



LVL Handbook

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Foreword

The use of wood is rapidly increasing in construction. Wood based systems are the solution for urban construction due to speed, light weight and environmental benefits. Wooden products enable increased prefabrication. This again means increased output, faster payback, high quality, and minimized disturbance. The use of laminated veneer lumber (LVL) has steadily been growing since the 1970's. The initial driver for the development of LVL has been the good efficiency of the used wood material. Low waste of material and uniform quality improves the overall material and time efficiency especially in industrial applications and off-site production of construction elements.

This European LVL Handbook serves as a guide for architects, structural engineers, technical wholesalers, off-site element manufacturers, contractors, carpenters and other construction professionals, and as an educational resource. The handbook has been prepared by the Federation of the Finnish Woodworking Industries during 2018-19.

The book has been divided into two main parts. Sections 1 to 3 provide general introduction to LVL. Section 1 introduces laminated veneer lumber (LVL) as a construction material, including its production, sustainability and basic properties; Section 2 describes the applications of LVL in various buildings and structures; and Section 3 provides instructions for purchasing, transport, storage, handling, protection and disposal of LVL. Sections 4 to 9 are intended primarily for designers and provide more detailed information about structural design, fire safety, durability and building physics, including structural calculation examples.

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Espoo, 4 September 2019

Authors

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1. LVL as a construction material

Stora Enso



Figure 1.1 Wooden multi-storey building, Wood City, Helsinki, Finland.

1.1 INTRODUCTION

Laminated veneer lumber (LVL) is an engineered wood product used in a diverse range of building and bridge construction applications. LVL beams, columns and panels have become established as essential components in modern timber construction due to their numerous advantages, versatility and proven structural performance. This handbook describes the state of the art of LVL from properties and applications to production and design methods.

LVL is made of 3 mm thick veneers bonded together with weather-resistant phenolic adhesive. This means that the dimensions of the final LVL product are not limited by the dimensions of the raw material, and even small-diameter logs can be used to produce large LVL beams and panels. When veneer logs are peeled to produce veneer, any natural defects in the wood, such as knots, are dispersed as small fragments across the veneer. This, together with the lamination effect, eliminates the impact of defects and results in exceptionally homogeneous material properties. Although the production costs of LVL, like all engineered wood products, are higher compared to sawn timber, with LVL the same constructions can be designed with smaller dimensions and LVL can also be used for applications where suitable sawn timber sizes are not available.

The low deviation of LVL's high strength and stiffness means that these properties can be fully utilized as characteristic values in structural design. In addition, due to the lack of sizeable defects, the strength to weight ratio of LVL is extremely high – LVL is twice as strong as steel in proportion to weight. Due to its laminated structure, LVL is also dimensionally stable and free of warps, splinters and splits. LVL also comes dry from the factory, eliminating the risk of shrinkage on site or in the ready building, as long as the LVL members are protected against weather exposure.

LVL is manufactured to exact dimensions, minimizing cross cutting and sawing waste. The resulting low material waste and uniform quality of LVL improve overall material and time efficiency, especially in industrial applications and off-site production of construction elements. LVL is easy to drill, cut, fasten and fit – only standard wood working tools are needed. LVL components are also highly portable due to their light weight. LVL can also be easily combined with other wood products and construction materials.

LVL is produced entirely from traceable, renewable, recyclable wood. It is a natural material from certified sources. LVL also serves as a carbon store in buildings: 1 m³ of LVL contains stored carbon equivalent to 789 kg of CO₂, making it an environmentally friendly choice ^{1, 2, 3}.

Features and Benefits

- Twice as strong as steel in proportion to weight.
- Dimensionally stable, no twists, splinters or splits due to laminated structure.
- Homogeneous material properties.
- Uniform quality and dimensions of the end product; key advantage in industrial applications.
- Easy to drill, cut, fasten and fit, only standard wood working tools needed.
- Precision-engineered and easily tailored.
- Can be produced to exact dimensions, minimizing cross cutting and sawing waste.
- Wide range of sizes: product dimensions not limited by raw material size.
- Light and highly portable.
- Dry from factory, moisture content 8-10% ensured minimal shrinkage in situ.
- Easily combined with other construction products.
- Off-site production reduces construction time.
- Entirely traceable, renewable, recyclable wood from certified sources.
- Environmentally friendly carbon store: 1 m³ LVL contains stored carbon equivalent to 789 kg CO₂.
- Production cost is higher than sawn timber, but less material is needed to meet design specifications when building with LVL ^{1, 2}.

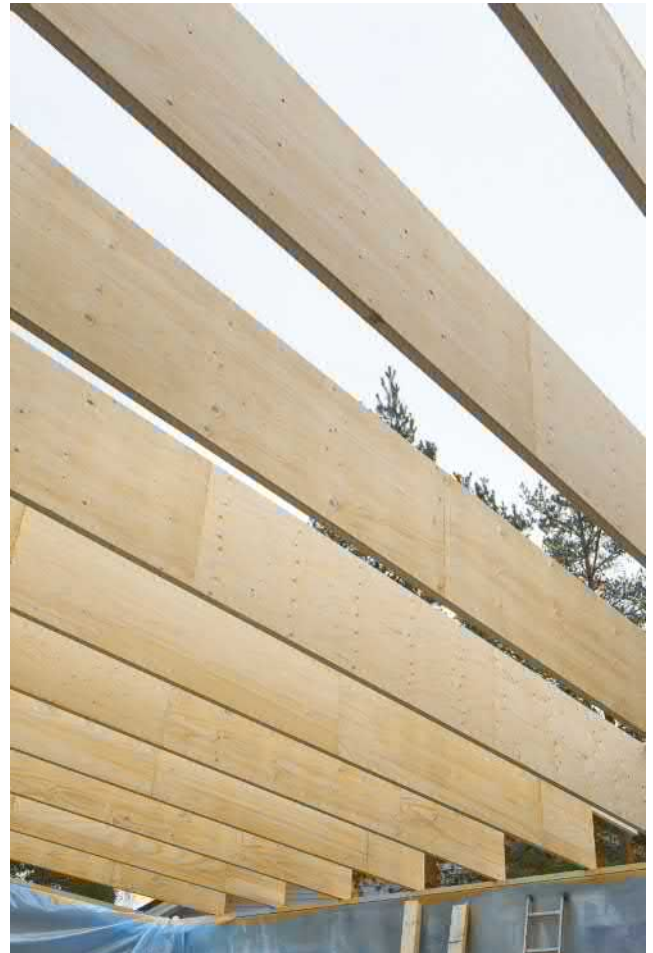


Figure 1.2 LVL roof rafters.

1. LVL AS A CONSTRUCTION MATERIAL

Stora Enso



Figure 1.3 LVL-P beams.

1.1.1 LVL-P beams and columns – all veneers in the length direction

LVL-P is made of 3 mm veneers laid in the same direction and bonded with weather- and boil-resistant phenolic adhesive. This structure enhances the material's strength properties that have significantly small deviation. LVL-P members can be used both as horizontal beams and vertical posts in various applications in construction systems. Erection and installation can be carried out without heavy machinery, even in confined spaces.

LVL-P beams are produced from the highest strength grade veneers to optimized beam dimensions and a height-to-thickness ratio that provides good material efficiency. LVL-P beams have an excellent strength-to-weight ratio allowing long spans with minimal deflection.

LVL-P studs are perfect for load-bearing or non-load-bearing structures in external and internal walls. LVL-P studs are easy to install, screw, drill, nail and cut. LVL studs are produced from lighter grade veneers, but their dimensional accuracy, structural strength and rigidity, straightness, and lack of twisting make them ideal for wall structures ^{1,2}.

Table 1.1. Applications of LVL-P.

Beams	Features and benefits
<ul style="list-style-type: none">• Headers, main beams, ridge beams• Lintels• Floor joists• Roof rafters• Purlins• Trusses• Frames• Components for roof and floor elements• Sole and top plate• Beam reinforcements	<ul style="list-style-type: none">• Strong and rigid: long spans with minimal deflection• Straight and dimensionally stable, does not warp or twist: improves construction quality; ideal for walls, also high constructions• Dry from factory: minimal shrinkage in situ• Customized product dimensions, with minimum waste: Applications in a wide variety of building types; savings in material costs and time• Great workability: easy to install, staple, nail, drill and cut without special tools• Excellent strength-to-weight ratio: Light structures• Light: easy to handle and lift manually or with light cranes• Can be used with any panel material: suitable for a wide variety of structures• Easy to combine with other structures and materials in a wide range of building types
Studs	
<ul style="list-style-type: none">• Wall studs for internal walls• Wall studs for external walls• Load-bearing and non-load-bearing-applications	
Industrial applications	
<ul style="list-style-type: none">• Support structures and moulds for concrete formwork• Scaffolding• Door- and window frames• Furniture components• Packaging industry	



Figure 1.4 LVL-C roof panels.

1.1.2 LVL-C with cross-bonded veneers for robust structural panels

LVL-C panel is a cross-bonded panel product with approximately 20% of the veneers glued in a crosswise direction. This enhances the transverse strength and stiffness and the connection ductility of the panel. It is dimensionally stable, as the cross veneers prevent shrinkage and swelling in the event of moisture changes in the panel width direction. LVL-C panels or beams can be used as both horizontal and vertical bearers in numerous construction applications. It is designed for use as a large-dimension load-bearing panel, as well as for structural bracing and stabilizing. LVL-C panel is an ideal component for floor, wall and roof elements, roof and wall panels, and pre-fabricated houses. The panels can be cut to tailored sizes and special shapes for modern timber structures ^{1,2}.

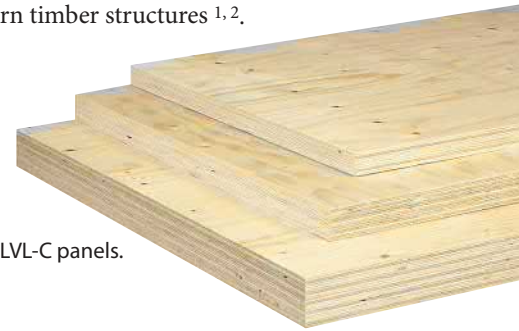


Figure 1.5 LVL-C panels.

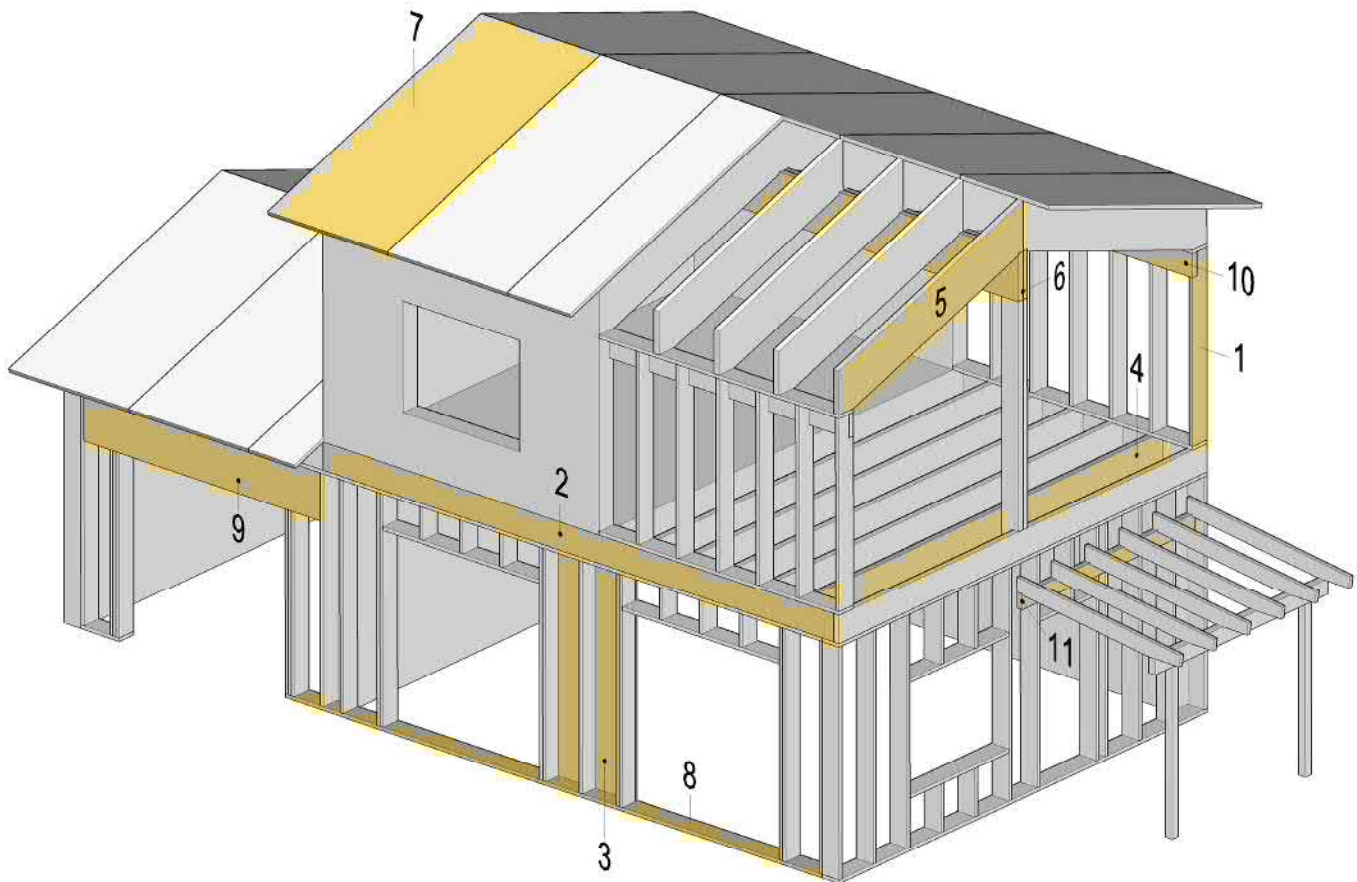
Table 1.2. Applications for LVL-C.

Panel applications	Features and benefits
<ul style="list-style-type: none"> • Large panel product for roof, floor and wall constructions • Roof overhangs • Pre-fabricated roof, floor and wall elements and modules • Stresses-skin panel elements • Door panels 	<ul style="list-style-type: none"> • Strong and rigid: long spans with minimal deflection • Large dimensions: applications in a wide variety of building types • Installation efficiency: large panel for floors, roofs and walls covers vast areas much faster than normal size wood-based panels reducing the amount of lifting required on the construction site. • High and thin beams: energy efficient constructions • Straight and dimensionally stable; does not warp or twist: improves construction quality • Dry from factory: minimal shrinkage in constructions • Customized product dimensions and shapes with minimum waste: applications in a wide variety of building types; savings in material costs and time on construction site • Excellent strength-to-weight ratio: light structures • Great workability: easy to install, staple, nail, drill and cut without special tools • Ductility of connections: improved safety • Strong and rigid in edgewise compression perpendicular to grain direction: better building quality with minimal deformation, suitable for narrow supports • Not sensitive to cracking under tension perpendicular to grain stressed structures: safe connections • Bracing of different size buildings, regardless of frame material: applications in a wide variety of building types; large window openings possible.
Other applications	
<ul style="list-style-type: none"> • Rim boards • High rafter beams • High lintels and header beams • Frames and truss members • Reinforcement of old structures • Curved components, free-form beams and panels (CNC machining) • Concrete formwork 	

1.2 WHERE CAN YOU USE LVL

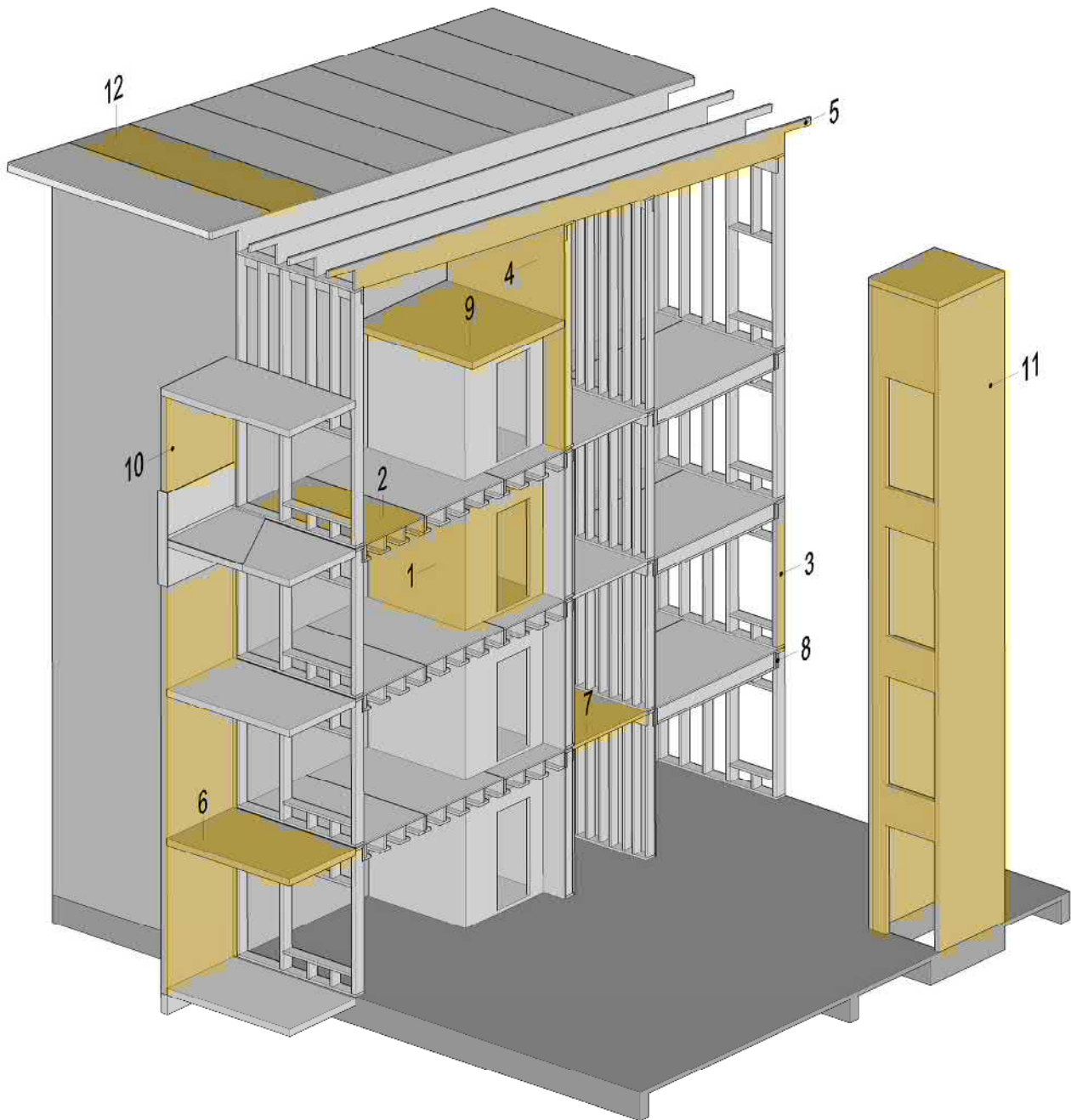
Figures 1.6-1.10 present a number of applications for LVL in low-rise buildings, multi-storey buildings, hall constructions, and some special applications.

Other common applications of LVL include scaffold planks for construction sites, concrete form work support beams, door panels, door and window frame components, and wooden I-joist flanges.



- | | | |
|---|--|--|
| 1. LVL-P load-bearing wall stud: straight and precise dimensions. | 4. LVL-P floor joist: strong and rigid. | 8. LVL-P/C sole plate: fits stud dimensions, thin members reduce settling. |
| 2. LVL-C rim board: dimensionally stable, minimal settling. | 5. LVL-P roof rafter (or LVL-C when height-thickness ratio is high): space for thermal insulation of low-energy buildings. | 9. LVL-P lintel over garage door opening: large openings possible. |
| 3. LVL-C bracing panel: narrow, but robust panel next to large openings in walls when there is no space for large panel fields. | 6. Double LVL-P ridge beam: strong and rigid. | 10. LVL-P lintel in wall to carry roof loads: straight and rigid. |
| | 7. LVL-C roof panel: fast installation, fewer joints, roof overhangs without additional supports. | 11. LVL-P ledger beam for canopy: straight and rigid, easy details. |

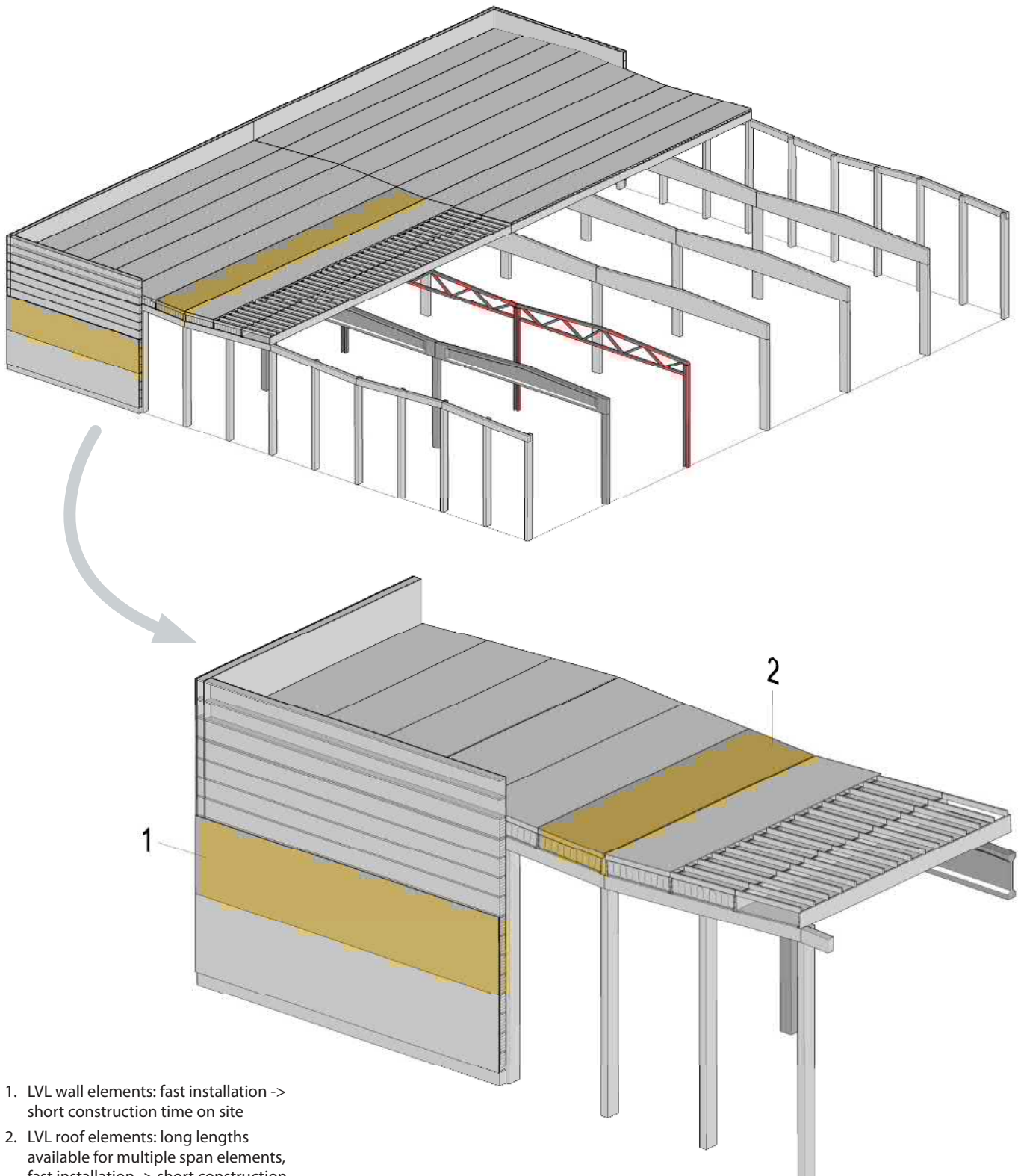
Figure 1.6 Applications of LVL in a timber frame house.



1. LVL-C panel structure of volumetric bathroom module: light structure, short construction time on site.
2. LVL ribbed slab intermediate floor elements: long spans with suitable floor thickness.
3. LVL-P wall studs (small dimensions for non-load-bearing walls, larger dimensions for load-bearing walls): straight and precise dimensions.
4. LVL-C bracing panel: stable and rigid building.
5. LVL-P roof beams/elements: space for thermal insulation of low energy buildings, long lengths available.
6. LVL-C balcony floor slab: simple structure.
7. LVL-C corridor floor slab for limited spans: simple structure.
8. LVL-C rim beams / Lintels: straight and rigid structure, simple geometry of element joints.
9. LVL-C mezzanine floor slab for loft spaces: minimal structural depth, better use of room height.
10. LVL-C panel structures for balcony walls (separate cladding needed): simple and rigid structure.
11. LVL-C panel structures for lift shafts: panel sizes available for full building height or storey-high elements. Acts as a part of the bracing system of the building.
12. LVL-C roofing panels: large sizes, fast installation, fewer joints

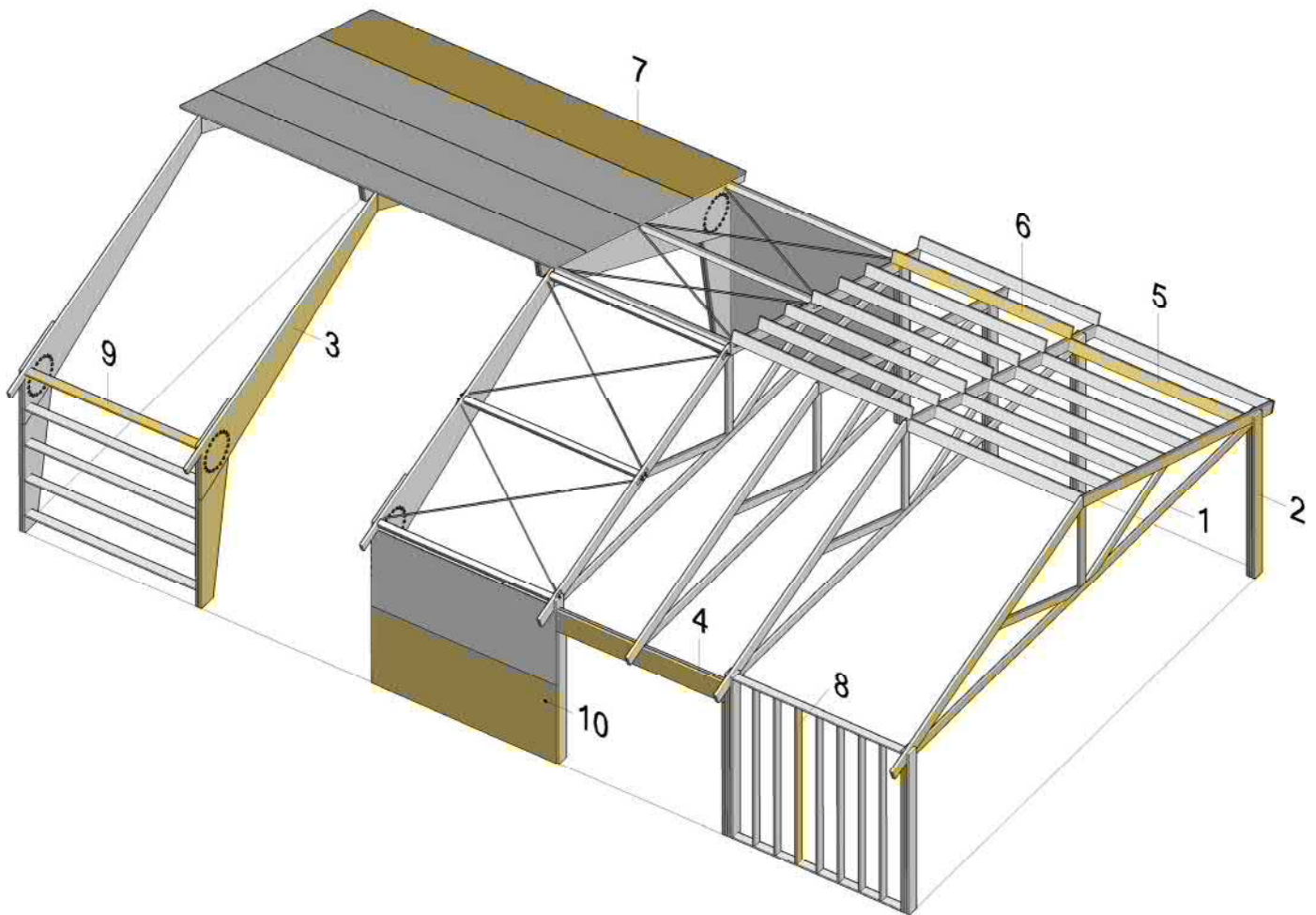
Figure 1.7 Applications of LVL in a wooden multi-storey building.

1. LVL AS A CONSTRUCTION MATERIAL



1. LVL wall elements: fast installation -> short construction time on site
2. LVL roof elements: long lengths available for multiple span elements, fast installation -> short construction time on site. Glued box slab elements for long spans.

Figure 1.8 LVL applications in hall constructions. LVL elements can be used together with any type of main frame, e.g. precast concrete beams, steel trusses or glulam beams.



- | | |
|---|---|
| 1. LVL king post or queen post roof trusses: impressive appearance. | 6. LVL-P purlins, multiple span: long lengths available. |
| 2. LVL columns: fit together with LVL roof trusses. | 7. LVL-C bracing panels for roofs: simple and robust. |
| 3. LVL-P and LVL-C portal frames: large clear height. | 8. LVL-P studs for high walls: straight and precise. |
| 4. LVL lintels for door and window openings: strong and rigid. | 9. LVL-P horizontal beams for walls: large spacing between main frames. |
| 5. LVL-P purlins, single-span: strong and rigid. | 10. LVL-C bracing panels for walls: simple and robust structure. |

Figure 1.9 LVL applications in hall constructions.

1. LVL AS A CONSTRUCTION MATERIAL

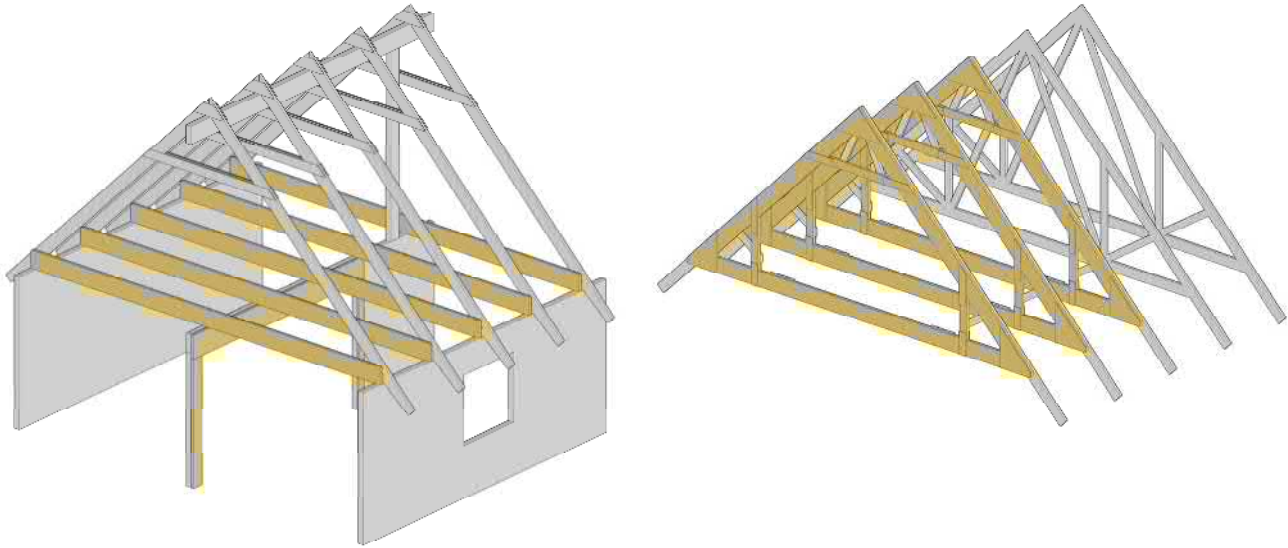


Figure 1.10 Renovation applications, left: LVL floor beam reinforcement, right: attic frames for room in roof space with LVL reinforced roof trusses.

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Figure 1.11 Off-site produced wooden volumetric elements, BoKloK, Vantaa, Finland.

1.3 ARCHITECT AND STRUCTURAL DESIGNER APPRAISALS OF LVL

*Architect's view of LVL,
Mika Ukkonen, Finland*

From the designer's point of view, wooden constructions give a different opportunity to create recognizable and distinctive architecture that is produced naturally, drawing on the natural structural properties of wood. I have worked as the principal designer and architectural designer for several residential multi-storey wooden building projects delivering 242 apartments to date. I have also designed around 300 apartments for architectural competitions and our office currently has approximately 200 apartments under design for new wooden multi-storey buildings. As a construction material wood is ideal for versatile urban design and provides possibilities for a modern life cycle approach to living.

Wood is the only 100% renewable construction material and the use of wood slows climate change. Wooden load-bearing structures create pleasant environments, healthy indoor air and comfortable acoustics. The high prefabrication rate of off-site production enables fast construction times on the building site. As a construction material, wood provides excellent opportunities for flexible modifications of the layout of a building during its whole life cycle.

The structural system in our projects is based on large timber frame panel elements. Each project involves product development of various building components, which gives us the ability to optimize the best solutions and architecture

for each case. For example, we have replaced concrete elevator shafts with LVL-C panel structures and built intermediate floors from offsite-produced LVL elements with 7-metre spans. Floor elements have been developed so that the required mass for acoustics is realized with dry screed panels pre-installed on the elements already in the factory. This has further increased the prefabrication rate and minimized the amount of site work needed.

*Structural engineer's view of LVL,
Wenzel von Fragstein, Germany*

As structural designers focusing on wooden buildings, we design entire wood and wood-hybrid constructions. From single-family houses to wooden multi-storey buildings and industrial hall constructions, our activities cover the whole range of wooden buildings. Renovation of churches and historical building as well as church towers of new buildings have also been part of our project portfolio. We typically prepare the work specifications and production drawings for manufacture all the way down to the steering files for CNC machining. Being involved in the whole construction process in this way, we

get a lot of feedback on the technical feasibility, assembly and cost efficiency of the structures in practice.

We use LVL in all kinds of applications, such as the renovation and reinforcement of existing wooden floors, where slim cross-section LVL reinforcement brings structural enhancement to existing joist structures. We also often use ribbed panel and box panel structures to achieve light and long-spanning floors. In cross-laminated timber buildings, we use LVL lintels where small cross-sections with high loadbearing capacity are required. Occasionally, we use LVL-C panels for walls or floors without separate stud or joist structures. Long overhangs from thin structural panels are commonly desired for front roofs, and LVL-C panels can be used to achieve them as they have better strength properties than most other structural panels. Their high strength is a result of their glued veneer structure, which eliminates the influence of individual defects, such as knots. LVL-C with cross veneers has significantly higher resistance to splitting than solid wood, so by choosing LVL-C for members that have high tension perpendicular to grain stresses, separate reinforcements can be avoided.

LVL enriches the product range of wood-based materials significantly.



Figure 1.12 Honkasuo multi-storey houses, Helsinki, Finland.

1.4 HISTORY, VOLUMES AND RAW MATERIALS OF LVL

1.4.1 History of the globally used engineered wood product LVL

Parallel oriented plywood products have been used in the furniture industry since the beginning of the 20th century. The history of LVL in its present form dates back to 1970s North America and the research and development of veneer-based wooden beams by Peter Koch and the USDA Forest Products Laboratory. The first commercial solution for manufacturing LVL was created by Al Troutner of the company Trus-Joist (Weyerhaeuser). In 1975, Finnish company Metsäliiton Teollisuus Oy (Metsä Wood) developed the first commercial LVL

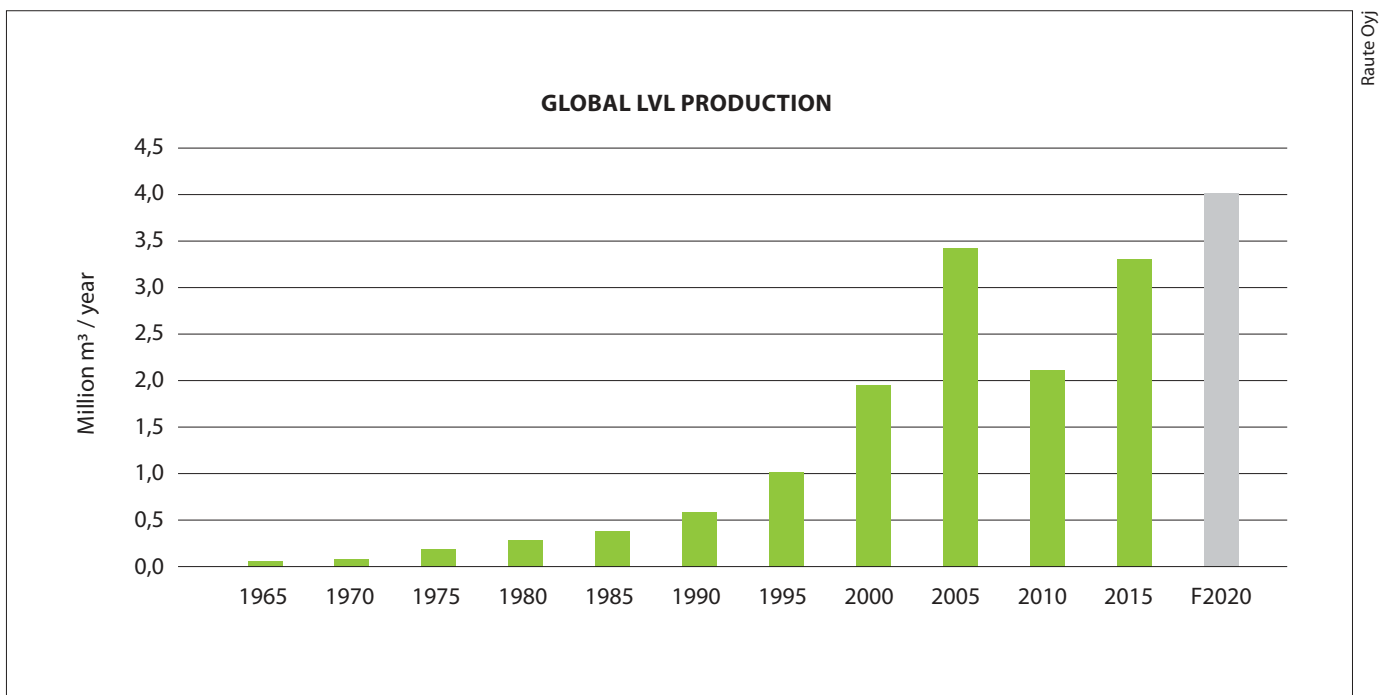
production line in Europe. Metsäliitto went on to develop its own manufacturing concept, which was further developed by another Finnish company, Raute Oyj, which is today the leading LVL machinery supplier worldwide. LVL production has since grown continuously and today LVL is manufactured in ten countries on four continents with a total production capacity of around 4 million cubic metres per year across 30 locations. In addition, a number of small plants produce LVL that does not meet the requirements of structural LVL ⁴.



Figure 1.13 LVL-P beam and LVL-C panels.

Table 1.3. Global LVL production. Active manufacturers of structural LVL produce about 3.9 million cubic metres per year ^{4,5}.

Europe			North America			Asia & Oceania		
Manufacturer	Mills	Capacity 1000 m ³ /year	Manufacturer	Mills	Capacity 1000 m ³ /year	Manufacturer	Mills	Capacity 1000 m ³ /year
Metsä Wood	2	300	Boise Cascade	3	890	JNL	2	140
Steico	1	160	Weyerhaeuser	4	530	Carter Holt Harvey	1	100
Stora Enso	1	100	Lousiana Pacific	2	260	Nelson Pine	1	100
MLT	1	100	Pacific Woodtech	1	220	First plywood	1	100
Pollmeier	1	80	Roseburg	1	200	Wesbeam	1	60
LVL Ugra	1	40	Forex Amos Inc.	1	140	Keyteck	1	60
			Murphy	1	120	Shin Yang	1	20
			West Fraser	1	90			
			RedBuilt	1	70			
			Global LVL	1	20			
Total	7	780		16	2540		8	580

**Figure 1.14** Development of structural LVL production volume globally 1965 – 2020 ⁵.

1. LVL AS A CONSTRUCTION MATERIAL

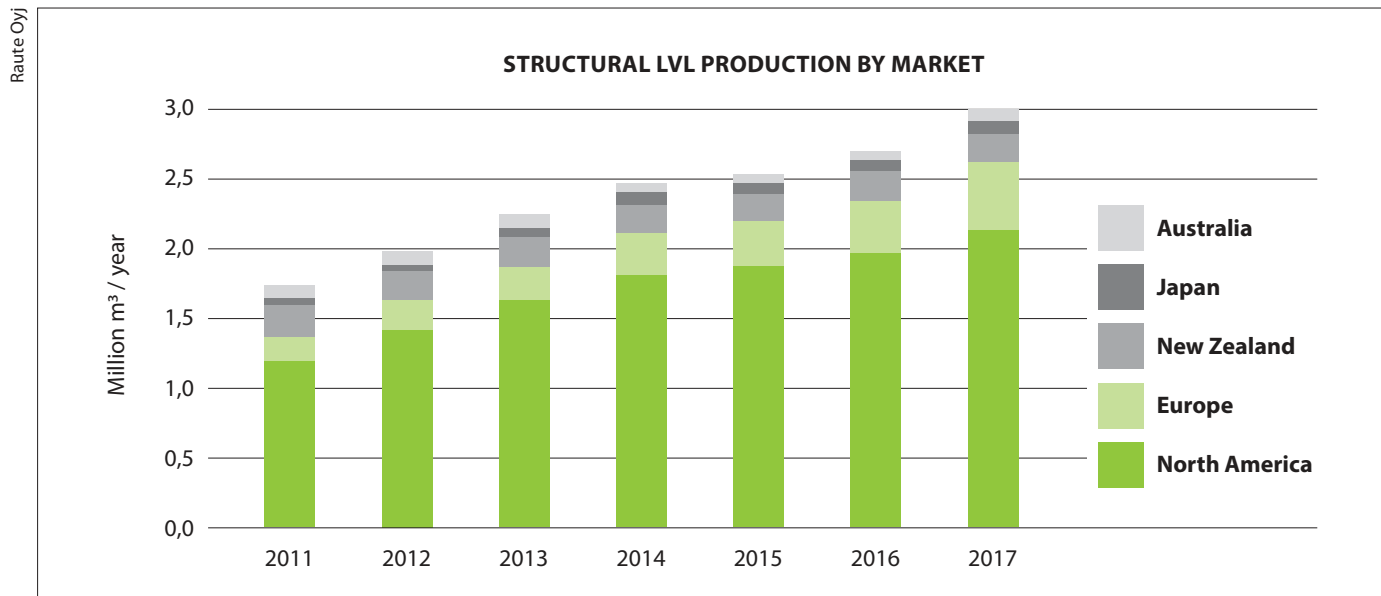


Figure 1.15 Structural LVL production volume by market 2011 – 2017⁵.

1.4.2 Material efficiency

The initial driver for the development of LVL has been the high efficiency of wood raw material use. Firstly, the dimensions of the final product are not limited by the dimensions of the raw material – even small-diameter logs can be used to produce long LVL beams. Secondly, the veneer structure of LVL products eliminates the impact of natural defects, such as knots, resulting in a homogeneous material with excellent structural properties. This means that logs that are not good enough for sawing can be suitable raw material for LVL. Thirdly, products can be manufactured to exact dimensions minimizing material waste in cross cutting and sawing, and product features

can be varied by adjusting production process parameters. In a modern production process about 2,5 m³ of logs (over bark) are needed to yield 1m³ of ready LVL product, which is efficient compared to other wood products intended for load-bearing structures. In addition, the end product yield of LVL consists almost exclusively of the desired product grades and sizes, unlike sawn timber production, which produces a mixture of heartwood and sideboards. 1 m³ of dried sawn timber needs about 2,0 m³ logs (over bark); the corresponding value for glulam and cross-laminated timber (CLT) is 2,6-2,8m³.

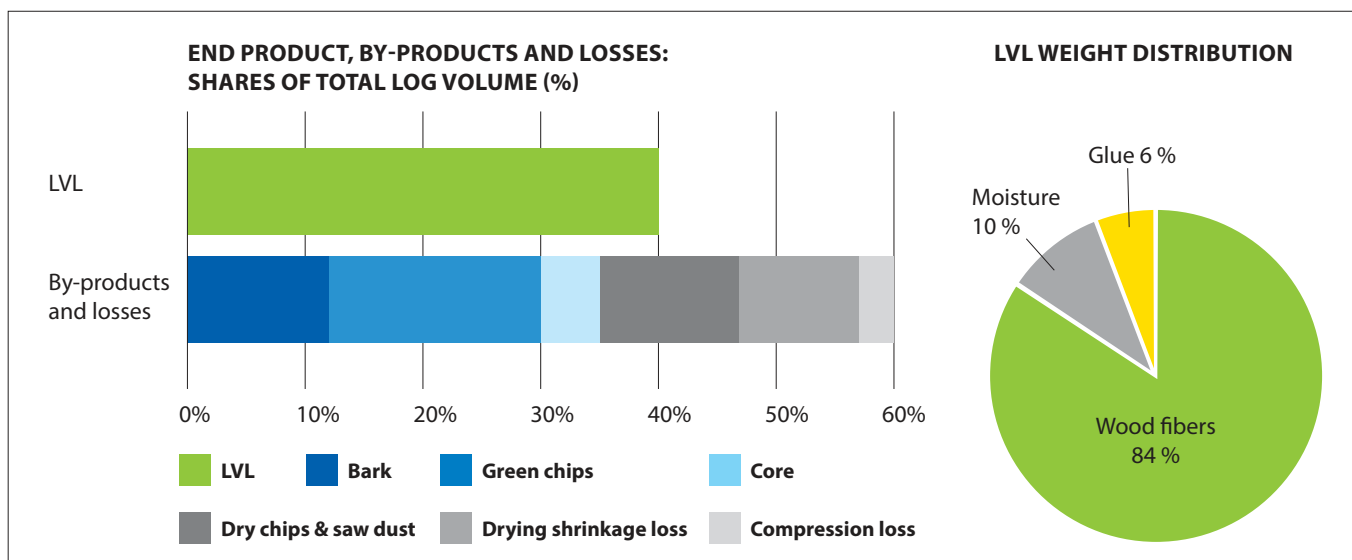


Figure 1.16 Left: Distribution of wood material in LVL production. 2,5 m³ of logs (over bark) is needed to produce 1m³ of LVL. Right: Weight distribution of wood fibres, glue and moisture in ready LVL product⁵.

1.4.3 Wood species used in LVL

LVL is commonly produced from softwood and, in Europe, spruce and pine are typically used. Spruce has the best strength to weight ratio, and its low resin content is also advantageous with respect to the production process. Pine veneer, on the other hand, has higher density, which gives the product slightly higher mechanical properties. Some LVL manufacturers in Europe also use the hardwood species beech and birch. The mechanical properties of hardwood LVL are higher due to the higher density. However, higher density introduces additional machining requirements, such as possible predrilling for screw connections. The surface of hardwood LVL is also more sensitive to mould growth in humid conditions.

North American LVL manufacturers use different pine species, Douglas fir, western hemlock, yellow poplar and red maple. In Australia LVL is produced from different pine species and karri (eucalyptus). In Japan, LVL is produced from larch and Japanese cedar (sugi). Other species that have the required mechanical and gluing properties may also be used.

1.4.4 Durable and inert gluing of LVL

In structural LVL the veneers are bonded together with weather- and boil-resistant phenol formaldehyde (PF) adhesive, which is cured in the hot pressing process. Curing converts the adhesive to a high temperature resistant inert polymer that does not dissolve or react with other materials in the surrounding environment. LVL also fulfils the most stringent formaldehyde emission requirements, with emissions 3 times lower than the limit value of the E1 classification tested according to standard EN 717-1. The dry solids content of adhesive in LVL is approximately 30 kg/m³, i.e. about 6% by weight.

1.5 SUSTAINABLE BUILDING WITH LVL

1.5.1 Traceable raw material and sustainable sources

Sustainable forest management and use of forest products play a key role in the mitigation of global warming and contribute to achieving climate policy objectives. Forest climate mitigation options include reducing greenhouse gas emissions from deforestation and forest degradation, enhancing the carbon sequestration rate in existing and new forests, providing wood residues as a substitute for fossil fuels, and replacing non-renewable and energy-intensive materials with wood products, particularly in the building sector.

Due diligence on the origin of wood

Due diligence systems differentiate wood and wood products from many other building materials by verifying the origin of the wood raw material. The European Union Timber Regulation (No 995/2010) was enacted to ensure that all wood placed on the European market from internal or external sources is covered by a due diligence system for verifying the legal origin of wood 6. The objective is to prohibit any product that contains wood raw material harvested in violation of national or international laws from entering the EU market. The regulation covers all wood and wood products, such as round wood, solid wood, engineered wood, pulp, paper and board.

The EU Timber Regulation covers the entire wood value chain. An 'operator' who first places a wood product on the European market is obliged to prove the legality of the wood to a national EU Timber Regulation authority. A Due Diligence system is applied for this purpose, including access to information on wood sources, risk assessment, and mitigation of risks. Chain of Custody systems (e.g. PEFC™ and FSC®) can be third-party certified according to Due Diligence requirements.

Promoting sustainable, certified forest management

European LVL producers are well positioned in forest certification as European forests owners have been active in applying forest certification schemes (most commonly PEFC™ and FSC®). Forest certification schemes provide third-party verification of sustainable forest management practices and the chain of custody from forest to product. They include requirements for sustained harvesting and forest regeneration practices, biodiversity protection, multiple and recreational uses of forests, social sustainability, training of employees, and occupational safety.

In order for a wood product to PEFC™ or FSC® labelled, it must contain at least 70% wood raw material from certified forests. When non-certified wood is used in the production of certified goods, the wood must originate from forests covered by an appropriate due diligence system.

1.5.2 Sustainable over the life cycle

Wood products offer renewable and sustainable solutions for construction. Wood as a renewable material has a lower global warming impact compared to alternative, non-renewable building materials. The infinite carbon cycle between the atmosphere, growing trees and wood products distinguishes renewable wood from non-renewable materials.

Life cycle assessment (LCA) is a holistic approach for assessing environmental impacts throughout a product's or system's life cycle, from extraction of raw materials to disposal of the product. The principles of LCA have been internationally agreed and standardized with the ISO 14040 and ISO 14044 standards, which enables third-party verification of life cycle calculations. LCA compiles and evaluates the inputs, outputs and potential environmental impacts of a product or system throughout its life cycle. LCA helps manufacturers to identify opportunities to improve the environmental and climate performance of a product and to inform customers and stakeholders. LCA includes four steps; defining of the goal and scope, inventory of material and energy flows, assessment of impacts, and interpretation of results.

Environmental performance of buildings and building products

The European standard series 'Sustainability of Construction Works' (CEN/TC 350) guides the assessment of the sustainability of buildings and building products. The standard series aims to enhance the supply and demand of products and buildings that have as low environmental impact as possible. Environmental assessment of a building is based on the life cycle approach, in which each of the different stages of the building's life cycle are included and assessed (Figure 1.17).

At the product and service level, the environmental product declaration (EPD) applies the life cycle assessment approach and presents quantified environmental information over a product's life cycle. EPDs enable comparison between different products with the same functional purpose at the building level. In the case of LVL, the comparison is most appropriately done at the structure type level, e.g. structures with the same load carrying capacity and stiffness. The EN 15804 standard provides product category rules (PCR) for an environmental product declaration for any construction product or construction service (Table 1.4). Biogenic carbon content

Table 1.4. Life cycle stages of building environmental assessment based on EN 15978.

Life cycle stages		Module	
Building life cycle information	Product stage	A1	Raw material supply
		A2	Transport
		A3	Manufacturing
	Construction process stage	A4	Transport
		A5	Construction installation process
	Use stage	B1	Use
		B2	Maintenance
		B3	Repair
		B4	Replacement
		B5	Refurbishment
		B6	Operational energy use
		B7	Operational water use
	End of life stage	C1	De-construction, demolition
		C2	Transport
		C3	Waste processing
		C4	Disposal
Additional information outside the system boundary	Potential benefits and loads	D	Reuse, recovery, recycling potential

calculation rules are provided in the EN 16485 standard.

The most used environmental indicator of EPDs is global warming potential (GWP), also known as carbon footprint. The GWP reflects the amount of greenhouse gas emissions in each stage of a product's life cycle and is mainly the outcome of fossil fuel use in the raw material supply stage and energy use in the production stage (A1–3).

At the building level, the EN 15978 standard provides systematic calculation rules for the assessment of the environmen-

tal performance of new and existing buildings. The environmental performance of wooden buildings derives from their light weight (compared to other building materials), energy efficiency of materials and buildings, life-time carbon storage in wood, and their renewable and sustainable origin. Wooden buildings typically achieve the same service time as other buildings, typically 50–100 years. Up to 100 years or longer service time is achievable with proper design and optimized maintenance.

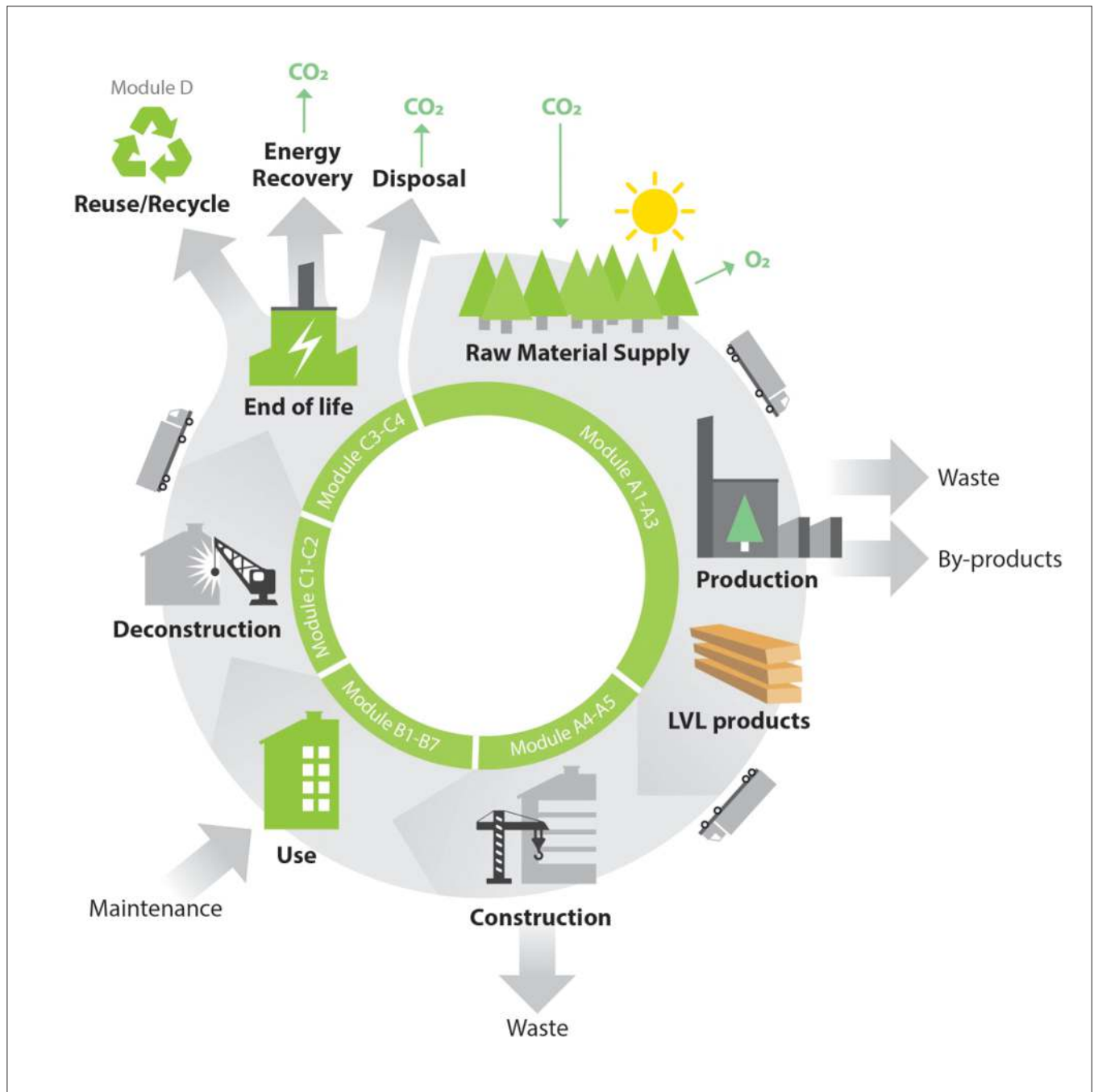


Figure 1.17 Life cycle approach of building product assessment.

1.5.3 Global Warming impact of LVL

In the manufacture of LVL, a large share of renewable energy is used (see Figure 1.18), resulting in lower fossil greenhouse gas emissions and global warming potential compared to other building materials. In addition, LVL stores biogenic carbon, which constitutes about one half of the dry weight of the wood. The biogenic carbon remains in the LVL throughout its lifetime until it is released back to the atmosphere to be absorbed by the next generation of growing trees.

The global warming potential of LVL when used in a load-bearing structure of a building has been determined by LVL producers as follows:

- Raw materials extraction and energy use are similar in different production units, but different energy mixes are used depending on the unit and country. Energy mix affects the greenhouse gas emissions of the production stage (modules A1–3). Biogenic carbon content is given separately.
- Construction stage (A4–5) covers transportation to the Central European market and normal construction works.
- Use stage (B1–7) is considered negligible.
- End of life stage C1–4 scenarios consider utilization of LVL as a source of energy.
- Module D, other benefits, originate mainly from the bio-energy substitution effect, compared to typical local energy mix, when LVL is used for bioenergy at end of life.

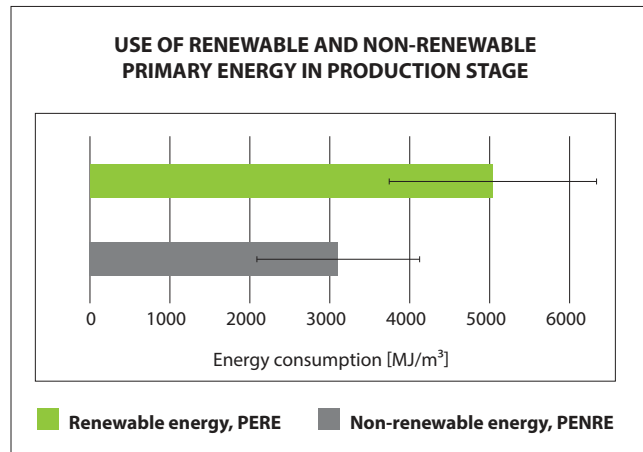


Figure 1.18 Primary energy (renewable and non-renewable) use in the manufacturing stage (A1–A3) of LVL ^{3, 7, 8}.

Biogenic Global Warming Potential is the sum of stored biogenic carbon in the production stage and release of biogenic carbon in the end of life stage. The sum is close to zero for the whole life cycle.

Consolidated results for fossil global warming potential show that the production stage (A1–3) constitutes 90% of the result (Figure 1.19). The construction stage (A4–5) constitutes 10%, and end of life stage (C1–4) constitutes 2.5% of the fossil GWP. In module D the GWP benefits are shown as the amount of fossil fuels substituted by biomass energy.

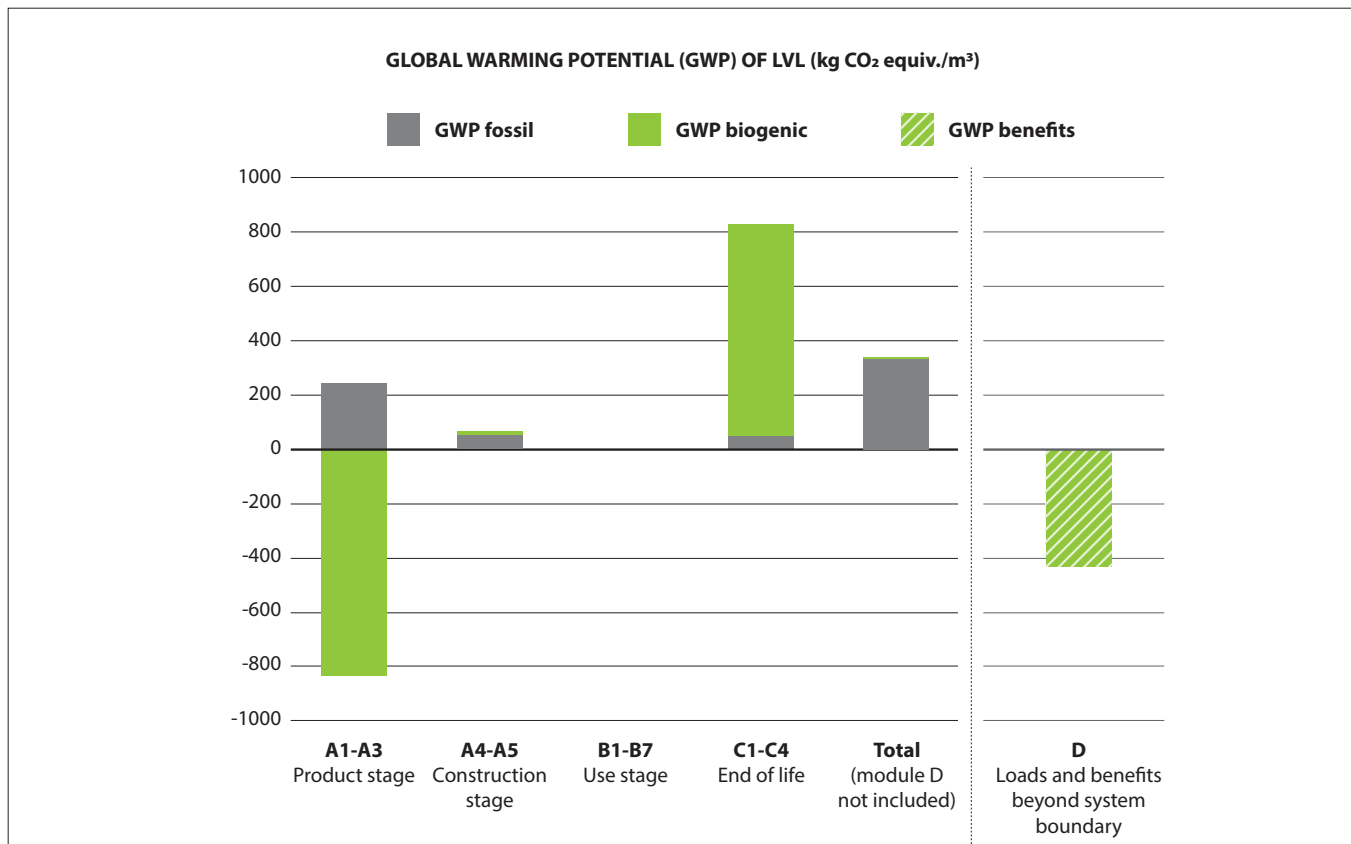


Figure 1.19. Example global warming potential of different life cycle stages ^{3, 7, 8}.

1.5.4 Global warming impact of buildings

The life cycle of a building covers all life cycle stages from 'cradle to grave' as well as, optionally, also the benefits and loads beyond the building's life cycle. These life cycle stages are presented as modules A1–C4 + D (Figure 1.17). The environmental performance of a wood-frame building, and a concrete-frame building were assessed based on a generic Finnish

building type (a 4-storey residential building) ^{9, 10}. The assessment was done according to the EN 15978 standard, with the specific assumptions presented in Table 1.5.

The global warming potential of the whole life cycle and beyond is 798 kg CO₂e/m² for a wood-frame building and 1022 kg CO₂e/m² for a concrete-frame building (Figure 1.20).

Table 1.5. Scenarios of a wooden frame building and a concrete frame building ^{9, 10}.

	Concrete-frame	Wood-frame
System boundary	4-storey residential building, gross floor area 1922 bm ² , concrete structure, wooden roof structure, all technical installation included (HVAC, piping, electricity), energy supply based on average Finnish heat and electricity	4-storey residential building, gross floor area 1922 bm ² , first floor concrete structure, other floors wood structures, all technical installation included (HVAC, piping, electricity), energy supply based on average Finnish heat and electricity
Reference period	50 years	50 years
Service life	Windows replaced once	Windows replaced once, external cladding (wood) painting every 10th year
End of life	Concrete: crushing and recycling for ground construction Wood: chipping and energy recovery	Concrete: crushing and recycling for ground construction Wood: chipping and energy recovery
Benefits and loads beyond system boundary (module D)	Concrete: carbonization Wood: bioenergy carbon emissions compared to natural gas emissions	Concrete: carbonization Wood: bioenergy carbon emissions compared to natural gas emissions

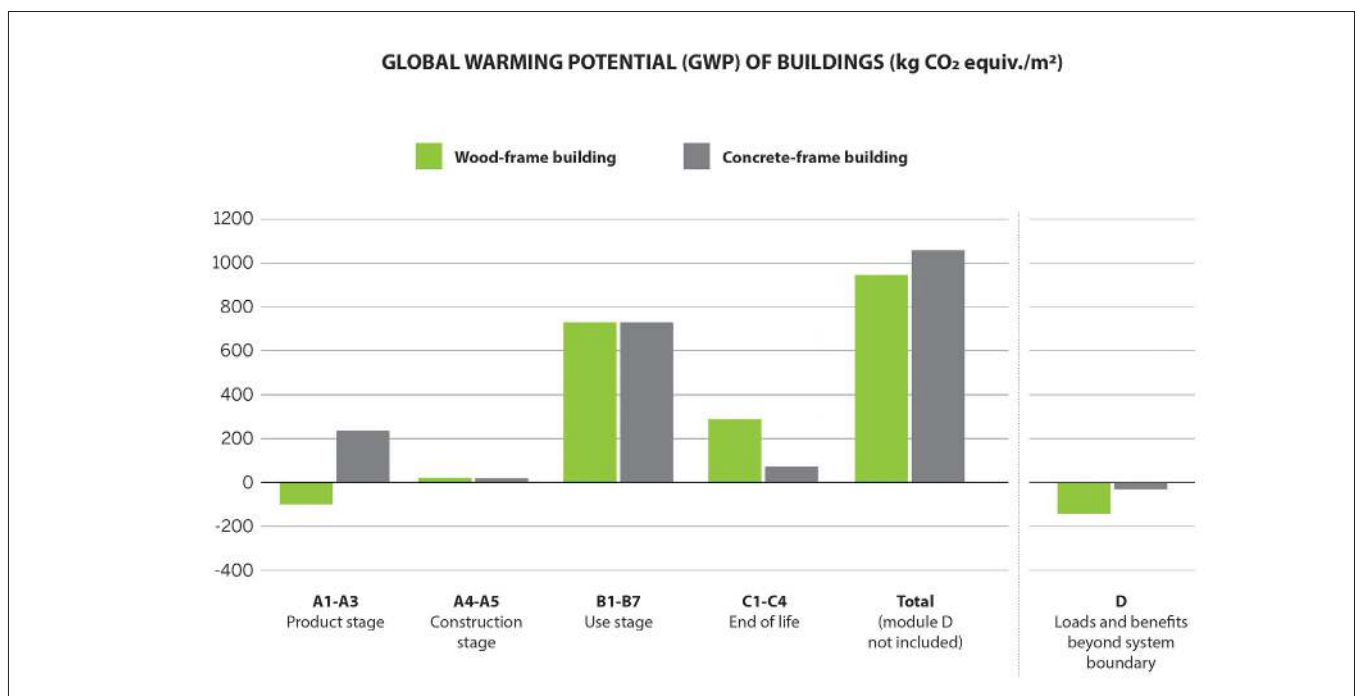


Figure 1.20. Global warming potential (GWP) of a wood-frame building vs. a concrete-frame building ¹⁰.

1. LVL AS A CONSTRUCTION MATERIAL

1.6 PRODUCTION OF LVL

The LVL production process is subject to strict quality standards and audited by inspectors to ensure the final product is safe to use and meets the specified end use requirements. LVL is made by gluing and layering wood veneers to produce a homogenous wood panel. The process distributes any natural defects in the wood raw material evenly throughout the product, eliminating any individual points of weakness. The following simplified production diagram illustrates the LVL production process inside the mill.

Veneer logs are delivered from the forest to the mill at a specified length. The logs are debarked, conditioned, and cut



Figure 1.21. Composition of LVL, from veneer mat to cut-to-size panel.

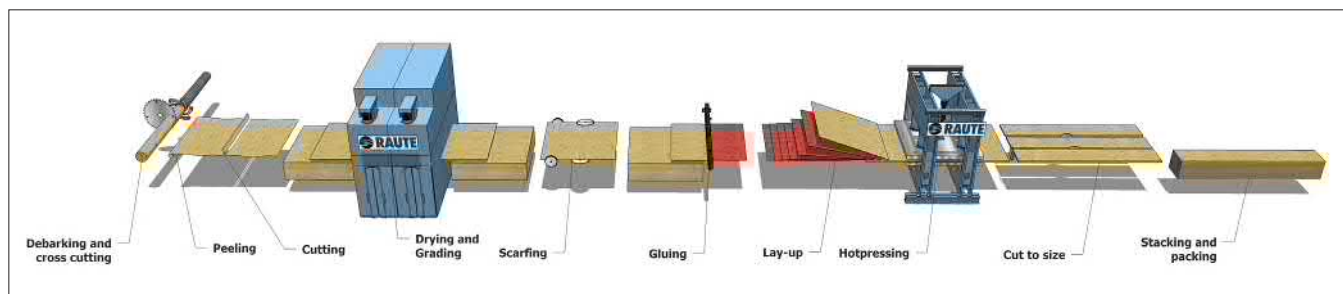


Figure 1.22. The LVL production process.

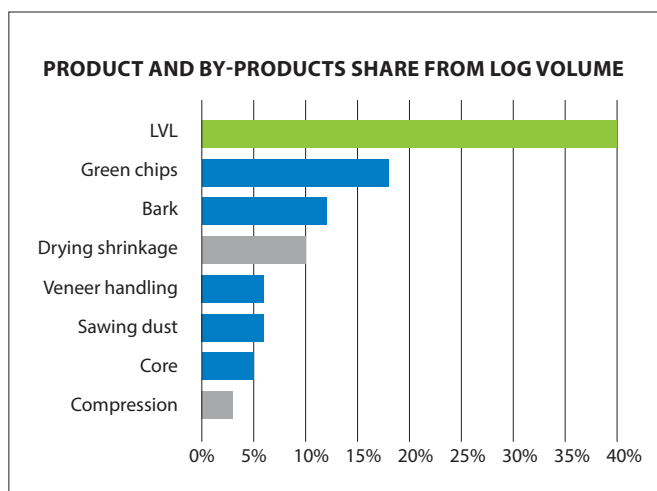


Figure 1.23. Log use distribution by volume: LVL and by-products (incl. shrinkage and compression losses).

to blocks. On the peeling line, the veneer block is turned to produce a veneer mat. The green veneer is clipped to size and then dried, graded and composed when necessary. Glue is then applied on the top side of each veneer and the veneers are staggered to form a LVL billet. In hot-pressing, the glue cures and binds the veneers together. Finally, the formed LVL panel is edge trimmed or cut to size (e.g. to a specific beam width) before stacking and packaging.

For every cubic metre of LVL about 2,5 m³ of logs (measured over bark) are processed, which is a similar raw material yield to plywood production. The LVL production process thus delivers more by-products than final product. None of these by-products go to waste. They are sold and used in pulp, fibreboard and other wood-based products, for energy production, and for decorative purposes. Glue residues are reused inside the mill.

1.6.1 Logs to blocks

To maintain high raw material quality, veneer logs are transported to the LVL mill immediately after harvesting. At the mill site, the log piles are sprayed with water to avoid drying and cracking and attack by pests.



Figure 1.24. Harvesting of logs.

The logs are cut to the required peeler block lengths on an automatic saw line and transferred to the debarker. The debarker removes the bark down to the cambium so that the blocks are free of bark. During debarking, damage to the peeler block surface is carefully avoided, as the most valuable veneer qualities are obtained from the outermost layers of the wood during peeling.

The conditioning increases the internal temperature of the blocks and makes the wood softer for peeling. In northern winter conditions the logs are thawed by soaking them in covered and heated conditioning chambers.

1.6.2 Blocks to veneer

A jack-ladder lifts the blocks one by one to the charger for XY centering. The block charger plays a key role in optimizing raw material utilization and the value of the veneer yield. To recover the maximum amount of veneer from the block, the block is optimally aligned between the lathe spindles using high precision laser measurement.

In the lathe, the peeler block is rotated between the spindles at a constant speed while the knife carriage moves toward the block core. For LVL production, the typical nominal thickness for softwood veneer is 3 mm. According to EN14374, the maximum thickness of veneers in structural LVL is 6 mm. The veneer is peeled through the gap between the peeler knife and the overhead nose bar. The knife gap is smaller than the veneer thickness to ensure sufficient compression and high-quality veneer.

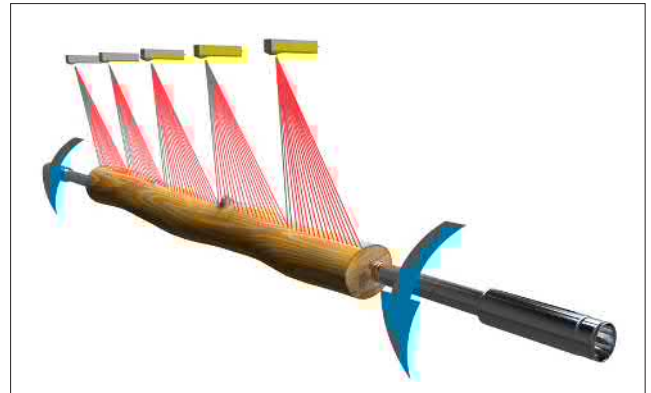


Figure 1.26. XY centring device with laser scanning

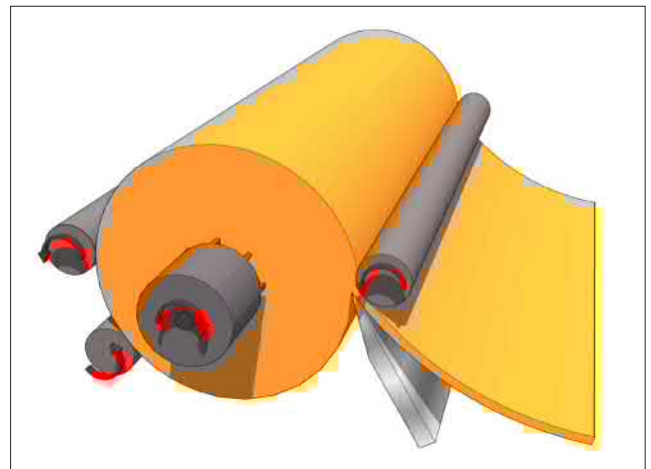


Figure 1.27. Peeling method



Figure 1.25. Peeler blocks being lifted from a conditioning chamber.

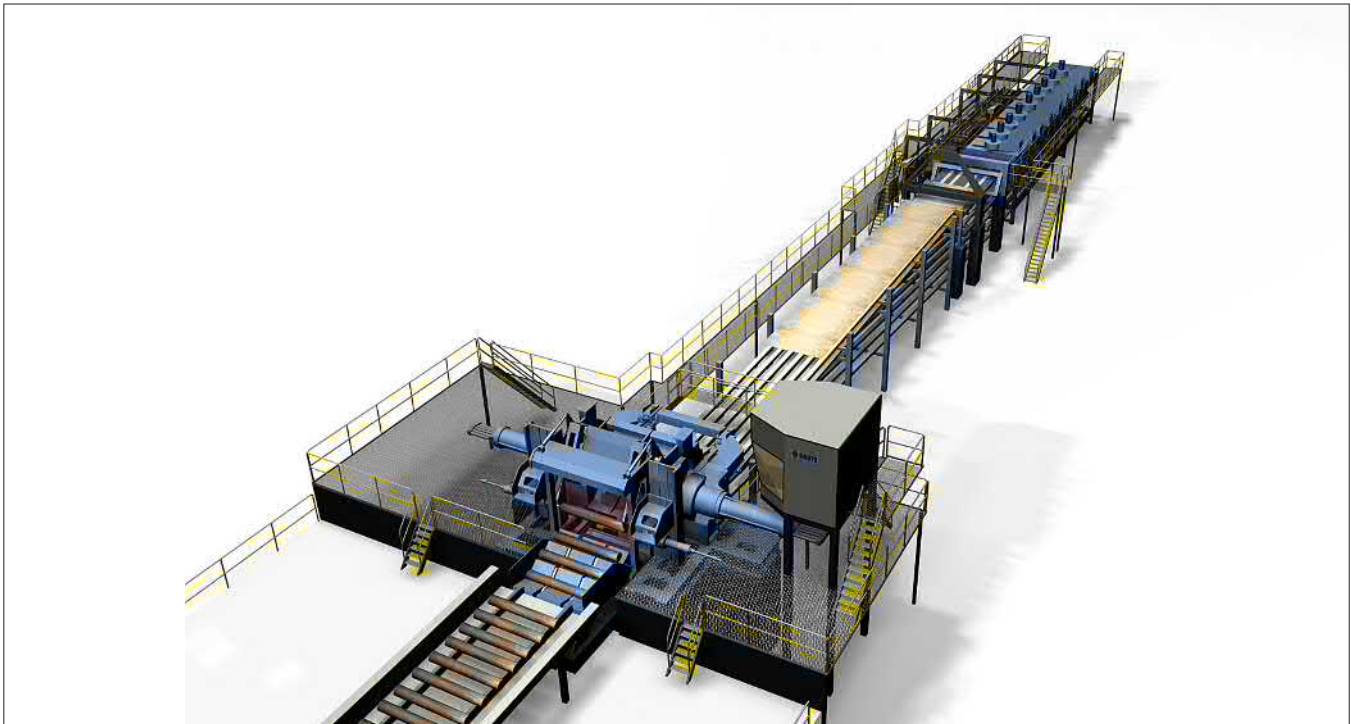


Figure 1.28. Peeling line, from block centring (left) to veneer mat clipping and stacking (right).

A colour camera technique is used to recognize and analyse even the smallest details on the veneer mat, such as knots, holes, splits, bark and rot. Based on this camera analysis, the system optimizes the points at which the mat is to be cut. The mat is then clipped into sheets. Defects such as splits, fishtails and large holes are cut out according to predefined parameters.

The clipped veneers are stacked in different bins according to their size and moisture content. Softwood is typically sorted into two or three moisture grades as the moisture content can

vary widely, from 30% to over 150%. Sorting of veneers according to moisture content enhances the efficiency of drying and attainment of the desired final moisture content.

1.6.3 Drying and grading the veneer

The aim of the drying process is to dry the green veneers to a moisture content suitable for gluing. Too high veneer moisture hampers gluing and generates steam during hot pressing.

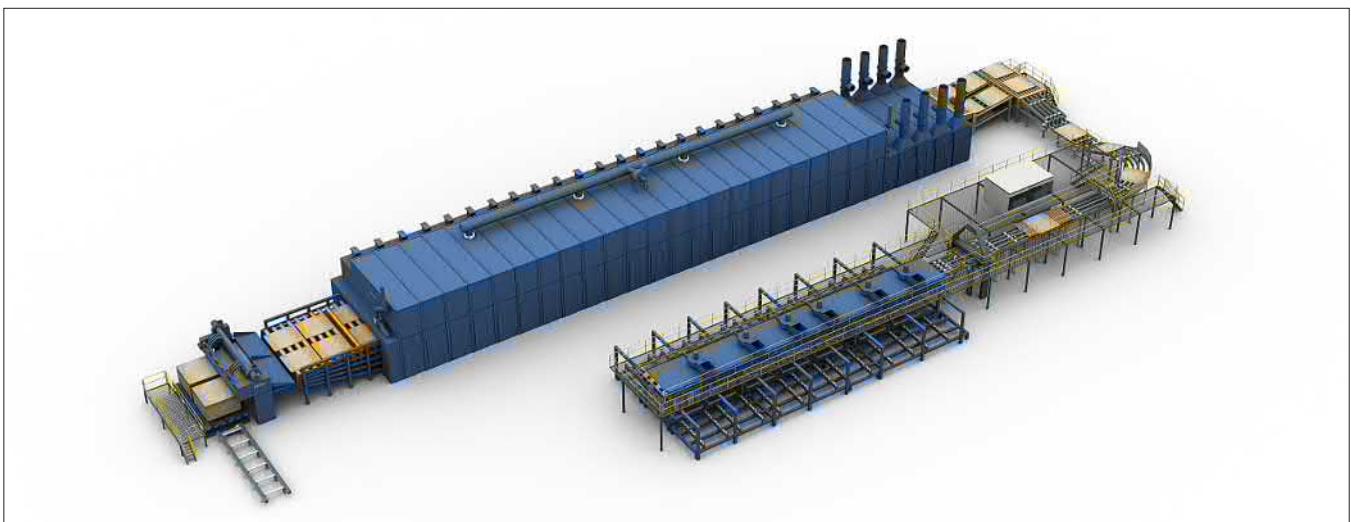


Figure 1.29. Drying line.

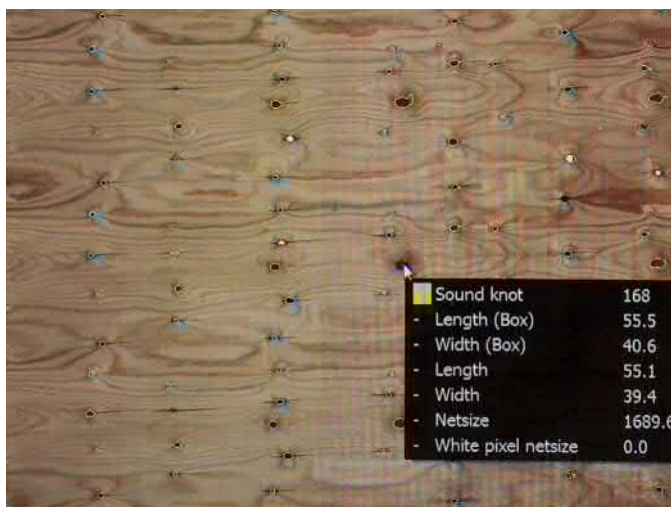


Figure 1.30. VDA camera image for visual defect analysis.



Figure 1.31. Dryer infeed.



Figure1.32. Stacker bins.



Figure1.33. Composed veneer.



Figure1.34. Storage of dried and graded veneers.

1. LVL AS A CONSTRUCTION MATERIAL

The target moisture content is below 5%. Automatic regulation maintains the speed, temperature and humidity inside the dryer at an optimal level. The energy required to dry the veneer can be produced from the mill's own by-products.

A fully automatic machine vision system analyses veneer defects, such as knots, splits, breakages, micro-splits, decay, resin pockets and discolouration, at process speed. The image data is analysed in milliseconds and used to stack the veneers into different bins at the stacker. The dried veneer is measured for moisture content to ensure the target moisture has been achieved. Density measurement and strength grading of the

veneer are also carried out based on radio frequency analysis and ultrasonic propagation time.

Veneer pieces of different sizes – i.e. 'randoms' – can be composed into veneer sheets of required width. Composed veneers are used as core veneers. The aim is to increase veneer yield by maximizing the use of peeled material. Randoms can be collected from peeling line and broken veneers from the drying line. Veneer can be composed green or dry. At the composer, the randoms are glued or taped together to form full-size veneer sheets for LVL production.

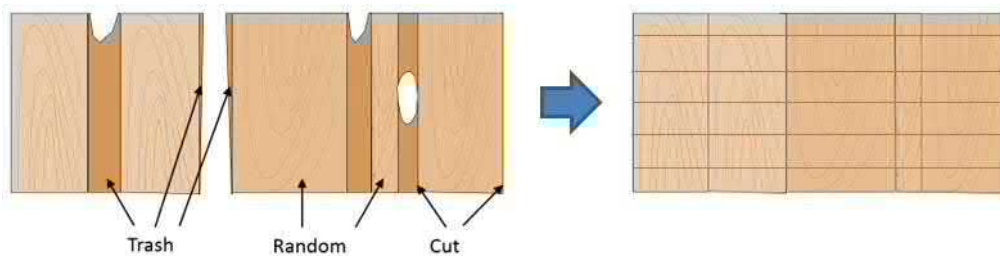


Figure1.35. Composing principle.

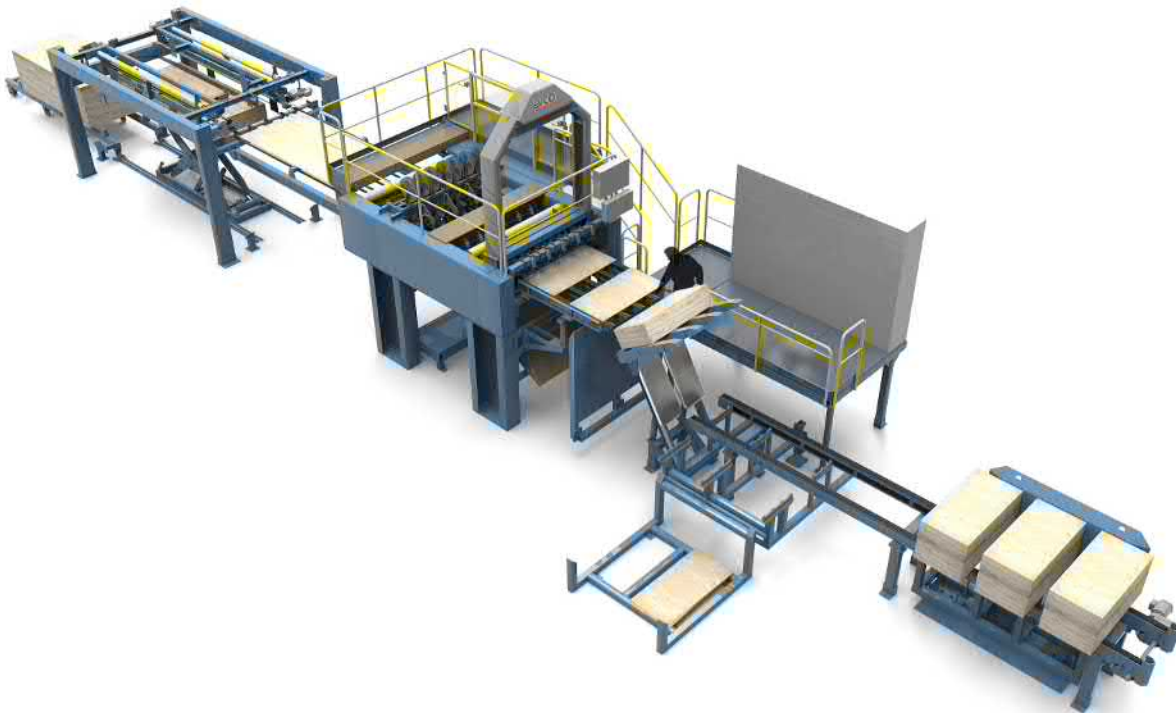


Figure1.36. Green veneer composing line.

1.6.4 Layup and hot pressing

Glue is mixed on site from resin, hardener and water. In the manufacture of LVL, phenol formaldehyde resins are mainly used at approximately 30 kg dry weight per cubic metre of LVL.

Veneer of the required length and proper bonding are achieved by scarfing a bevel at both ends of the veneer. The scarfed veneers are fed to the layup line, glued on one side, and then laid in a staggered configuration (see Fig. 1.40) to form a continuous billet of LVL. The veneers are normally fed parallel to the grain direction, although some veneers are laid cross-wise in certain products. LVL is typically assembled from an odd number of plies. Special structures can also be produced to meet specific end use requirements.

Layup starts at the layup table where the veneers are laid with a staggered edge spacing of approximately 120 mm (see Fig.19). The resulting staggered jointing maximizes the strength properties of the LVL.



Figure1.37. Scarfed veneer end.

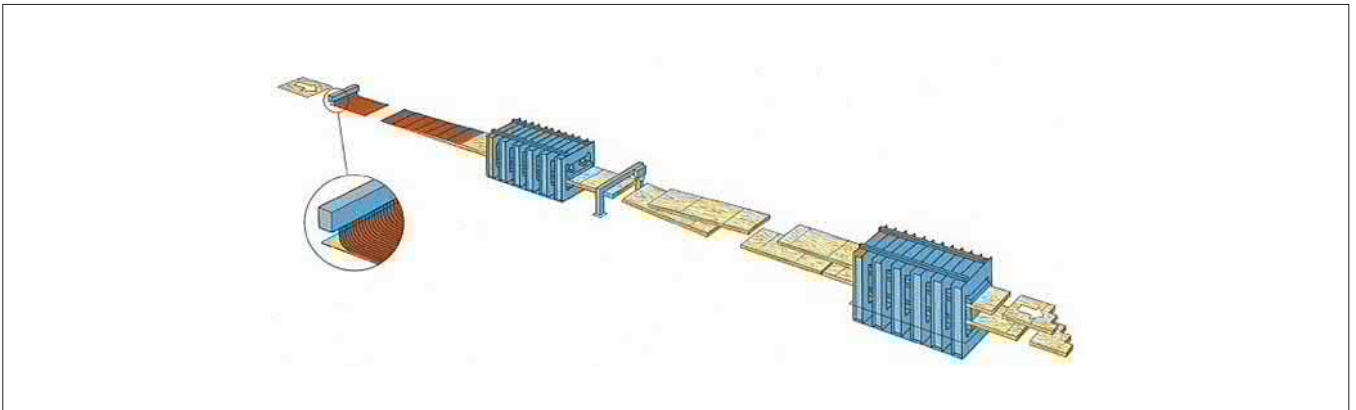


Figure 1.38 Layup line principle.



Figure1.39. Liquid extruder gluing (LEG): glue is applied to the top of the veneer as it passes on the belt conveyor.



Figure1.40. Staggered layup ahead of the pre-press.

1. LVL AS A CONSTRUCTION MATERIAL

Hot pressing is typically continuous to allow for variation in product lengths. The maximum LVL length (18-25 m) is limited either by the mill building or by delivery by road to the customer. Hot pressing must ensure that all glue lines reach the proper curing temperature. Once cured, the glue becomes resistant to melting and water insoluble, i.e. highly weather

resistant. With phenolic resins the pressing time ranges from 15 to 90 minutes depending on the thickness of the product. Long pressing times and high pressures cause the wood fibres to compress, resulting in higher product density.

1.6.5 Finishing

The LVL panels are normally cut to size according to the width and length specifications of the customer. Additionally, the LVL can be sanded or otherwise treated. Some manufacturers use portal-type sawing units that enable inclined, diagonal or chamfered cuts.

Finally, appropriate packaging protects the finished panels against soiling, moisture and handling damage, keeping them straight and easy to store and handle throughout the delivery chain from mill to customer.

1.6.6 Production summary

The homogenous structure of LVL together with production process optimization enable a wide range of structures and products to be cut to size with virtually no raw material losses. The veneer peeling system ensures that the best properties of the veneer log are used with maximum efficiency. Production volumes have increased globally and availability is improving as new manufacturers emerge in the market. Demand for LVL is continuing to rise as residential construction and awareness of the environmental friendliness of wood products are increasing. More detailed production descriptions can be found in the Wood-based Panel Industry publication released by Finnish Woodworking Engineers' Association 2018 ⁴.



Figure 1.41. Pre-pressing: the billet is cold pressed to spread the glue evenly between the veneers.



Figure 1.42. Hot press.



Figure 1.43. Feeding LVL billets into 3-level hot press.



Figure1.44. Sawing and packaging line.



Figure1.45. LVL packages.

1.7 FURTHER PROCESSING

LVL manufacturers offer further processing of their products according to customer specifications as value-added services. This is done either directly at the mill or by subcontractors equipped with special machinery for LVL processing. The value-added services save time and minimize waste for the customer and on the building site.

1.7.1 Sanded surfaces: optical or calibration sanding

Standard LVL is delivered unsanded. It can, however, be sanded with two alternative specifications. The visual appearance of LVL can be improved by optical sanding, which cleans and smoothens the surface by removing dark glue stains and equalizing local colour differences of the veneers. Optical sanding can be carried out on one or both sides and reduces the thickness of the product by approximately 2 mm (1 mm per surface). In visible applications it must be noted that on the front face of the LVL member the scarf-joints of the surface veneers are glued with colourless glue, but on the bottom side the scarf-joint glue line is dark brown, similar to the glue lines between veneers ¹¹.

Another sanding treatment that can be performed is calibration sanding for thickness calibration, which is carried out on both sides of the LVL. Calibration sanding reduces the thickness of the product by approximately 3 mm (1.5 mm per surface) and the thickness tolerance after calibration is ± 0.5 mm. Unless a non-transparent coating is used on the surfaces, calibration sanding is not recommended for visible applications because it can sand through the surface veneers revealing the dark glue line, especially in thicker products.

The nominal sanded dimensions of the product must be used in structural design calculations.

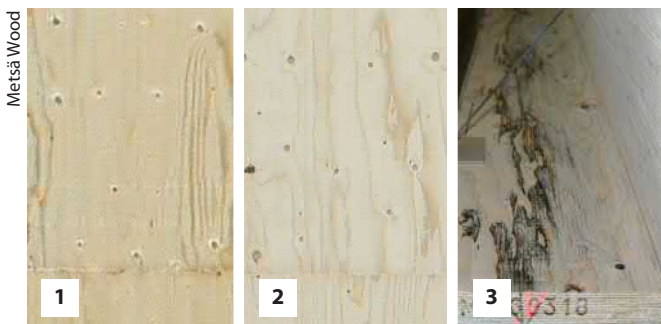


Figure 1.46. Appearance of LVL veneer surface 1. Unsanded face side. 2. Optical sanded face side. 3. Calibration sanding revealing the dark glue line.

1.7.2 Special cutting

LVL panels or beams can be sawn to special shapes or sawn diagonally to produce tapered beams or columns. Manufacturers may also have special cutting tolerances for tailored products, e.g. of industrial customers.

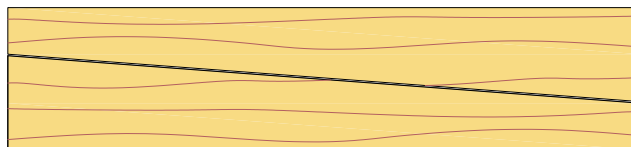


Figure 1.47. Principle of diagonal sawing of LVL panel to produce single-tapered column and rafter members of portal frames.

1.7.3 CNC Machining

CNC machining enables drillings, holes, notches and end sloping of beams. The machining requires a geometry file of the members from the customer, e.g. a DWG drawing at 1:1 scale drawn with closed lines for each different type of member to be machined.



Figure 1.48. Holes in LVL panels.



Metsä Wood

Figure 1.49. LVL-P roof beam cut to special shape.



Stora Enso

Figure 1.50. Hole in an LVL beam.

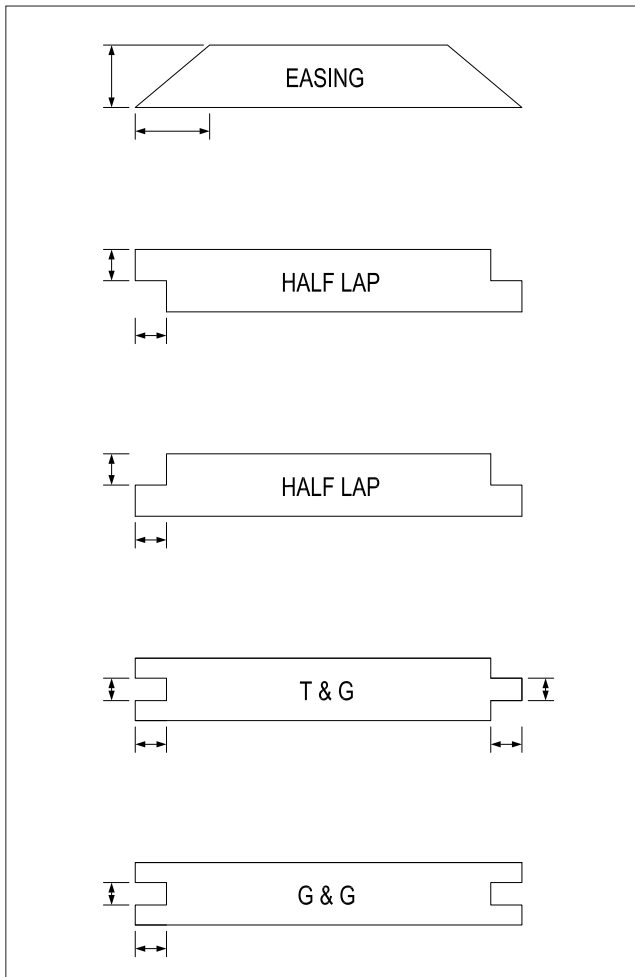


Figure 1.51. Edge profile alternatives for LVL components.

1.7.4 Edge profiling

Edges of LVL panels can be profiled with easing, half lap, tongue & groove (T&G) or groove & groove (G&G) profiles. Long edges can be profiled on a milling cutter line, and profiling of all edges can be done on a 5-axis CNC machine.

1.7.5 Surface treatments

LVL manufacturers provide different kind of surface treatments for their products:

- Moisture protection provides improved resistance against temporary weather exposure during storage, transport and on the building site.
- Treatment against mould reduces the risk of mould growth and blue stain and is recommended for use in service class 2 conditions, e.g. in roof structures of attics and shelters.
- Fire retardant treatments improve the reaction to fire classification of LVL members from D to C or B class, enabling a wider range of applications. Except for certain paints, fire retardant treatments can be used only in service class 1 conditions.
- LVL surface can be painted with fully pigmented non-transparent paints or stained with translucent stain or varnish.

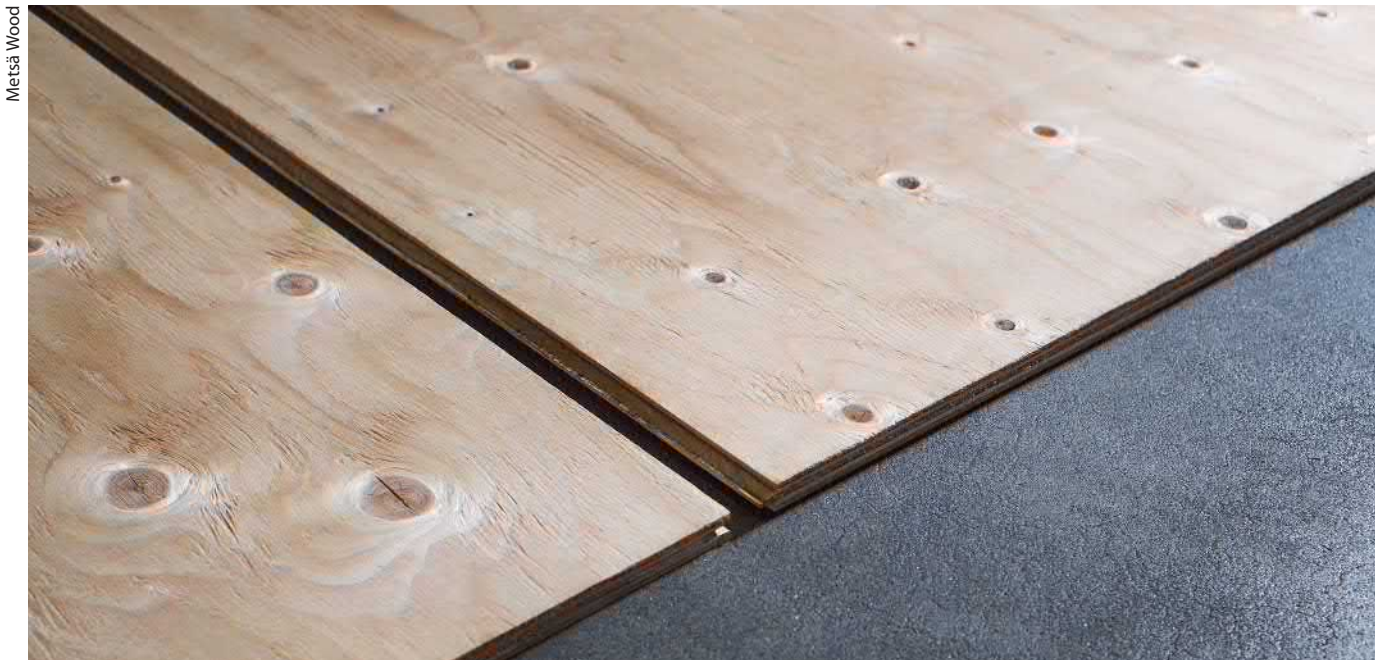


Figure 1.52. Edge profiled LVL-C panels.



Metsä Wood

Figure 1.53. Moisture protection treatment WeatherGuard®, Metsä Wood.

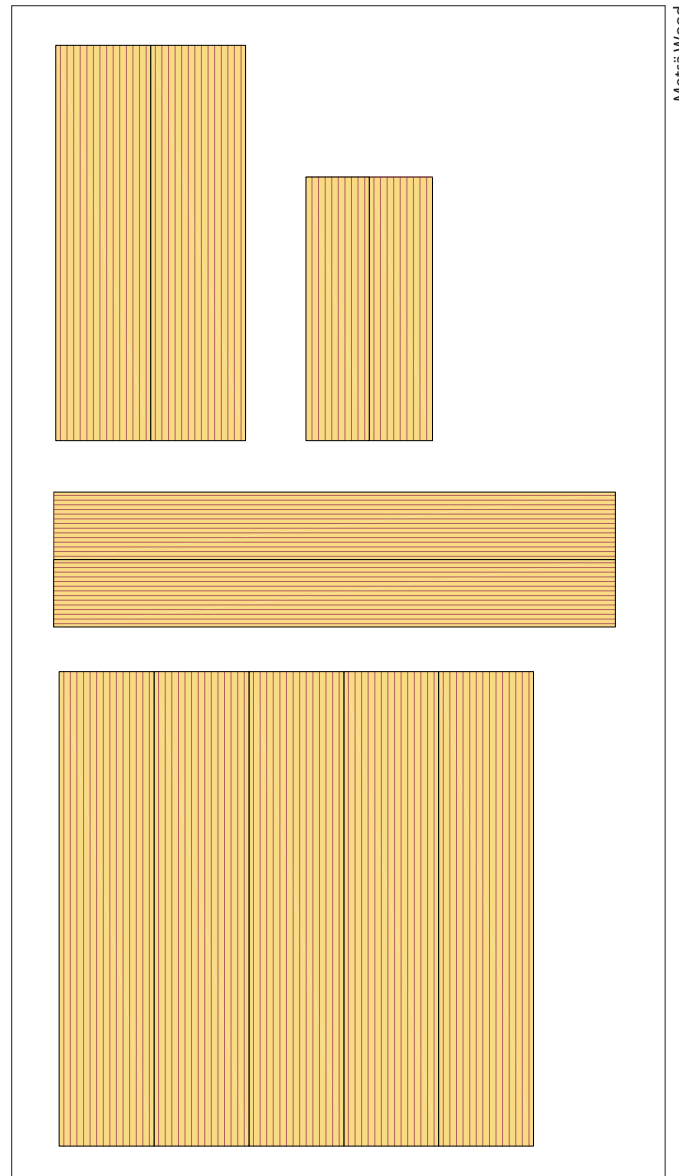


Stora Enso

Figure 1.54. Moisture protection treatment of LVL-P beams by Stora Enso.

1.7.6 Multiple-glued members, GLVL

Hot press gluing of LVL is feasible up to max. 75 mm product thickness, but larger thicknesses can be produced by multiple gluing of LVL laminas to produce GLVL. Multiple-glued GLVL members can achieve glulam beam dimensions. Wide panels can also be multiple-glued. Maximum thickness is limited in practice to around 500 mm by the capacity of saw units and lifting devices. PU, MUF and PRF adhesives that do not need hot pressing and that are approved for load-bearing structures can be used for multiple gluing of GLVL.



Metsä Wood

Figure 1.55. Multiple-glued GLVL beams and panels.

1. LVL AS A CONSTRUCTION MATERIAL

1.7.7 Combined structures

Structural elements can be produced by combining LVL members to form stressed-skin panels, box slabs, I-beams, box beams or built-up trusses.

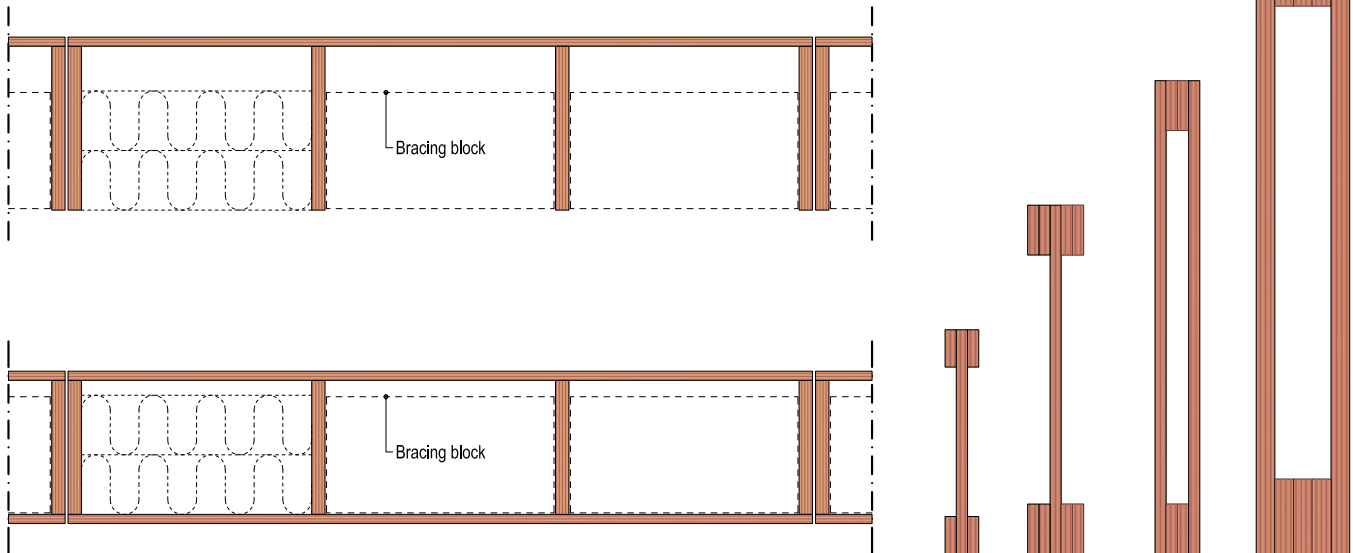


Figure 1.56. Stressed-skin panels, large I-beams and box beams from LVL.



Figure 1.57: Manufacture of structurally glued LVL roof elements.

1.8 LVL SIZES AND LAYUPS

LVL is produced in a continuous layup process from 3 mm veneers. This process optimally produces fairly thin, but wide and long beams, panels and studs cut from 1200-2500 mm wide billets. Maximum lengths range from 18-25 m depending on the production line. Due to the continuous process, the length can be individually specified for each billet. The minimum length is however limited to 2000 mm due to handling in the production process, so shorter components need to be cut separately. Standard widths are defined to minimize waste and to complement the dimensions of other building structures. The standard layups and product thicknesses are shown in Table 1.6.

The thickness to height ratio of LVL-P standard beams is limited to 1:8 to ensure dimensional stability in normal building site humidity conditions. Weather exposure on one side of a beam may lead to cupping of higher beams. Table 1.7 shows the standard sizes of LVL-P available from several manufacturers; these sizes are thus usually available in the shortest delivery times. LVL is easy to cut, so custom sizes are possible on separate request, making it an attractive option for many building systems. A more slender beam thickness to height ratio can be used if moisture conditions are controlled throughout the whole production chain, e.g. in off-site production of wooden roof elements for hall constructions.

Table 1.6. Nominal product thicknesses and layups of LVL-P and LVL-C.

Thickness [mm]	Number of veneers	Layup of LVL-P	Layup of LVL-C	Number of cross veneers in LVL-C
24	8		II-II-II	2
27	9		II-III-II	2
30	10		II-III-II	2
33	11		II-III-II	2
39	13		II-III-III-II	3
42	14			-
45	15		II-III-III-II	3
48	16			-
51	17		II-III-III-II	3
57	19		II-III-III-III-II	4
63	21		II-III-III-III-III-II	5
69	23		II-III-III-III-III-II	5
75	25		II-III-III-III-III-II	5

Table 1.7. Standard sizes of LVL-P beams.

Beam thickness [mm]	Beam height [mm]										
	200	220	225	240	260	300	360	400	450	500	600
27											
33											
39											
42											
45											
48											
51											
57											
63											
69											
75											

1. LVL AS A CONSTRUCTION MATERIAL

For structures that are sensitive to dimensional changes due to changing humidity conditions, LVL-C is the most suitable choice.

LVL-C beams and panels are usually produced according to customer specifications and their sizes are thus not standardized. The beam heights given in Table 1.6, however, ensure efficient use of material. Other common widths are 900 mm, 1200 mm, 1800 mm and 2500 mm, although the maximum panel width depends on the production line.

Standard sizes for LVL-P studs are smaller than beams. Thickness is typically 39 and 45 mm and the width is normally limited to 200 mm. Standard stud sizes for non-load-bearing

walls are 39x66 and 39x92 in lengths of 2550 mm, 2700 mm, 3000 mm, 3600 mm and 6000 mm.

No pre-cambering: Because LVL members are cut from straight billets, beams cannot be pre-cambered in normal production and pre-cambering should therefore not be included in the design.

Larger thicknesses are available as multiple-glued GLVL beams and panels. While standard sizes ensure the most efficient material use, custom sizes of GLVL are also available from GLVL suppliers for project-specific needs, but the availability shall to be checked case by case.

Table 1.8. Standard sizes of multiple-glued GLVL beams and panels. GLVL can be manufactured from LVL-P or LVL-C laminas.

Thickness [mm]	Beam height or panel width [mm]											
	200	220	225	240	260	300	360	450	600	900	1800	(2500)
84	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C	C
90	P	P	P	P	P	P	P	P	P			
96	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C	C
108	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C	C
120	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C
133	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C
144	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	P/C	C	C



1.58 LVL-C panel.



Figure 1.59 LVL-P wall studs.

Metsä Wood



Figure 1.60. Straight and accurate LVL-P beams.

1.9 TOLERANCES

The tolerances of LVL members are defined in FprEN 14374:2018 and depend on the member sizes. The tolerance values are shown in Table 1.9 and the dimension definitions in Figure 1.61.

Table 1.9. Maximum deviations from nominal sizes and nominal angles for LVL, unsanded and not pressure treatment (FprEN 14374:2018).

Nominal sizes		Maximum deviations
Thickness t	$t \leq 27$ mm	± 1 mm
	27 mm $< t \leq 57$ mm	± 2 mm
	$t > 57$ mm	± 3 mm
Width b	$b \leq 300$ mm	± 2 mm
	300 mm $< b \leq 600$ mm	± 3 mm
	$b > 600$ mm	$\pm 0.5\%$
Length l	$l \leq 5$ m	± 5 mm
	5 m $< l \leq 20$ m	$\pm 0,1\%$
	$l > 20$ m	± 20 mm
Maximum deviation α of the right angles of the cross section, see Figure 1.61		1:50 (approx. $1,1^\circ$)

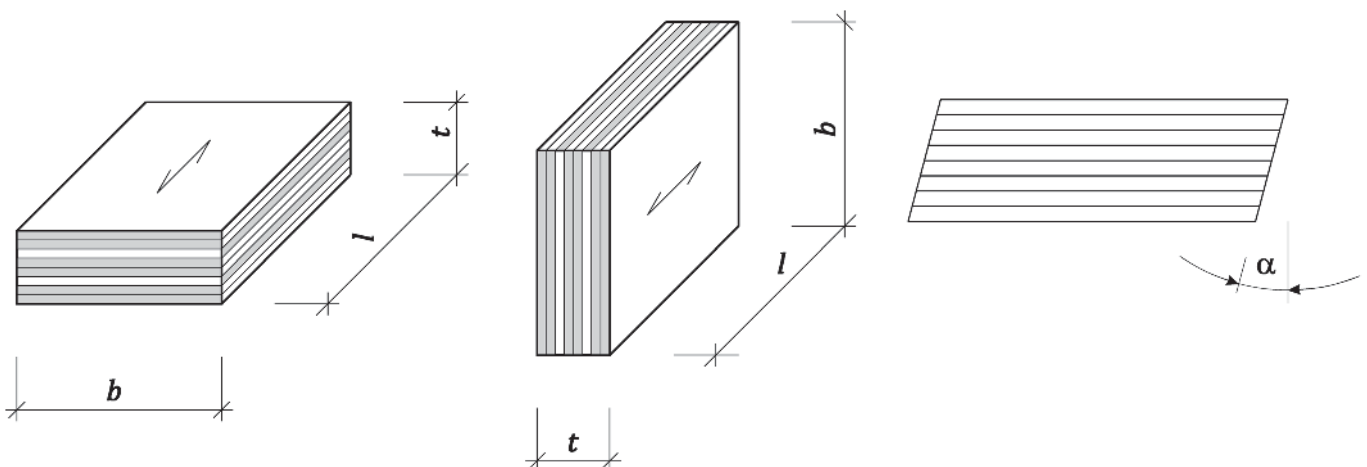


Figure 1.61. Dimensions of LVL. b = width (H =height), l = length, t = thickness. Arrow shows the grain direction of the surface veneer. Bottom: Example of the angle α deviation from the right angle of a cross section of LVL.

1.10 CE MARKING AND CERTIFICATION OF LVL PRODUCTS

Structural LVL has its own harmonized European Standard, EN 14374, which provides the basis for mandatory CE marking and Declaration of Performance (DoP) of LVL products. Structural elements e.g. stressed skin panels made of LVL can be CE marked based on supplier specific European Technical Assessments (ETA).

As a glued engineered wood product for load-bearing applications, structural LVL has high requirements for assessment and verification of constancy of performance (AVCP). European Commission decision 97/176/EC defines AVCP System 1 for structural LVL and the requirements of the AVCP System 1 are defined in the construction product regulations (CPR) of the European Union (Regulation (EU) 305/2011, Annex V amended by Commission Delegated Regulation (EU) No 568/2014). In the AVCP System 1 a notified product certification body decides on the issuing, restriction, suspension or withdrawal of the certificate of constancy of performance of the construction product on the basis of the outcome of the following assessments and verifications carried out by that body ¹²:

- An assessment of the performance of the construction product carried out on the basis of testing (including sampling) to determine the bonding strength (glue bond quality) and reaction to fire of the product.
- Initial inspection of the manufacturing plant and of factory production control
- Continuing surveillance, assessment and evaluation of factory production control covering e.g. the essential strength characteristics.

To fulfil the requirements of AVCP System 1, LVL manufacturers must carry out:

- Testing or assessment of essential characteristics relevant for the intended uses which are declared (modulus of elasticity, bending strength, compression strength, tension strength, release of formaldehyde and durability)
- Factory production control
- Further testing of samples taken at the manufacturing plant by the manufacturer in accordance with the prescribed test plan
- Declaration of performance (DoP) of the LVL products
- CE marking of the LVL products

Requirements for building products are set at the national level. DoPs and CE marking provide harmonized means of showing compliance with these requirements with respect to the properties included within the scope of the harmonized product standards. Based on these properties the structural LVL products can be designed in accordance with the Eurocode design standards system for load-bearing structures.

Note: LVL products treated against fire and biological attack cannot be CE marked according to EN 14374:2004, since the treatments are currently not included in the scope of the standard.

LVL products may also have other voluntary certificates for properties that are not included in the CE marking and DoPs, such as building physics properties, emissions, or certain nationally required design parameters. Examples of such certificates are the Eurofins Product Certificate, the M1 emission certificate in Finland, and Allgemeine Bauartgenehmigung in Germany.



Figure 1.62. Left: Example of CE-mark label in LVL package, right: Example of CE-mark label in LVL product.

1.11 DESIGN TOOLS

LVL structures are designed with similar computer-aided design tools to other load-bearing construction materials. Some structural calculation programs that support Eurocode compliant design also include LVL product libraries, but usually the user needs to enter the LVL product-specific properties into the software database. Examples of structural calculation software are Autodesk Robot Structural Analysis, Dlubal RF Timber, Frilo HO11+ and Mitek Roofcon/Trusscon. Some of their design results, e.g. support reaction capacity, however need to be verified manually as they do not include all of the LVL-specific parameters for EN 1995 (Eurocode 5). The definition of LVL strength classes in product standard EN 14374 will guide software development in the future. LVL manufacturers are also specifying the properties of their product brands individually and also providing tailored calculation software, such as Finnwood® and Calculatis®.

For basic structural drawings 2D design tools such as Autodesk AutoCad are commonly used. More detailed LVL component information can be utilized with 3D building information modelling (BIM). In addition to the planning phase of the project, BIM supports the whole building process from cost and construction management to facility operation and the whole life cycle of the building. LVL suppliers have created BIM libraries of LVL components for the Autodesk Revit, Archicad, Vertex BD, HSB Cad, Cad Works and Trimble Sketch-Up design software environments, among others. The libraries are distributed, for example, via manufacturers' websites or portal program tools such as ProdLib 13, 14.

Design software can create steering files for CNC machines used for cutting LVL components (CAM) to improve the efficiency of the whole manufacturing process. HSB Cad and Vertex BD software's, for instance, have a strong position in integrated chain support, from design to off-site manufacture.

The IFC file transfer formats 2x3 and ifc4 support some basic properties of engineered wood products and the next generation formats will take these better into account. For example, due to the orthotropic properties of the products it is important to be able to accurately define their orientation (lengthwise, edgewise, flatwise parallel to surface veneer and flat across the surface veneer) in the IFC objects.



Figure 1.63. Design software for LVL structures: Calculatis by Stora Enso, Finnwood by Metsä Wood and portal of BIM objects by ProdLib.

1.12 BASIC PROPERTIES OF LVL

1.12.1 Strength and stiffness properties

LVL has homogeneous material properties, firstly, due to the breakdown and uniform distribution of natural defects, such as knots, in the product and, secondly, due to the effect of lamination, which further eliminates their impact. Strength grading of the veneers also reduces variation within each strength class of the product. This results in strength levels that are close to defect-free wood for the highest LVL grades and, due to low variation, the characteristic 5% fractile values used in structural design are also high.

LVL-P has the highest strength and stiffness properties parallel to grain. LVL-C has about 20% lower values parallel to grain due to its cross-bonded veneers, but is stronger and stiffer perpendicular to the grain direction of the surface veneer, properties which can be utilized in panel structures. Table 1.11 presents the basic mechanical properties of the typical strength classes of LVL.

The variation in bending strength and stiffness properties for LVL is typically less than 10% compared to 12-20% for glulam and plywood and 15-30% for structural timber. Therefore, the characteristic 5% fractile values of non-LVL materials for structural design are significantly lower¹⁶. Table 1.12 compares the basic mechanical properties of some common structural wood products.

For more information on the mechanical properties of LVL, see Section 4.2.

1.12.2 Building physics properties

Moisture

LVL products are delivered from the factory at a moisture content (MC) of 8-10%, which is close to the MC of service class 1 end uses. This significantly reduces initial dimensional changes due to moisture in structures if the members are protected against weather exposure. LVL swells when its moisture content increases and shrinks when its moisture content decreases¹⁸.

The extent of these dimensional changes depends on the grain direction and the product type. Table 1.10. shows an example dimensional change for a 3% increase in moisture content. LVL-C undergoes a much smaller change in beam height because the cross veneers efficiently prevent movement in the height direction.

Untreated wood surfaces are hygroscopic, meaning that they absorb moisture from humid air and release moisture to the surrounding air when the RH is low. This moisture buffering phenomenon may be useful for improving the indoor air quality of buildings.

Thermal properties

LVL has a thermal conductivity λ of about 0,13 W/mK depending on its density and moisture content, and a specific heat capacity c_p of 1600 J/(kg K) according to EN ISO 10456.

Thermal expansion of LVL is negligible and its dimensions remain stable during temperature changes. Therefore, temperature variation does not need to be considered in structural design, unlike swelling and shrinkage due to moisture changes.

For further information on building physics, see Chapter 8.

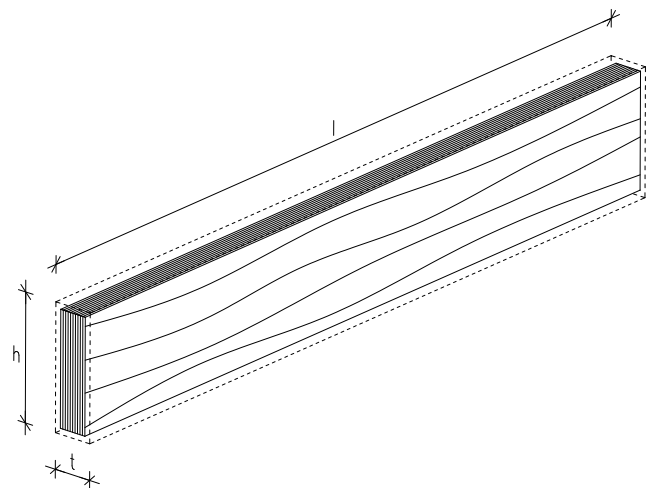


Figure 1.64. Dimensional changes due to increased moisture content.

Table 1.10. Example dimensional changes due to a 3% increase in moisture content (MC) %.

Product type	Direction	Original dimension	Dimension after +3% increase in MC	Difference
LVL-P	Length l [mm]	5000	5001,5	+1,5 mm
	Thickness t [mm]	57	57,5	+0,5 mm
	Height h [mm]	260	262,6	+2,6 mm
LVL-C	Length l [mm]	5000	5001,5	+1,5 mm
	Thickness t [mm]	57	57,5	+0,5 mm
	Height h [mm]	260	260,3	+0,3 mm

Table 1.11. Basic mechanical properties of common LVL strength classes.

		LVL 48 P	LVL 32 P	LVL 36 C	LVL 25 C
Typical use		Beam	Stud	Panel	Panel
Characteristic strength values, N/mm²					
Bending strength edgewise, $h = 300$ mm	$f_{m,0,edge,k}$	44	27	32	20
Bending strength flatwise	$f_{m,0,flat,k}$	48	32	36	25
Bending strength flatwise perpendicular to grain	$f_{m,90,flat,k}$	-	-	8	-
Compression parallel to grain	$f_{c,0,k}$	29	21	21	15
Compression perpendicular to grain edgewise	$f_{c,90,edge,k}$	6	4	9	8
Tension parallel to grain	$f_{t,0,k}$	35	22	22	15
Shear edgewise parallel to grain	$f_{v,edge,0,k}$	4,2	3,2	4,5	3,6
Shear flatwise parallel to grain	$f_{v,flat,0,k}$	2,3	2,0	1,3	1,1
Size effect parameter	$s, [-]$	0,15	0,15	0,15	0,15
Mean stiffness values, N/mm²					
Modulus of elasticity parallel to grain	$E_{0,mean}$	13800	9600	10500	7200
Modulus of elasticity perpendicular to grain in flatwise bending	$E_{m,90,mean}$	-	-	2000	-
Shear modulus edgewise	$G_{0,edge,mean}$	600	500	600	500
Density, kg/m³					
Mean value	ρ_{mean}	510	440	510	440
Characteristic value	ρ_k	480	410	480	410

Table 1.12. Basic mechanical properties of common structural wood products.

		Sawn timber C18 (EN 338:2016)	Glulam GL24h (EN 14080:2013)	Spruce plywood 21 mm ¹⁷
Typical use		Beam / stud	Beam	Panel
Characteristic strength values, N/mm²				
Bending strength	$f_{m,0,k}$	18	24	20,6
Bending strength flatwise perpendicular to grain	$f_{m,90,flat,k}$	-	-	12,8
Compression perpendicular to grain	$f_{c,90,k}$	2,2	2,5	-
Shear parallel to grain	$f_{v,k}$	3,4	3,5	3,5
Mean stiffness values, N/mm²				
Modulus of elasticity parallel to grain	$E_{0,mean}$	9000	11500	8230
Modulus of elasticity perpendicular to grain in bending	$E_{m,90,mean}$	-	-	3770
Density, kg/m³				
Mean value	ρ_{mean}	380	420	460
Characteristic value	ρ_k	320	385	400

1.12.3 Emissions and product safety

Phenolic adhesives that are cured at high temperature and wood raw material both contain small quantities of free formaldehyde. In Europe, the formaldehyde emissions of LVL products are tested according to the standards EN 717-1 (chamber method) or EN ISO 12460-3 (gas analysis method). The Class E1 requirement according to EN 14374 corresponding to $\leq 0,1$ ppm (EN 717-1) can be easily achieved with LVL products. LVL manufacturers commonly report significantly lower formaldehyde emission levels of $\leq 0,03$ ppm, and this limit stated in the voluntary certification of some construction product associations to demonstrate the low emissions of their products. For example, the German Qualitätsgemeinschaft Deutscher Fertigtbau (QDF) für Holzwerkstoffe QDF-Positivliste criteria states a formaldehyde limit of $\leq 0,03$ ppm.

For volatile organic compounds (VOC) a European classification is currently under preparation, but for the present different classification systems are used in different countries either voluntarily or based on legislation. For example, in Finland, LVL products are certified to show that they fulfil the M1 emission classification requirements of the Finnish Building Information Foundation RTS for building materials. The M1 classification criteria set limit values for total volatile organic compounds (TVOC), formaldehyde, ammonia, carcinogens and sensory evaluation ¹⁹.

After curing at high temperature, the adhesive bond between the LVL veneers becomes an inert polymer that does not dissolve or react with other materials in the surrounding environment. It is safe and non-hazardous to humans and animals.

Standard LVL products do not contain more than 0,1% of any of the Substances of Very High Concern (SVHC) listed in the Candidate List of the European Chemicals Agency, as these substances are not intentionally added to the products ²⁰. Manufacturers continuously monitor the Candidate List for updates.

LVL does not contain anything classified as hazardous waste, and has the following waste code in the consolidated European Waste Catalogue:

- 17 02 01 Wood (Construction and Demolition Wastes)

After use at the end of its life cycle, LVL can be utilized e.g. for bioenergy production ³.

1.12.4 Acoustics

LVL members can be used in timber frame structures in similar ways to other wood members to achieve the required sound insulation levels. Sound insulation of timber structures is usually good at high and middle frequencies if the structures have careful detailing so that they are airtight. However, timber structures are light, which makes it challenging to design adequate sound insulation structures against low frequency sounds. Therefore, layered structures are necessary for airborne sound insulation of intermediate walls between apartments. In floor structures additional mass and resilient layers

are needed to keep the impact sound level at low frequencies low enough.

Despite the challenges of low frequencies, real-life feedback from people living in well-designed wooden multi-storey buildings has been positive. The buildings are considered silent and the room acoustics of wooden buildings is usually regarded to be comfortable. One reason for this is that the surfaces of timber elements are less dense compared to steel or concrete, which is beneficial for sound absorption.

Robust perforated LVL panels can be used together with mineral wool insulation installed in cavities behind the panels for sound absorption, e.g., in sport halls and schools. The good impact resistance of these panels is also advantageous, e.g., in walls of halls for ball games.

1.12.5 Fire safety

When wood burns, a layer of char forms on the wood surface. This char layer serves as protective thermal insulation, inhibiting further burning of the remaining wood cross section. This makes the behaviour of wooden structures in fire predictable and their resistance to fire can be calculated based on the charring rates defined in EN 1995-1-2 (Eurocode 5). The one-dimensional charring rate β_0 of LVL is 0,65 mm/min, and the notional charring rate β_n for beams and columns is 0,70 mm/min when the characteristic density is ≥ 480 kg/m³.

As LVL cross sections are typically thin, with a product thickness of max. 75 mm, they usually require additional protection to achieve the required fire resistance. This is usually achieved with gypsum plasterboard panelling directly onto the LVL members, or onto LVL frame structures with cavities that are left empty or filled with mineral wool insulation. Eurocode 5 provides instructions for calculating resistance to fire.

The risk of flame spread is controlled by reaction to fire class classifications of construction products. The class for untreated LVL is D-s2,d0, which is the same as solid wood, where D is the combustibility class, s is smoke production and d is burning droplets. The classification may be improved with fire retardant treatments up to class B-s1,d0 for some structures, mainly in indoor applications.

For more information about fire safety, please see Chapter 6.

1.12.6 Structures for seismic areas

LVL structures can be used in seismic areas. Unless special assessments are made, LVL structures are intended to be used subject to static or quasi-static actions. In seismic areas the behaviour factor of LVL panels used for the design is limited to non-dissipative or low-dissipative structures ($q \leq 1,5$), defined according to Eurocode 8 (EN 1998-1:2004 clauses 1.5.2 and 8.1.3 b) and to applicable national rules on construction work.

LVL structures have low weight which reduces the design loads in seismic design. LVL-C panels are available in large sizes, so they can be used as robust panel bracing components to

resist seismic loads. LVL-C members, in particular, are not sensitive to cracking or brittle failures of connections, so they can yield and absorb more energy. Full utilization of these properties and use of LVL in higher dissipative classes for structures in seismic design requires testing in cyclic loading according to EN 12512 for combinations of LVL panels and dowel type connections.

1.12.7 Visual properties of LVL surface

Softwood LVL is produced from peeled conifer softwood veneers. An inherent characteristic of conifer wood is that branches are located in star-like clusters along the stem (knot ringlets) and therefore there may be frequent knots in a peeled veneer.

LVL is mainly used as a structural load-bearing product for non-visible applications. Therefore, the sorting of veneers in production is based mainly on the strength properties of the veneers, not on their visual properties. During peeling, small peeling cracks are formed in the veneer. These cracks may become visible due to swelling or shrinkage caused by moisture variations or sanding. In addition, the scarf-joints of face veneers can sometimes overlap, leaving the joint slightly open. Due to moisture variations this may also occur later on the surface of sanded products. Other possible surface defects include resin pockets, bark, and splits. Unsanded surfaces may have

some stains of adhesive from the production process. Some manufacturers provide higher grade face veneers on special request.

On the front side of the product a light-coloured melamine adhesive is used for scarf-jointing the surface veneers. On the reverse side the scarf-joints are glued with the same dark brown phenol resin adhesive used in the glue lines between veneers. The distance between scarf-joints is usually 1.9 m or 2.5 m depending on the production process. When LVL members are used in visible applications, the designer specifies which side of the member is the visible surface. This must also be taken into consideration especially when the LVL is to be cut into special shapes.

Standard LVL is delivered unsanded. It can, however, be sanded to improve the visual appearance of the LVL surface by so-called optical sanding which cleans and smoothens the surface by removing any dark glue stains and equalizing any local colour differences of the veneers. Another alternative sanding treatment is calibration sanding, which can be carried out to achieve more precise thickness tolerances for, e.g., LVL door components. Unless some non-transparent coating is used on the surfaces, the calibrated sanding is not recommended for visible applications, because it can sand through the surface veneers revealing the dark glue line especially in the thicker range of the products 11. For more information on sanding specifications, see subsection 1.7.1.

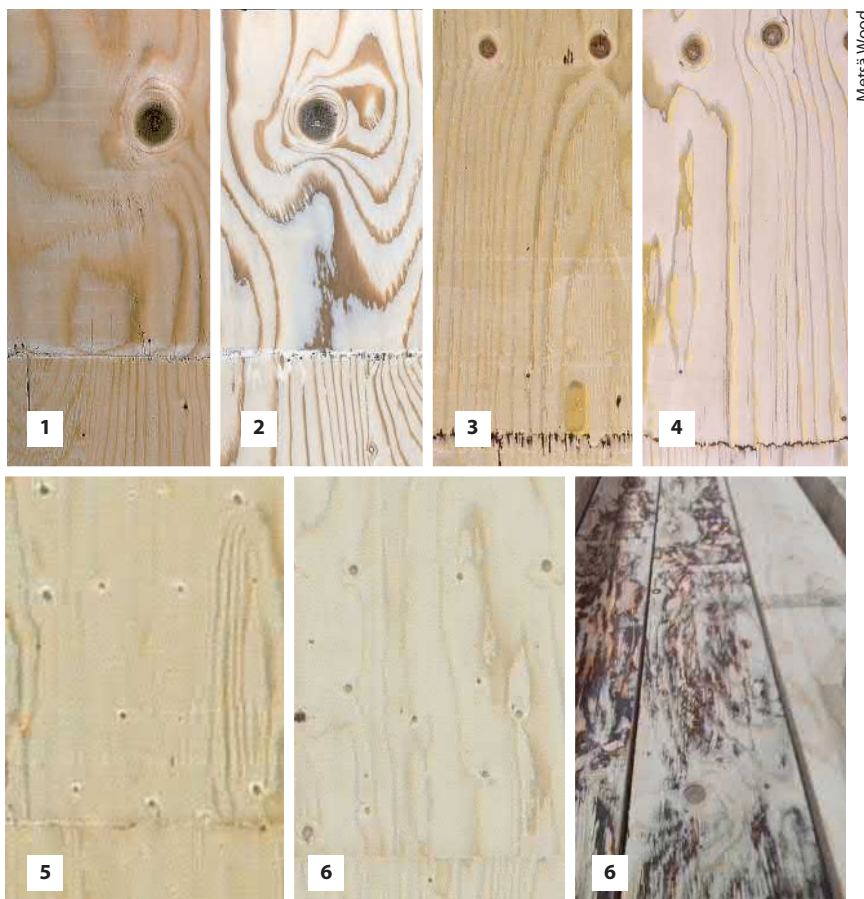


Figure 1.65: Appearance of LVL veneer surfaces
1. Unsanded front side, close image 2. Optical sanded front side, close image 3. Unsanded reverse side with dark scarf-joint glue line, close image 4. Optical sanded reverse side with dark scarf-joint glue line, close image 5. Unsanded front side, general view 6. Optical sanded front side, general view 7. Calibration sanding revealing the dark glue line ¹¹.



Figure 1.66. Coated surfaces of LVL beams.

1.12.8 Surface coating of LVL

LVL can be painted with fully pigmented non-transparent paints or stained with translucent stain or varnish. Due to peeling cracks, the surface of LVL is more demanding to paint than, e.g., solid wood. The cracks occur in the face veneers as a result of contraction and swelling of the product. In indoor conditions these cracks are not usually problematic, but in applications exposed to weather, a pigmented covering paint is required to protect the LVL. If the moisture nevertheless penetrates the LVL member, for example at the penetration points of connectors, at the edges or other similar areas, a thick paint coating will tend to flake. This is especially evident on large continuous surfaces. If surface cracks are not problematic for the application in question, a light non-film-forming surface treatment can be a suitable alternative. This may require more frequent maintenance, but the maintenance treatment is easier to perform.

The durability of the coating depends on various factors: the base material and how it has been prepared; the level of

exposure to sunlight; the colour and darkness of the coating; and the level of exposure to moisture. General guidelines for coating LVL:

- Apply the coating as soon as possible to prevent the effects of UV radiation on the wood surface. If needed, sand the surface prior to coating.
- Fill any voids, such as fallen knots, with substitute wood compound.
- Choose a primer with blue stain and mould protection.
- Round the edges of the member slightly to ensure adherence of an adequate thickness of coating also on the edges. Edge coating is especially important for all veneer-based wood products.
- Apply the coating agent to an adequate thickness. Apply the coating agent in at least two layers to ensure proper drying of the layers.

In all cases of coating LVL the application should be discussed in detail with the coating agent manufacturer to ensure the suitability of the product for the application ²¹.

1.13 JOINTS BETWEEN LVL PANELS

Structurally supported or self-supported joints may be used for the connections of LVL-C panels. The load-bearing capacity depends on the project and must be verified case-specifically, including the connector diameter, minimum insertion depths, spacing, and edge distances. For connector positioning, the acting force direction is parallel to the joint as a default.

Figure 1.67 illustrates the principles of panel jointing. In structurally supported panel joints, the panel edges are usually straight and the force acting parallel to the joint is transferred through the load-bearing structure from one panel to the next.

Where other structural features allow, a simple solution for a self-supporting joint is to use straight-edged panels and fasten a wooden or steel batten on or under them. Otherwise, the edges of the panels require machining. A common type of machined joint is a half-lap connection where part of the panel thickness is machined to create matching pairs of edges. A separate batten can be placed on the half-lap edges as a surface spline or as an interior spline when the edges are grooved to a tongue profile. Suitable nail and screw sizes are defined in Section 4.7.

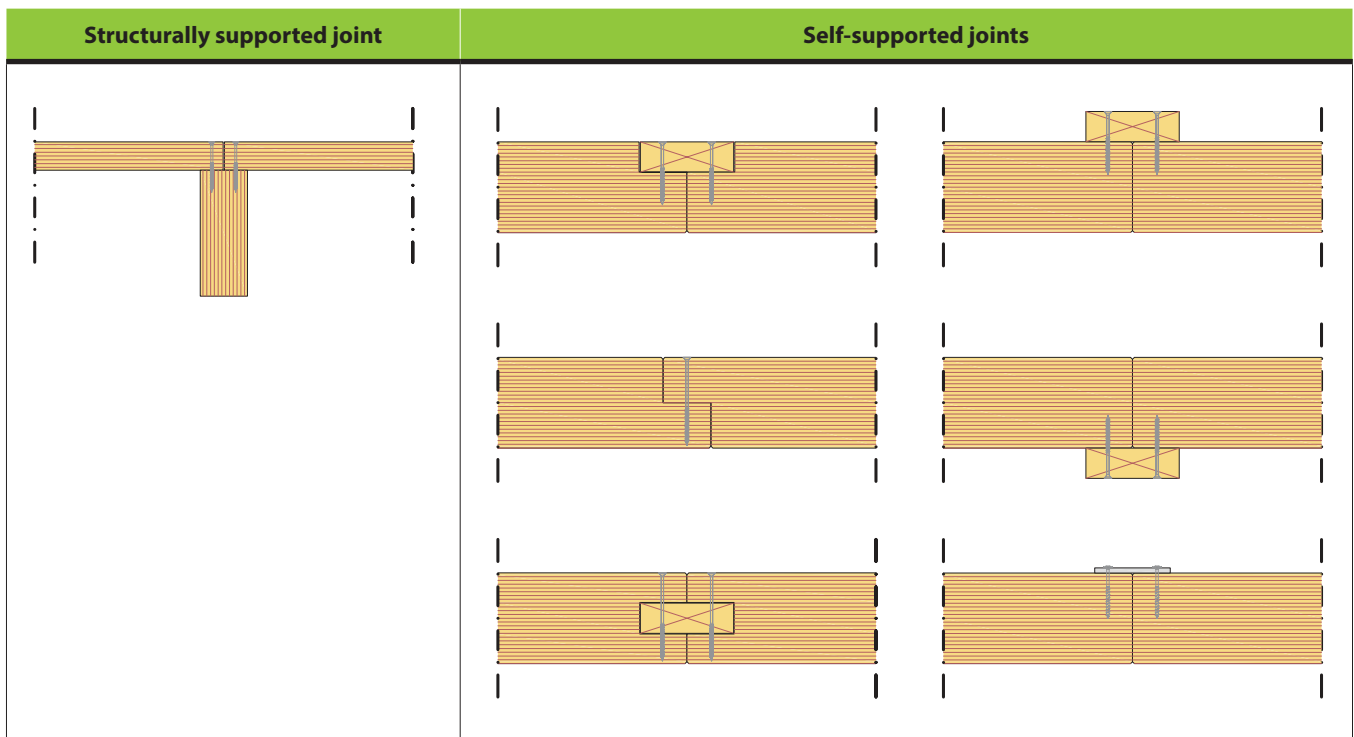


Figure 1.67. Structurally supported joint and self-supporting joints of LVL-C panels.

2. LVL structures in floors, walls, roofs and in special applications

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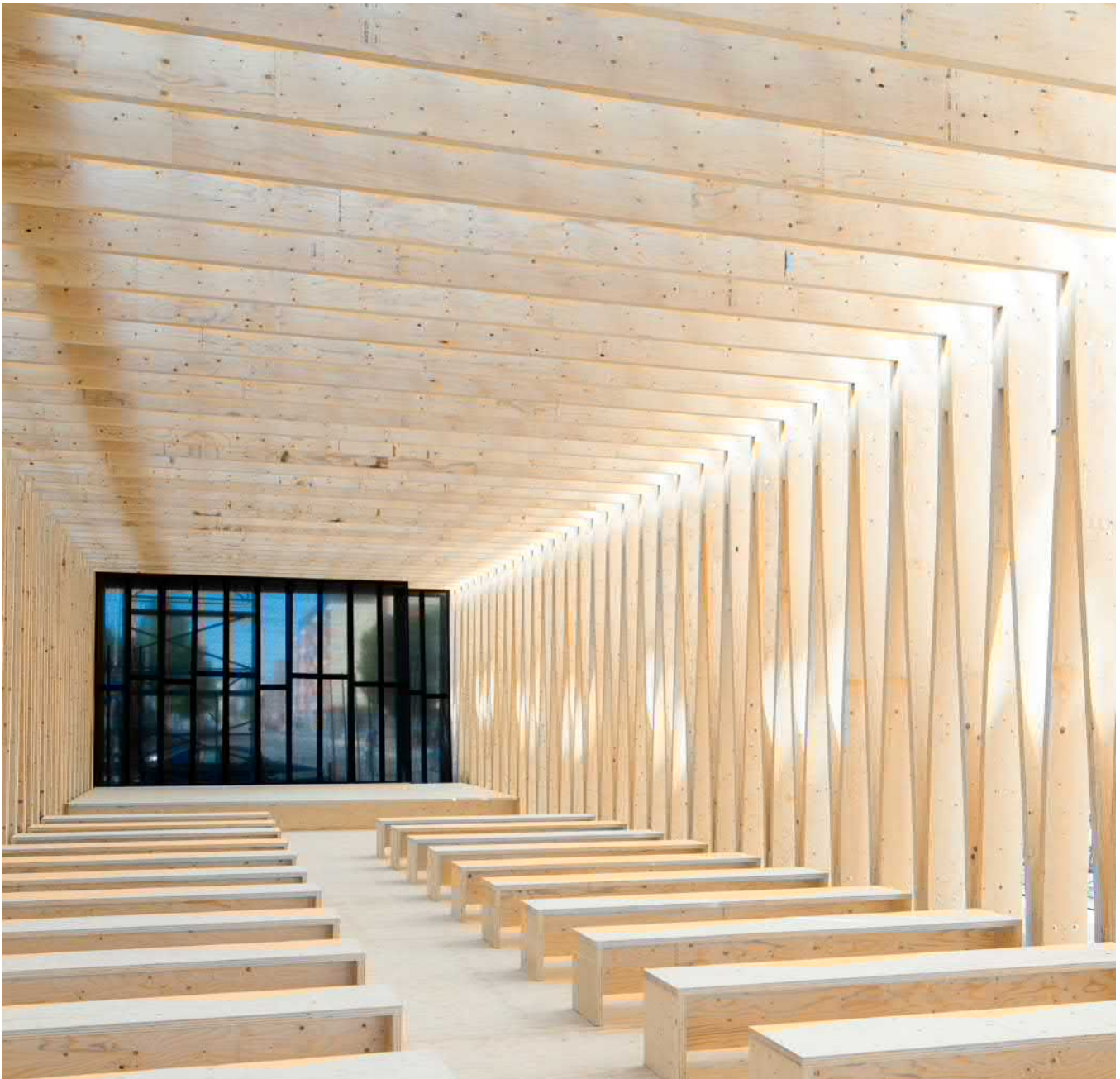


Figure 2.1 Aika stage, Aalto University, Finland.

2.1 GENERAL

LVL is available in a large variety of standard stock sizes, as well as in user-defined cross sections. Tall and slim beams and studs are useful, for example, for reducing heat loss through wall, floor and roof structures, while optimized cross section improve material efficiency and high bending stiffness minimizes deflection. Long LVL beams are well suited as headers and multi-span purlins in roof constructions. LVL can be easily combined with other structures with standard connection parts and simple joint details. LVL material values and design methods are carefully tested to provide reliable information for designer to specify robust load-bearing wooden structures.

LVL-P has the highest bending strength and stiffness properties and is therefore the standard choice for beam structures. However, sometimes structural details can cause local stresses, such as connection forces at an angle to the grain, and, in such cases, LVL-C with its reinforcing cross-bonded veneers is the preferred product. LVL-C has better stiffness, resistance and dimensional stability in the cross direction and is therefore recommended for members mainly loaded in the cross direction, such as rim boards.

2.2 FLOOR STRUCTURES

LVL offers numerous alternatives for creating floor structures, from traditional joist floors to stressed-skin panels and massive floor panels. LVL-P joists have precise dimensions and good modulus of elasticity, which are important for a robust and convenient floor. Stressed-skin panels can achieve the longest spans, and massive LVL panels minimize the height of the floor structure. Straight, precise and dry LVL-P joists are ideally suited for off-site production of floor elements.

2.2.1 Beam & joist floors

In wooden joist floors the serviceability design of deflections and human-induced vibration from walking are usually the most important design tasks. Wooden joists are strong, so the strength design is rarely an issue if the supports are wide enough. LVL joists are ideal for timber frame floors as they are rigid and straight with accurate dimensions. They are supplied at a moisture content close to that of the ready building, which reduces deformation and creaking.



Figure 2.2. LVL-P joist floor structures.



Figure 2.3. Joist hanger connections of floor joists.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

LVL joist floors can be built on a wooden, steel or concrete frame or walls. The 1:8 width to depth ratio of rigid LVL-P joists is ideally suited to the structural depths of floor constructions even with long spans. The recommended minimum width of supports and LVL floor joists is 45 mm in order to provide proper support for decking panels.

Floor elements can be prefabricated off-site. This saves time on the construction site, reduces waste, and improves quality as the elements can be made in dry factory conditions. An element floor usually has a slightly higher material consumption as the edge joists that seal the elements at the sides form double joists in the completed floor structure.

Floor deflections and vibrations are not only dependent on the floor joists themselves, but on the decking on the joists and on the blockings installed for transverse bracing perpendicular to the span of the floor. Due to the rigidity of typical decking panels, a joist spacing of c/c 400 mm is recommended. The spacing should not exceed 600 mm to avoid excessive deflection of the panels in the perpendicular direction to the joists. It is also recommended to glue the decking panels to the joists with polyurethane adhesive for best results. Depending on the gluing conditions (on-site or off-site) and country-specific requirements, at least half of the advantage of the composite effect of gluing can be utilized in structural calculations.

Blockings between the joists reduce deflection under point loads, but they need to be fixed well to the joists to give the desired transverse stiffness and to avoid creaking in the long term. The best improvement can be achieved by fixing the

blockings to a transverse tension flange board installed under the floor joists, see Figure 2.6.

Multi-span structures reduce deflection and enable longer maximum span lengths compared to single-span structures. However, if the floor joists are continuous between separate rooms more conservative span lengths are recommended as people are more sensitive to vibrations originating from spaces external to the space that they are occupying.

Floor vibration is controlled in the design based on: 1) deflection under point 1kN load when the lowest natural frequency f_1 is higher than 8-9 Hz depending on country-specific design rules and 2) acceleration when f_1 is 4,5-8 Hz. It is recommended to design joist floors to $f_1 > 8$ Hz, because achieving the requirements of the acceleration criterion requires the addition of considerable extra weight to the structure. For more information on floor vibration design, see subsection 4.3.7.

Residential floors within apartments can be light, without acoustic or fire resistance requirements, in which case deflection under 1kN point load is the governing requirement. Intermediate floors between apartments require protective cladding underneath the floor and extra mass on the top of the floor, which usually make the fundamental natural frequency $f_1 > 8-9$ Hz the most critical requirement. Figures 2.7 and 2.8 gives the maximum span lengths for different joist sizes. Span lengths up to ~6 m can be achieved with normal floor thicknesses, but with the biggest joists over 8 m spans are possible.

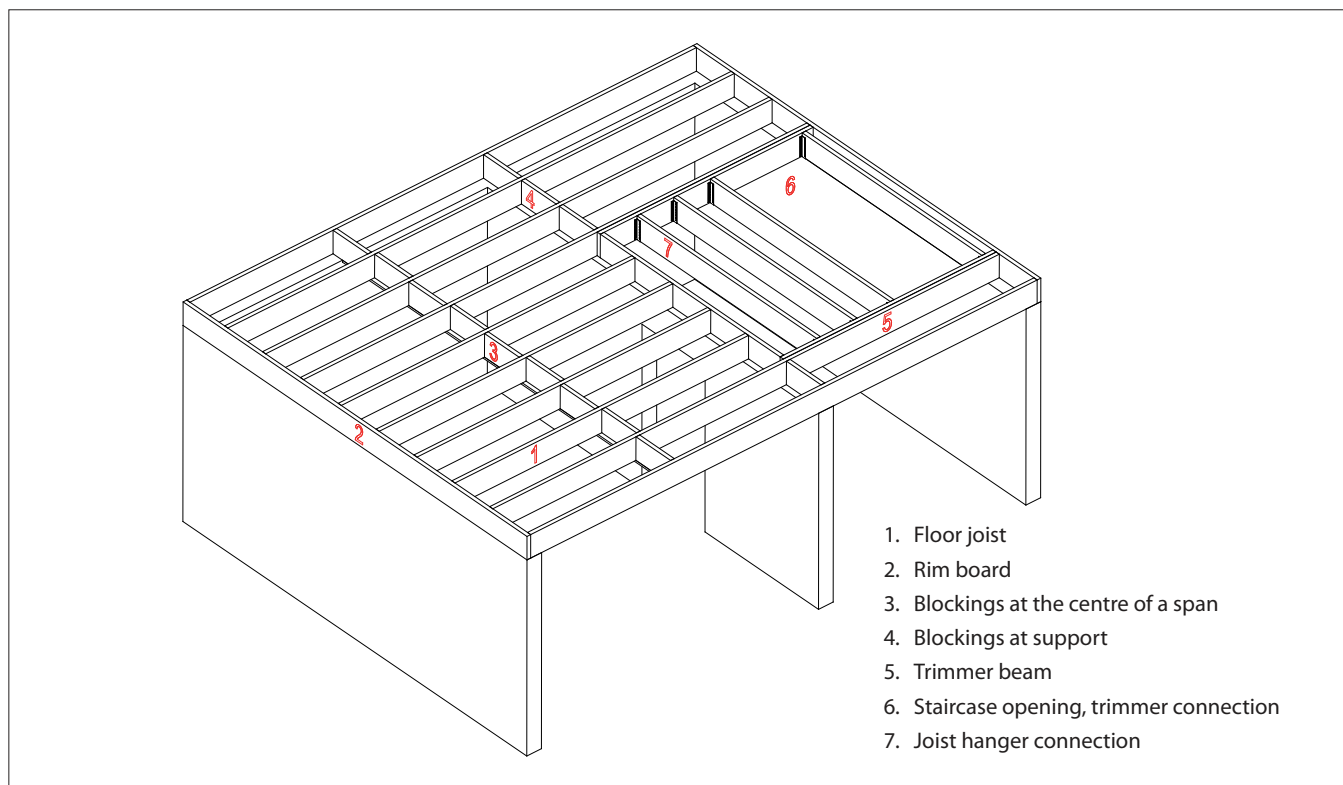


Figure 2.4. LVL-P joist floor members and details.

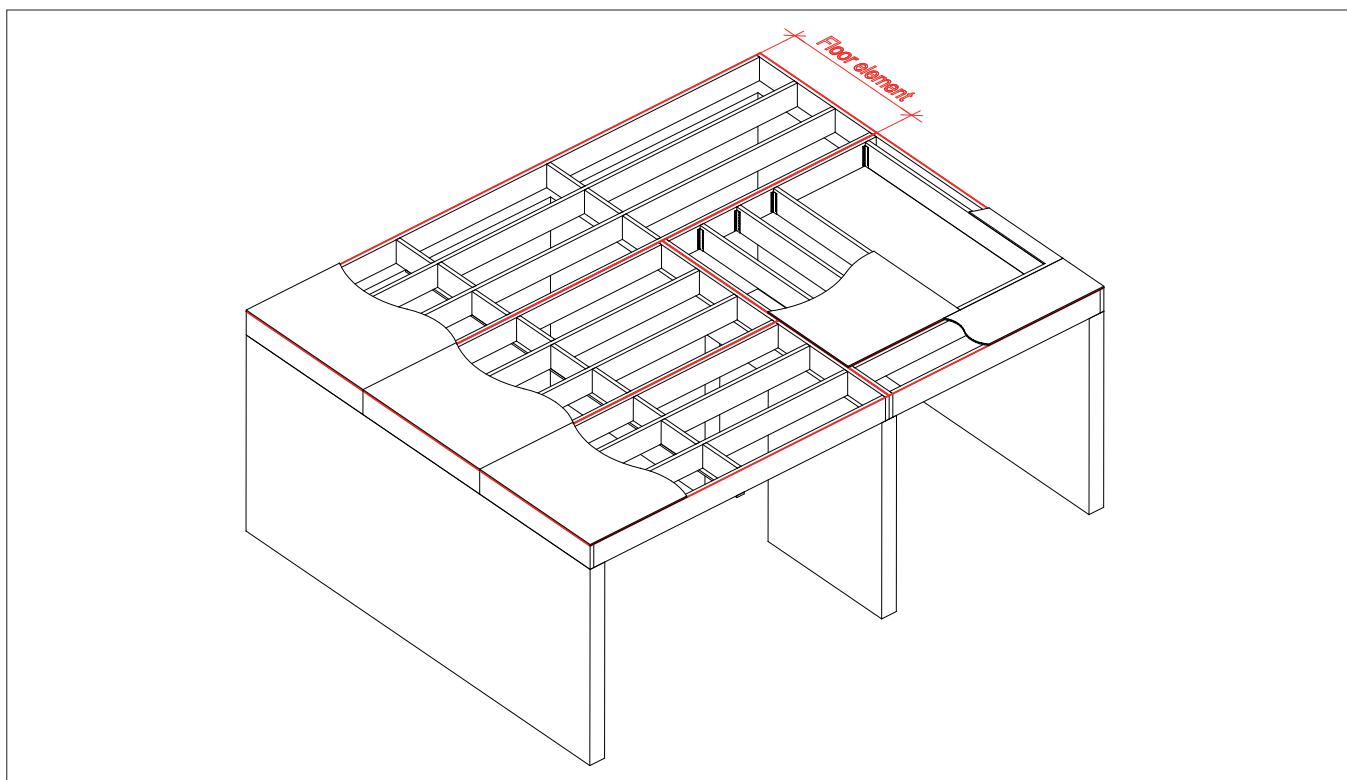


Figure 2.5. LVL joist floor with decking from offsite produced elements.

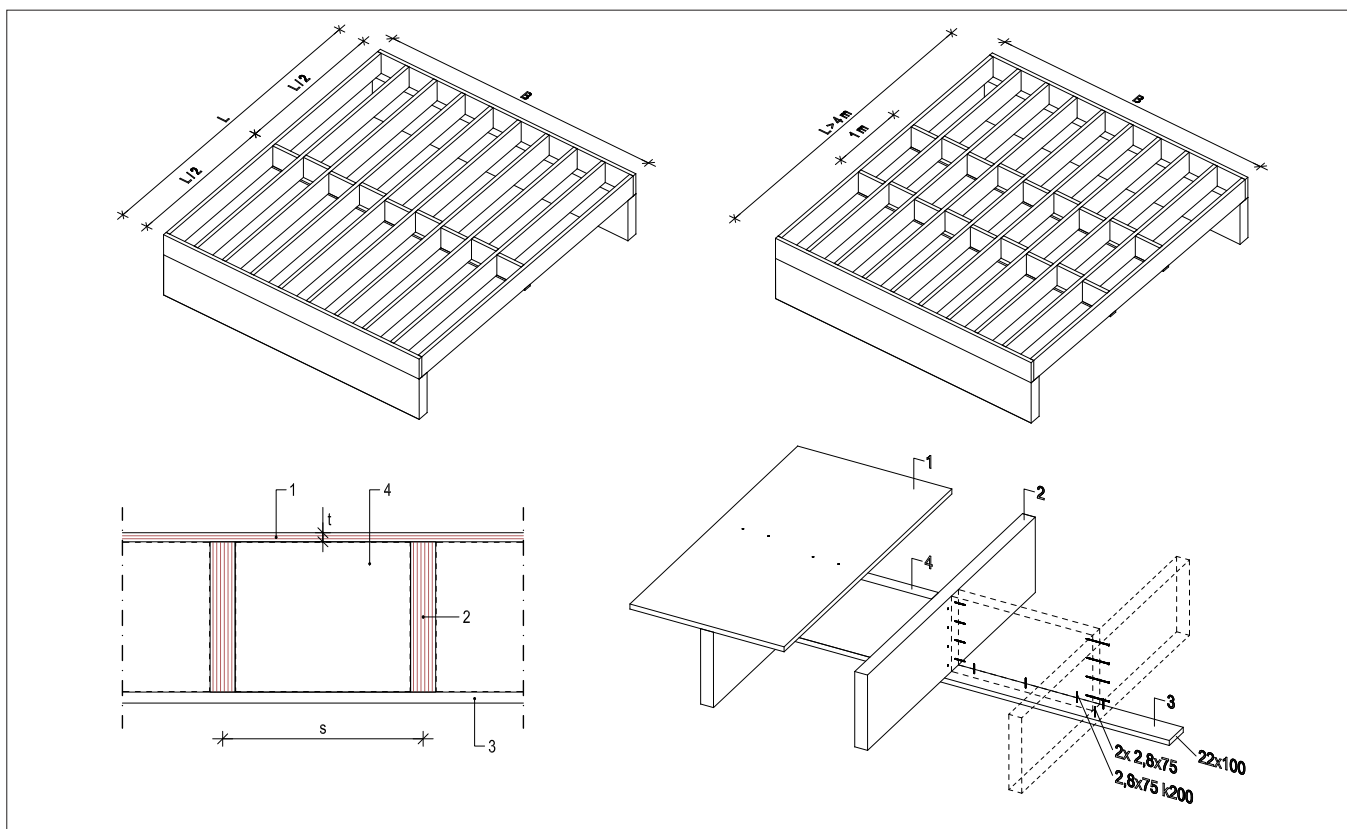


Figure 2.6. Transverse blocking and tension board below the floor joists for reducing deflection under point load and improved performance against floor vibration. Two blocking lines at 1m spacing at the centre of the span are recommended for $L > 4\text{m}$ span length. 1. Decking panel, 2. Floor joists, 3. Transverse tension board (C18, min 22x100) under the joists fixed with 2,8x75 nails to the joists and blockings. 4. Blocking.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

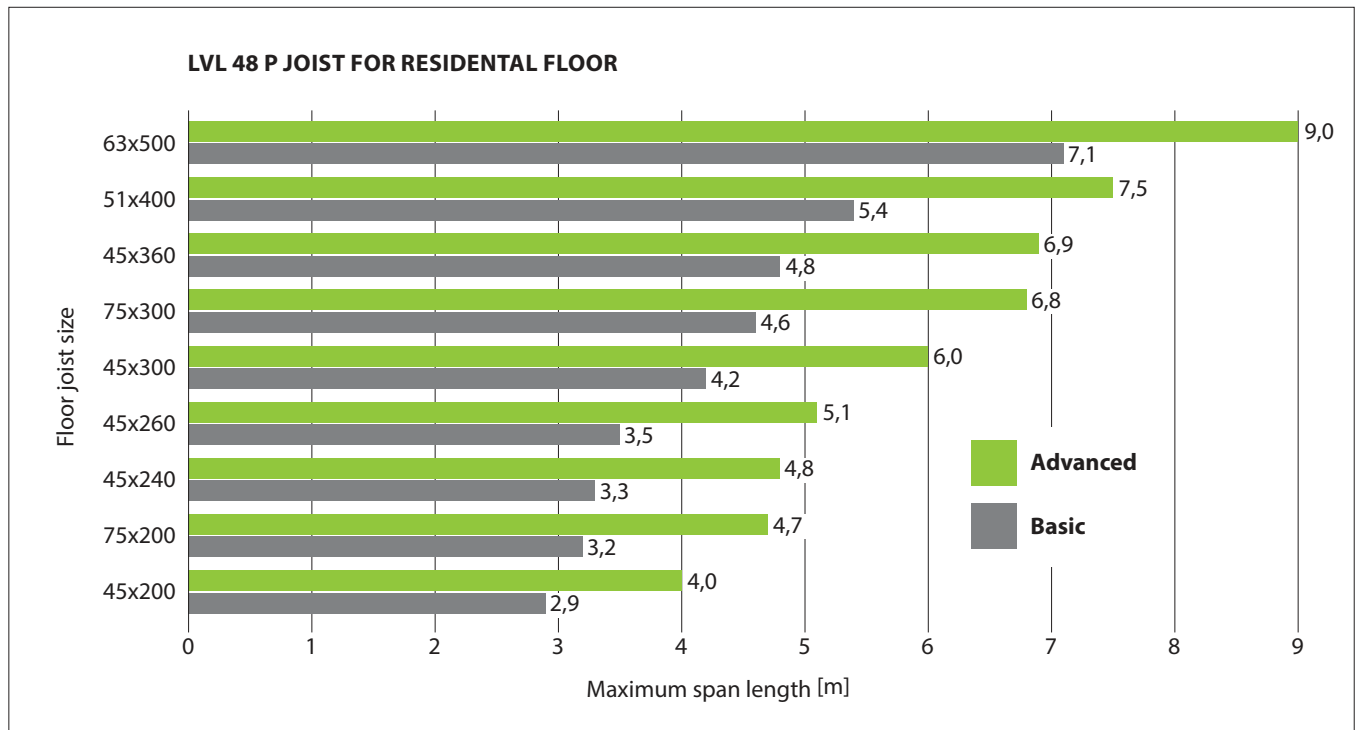


Figure 2.7. Maximum span lengths of LVL 48 P floor joists for predesign of residential floors. 2,0 kN/m² live load, 0,3 kN/m² partition load and 0,6 kN/m² self-weight. The **basic** option has c/c400 mm joist spacing, 22 mm chipboard decking without gluing and no transverse bracings. The **advanced** option has transverse bracing, glued deck panel and 45x45 c/c 400mm cross batten underneath the joists. Lowest natural frequency $f_1 > 8$ Hz and maximum deflection under 1kN point load is 0,5 - 0,8 mm depending on the span length (FI NA requirement for EN1995-1-1).

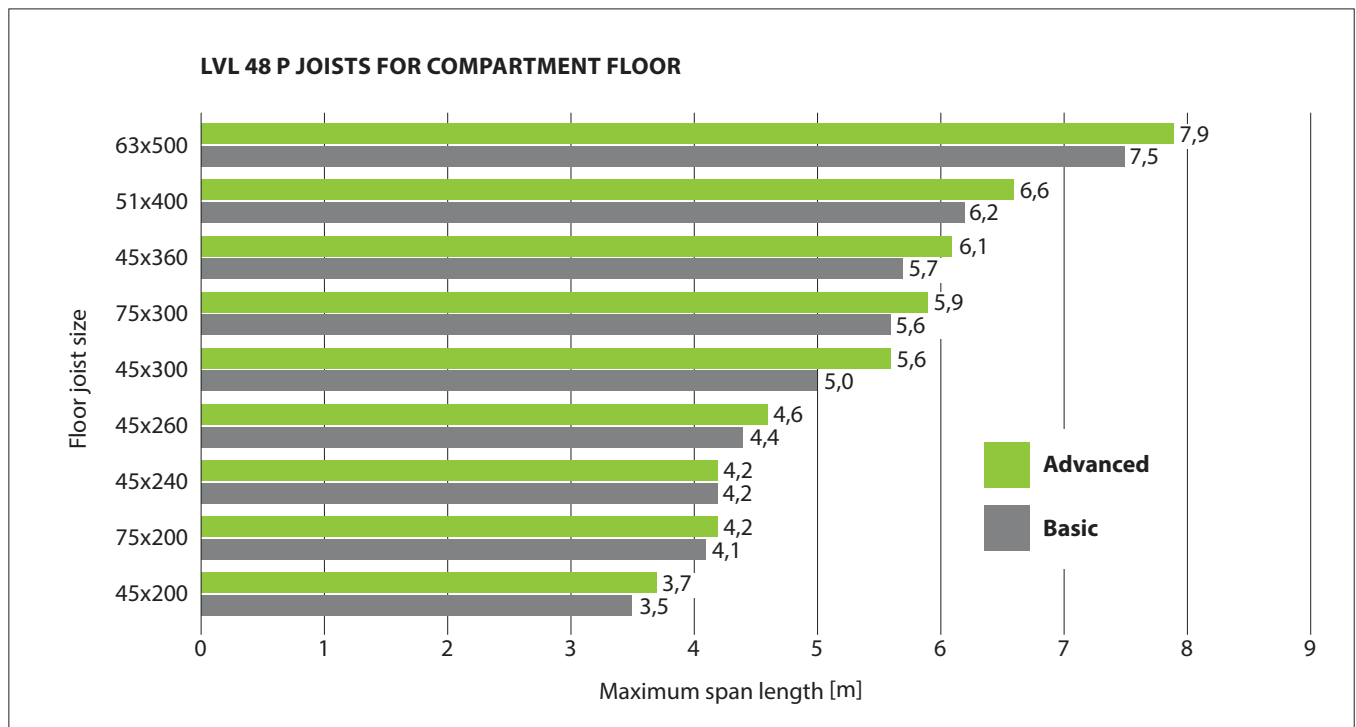


Figure 2.8. Maximum span lengths of LVL 48 P floor joists for predesign of compartment floors. 2,0 kN/m² live load, 0,3 kN/m² partition load and 1,6 kN/m² self-weight including a 50 mm screed in addition to the LVL members. The **basic** option has c/c400 mm joist spacing, 22 mm chipboard decking without gluing and no transverse bracings. In **advanced** option has transverse bracing, glued decking and 45x45 mm c/c 400 mm cross batten underneath the joists. Lowest natural frequency $f_1 > 8$ Hz and maximum deflection under 1kN point load is 0,5 - 0,8 mm depending on the span length (FI NA requirement for EN1995-1-1).

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

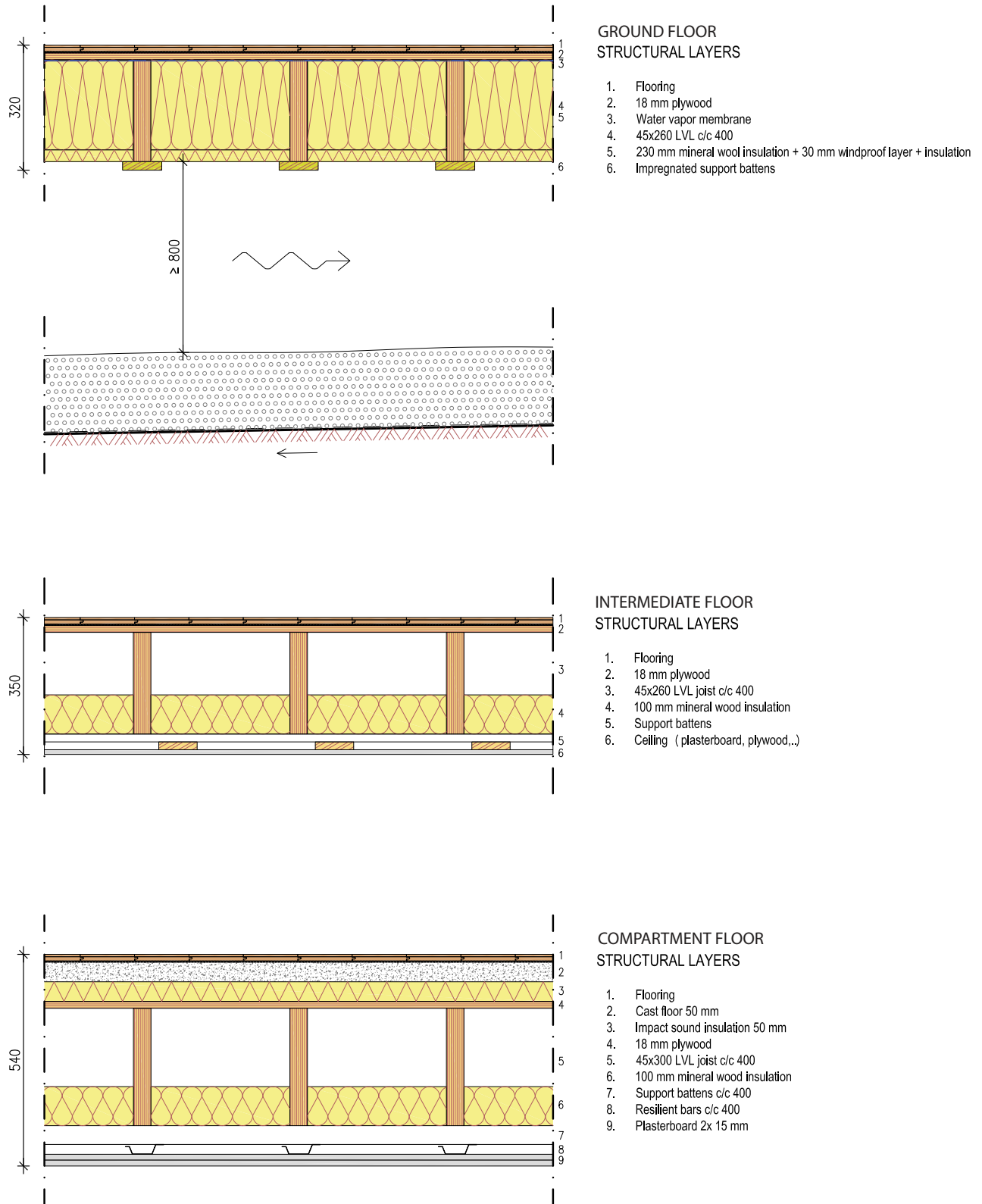


Figure 2.9. LVL joist floor structures for ground floors, intermediate floors and compartment floors.

2.2.2 Main beams for floors

LVL-P members are used as supporting beams for multiple span joist floors, headers over openings and trimmers around e.g. staircase openings. The supporting beams can be under the joists or the joists are connected to the sides of the supporting beams with e.g., joist hangers or diagonal screwing.

To save structural height while retaining adequate load-bearing capacity, multiple-member LVL beams are used.

Nailing, screwing or bolting patterns are required to ensure stability, especially when the beams are side loaded. At minimum, the plies are nailed together in 2 rows at c/c 300 mm. Loading from only one side is allowed for beams with a maximum of 3 members. Top-loaded beams or beams equally loaded from both faces may have at maximum 4 members. LVL suppliers provide connector-specific capacity tables for side-loaded beams.

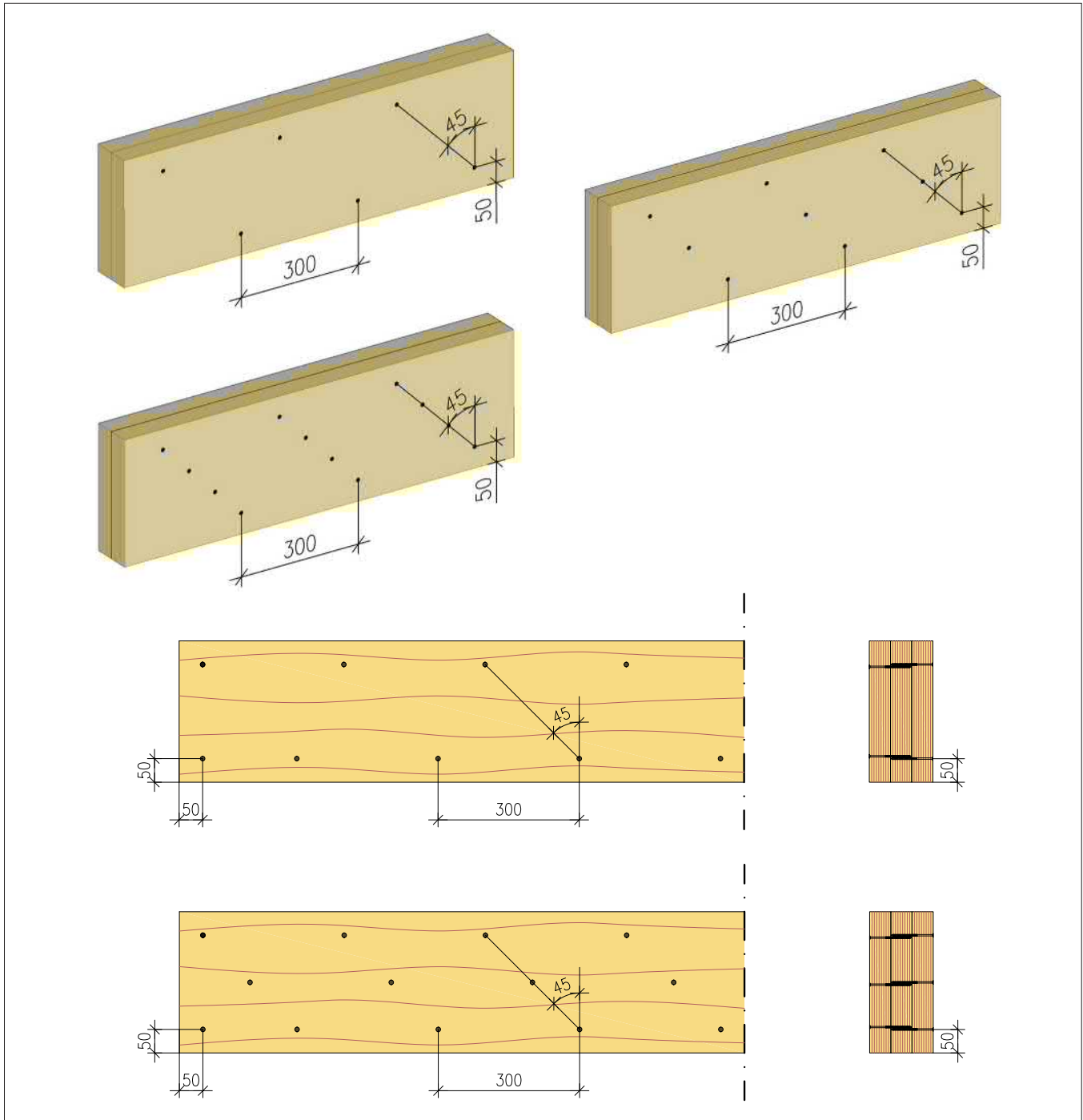


Figure 2.10. Instructions for nailing and screwing patterns for multiple-member LVL beams.

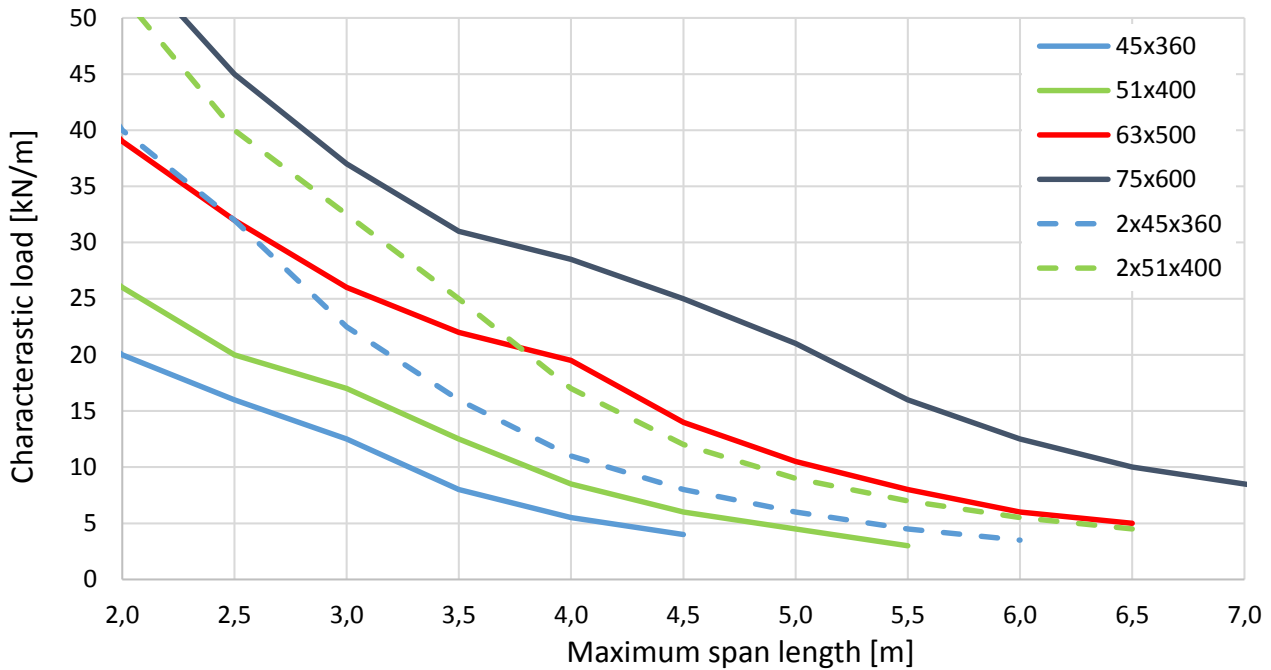
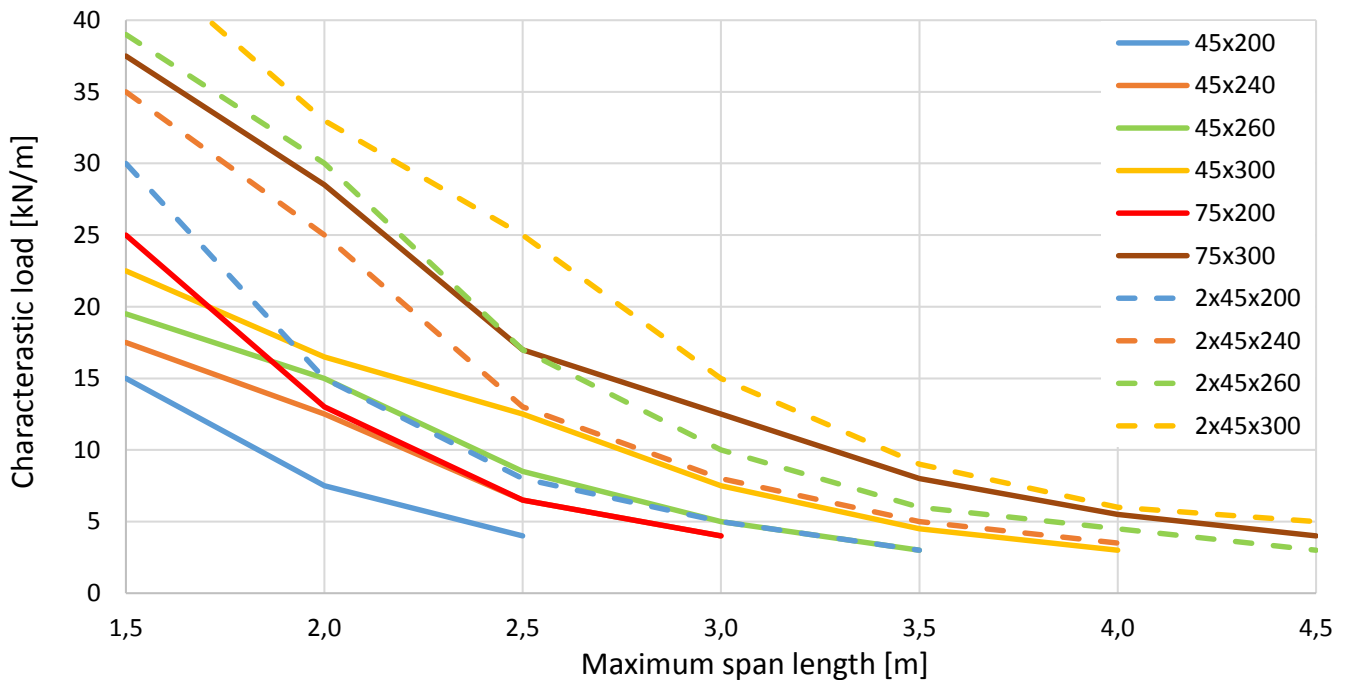
LVL 48 P main beams $h = 360 - 600\text{mm}$ for floorsLVL 48 P main beams $h = 200 - 300\text{mm}$ for floors

Figure 2.11. Span and capacity table of LVL 48 P main beams for predesign of floor structures. Calculations according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex. The permanent load is 20% of the characteristic load kN/m^2 . The service class is 1 or 2 and the consequences class is CC2. The rafter has lateral torsional buckling supports on the top surface with spacing $\leq 600\text{ mm}$ and the loads are located at the lateral torsional buckling supports. The support length shall be calculated separately. The initial deflection $w_{\text{inst}} \leq L/400$ and net final deflection $w_{\text{net,fin}} \leq L/300$. $\gamma_M = 1.2$. The table does not replace project-specific structural design. Double beams are calculated as separate beams in lateral torsional buckling.

2.2.3 Rim boards and rim beams

Rim boards and rim beams are used in the walls around the timber frame floors and, depending on the structural system, they may have several different load-bearing functions. Rim boards work as edge binders preventing the joists from tilting, and as load transfer members for the timber floor assemblies by carrying the line loads from the walls above the floor level to the supporting walls below the floor level. Continuous rim beams are required for floor panel diaphragms to absorb the tensile forces from the floor planes.

LVL-C products are supplied dry in long lengths and they have good resistance, stiffness and dimensional stability in the floor joist height direction, which makes them an ideal product for rim boards. LVL-C settlements are minimal, which

prevents the risk of compression cracks in plasterboard walls or facade rendering and enhances building envelope airtightness.

In partition walls between apartments rim boards must be designed for adequate resistance to fire to prevent fire spread in wall or floor cavities. A 30 mm rim board can serve as a fire separating component to meet the integrity E and insulation I performance criteria for 30 minutes for the structure.

Rim beams may also be designed in more robust dimensions to act at the same time as lintels to carry loads, e.g., above door or window openings. In such cases, bending strength and stiffness are important. In some structural systems rim boards work as secondary load transfer routes to prevent disproportionate collapse if, e.g., a part of the wall structure is destroyed.

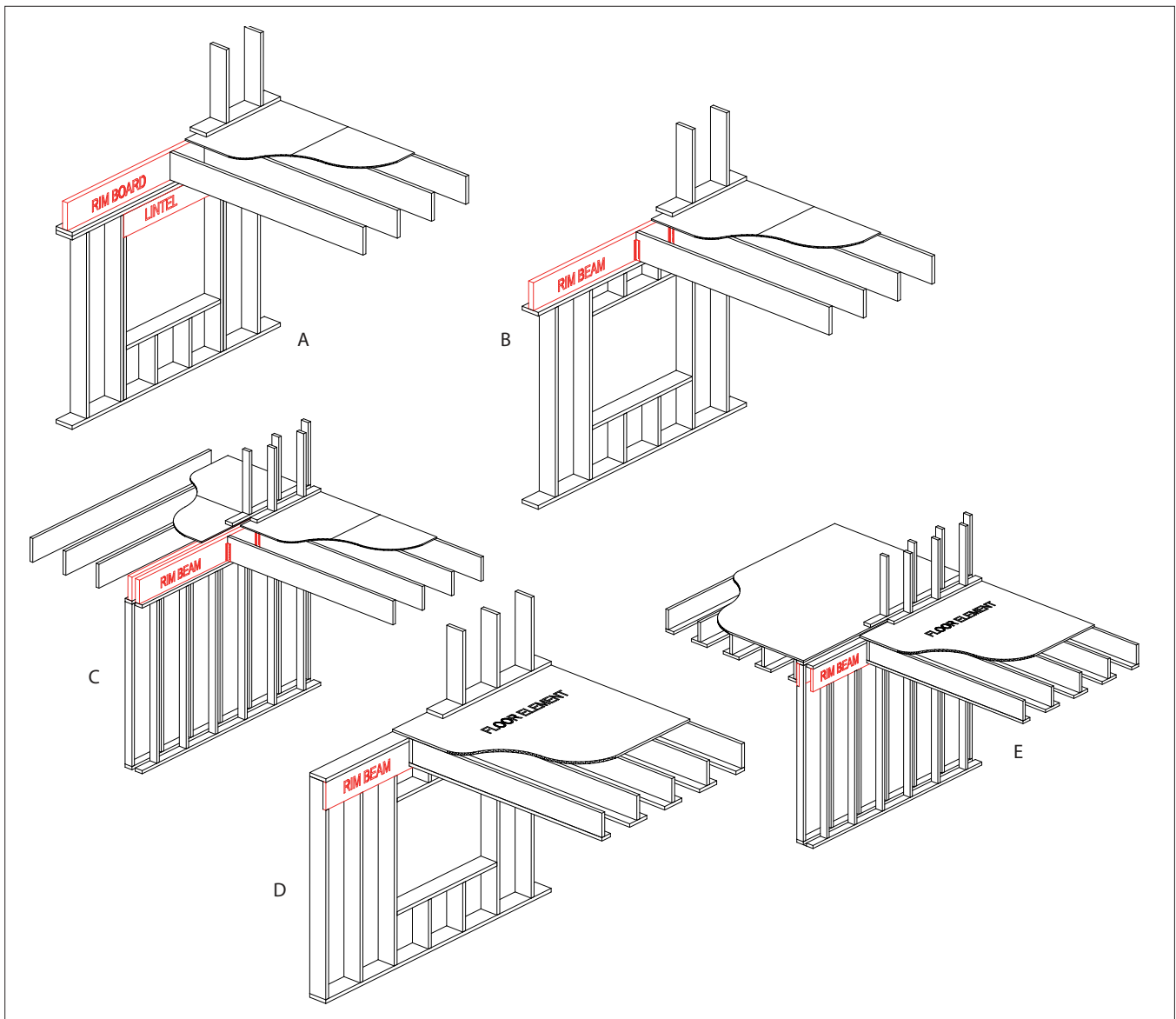


Figure 2.12. Rim boards and rim beams in different structural solutions of timer frame structures A) Rim board as egde binder and transferring loads from upper walls; B) Rim beams acting at the same time as a lintel over a window opening; C) Rim beam in compartment wall connection; D) Rim beam integrated in exterior wall element; E) Rim beam integrated to compartment wall element.

2.2.4 Panel structures

Robust LVL-C decking panels can be used on floor joists to utilize the diaphragm action of timber frame floors. The stiffness of LVL-C panels fulfils the deflection requirements of decking with large joist spacing. Large panels are fast and easy to install with a crane. They are also a straightforward structural design solution as fewer joints are needed between the panels and the substructure.

LVL-C panels can be used as a simple floor structure for up to 2.6 m single spans, which is an attractive option for, e.g., corridor floors or loft apartment mezzanine floors, where the space height limits the height of the floor structure. Multiple-glued GLVL-C panels can be used for over 4 m long spans, see Figure 2.17.

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Figure 2.13. Installation of a multiple-glued GLVL-C floor element.

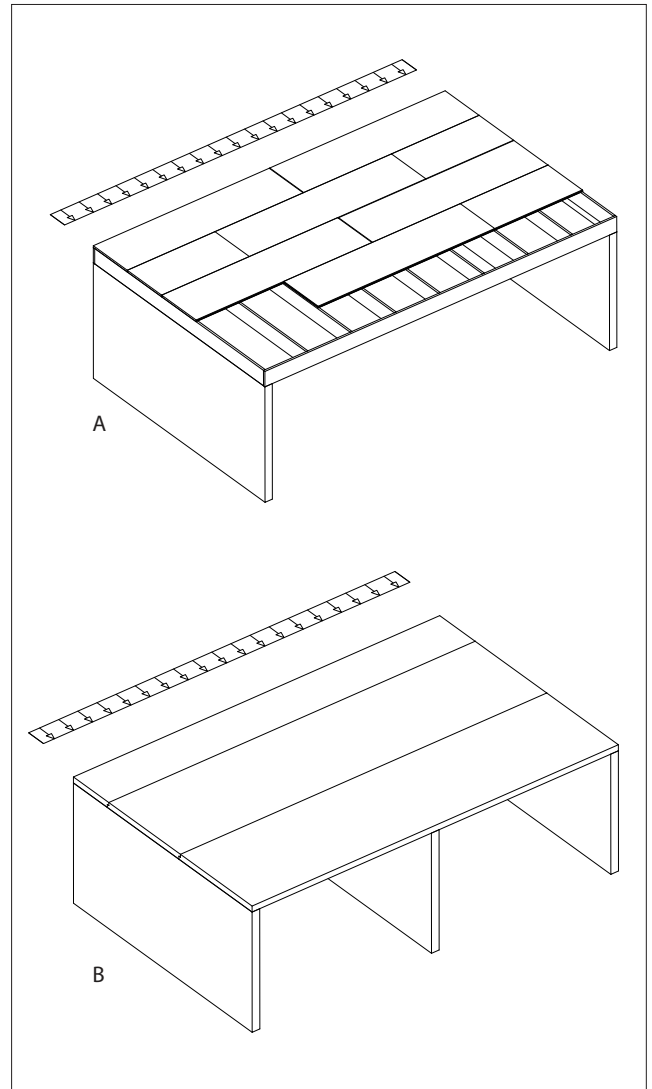


Figure 2.14. Robust LVL-C floor panels can be used for bracing and enable larger spacing between floor joists (A). Thicker LVL-C panels can be used as floor slabs (B).



Figure 2.15. Installation of LVL floor elements.

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2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

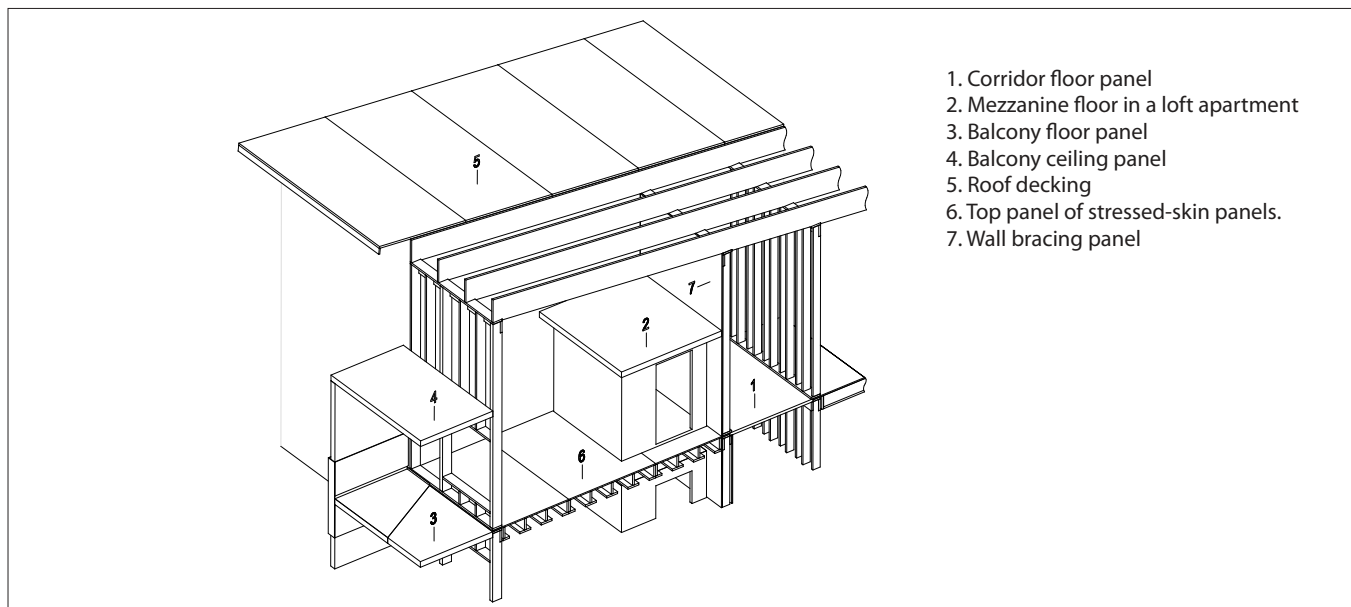


Figure 2.16. LVL-C panel applications in a multi-storey building.

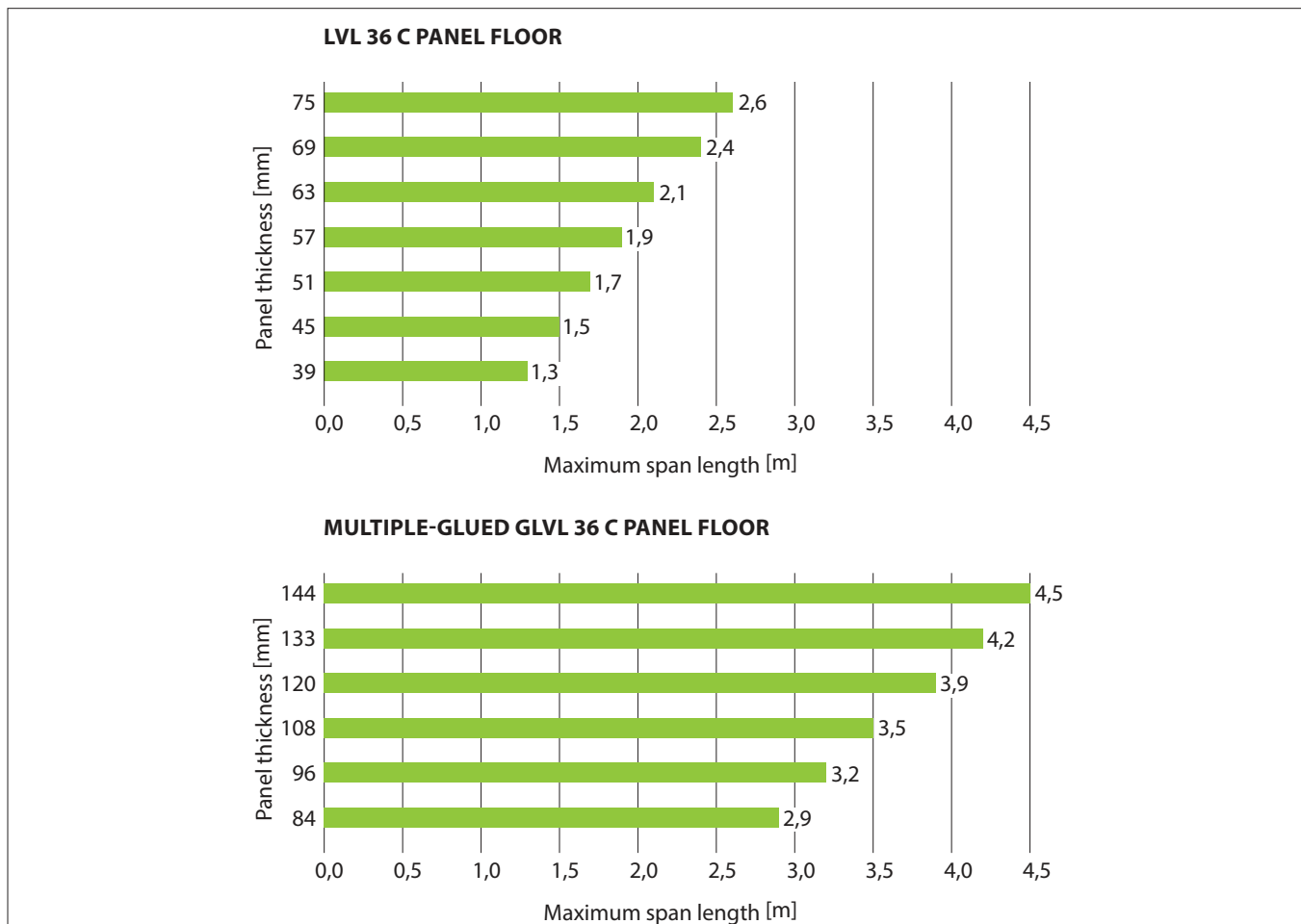


Figure 2.17. Span table of LVL 36 C and multiple-glued GLVL 36 C panel for predesign of floors supported from the ends. Calculations are according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex including floor vibrations. Permanent load is 0.4 kN/m^2 + panel's own weight, imposed load is 2.0 kN/m^2 (category A). Service class is 1 or 2 and consequences class CC2. The support length is $\geq 45 \text{ mm}$. Instantaneous deflection $w_{\text{inst}} \leq L/400$ and net final deflection $w_{\text{net,fin}} \leq L/300$. $\gamma_M = 1,2$. Double span structures may have $0,1 - 0,3 \text{ m}$ longer maximum span lengths. Lowest natural frequency $f_1 > 8 \text{ Hz}$ and maximum deflection under 1 kN point load is $0,5 - 0,8 \text{ mm}$ depending on the span length.

2.2.5 Multiple-glued beams, GLVL

For longer spans or higher loads on main beams, larger cross sections of multiple-glued GLVL can be used. Beam heights can be tailored for the projects case-specifically, but for optimized material efficiency, standard LVL beam heights or 5-10 mm smaller heights are recommended. The beam thicknesses are multiple thicknesses of sanded LVL laminae (e.g. $2 \times 42 \text{ mm} = 84 \text{ mm}$). Massive multiple-glued GLVL cross sections have the advantage that they are not sensitive to cracking in dry conditions. However, when they are used in visible applications, the sanding and face veneer specifications must be separately agreed with the supplier to achieve clean veneer surfaces.

- Recommended beam thicknesses: 84, 96, 90, 108, 120, 133 and 144 mm
- Recommended beam heights: 200, 225, 240, 260, 300, 360, 450 and 600 mm

2.2.6 Stressed-skin panels: rib panels, box panels or open box panels

Rib panels are efficient structures for long span floors or for cases where the floor height is limited. They utilize the composite action of the panels and the ribs achieved by mechanical connections or glued joints. Glue joints are recommended as they are much stiffer and more efficient but, as they constitute structural gluing, their manufacturing requirements are higher.

The simplest alternative is a T-cross section where a 25-37 mm thick LVL-C panel is structurally glued onto 200-400 mm

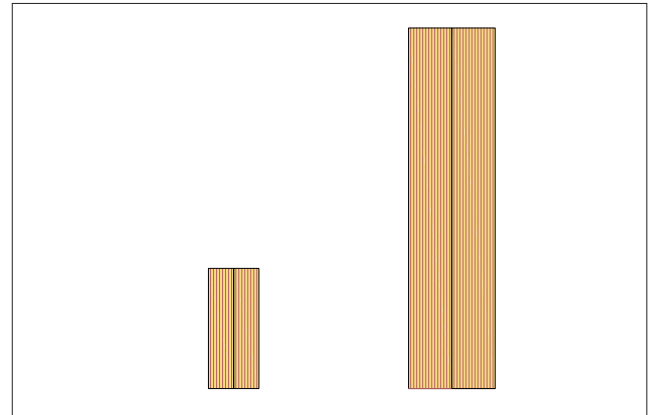


Figure 2.18. Multiple-glued GLVL beams.

high LVL-P ribs. Compared to typical LVL joist floor dimensions this can save ~100 mm in floor height or enable ~1 metre longer spans. Other alternatives include a box slab where another LVL-C panel is glued to the bottom of the element, or an open box element where LVL-P flanges are glued in the flatwise direction under the ribs. With typical dimensions these options can save another 100 mm in floor height or add an extra metre to the maximum span length compared to T-sections, see Figure 2.23. However, due to the high stiffness of box slabs the bottom panel emits sounds easier through the structure, making it difficult to fulfil the acoustic requirements of compartment floors between apartments. Therefore, an open box structure is recommended for compartment floors.

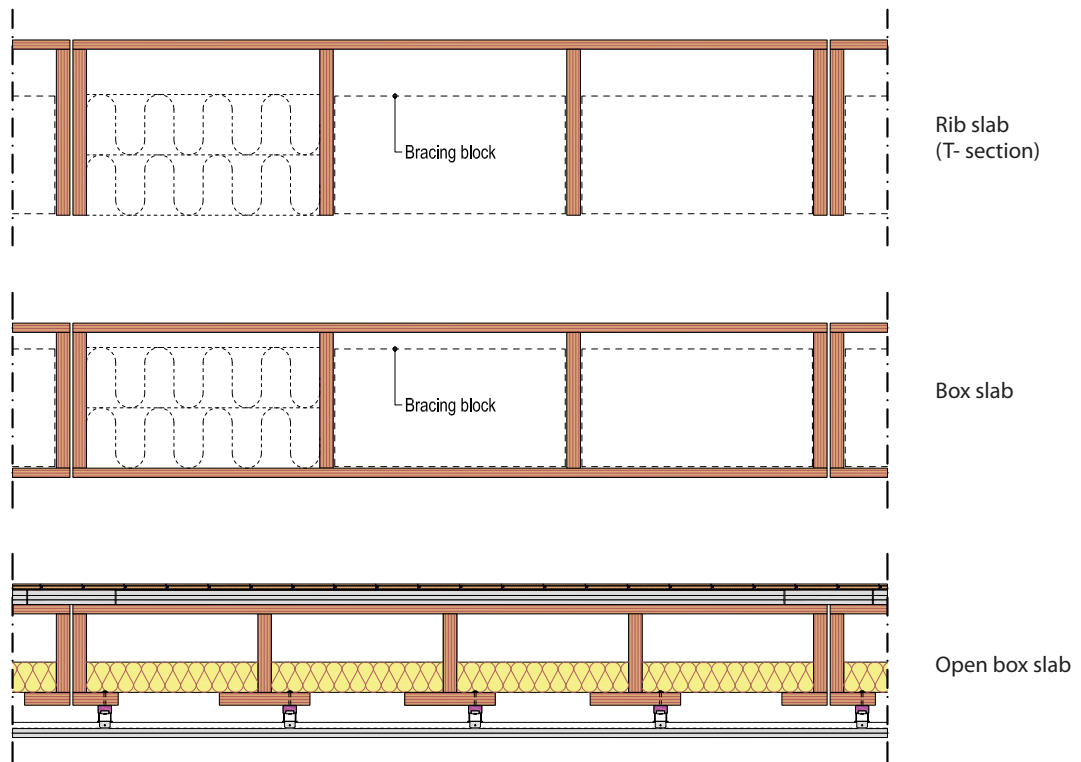


Figure 2.19. LVL rib slab, box slab and open box slab for floor constructions.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

Metsä Wood



Figure 2.20. LVL open box slabs of intermediate floor constructions in multi-storey buildings.



Figure 2.21. Installation of an LVL open box slab element.

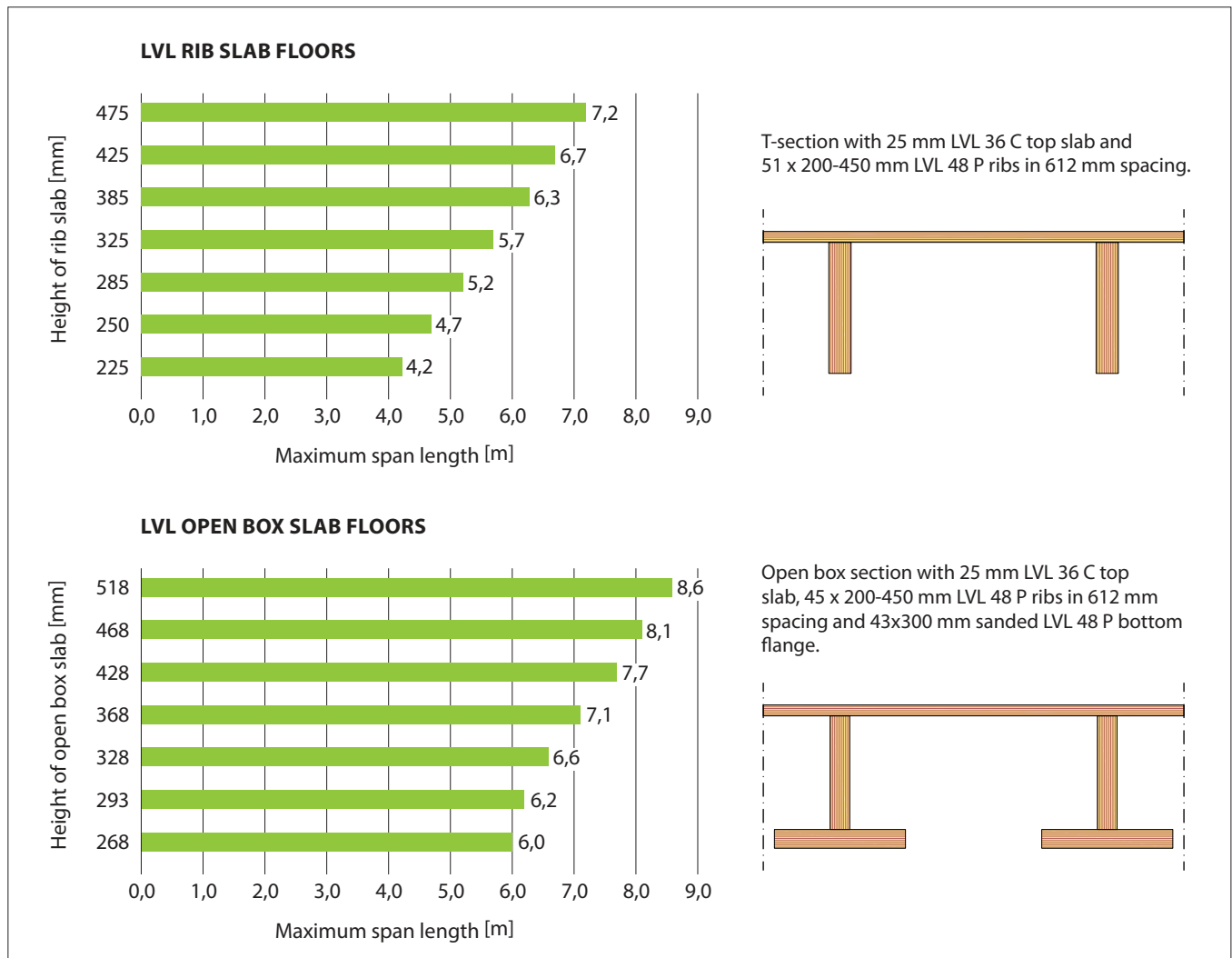


Figure 2.22. Span table of LVL rib slab and open box slabs for predesign of compartment floors. Loads: 2,0 kN/m² imposed load, 0,3 kN/m² partition load and 1,6 kN/m² self-weight including 50mm screed in addition to the LVL members. Transverse bracings are according to Figure 2.5. Lowest natural frequency $f_1 > 8$ Hz and maximum deflection under 1kN point load is 0,5 - 0,8 mm depending on the span length (Finnish National Annex requirements for EN1995-1-1).

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

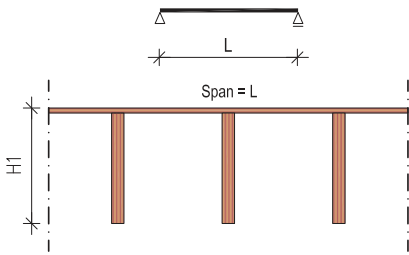
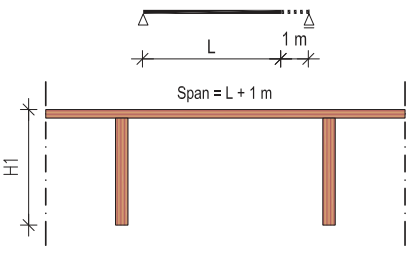
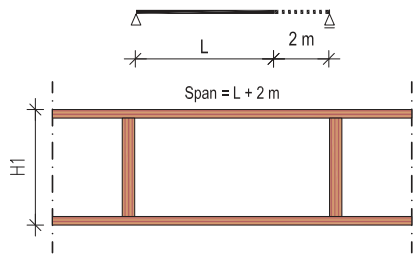
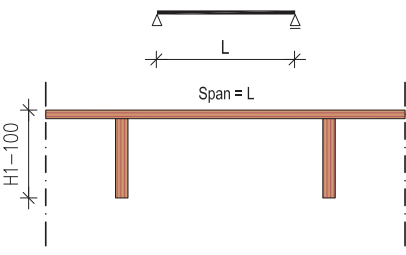
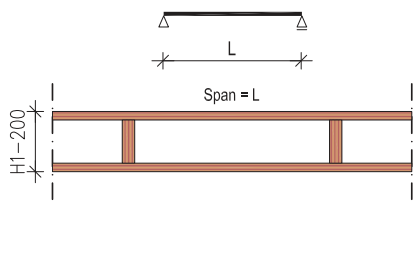
ADVANTAGES COMPARED TO JOIST FLOOR:		
Joist floor	Rib slab floor	Open box or box slab floor
 <p>Span = L</p>	1 metre longer span with the same floor height  <p>Span = L + 1 m</p>	2 metre longer span with the same floor height  <p>Span = L + 2 m</p>
	100 mm smaller height of the structure with the same span length  <p>Span = L</p>	200 mm smaller height of the structure with the same span length  <p>Span = L</p>

Figure 2.23. Benefits of rib slab, open box and box slab structures compared to joist floors. Savings in floor height or longer spans.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

Stressed-skin panels can be supported from the bottom as with normal joist floors. Another alternative is top slab support where the LVL-C panels are longer than the ribs which are anchored to the top panel with screws, and only the cantilever parts of the top panels are on the supports. This detailing facilitates site installation of the floor elements and simplifies the geometry of the supporting wall elements. It must be noted,

however, that top slab supported elements may have slightly smaller maximum spans or may need thicker top panels and higher ribs to ensure adequate capacity of the support connection.

Stressed-skin panels can be CE marked based on the European Technical Assessment (ETA) of the element supplier^{22, 23}.

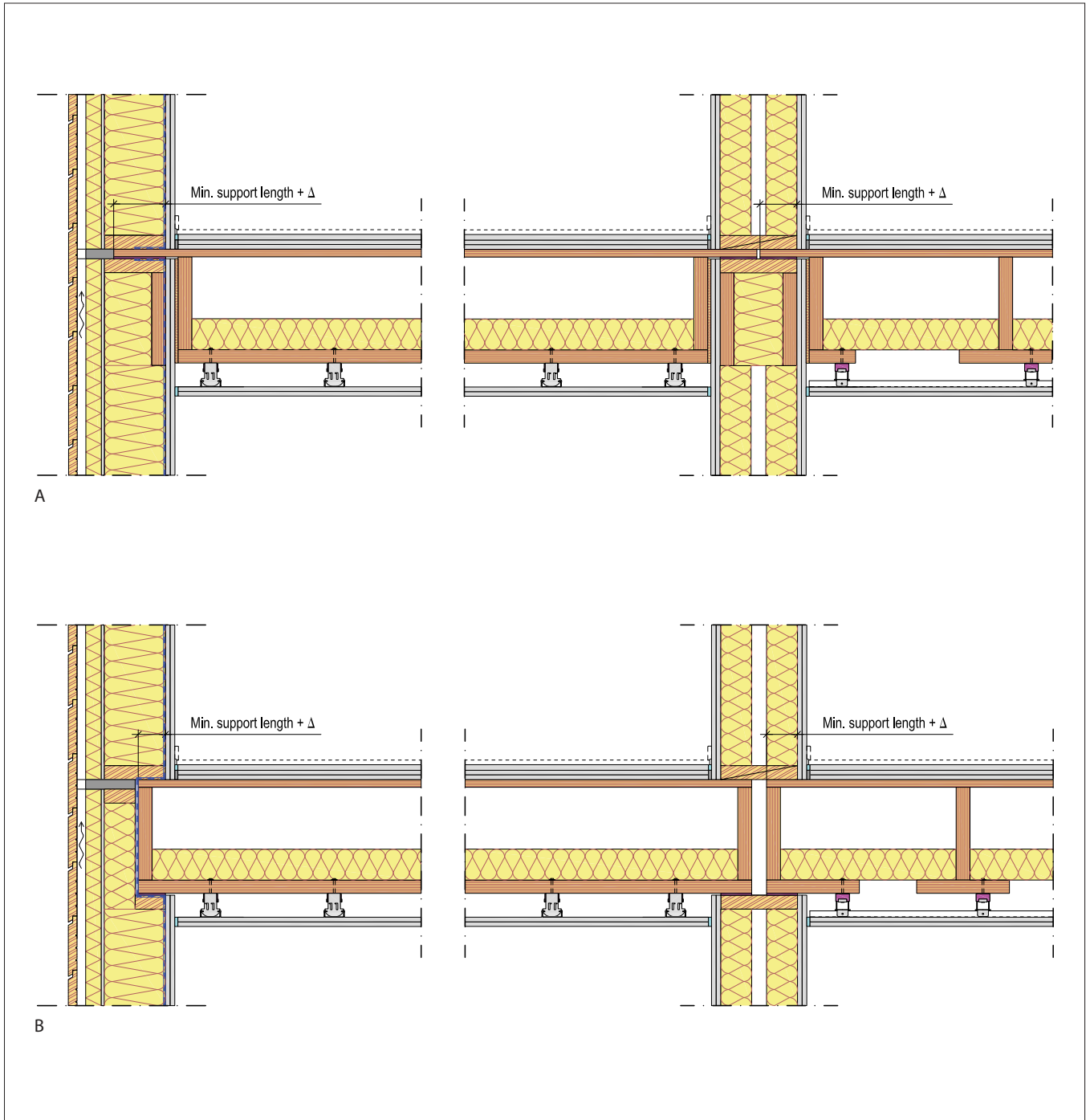


Figure 2.24. A) Top slab support detail of LVL rib slab or open box slab floor structure on exterior wall and intermediate wall. B) Rib slab or open box slab floor structure supported from the bottom on exterior wall and intermediate wall. Alternative A) has simpler geometry of the wall elements and less wood loaded perpendicular to grain.

2.2.7 Renovation applications

In renovation, old floors can be reinforced with LVL without increasing the loading of old supporting structures significantly. LVL-P joists can be fixed to the sides of old joists. This is practical because LVL beams are light to lift and fit into the cavities between the joists. Another alternative is to install LVL-C panels on top of old joists. This method provides a rigid structure and can be used to improve the overall bracing of a building. Structural gluing in on-site conditions is demanding, but gluing can be used to reduce deflections. Another connection method is inclined screwing, which gives good stiffness to the connections between old and new components.

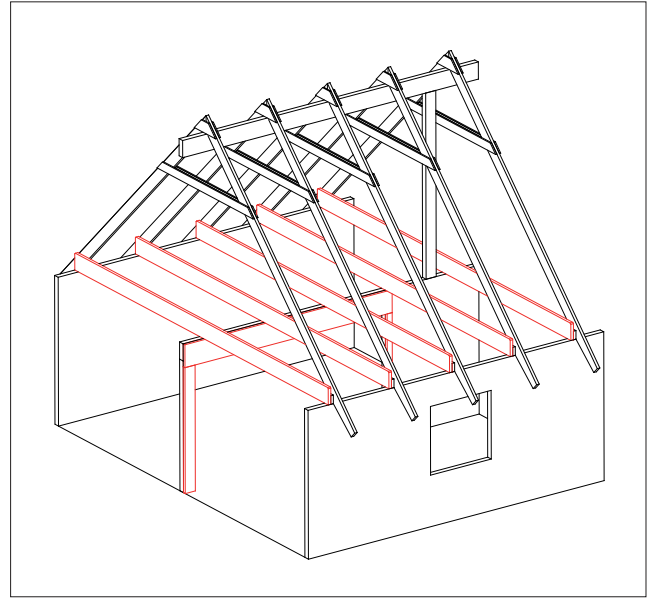


Figure 2.25. LVL-P joist fixed on one side or both sides of old timber beams as reinforcements.

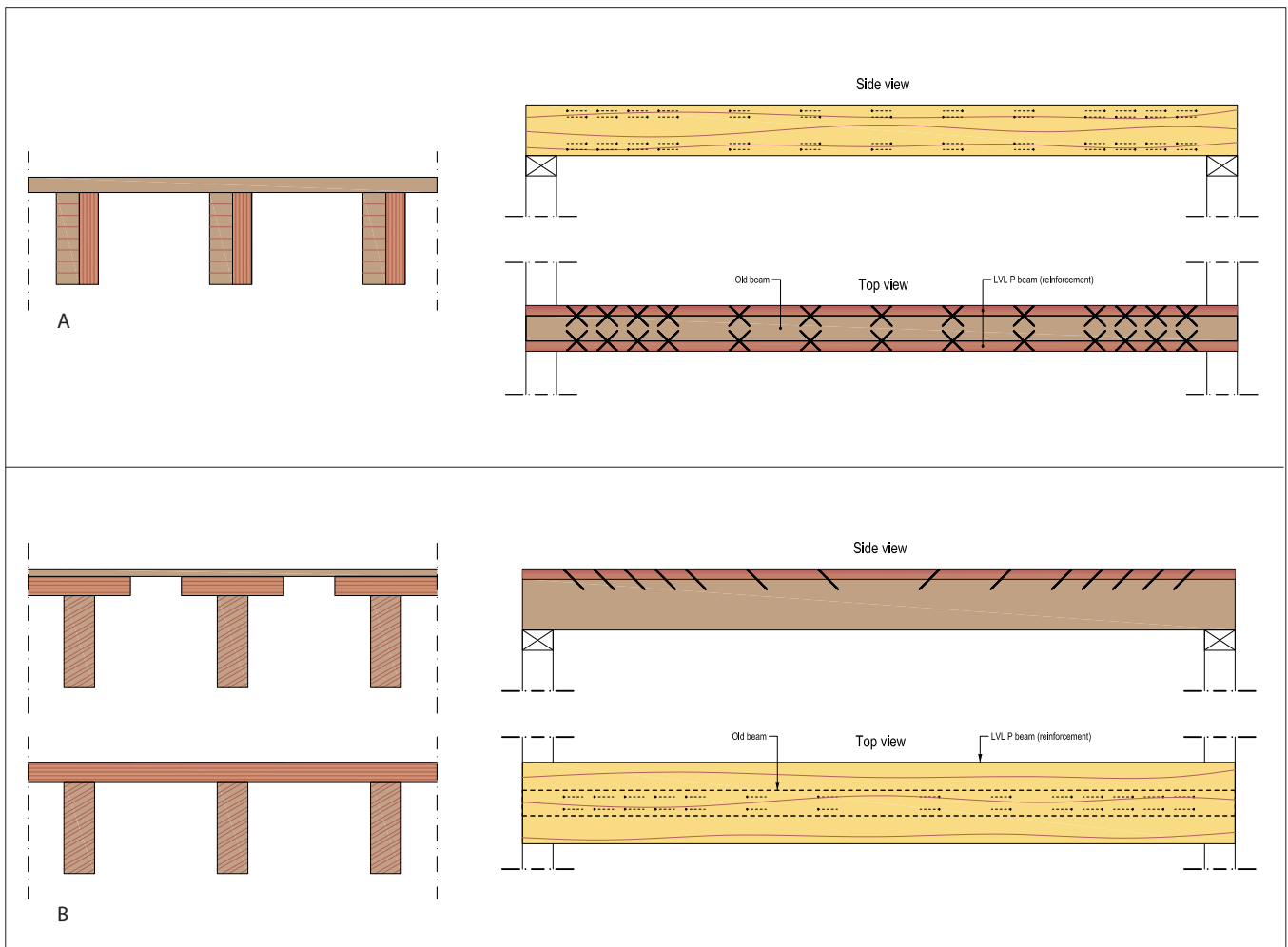


Figure 2.26. A) LVL-P reinforcement members connected with inclined screws to the sides of an existing joists, B) LVL-C panel reinforcement connected with inclined screws on the top of an existing joists.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

2.3 ROOF STRUCTURES

Tall and slim LVL-P beams are ideally suited for thermally efficient roof structures. LVL-P rafters provide long span lengths and simple detailing for pitched or ridge roofs in residential buildings. Multiple span LVL-P purlins in hall constructions can be installed on site or as components of roof elements produced offsite. Large and robust LVL-C roof panels are fast to install, they work effectively in diaphragms, and they make it possible to create roof overhang eaves that have a light appearance without supporting beams.

2.3.1 Beam & rafter roofs

LVL-P roof rafters are available in a wide range of sizes for different span lengths of pitched or ridge roofs. Energy efficiency requirements for roof structures set the minimum U-value at 0.09-0.15 W/mK depending on the country. With conventional mineral wool or wood fibre insulation this means that the insulation thickness needs to be ~250-450 mm. To provide adequate ventilation space between the insulation and roof covering in cold roofs, the rafters should be at least 100 mm taller or a separate batten structure is needed. In some countries a warm roof without a ventilation gap is the preferred solution depending on the climate and building physics design. LVL-P rafters provide an easy solution with minimum cold bridging. The rafters have good load-bearing capacity and stiffness, enabling wide rafter spacing, e.g. c/c 1200 mm.

Ends of the rafters can be notched at the roof overhang section. This can be done on site with normal woodworking tools, or for faster installation the rafters can be ordered from the supplier with ends ready machined.



Figure 2.28. LVL-P rafters in a ridge roof structure.



Figure 2.27. LVL-P rafters in a mono-pitch roof structure.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

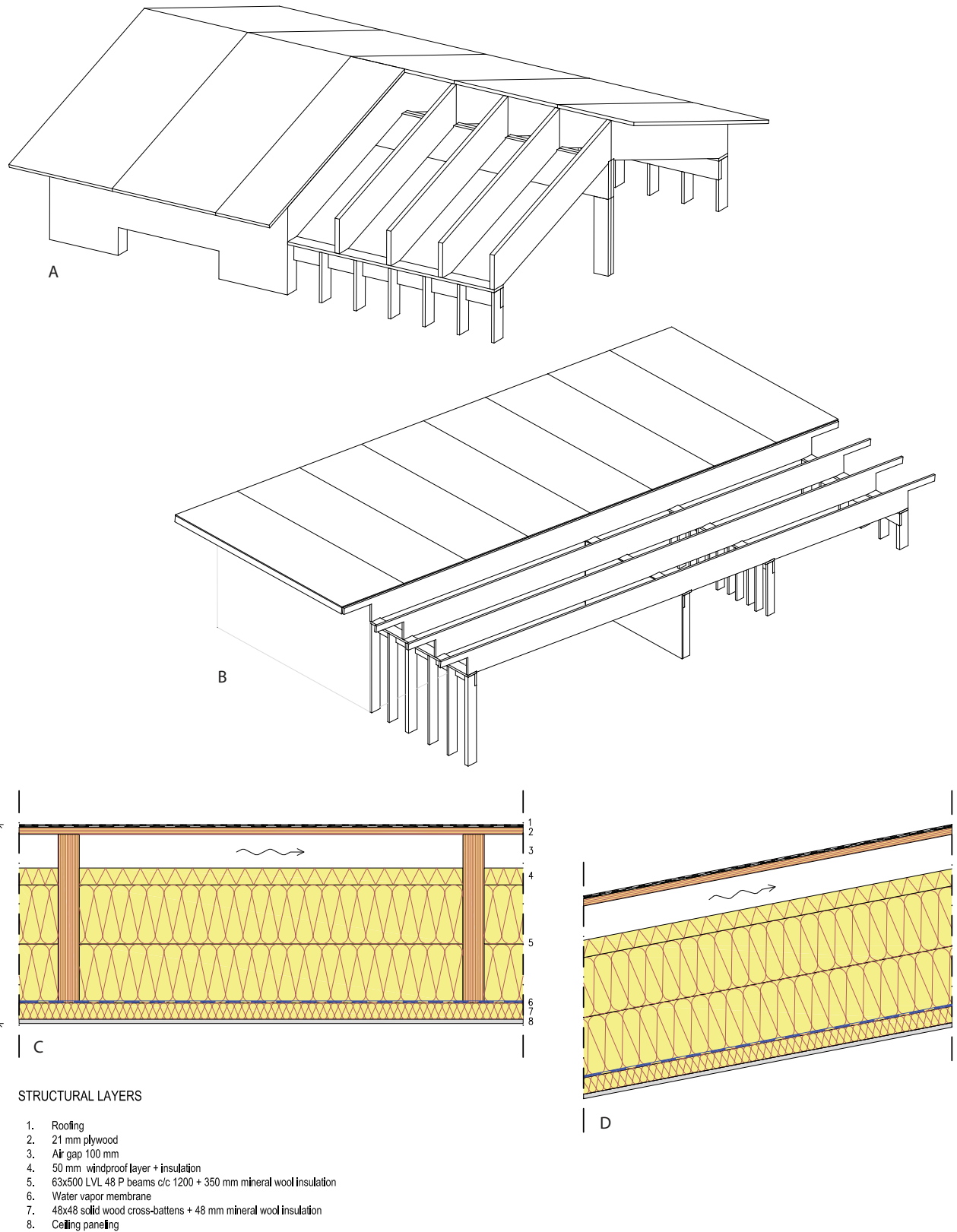
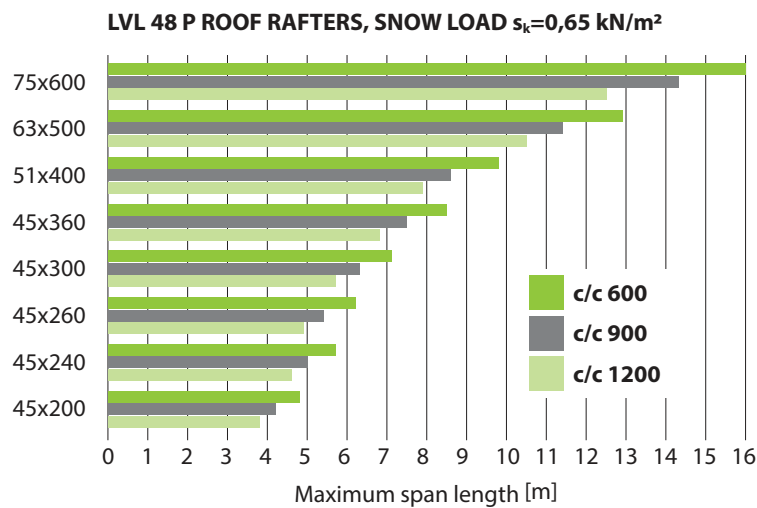


Figure 2.29. A) Ridge roof with LVL-P rafters, B) Pitched roof with multiple span LVL-P rafters, C) End section of LVL rafter structure, D) Side section of LVL rafter structure.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS



Calculations according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex. Roof slope is 1:3. Permanent load is 0.9 kN/m^2 , snow load on the ground s_k is 0.65 , 1.5 or 2.75 kN/m^2 (snow load at roof level = $0.8 \times s_k$) and wind load = 0.2 kN/m^2 . Service class 1 or 2 and consequences class CC2. Rafters have lateral torsional buckling supports on the top surface at $\leq 900 \text{ mm}$ spacing and the loads are located at the lateral torsional buckling supports. Rafters must be supported for the buckling at the midspan. The support length shall be calculated separately. Net final deflection $w_{\text{net,fin}} \leq L/200$. $\gamma_M = 1.2$. This table does not replace project-specific structural design.

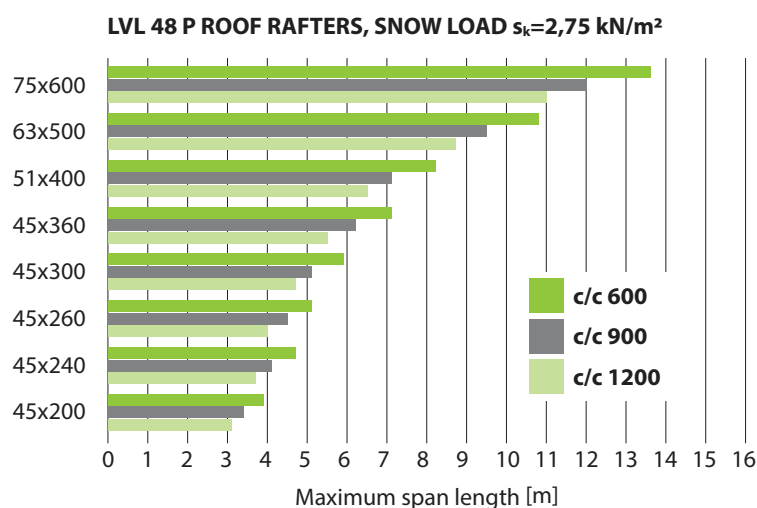
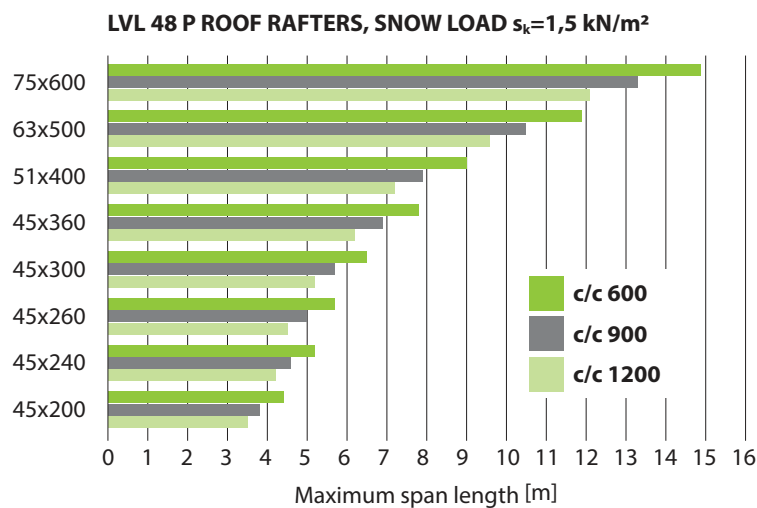


Figure 2.30. Span table of single-span roof rafters at c/c 600, 900 and 1200 mm spacing for predesign. The spans lengths are given as horizontal projections between the centres of supports, not the inclined length of a rafter.



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Figure 2.31. Notched rafter end at the front roof overhang.

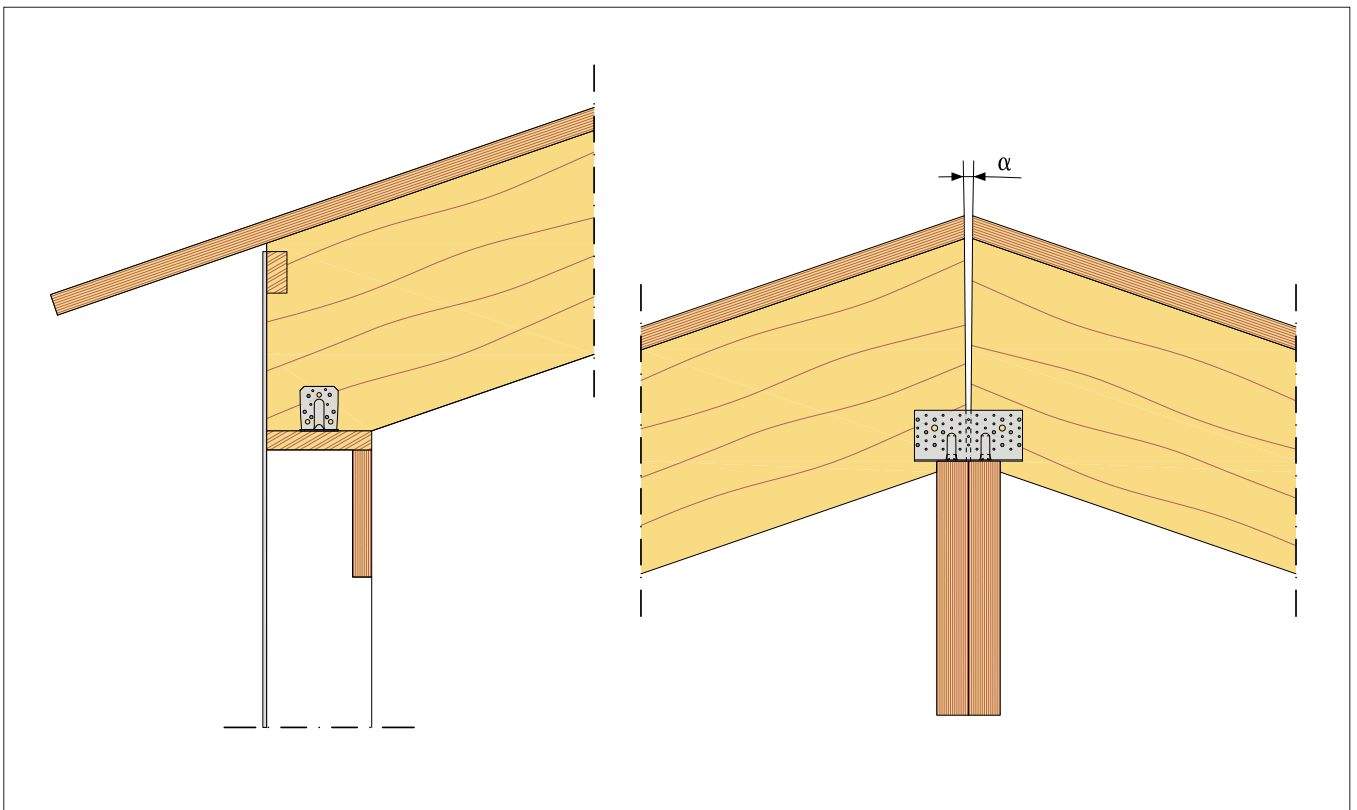


Figure 2.32. Connections of roof rafters at supports. Notch details to be verified in structural design.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

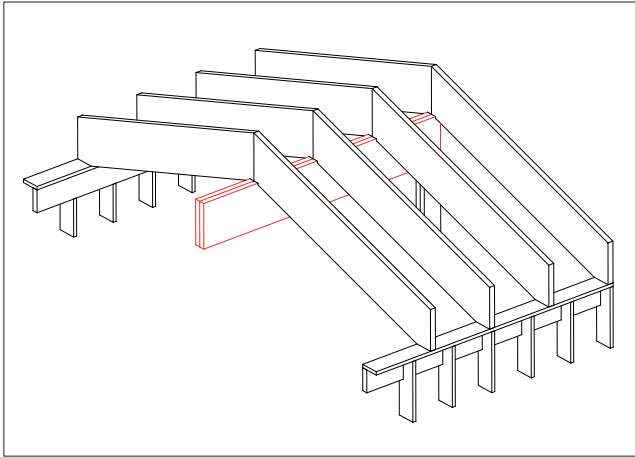


Figure 2.33. LVL-P ridge beam. Due to the height/thickness ratio recommendations, LVL ridge beams are usually multiple-member beams.

2.3.2 Ridge beams

LVL-P beams can be used as load-bearing main beams of ridge roofs. As LVL beams are slim, LVL ridge beams are normally multiple-member beams. Rafters can be supported on the ridge beam or they can be connected to the sides of the ridge beam with, e.g., joist hangers. Section 2.2.2 and Figure 2.10. have instructions for the nailing and screwing patterns of multiple-member LVL beams. In the case of side connections, nailing or screwing between the LVL plies must be verified case-specifically.

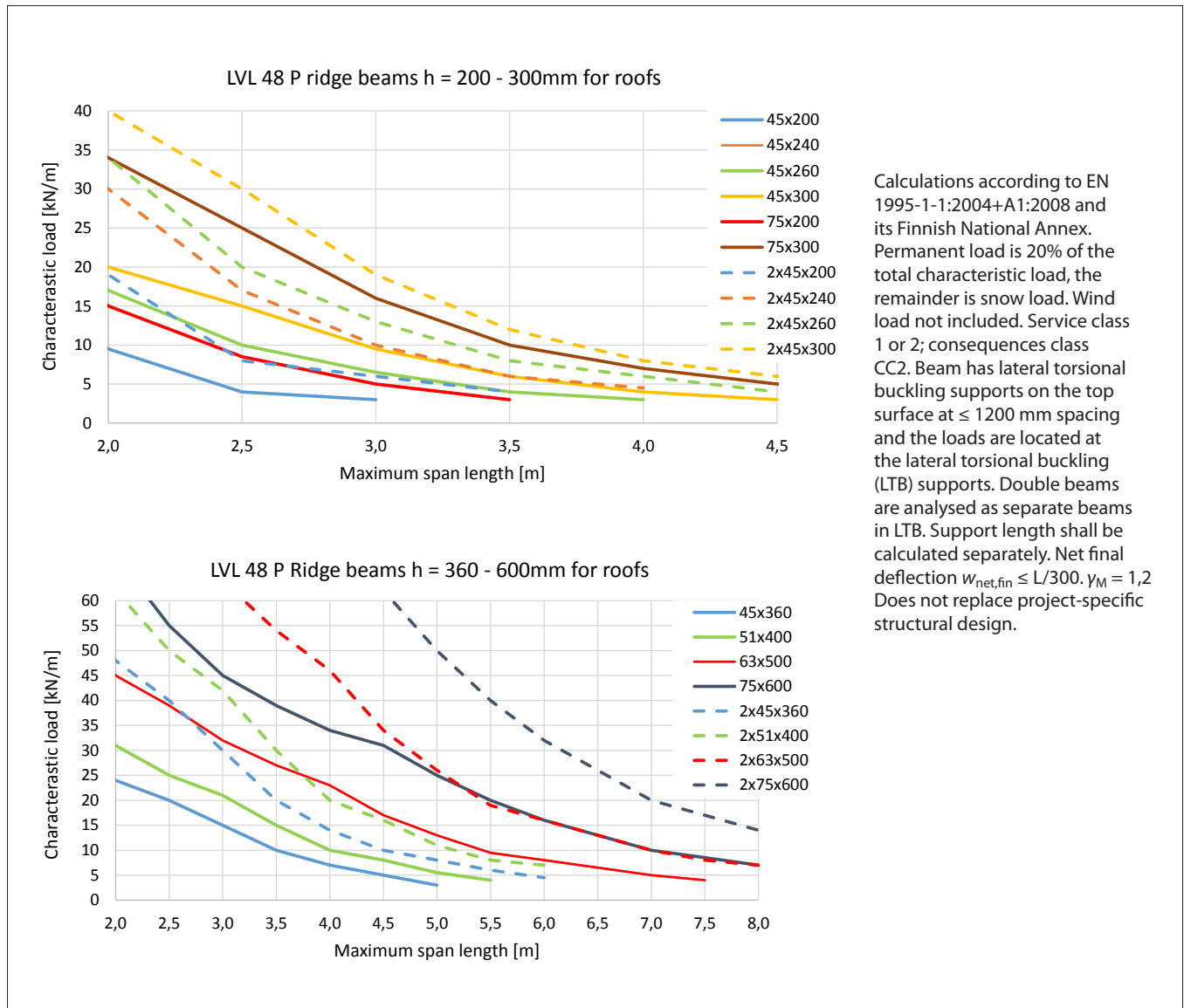


Figure 2.34. Capacity table [kN/m] of single-span LVL 48 P ridge beams for predesign.

2.3.3 Purlins

In hall constructions LVL-P purlins are suitable secondary structures for wooden or other main frames. Since LVL is available in long lengths, multiple span purlins are structurally efficient and fast to install. However, purlins often also act as supports against lateral torsional buckling of the main frames or beams setting capacity requirements for connection detailing. Therefore single-span purlins supported with hangers from the sides of rafter beams are recommended for 3-pin frames.

With the exception of flat roofs, purlins are biaxially loaded. As LVL purlins are thin, they need to be side supported in the weaker direction.

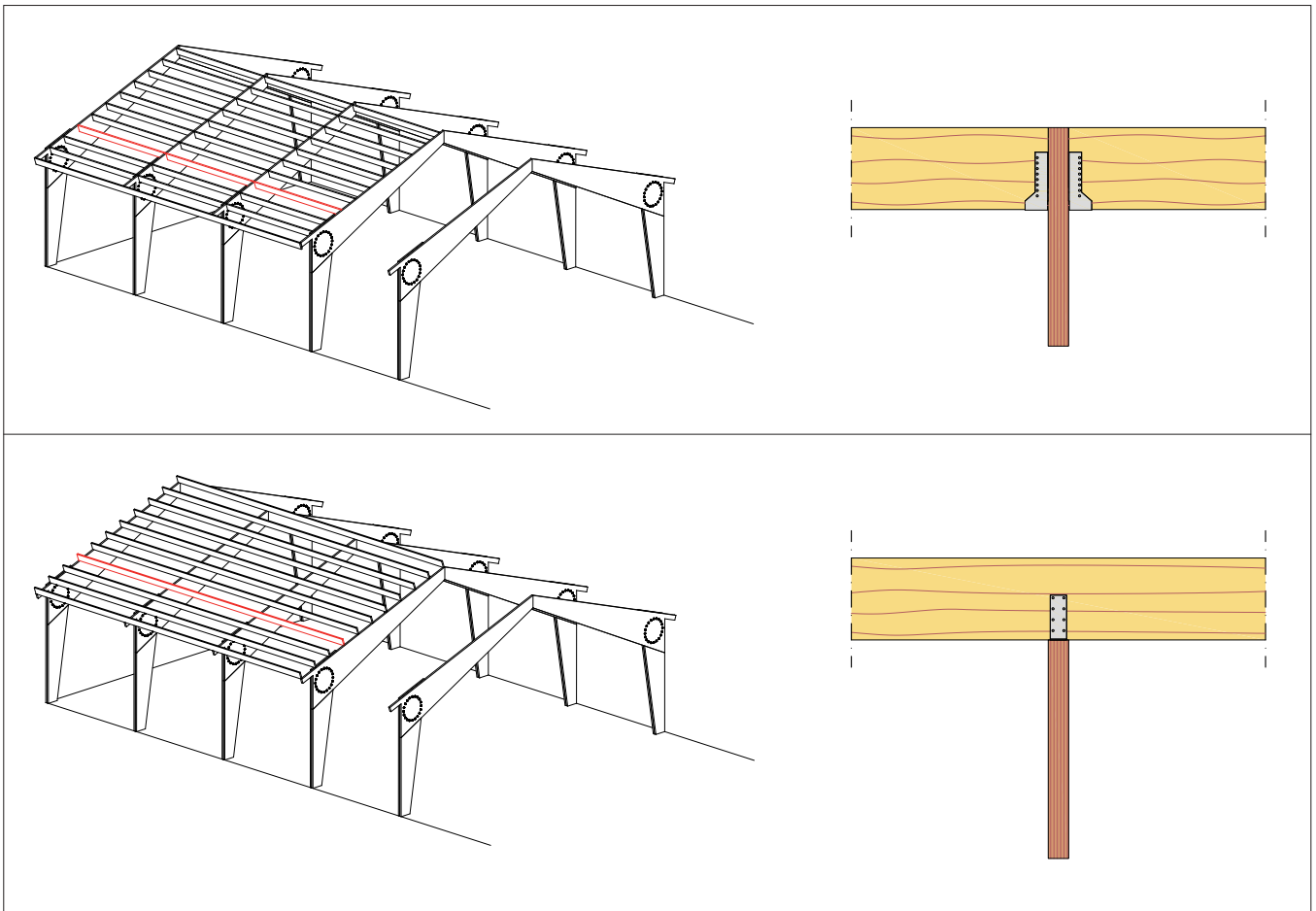


Figure 2.35. Single-span and multiple span purlins for hall construction.

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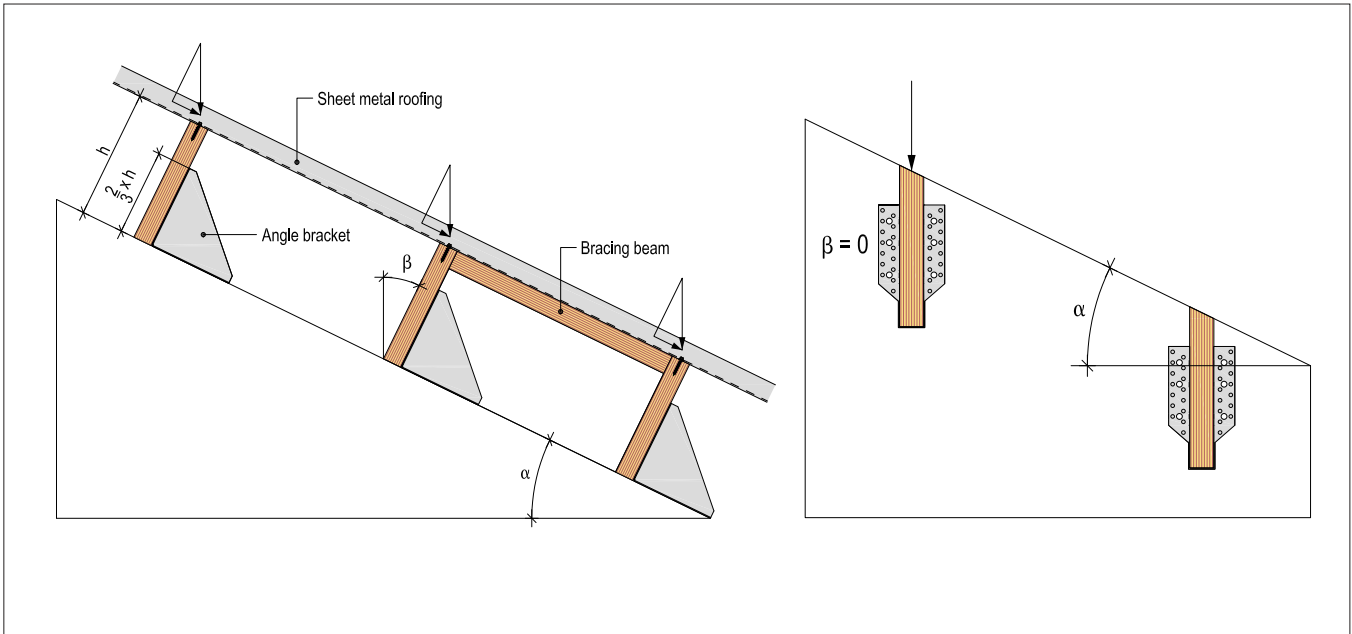


Figure 2.36. Support details of purlins installed vertically or perpendicular to the roof surface.

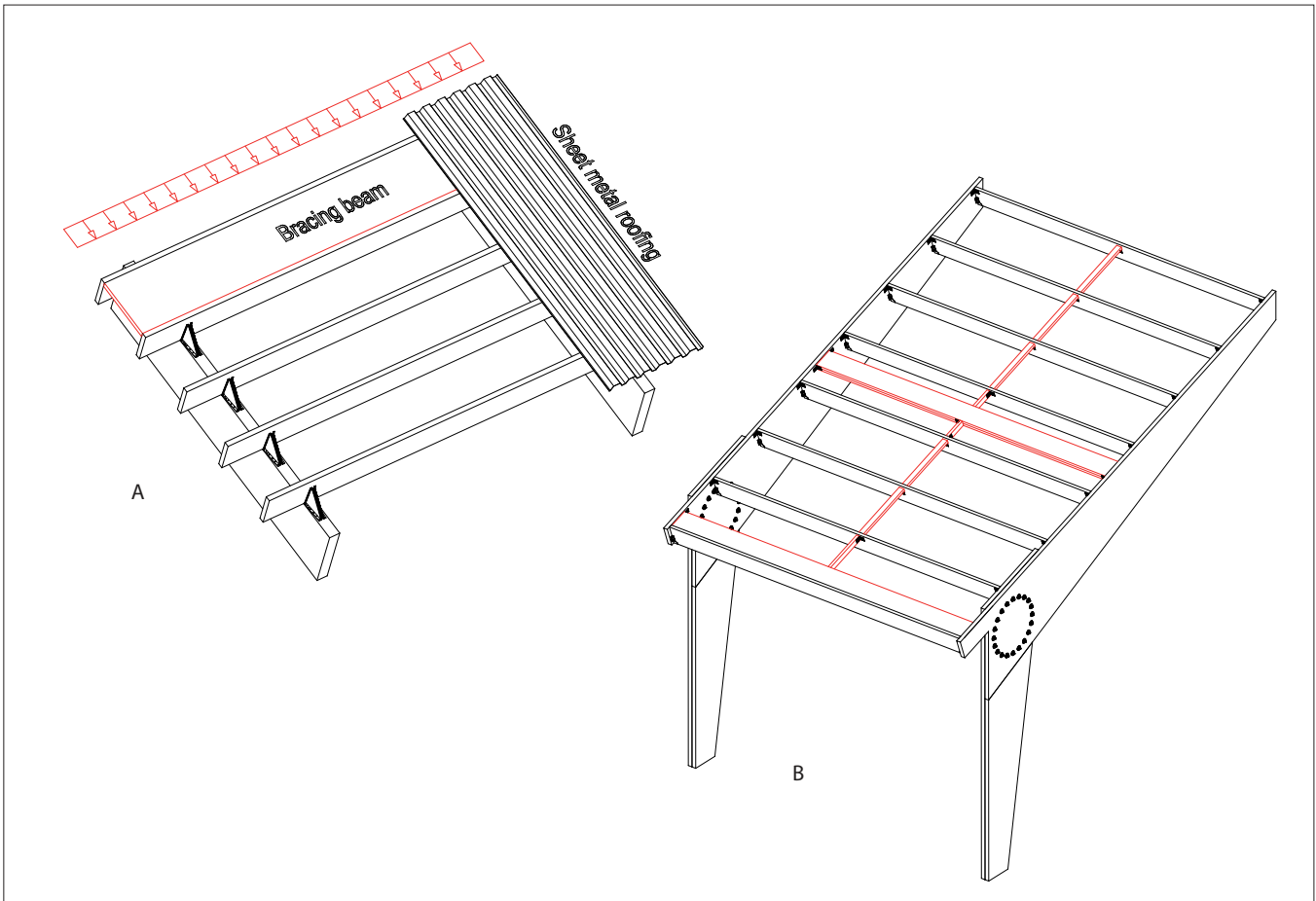


Figure 2.37. LVL purlins installed perpendicular to roof surface. Bracing beams are needed in purlin width directions. A) Steel sheeting transfers lateral torsional buckling support forces to the bracing beam B) Wooden batten side supports at the mid span transfer loads perpendicular to the span to bracing beams.

2.3.4 Roof elements

LVL purlins can be combined with off-site produced roof elements that are fast to install on site on all kinds of hall construction main frames. An installation speed of 1000 m² per day for a ready roof is commonly achieved. The maximum dimensions of the elements depend on transportation limits, but 2.5 m x 20-25 m sizes are commonly used as a 3- or 4-span structure. For finishing of interior and exterior surfaces, gypsum board ceiling panels, vapour barrier, thermal insulation,

decking panels and water proof membrane the elements are delivered as a single complete package. Secondary beams and battens may be made of LVL or solid wood depending on the choice of manufacturer. Roof overhangs may also be integrated with the edge elements. The roof elements are typically ventilated (cold roof) with a U-value of 0.07-0.15 W/mK and fire resistance class R(EI) 15-60 depending on the local requirements and customer specifications.

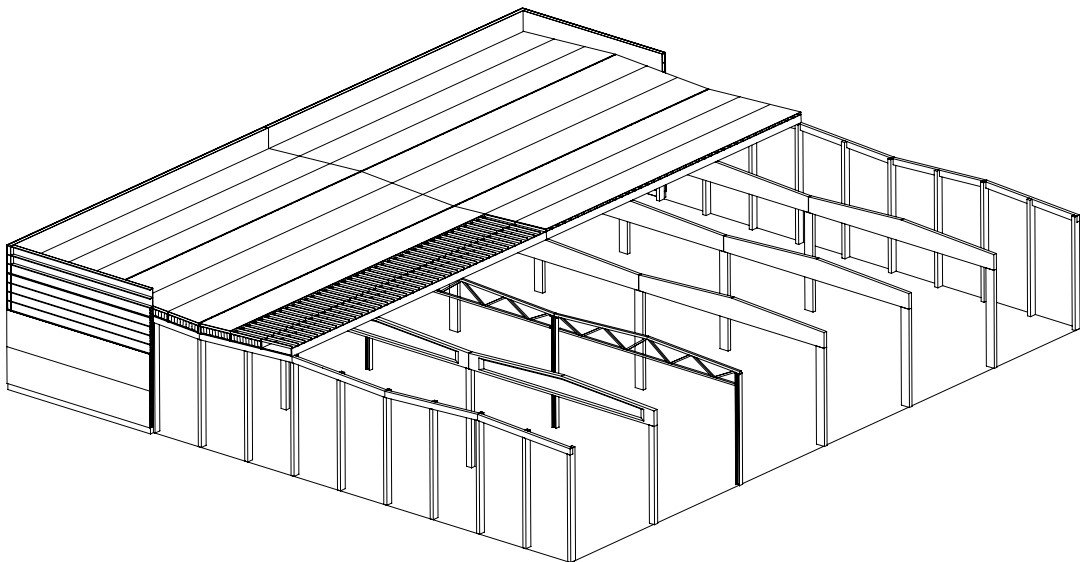


Figure 2.38. Roof elements with LVL purlins for hall construction can be used on any type of main frame.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

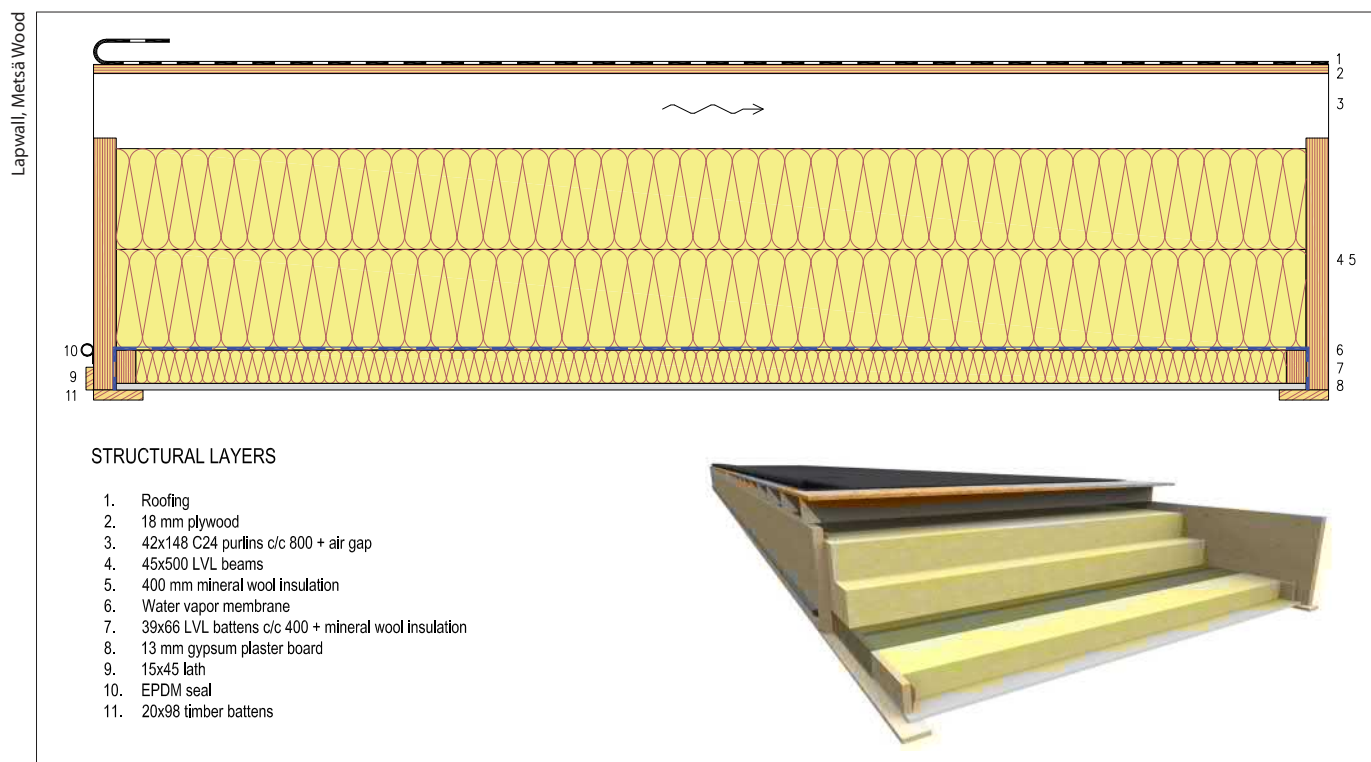


Figure 2.39. Cross section and detailing of roof elements with LVL purlins.



Figure 2.40. Installation of roof elements with LVL purlins. Metsä Fibre pulp terminal, Vuosaari, Helsinki, Finland; Karisma, Lahti, Finland; Skanssi, Turku, Finland.

Stressed skin rib panels and box panels

For long-span roof elements, the composite action of glued stressed-skin panels can be utilized between LVL-P ribs and LVL-C panels. Rib panels can be used, e.g., for non-heated shelters, and insulated box elements for hall constructions. Depending on the snow loads and height of the elements, span lengths can be 10-20 metres.

The structure is simpler than elements with purlins as no secondary beams or battens are needed. However, if the rib spacing is large, up to 1250 mm, the top panel must be thicker because the main direction of the LVL-C is along the ribs, and the panels need to transfer the snow load to the ribs in the secondary direction.

The ceiling panel thickness depends on the fire resistance requirements. Depending on the reaction to fire requirements of the project, LVL-C bottom flange panels can be left visible or covered with gypsum board. A great advantage of box elements is that suspension installations, such as ventilation channels, can be freely fixed with screws anywhere on the ceiling surface because the LVL-C panels provide a solid base for axially loaded connections.

An LVL box element roof can be realized as a cold roof or warm roof solution, but the building physics design needs to be done separately based on the project-specific indoor and outdoor conditions. Stressed-skin panels can be CE marked based on the ETA assessments of the element suppliers ^{22, 23}.

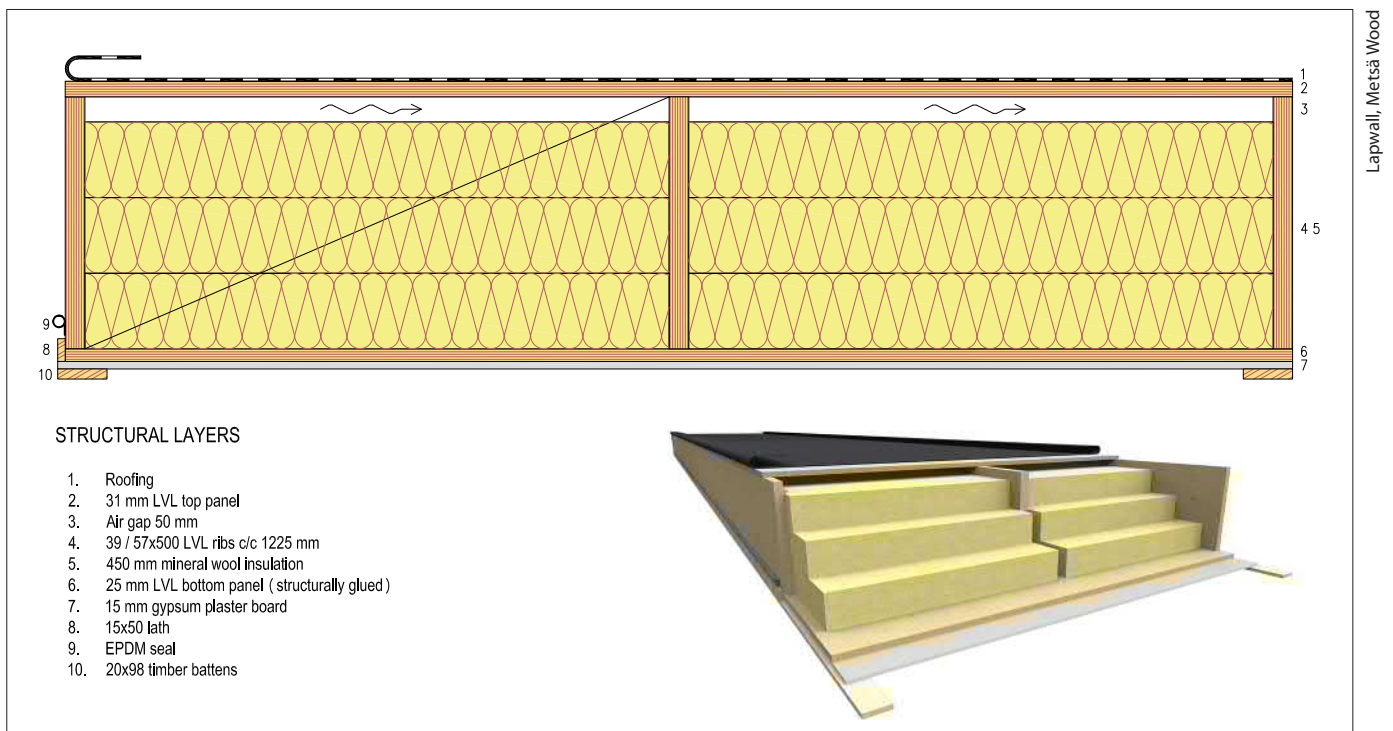


Figure 2.41. LVL box panel element.

2.3.5 Roof panels

LVL-C panels are suitable for load-bearing panel structures in roofs. Thicker panels allow longer spans between supports, and LVL-C forms a durable base for waterproof roofing. As most of the veneers are parallel to the surface veneer direction, the capacity of the panels is better in the main direction, enabling longer maximum spans. Sometimes, however, it is more practical to install the panels so that they are supported from their long edges. The charts in Figures 2.42. and 2.44. provide information on suitable panel thicknesses for different spans in preliminary design, and sections 1.13 and 5.7 give information on joints and connections. It is recommended to treat LVL-C

panels against mould growth when used in unheated spaces.

From large, robust LVL-C it is possible to build roof overhangs that have a very light appearance because no other load-bearing support structure is needed even in corner overhangs. Information on suitable panel thicknesses and spans when the panel has an overhang in only one direction (parallel or perpendicular to grain of the surface veneer) is given in Figure 2.50. If the panel is used as an overhang in both the parallel and perpendicular direction to also provide a corner overhang, the maximum span length L_C must be in accordance with Figure 2.51. in both directions.

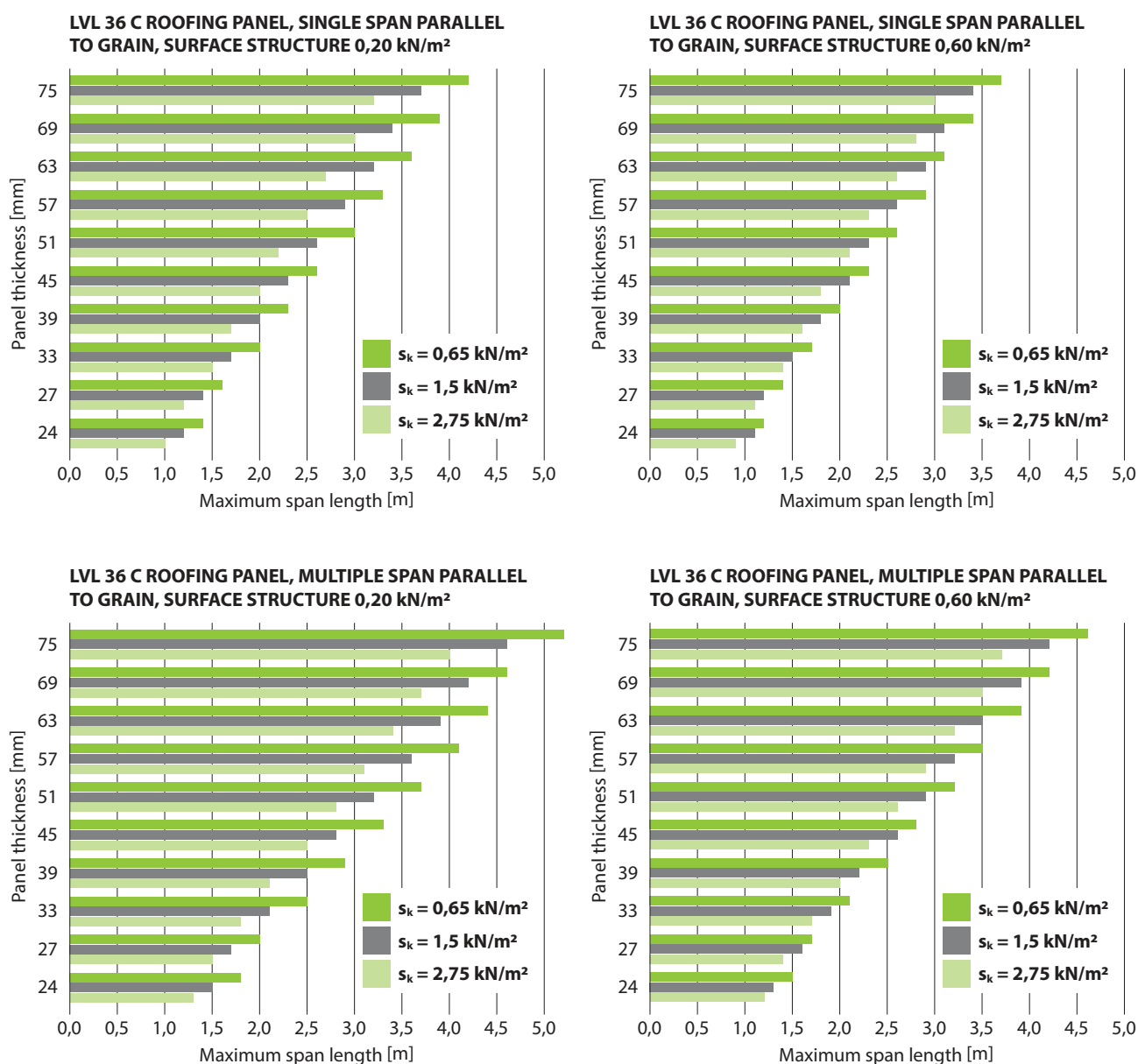


Figure 2.42. Span tables of LVL-C roof panels for predesign. Span length perpendicular to grain of surface veneers. Calculations according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex. Service class 2, Loads: g_2 = own weight of surface structure, s_k = snow load at ground level; $0,8 \times s_k$ = snow load at roof level. Wind load $w_k = 0,4 \text{ kN/m}^2$, maintenance load $q_H = 0,4 \text{ kN/m}^2$. Point loads are not taken into account. Deflection limit $w_{\text{net,fin}} \leq L/100$.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

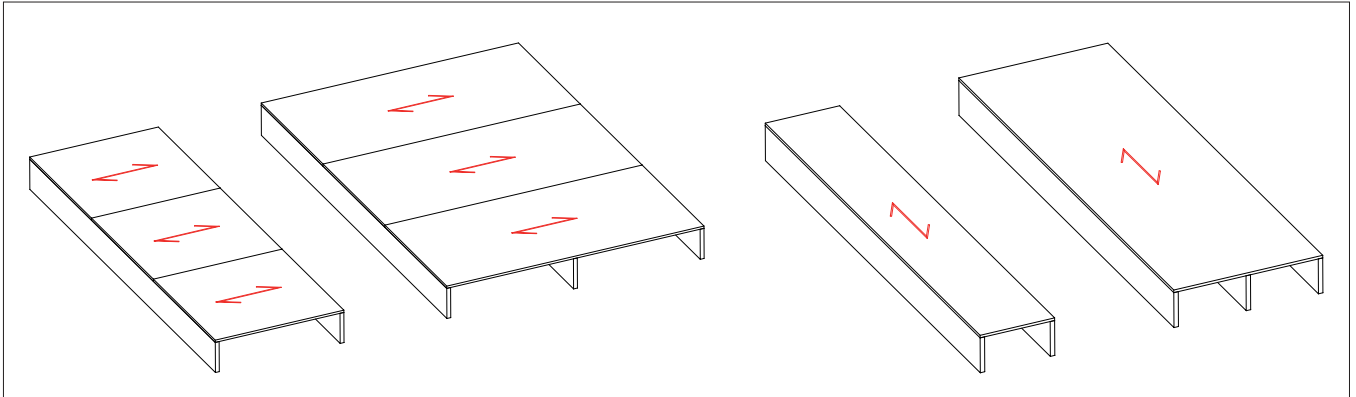


Figure 2.43. LVL-C roof panel orientations. Left: single span and multiple span parallel to grain of surface veneers. Right: single span and multiple span perpendicular to grain of surface veneers.

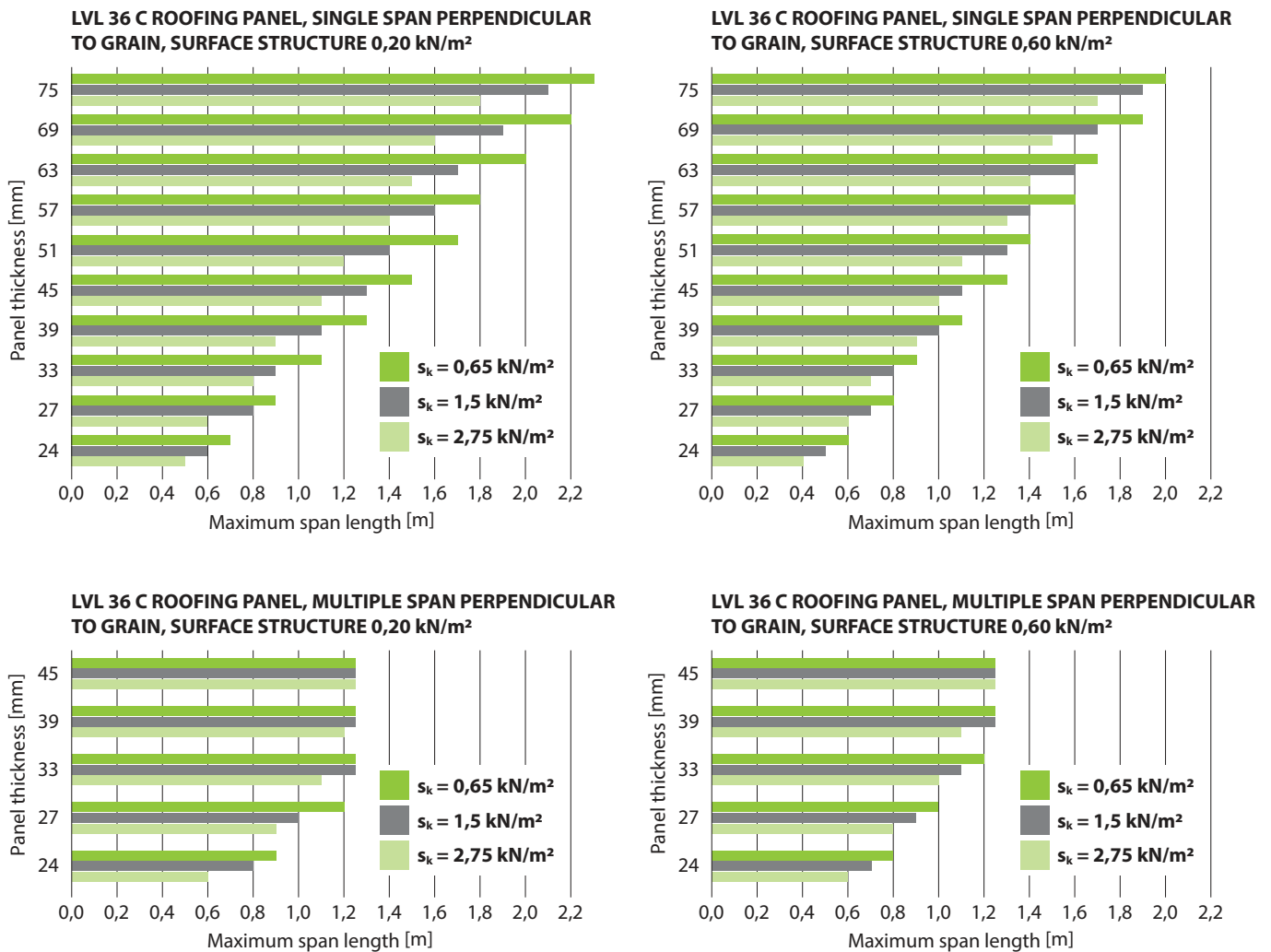


Figure 2.44. Span tables of multiple span LVL-C roof panels for predesign. Left: span length parallel to grain of surface veneers, Right: span length perpendicular to grain of surface veneer. Calculations according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex. Service class 2, Loads: g_2 = own weight of surface structure, s_k = snow load at ground level; $0,8 \times s_k$ = snow load at roof level. Wind load $w_k = 0,4 \text{ kN/m}^2$, maintenance load $q_H = 0,4 \text{ kN/m}^2$. Point loads are not taken into account. Deflection limit $w_{\text{net,fin}} \leq L/100$.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

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Figure 2.45. Installation of LVL-C roof panels.



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Figure 2.46. LVL-C roof panels.

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Figure 2.47. Eaves and roof overhangs from LVL-C panels.



Figure 2.48. Surface treated eave panel.

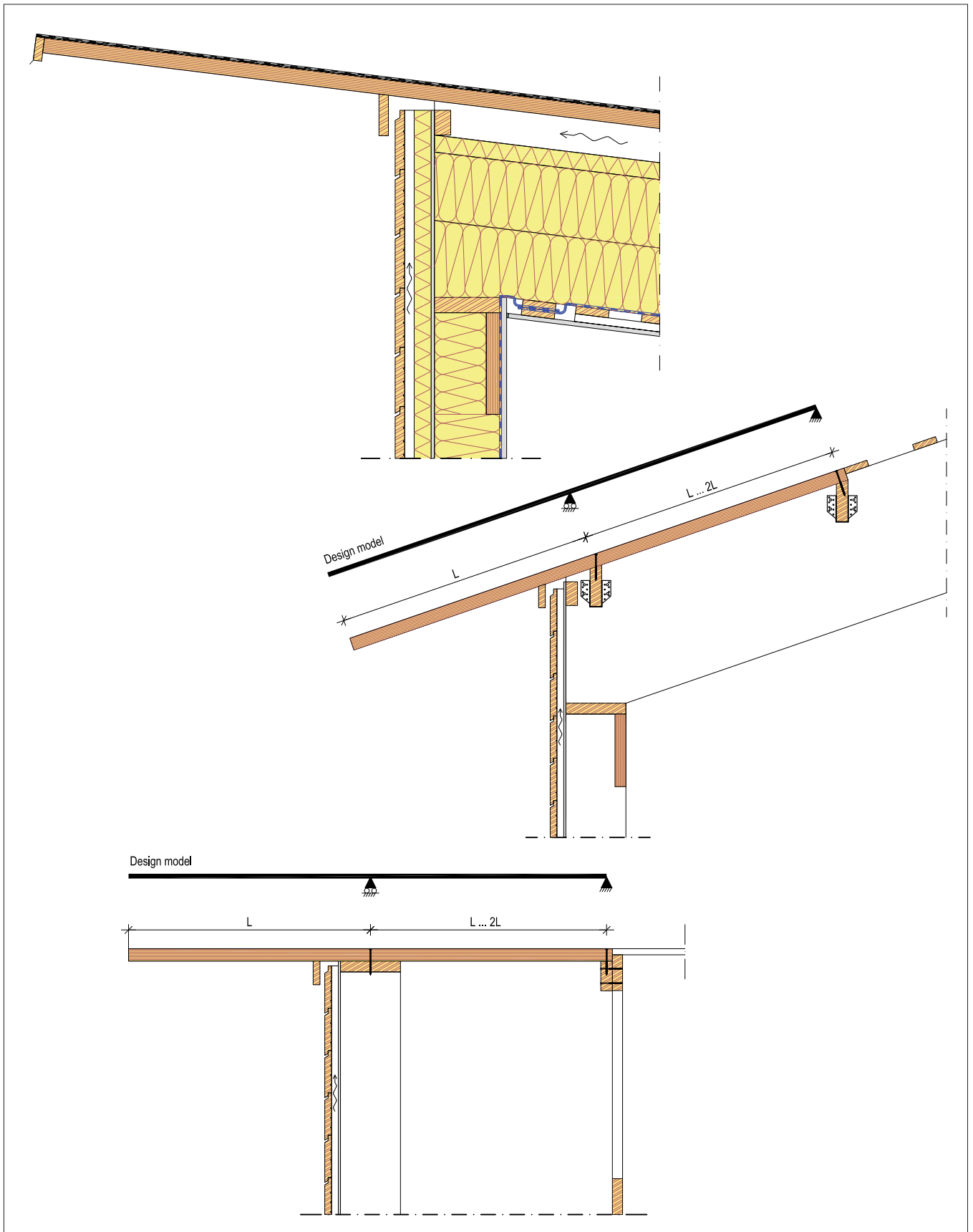
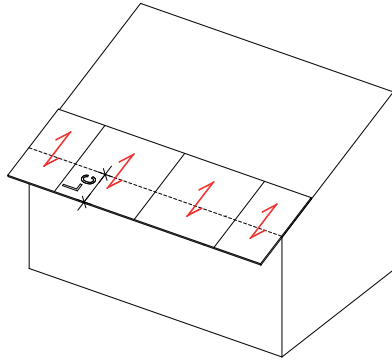
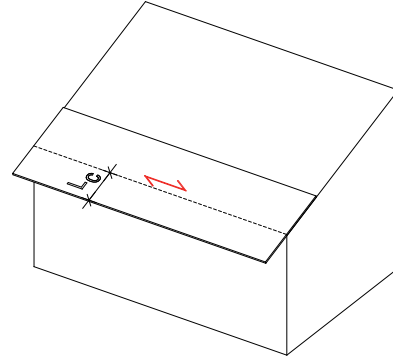
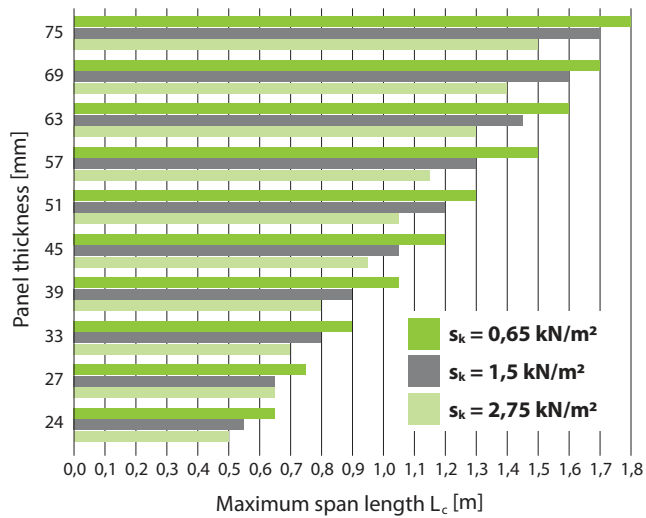


Figure 2.49. LVL-C roof panel overhang eaves.

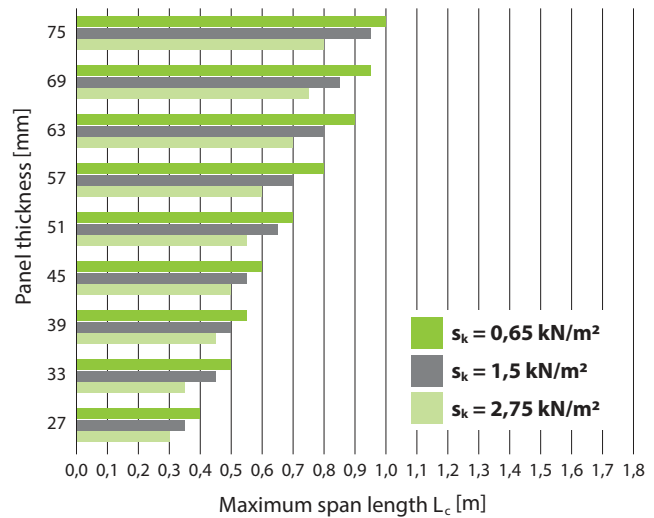
2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS



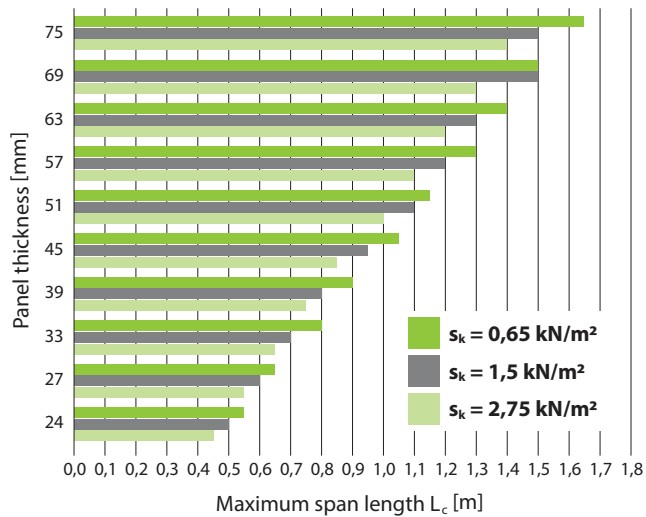
LVL 36 C ROOF OVERHANG PANEL, SPAN PARALLEL TO GRAIN OF SURFACE VENEERS, SURFACE STRUCTURE 0,20 kN/m²



LVL 36 C ROOF OVERHANG PANEL, SPAN PERPENDICULAR TO GRAIN OF SURFACE VENEERS, SURFACE STRUCTURE 0,20 kN/m²



LVL 36 C ROOF OVERHANG PANEL, SPAN PARALLEL TO GRAIN OF SURFACE VENEERS, SURFACE STRUCTURE 0,60 kN/m²



LVL 36 C ROOF OVERHANG PANEL, SPAN PERPENDICULAR TO GRAIN OF SURFACE VENEERS, SURFACE STRUCTURE 0,60 kN/m²

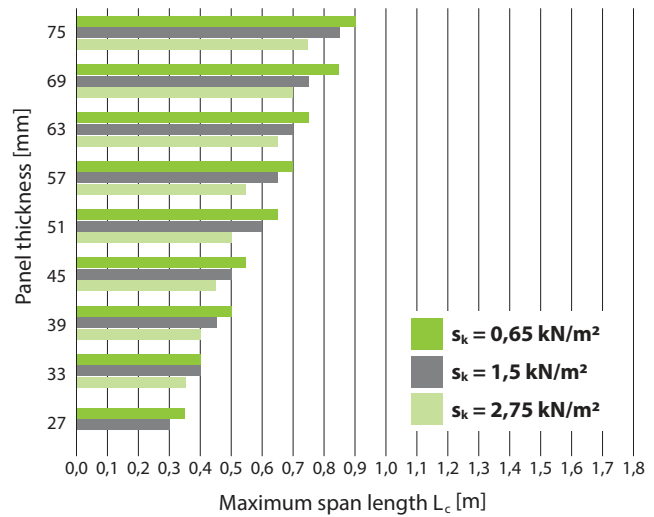


Figure 2.50. Span tables of LVL-C roof overhang panels for predesign. Left: Span length parallel to grain of surface veneers Right: Span length perpendicular to grain of surface veneer. Calculations according to EN 1995-1-1:2004+A1:2008 and its Finnish National Annex. Service class 2, Loads: own weight of surface structure 0,20 or 0,60 kN/m², s_k = snow load on ground level 0,65, 1,5 or 2,75 kN/m² and snow load on roof level = $0,8 \times s_k$. Wind load $w_k = 0,4 \text{ kN/m}^2$; maintenance load $q_H = 0,4 \text{ kN/m}^2$. Point loads are not taken into account. Deflection limit $w_{net,fin} \leq L_c/100$.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

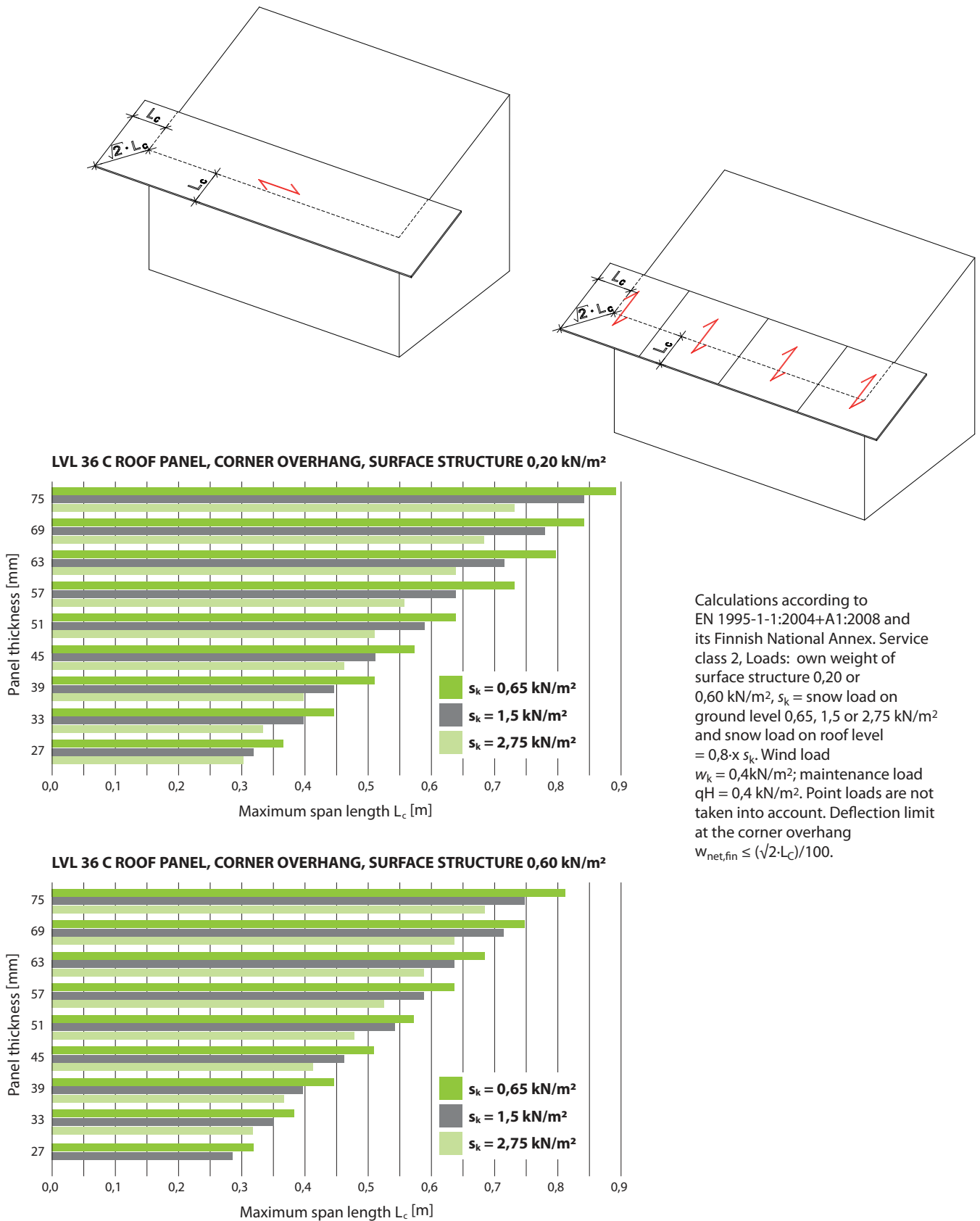


Figure 2.51. Span tables of LVL-C roof corner overhang panels for predesign. Span length L_c is the same parallel and perpendicular to grain of the surface veneer.

2.3.6 Bracing panels

Bracing is needed to transfer horizontal loads, like wind loads, from the building frame to the foundations. When panel bracing is used, additional diagonals or other bracing systems are usually not necessary and the structure becomes simpler. LVL-C panels are ideal for bracing in various types of buildings as they are easy to fasten to most materials and LVL-C panels are available in large sizes. Frame spacing can be large without risk of bracing panel buckling. Horizontal bracing panels transfer horizontal loads to the vertical wall bracing panels which, in turn, transfer the forces downwards. If the edges of the panels are not fully supported, the resistance against lateral torsional buckling of the compressed edge must be verified as a beam. Fastening the panels to the frame structures is essential for the capacity of the bracing. Preliminary design values for panel fastenings are presented in Section 5.7.

In roofs and walls of conventional buildings, LVL-C panels can also be utilized as a water vapour and air barrier on the interior side of the structure, so no separate barrier layer is needed. The fire behaviour of LVL-C is highly predictable, which makes it a suitable bracing panel also with respect to fire design ²⁴.

2.3.7 Large LVL I-beams and box beams

LVL can be used for the main beams of roof structures in large buildings when they are structurally glued to larger I-beams or

box beams. The advantage of these LVL beams is that they are not sensitive to cracking in dry or changing moisture conditions. The beam heights can also be tailored project-specifically. The beams have LVL-C webs and LVL-P flange components. Recommended beam heights are 900, 1200, 1800 or 2500 mm. I-joists have the best material efficiency, but if the fire resistance requirement is higher than R15, box beams are required.

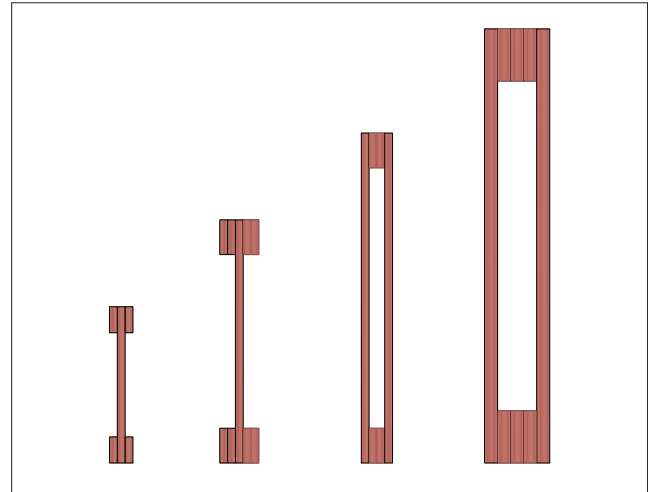


Figure 2.53. Large LVL I-beams and box beams for main beams of roofs.

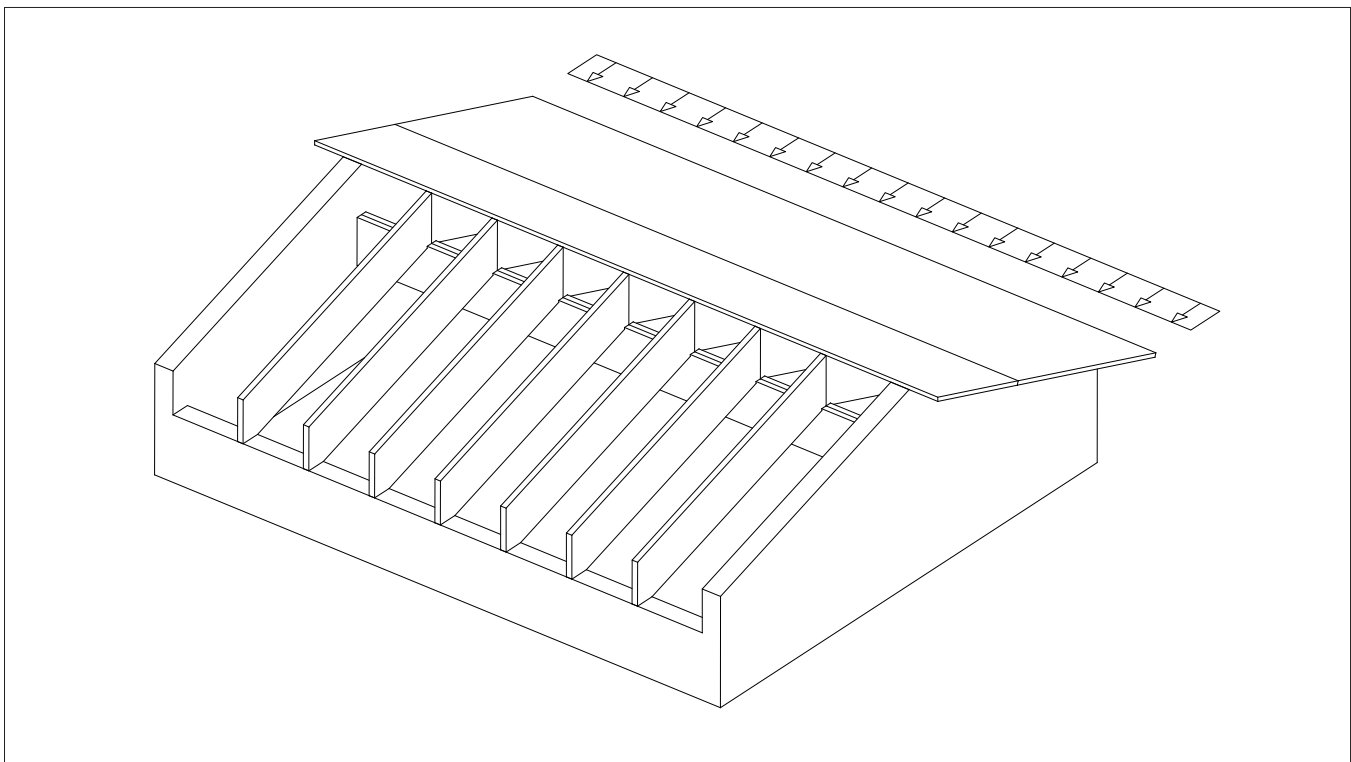


Figure 2.52. Panel bracing of roof structure.

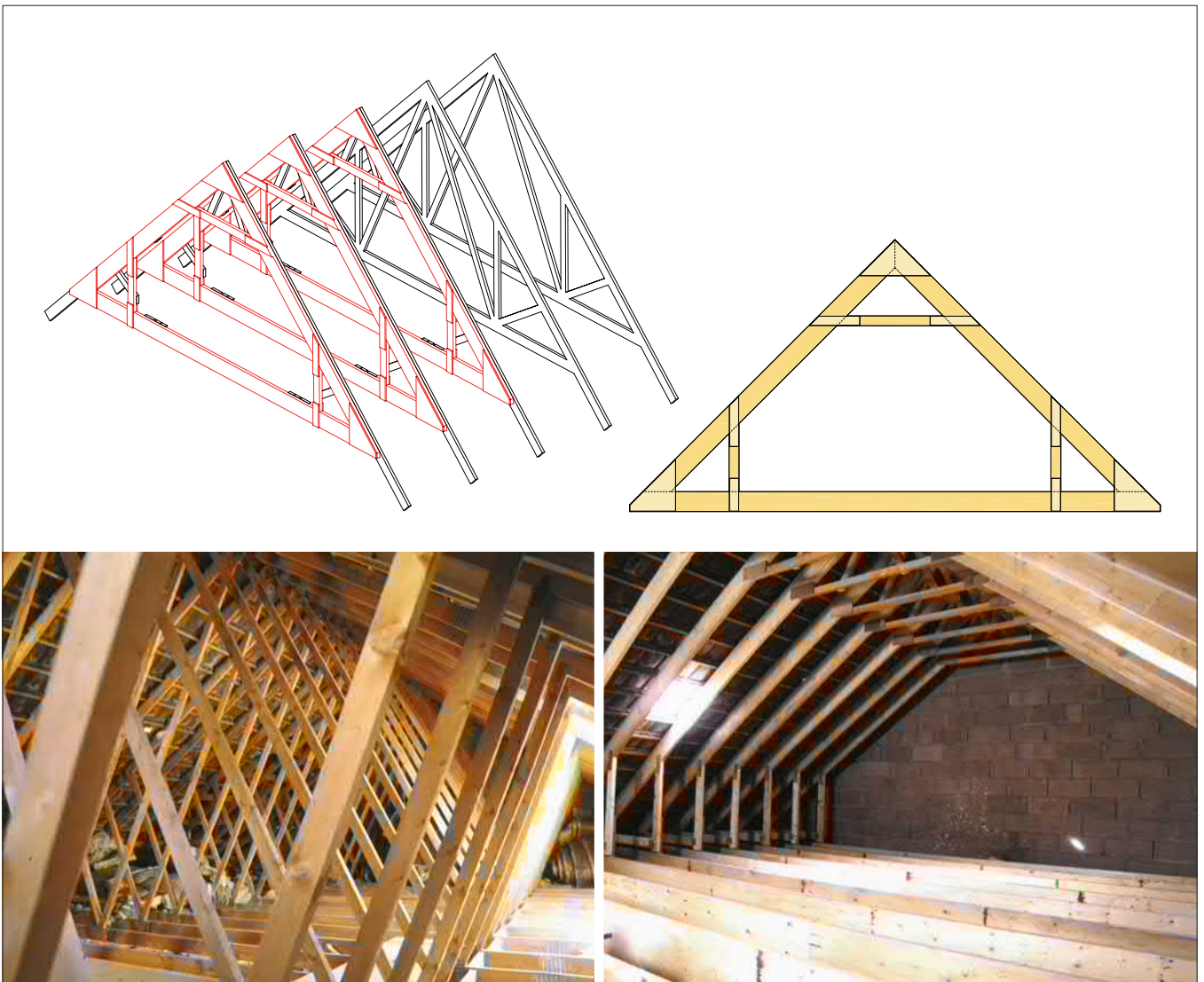


Metsä Wood

Figure 2.54. Large LVL I-beams as main beams of a warehouse roof structure at Herttoniemi, Helsinki, Finland.

2.3.8 Roof renovation applications

In the renovation of residential buildings, nail plate truss roof structures can be converted to room-in-the-roof attics by reinforcing or replacing the top and bottom cords with LVL-P beams and removing the truss diagonals. LVL-C or plywood connection plates are nailed or screwed to the corners to build a new rigid frame structure. The economical span range for the solution is 8-10 m depending on snow loads and thermal insulation requirements. It is recommended to leave at least a 100 mm ventilation gap between the thermal insulation and roof covering. The building physics design must, however, be done project-specifically based on national requirements.



Metsä Wood

Figure 2.55. Conversion of nail plate truss structure to room-in-roof attic. Top and bottom cords have been reinforced with LVL-P beams and truss diagonals removed. LVL-C or plywood is used for the frame corner connections.

2.4 WALL STRUCTURES

LVL is used as straight, dimensionally precise vertical studs in timber frame walls. Small dimension studs can be used for non-load-bearing walls, long lengths for high walls, and GLVL columns to support large constructions. Horizontal LVL members: rim boards, lintels, headers, top cords, ledger beams and sole plates transfer loads from upper walls, floors and the roof to the load-bearing studs. Robust LVL-C panels brace the buildings where loads are high or where space for the bracing system is limited, e.g. due to large door and window openings.



Figure 2.56. Installation of plasterboards to LVL-P wall studs.

2.4.1 Studs for timber frame walls

Straight studs with accurate dimensions are essential for timber frame walls. LVL-P studs create a straight and even base for good quality wall panelling without extra skimming work in the finishing phase. As in vertical structures the highest strength properties are usually not required, LVL 32P is a suitable choice and is available in sizes and lengths tailored for stud applications. It is easy to cut and fasten without special tools. This makes construction work faster and reduces waste. The smallest cross sections are ideal for non-load-bearing interior walls as the smaller stud sizes allow more space in the rooms. Larger sizes can be used e.g. in tall load-bearing walls when the required lengths of other stud types are not available ²⁵. In the lowest floors of timber frame multi-storey buildings, where the loads are the largest, the LVL 48 P stud is an ideal option as it has twice the compression strength of C24 solid wood. However, compression perpendicular to the grain of a sole plate under the studs may set a limit for the minimum stud size.

For the dimensioning of non-load-bearing interior walls, it is recommended to check their resistance and deformation based a 0,5 kN/m horizontal rail load at a height of 1 m. When stud spacing is 600mm and instantaneous deformation is limited to $H/200$, the standard stud size 39x66mm is suitable for a maximum 2500mm high walls and the size 45x66mm for 3000mm high walls ²⁵. Chapter 4 provides further information on the dimensioning of LVL studs in load-bearing walls.

As the building physics properties of LVL-P studs are similar to solid wood, the acoustic and thermal properties specified for timber frame walls are valid also for structures with LVL-P studs with similar dimensions and detailing.

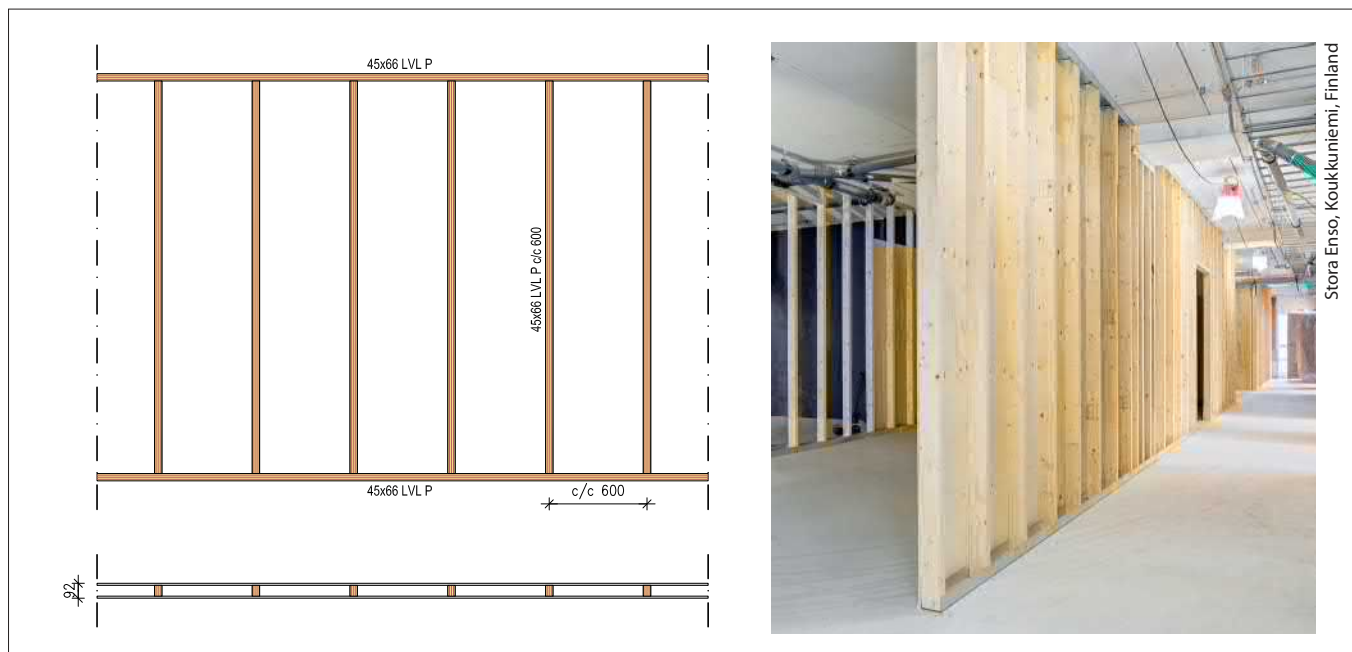
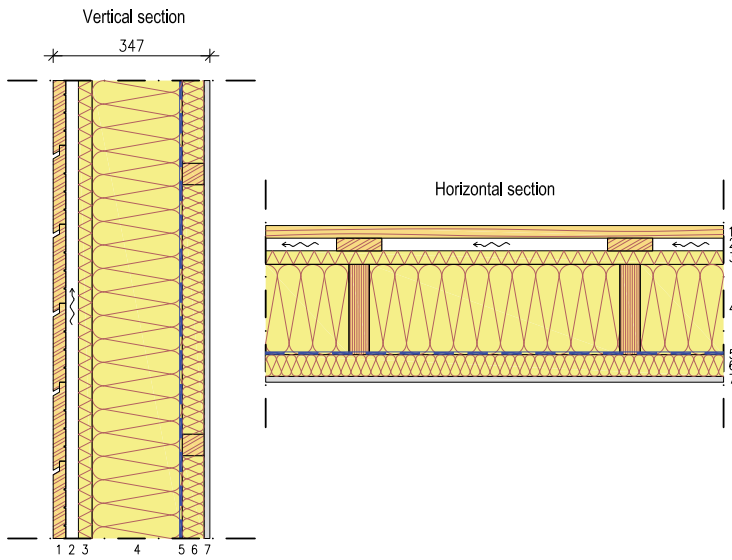


Figure 2.57. LVL-P studs for non-load-bearing interior walls.



STRUCTURAL LAYERS

1. External cladding
2. 28x98 timber battens c/c 600 + air gap
3. 30 mm windproof layer + insulation
4. 45x200 LVL P c/c 600 + 200 mm mineral wool insulation
5. Water vapor membrane
6. 48x48 timber cross-battens c/c 600 + mineral wool insulation
7. Internal paneling



Figure 2.58. LVL-P studs for tall load-bearing exterior walls.

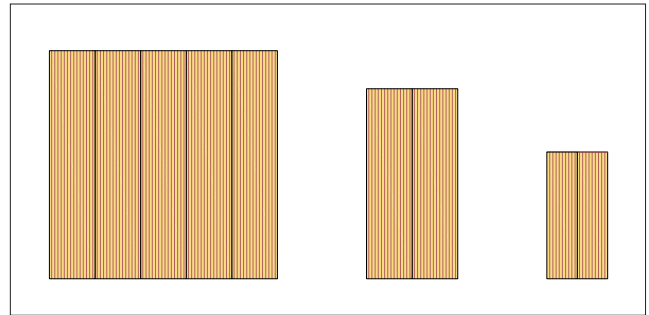


Figure 2.60. Multiple-glued GLVL columns for beam and post structures.

2.4.2 Multiple-glued GLVL columns

Multiple-glued GLVL can be used in larger cross sections for load-bearing columns in post and beam structures. They are glued together from several LVL-P or, in some cases, LVL-C laminas. The column cross sections can be tailored project-specifically, but for optimized material efficiency the standard widths or 5-10 mm smaller widths are recommended. Column thicknesses are derived from multiple sanded LVL laminas.

- Recommended column thicknesses: 84, 90, 96, 108, 120, 133 and 144 mm
- Recommended column widths: 200, 225, 240, 260, 300, 360, 450 and 600 mm

Massive multiple-glued GLVL columns are possible for heavy-duty structures up to glulam beam dimensions. The maximum width is usually 400 mm, but the thickness can be up to 2000 mm. In visible applications, massive multiple-glued GLVL cross sections have the advantage that they are not sensitive to cracking in dry conditions. However, for visible applications the sanding and lamina specifications must be separately agreed with the supplier to ensure clean veneer surfaces.

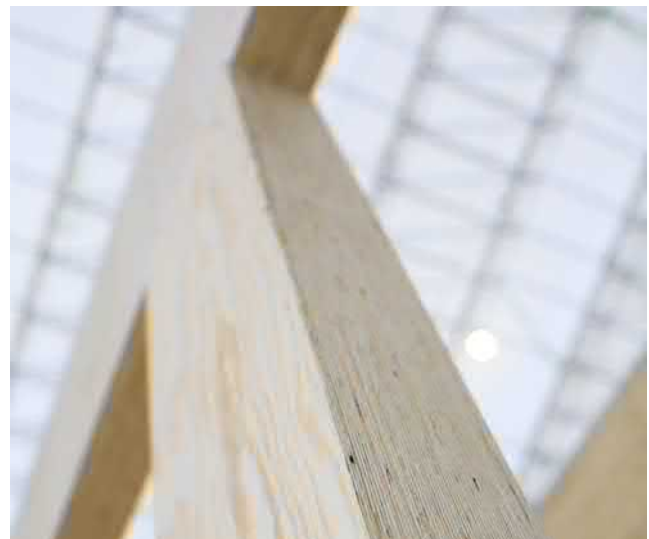


Figure 2.59. Multiple-glued GLVL beam and post structures. Pro Nemus, Äänekoski, Finland.

2.4.3 Lintels

Lintels are one of the most common applications of LVL due to its ideal dimensions and good strength and stiffness properties. LVL's good shear strength also means slim cross sections can be used as lintels for window openings. This improves the energy efficiency of timber frame wall structures as there is more space for thermal insulation and the cold bridging effect can be reduced. For large openings, e.g. of garage doors and car-ports, rigid LVL-P lintels reduce deflection and enable longer span lengths.

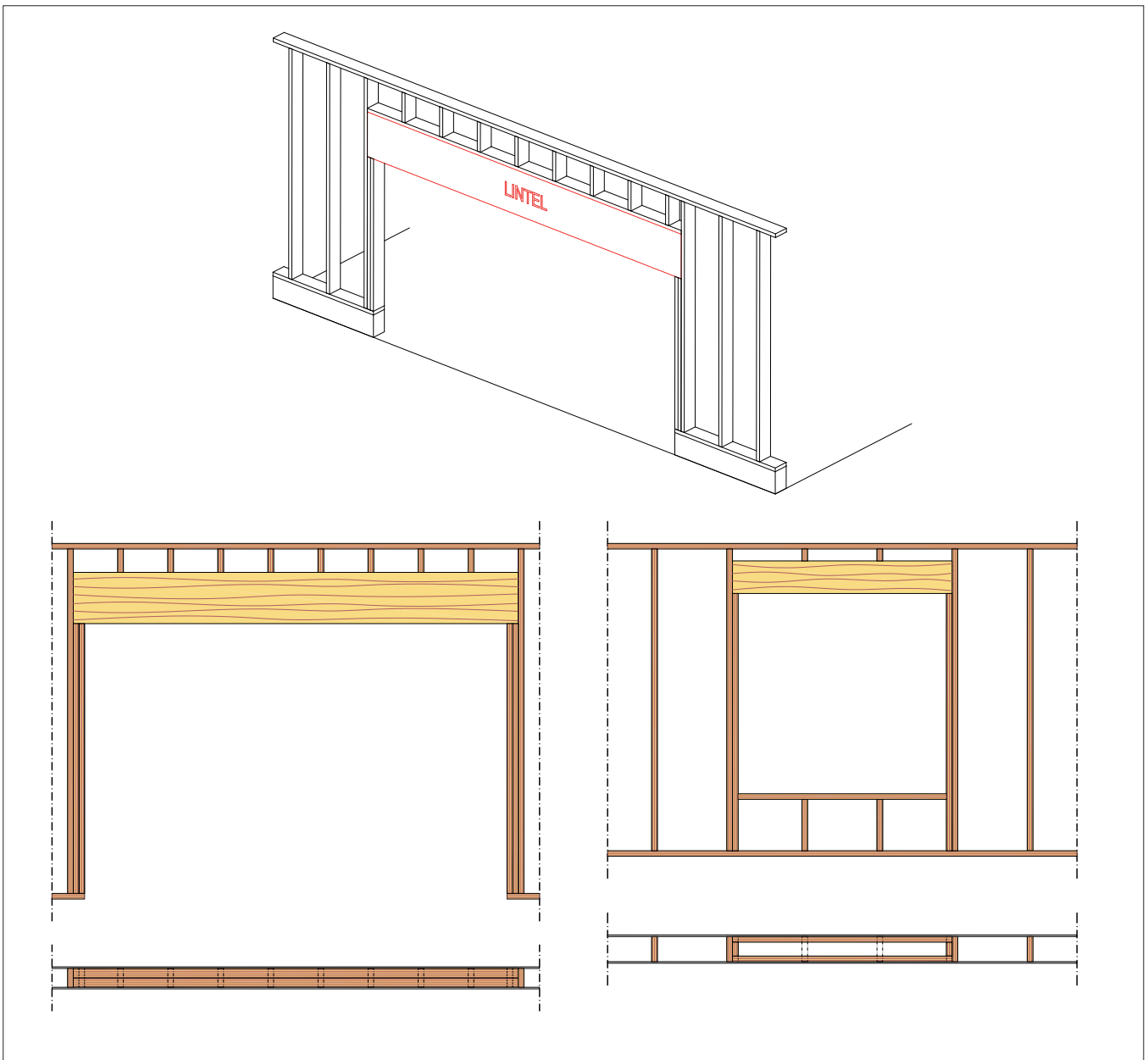


Figure 2.61. Left: Straight and stiff LVL lintel for long-span garage door opening in timber frame wall; the straight, slim member has minimal deflection. Right: Lintel for window opening inside the timber-frame wall. Key benefit is larger adaptability of opening positioning with fewer load-bearing columns and more space for thermal insulation to improve the energy efficiency of the building envelope.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

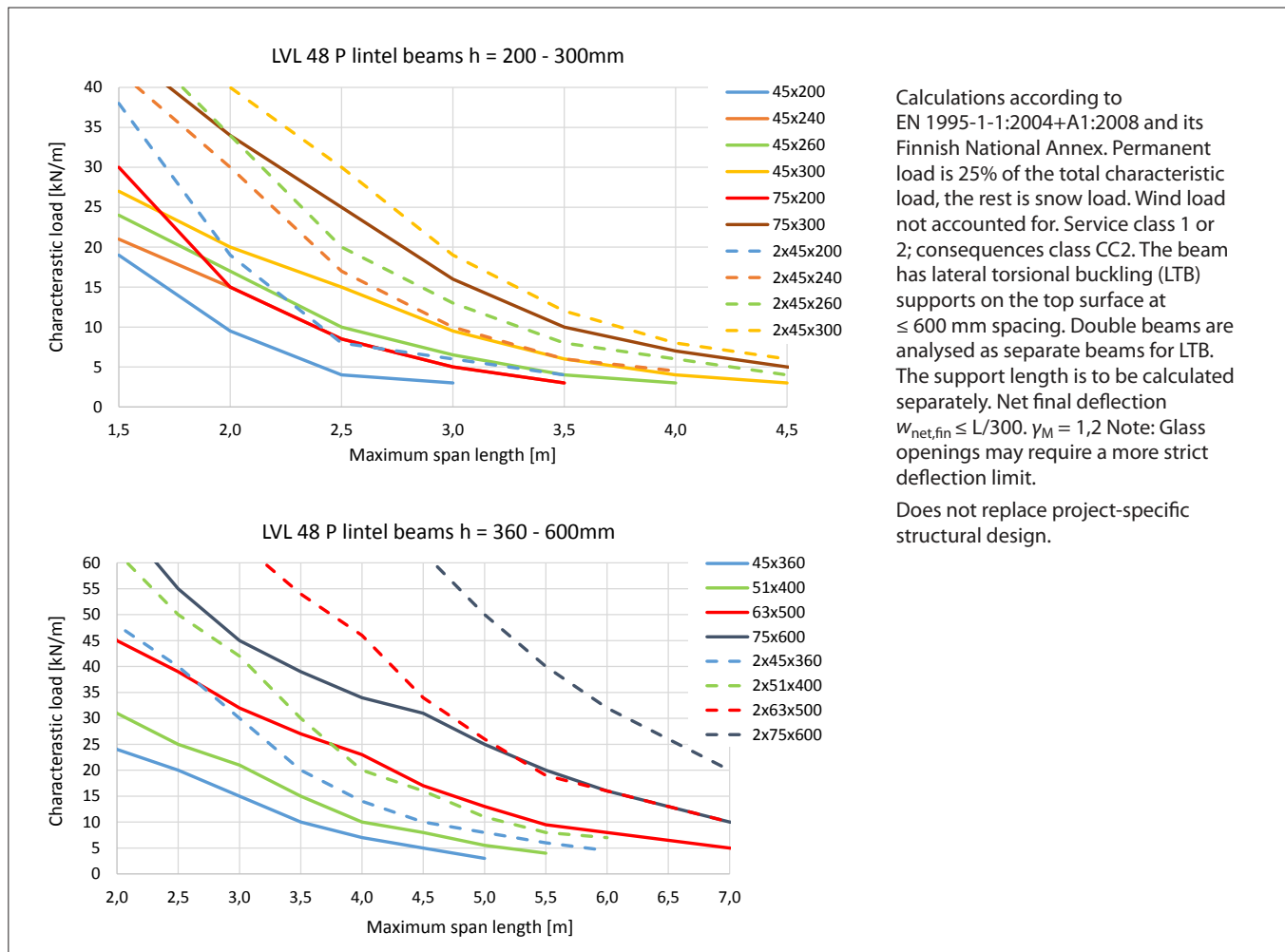


Figure 2.62. Capacities [kN/m] of single-span LVL 48 P lintel beams for predesign.



Figure 2.63. Integrated rim beam in timber frame wall structure.

2.4.4 Integrated rim beams

Integrated rim beams of timber frame walls are an essential and common application for LVL-P in Europe. They are installed on top of notched wall studs and carry the loads from the roof or other structures above to the studs. Since the integrated rim beams transfer the loads, the floor joist, roof rafter or roof truss spacing can be independent of the wall stud locations. Integrated rim beams simplify the structure so that separate lintels are not required except for the largest openings. In off-site production integrated rim beams are installed integrally in the wall elements.

As LVL is available in long lengths, builders often use the dimensionally accurate and straight LVL-P members to align and brace the wall frame when they are assembled and to ensure that the overall dimensions of the building are correct. Similar to lintels, LVL integrated rim beams improve the building physics detailing by reducing cold bridging compared to massive beams, as there is more space for thermal insulation. LVL integrated rim beams are typically 45-51 mm thick and 200-300 mm high.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

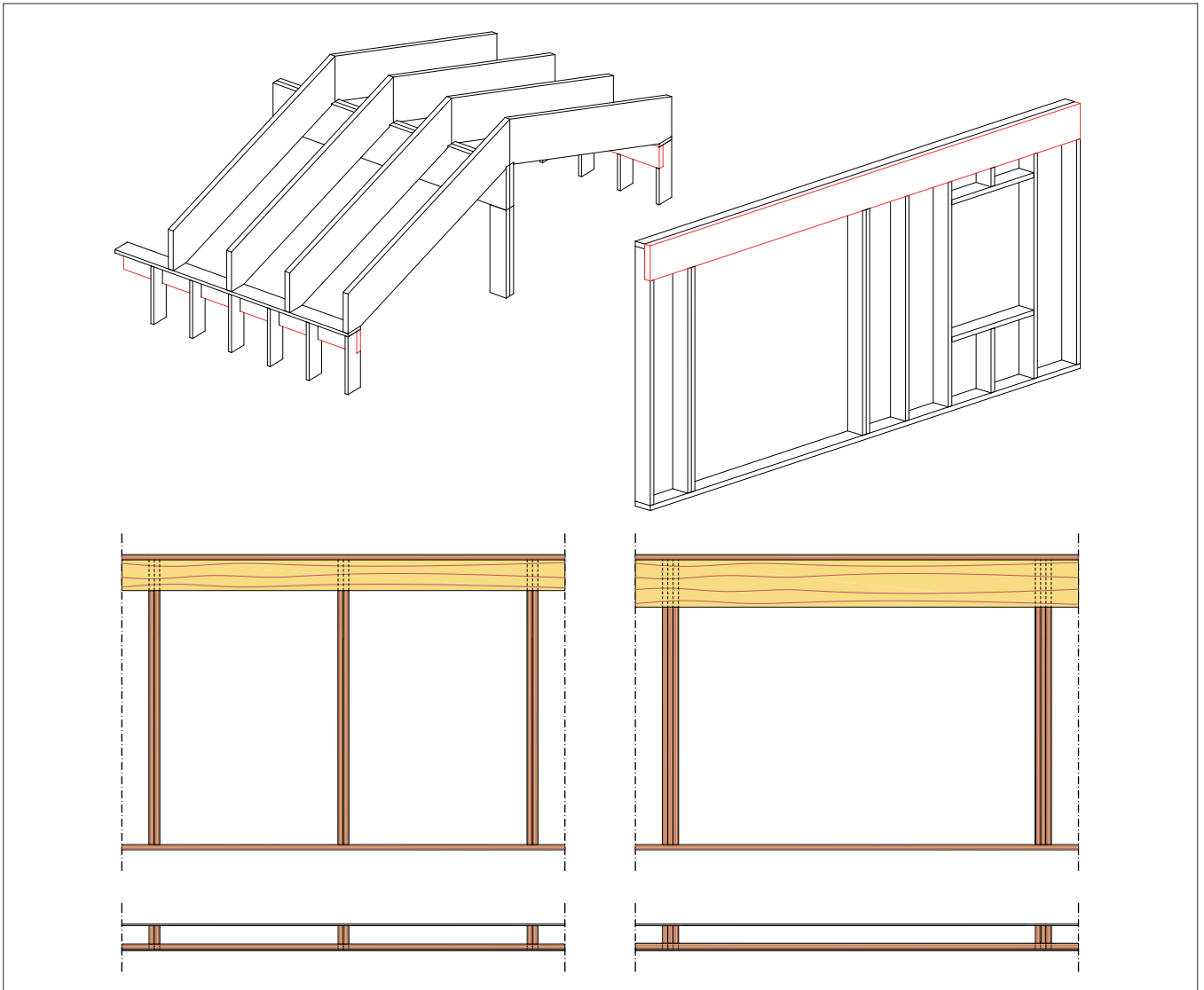


Figure 2.64. LVL-P integrated rim beam for timber frame wall structures. The bracing and load-bearing beam fits the thickness of timber-frame walls. Key benefit: More possibilities for opening positioning with fewer load-bearing columns.

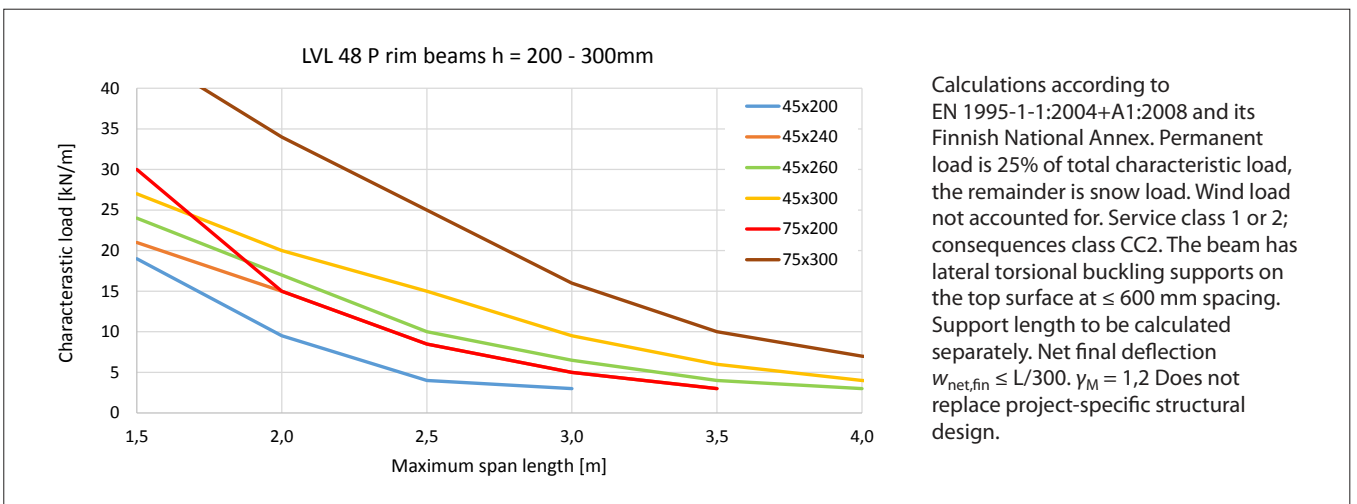


Figure 2.65. Capacities [kN/m] of single-span LVL 48 P integrated rim beams for predesign.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

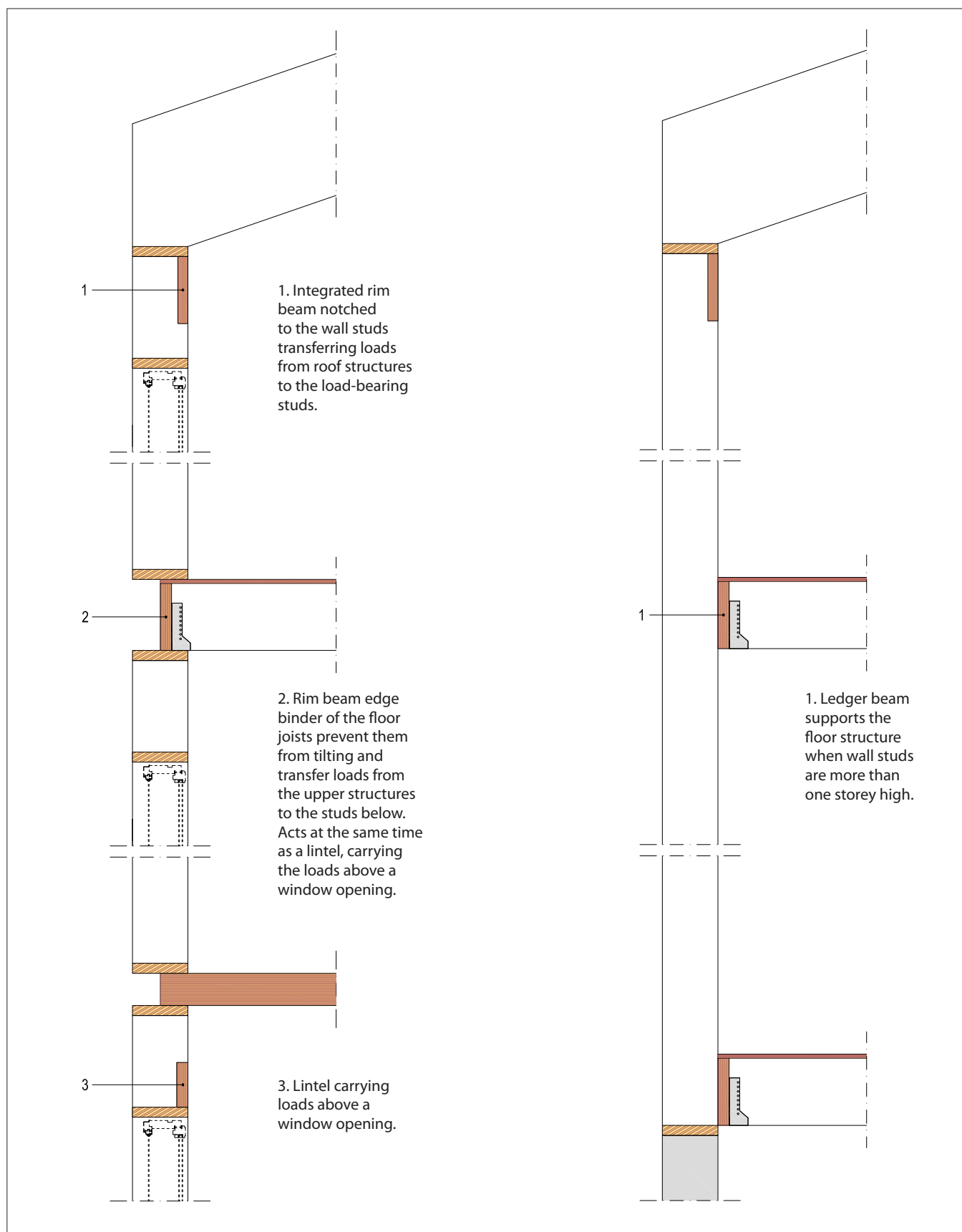


Figure 2.66. Different alternatives of rim beams, lintels and ledger beams in wall structures.

2.4.5 Ledger beam supports for floors or canopies

In a timber frame wall of a balloon construction where the walls are higher than one storey, the floor structures are connected to the walls with a ledger beam which is either fixed to the wall stud with nail or screw connections or sits in notches in the wall studs. Similarly, canopies for, e.g., entrances can be fixed to the exterior wall frames with ledger beams. They should be at least 45 mm thick to provide adequate bearing length for the joists, or 39 mm thick if the joists are connected with hangers. The beam height depends on the stud spacing and the number of nails or screws needed for the connections. The bending moment from the ledger beam supported structure must be taken into account in the stud design.

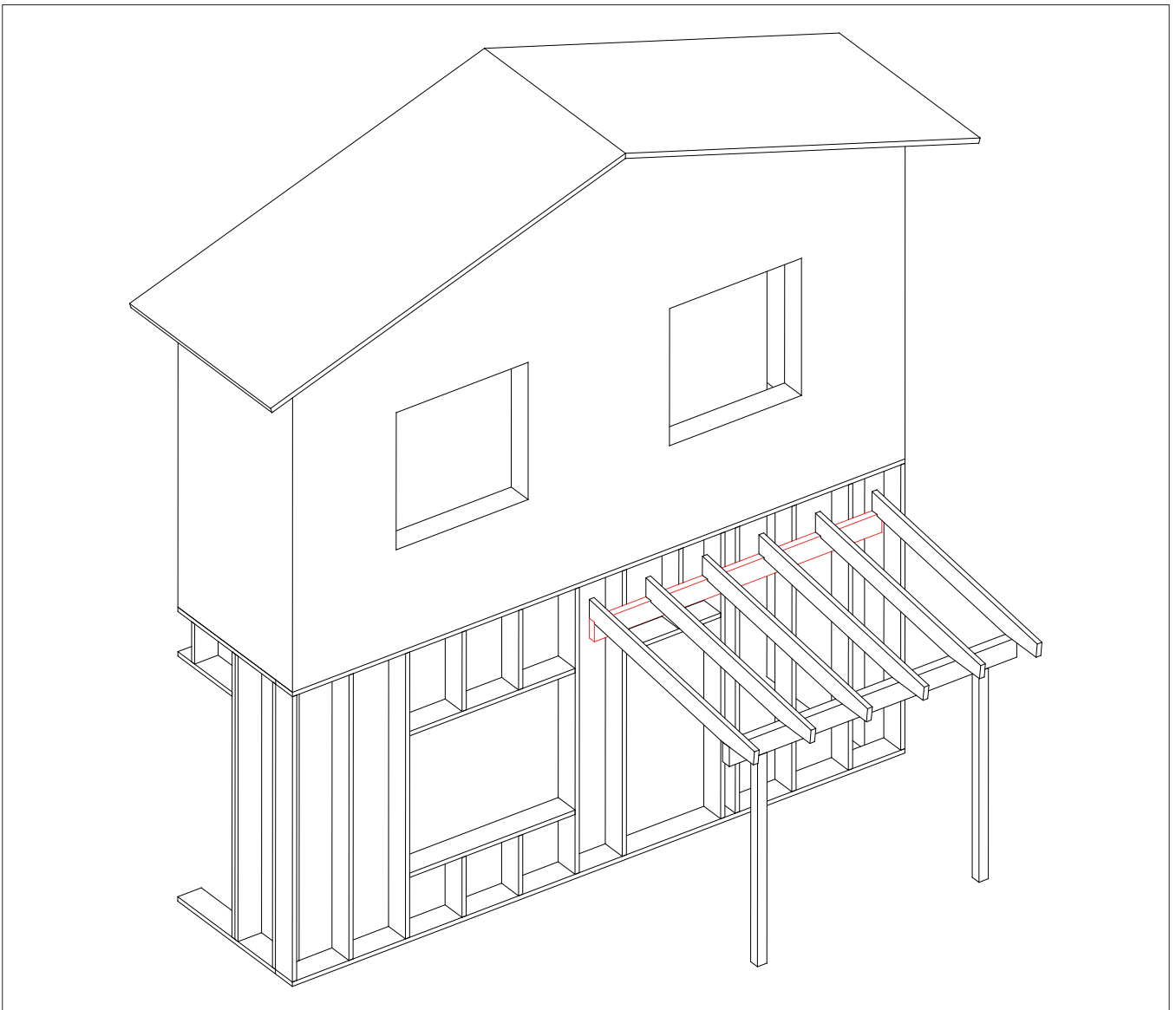


Figure 2.67. Ledger beam supporting a canopy. Can be fixed to the timber frame wall studs with nail or screw connections or notched in to the studs.

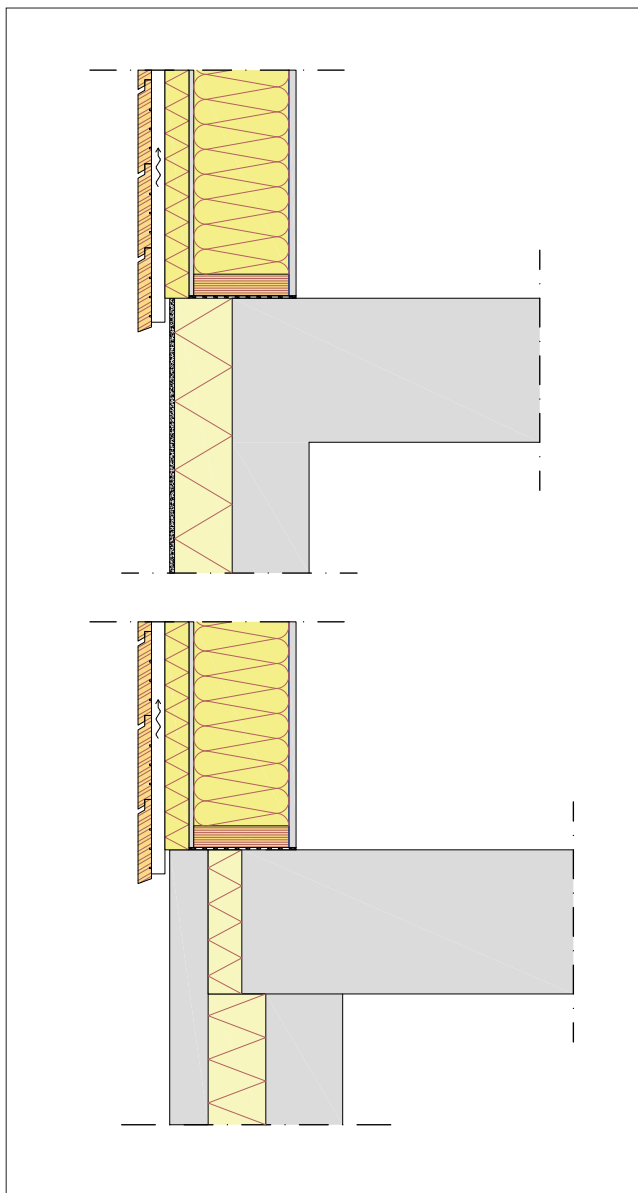


Figure 2.68. Sole plates from LVL. Thin plates reduce settling perpendicular to grain and reduce cold bridges. Left: LVL-C sole plates may be extended beyond the edge of foundation to reduce cold bridges in the structure. It is recommended to connect the anchoring hold-downs to the studs.

2.4.6 Sole plates, bottom cords, top cords and top binders

LVL-P or LVL-C boards function flatwise as sole plates, bottom cords under wall studs, and top cords or top binders above studs in timber frame walls. The members can be designed to thinner thicknesses than solid wood, e.g. 45 mm, as they are dimensionally stable and not sensitive to twisting or cracking. Smaller thickness reduces settlement of the frame in the perpendicular to grain direction. LVL sole plate and bottom cord sizes also make good combinations with I-studs in timber frame walls. For the best wall bracing performance it is recommended to connect the hold-down anchors to the wall studs.

LVL-C sole plates can be extended beyond the edge of the concrete foundation slab to create a drip edge. This helps to insulate the foundation, e.g. rendering panels, and to avoid cold bridges.

As compression perpendicular to the grain is lower flatwise compared to an edgewise orientation, an LVL product with higher strength made from pine veneer can be supplied for this application to help reduce the cross-section sizes of highly loaded studs, e.g. next to windows.

The need for treatment against insects or decay is to be verified according to national requirements. However, if a bitumen felt strip is installed between the sole plate and the concrete foundation service class 1 conditions are met for the sole plate and chemical treatment is usually not mandatory.

2.4.7 Window frame boards

Long and straight LVL-C boards offer a practical solution for the installation of window elements or other cladding elements in concrete buildings. The boards are extended beyond the concrete frame to create a flat and aligned surface for the facade and to compensate the tolerances of the main frame. This saves time and labour costs and improves the building quality. Wedge anchors can be used for the connections. Board sizes depend on the structure of the facade, in the example of Figure 2.69. the size is 39x260 mm.

2.4.8 Bracing panels

Robust LVL-C panels are suitable for panel bracing of timber frame walls and beam & post structures when high capacity is required or there is only limited space for the bracing structure, e.g. due to large window openings.

Panel bracing simplifies the structure, as trusses or diagonal bars are usually not necessary. LVL-C panels up to 75 mm thickness allow large frame spacing without risk of buckling the bracing panel.

In timber frame walls the LVL-C panels are screwed or nailed to the studs and to sole and top binders which together form the diaphragm. The thickest panels or multiple-glued elements can act as bracing columns when they are anchored to the foundations.

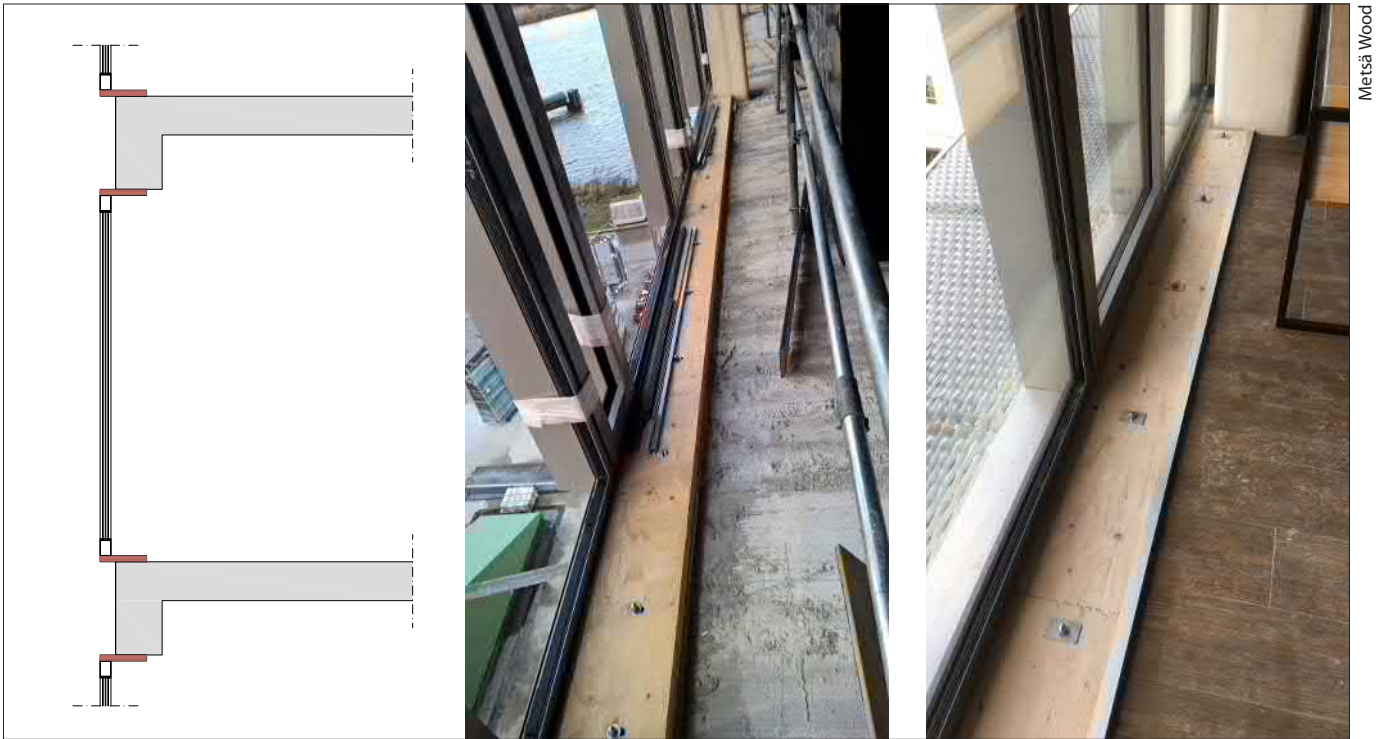


Figure 2.69. LVL-C boards for installing window and cladding elements in concrete buildings. The boards may be a visual element of the architectural detailing.

LVL-C is available in panel widths up to 2500 mm. Wide panels reduce the number of joints to be nailed or screwed, thus reducing installation work. LVL-C panels also provide an ideal solution for higher than normal wall heights. Tables 2.1 and 2.2 show the capacities of 1.2 m and 2.4 m wide single panel sections installed in wooden frames calculated according to method A of Eurocode 5, subsection 9.2.4. Multiple sections are summed together to get the total capacity of a wall. Sections with openings are not taken into consideration, except for the

transfer of horizontal forces to adjacent sections. Anchoring of robust diaphragms can be critical, especially if the sections are narrow. Small fastener spacings increase the capacity of the bracing. 75-150 mm is a suitable spacing for nailing and 100-200 mm for screwing at the panel edges. At the centre studs of the panels the fastener spacing can be doubled without reducing the bracing capacity. The capacities are based on the panel, nail and stud combinations specified in Section 5.7.

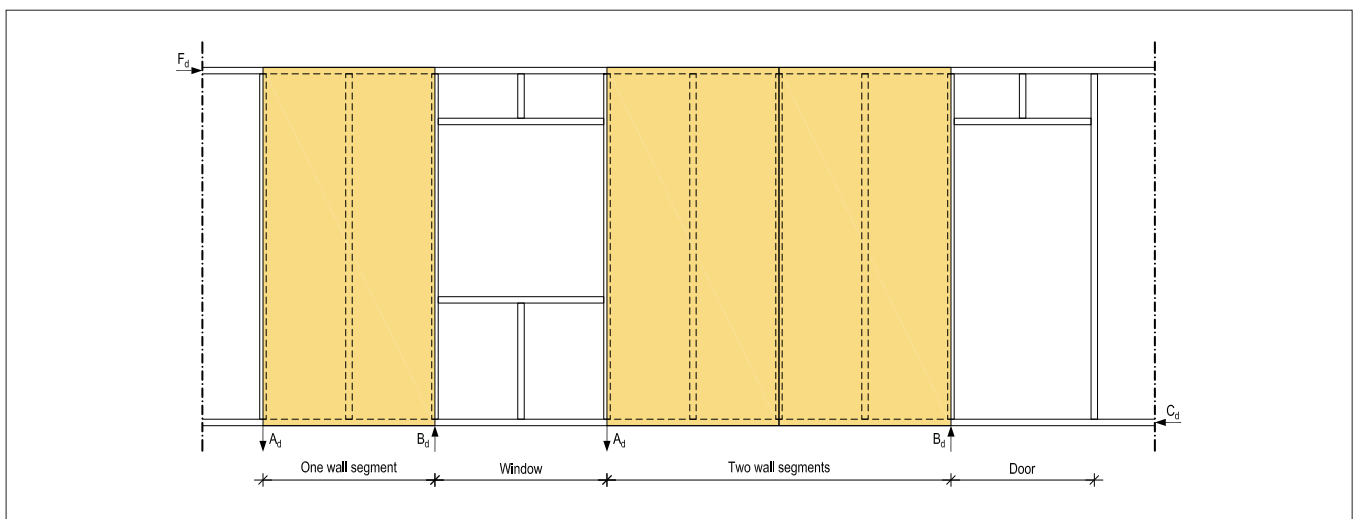


Figure 2.70. Principle of panel bracing to carry horizontal force F_d .

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS



Figure 2.71. Multiple-glued GLVL-C bracing column.



Figure 2.72. LVL-C bracing panels.

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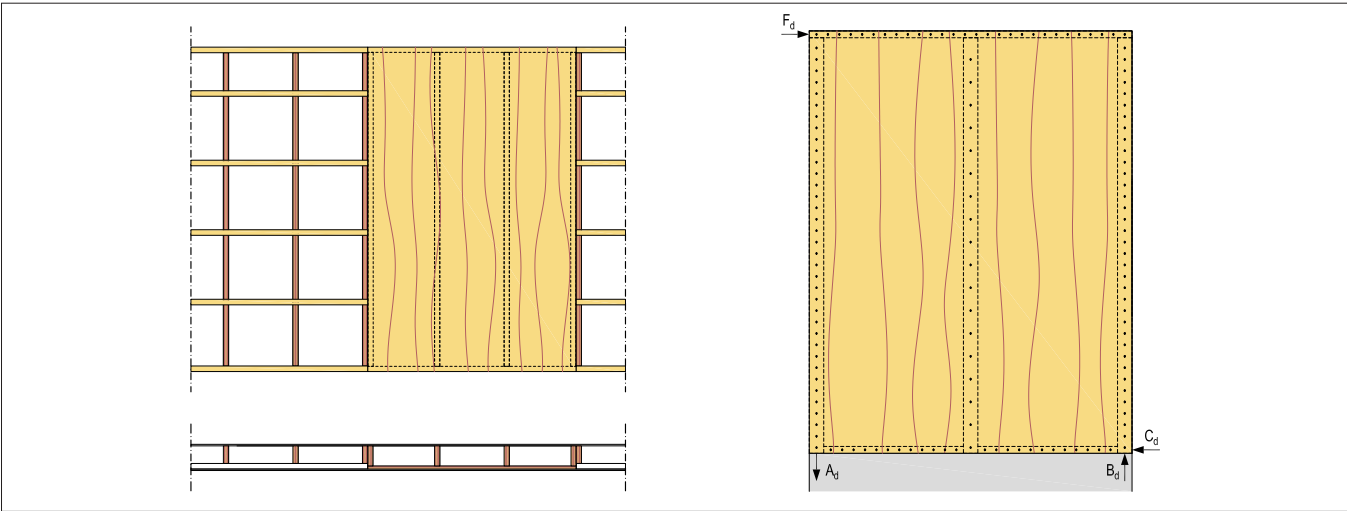


Figure 2.73. Robust LVL-C bracing panel integrated into the wall structure.

Table 2.1. Characteristic racking load capacity $F_{i,V,Rk}$ [kN] of LVL 36C panels nailed to LVL 48 P frame for predesign. The height of the LVL 36 C panel is 3,0 m. The distance between the frame studs should not be more than the panel width.

LVL 36 C PANELS NAILED TO LVL 48 P FRAME		Panel width					
		1200 mm			2400 mm		
Nail spacing at panel edges [mm]		150	100	75	150	100	75
Nail spacing at centre studs [mm]		300	200	150	300	200	150
Panel thickness [mm]	Nail size $d \times L_{min}$ [mm]	Racking load capacity $F_{i,V,Rk}$ [kN] of wall panel					
24	2,1x50	3,6	5,4	7,2	8,9	13	17
27	2,5x60	4,8	7,2	9,5	11,5	17	23
33	2,8x70	5,7	8,6	11,5	14	21	28
45	3,1x90	6,8	10	13	17	25	34

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

Table 2.2. Characteristic racking load capacity $F_{i,V,Rk}$ [kN] of LVL 36C panels nailed to C24 frame for predesign. The height of the LVL 36 C panel is 3,0 m. The distance between the frame studs should not be more than the panel width.

LVL 36 C PANELS NAILED TO C 24 FRAME		Panel width					
		1200 mm			2400 mm		
Nail spacing at panel edges [mm]		150	100	75	150	100	75
Nail spacing at centre studs [mm]		300	200	150	300	200	150
Panel thickness [mm]	Nail size $d \times L_{min}$ [mm]	Racking load capacity $F_{i,V,Rk}$ [kN] of wall panel					
24	2,1x50	3,3	4,9	6,6	8,2	12	16
27	2,5x60	4,4	6,6	8,8	10,5	16	21
33	2,8x70	5,3	7,9	10,5	13	19	26
45	3,1x90	6,2	9,4	12	15	23	31

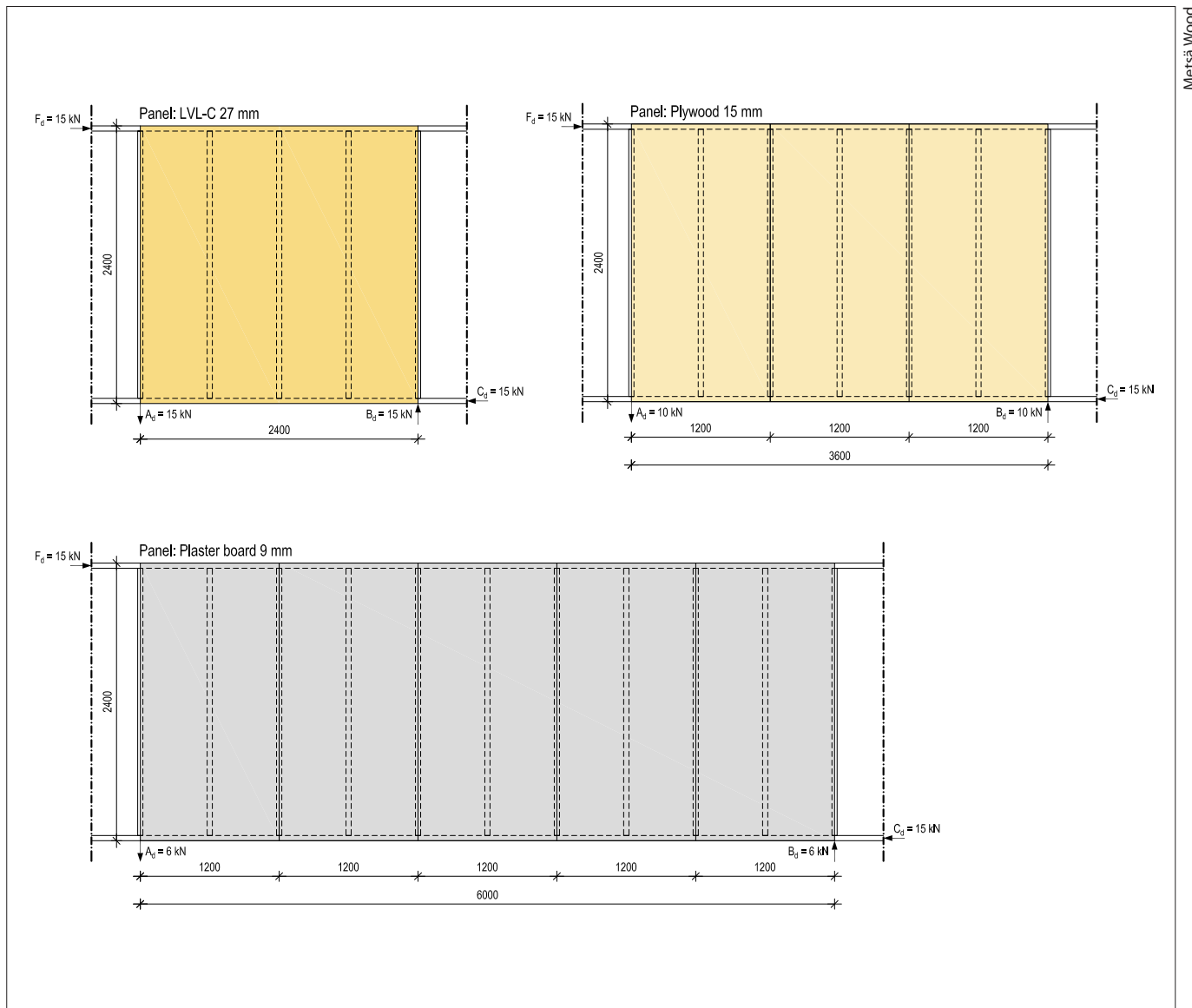


Figure 2.74. A wall diaphragm made with 27 mm LVL 36C panels, 15 mm plywood panels and 9 mm plasterboard panels have equal capacity. LVL-C panels are the best solution where space for bracing is limited; however, the increased anchoring force needs to be taken into account in connection detailing ²⁴.

2.4.9 LVL-C panel walls

In addition to panel bracing, LVL-C panels can be used in walls for carrying vertical loads as a combined structure together with timber frame studs. When multiple-glued to provide more robust GLVL panel thicknesses or if the LVL-C panels are part of a glued sandwich structure with rigid insulation, they may form the main load-bearing structure. LVL-C panels may contribute to providing EI or K class fire protection for walls. For the required panel thicknesses, see the tables in Sections 6.3 and 6.4.

Stora Enso



Figure 2.75. Multiple-glued GLVL-C panel walls.

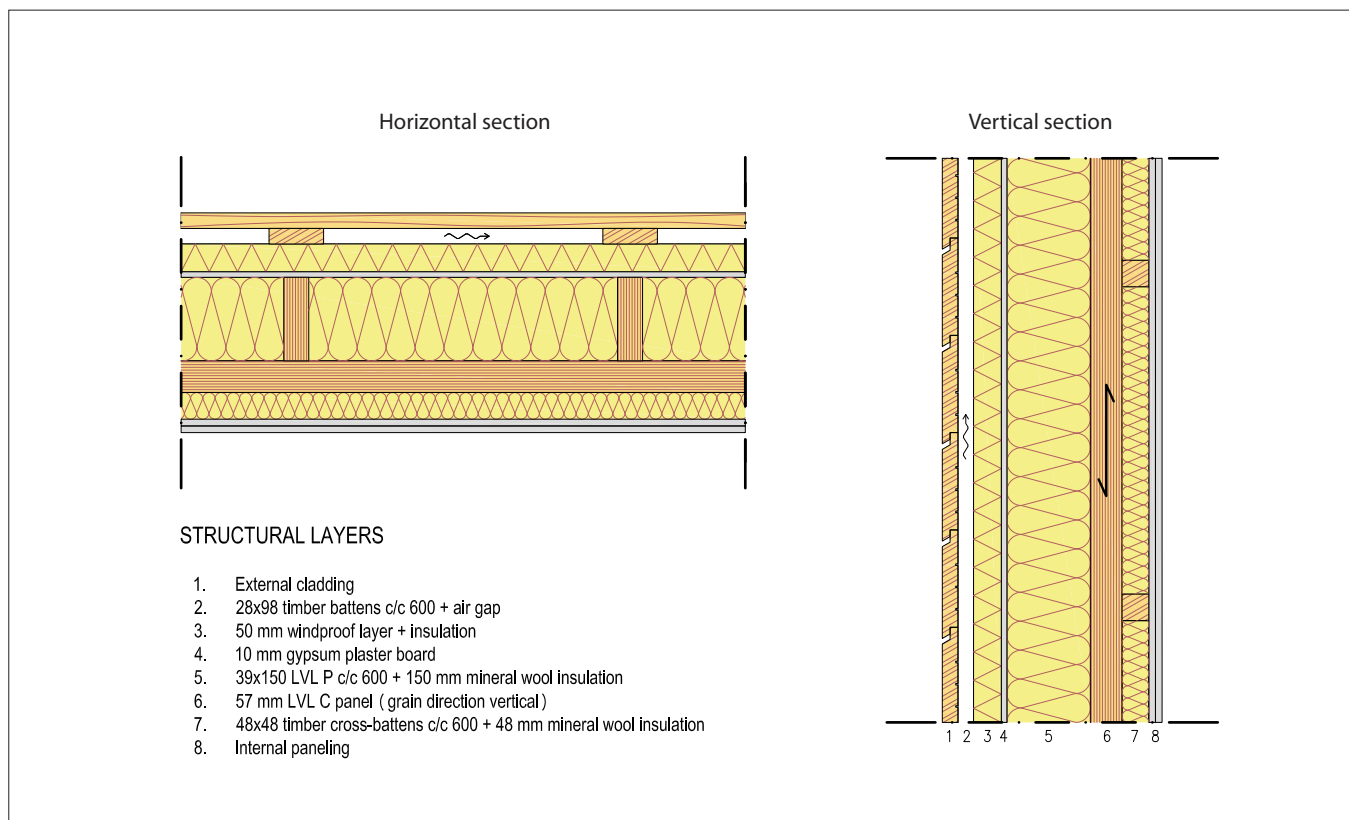


Figure 2.76. LVL-C panels as load-bearing structure together with LVL-P wall studs.

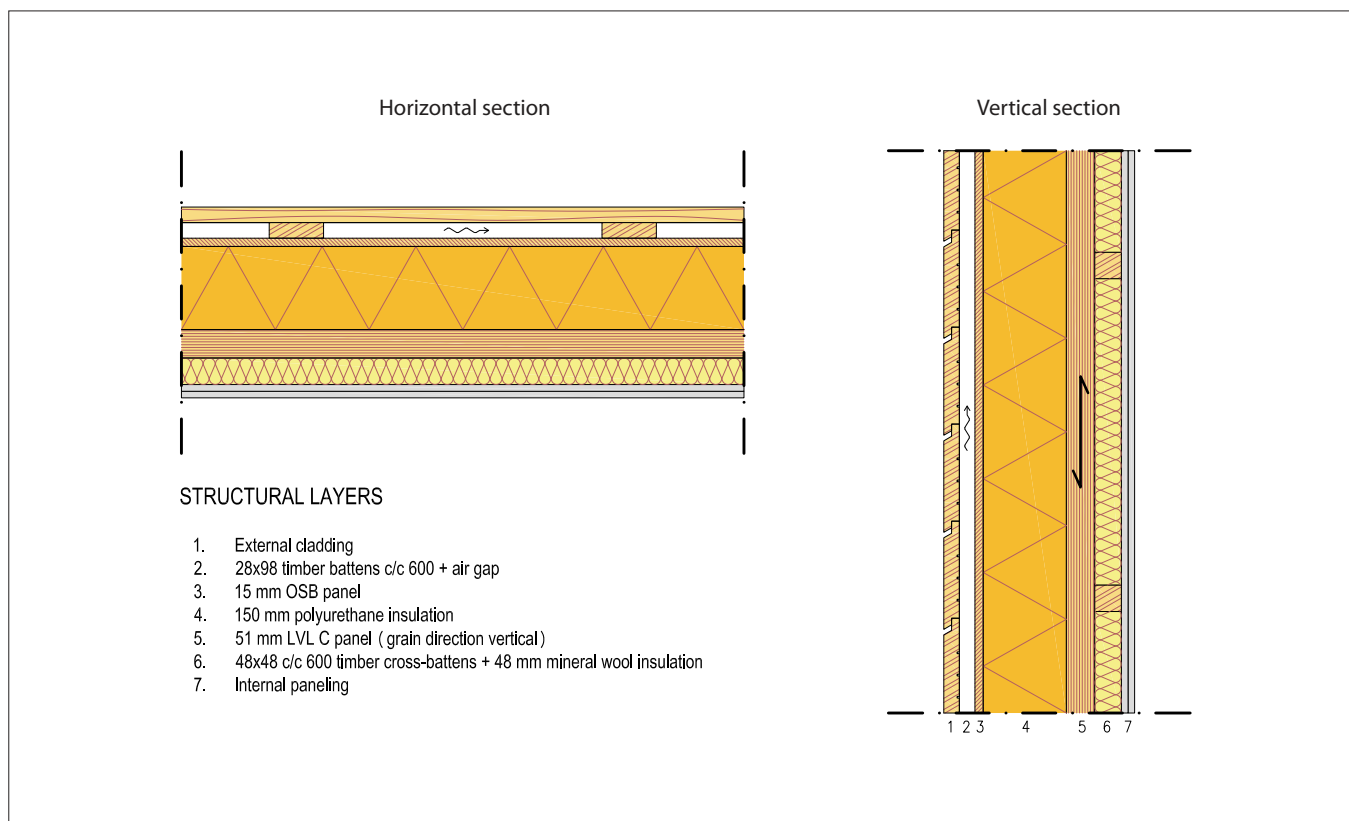


Figure 2.77. LVL-C panels as load-bearing structure of sandwich wall element structure.

2.5 SPECIAL STRUCTURES

The strength, stiffness, light weight, precise dimensions, large range of sizes, free forms of LVL-C members and easy machining of LVL are advantages that can be utilized in many special structures. LVL cords give extra capacity to nail plate trusses when longer resistance to fire is required. Visually impressive LVL trusses or portal frames can enable longer spans or maximize the interior height of a hall. Door panels and frames gain form stability and improved burglar safety from LVL components. Complex geometry and ambitious architectural designs can be realized with free-form LVL-C members that can be cut to shape as required. LVL can be used to reinforce critical parts of old structures and light engineered LVL wood structures enable facilitated extensions of existing buildings. The entire deck

of a pedestrian bridge can be realized from just two LVL-C panels saving considerable site work and achieving a simple and robust bracing solution. In other applications LVL-P is used, e.g., as safe, proof-loaded scaffold planks and stiff flange members of wooden I-joists for floor systems.

2.5.1 Trusses and portal frames

LVL-P bottom chords provide advantages for nail plate trusses, and the LVL-P thicknesses (42 mm in Scandinavia) match the solid wood members used in them. In attic trusses, the slim and tall LVL-P bottom chords give the rigidity needed to fulfil the vibration design requirements of floor constructions.

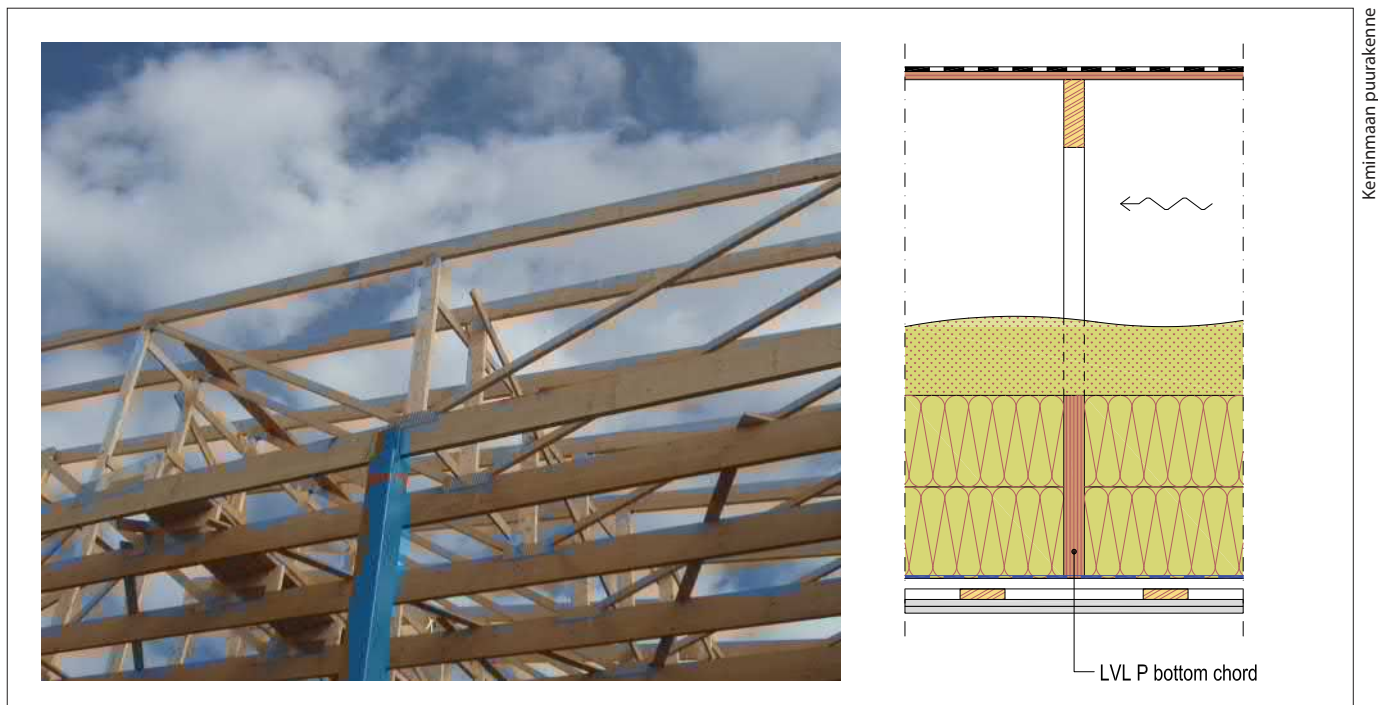


Figure 2.78. LVL-P bottom chord for nail plate trusses for attic floors and R30 fire resistant roof structures.

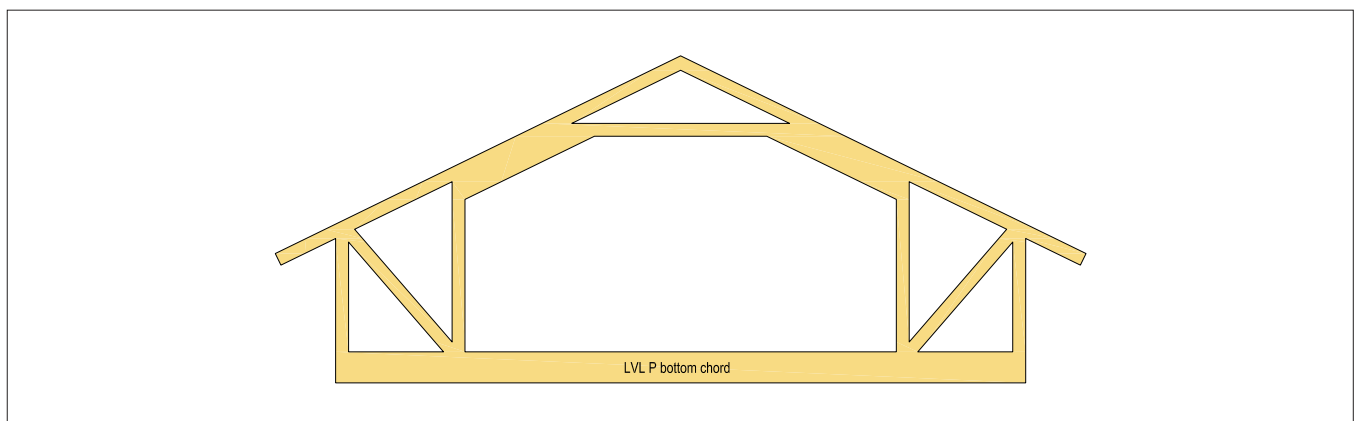


Figure 2.79. LVL-P bottom chord provides stiffness as a floor member of an attic truss.



Figure 2.80. Left: LVL queen post trusses, Fupicsa production hall, Spain. Right: King post trusses, Manese Wasseström, Tammisaari, Finland.

In roof structures that have higher resistance to fire requirements than the nail plate trusses can achieve, an LVL-P bottom chord can be designed to carry the loads in the case of fire as a beam, allowing the rest of the frame to be designed to normal temperature requirements. Sides are protected with stone wool insulation and the top edge is supported against lateral torsional buckling.

When roof trusses are left visible, LVL-P king post or queen post trusses with dowel type connections are an aesthetically pleasing solution. They are the most competitive option for 15-22 metre spans in cases where double-tapered solid beams are uneconomical due to the roof slope and where the fire resistance requirement is R15. With larger trusses, e.g. for sports halls, LVL can be used for all truss components, as diagonals combined with glulam chords, or in hybrid structures with steel members.

A 3-pin frame structure is a good solution when the interior height of a hall needs to be maximized. The built-up column part of the frame has two LVL-C panels which are screwed and glued together with wooden battens to form a box structure. The intermediate battens are shorter than the LVL-C panels which creates a space for the rafter part of the 3-pin frame to sit in. The battens should therefore have the same width as the rafter. The rafter part of the 3-pin frame is a single LVL-P beam, box beam or a glulam beam. The moment-rigid corner is a timber-to-timber connection made with bolt, screw or dowel circle without separate steel parts. The size and number of fasteners in the corner connection depend on the size of the hall and the loads. The crosswise veneers in LVL-C panels provide good connection strength and prevent the risk of corner cracking²⁴.

To optimize material use, it is recommended to choose column panel and rafter sizes that fit the LVL panel width when they are diagonally cut as a single taper. The wider end is at the moment-rigid corner connection and the narrower end to-



Figure 2.81. LVL roof trusses. Ydalir, France.

wards the pin points: the ridge and foundations. 3-pin frames are the most competitive when the capacity of a single LVL-P rafter is adequate for the loads. In areas with high wind loads a small roof angle is preferable and in areas with high snow load a larger roof angle is preferable. The economical span range is 10-30 m¹⁹.

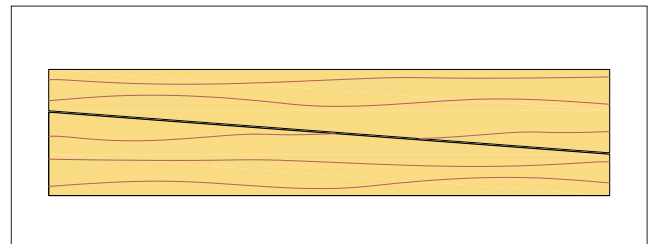
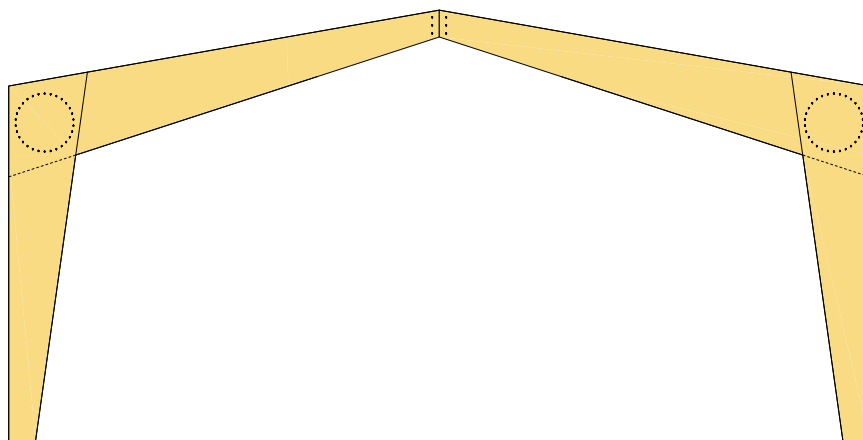


Figure 2.82. Principle of diagonal sawing of LVL panels to produce single-tapered column and rafter members for 3-pin frames. The dimensions should be chosen so that the full panel width can be used.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

Parthenay, France, Stora Enso



Siuntio, Finland, Metsä Wood

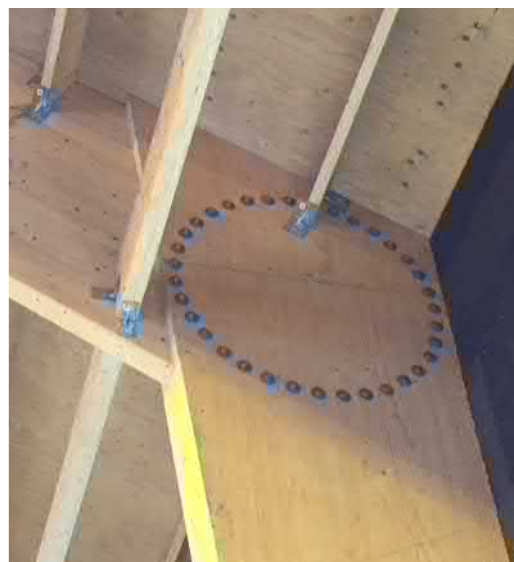


Figure 2.83. 3-pin portal frame structures for hall constructions with moment-resistant bolt circle corner connection.

2.5.2 Doors and windows

Small dimension LVL-P battens can be used to ensure the straightness and stability of door and window frames. LVL-C panels can form the core panels of fire and safety doors. LVL suppliers can provide special tolerances to ensure that the LVL components are suited to the production processes of the door and window industry.

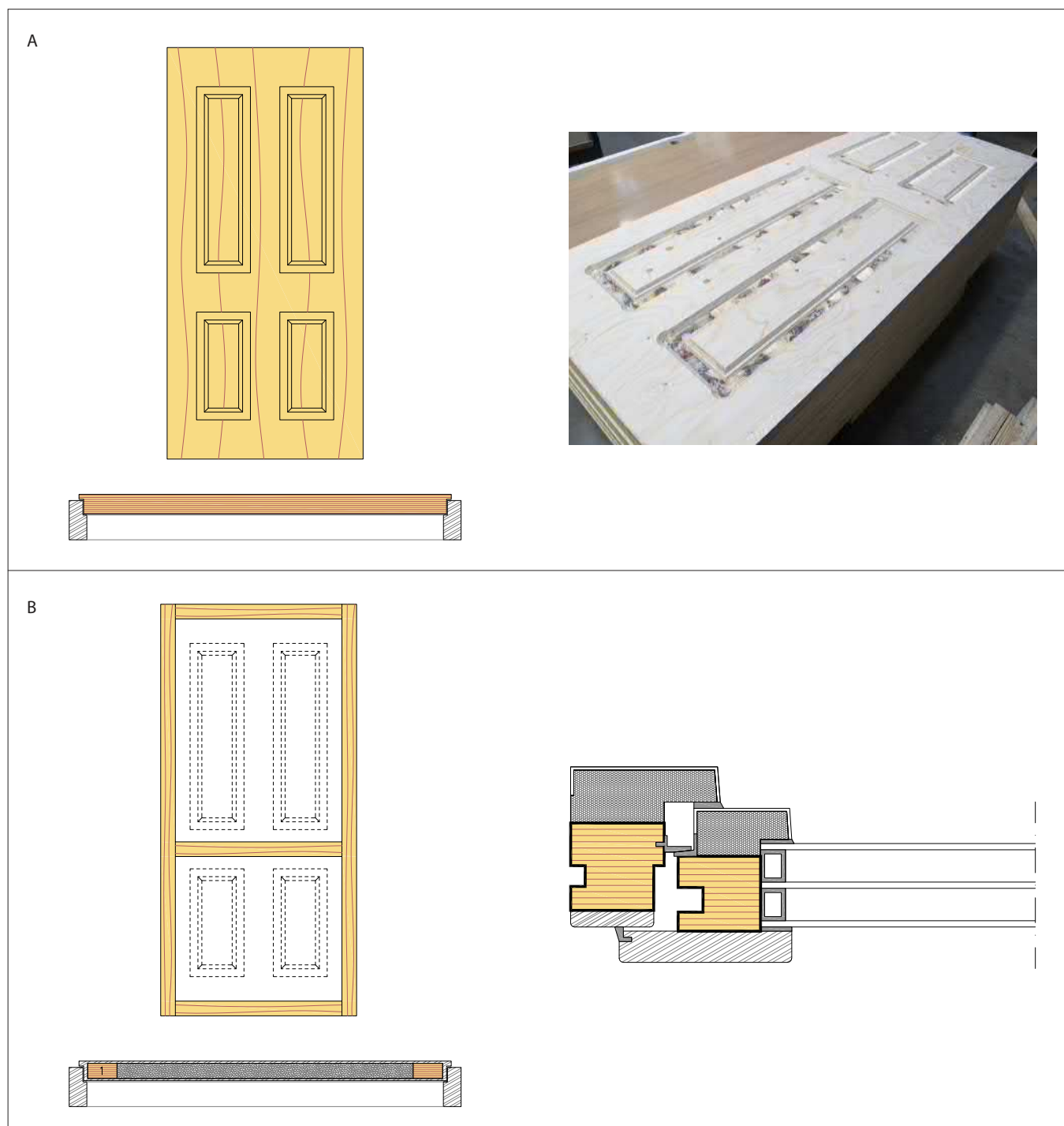


Figure 2.84. A) LVL-C core panel of fire and safety doors, B) LVL-P battens to ensure the straightness of door and window frames.

2. LVL STRUCTURES IN FLOORS, WALLS, ROOFS AND IN SPECIAL APPLICATIONS

2.5.3 Reinforcement of old structures

Old wooden structures can be reinforced by screwing or nailing LVL-C panels to their sides in areas subjected to high shear forces or tension perpendicular to the grain. The method is useful for notches, holes in beams, and for glulam beams with large cracks. Old joist floors can be reinforced by fastening LVL-P beams to the side or on top of them. Inclined screw connections have good stiffness and provide efficient joint action between the old joists and LVL-P reinforcement. In some cases the reinforcements can be glued to the old structures, but to fully utilize the advantages of rigid structural gluing the requirements for gluing conditions and quality control of the process must be met.

2.5.4 Stair cases

Large LVL-C panels can be used for the load-bearing structure of staircases, forming at the same time an architecturally interesting railing element. In smaller sizes, LVL boards are used as stair stringers which are covered with decorative panelling.

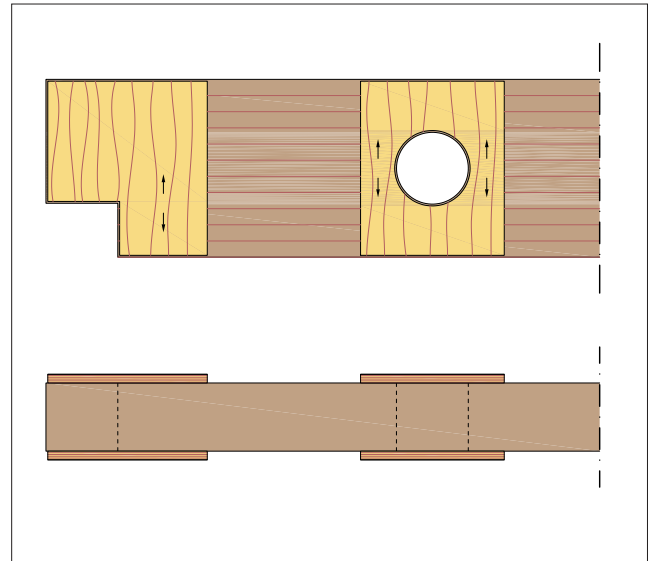


Figure 2.85. Reinforcing structures with LVL-C panels. Glulam beams with notches, holes, prevention of drying cracks.

Metsä Wood

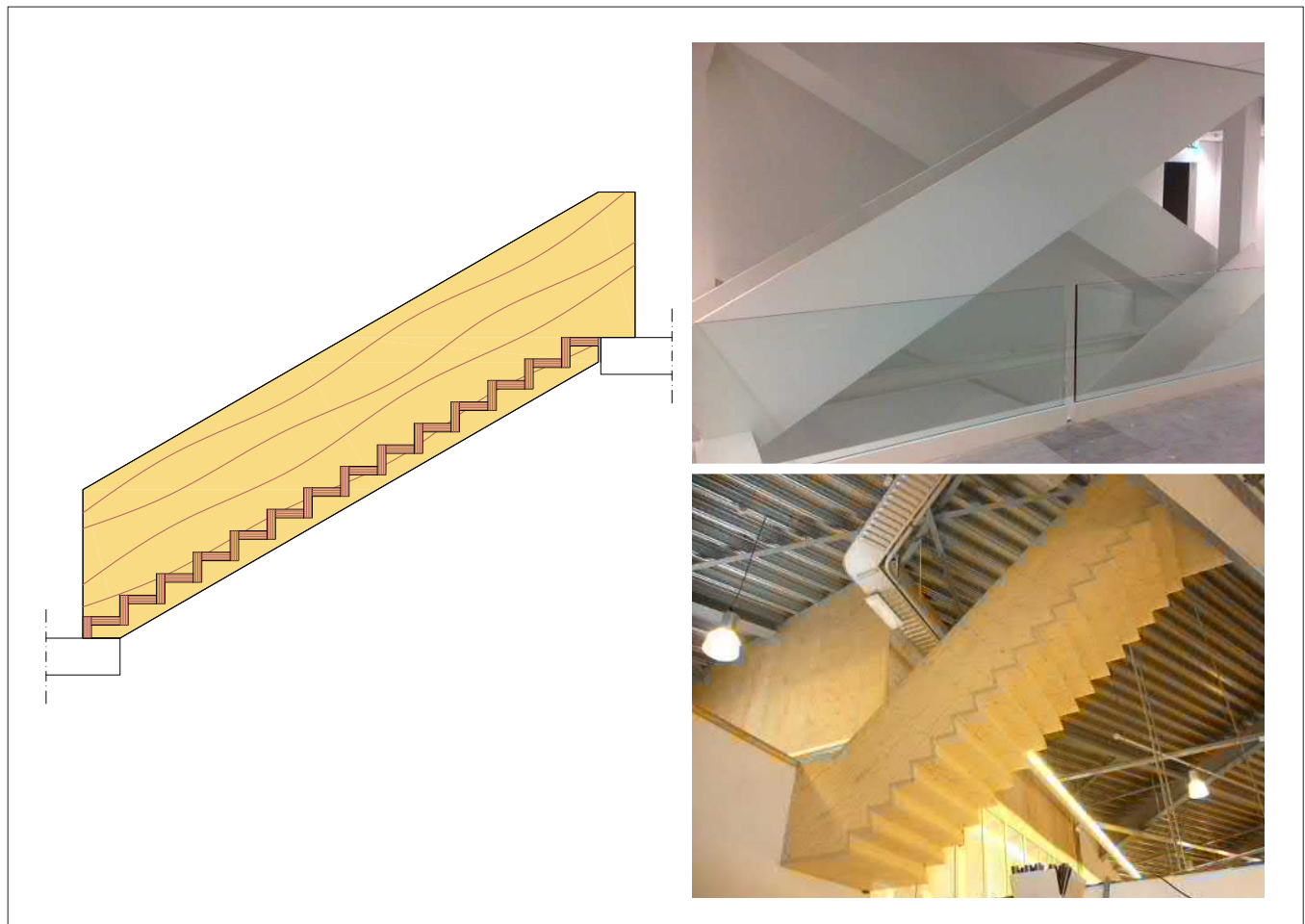


Figure 2.86. Staircase of LVL-C.

2.5.5 Lift shafts of multi-storey buildings

In wooden multi-storey buildings LVL-C panel structures can be used for lift shafts where they can also act as a part of the bracing system of the building. Panel sizes are suited for full building height or storey-high elements and the lifts structures can be installed to the shafts with conventional laterally loaded screw connections.

2.5.6 Extension of multi-storey buildings

With the ever increasing population shift to urban areas, an interesting possibility for creating additional housing is to build extensions on the roofs of existing buildings. To achieve this, the new structures must be light enough so that the existing structures can carry the load. Robust, but light LVL structures provide the ideal solution.



Figure 2.87. Light wooden extension floor to an existing multi-storey building from volumetric elements. Poissy, France.

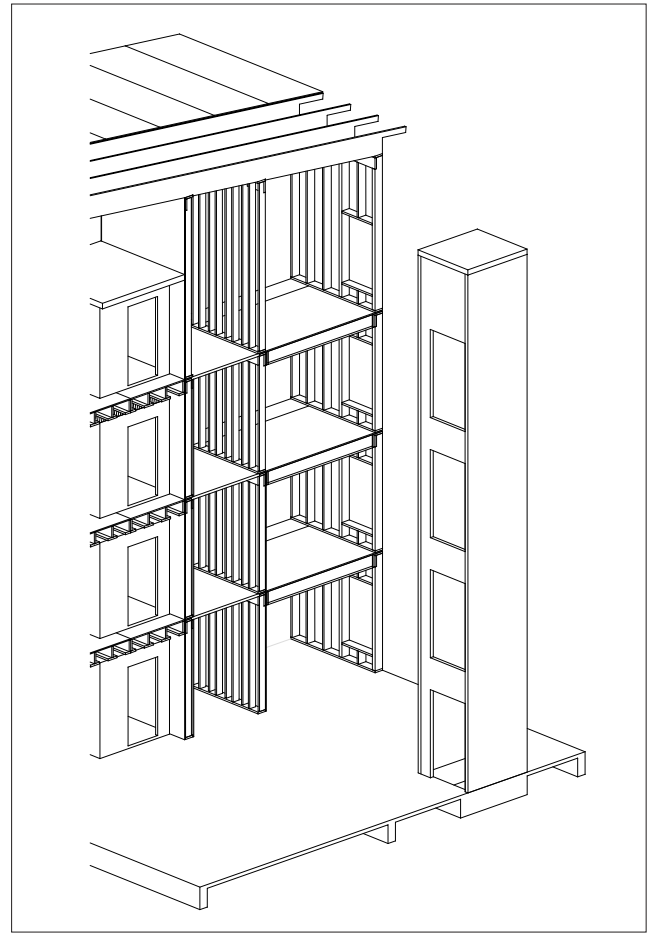


Figure 2.89. LVL-C panel structures for lift shafts. Panel sizes are suited for full building height or storey-high elements.



Figure 2.88. Tammelan Kruunu, Lisa Voigtländer & Sung Bok Song, 1st prize in 'City Above the City' architectural design competition, Metsä Wood ²⁶.

2.5.7 Bent and curved structures

LVL boards and panels can be bent in the flatwise direction to form curved structures when the bending and shear stress is taken into account in the design. Unless otherwise specified in more detail, LVL-P boards and LVL-C panels can be bent in the grain direction of the surface veneer according to the following specifications:

- Radius of curvature $R \geq 450 \times$ panel thickness
- Bending only in the grain direction of the surface veneer
- Unless otherwise specified more in detail, LVL-C panels can also be bent flatwise perpendicular to the grain direction of the surface veneer according to the following specifications:
 - Radius of curvature R perpendicular to surface veneer grain direction $\geq 350 \times$ panel thickness
 - Bending only perpendicular to the grain direction of surface veneer

As the cross veneers of LVL-C reinforce the product in the panel width direction, it can be cut to various shapes and used as the load-bearing structure of ambitious architectural designs or to create unique interiors. When production series are small and unique, CNC machining the shapes from panels is an economical solution. In structural design, strength properties at an angle to the grain must be taken into account according to the instructions given in subsections 4.3.6-7 or according to the LVL supplier's instructions.

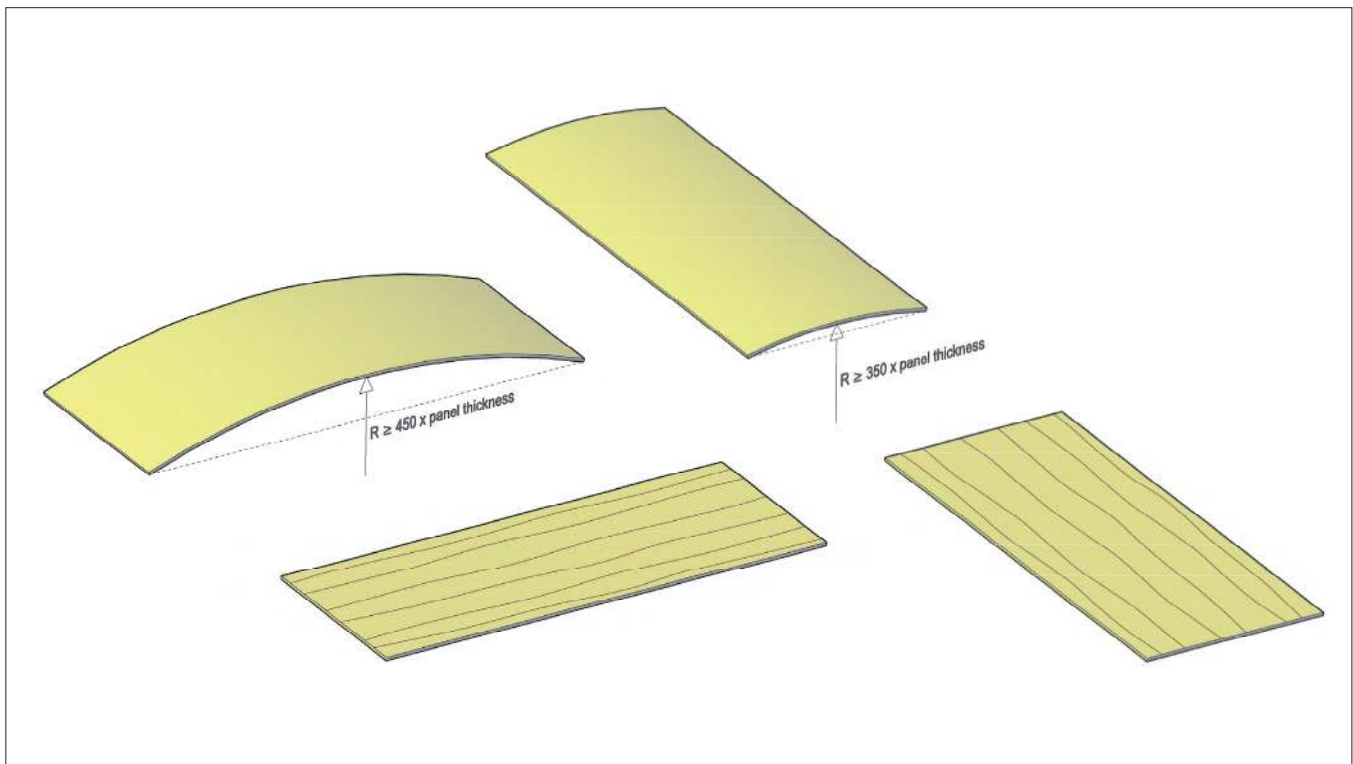


Figure 2.90. LVL panels bent flatwise in parallel to grain direction (left) and perpendicular to grain direction (right).



Figure 2.91. LVL-C arches for the roof structure of a windmill, UK. The structure consists of several curved members.

2.5.8 Bridges

Robust LVL-C panels can be used as pedestrian bridge decks supported on, e.g., glulam main beams. A 75 mm thick panel has the required resistance against maintenance vehicle wheel point loads and the bracing design of the bridge is straightforward using the solid panels. LVL-C panels allow up to 1.8 m spacing of main beams and secondary beams or, if the main beam spacing is small, secondary beams are not needed. Only two panels are needed for a typical 3.6 m bridge width. The deck needs a protective layer against water ingress on the top side, but impregnation against decay is not needed.

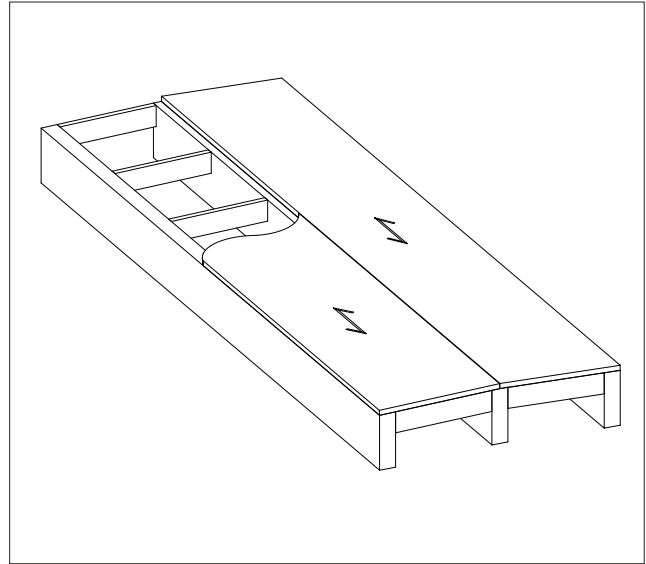


Figure 2.93. LVL-C deck parallel to the span of glulam beams of a pedestrian bridge.



Figure 2.92. 75mm LVL-C deck on a pedestrian bridge. Matinpuro Espoo, Finland.

2.5.9 Other applications

LVL-P is an essential component of the common engineered wood product, the I-joist. As LVL-P has a high and consistent E-modulus and is available in long lengths, it is well suited as flange stock material for the production process of I-joists for floor structures. I-joists are usually sold as a complete system containing the specific design and detailing for individual building projects and are CE marked according to the supplier's ETA assessment. Typical I-joist heights are 200-400 mm and flange widths 38-96 mm. In low-energy housing I-joists and studs are used also in exterior wall and roof structures.

LVL-P is commonly used as scaffolding planks, e.g. in the US and Middle East. They are proof loaded to ensure reliable and safe working on the construction site. LVL-P planks have a good working life and are light to carry and easy to install. One advantage of LVL scaffold planks is their high fire safety, as there is no risk of sparks if a metal member hits a plank, making them ideal for applications such as oil refineries and shipyards.

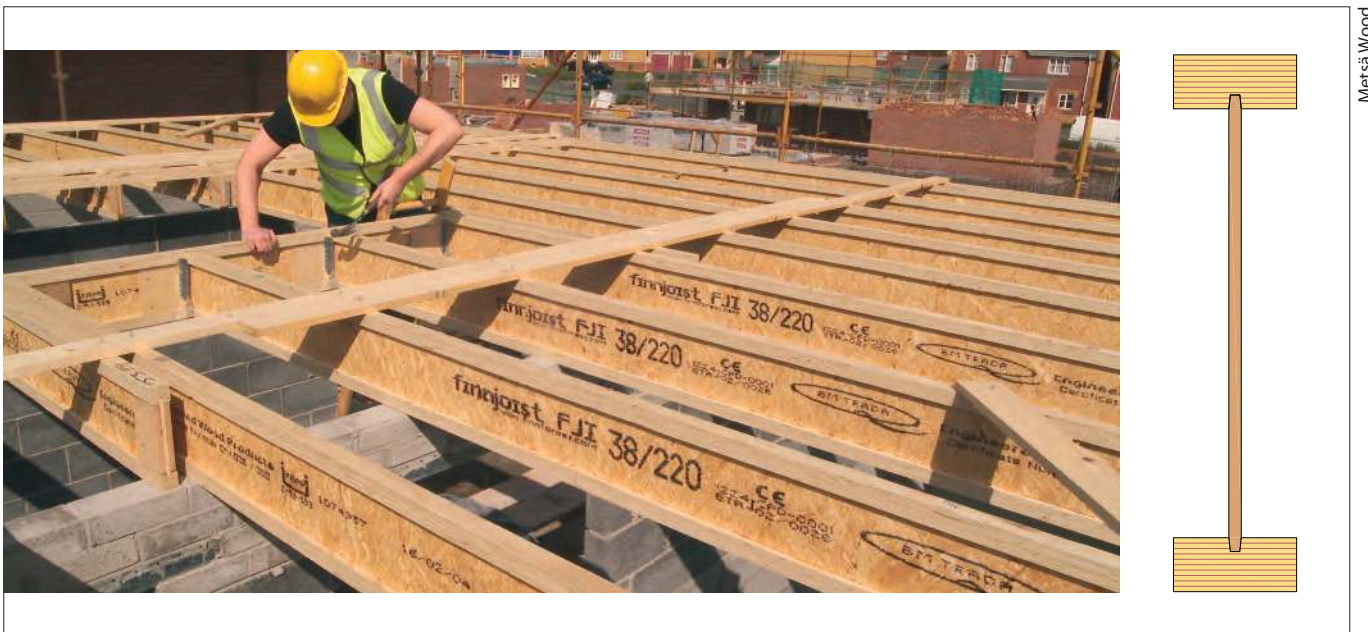


Figure 2.94. Wooden I-joist with LVL flanges.



Figure 2.95. Scaffolding planks from LVL-P.

3. Purchasing, transport, handling and storage of LVL

Stora Enso



Figure 3.1. Handling of LVL beams



Figure 3.2. Loading LVL packages with an overhead crane for truck transport.

3.1 PURCHASING LVL

Standard sizes of LVL-P beams and studs are readily available from local wholesalers or distributors' stocks, and in some countries LVL-C panels are also stocked. Larger quantities and special specifications are produced to order. LVL manufacturers have their own sales offices or agencies in most European countries.

The following information should be specified in a quotation request and order specification for LVL:

- Name of customer
- Delivery address
- Requested week of delivery to customer
- Invoicing address and VAT number of customer
- Product type (LVL-P or LVL-C and the grade)
- Cross section dimensions (thickness x width)
- Length
- Number of pieces of each product type and size
- Surface: unsanded, sanded and/or calibration
- Special tolerances, if non-standard
- Product labelling: with CE-mark stamp on each product or without stamping
- Machining drawing reference, if relevant
- Packaging:
 - Package size: max. weight or package height
 - Wrapping: LVL supplier will propose a suitable wrapping type, but customer specific requirements are taken into consideration
- Required certification (PEFC or FSC on request)
- Terms of delivery:
 - Road, rail or ship
 - For rail transport: if there are several delivery addresses for the same load, a loading order is required
- Availability of special surface qualities to be verified separately. As a default, all products have the same visual grade.

An individual production instruction is prepared for each order at the mill. Any additional information needed is obtained from the customer before production.

3.2 TRANSPORT, HANDLING AND STORING OF LVL

LVL products – as with all engineered wood products – must be handled and stored properly and carefully to protect them from damage and dirt. Incorrect handling and storage may damage or blemish the surfaces, edges or corners or reduce the dimensional stability of the product.

Transport

Exposure to moisture, e.g. from rain or splashing, must be avoided during transportation and storage of the product. If moving the product with a forklift truck, wide enough forks must be used to avoid damage. When lifting several packs at a time, the distance between forks must be wide enough to ensure safe lifting. Surface-treated products should be delivered direct to site without additional unnecessary off-loading during delivery ^{2, 27}.



Figure 3.3. Use wide enough forks when moving LVL packages with forklift to avoid damage.

3. PURCHASING, TRANSPORT, HANDLING AND STORAGE OF LVL



Figure 3.4. Storage of LVL packages



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Storage

LVL products must be stored under cover. When storing the products temporarily on site, a solid, straight and dry platform should be used. The height of ground bearers (skids) must be at least 30 cm. To avoid twisting or cracking of the product, the bearers between packs must be aligned vertically with the ground bears. Bearers must be of a suitable size and number and spread evenly.

The plastic wrapping of each pack must be cut open from underneath to enable air circulation and moisture to evaporate from the bundles. If the products are stored on site for longer than one week, the bundles must be covered with an additional protective covering. The condition of the product and the protective cover should be monitored regularly during storage^{2,27}.

Handling and processing

LVL product packs can be unloaded on site with either a forklift or a crane. When unloading with a forklift, follow the instructions given in the 'Transport' section. When unloading by crane, approved webbing slings in proper condition and of the appropriate strength class must be used. Do not use chains or wires. If unloading is done manually, open the pack and carry the products one-by-one. When cutting the banding, beware of the band ends. LVL products should not be dragged, pushed or dropped.

LVL is a light-weight material and is easy to shape, which means notable time and cost savings in construction. LVL products can be processed with conventional wood working and power tools. There is no need for separate specialist tools. Surface-treated products should be unloaded individually. If needed, a cellular plastic padding that does not stain, should be used under the webbing slings^{2,27}.



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Figure 3.5. Handling of LVL beams

3.3 PROTECTING THE STRUCTURE DURING CONSTRUCTION

LVL structures must be protected on construction site to avoid dimensional changes and surface mould due to exposure to moisture. Exposure to rain, splashing, and wetting from water convection from other structures must be avoided. In addition, the designer must ensure in the detailed design that no water pockets can form on the product in situ.

The product may be exposed to weather for a short period during installation. During the erection of the building, structural LVL products and elements, which are structurally glued from LVL components, have good resistance to temporary ex-

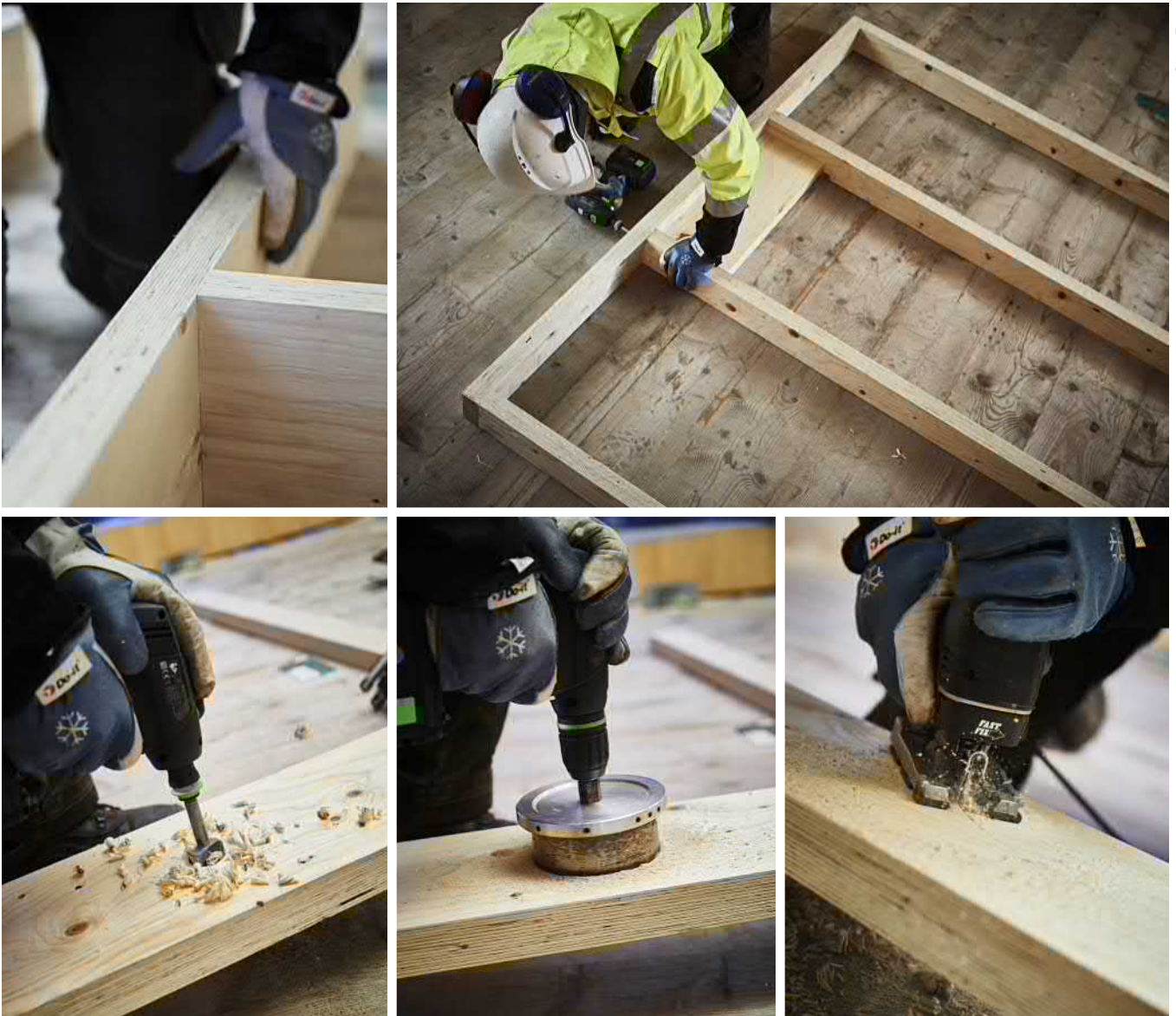


Figure 3.6. LVL structures can be processed with conventional wood working and power tools.

posure to water without damage or decay, provided that it is ensured that the products can afterwards dry to the desired moisture content before the structures are closed. The integrity of the adhesive bonds is maintained according to the designated service class throughout the expected life of the structure ²⁸. Further information on durability is presented in Chapter 7.

3.4 HANDLING AFTER USE OF LVL

After use, LVL products are to be disposed of according to national regulations and directives. In general, LVL products can be reused or recycled as the preferred handling options; alternatively, they can be composted or burned for energy recovery. LVL can be safely burned when the combustion temperature

is at least 850°C; the combustion air and gases are well mixed; the retention time of the combustion gases in the furnace is over two seconds; and the residual oxygen content of the flue gases is over 6%. In these combustion conditions the flue gases are identical to the gases produced in burning untreated wood. LVL has a gross heating value of 19.4 MJ/kg

Composting requires the panels to be chipped and the long composting process should also be taken into consideration. The products can also be taken to landfill, although LVL degrades very slowly ³.

LVL does not contain anything classified as hazardous waste and has the following waste code under the consolidated European Waste Catalogue:

- 17 02 01 Wood (Construction and Demolition Wastes)

4. Structural design of LVL structures

Metsä Wood



Figure 1.1 LVL roof structure, K-Rajamarket, Utsjoki, Finland.

The structural design of LVL structures is similar in purpose and principle to any load-bearing wood-based structure: to verify that the structure fulfils its strength, serviceability and other structural requirements. The basis of limit state design is to verify, based on the partial safety coefficient method, that the design value of an action E_d is smaller than the design value of the resistance R_d of a section, structural member or connection:

$$E_d \leq R_d$$

where

E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments

R_d is the design value of the corresponding resistance.

4.1 BASIS OF STRUCTURAL DESIGN

The Eurocode design system has been used for this purpose in Europe since 2010 with country-specific adjustment factors that are defined in national annexes. For timber construction the important parts of the Eurocode system are:

EN 1990 Eurocode 0	Basis of structural design
EN 1991 Eurocode 1	Actions on structures
EN 1993 Eurocode 3	Design of steel structures
EN 1995 Eurocode 5	Design of timber structures

Eurocodes are limit state design codes that have two main limit states: Ultimate limit state (ULS) and serviceability limit state (SLS). In ultimate limit state (ULS) design the requirement is to verify that a structure has adequate safety against failure during its whole designed service life. What is regarded adequate is defined in the relevant building regulations. Serviceability limit state (SLS) design evaluates whether the structure is fit for purpose. In most cases, the building regulations do not define exact limit values for this evaluation. They provide recommendations for the maximum level of deformation and, e.g., human-induced floor vibration, but it is ultimately up to the contractor and the client to agree what is acceptable.

The risk of structural failure is depending on the probability that the expected actions are exceeded and the probability that resistance of the structure is lower than calculated in ULS design. Normally it can be presumed that the actions and the resistance of a structure are random variables. When their distribution functions are known, it is possible to calculate the risk of failure by the methods of probability theories. These methods, however are usually too complicated for structural

design in practice, but they can be used for code calibration when the requirements are defined for building regulations and in comparison between different construction materials. They also have an important role e.g. in verification of the safety level of simplified design methods, such as the partial safety factor method used in Eurocodes ^{29, 30}.

The primary purpose of the design rules for load-bearing structures is to prevent failures that can lead to the risk of human injury and to ensure buildings function properly according to their intended use.

Building codes such as the Eurocodes provide approved verification methods for verifying that the requirements are fulfilled. They also present the loads and loading combinations that the structures must resist. The Eurocodes have common parts that are applied in all countries in connection with national annexes. The annexes specify country-specific safety factors and parameters that take into consideration climate and geology specific conditions. In addition, national choices are made in some alternative verification methods or adjustment factors within them.

The general design requirements according to the Eurocodes are ^{29, 30}:

- Choice of structural system, structural design and construction work must be done by sufficiently qualified and experienced persons.
- Work must be supervised adequately and quality assurance implemented throughout the construction process from design offices and factories to workshops and building sites.
- Structures must be made of construction materials and products defined in the Eurocodes or their harmonized standards or in other harmonized technical specifications.
- Buildings must be adequately and regularly maintained throughout their design working life.
- Buildings must be used for the purpose for which they have been designed

The next generation of Eurocodes is currently under development and scheduled to be ready for use after some years.

4.1.1 Actions (Loads)

Actions are loads that cause, e.g., bending moment, shear and axial stresses and deformation of the structures. The determining actions and their combinations for each structure are evaluated based on the type, magnitude and duration of the action at the most unfavourable locations of the structure. Different load combinations are determined as load cases. The load combinations consist of the main loads combined with reduced values of other loads that may be acting at the same time. Reduction

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Table 4.1. Load-duration classes and examples of loads ³¹.

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading	Notes
Permanent	More than 10 years	<ul style="list-style-type: none"> • Self-weight • Permanently installed machinery • Compartment walls in some countries 	
Long-term	6 months – 10 years	<ul style="list-style-type: none"> • Long-term storage 	
Medium-term	1 week – 6 months	<ul style="list-style-type: none"> • Medium-term imposed floor load • Snow 	Snow in Finland
Short-term	Less than one week	<ul style="list-style-type: none"> • Short-term snow • Wind • Staircase loads • Imposed point loads • Service maintenance loads on roofs 	Snow in several countries Wind in several countries
Instantaneous		<ul style="list-style-type: none"> • Instantaneous wind • Accidental load 	Wind in Finland

in load is defined by multiplying the characteristic value of a load Q_k by factor ψ_0 , ψ_1 or ψ_2 depending on the case:

- Characteristic combination ($\psi_0 Q_k$) is used for verification of ultimate limit states and for irreversible (permanent) deformations of a structure in serviceability limit states
- Frequent combination ($\psi_1 Q_k$) is used for verification of ultimate limit states involving accidental actions and for verification of reversible deformations of a structure in serviceability limit states
- Quasi-permanent combination ($\psi_2 Q_k$) is used for verification of ultimate limit states involving accidental actions and for verification of reversible serviceability limit states. Quasi-permanent values are also used for the calculation of long-term effects.

Factor ψ_2 can be concluded as a factor that converts short-term loads to permanent loads which have a similar long-term influence in the calculation of creep deformations.

Loads are defined in EN 1991 and the load combinations in EN 1990. The rules in these standards define how permanent actions and variable actions shall be taken into consideration in load combinations. The general equation of load combination in the ultimate limit state is:

$$E_d = \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (4.1)$$

where

- $\gamma_{G,j}$ = partial safety factor for permanent actions j ;
- $G_{k,j}$ = characteristic value of permanent load j ;
- $\gamma_{Q,1}$ = partial safety factor for decisive variable action 1;
- $Q_{k,1}$ = characteristic value of decisive variable load 1;
- $\gamma_{Q,i}$ = partial safety factor for variable action i ;
- $Q_{k,i}$ = characteristic value of variable load i ; and
- $\psi_{0,i}$ = reduction factor in load combination for variable action i .

The values of γ_G and γ_Q are set in national annexes, but in common ULS design cases for unfavourable actions $\gamma_G = 1,15 - 1,35$ and $\gamma_Q = 1,5 - 1,6$. In SLS design γ_G and γ_Q are 1,0.

4.1.2 Consequence class, reliability class and factor K_{FI}

For the purpose of reliability differentiation, consequences classes (CC1-CC3) may be established by considering the consequences of a failure or malfunction of the structure. Class CC1 is used for low consequence for loss of human life or when economic, social or environmental consequences are small or negligible. For example, agricultural buildings and storage houses may belong to class CC1. CC2 is a normal class with a medium consequence level and is used as a default class for residential and office buildings. CC3 is used for buildings where the consequences of failure are high such as concert halls or similar monumental structures.

The requirements for different consequence classes are set in the associated reliability classes RC1-RC3. They include requirements for the level of reliability index β , the supervision of design and execution of the structures and resistance properties of materials and products. In partial safety factor design the reliability classes are taken into account by the K_{FI} factor for actions. The values of K_{FI} are given in national annexes, but according to the default values of EN1990, in RC1 the actions in equation (4.1) of the ULS are multiplied by $K_{FI} = 0,9$, in RC2 by $K_{FI} = 1,0$ and in RC3 by $K_{FI} = 1,1$. K_{FI} is not used in SLS.

4.1.3 Load duration

In the Eurocodes, load-duration classes are characterized by the effect of a constant load acting for a certain period of time in the life of the structure. For a variable action, the appropriate class is determined on the basis of an estimate of the typical variation of the load with time, see Table 4.1.

4.1.4 Service classes

Moisture content and changing humidity conditions have a significant influence on the strength and stiffness properties of LVL as well as on other wood-based materials. In Eurocode 5 this is taken into account by defining 3 service classes:

- **Service class 1 (SC1)** is characterized by a moisture content of the materials corresponding to a temperature of 20 °C and the relative humidity of the surrounding air only exceeding 65% for a few weeks per year. This corresponds typically to heated indoor air conditions. In service class 1 the average moisture content (MC) of softwood LVL is usually between 6 and 10%. The MC of most solid woods is in those conditions typically a couple of % higher, but will not exceed 12%. The manufacturing process of LVL keeps the product dryer because the veneers are dried at high temperature to less than 5% MC, which alters the wood cell structure making the material less moisture absorbent.

- **Service class 2 (SC2)** is characterized by a moisture content of the materials corresponding to a temperature of 20 °C and the relative humidity of the surrounding air only exceeding 85% for a few weeks per year. This corresponds typically to ventilated outdoor conditions under a roof protecting from direct weather exposure. In service class 2 the average moisture content of softwood LVL is usually between 10 and 16%. The MC of most softwoods will not exceed 20% in SC2.
- **Service class 3 (SC3)** is characterized by climatic conditions leading to higher moisture contents than in service class 2. This corresponds typically to conditions where the structures are under direct weather exposure, high humidity or direct contact with water. LVL cannot be used in service class 3 without preservative treatment against decay.

Standard EN 335 defines use classes with respect to durability. These correspond to the service classes of Eurocode 5 with the exception of service class 3, which is divided into sub-classes UC3.1, 3.2, 4 and 5, which describe the conditions more precisely.

When choosing the service class for the design, in addition to the moisture content of the wood material, special attention must be paid to cyclic changes in the moisture conditions, which may have a more significant effect on timber constructions than higher constant humidity conditions. In service class 1 special attention must be paid to the risk of cracking of timber structures.

4.1.5 Partial safety factor of material γ_M and modification factors k_{mod} and k_{def}

The partial safety factor γ_M of different materials are defined in the National annex of Eurocode 5. For LVL it is typically $\gamma_M = 1,2 - 1,3$.

k_{mod} is a modification factor taking into account the effect of the duration of load and moisture content. In Eurocode 5 the same values of k_{mod} are used for all mechanical properties of LVL in service class 1 and 2. However, experimental studies have shown that the compression strength $f_{c,0,k}$ is lower in SC2 than in SC1. This is taken into consideration in the strength classes by defining a 20% lower strength value of $f_{c,0,k}$ for SC2 15.

If a load combination consists of actions belonging to different load-duration classes a k_{mod} value that corresponds to the action with the shortest duration should be chosen. For example, for a combination of dead load and a short-term load, a value of k_{mod} corresponding to the short-term load should be used.

In serviceability limit state design the influence of creep is dependent on the service class and the duration class of the loads. ψ_0 and ψ_2 factors for the quasi-permanent value of the action take this into account in load combinations. The deformation factors k_{def} take into account the service classes. For characteristic combination of actions, the final deflection including creep is calculated using equation (4.2).

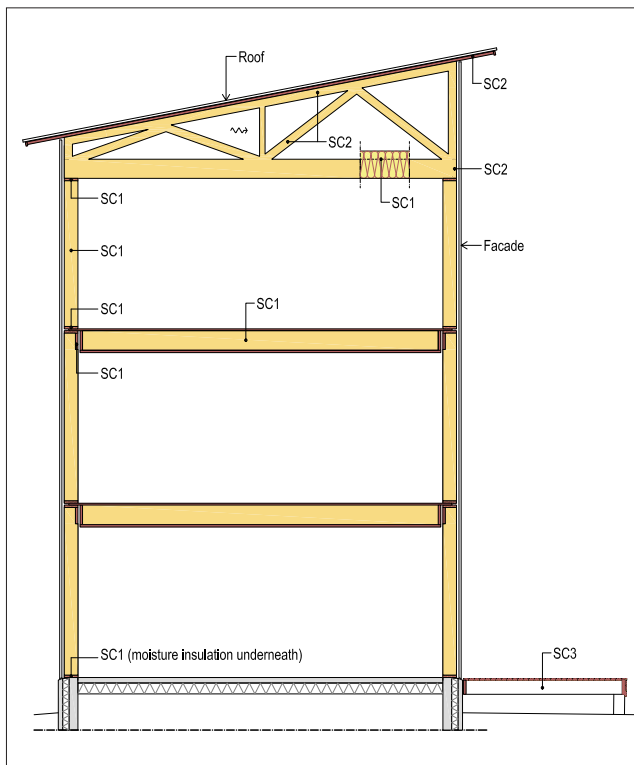


Figure 4.2. Examples of service class 1 (SC1), 2 (SC2) and 3 (SC3) conditions in wooden structures. Note: Assignment of structures to service classes are given in the National Annex to Eurocode 5 for each country.

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Table 4.2. Values of k_{mod} for LVL (the same values are used for structural timber, glulam and CLT and plywood).

Service class	Permanent	Long-term	Medium-term	Short-term	Instantaneous
1	0,60	0,70	0,80	0,90	1,10
2	0,60	0,70	0,80	0,90	1,10
3	0,50	0,55	0,65	0,70	0,90

Table 4.3. Values of k_{def} for different LVL types in different service classes.

Product type	Service class 1	Service class 2	Service class 3
LVL-P	0,60	0,80	2,00
LVL-C in edgewise loading	0,60	0,80	2,00
LVL-C in flatwise loading	0,80	1,00	2,50

$$u_{\text{fin}} = u_{\text{inst,G}}(1 + k_{\text{def}}) + u_{\text{inst,Q,1}}(1 + \psi_{2,1} \cdot k_{\text{def}}) + u_{\text{inst,Q,i}}(\psi_{0,i} + \psi_{2,i} \cdot k_{\text{def}}) \quad (4.2)$$

where

u_{fin} is the final deflection including creep deformation;

$u_{\text{inst,G}}$ is the instantaneous deflection due to permanent actions;

$u_{\text{inst,Q,1}}$ is the instantaneous deflection due to the leading variable action; and

$u_{\text{inst,Q,i}}$ is the instantaneous deflection due to accompanying variable actions.

LVL-C has a higher k_{def} value when the loading causes deformation in the flatwise direction due to rolling shear deformation of the cross veneers similar to plywood. When LVL-C panel is used as a component of stressed-skin panels, the k_{def} value in the longitudinal direction of the element is the same as for LVL-P, because the loading causes mainly axial stresses on the panel.

4.1.6 Design resistance and stiffness

Design resistance in ULS design is determined from the design values of the strength properties by modifying the characteristic properties with partial safety factor γ_M and modification factor k_{mod} .

$$f_d = \frac{k_{\text{mod}} \cdot f_k}{\gamma_M} \quad (4.3) \text{ (EC5 2.17)}$$

where

f_k is the characteristic 5% value of a strength property;

k_{mod} is the modification factor that takes into account the duration of load and service class, see Table 4.2; and

γ_M is the partial safety factor of the material.

Mean values for stiffness properties are used in SLS design and are determined for short-term loading in service class 1 conditions. The influence of creep deformation is taken into account by the k_{def} factor. Characteristic 5% stiffness values are used for stability calculation in ULS design.

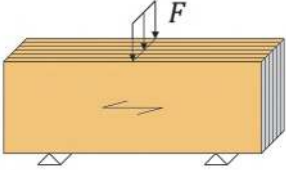
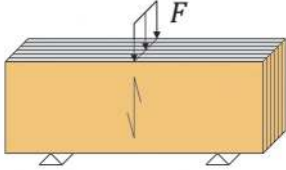
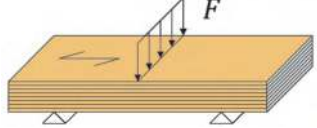
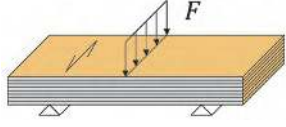

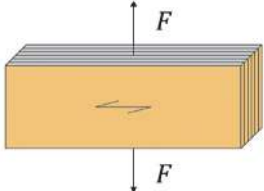
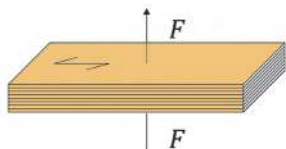

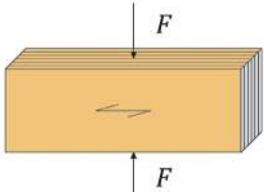
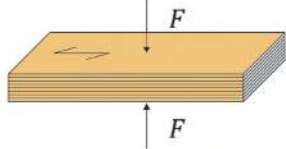
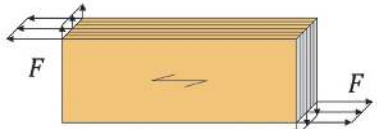

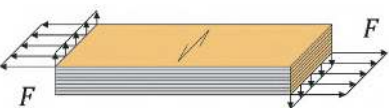
4.2 STRUCTURAL PROPERTIES OF LVL AND STRENGTH CLASSES

The mechanical properties of structural LVL are determined according to the harmonized product standard EN 14374. The properties are assessed and their constancy of performance verified according to the AVPC system 1 of the EU construction product regulations. LVL suppliers declare their individual product properties in their Declaration of Performance (DoPs).

In the future EN 14374 LVL product categories will be introduced but in the meanwhile LVL industry has decided to launch them as LVL strength classes. Information about the strength classes can be found from the Laminated Veneer Lumber (LVL) bulletin: New European strength classes ¹⁵, FprEN 14374 Annex B, and these are described in the following subsections 4.2.1 and 4.2.2.

The bending and axial strength and stiffness properties of LVL-C with crossband veneers having different layups (i.e. veneer orientation) may be calculated from one set of tests according to FprEN 14374 Annex A by applying layup factors according to Annex C for thickness ranges specified in Annex A. LVL-C properties are defined with the assumption that the cross veneers are zero layers.

Table 4.4. Symbols for strengths, moduli of elasticity and shear moduli in different directions of LVL ¹⁵

 <p>$f_{m,0,edge}$, s and $E_{m,0,edge}$</p>	 <p>$f_{m,90,edge}$, $E_{m,90,edge}$ and $f_{v,90,edge}$</p>	 <p>$f_{m,0,flat}$, $s_{flat,m}$ and $E_{m,0,flat}$</p>
 <p>$f_{m,90,flat}$ and $E_{m,90,flat}$</p>	 <p>$f_{t,0}$ and $E_{t,0}$</p>	 <p>$f_{t,90,edge}$</p>
 <p>$f_{t,90,flat}$</p>	 <p>$f_{c,0}$ and $E_{c,0}$</p>	 <p>$f_{c,90,edge}$ and $E_{c,90,edge}$</p>
 <p>$f_{t,90,flat}$ and $E_{m,90,flat}$</p>	 <p>$f_{v,0,edge}$ and $G_{0,edge}$</p>	 <p>$f_{v,0,flat}$, $s_{flat,v}$ and $G_{0,flat}$</p>
 <p>$f_{t,90,flat}$ and $G_{90,flat}$</p>		

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4.2.1 Strength classes for LVL-P without crossband veneers

For structural LVL made of spruce or pine the most relevant class is LVL 48 P for beam applications. LVL 32 P is suitable for stud applications where mechanical property requirements are lower. LVL 80 P is made from beech hardwood. The symbols of properties related to different directions of LVL are described in Table 4.4.

4.2.2 Strength classes for LVL-C with crossband veneers

For structural LVL made of spruce or pine the most relevant class is LVL 36 C for load-bearing panels. LVL 25 C is suitable for panel applications where mechanical property requirements are lighter. LVL 70 C and LVL 75 C are made from beech hardwood. The symbols of properties related to different directions of LVL are described in Table 4.4.

Table 4.5. Strength classes for structural LVL-P without crossband veneers ¹⁵.

Property ^a		Symbol	Unit	Strenght class				
				LVL 32 P	LVL 35 P	LVL 48 P	LVL 50 P	LVL 80 P
Bending strength	Edgewise, parallel to grain (depth 300 mm)	$f_{m,0,edge,k}$	N/mm ²	27	30	44	46	75
	Flatwise, parallel to grain	$f_{m,0,flat,k}$	N/mm ²	32	35	48	50	80
	Size effect parameter	s	-	0,15	0,15	0,15	0,15	0,15
Tension strength	Parallel to grain (length 3 000 mm)	$f_{t,0,k}$	N/mm ²	22	22	35	36	60
	Perpendicular to grain, edgewise	$f_{t,90,edge,k}$	N/mm ²	0,5	0,5	0,8	0,9	1,5
Compression strength	Parallel to grain for service class 1	$f_{c,0,k}$	N/mm ²	26	30	35	42	69
	For service class 2 according to EN 1995-1-1 ^b			21	25	29	35	57
	Perpendicular to grain, edgewise	$f_{c,90,edge,k}$	N/mm ²	4	6	6	8,5	14
	Perpendicular to grain, flatwise (except pine)	$f_{c,90,flat,k}$	N/mm ²	0,8	2,2	2,2	3,5	12
	Perpendicular to grain, flatwise, pine	$f_{c,90,flat,k,pine}$	N/mm ²	MDV ^c	3,3	3,3	3,5	- ^d
Shear strength	Edgewise parallel to grain	$f_{v,0,edge,k}$	N/mm ²	3,2	3,2	4,2	4,8	8
	Flatwise, parallel to grain	$f_{v,0,flat,k}$	N/mm ²	2,0	2,3	2,3	3,2	8
Modulus of elasticity	Parallel to grain	$E_{0,mean}$ ^e	N/mm ²	9 600	12 000	13 800	15 200	16 800
	Parallel to grain	$E_{0,k}$ ^f	N/mm ²	8 000	10 000	11 600	12 600	14 900
	Perpendicular to grain, edgewise	$E_{c,90,edge,mean}$ ^g	N/mm ²	MDV ^c	MDV ^c	430	430	470
	Perpendicular to grain, edgewise	$E_{c,90,edge,k}$ ^h	N/mm ²	MDV ^c	MDV ^c	350	350	400
Shear modulus	Edgewise, parallel to grain	$G_{0,edge,mean}$	N/mm ²	500 ⁱ	500 ⁱ	600	650	760
	Edgewise, parallel to grain	$G_{0,edge,k}$	N/mm ²	300 ⁱ	350 ⁱ	400	450	630
	Flatwise, parallel to grain	$G_{0,flat,mean}$	N/mm ²	320 ⁱ	380 ⁱ	380	600	850
	Flatwise, parallel to grain	$G_{0,flat,k}$	N/mm ²	240 ⁱ	270 ⁱ	270	400	760
Density		ρ_{mean}	kg/m ³	440	510	510	580	800
		ρ_k	kg/m ³	410	480	480	550	730

^a Additional strength, stiffness and density properties not covered by the classes given in this Table may be declared as individual values

^b Value may also be applied in Service Class 1 as a conservative value

^c Property is not expressed as strength class but rather as individual manufacturer's declared value (MDV)

^d Class not produced from pine

^e Covering $E_{m,0,edge,mean}$, $E_{t,0,mean}$, $E_{m,0,flat,mean}$, and $E_{c,0,mean}$

^f Covering $E_{m,0,edge,k}$, $E_{t,0,k}$, $E_{m,0,flat,k}$, and $E_{c,0,k}$

^g Also covering $E_{t,90,edge,mean}$

^h Also covering $E_{t,90,edge,k}$

ⁱ Property need not be tested if all other properties meet the minimum values for the strength class

Table 4.6. Strength classes for structural LVL-C with crossband veneers ¹⁵.

				Strenght class					
Property ^a		Symbol	Unit	LVL 22 C	LVL 25 C	LVL 32 C	LVL 36 C	LVL 70 C	LVL 75 C
Bending strength	Edgewise, parallel to grain (depth 300 mm)	$f_{m,0,edge,k}$	N/mm ²	19	20	28	32	54	60
	Flatwise, parallel to grain	$f_{m,0,flat,k}$	N/mm ²	22	25	32	36	70	75
	Size effect parameter	s	-	0,15	0,15	0,15	0,15	0,15	0,15
	Flatwise, perpendicular to grain	$f_{m,90,flat,k}$	N/mm ²	MDV ^c	MDV ^c	7	8	32	20
Tension strength	Parallel to grain (length 3 000 mm)	$f_{t,0,k}$	N/mm ²	14	15	18	22	45	51
	Perpendicular to grain, edgewise	$f_{t,90,edge,k}$	N/mm ²	4	4	5	5	16	8
Compression strength	Parallel to grain for service class 1	$f_{c,0,k}$	N/mm ²	18	18	18	26	54	64
	For service class 2 according to EN 1995-1-1 ^b			15	15	15	21	45	53
	Perpendicular to grain, edgewise	$f_{c,90,edge,k}$	N/mm ²	8	8	9	9	45	23
	Perpendicular to grain, flatwise (except pine)	$f_{c,90,flat,k}$	N/mm ²	1,0	1,0	2,2	2,2	16	16
	Perpendicular to grain, flatwise, pine	$f_{c,90,flat,k,pine}$	N/mm ²	MDV ^c	MDV ^c	3,5	3,5	- ^d	- ^d
Shear strength	Edgewise parallel to grain	$f_{v,0,edge,k}$	N/mm ²	3,6	3,6	4,5	4,5	7,8	7,8
	Flatwise, parallel to grain	$f_{v,0,flat,k}$	N/mm ²	1,1	1,1	1,3	1,3	3,8	3,8
	Flatwise, perpendicular to grain	$f_{v,90,flat,k}$	N/mm ²	MDV ^c	MDV ^c	0,6	0,6	MDV ^c	MDV ^c
Modulus of elasticity	Parallel to grain, edgewise	$E_{0,edge,mean}$ ^e	N/mm ²	6 700	7 200	10 000	10 500	11 800	13200
	Parallel to grain, edgewise	$E_{0,edge,k}$ ^f	N/mm ²	5 500	6 000	8 300	8 800	10 900	12200
	Perpendicular to grain, edgewise	$E_{90,edge,mean}$ ^g	N/mm ²	MDV ^c	MDV ^c	2 400	2 400	MDV ^c	MDV ^c
	Perpendicular to grain, edgewise	$E_{90,edge,k}$ ^h	N/mm ²	MDV ^c	MDV ^c	2 000	2 000	MDV ^c	MDV ^c
	Perpendicular to grain, flatwise	$E_{m,90,flat,mean}$	N/mm ²	MDV ^c	MDV ^c	1 200	2 000	MDV ^c	MDV ^c
	Perpendicular to grain, flatwise	$E_{m,90,flat,k}$	N/mm ²	MDV ^c	MDV ^c	1 000	1 700	MDV ^c	MDV ^c
Shear modulus	Edgewise, parallel to grain	$G_{0,edge,mean}$	N/mm ²	500 ⁱ	500 ⁱ	600	600	820	820
	Edgewise, parallel to grain	$G_{0,edge,k}$	N/mm ²	300 ⁱ	300 ⁱ	400	400	660	660
	Flatwise, parallel to grain	$G_{0,flat,mean}$	N/mm ²	70 ⁱ	70 ⁱ	80	120	430	430
	Flatwise, parallel to grain	$G_{0,flat,k}$	N/mm ²	55 ⁱ	55 ⁱ	60	100	380	380
	Flatwise, perpendicular to grain	$G_{90,flat,mean}$	N/mm ²	MDV ^c	MDV ^c	22	22	MDV ^c	MDV ^c
	Flatwise, perpendicular to grain	$G_{90,flat,k}$	N/mm ²	MDV ^c	MDV ^c	16	16	MDV ^c	MDV ^c
Density		ρ_{mean}	kg/m ³	440	440	510	510	800	800
		ρ_k	kg/m ³	410	410	480	480	730	730

^a Additional strength, stiffness and density properties not covered by the classes may be declared as individual values^b Value may also be applied in Service Class 1 as a conservative value^c Property is not expressed as strength class but rather as individual manufacturer's declared value (MDV)^d Class not produced from pine^e Covering $E_{m,0,edge,mean}$, $E_{t,0,mean}$, $E_{m,0,flat,mean}$, and $E_{c,0,mean}$ ^f Covering $E_{m,0,edge,k}$, $E_{t,0,k}$, $E_{m,0,flat,k}$, and $E_{c,0,k}$ ^g Covering $E_{m,90,edge,mean}$, $E_{t,90,edge,mean}$ and $E_{c,90,edge,mean}$ ^h Covering $E_{m,90,edge,k}$, $E_{t,90,edge,k}$ and $E_{c,90,edge,k}$ ⁱ Property need not be tested if all other properties meet the minimum values for the strength class

4.3 LVL DESIGN ACCORDING TO EUROCODE 5

In EN 1995-1-1 (Eurocode 5) the structural design calculations are made in the ultimate limit state (ULS) for bending moment, shear, tension and compression resistance, stability, connections, and stress concentrations in notches, holes and tapered beams. Serviceability limit state (SLS) design calculations are made for deformations and floor vibrations. The following subsections provide instructions on these design calculations with remarks regarding LVL-specific properties. The equations in this Section have individual numbers and when an equation is in the same form in Eurocode 5, it is marked with EC5 and the equation reference number there, e.g., (EC5 6.11).

4.3.1 Bending

The following expressions shall be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.4) \text{ (EC5 6.11)}$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.5) \text{ (EC5 6.12)}$$

where

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes as shown in Figure 4.2;
 $f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths. In addition to k_{mod} and γ_M , for LVL in edgewise bending the design value is dependent on the member height h . That is taken into consideration by factor k_h which is defined as

$$k_h = \left(\frac{300}{h}\right)^s \leq 1,1 \quad (4.6) \text{ (EC5 3.3)}$$

where

h is the member height in bending

s is the size effect parameter. For LVL-P and LVL-C strength classes it is 0,15, but manufacturer-defined values are also possible

Note: Multiple-glued GLVL products are subject to manufacturer-specific rules for size effect in flatwise bending in flatwise bending (reference size and size effect parameter $s_{flat,m}$).

The value of the factor k_m should be taken as follows LVL:

for rectangular sections: $k_m = 0,7$

for other cross sections: $k_m = 1,0$

Note: The factor k_m makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in the cross-section.

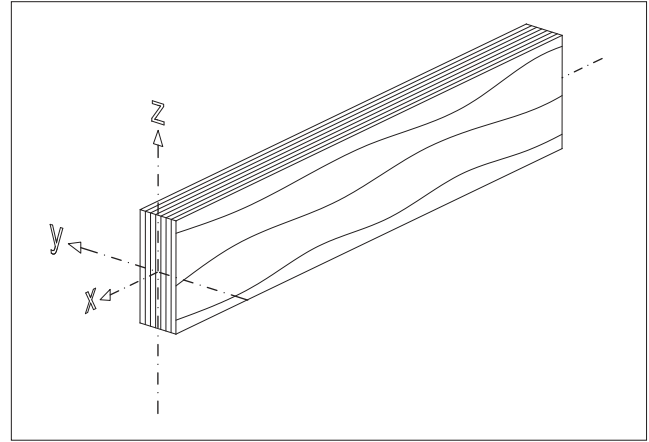


Figure 4.3. Member axes of LVL. X is in the grain direction of the surface veneers, Z is in the edgewise direction and Y in the flatwise direction.

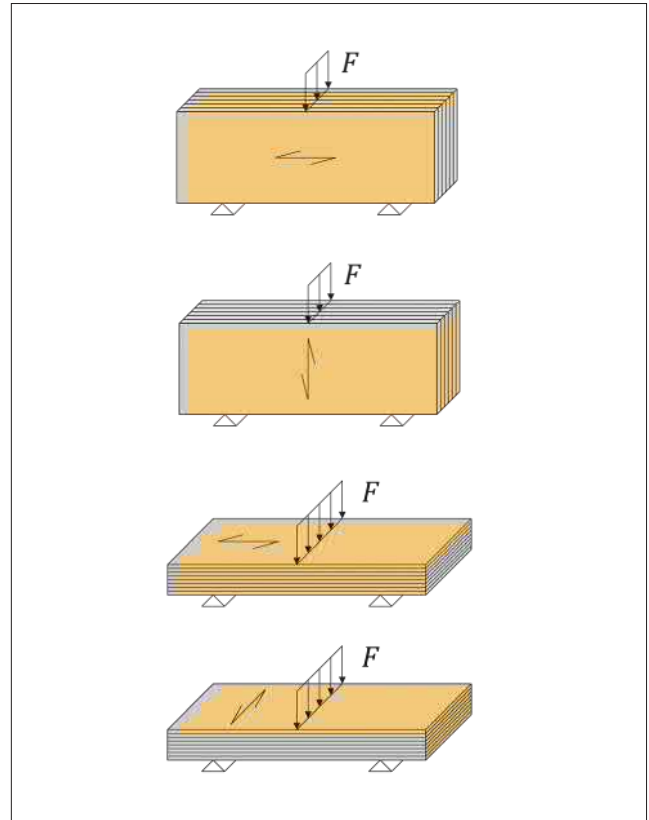


Figure 4.4. Edgewise and flatwise bending orientations of LVL.

Due to the cross veneers, LVL-C panels have a bending strength flatwise perpendicular to the grain $f_{m,90,flat,k}$ that is about 20% of the value along the grain. In the edgewise direction, $f_{m,90,edge,k}$, the strength is not defined in the DoPs of the product, but it may be calculated using the instructions provided with manufacturer's technical information. Bending strength $f_{m,90,flat,k}$ and $f_{m,90,edge,k}$ of LVL-P are negligible and cannot be utilized in structural design.

4.3.2 Shear

For shear with a stress component parallel to the grain, see Figure 4.5(a, b, d and e), and for shear with both stress components perpendicular to the grain, see Figure 4.4(c and f), the following expression shall be satisfied:

$$\tau_d \leq f_{v,d} \quad (4.7) \text{ (EC5 6.13)}$$

where

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

LVL is not sensitive to cracking and therefore the factor $k_{cr} = 1,0$. This means that the full member width b can be used in equation (4.8) of an effective width b_{ef} of the member in the verification of shear resistance of members in bending.

$$b_{ef} = k_{cr} \cdot b \quad (4.8) \text{ (EC5 6.13a)}$$

At supports, the contribution to the total shear force of a concentrated load F acting on the top side of the beam and within a distance h or h_{ef} from the edge of the support may be disregarded, see Figure 4.5. For beams with a notch at the support this reduction in the shear force applies only when the notch is on the opposite side to the support. For uniformly distributed loads, the determining shear force maybe taken at a distance of the member height h from the support.

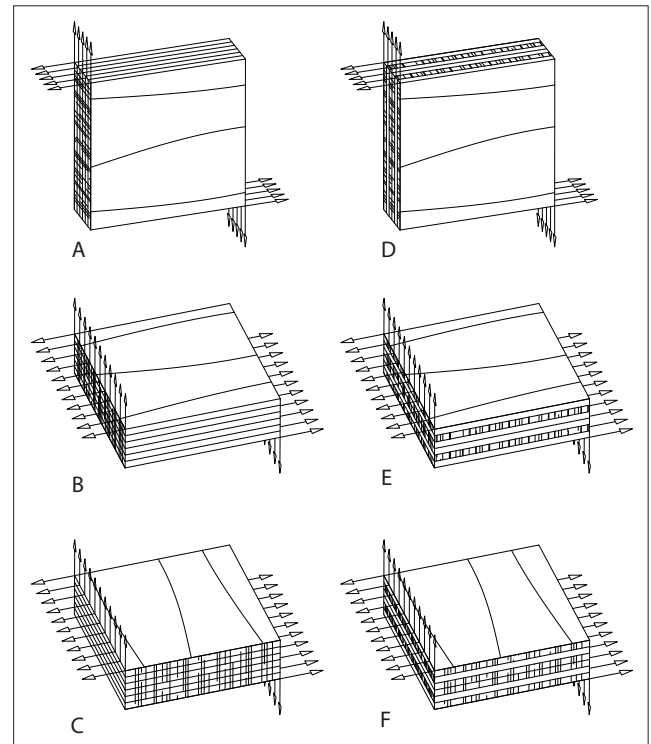


Figure 4.5. A) LVL-P shear stress edgewise parallel to grain B) LVL-P shear stress flatwise parallel to grain C) LVL-P shear stress flatwise perpendicular to grain (rolling shear) D) LVL-C shear stress edgewise parallel to grain E) LVL-C shear stress flatwise parallel to grain (rolling shear of cross veneers) F) LVL-C shear stress flatwise perpendicular to grain (rolling shear of parallel veneers).

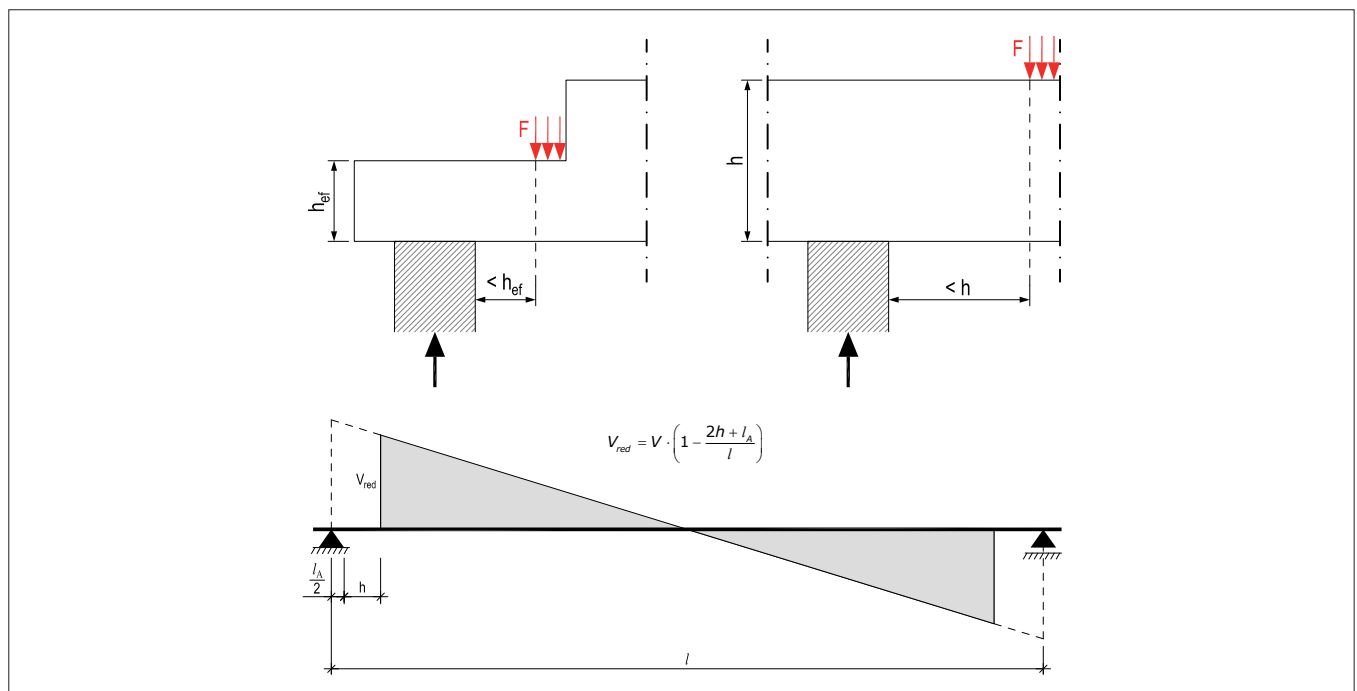


Figure 4.6. Conditions at a support, for which the concentrated force F may be disregarded in the calculation of the shear force. In the case of uniformly distributed loads, the shear force maybe reduced to the value which it has at a distance of the member height h from a support ³¹.

4. STRUCTURAL DESIGN OF LVL STRUCTURES

The shear strength of LVL is highly dependent on the direction of the shear stress. In the edgewise orientation the shear strength $f_{v,0,edge,k}$ is the highest. LVL-P and LVL-C strength values are quite similar at 3,2-4,5 N/mm², but in practice LVL-C behaves more ductile (i.e. less brittle) under loading due to the cross veneer. In the flatwise orientation the shear strength $f_{v,flat,k}$ is smaller. For LVL-P the strength $f_{v,0,flat,k}$ is 2-3,2 N/mm², but for LVL-C it is 1,1-1,3 N/mm² due to the cross veneers which are in the rolling shear direction compared to the main direction of the panel. In the across direction of the panel the strength $f_{v,90,flat,k}$ is 0,6 N/mm².

Note: Multiple-glued GLVL products have manufacturer-specific rules for size effect in flatwise shear (reference size and size effect parameter $s_{flat,v}$).

4.3.3 Tension parallel to the grain

The following expression shall be satisfied:

$$\sigma_{t,0,d} = \frac{F_{t,0,d}}{A} \leq f_{t,0,d} \quad (4.9)$$

where

$\sigma_{t,0,d}$ is the design tensile stress along the grain;
 $F_{t,0,d}$ is the design compressive force;
 A is the cross-sectional area of the member;
 $f_{t,0,d}$ is the design tensile strength along the grain. In addition to k_{mod} and γ_M , for LVL the design value is dependent on the length l of the member in tension. This is taken into consideration by a factor k_1 which is defined as

$$k_1 = \left(\frac{3000}{l} \right)^{\frac{s}{2}} \leq 1,1 \quad (4.10) \text{ (EC5 3.4)}$$

where

l is the member length in tension; and
 s is the size effect parameter. For LVL-P and LVL-C strength classes it is 0,15, but individual manufacturer-defined values are possible.

4.3.4 Tension perpendicular to the grain

The following expression shall be satisfied:

$$\sigma_{t,90,d} \leq f_{t,90,d} \quad (4.11)$$

where

$\sigma_{t,90,d}$ is the design tensile stress perpendicular to the grain;
 $f_{t,90,d}$ is the design tensile strength perpendicular to the grain.

Although in LVL-P the veneers are parallel to the main direction of the product, there is a small difference between the grain directions of the veneers. This makes the product less sensitive to cracking and the tension strength perpendicular to the grain edgewise of LVL-P $f_{t,90,k} = 0,5-0,8$ N/mm² is slightly higher than solid wood or glulam $f_{t,90,k} = 0,4-0,5$ N/mm².

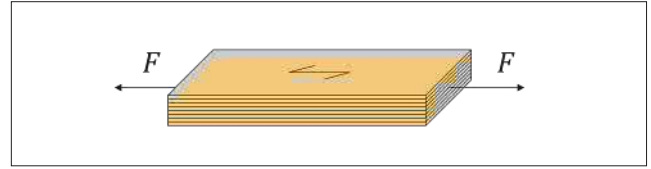


Figure 4.7. Tension parallel to grain of surface veneers.

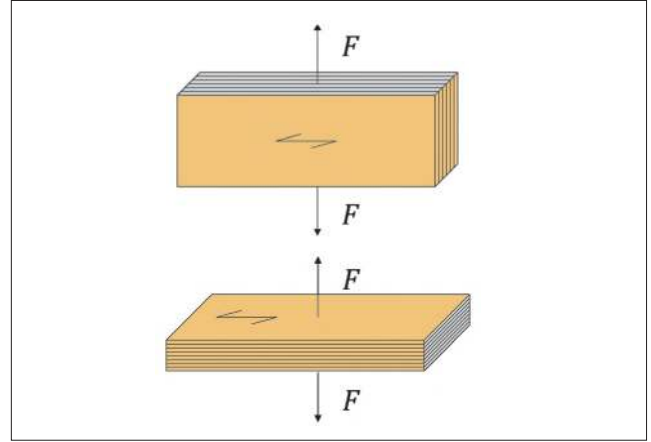


Figure 4.8. Edgewise and flatwise tension perpendicular to the grain of surface veneers.

The cross veneers of LVL-C improve the tension strength perpendicular to the grain in edgewise direction $f_{t,90,k} = 4-5$ N/mm² is many times better than the strength of LVL-P. This property is an advantage especially in suspended connections and between main beams and secondary beams or diagonal struts.

Tension strength flatwise perpendicular to the grain of LVL-P and LVL-C is low and it is not recommended to design a structure so that the stresses in this direction would become critical. The strength value in this direction is normally not defined in the DoPs for LVL products, but to have an idea of the strength level, $f_{c,90,k,flat} = \sim 0,2-0,3$ N/mm² could be used for estimations in predesign.

4.3.5 Compression parallel to the grain

The following expression shall be satisfied:

$$\sigma_{c,0,d} = \frac{F_{c,0,d}}{A} \leq f_{c,0,d} \quad (4.12)$$

where

$\sigma_{c,0,d}$ is the design compression stress along the grain;
 $F_{c,0,d}$ is the design compressive force;
 A is cross-sectional area of the member; and
 $f_{c,0,d}$ is the design compression strength along the grain.

For LVL strength classes a separate, 20% higher $f_{c,0,k}$ value has been defined for service class 1 in Table 4.5 and 4.6. The k_{mod} factor of Eurocode 5 is similar in SC1 and SC2, but material testing has shown compression strength values to be different in SC1 and SC2. The same phenomenon is also known for other load-bearing wood products, but the strength values

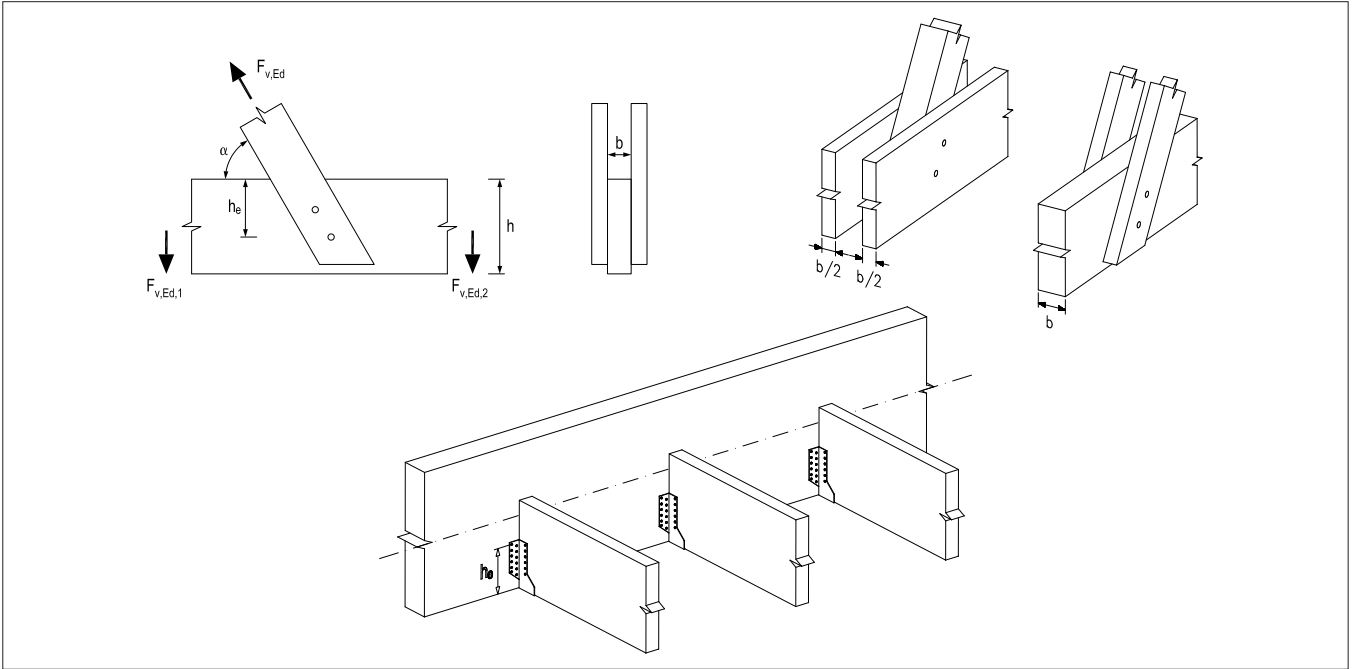


Figure 4.9. Connection between a cord and diagonal strut or secondary beams connected to the lower part of a main beam create tension stresses perpendicular to the grain. When the cord member is LVL-C and the connection is on the face side, equation 8.4 of EN1995-1-1 is not applied, since LVL-C is not sensitive to splitting (Modified from EC5 figure 8.1).

for their standards classes are not defined directly from testing according to EN 408.

In addition to the compression resistance itself, usually the stability design (buckling) of the members is more critical, see subsection 4.3.9.

4.3.6 Compression perpendicular to the grain

Compression perpendicular to the grain design is defined in EN1995-1-1, Clause 6.1.5. The following expression shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,d} \quad (4.13) \text{ (EC5 6.3)}$$

with

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} \quad (4.14) \text{ (EC5 6.4)}$$

Where

$\sigma_{c,90,d}$ is the design compressive stress in the contact area perpendicular to the grain;

$F_{c,90,d}$ is the design compressive load perpendicular to the grain;

A_{ef} is the effective contact area in compression perpendicular to the grain;

$f_{c,90,d}$ is the design compressive strength perpendicular to the grain; and

$k_{c,90}$ is a factor taking into account the load configuration, the possibility of splitting, and the degree of compressive deformation.

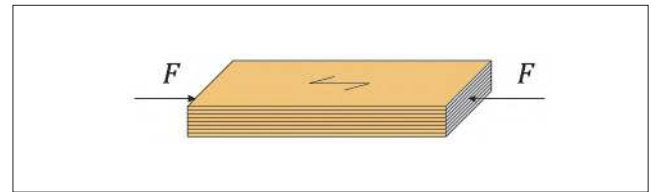


Figure 4.10. Compression parallel to grain of surface veneers.

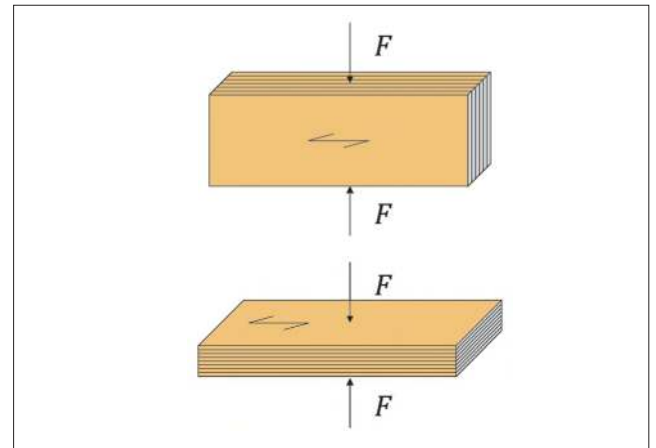


Figure 4.11. Compression perpendicular to the grain of surface veneers.

4. STRUCTURAL DESIGN OF LVL STRUCTURES

The effective contact area perpendicular to the grain, A_{ef} , should be determined taking into account an effective contact length parallel to the grain, where the actual contact length, l , at each side is increased.

Eurocode 5 does not include the parameters $k_{c,90}$ and A_{ef} for LVL in different orientations. The value of $k_{c,90}$ should be taken as 1,0 for LVL in the edgewise loading direction. For LVL in the flatwise loading direction, $k_{c,90} = 1,4$ may be used, when the distance $\ell_1 \geq 2h$ ¹⁵. The contact length and width may be increased according to the Table 4.7, but not more than a , ℓ or $\ell_1/2$, see Figure 4.11. LVL suppliers also have their own tailored instructions that may give more favourable design results.

The increased contact length and factor $k_{c,90}$ are less favourable for LVL in edgewise direction, than in the flatwise direction or compared to other wood products have, due to the failure behaviour of the products. LVL in the flatwise direction, solid wood and glulam have ductile behaviour under compression perpendicular to the grain. LVL has high strength values in the edgewise direction $f_{c,90,edge,k} = 6-9 \text{ N/mm}^2$. However, it breaks in a more brittle manner, see Figure 4.13. Figure 4.14 shows a calculation example for an LVL beam supported on an LVL sole plate.

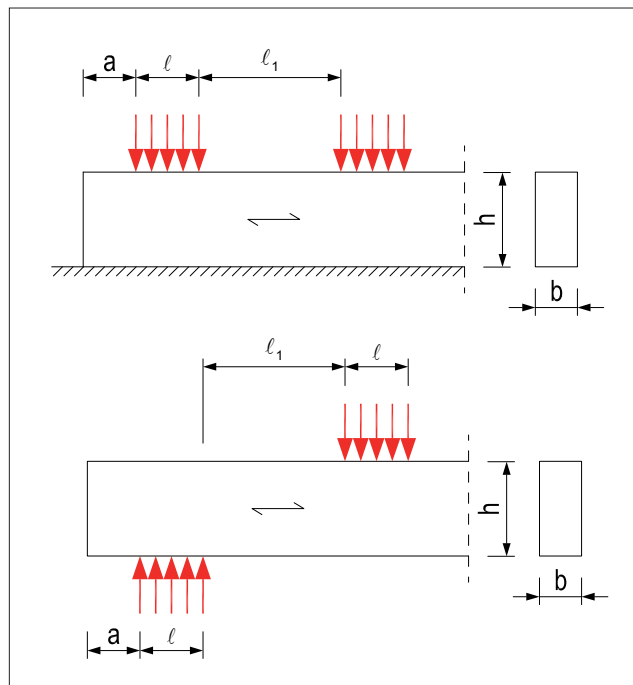


Figure 4.12. (a) Compression $F_{c,90}$ of a sole plate (b) Compression $F_{c,90}$ of beam on supports (EC5 Figure 6.2).

Table 4.7. $k_{c,90}$ -values and increase of the actual contact length for the design of compression strength perpendicular to the grain of LVL¹⁵.

Loading direction		$k_{c,90}$ -values	Increase of the actual contact length ^a [mm]
Edgewise compression strength	$f_{c,90,edge,k}$	1,0	15
Flatwise compression strength	$f_{c,90,flat,k}$	1,4	
— parallel to the grain of the surface veneers			30
— perpendicular to the grain of the surface veneers			15

^a One-sided or two-sided increase of the actual contact length, but not more than a , ℓ or $\ell_1/2$ according to EN 1995-1-1.

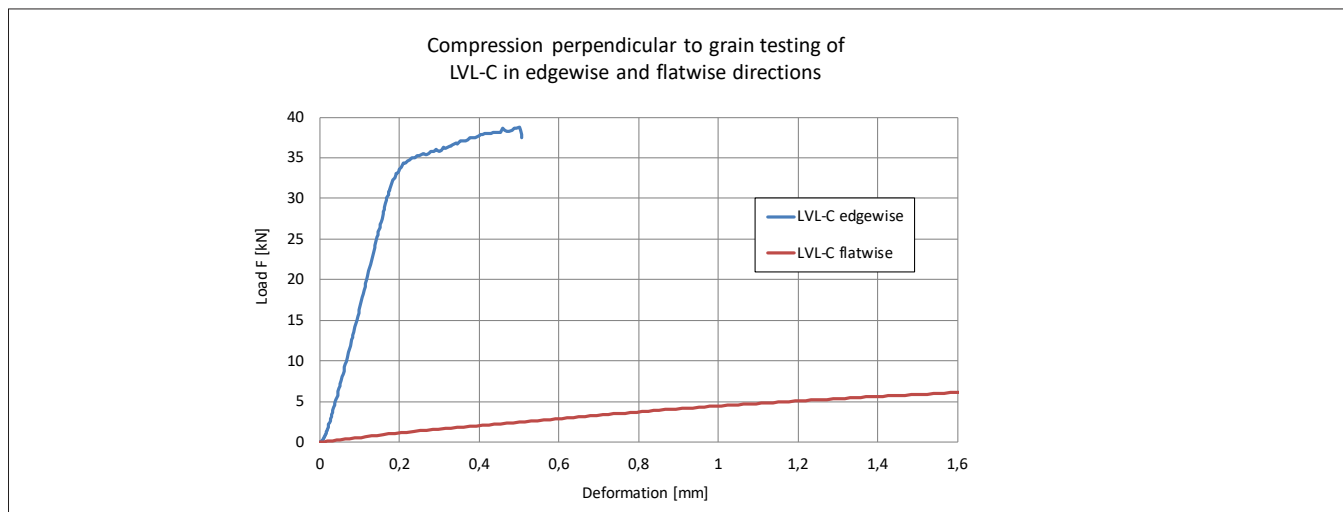
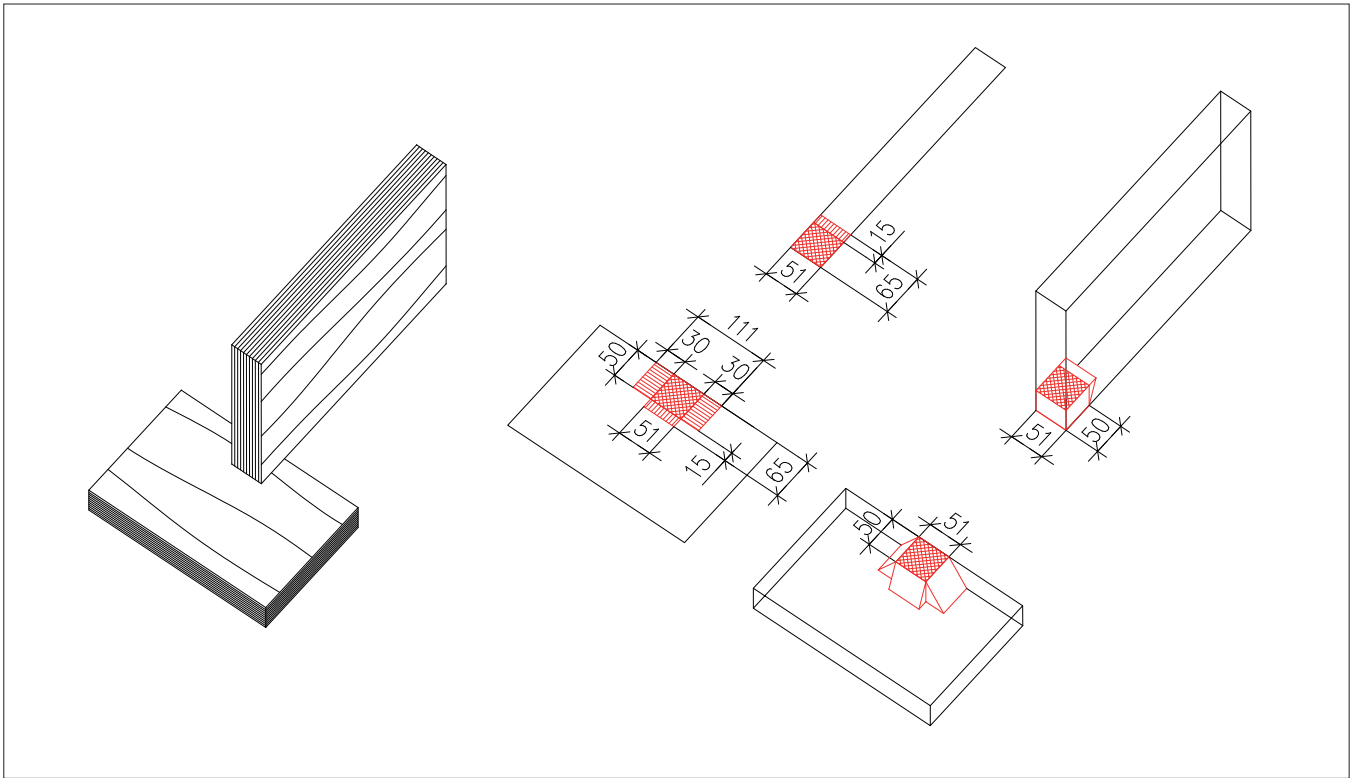


Figure 4.13. Example of LVL-C specimens in compression perpendicular to grain testing. In the flatwise direction LVL-C exhibits a ductile behaviour. In the edgewise direction orientation LVL-C is much stronger and stiffer, but undergoes more brittle failure due to buckling of the veneers.



LVL beam							
Product type	Beam thickness b [mm]	Support length l [mm]	Increase in actual contact length l_1 [mm]	Effective contact area $A_{ef} = b \cdot (l + l_1)$ [mm ²]	Compression strength $f_{c,90,edge,k}$ [N/mm ²]	$k_{c,90}$ [-]	Characteristic bearing capacity $F_{c,k} = A_{ef} \cdot k_{c,90} \cdot f_{c,90,edge,k}$ [kN]
LVL 48 P	51	50	15	3 315	6,0	1,0	20
LVL 36 C	51	50	15	3 315	9,0	1,0	30

LVL or solid wood sole plate							
Product type	Contact width b [mm]	Contact length l [mm]	Increase in actual contact length parallel l_1 and perpendicular l_2 [mm]	Effective contact area $A_{ef} = b \cdot (l + l_1) + (b \cdot l_2)$ [mm ²]	Compression strength $f_{c,90,edge,k}$ [N/mm ²]	$k_{c,90}$ [-]	Characteristic bearing capacity $F_{c,k} = A_{ef} \cdot k_{c,90} \cdot f_{c,90,edge,k}$ [kN]
LVL 48 P or 36 C	50	51	$l_1 = 30, l_2 = 15$	6 315	2,2	1,4	19,5
LVL 48 P or 36 C of pine	50	51	$l_1 = 30, l_2 = 15$	6 315	3,3	1,4	29
C24 solid wood	50	51	$l_1 = 30, l_2 = 0$	5 550	2,5	1,25	17,3

Figure 4.14. Example of an LVL beam supported on an LVL sole plate. Effective contact area is increased by 15 mm in the edgewise direction on the beam and separately by 60 mm (2x30 mm) in the length direction and 15mm in width direction on the sole plate.

4.3.7 Bending stress at an angle to the grain

The bending stresses at an angle α to the grain should satisfy the following expression 32:

$$\sigma_{m,\alpha,d} \leq \frac{f_{m,0,d}}{\frac{f_{m,0,d}}{f_{m,90,d}} \cdot \sin^2 \alpha + \frac{f_{m,0,d}}{f_{v,d}} \cdot \sin \alpha \cdot \cos \alpha + \cos^2 \alpha} \quad (4.15)$$

where

$\sigma_{m,\alpha,d}$ is the bending stress at an angle α to the grain

When the characteristic bending strength $f_{m,90,k}$ is not declared for the product, it should be assumed that $f_{m,90,k} = f_{t,90,k}$. In Figure 4.14, the edgewise bending strength values of LVL 48 P and LVL 36 C at different angles are shown as graphics with this assumption.

4.3.8 Tension stress at an angle to the grain

The tensile stresses at an angle α to the grain should satisfy the following expression 32:

$$\sigma_{t,\alpha,d} \leq \frac{f_{t,0,d}}{\frac{f_{t,0,d}}{f_{t,90,d}} \cdot \sin^2 \alpha + \frac{f_{t,0,d}}{f_{v,d}} \cdot \sin \alpha \cdot \cos \alpha + \cos^2 \alpha} \quad (4.16)$$

where

$\sigma_{t,\alpha,d}$ is the tensile stress at an angle α to the grain

In Figure 4.16, graphically illustrates tensile strength values of LVL 48 P and LVL 36 C at different angles.

More advanced instructions on determining the effect of an angle to the grain on LVL strength properties may be found in the manufacturers' technical documentation.

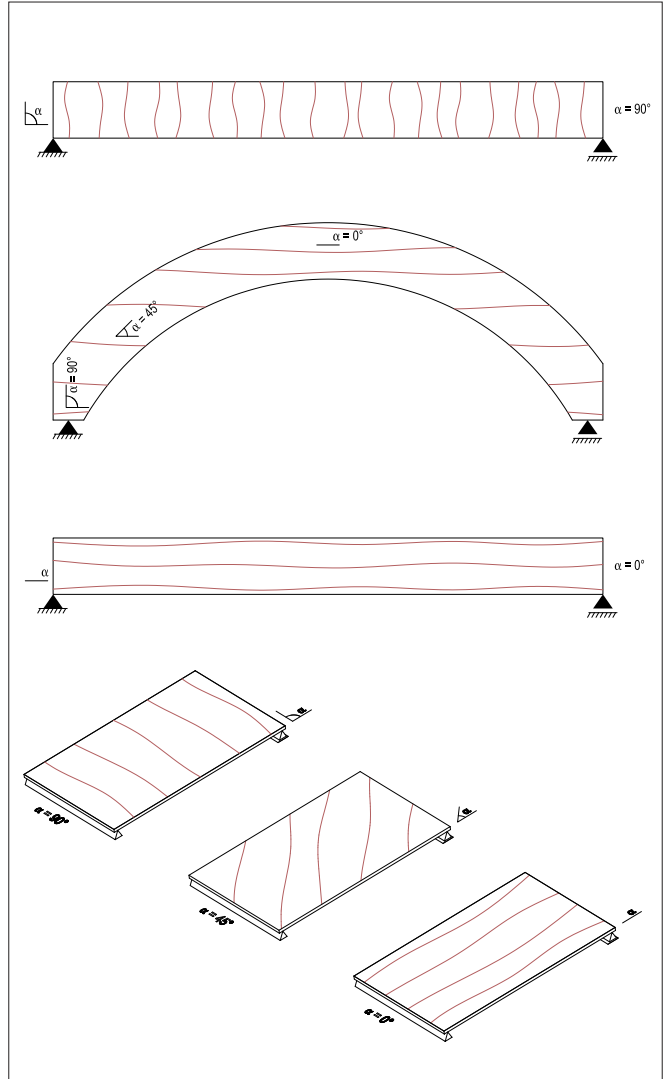


Figure 4.15. Angle α between span direction and grain direction of surface veneer.

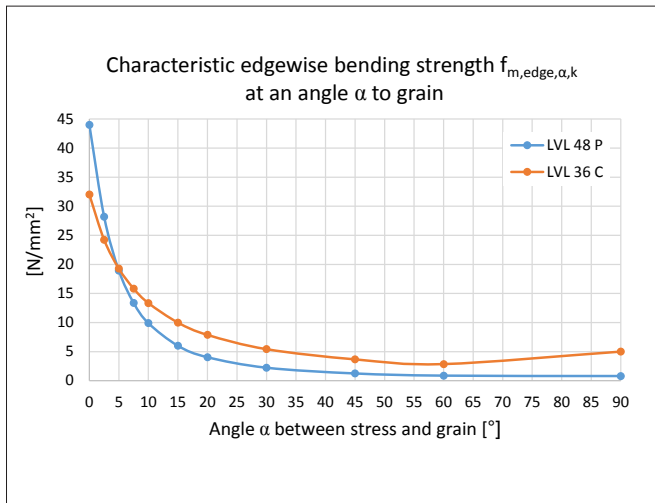


Figure 4.16. Characteristic edgewise bending strength at an angle α to grain.

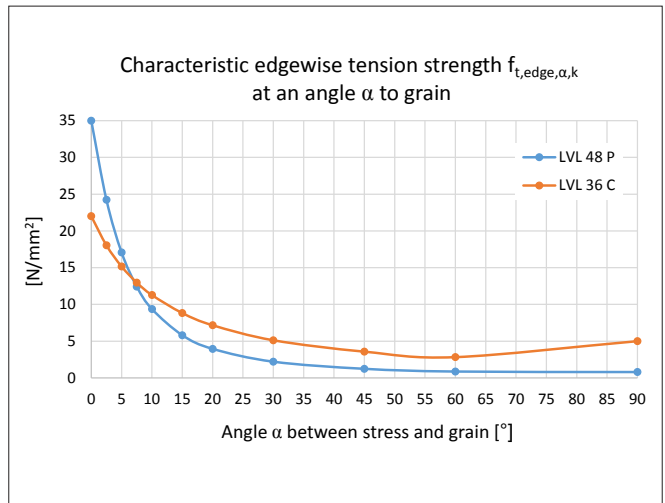


Figure 4.17. Characteristic edgewise tension strength at an angle α to grain.

4.3.9 Stability of LVL members

LVL cross sections are usually slender, as it is economical to produce panel billets and cut thin and high or deep beams and stud dimensions. Stability calculation is therefore particularly important for LVL beams.

Column stability and lateral torsional stability shall be verified using the characteristic stiffness properties $E_{0,05}$ and $G_{0,05}$.

4.3.9.1 Members subjected to combined bending and axial compression or tension

Roof rafters of pitched roofs are typical members which shall be analysed for combined bending and compression. According to Eurocode 5, the expressions (4.17) & (4.18) or (4.19) & (4.20) shall be fulfilled.

For combined bending and axial tension the following expressions shall be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.17) \text{ (EC5 6.17)}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.18) \text{ (EC5 6.18)}$$

For combined bending and axial compression the following expressions shall be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.19) \text{ (EC5 6.19)}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + k_m \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.20) \text{ (EC5 6.20)}$$

where

k_m makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in cross-section. For rectangular LVL cross sections $k_m = 0,7$ and for other cross sections $k_m = 1,0$; and

$k_{m,\alpha}$ is a factor for combined stresses in tapered beams, see subsection 4.3.11. For straight beams $k_{m,\alpha} = 1,0$.

4.3.9.2 Columns subjected to either compression or combined compression and bending

According to Eurocode 5, the expressions (4.20) & (4.21) shall be fulfilled.

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.29) \text{ (EC5 6.23)}$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{k_{m,\alpha} f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (4.30) \text{ (EC5 6.24)}$$

When both $\lambda_{rel,z} \leq 0,3$ and $\lambda_{rel,y} \leq 0,3$ the stresses should satisfy the expressions (4.19) and (4.20) of combined bending and axial compression. In all other cases the stresses, which will

be increased due to deflection, should satisfy the expressions (4.29) and (4.30).

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \quad (4.31) \text{ (EC5 6.25)}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \quad (4.32) \text{ (EC5 6.26)}$$

$$k_y = 0,5(1 + \beta_c(\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2) \quad (4.33) \text{ (EC5 6.27)}$$

$$k_z = 0,5(1 + \beta_c(\lambda_{rel,z} - 0,3) + \lambda_{rel,z}^2) \quad (4.34) \text{ (EC5 6.28)}$$

Factor β_c is 0,10 for LVL members within the straightness limit of $L/500$. The limit is defined in Eurocode 5, Section 10 as the deviation from straightness measured midway between the supports of frame members, columns and beams where lateral instability can occur.

The relative slenderness ratio should be taken as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (4.35) \text{ (EC5 6.21)}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (4.36) \text{ (EC5 6.22)}$$

Where

λ_y and $\lambda_{rel,y}$ are the slenderness ratio corresponding to bending about the y-axis (deflection in the z-direction);

λ_z and $\lambda_{rel,z}$ are the slenderness ratio corresponding to bending about the z-axis (deflection in the y-direction); and

$E_{0,05}$ is the characteristic value of modulus of elasticity parallel to the grain.

For rectangular cross section the slenderness of a member is defined as:

$$\lambda = \frac{l_c}{i} = \frac{l_c}{\sqrt{\frac{I}{A}}} = \frac{l_c}{\sqrt{\frac{bh^3}{12}}} = \sqrt{12} \left(\frac{l_c}{h} \right) \quad (4.37)$$

where

l_c is the buckling length; and

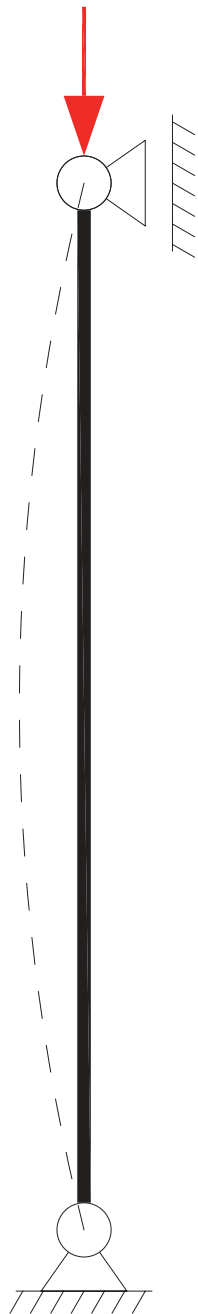
h is the height of the member in the direction of the buckling analysis.

The buckling coefficient k_c for different LVL classes and slenderness ratios are shown in Figure 4.18 and Table 4.8. The values are very similar for all shown LVL classes.

More advanced instructions on determining the effect of an angle to the grain on LVL strength properties may be found in the manufacturers' technical documentation.

4. STRUCTURAL DESIGN OF LVL STRUCTURES

Table 4.8. Buckling coefficient k_c of different LVL classes for different slenderness ratios λ .



λ [-]	k_c [-]			
	LVL 32 P	LVL 48 P	LVL 25 C	LVL 36 C
15	1,00	1,00	1,00	1,00
20	0,99	0,99	0,99	0,99
25	0,98	0,98	0,98	0,98
30	0,97	0,97	0,97	0,97
35	0,95	0,95	0,95	0,96
40	0,92	0,93	0,93	0,93
45	0,89	0,90	0,90	0,90
50	0,84	0,86	0,86	0,86
55	0,77	0,80	0,80	0,81
60	0,69	0,73	0,73	0,74
65	0,62	0,65	0,66	0,66
70	0,55	0,58	0,58	0,59
75	0,49	0,52	0,52	0,53
80	0,43	0,46	0,47	0,47
85	0,39	0,41	0,42	0,42
90	0,35	0,37	0,38	0,38
95	0,31	0,34	0,34	0,34
100	0,29	0,31	0,31	0,31
110	0,24	0,26	0,26	0,26
120	0,20	0,22	0,22	0,22
130	0,17	0,18	0,19	0,19
140	0,15	0,16	0,16	0,16
150	0,13	0,14	0,14	0,14
160	0,11	0,12	0,12	0,13
170	0,10	0,11	0,11	0,11
180	0,09	0,10	0,10	0,10
190	0,08	0,09	0,09	0,09
200	0,07	0,08	0,08	0,08
220	0,06	0,07	0,07	0,07
240	0,05	0,06	0,06	0,06

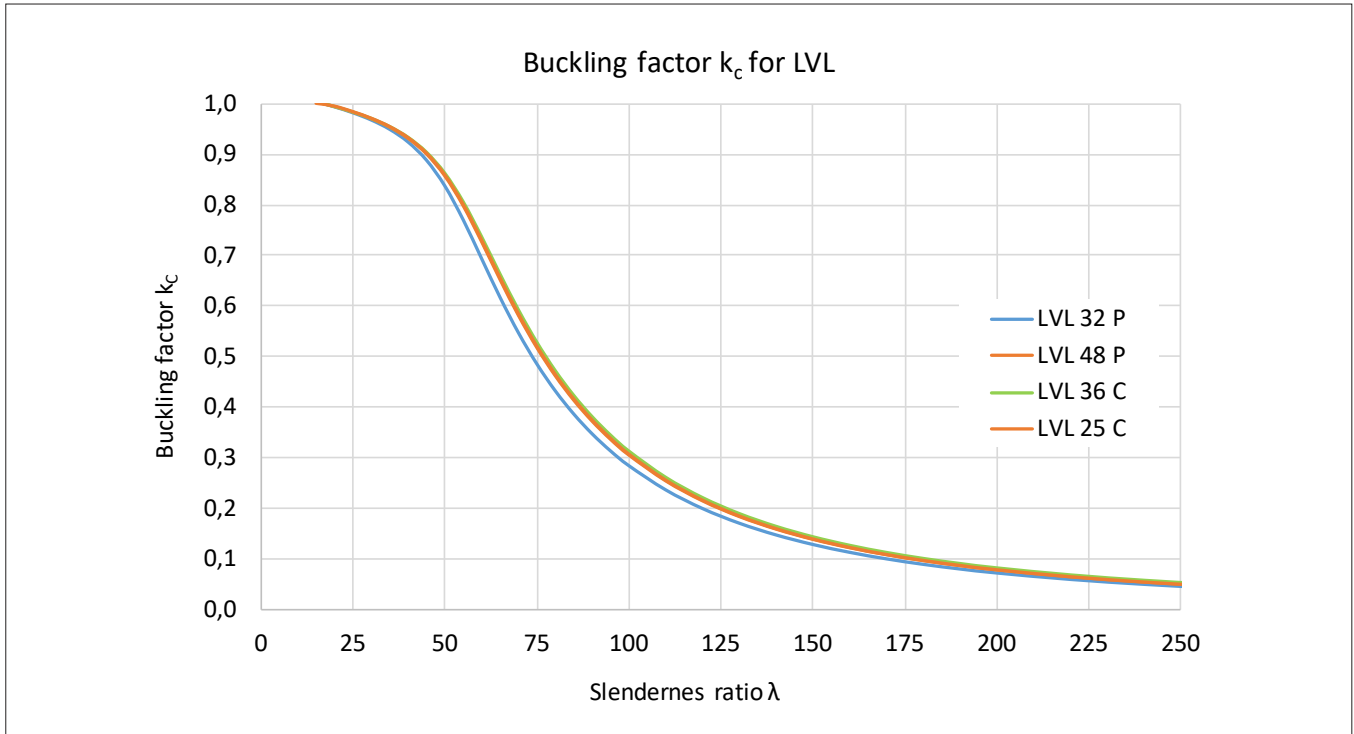


Figure 4.18. Buckling coefficient k_c of different LVL classes for different slenderness ratios λ .

4.3.9.3 Beams subjected to either bending or combined bending and compression – Lateral torsional buckling (LTB)

Lateral torsional stability shall be verified both in the case where only a moment M_y exists about the strong axis y and where a combination of moment M_y and compressive force N_c exists.

In the case where a bending moment M exists only on one axis, the stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_{crit} \cdot f_{m,d} \quad (4.38) \text{ (EC5 6.33)}$$

where

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength; and

k_{crit} is a factor that takes into account the reduced bending strength due to lateral buckling.

In the case where a combination of moment M_y about the strong axis y and compressive force N_c exists, the stresses should satisfy the following expression

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,d}} \right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leq 1 \quad (4.39) \text{ (EC5 6.35)}$$

For beams with an initial lateral deviation from straightness within the limits defined in Section 10 of Eurocode 5, k_{crit} may be determined from expression:

$$k_{crit} = \begin{cases} 1 & \text{when } \lambda_{rel,m} \leq 0,75 \\ 1,56 - 0,75\lambda_{rel,m} & \text{when } 0,75 < \lambda_{rel,m} \leq 1,4 \\ \frac{1}{\lambda_{rel,m}^2} & \text{when } 1,4 < \lambda_{rel,m} \end{cases} \quad (4.40) \text{ (EC5 6.34)}$$

The factor k_{crit} may be taken as 1,0 for a beam where lateral displacement of its compressive edge is prevented throughout its length and where torsional rotation is prevented at its supports

The relative slenderness for bending should be taken as

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \quad (4.41) \text{ (EC5 6.30)}$$

where $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values. The critical bending stress should be taken as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{l_{ef} W_y} \quad (4.42) \text{ (EC5 6.31)}$$

where

$E_{0,05}$ is the fifth percentile value of modulus of elasticity parallel to grain;

$G_{0,05}$ is the fifth percentile value of shear modulus parallel to grain; Note: $G_{edge,0,05}$ of LVL shall be used;

I_z is the second moment of area about the weak axis z ;

I_{tor} is the torsional moment of inertia;

l_{ef} is the effective length of the beam, depending on the support conditions and the load configuration, according to Table 4.9;

W_y is the section modulus about the strong axis y .

4. STRUCTURAL DESIGN OF LVL STRUCTURES

In the case of rectangular cross sections:

$$I_{\text{tor}} = k_1 \cdot h \cdot b^3 \quad (4.43)$$

where

$$k_1 = \frac{1}{3} \left(1 - \frac{0,63 \cdot b}{h} \right) \quad (4.44)$$

$k_1 = 0,14$ for square cross sections;

$k_1 = 0,23$, when $h/b = 2$;

$k_1 = 0,28$, when $h/b = 4$ and

$k_1 = 0,30$, when $h/b = 6$

$k_1 = 0,31$, when $h/b = 10$

Table 4.9. Effective length as a ratio of the span (Modified from EC5 Table 6.1.).

Beam type	Loading type	I_{ef} / I^a
Simply supported	Constant moment	1,0
	Uniformly distributed load	0,9
	Concentrated force at the middle of the span	0,8
Cantilever	Uniformly distributed load	0,5
	Concentrated force at the middle of the span	0,8

^a The ratio between the effective length l_{ef} and the span l is valid for a beam with torsionally restrained supports and loaded at the centre of the gravity. If the load is applied at the compression edge of the beam. l_{ef} should be increased by $2h$ and may be decreased by $0,5h$ for a load at the tension edge of the beam.

When a beam is supported against lateral torsional buckling (LTB) from the compressive edge and the beam is loaded from the compressive side, the effective length l_{ef} in the design is the distance between the LTB supports $a + 2h$. When the beam is loaded from the tensile side, the effective length $l_{\text{ef}} = a - 0,5h$. When the compressive edge of the beam is loaded only with point loads at the locations of the LTB supports, the effective length $l_{\text{ef}} = a$ ³¹.

The equation (4.42) of $\sigma_{\text{m,crit}}$ may be replaced by a simplified equation:

$$\sigma_{\text{m,crit}} = \frac{c \cdot b^2}{h \cdot l_{\text{ef}}} E_{0,05} \quad (4.45)$$

where

c is 0,58 for LVL 48 P and 0,67 for LVL 36 C;

b is the beam thickness [mm]; and

h is the beam height [mm].

Note: More advanced design instructions for LTB can be found from manufacturers' technical documentation.

4.3.10 Notches

The effects of stress concentrations at the notch shall be taken into account in the strength verification of members. The effect of stress concentrations may be disregarded in the following cases:

- Tension or compression parallel to the grain;
- Bending with tensile stresses at the notch, if the taper is not steeper than $1:i = 1:10$, that is $i \geq 10$, see Figure 4.20 a);
- Bending with compressive stresses at the notch, see Figure 4.20 b)



Figure 4.19. Installation of notched rafter beam.

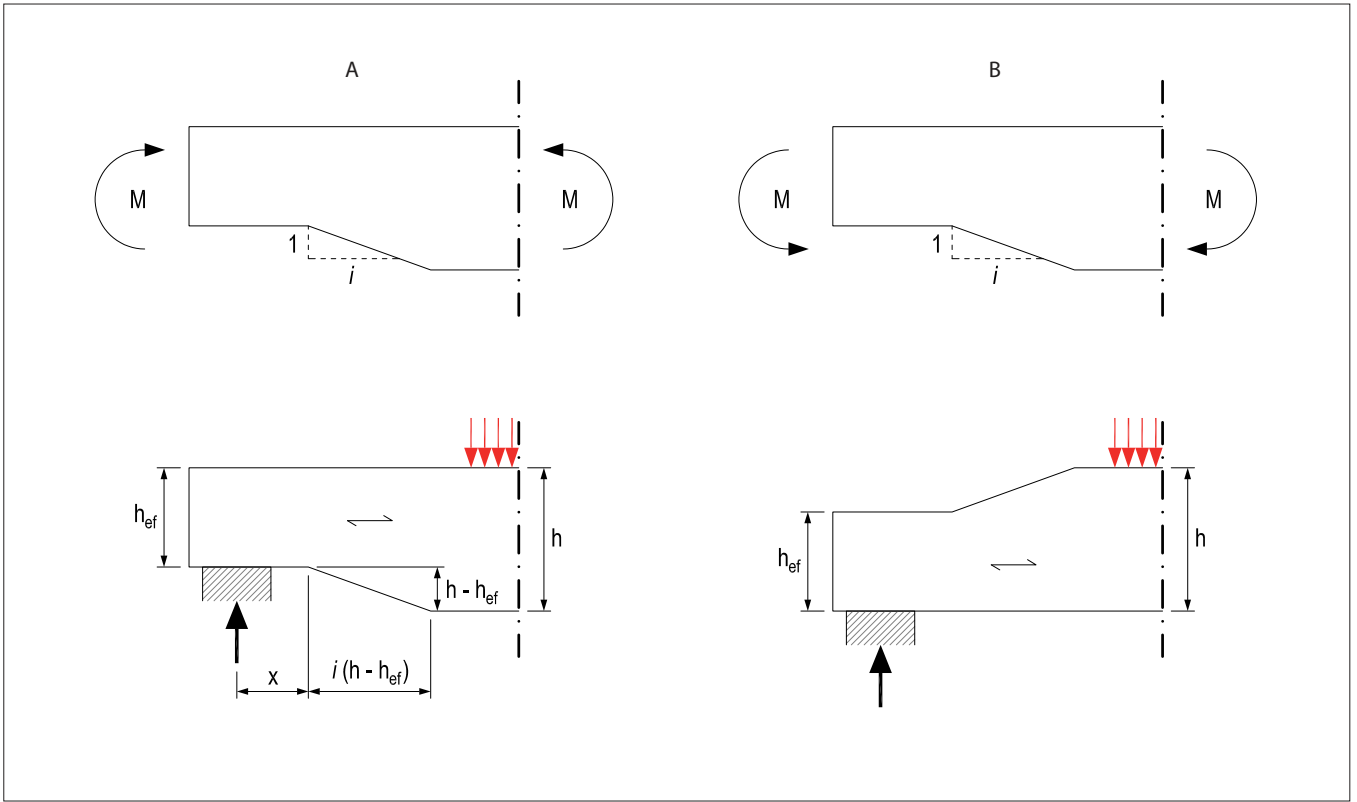


Figure 4.20. Bending at a notch A) with tensile stresses at the notch B) with compressive stresses at the notch (EC5 Figure 6.10 and 6.11).

When the taper at the notch at tensile side is steeper than 1:10, it can be located only at the support.

For beams with rectangular cross sections and where the grain runs essentially parallel to the length of the member, the shear stresses at the notched support should be calculated using the effective (reduced) depth h_{ef} , see Figure 4.18 b).

It should be verified that

$$\tau_d = \frac{1,5 V_d}{b \cdot h_{ef}} \leq k_v \cdot f_{v,d} \quad (4.46) \text{ (EC5 6.60)}$$

where k_v is a reduction factor defined as follows:

- For beams notched at the opposite side to the support, see Figure 4.20 b), $k_v = 1,0$.
- For beams notched on the same side as the support, see Figure 4.18 a)

$$k_v = \min \left(1 ; \frac{k_n \left(1 + \frac{1,1 i^{1,5}}{\sqrt{h}} \right)}{\sqrt{h} \left(\sqrt{\alpha (1 - \alpha)} + 0,8 \frac{x}{h} \sqrt{\frac{1}{\alpha} - \alpha^2} \right)} \right) \quad (4.47) \text{ (EC5 6.62)}$$

where

i is the notch inclination, see Figure 4.20 a);

h is the beam depth in mm;

x is the distance from line of action of the support reaction to the corner of the notch, in mm; and

$$\alpha = \frac{h_{ef}}{h} \quad (4.48)$$

k_n is 4,5 for LVL in general. Note: manufacturers provide product-specific information on the k_n values of their products, especially where the advantages LVL-C are evident.

4.3.11 Tapered beams

The influence of the taper on the bending stresses parallel to the surface shall be taken into account.

The design bending stresses, $\sigma_{m,\alpha,d}$ and $\sigma_{m,0,d}$ (see Figure 4.21) may be taken as:

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{6M_d}{bh^2} \quad (4.49) \text{ (EC5 6.37)}$$

At the outermost fibre of the tapered edge, the stresses should satisfy the following expression:

$$\sigma_{m,\alpha,d} \leq k_{m,\alpha} \cdot f_{m,d} \quad (4.50) \text{ (EC5 6.38)}$$

where

$\sigma_{m,\alpha,d}$ is the design bending stress at an angle to grain;

$f_{m,d}$ is the design bending strength; and

$k_{m,\alpha}$ is calculated as follows:

For tensile stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{a \cdot f_{v,d}} \tan \alpha\right)^2 + \left(\frac{f_{m,d}}{f_{t,90,d}} \tan^2 \alpha\right)^2}} \quad (4.51)^{32}$$

where

$$a = \begin{cases} 0,75 & \text{for LVL - P} \\ 1,0 & \text{for LVL - C} \end{cases} \quad (4.52)^{32}$$

For compressive stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{b \cdot f_{v,d}} \tan \alpha\right)^2 + \left(\frac{f_{m,d}}{f_{c,90,d}} \tan^2 \alpha\right)^2}} \quad (4.53)^{32}$$

where

$$b = \begin{cases} 1,5 & \text{for LVL - P} \\ 1,0 & \text{for LVL - C} \end{cases} \quad (4.54)^{32}$$

It is not necessary to take $k_{m,\alpha}$ into consideration in the resistance against lateral torsional buckling of the beam equation (4.38). The effects of combined axial force and bending moment shall be taken into account. When the tapered edge is under tension stress, $k_{m,\alpha}$ is used to reduce the bending strength in the equations for combined stresses equation (4.17) and (4.18). When the tapered edge is under compression stress, $k_{m,\alpha}$ is used to reduce the bending strength in the equations for combined stresses equations (4.19) and (4.20).

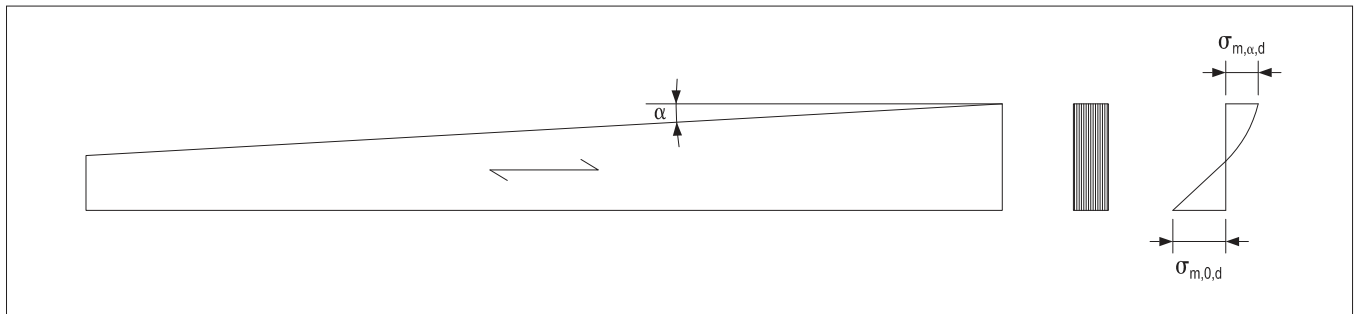


Figure 4.21. Single-tapered beam. α is the angle between the tapered edge and the grain direction of the beam (EC5 Figure 6.8).

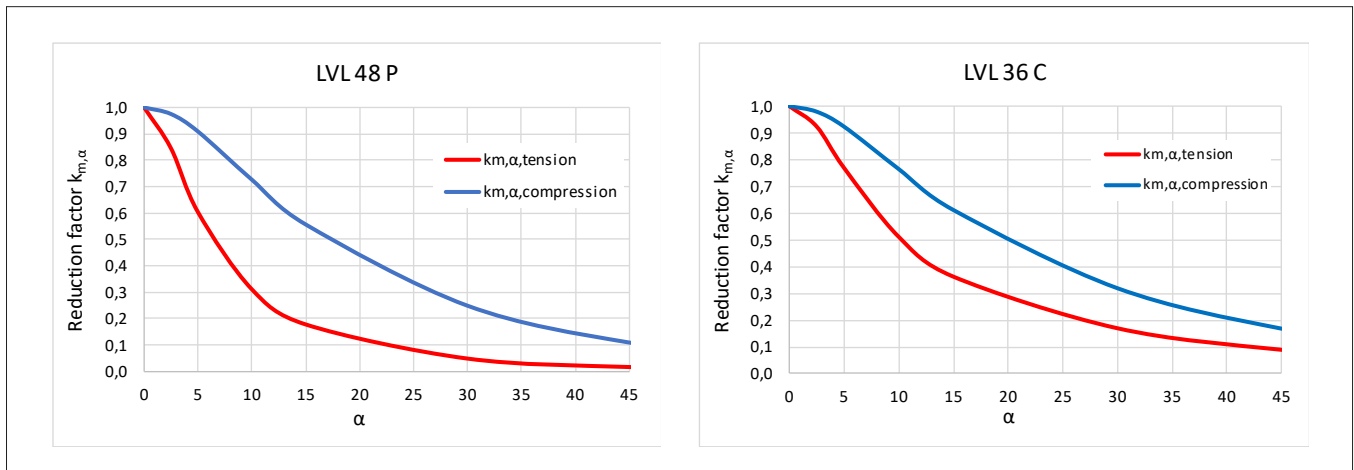


Figure 4.22. Strength reduction factor $k_{m,\alpha}$ for tensile or compression stress parallel to the tapered edge. Left LVL 48 P, right LVL 36 C.

It is recommended to have the tapered edge on the compressive side, especially for LVL-P, since the tension perpendicular to grain strength $f_{t,90,edge,k}$ is low, which can lead to cracks and brittle failure. LVL-C may be used for special shapes, also when the tapered edge is on the tensile side, as its $f_{t,90,edge,k}$ is higher due to the cross veneers and it behaves more ductile. Figure 4.21 shows the $k_{m,\alpha}$ factors as a function of the angle α .

For high pitched roof beams ($\alpha \geq 10^\circ$) the maximum shear stress $\tau_{v,max,d}$ and tension perpendicular to the grain $\sigma_{90,max,d}$ shall be calculated at the point of the maximum bending moment stress with the equations:

$$\tau_{v,max,d} = \sigma_{m,0,max,d} \cdot \tan \alpha \quad (4.55)^{30}$$

$$\sigma_{90,max,d} = \sigma_{m,0,max,d} \cdot \tan^2 \alpha \quad (4.56)^{30}$$

For double-tapered, curved and pitched camber beams design instruction are given in Eurocode 5 clause 6.4.3. Additional information to the clause:

- Factor k_r is 1,0 for LVL in the edgewise direction, as the shape of the beam is cut directly from a panel and no reduction due to bending of the laminates during production is needed.
- $k_{m,\alpha}$ is not used together with the equations for checking the stresses at the apex point.
- It is not necessary to take kl into consideration in the resistance against lateral torsional buckling of the beam (4.38).

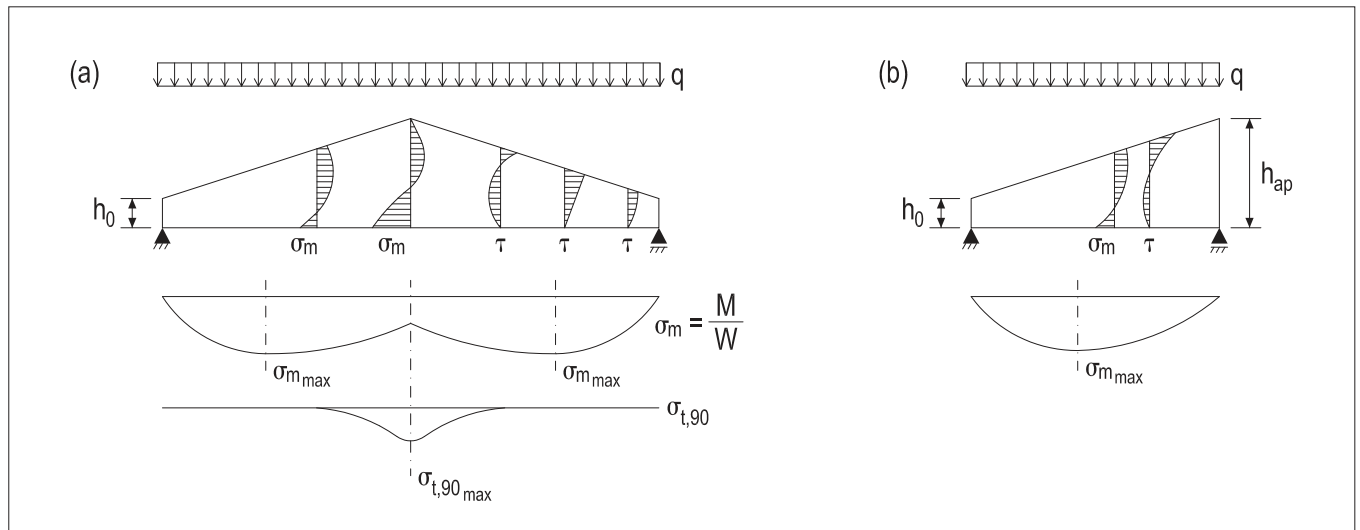


Figure 4.23. Stress distributions in single and double-tapered beams. When the angle between loading and the grain is large ($\alpha \geq 10^\circ$), shear stress at the point of maximum bending moment stress may become more critical than the shear stress at the support ³⁰.

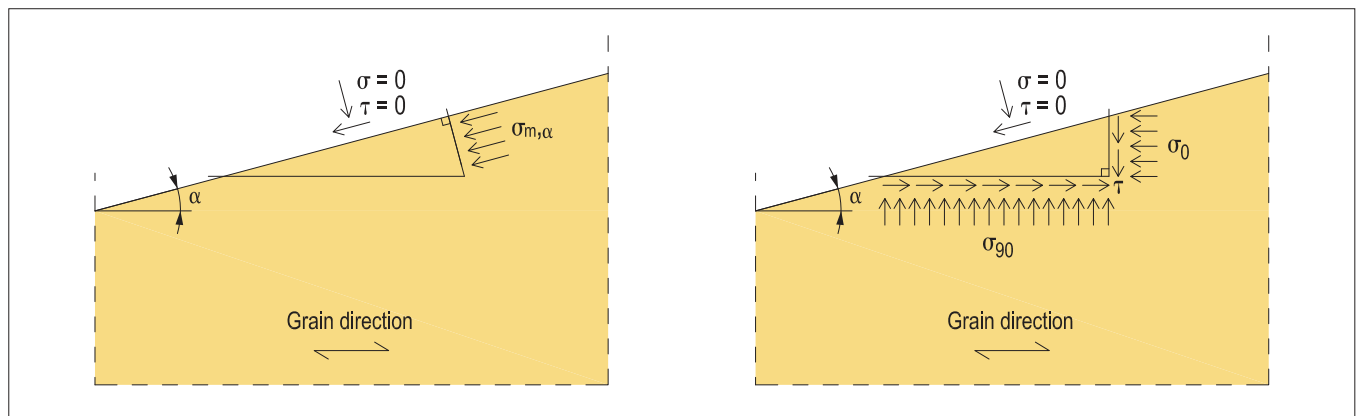


Figure 4.24. Stresses at the tapered edge of a beam: bending stress $\sigma_{m,\alpha}$ at the direction of the edge, bending stress at the grain direction σ_0 , shear stress $\tau = \sigma_0 \cdot \tan \alpha$ and stress perpendicular to the grain $\sigma_{90} = \sigma_0 \cdot \tan^2 \alpha$ ³⁰.

4.3.12 Holes

Eurocode 5 does not provide instructions for designing holes in beams, but such instructions are presented in the non-conflicting complementary instructions (NCCI) for Eurocode 5. The design method presented in this subsection is based on the Austrian NCCI document ÖNORM B 1995-1-1:2015, annex F ³³ and it can be applied to holes in LVL beams in service class 1 and 2 conditions. LVL suppliers have in their technical documentation also their own tailored instructions for designing holes in LVL beams with different boundary conditions.

For all beams with holes the bending, shear and tension/compression resistance shall be verified at the location of the hole. When the diameter d of the hole is ≥ 50 mm or $\geq h/10$, the resistance against tension perpendicular to the grain shall be verified by equation (4.57), shear stress concentration shall be verified by equation (4.62). The bending stress at the location of the hole shall be verified by equations (4.64) and (4.65) for rectangular holes or (4.71) for round holes. The corners of rectangular holes shall be a rounding radius $r \geq 15$ mm. The boundary conditions of the geometry are specified in Figure 4.25.

The verification of the resistance against tension perpendicular to the grain stresses can be the most critical condition to fulfil in the design of holes in LVL-P beams. LVL-C beams, on the other hand, offer a significant advantage for beams with holes, as the cross veneers act as reinforcement around the holes preventing cracking due to tension stresses perpendicular to the grain. Their resistance is therefore superior and the larger hole size limit for reinforced holes specified in the Austrian NCCI document may be applied to LVL-C beams.

Tension stress perpendicular to the grain is verified by the equation

$$\sigma_{t,90,d} = \frac{F_{t,90,d}}{0,5 \cdot l_{t,90} \cdot b \cdot k_{t,90}} \leq f_{t,90,d} \quad (4.57)$$

where

$$k_{t,90} = \min \left\{ \left(\frac{450}{h} \right)^{0,5} \right\} \quad (4.58)$$

$\sigma_{t,90,d}$ is the design value of tension stress perpendicular to the grain [N/mm²];

$F_{t,90,d}$ is the design value of tension force perpendicular to the grain [N];

$l_{t,90}$ is the load distribution length [mm], see Figure 4.26;

b is the beam thickness [mm];

$f_{t,90,d}$ is the design value of tension strength perpendicular to the grain [N/mm²]; and

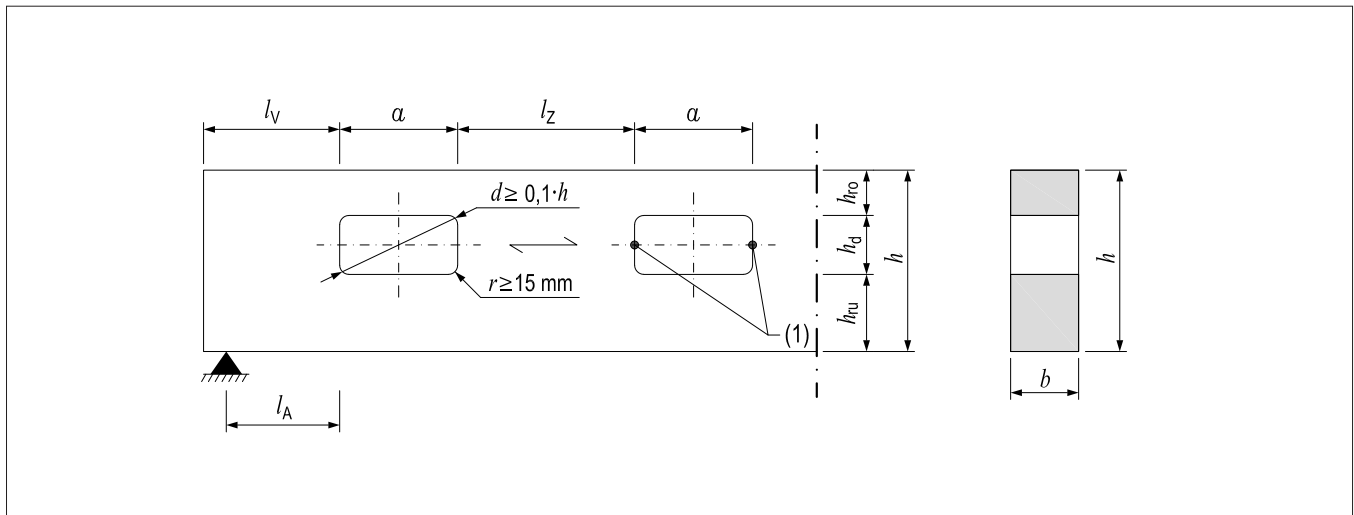
h is the beam height [mm].

The tension perpendicular to the grain force $F_{t,90,d}$ depends on the shear force V_d and bending moment M_d at the edge of the hole:

$$F_{t,90,d} = \frac{V_d \cdot h_d}{4 \cdot h} \cdot \left[3 - \left(\frac{h_d}{h} \right)^2 \right] + 0,008 \cdot \frac{M_d}{h_r} \quad (4.59)$$

where

$$h_r = \begin{cases} \min(h_{r0}; h_{ru}) & \text{for rectangular holes} \\ \min(h_{r0} + 0,15 \cdot d; h_{r0} + 0,15 \cdot d) & \text{for round holes} \end{cases} \quad (4.60)$$



Product type	l_v	l_A	l_z	h_{r0} and h_{ru}	a	h_d
LVL-P	$\geq h$	$\geq 0,5 h$	Max ($\geq 1,5h$; 300 mm)	$\geq 0,35 h$	$\leq 2,5 h_d$	$\leq 0,15 h$
LVL-C	$\geq h$	$\geq 0,5 h$	Max ($\geq 1,5h$; 300 mm)	$\geq 0,25 h$	$\leq 2,5 h_d$	$\leq 0,4 h$

Figure 4.25. Geometrical boundary conditions of holes in beams ³³.

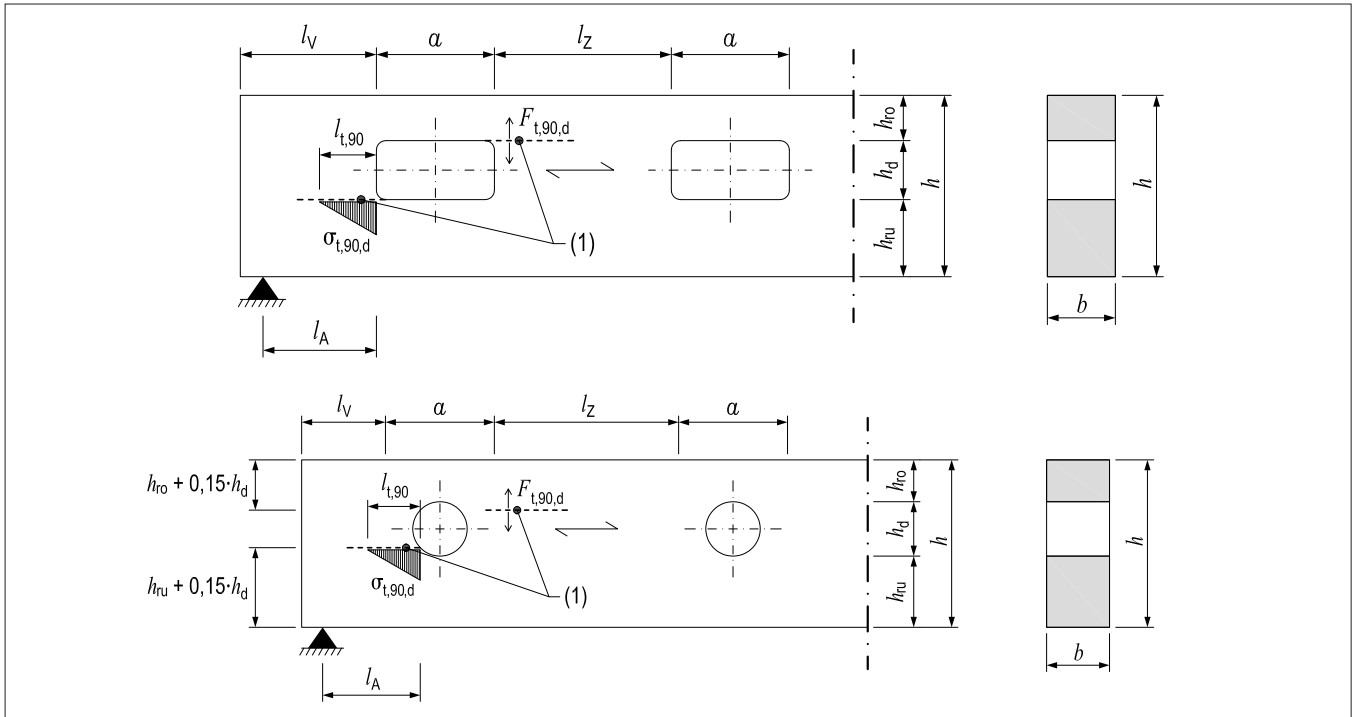


Figure 4.26. Tension perpendicular to the grain stresses at the hole edges. (1) Risk of cracks due to the tension in perpendicular to grain ³³.

h_d is the height of the hole for rectangular holes. For round holes $h_d = 0,7d$ may be used in the equation (4.59).

Load distribution length $l_{t,90}$ is

$$l_{t,90} = \begin{cases} 0,5 \cdot (h_d + h) & \text{for rectangular holes} \\ 0,35 \cdot d + 0,5 \cdot h & \text{for round holes} \end{cases} \quad (4.61)$$

Verification of shear stress concentration at the hole edge shall fulfil the condition:

$$\tau_d = k_\tau \cdot \frac{1,5 \cdot V_d}{b \cdot (h - h_d)} \leq f_{v,d} \quad (4.62)$$

Where

$$k_\tau = 1,85 \cdot \left(1 + \frac{a}{h}\right) \cdot \left(\frac{h_d}{h}\right)^{0,2} \quad (4.63)$$

τ_d is the design value of shear stress;

k_τ is the factor to determine maximum shear stress due to stress concentration;

a is the length of a hole [mm], for round holes $a = hd$; and

$f_{(v,d)}$ is the design value of edgewise shear strength

Bending stress at the location of a rectangular hole is verified by the equations:

$$\frac{M_d}{W_n} + \frac{M_{o,d}}{W_o} \leq 1 \quad (4.64)$$

$$\frac{M_d}{W_n} + \frac{M_{u,d}}{W_u} \leq 1 \quad (4.65)$$

where

$$W_n = \frac{b \cdot (h^2 - h_d^2)}{6} \quad (4.66)$$

$$M_{o,d} = \frac{A_o}{A_u + A_o} \cdot V_d \cdot \frac{a}{2} \quad (4.67)$$

$$M_{u,d} = \frac{A_u}{A_u + A_o} \cdot V_d \cdot \frac{a}{2} \quad (4.68)$$

$$A_o = b \cdot h_{ro} \text{ and } W_o = \frac{b \cdot h_{ro}^2}{6} \quad (4.69)$$

$$A_u = b \cdot h_{ru} \text{ and } W_u = \frac{b \cdot h_{ru}^2}{6} \quad (4.70)$$

W_o and W_u is the effective section modulus of the beam at the location of a hole [mm³]

$f_{m,d}$ is the edgewise bending strength [N/mm²]

Bending stress at the location of a round hole is verified by the equations:

$$\frac{M_d}{W_n} \leq 1 \quad (4.71)$$

The resistance of LVL-P beams at the location of holes maybe improved and larger hole sizes are allowed when they are reinforced by gluing wood-based panels such as plywood to both sides of the beam around the holes. Detailed design instructions are given e.g. in chapter F3.2 of the Austrian ÖNORM B 1995-1-1:2015 ³³. As LVL beams are thin, internal reinforcement with screws or glued-in-rods is not recommended.

4.3.13 Serviceability limit state design: Deflections

Instantaneous deflection of a member is calculated using the actions in equation (4.1). In serviceability limit state (SLS) the partial safety factors γ_G and γ_Q of loads are 1,0. The following serviceability limit state principles are defined in Eurocode 5, clause 2.2.3:

The deformation of a structure resulting from the effects of actions (such as axial and shear forces, bending moments and joint slip) and from moisture shall remain within appropriate limits, having regard to the possibility of damage to surfacing materials, ceilings, floors, partitions and finishes, and to the functional needs as well as any appearance requirements.

Instantaneous deformation, u_{inst} , see Figure 4.27, should be calculated for the characteristic combination of actions, see EN 1990:2002, clause 6.5.3(2) a), using mean values of the appropriate moduli of elasticity, shear moduli and slip moduli.

Final deformation u_{fin} , see e.g. w_{fin} in Figure 4.27, should be calculated by superimposing the creep deformation u_{creep} calculated using the quasi-permanent combination of actions, see EN 1990:2002, 6.5.3(2)(c), onto the instantaneous deformation u_{inst} calculated from 2.2.3(2). The creep deformation should be calculated using mean values of the appropriate moduli of elasticity, shear moduli and slip moduli and the relevant values of k_{def} given in Table 4.3.

Note: When calculating the creep deflection of LVL-C in the edge-wise direction, the k_{def} value is similar to LVL-P. In flatwise direction, the k_{def} is larger due to the rolling shear deformation of the cross veneers, similar to plywood, see Table 4.3 of k_{def} values.

If the structure consists of members or components having different creep behaviour, the long-term deformation due to the quasi-permanent combination of actions should be calculated using the final mean values of the appropriate moduli of elasticity, shear moduli and slip moduli according to Eurocode 5, clause 2.3.2.2 (1). The final deformation u_{fin} is then calculated by superimposing the instantaneous deformation, due to the difference between the characteristic and the quasi-permanent combination of actions on the long-term deformation.

For structures consisting of members, components and connections with the same creep behaviour and under the

assumption of a linear relationship between the actions and the corresponding deformations, as a simplification of EN 1990:2002, 2.2.3(3), the final deformation, u_{fin} , may be taken as:

$$u_{fin} = u_{fin,G} + u_{fin,Q_1} + \sum u_{fin,Q_i} \quad (4.72) \quad (EC5 \ 2.2)$$

where

$u_{fin,G} = u_{inst,G} (1 + k_{def})$	for a permanent action G;
$u_{fin,Q_1} = u_{inst,Q_1} (1 + \psi_{2,1} k_{def})$	for the leading variable action, Q_1 ;
$u_{fin,Q_i} = u_{inst,Q_i} (\psi_{0,i} + \psi_{2,i} k_{def})$	for accompanying variable actions, Q_i ($i > 1$);
$u_{inst,G}, u_{inst,Q_1}, u_{inst,Q_i}$	are the instantaneous deformations for actions G, Q_1, Q_i respectively;
$\psi_{2,1}, \psi_{2,i}$	are the factors for the quasi-permanent value of variable actions;
$\psi_{0,i}$	are the factors for the combination value of variable actions; and
k_{def}	is the creep deformation factor for timber and wood-based materials.

The components of deflection resulting from a combination of actions are shown in Figure 4.27, where the symbols are defined as follows:

- w_{inst} is the instantaneous deflection;
- w_{creep} is the creep deflection; and
- w_{fin} is the final deflection.

Note: LVL is not pre-cambered. Only in some very special cases LVL beams may be cut to a camber by special sawing from an LVL billet.

The net deflection below a straight line between the supports, $w_{net,fin}$, should be taken as:

$$w_{net,fin} = w_{inst} + w_{creep} \quad (4.73)$$

Note: The recommended range of limiting values of deflections for beams with span l is given in Table 4.10 depending upon the level of deformation deemed to be acceptable. Information on national limit values can be found in the National Annex for Eurocode 5.

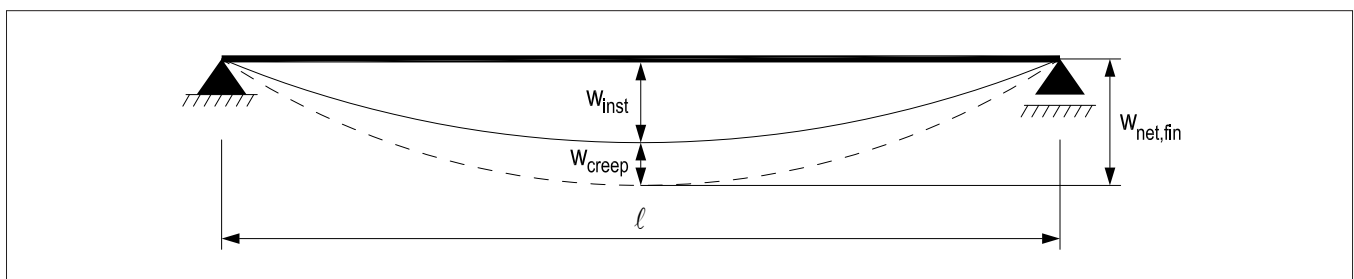


Figure 4.27 Components of deflection of LVL members.

Table 4.10. Example limiting values for beam deflection.

	w_{inst}	$w_{net,fin}$
Beam on two supports	$l/300$ to $l/500$	$l/250$ to $l/350$
Cantilevering beams	$l/150$ to $l/250$	$l/125$ to $l/150$

Deflection due to bending moment and shear should be taken into account for all load-bearing timber products. As an example, the deflection of a single-span beam under uniformly distributed load is calculated from equation:

$$w = \frac{5 \cdot q_{d,i,SLS} \cdot L^4}{384 \cdot E_{mean} \cdot I} + \frac{\zeta \cdot q_{d,i,SLS} \cdot L^2}{8 \cdot G_{mean} \cdot A} \quad (4.74)$$

and for a point load in the middle of the span

$$w = \frac{F_{d,i,SLS} \cdot L^3}{48 \cdot E_{mean} \cdot I} + \frac{\zeta \cdot F_{d,i,SLS} \cdot L^2}{4 \cdot G_{mean} \cdot A} \quad (4.75)$$

where

$q_{d,i,SLS}$ is the design value of a uniformly distributed action in serviceability limit state [N/mm];

$F_{d,i,SLS}$ is the design value of a point load in serviceability limit state [N/mm];

L is the span of the beam [mm];

I is the moment of inertia of the LVL cross section [mm⁴];

A is the cross-sectional area of the LVL beam [mm²];

ζ is the shear deformation factor, for rectangular cross section $\zeta = 6/5$

E_{mean} is the mean value of the modulus of elasticity of the LVL class [N/mm²]; and

G_{mean} is the mean value of the modulus of rigidity of the LVL class [N/mm²].

Instructions for the deflection calculation of other loading and span configurations can be found, e.g. from general handbooks on mechanics or can be calculated with FEM calculation software.

Note: In the EN standards modulus of elasticity E is defined as the local value, E_{local} , which does not include shear deflection. Therefore shear deflection needs to be calculated separately, see equation (4.75) and (4.76). Another way to define modulus of elasticity is the global value, E_{global} , in which shear deflection is included. In edgewise bending of LVL the value of E_{global} is about 5-7% smaller than E_{local} , but its use makes the calculation easier, because separate calculation of shear deformation is not needed. E_{global} is commonly used in e.g. Australia and USA. Another name for E_{global} is $E_{apparent}$ and another name for E_{local} is E_{true} .

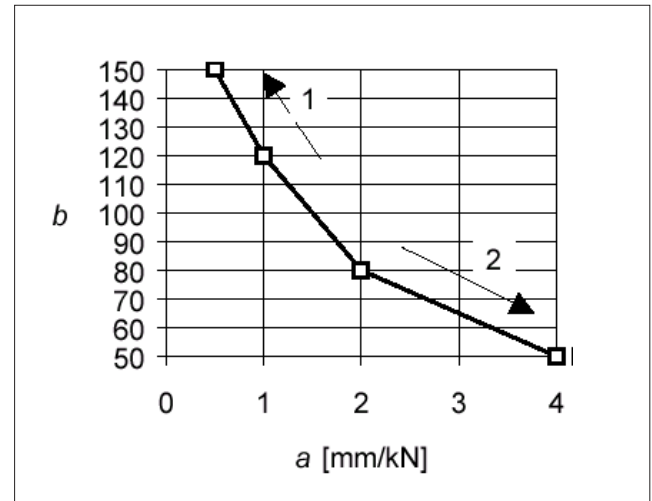


Figure 4.28. Recommended range of and relationship between a and b. Performance improves in the arrow 1 direction and decreases in the arrow 2 direction (EC5 Figure 7.2).

4.3.14 Serviceability limit state design: Floor vibrations

Eurocode 5, Section 7.3.3, gives requirements and some instructions for the design of residential floors. However, most national annexes deviate significantly from these.

Wooden floor structures can be divided into high frequency floors and low frequency floors based on their lowest fundamental frequency.

For residential floors with a fundamental frequency greater than 8 Hz ($f_1 > 8$ Hz), the following requirements should be satisfied:

$$\frac{w}{F} \leq a \text{ [mm/kN]} \quad (4.76) \text{ (EC5 7.3)}$$

and

$$v \leq b^{(f_1 \xi - 1)} \text{ [m/Ns}^2\text{]} \quad (4.77) \text{ (EC5 7.4)}$$

where

w is the maximum instantaneous vertical deflection caused by a vertical concentrated static force F applied at any point on the floor, taking account of load distribution;

v is the unit impulse velocity response, i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response. Components above 40 Hz may be disregarded; and

ζ is the modal damping ratio.

Values for factors a and b can be chosen from the diagram in Figure 4.28 depending on the desired performance level.

4. STRUCTURAL DESIGN OF LVL STRUCTURES

The calculations are made assuming that the floor is unloaded, i.e. only the mass of the floor and other permanent actions are accounted for. Note: in some National Annexes a part of the live load is also taken into consideration, e.g. in Finland 30 kg/m²³¹. For a rectangular floor with span l , the fundamental frequency f_1 may be approximately calculated as follows:

$$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_1}{m}} \quad (4.78) \text{ (EC5 7.5)}$$

where

m is mass per unit area [kg/m²];

l is the floor span [m]; and

$(EI)_1$ is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction calculated for 1 metre wide section [Nm²/m].

For a rectangular floor with overall dimensions $b \times l$, simply supported along all four edges, the value may, as an approximation, be taken as:

$$v = \frac{4 \cdot (0,4 + 0,6 n_{40})}{m \cdot b \cdot l + 200} \quad (4.79) \text{ (EC5 7.6)}$$

where

v is the unit impulse velocity response [m/Ns²];

n_{40} is the number of first-order modes with natural frequencies up to 40 Hz;

b is floor width [m];

m is the mass [kg/m²]; and

l is the floor span [m].

The value of n_{40} may be calculated from:

$$n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{b}{l} \right)^4 \frac{(EI)_l}{(EI)_b} \right\}^{0,25} \quad (4.80) \text{ (EC5 7.7)}$$

where $(EI)_b$ is the equivalent plate bending stiffness of the floor about an axis parallel to the beam direction calculated for a 1 metre wide section [Nm²/m] and $(EI)_b < (EI)_l$.

The deflection under $F=1$ kN point load can be calculated from equations:

$$w = \min \begin{cases} \frac{F \cdot l^2}{42 \cdot k_\delta \cdot (EI)_1} \\ \frac{F \cdot l^3}{48 \cdot s \cdot (EI)_1} \end{cases} \quad (\text{EC5 NA, Finland})^{31}$$

or

$$w = \frac{F \cdot l^2}{43,6 \cdot k_\delta \cdot (EI)_1} \quad (\text{EC5 NA, Austria})^{33}$$

where

s is the spacing of the floor beams [m]

$$k_\delta = \sqrt[4]{\frac{(EI)_b}{(EI)_1}} \quad (4.83)$$

with the limitation $k_\delta \leq b/l$

For multiple span floor additional instructions can be found, e.g., from the National annex of Austria³³.

For residential floors with a fundamental frequency less than 8 Hz ($f_1 \leq 8$ Hz) a special investigation should be made. Instructions for 4,5 Hz $\leq f_1 \leq 8$ Hz cases are defined, e.g., in the National Annexes of Austria³³ or Germany. In practice, their requirements can be fulfilled only when the own weight of the floor is >250 kg/m², which is quite heavy for an LVL floor structure.

4.4 COMBINED CROSS SECTIONS

4.4.1 Basic principles

Glued composite cross sections utilize the joints between members, significantly increasing the stiffness and resistance of the whole cross section compared to the members acting separately. This composite action can be calculated for a mechanically jointed cross section, but the influence of joint slip must then be taken into consideration and, therefore, the overall stiffness is much lower. The specific properties of the composite cross section that are essential to the structural analysis – effective stiffness EI_{eff} , normal stresses from bending moment, and shear stress at the glued joints – can be defined according to equations (4.84) – (4.88).

The effective stiffness EI_{eff} of a glued composite cross section is calculated according to equation:

$$EI_{\text{eff}} = \sum_i E_i I_i + E_i A_i e_i^2 \quad (4.84)$$

where

EI_{eff} is the effective stiffness of the composite cross section [Nmm²];

E_i is the modulus of elasticity of a part i [N/mm²];

I_i is the moment of inertia of a part i [mm⁴], for rectangular cross section $I_i = b_i \cdot h_i^3 / 12$, where b_i is the width [mm] of the part and h_i is the height [mm] of the part;

A_i is the cross-sectional area of a part i [mm²]; and

e_i is the eccentricity of the part i = distance between the centre of gravity of part i and neutral axis of the entire composite cross section [mm].

The location of the neutral axis of a composite cross section related to the bottom of the section is:

$$e_0 = \frac{\sum_i E_i \cdot A_i \cdot a_i}{\sum_i E_i \cdot A_i} \quad (4.85)$$

where

a_i is the distance between the centre of gravity of part i and the bottom of the entire composite cross section [mm].

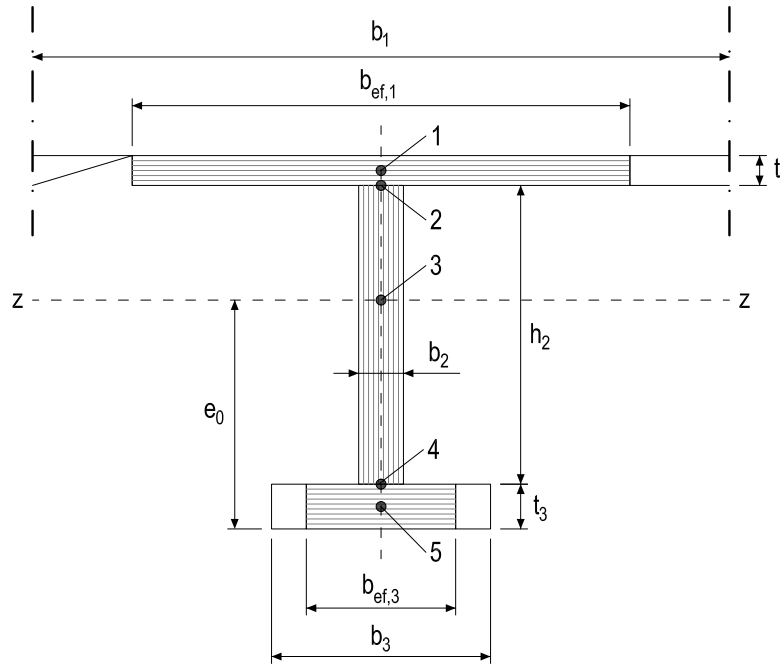


Figure 4.29. Composite cross section. In thin-flanged beams axial stresses are checked at points 1, 3 and 5. Shear stresses are checked at points 2, 3 and 4.

Normal stress from bending moment is calculated for composite cross sections according to the equation:

$$\sigma_{i,d}(z) = \frac{E_i \cdot e_{(z)i} \cdot M_d}{EI_{eff}} \quad (4.86)$$

where

$\sigma_{i,d}$ is the design value of normal stress at coordinate z in the section [N/mm²];

E_i is the modulus of elasticity of a part i [N/mm²];

$e_{(z)i}$ is the coordinate z of the point i where the stress is analysed = distance to the neutral axis of the entire composite cross section [mm];

M_d is the design value of the bending moment at the evaluated location of the member [Nmm]; and

EI_{eff} is the effective stiffness of the composite cross section [Nmm²].

Shear stresses at the glued joints of composite cross sections are calculated according to equation:

$$\tau_{(z)d} = E_i \cdot \frac{S_{(z)} \cdot V_d}{EI_{eff} \cdot b_{(z)}} \quad (4.87)$$

where

$\tau_{(z)d}$ is the design value of the shear stress at coordinate z in the section [N/mm²];

E_i is the modulus of elasticity of a part i [N/mm²];

$S_{(z)}$ is the static moment at coordinate z [mm³];

V_d is the design value of shear force at the evaluated location of the member [Nmm];

EI_{eff} is the effective stiffness of the composite cross section [Nmm²];

$b_{(z)}$ is the width of the section at coordinate z [mm];

$$S_{(z)} = \sum_i A_i \cdot e_{(z)i} \quad (4.88)$$

A_i is the cross-sectional area of a part i [mm²]; and

$e_{(z)i}$ is the coordinate z of the point i where the stress is analysed = distance to the neutral axis of the entire composite cross section [mm].

4. STRUCTURAL DESIGN OF LVL STRUCTURES

4.4.2 Beams and columns

When LVL-P or LVL-C members are glued together in the same direction as rectangular GLVL cross sections, the material values of the LVL lamina may be used. Additionally, the size effect shall be taken into account for the shear strength and bending strength flatwise to the LVL lamellas. The size effect parameters are declared in manufacturers' technical documentation.

LVL I-beams and box beams have LVL-P flanges and LVL-C webs which keep the beams straight and stable in changing moisture conditions. Glued thin-webbed I-beam and box beam specific design instructions are given in Eurocode 5 clause 9.1.1. For the flanges it shall be verified that:

- Axial tension stress $\sigma_{f,t,max}$ at the extreme fibre of the flange edge does not exceed the bending strength $f_{m,d}$ of the flange material
- Axial tension stress $\sigma_{f,t,d}$ at the centre of the flange does not exceed the tension strength $f_{t,0,d}$ of the material
- Axial compression stress $\sigma_{f,c,max}$ at the extreme fibre of the flange edge does not exceed the bending strength $f_{m,d}$ of the flange material
- Axial compression stress $\sigma_{f,c,d}$ at the centre of the flange does not exceed the compression strength $f_{c,0,d}$ of the material taking into consideration lateral instability

For the web material it shall be verified that:

- Compressive and tension stresses $\sigma_{w,c,max}$ and $\sigma_{w,t,max}$ do not exceed the respective strengths $f_{w,t,0,d}$ and $f_{w,c,0,d}$ of the web material
- Design shear force acting on the web $F_{v,w,Ed}$ does not exceed the shear capacity $R_{v,w,d}$ of the beam taking into consideration buckling as a plate

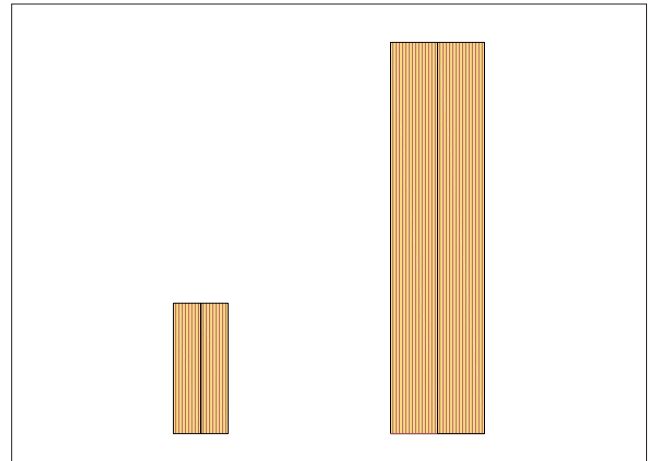


Figure 4.30. Multiple-glued GLVL beams.

For glued joints between web(s) and flanges (section 1-1 in Figure 4.31), it should be verified by equation (9.10) of Eurocode 5 that the shear stress $\tau_{mean,d}$ does not exceed the planar shear strength of the flange or web materials.

In SLS design, bending and shear deformations are taken into account. For simplicity, the shear rigidity GA may be evaluated based on the webs only.

LVL I-beam and box beam suppliers also have more advanced design instructions tailored and approved for their elements as part of their technical documentation.

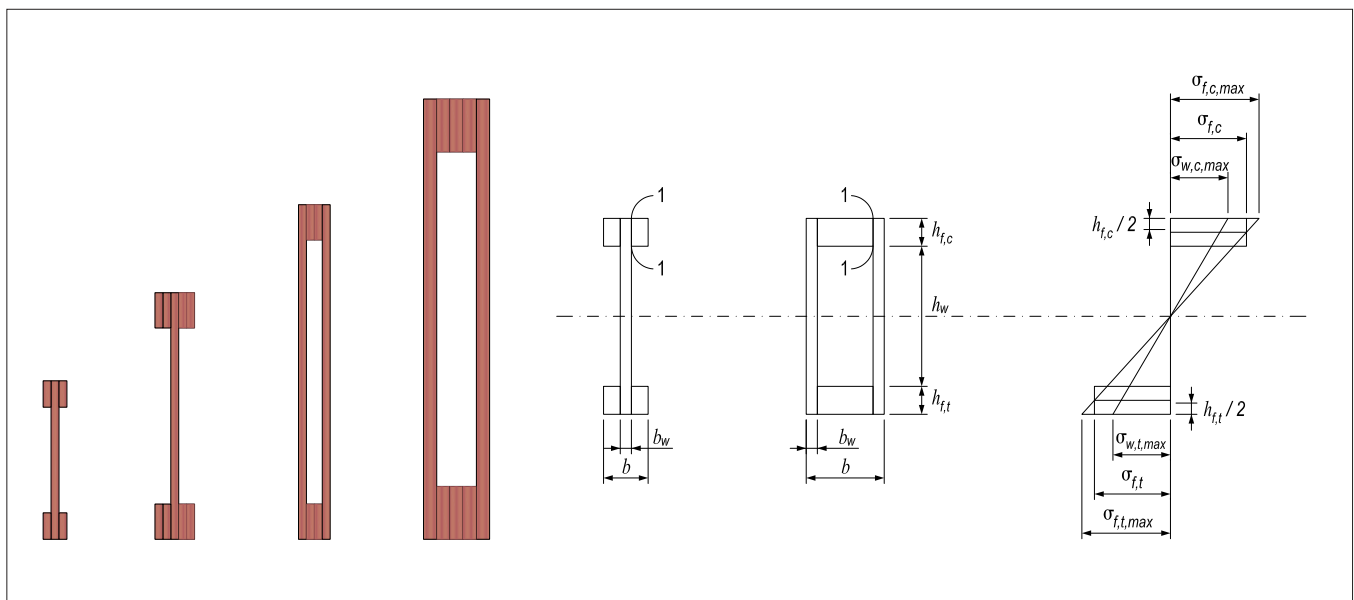


Figure 4.31. GLVL I-beams, box beams and stress distribution in the cross section. Section 1-1 is glued joint between web and flange.

4.4.3 Multiple-glued GLVL panels

In GLVL-C panels the mechanical properties are declared as homogenized values for the cross section assuming the cross veneers as zero layers. The cross layers are thin and a gamma method of effective cross section is not needed for multiple-glued GLVL-C panels when the LVL-C layers are all in the same direction. They can be designed based on the mechanical values of the LVL components with the addition that a size effect shall be taken into account for the flatwise bending strength and flatwise shear strengths. The reference sizes and size effect parameters s , $s_{\text{flat,m}}$ and $s_{\text{flat,v}}$ are declared in manufacturers' technical documentation.

4.4.4 Rib slabs and box slabs

LVL rib slab and box slab elements have LVL-P ribs and LVL-C panels that distribute the loads to the ribs and work as thin flanges of the composite cross sections. Structural design is made separately for each rib section. Their specific design instructions are given in Eurocode 5, clause 9.1.2. The effective width b_{ef} of the flange panel that can be taken into account in the calculations is limited due to shear lag and due to plate buckling in ULS design on the compression side according to Table 4.11.

In ULS design the following verifications are required:

- The mean axial compression stress $\sigma_{f,c,d}$ and tension stress $\sigma_{f,t,d}$ shall be smaller than $f_{t,c,d}$ and $f_{t,t,d}$ strengths respectively. Since the flange panels are thin, it is not required to verify the resistance against tension and compression stresses at the extreme fibres of the flanges.
- Resistance against shear stress for the ribs at the neutral axis and with the equation (9.14) of Eurocode 5 for the glued joints between the ribs and flange panels. The critical material property is typically the flatwise shear strength $f_{v,\text{flat},0,d}$ of the flanges due to the rolling shear of the cross veneers in LVL-C.
- Resistance against bending and shear stress of the flange panels in the perpendicular direction of the slab

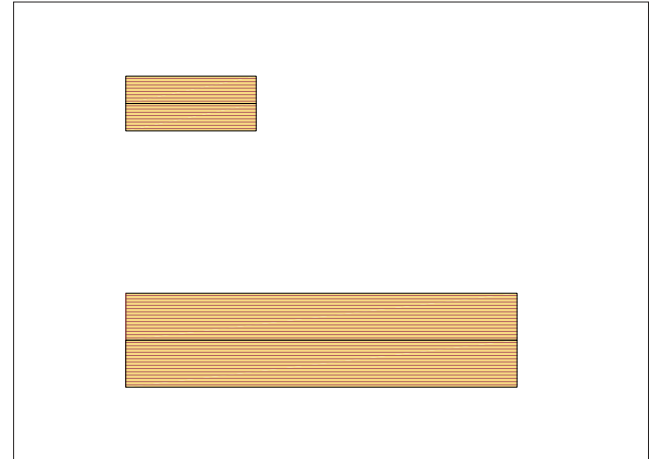


Figure 4.32. Multiple-glued GLVL panels.

Table 4.11. Maximum effective width b_{ef} of flange LVL-C panel parallel to the ribs (Modified from EC5 Table 9.1).

	Shear lag	Plate buckling in ULS design
I-section	$b_{\text{ef}} = b_w + 0,1 \cdot l$	$b_{\text{ef}} = b_w + 20 \cdot h_{f,c}$
C-section	$b_{\text{ef}} = b_w + 0,05 \cdot l$	$b_{\text{ef}} = b_w + 10 \cdot h_{f,c}$
Where b_w is rib thickness, l is span length and $h_{f,c}$ is the thickness of the flange panel on the compression side.		

In SLS design, bending and shear deformations are taken into account. For simplicity, the shear rigidity GA may be evaluated based on the ribs only. In that case, the k_{def} factor of LVL-P can also be used for the LVL-C flange panels in the flatwise direction in the calculation of final deformation, as the thin flanges are mainly axially loaded.

Rib slab suppliers also have more advanced design instructions tailored and approved for their elements as part of their technical documentation.

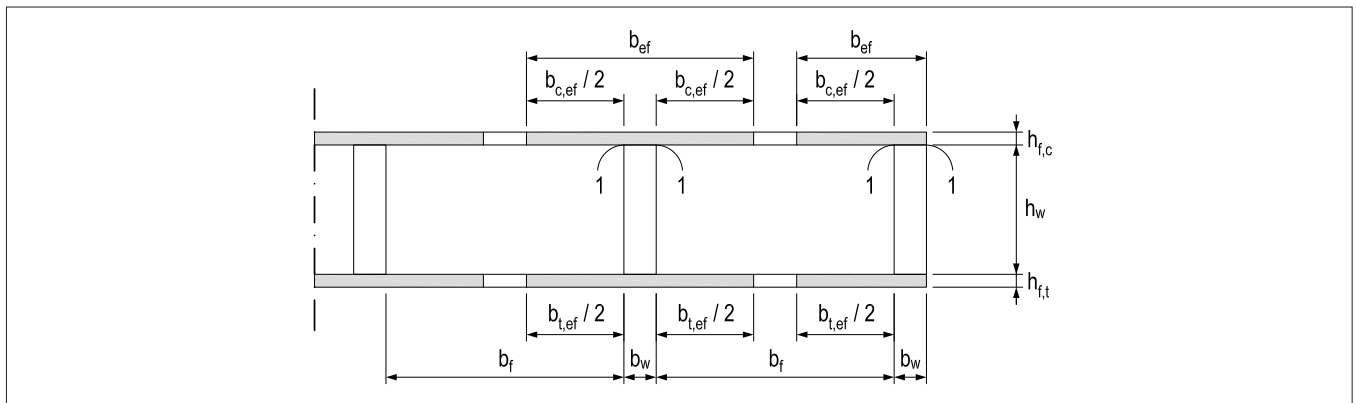


Figure 4.33. Definitions of the parts of rib slab part (EC5, Figure 9.2). I-section is used for the intermediate rib and C-section for the edge rib of the element.

5. Structural design of connections

Stora Enso

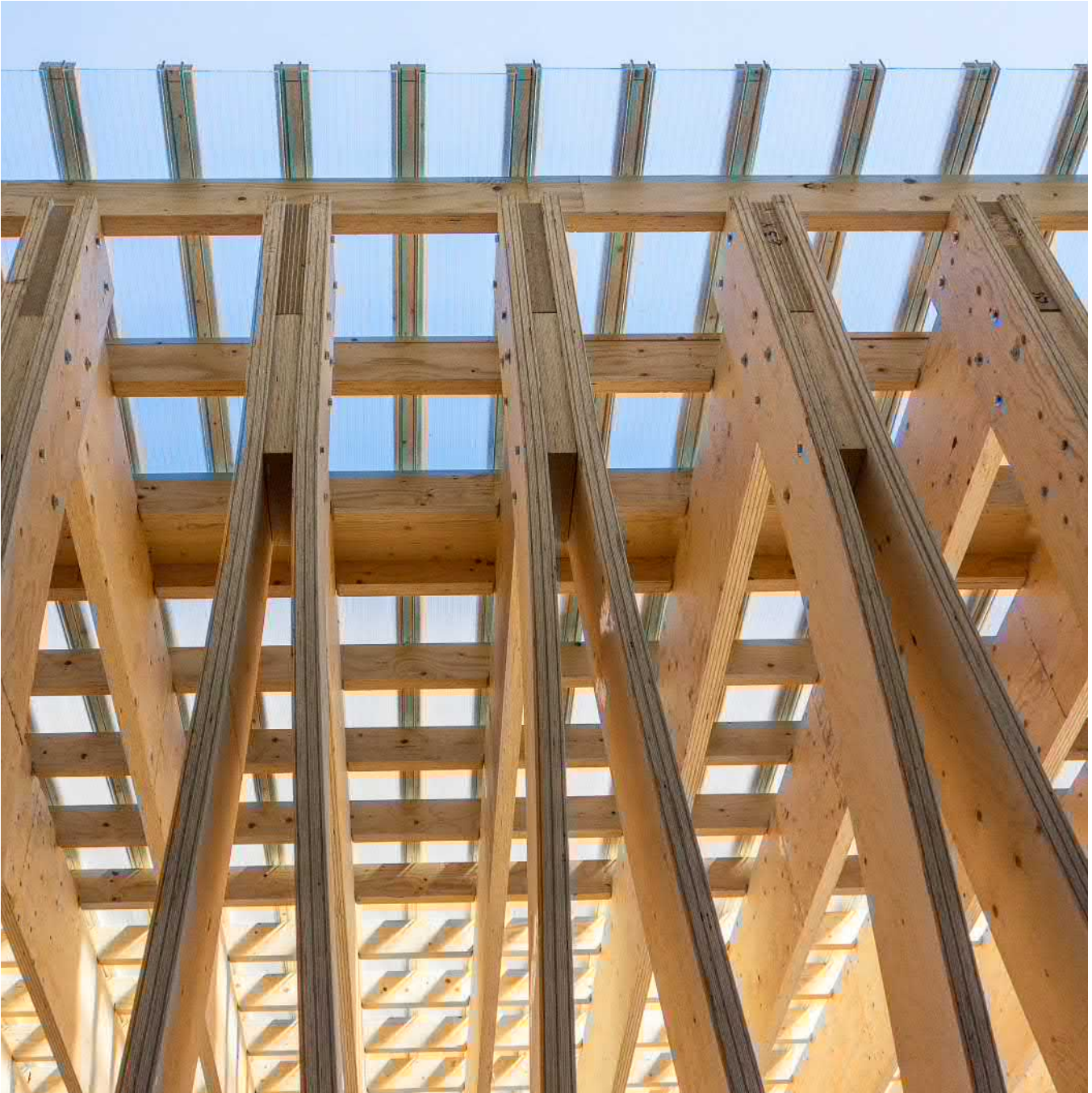


Figure 5.1. LVL connections.

5.1 DESIGN OF LVL CONNECTIONS ACCORDING TO EUROCODE 5

The design of connections with metal fasteners is specified in Section 8 of Eurocode 5. However, the design instructions do not fully cover LVL products. This section therefore provides additional definitions for LVL based on the following documents:

- Design rules for LVL to Eurocode 5, Proposal for discussion in CEN/TC250/SC5, Prof. Dr.-Ing. H.J. Blaß and Dr. -Ing.M. Flaig, Blaß & Eberhart GmbH, 30.6.2017, CEN/TC250/SC5/N0764 ³²
- Finnish Handbook for Eurocode 5 (RIL205-1:2017, Puurakenteiden suunnitteluohje) ³¹
- Connection chapters of Metsä Wood Kerto Manual ^{34, 35}

The equations in this Section have individual numbers and when an equation is in the same form in Eurocode 5, it is marked with EC5 and the equation reference number there, e.g., (EC5 8.2).

For the connection design of LVL members it is essential to note that the behaviour of the product may differ depending on whether the connections are on the wide face (flatwise) or on the edge face (edgewise) of the LVL, see Figure 5.2. The edge face of LVL is more sensitive to splitting, which must be taken into consideration when determining the geometry and maximum fastener sizes of connections. LVL-C, on the other hand, has the advantage of ductile behaviour of connections on the wide face due to its cross veneers, thus eliminating a number of wood failure modes related to connections and enabling denser groups of fasteners.

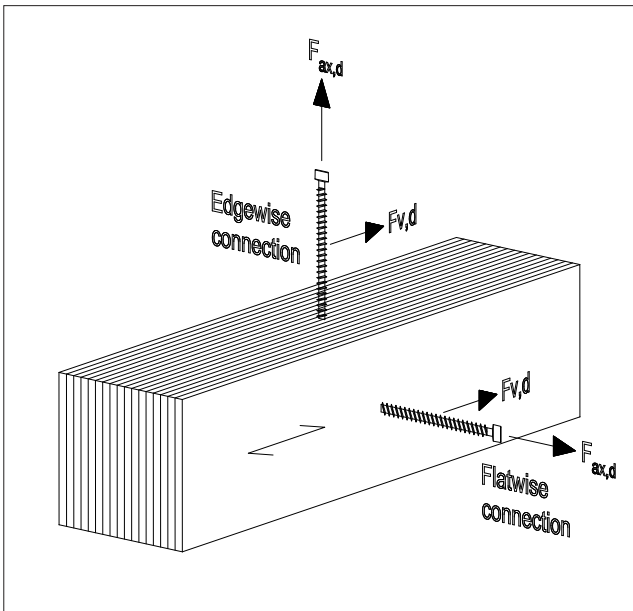


Figure 5.2. Edgewise (edge face) and flatwise (wide face) orientations and loading types of connections. $F_{ax,d}$ are forces of axially loaded and $F_{v,d}$ are forces of laterally loaded connections ³⁴.

A connection may be laterally loaded, axially loaded, or both laterally and axially loaded. For the determination of the characteristic lateral load-carrying capacity of connections with metal dowel-type fasteners, the contributions of the yield strength, embedment strength, and withdrawal strength of the fastener shall be considered. Section 8.2 of Eurocode 5 provides equations for calculating the characteristic load-carrying capacity of nails, staples, bolts, dowels and screws for different failure modes according to the Johansen yield theory. The minimum value, based on the relevant failure modes for the connection type, should be taken as the capacity per shear plane per fastener.

Fastener type specific instructions for calculating the embedment strength $f_{h,k}$, yield moment $M_{y,Rk}$ and axial withdrawal capacity $F_{ax,Rk}$, which are needed for the capacity calculation of laterally and axially loaded connections, are defined in sections 8.3-8.7 of Eurocode 5. Alternatively, fastener suppliers provide their own design instructions for connections using their products in their ETA assessment documents.

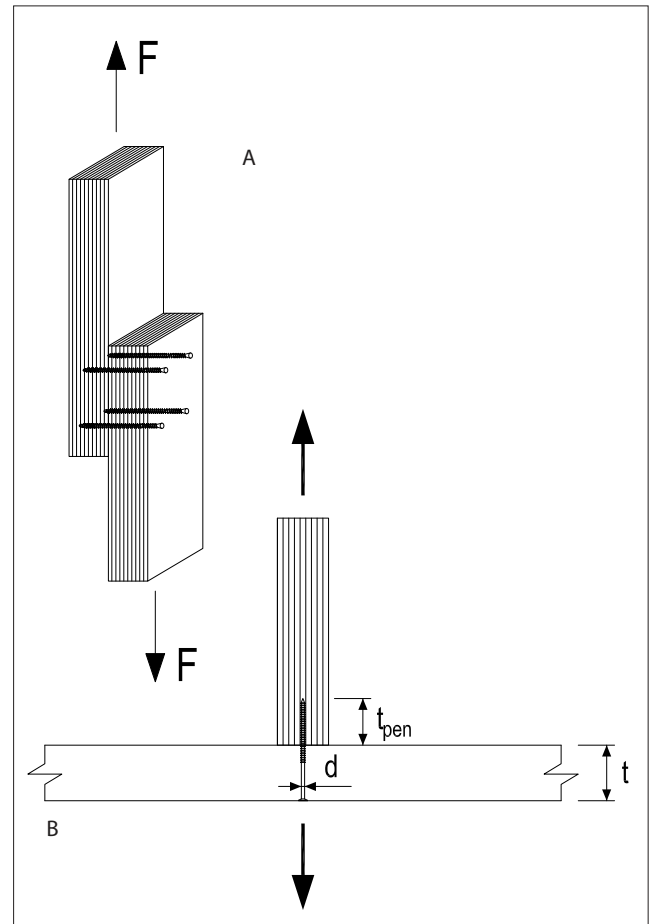


Figure 5.3. A) Laterally loaded connection ³⁴ B) Axially loaded connection.

5.2 FASTENER SPACING AND EDGE AND END DISTANCES

The rules for fastener connection geometry without predrilling of LVL-P on the wide face (flatwise connections) are similar to solid wood. In LVL-C connections, the spacing between fasteners as well as end and edge distances can be smaller because the product is not sensitive to splitting, due to its cross veneers. Connections at the edge face (edgewise), however, have a risk of splitting and require larger fastener spacing and end and edge distances for both LVL-P and LVL-C. Predrilling reduces the risk of splitting and smaller spacing and end and edge distances can be used.

Figures 5.4–5.7 describe the spacings a_1 , a_2 , end distances $a_{3,c}$, $a_{3,t}$, edge distances $a_{4,c}$, $a_{4,t}$ and angles α , β , and ϵ in different cases of laterally loaded connections. Their values for different faces of LVL-P and LVL-C products are given in:

- Table 5.1 for nails and screws with outer thread diameter < 12 mm
- Table 5.3. for bolts and screws with outer thread diameter ≥ 12 mm
- Table 5.4 for dowels

Figures 5.6–5.7 and Table 5.1 give the definitions and values for axially loaded screw connections.

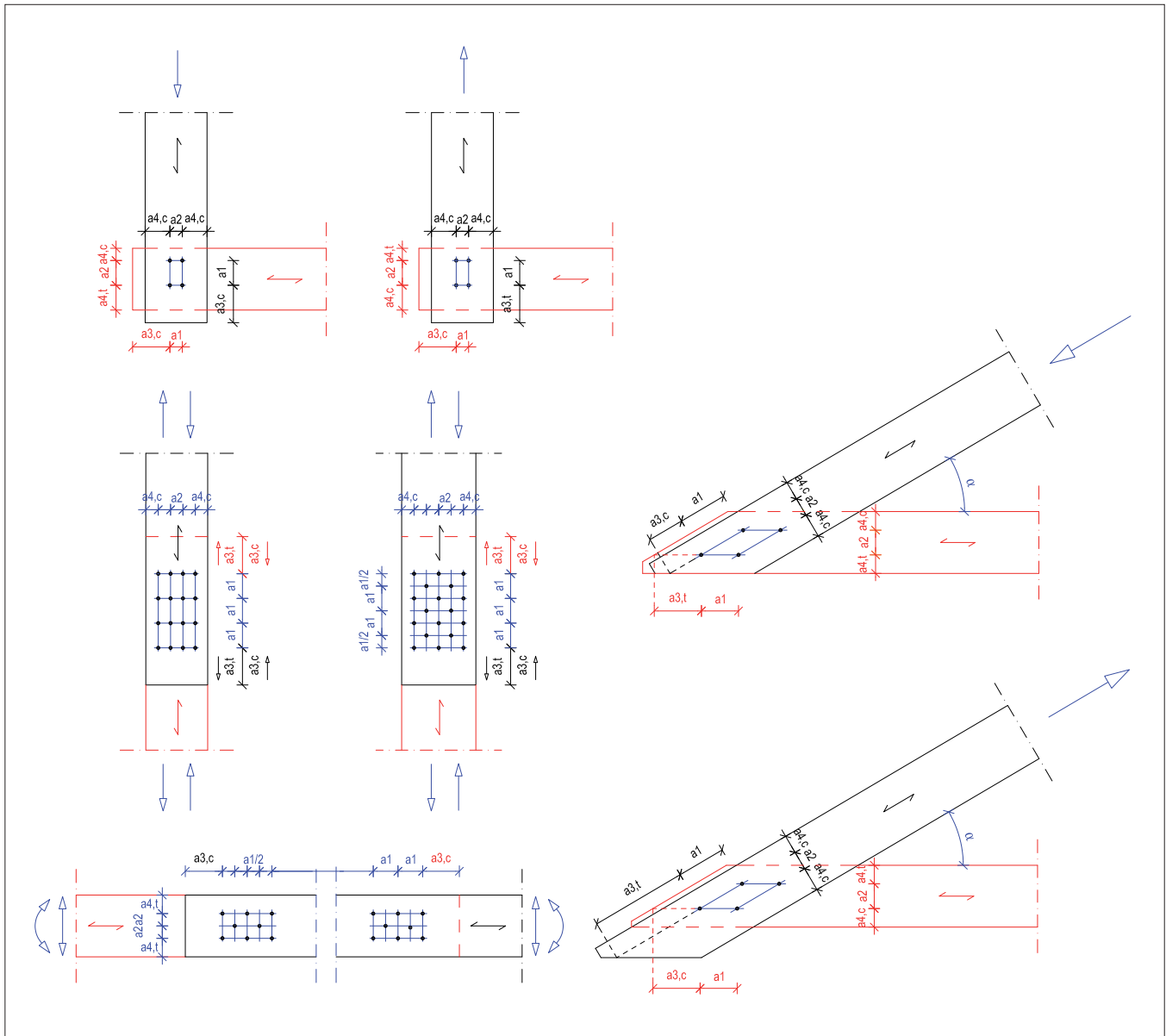


Figure 5.4. Spacings (a_1 and a_2), end distances ($a_{3,t}$ and $a_{3,c}$) and edge distances ($a_{4,t}$ and $a_{4,c}$) for fasteners ³⁴.

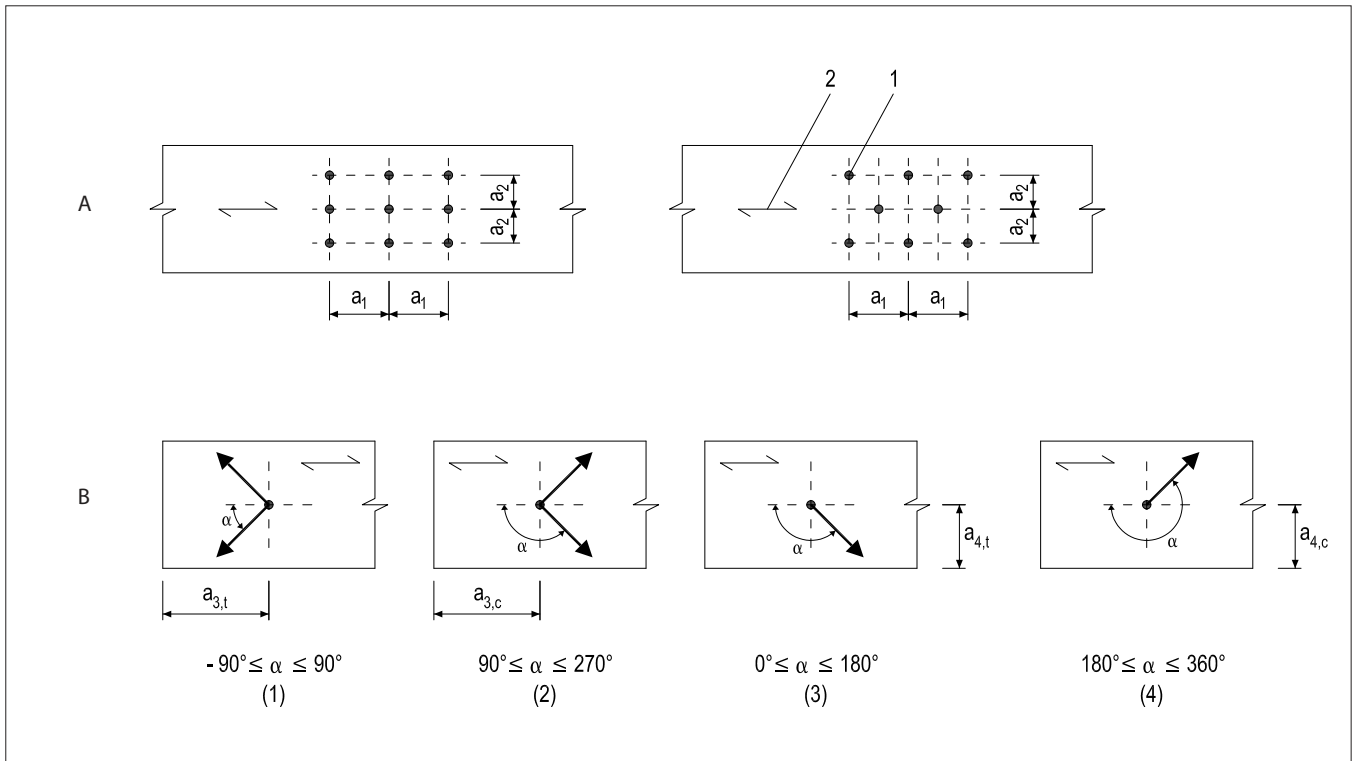


Figure 5.5. Spacings and end and edge distances: (A) Spacing parallel to grain in a row and perpendicular to grain between rows; (B) Edge and end distances; (1) Loaded end, (2) Unloaded end, (3) Loaded edge, (4) Unloaded edge; 1 Fastener, 2 Grain direction (EC5 figure 8.7).

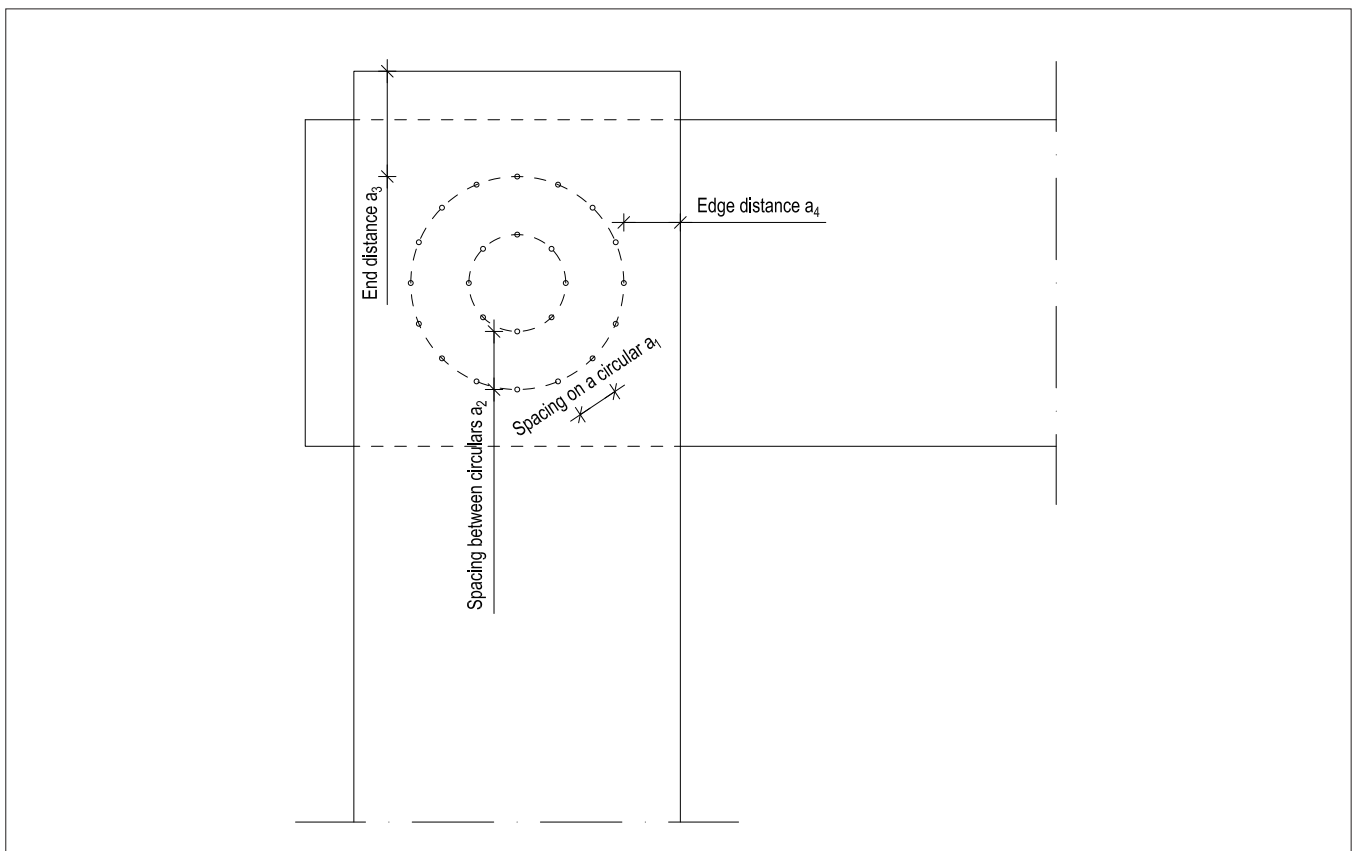


Figure 5.6. Moment-resisting multi-shear LVL-to-LVL flatwise connections with circular patterns of fasteners.

5. STRUCTURAL DESIGN OF CONNECTIONS

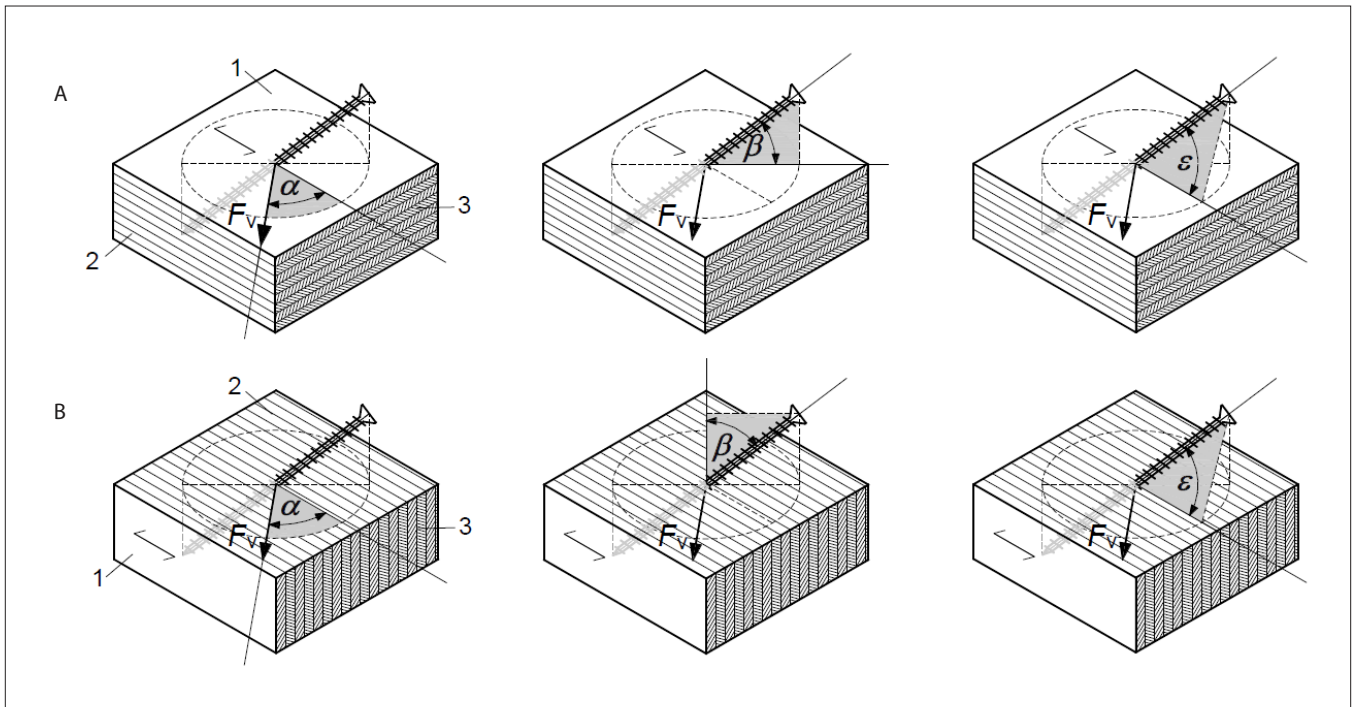


Figure 5.7. Definitions of angles α , β and ϵ for screws (A) in the wide face and (B) in the edge face of LVL. 1) wide face, 2) edge face and 3) end grain. α is the angle between the load and grain direction of laterally loaded connections. β is the angle between the screw axis and wide face. ϵ is the angle between screw axis and grain direction ³².

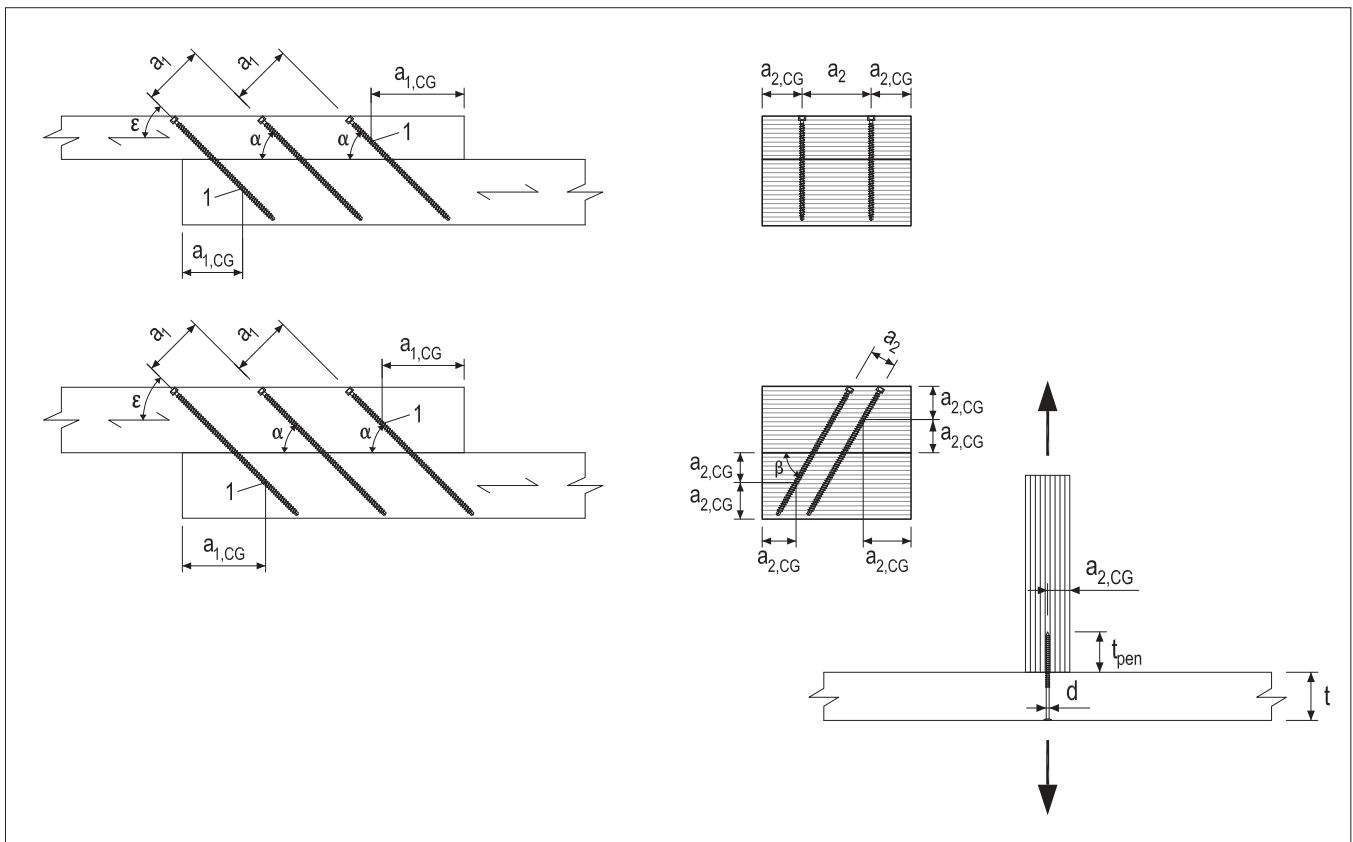


Figure 5.8. Spacings and end and edge distances and definitions of angles α , β and ϵ for axially loaded screws in LVL. α is the angle between the shear plane and screw axis. β is the angle between the screw axis and wide face. ϵ is the angle between screw axis and grain direction. (modified from EC5 Figure 8.11a).

Table 5.1. Minimum spacings, end distances and edge distances for nails and screws with outer thread diameter <12 mm ³².
 Note: EN 1995-1-1:2004 (Eurocode 5) has a limit of $d_{ef} < 6$ mm which corresponds with 9 mm outer thread diameter.

Laterally loaded connections Spacing or distance, see Figures 5.4-5.5 and 5.7	Angle α	Minimum spacing or end/edge distance			
		without predrilled holes			with predrilled holes
		LVL or GLVL wide face	LVL or GLVL edge face	LVL-C or GLVL-C wide face when pointside penetration length of at least $10d$ ^{a)}	
Spacing a_1 (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$d < 5$ mm: $(5 + 5 \cos \alpha) d$ $d \geq 5$ mm: $(5 + 7 \cos \alpha) d$	$(7 + 8 \cos \alpha) d$	$5 + 2 \cos \alpha d$	$(4 + \cos \alpha) d$
Spacing a_2 (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$5d$	$(3 + \sin \alpha) d$
Distance $a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$(10 + 5 \cos \alpha) d$	$(15 + 5 \cos \alpha) d$	$(4 + 3 \cos \alpha) d$	$(7 + 5 \cos \alpha) d$ ^{b)}
Distance $a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha \leq 270^\circ$	$10d$	$15d$	$5d$	$7d$ ^{c)}
Distance $a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$d < 5$ mm: $(5 + 2 \sin \alpha) d$ $d \geq 5$ mm: $(5 + 5 \sin \alpha) d$	$d < 5$ mm: $(7 + 2 \sin \alpha) d$ $d \geq 5$ mm: $(7 + 5 \sin \alpha) d$	$(3 + 4 \sin \alpha) d$	$d < 5$ mm: $(3 + 2 \sin \alpha) d$ $d \geq 5$ mm: $(3 + 4 \sin \alpha) d$
Distance $a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$3d$	$3d$
Axially loaded screws Spacing or distance, see Figures 5.7-5.8					
Spacing a_1 (parallel to grain)		$7d$	$10d$	$7d$	$7d$
Spacing a_2 (perpendicular to grain)		$5d$	$5d$	$5d$	$5d$
Minimum end distance of the centre of gravity of the threaded part of the screw in the member $a_{1,CG}$		$10d$	$12d$	$10d$	$10d$
Minimum edge distance of the centre of gravity of the threaded part of the screw in the member $a_{2,CG}$		$4d$	$4d$	$4d$	$4d$

^{a)} When pointside penetration length is less than $10d$, the rules in the column LVL or GLVL wide face apply
^{b)} for LVL-C or GLVL-C wide face and pointside penetration length of at least $10d$: $(4 + 3 \cos \alpha) d$
^{c)} for LVL-C or GLVL-C wide face and pointside penetration length of at least $10d$: $5d$

5. STRUCTURAL DESIGN OF CONNECTIONS

Table 5.2. Maximum nail and screw size d [mm] for edge face (edgewise) connections

LVL thickness	Laterally loaded connections		Axially loaded connections	
	Without pre-drilling $a_{4,c} \geq 7d$	With predrilling $a_{4,c} \geq 3d$	$a_{2,CG} \geq 4d$	
	Nails and screws	Nails	Screws	Screws
27 mm	1,9	4,5	4,5	3,4
33 mm	2,4	5,5	5,5	4,1
39 mm	2,8	6,5	6,5	4,9
45 mm	3,2	7,5	7,5	5,6
51 mm	3,6	8,0	8,5	6,4
57 mm	4,1	8,0	9,5	7,1
63 mm	4,5	8,0	10,5	7,9
69 mm	4,9	8,0	11,5	8,6
75 mm	5,4	8,0	12,0	9,4

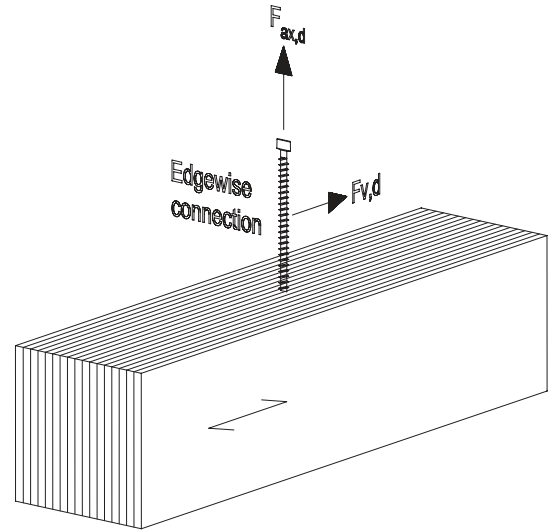


Table 5.3. Minimum spacings, end distances and edge distances for bolts and screws with max outer thread diameter >12 mm with predrilled holes ³². Note: EN 1995-1-1:2004 (Eurocode has a limit of $\text{def} < 6$ mm which corresponds with 9 mm outer thread diameter).

Spacing or distance, see Figures 5.4-5.5 and 5.7	Angle α	Minimum spacing or end/edge distance	
		LVL-P / GLVL-P or LVL-C / GLVL-C edge face	LVL-C / GLVL-C wide face
Spacing a_1 (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(4 + 3 \cos \alpha) d^a$	$4d$
Spacing a_2 (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$4d$	$4d$
Distance $a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$\max(7d; 105 \text{ mm})^b$	$\max(4d; 60 \text{ mm})^c$
Distance $a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$	$(1 + 6 \sin \alpha) d$	$4d$
	$150^\circ \leq \alpha < 210^\circ$	$4d$	$4d$
	$210^\circ \leq \alpha \leq 270^\circ$	$(1 + 6 \sin \alpha) d$	$4d$
Distance $a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$\max[(2 + 2 \sin \alpha) d; 3d]$	$\max[(2 + 2 \sin \alpha) d; 3d]$
Distance $a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$3d$	$3d$

^{a)} minimum spacing a_1 may be reduced to $5d$ if $f_{h,0,k}$ is multiplied by $\sqrt{a_1 / (4 + 3 |\cos \alpha|) d}$

^{b)} minimum end distance $a_{3,t}$ may be reduced to $7d$ for $d < 15$ mm if $f_{h,0,k}$ is multiplied by $a_{3,t} / 105$ mm

^{c)} minimum end distance $a_{3,t}$ may be reduced to $4d$ for $d < 15$ mm if $f_{h,0,k}$ is multiplied by $a_{3,t} / 60$ mm

Spacing and end/edge distances on circular patterns for double shear moment-resisting connections, see Figure 5.6	Minimum spacing or distance		
	LVL-P, GLVL-P wide face	LVL-C / GLVL-C wide face	Side member LVL-C / GLVL-C wide face Middle member LVL-P, GLVL-P or LVL-C wide face
a_1 (spacing on circle)	$6d$	$4d$	$5d$
a_2 (spacing between circles)	$5d$	$4d$	$5d$
$a_{3,t}$ (loaded end)	$6d$	$4d$	$6d$ in middle member $4d$ in side member
$a_{4,t}$ (loaded edge)	$4d$	$3d$	$4d$ in middle member $3d$ in side member

Table 5.4. Minimum spacings, end distances and edge distances for 6-30 mm dowels ³².

Spacing or distance, see Figures 5.4-5.7	Angle α	Minimum spacing or end/edge distance	
		LVL-P / GLVL-P or LVL-C / GLVL-C edge face	LVL-C / GLVL-C wide face
Spacing a_1 (parallel to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$(4 + 3 \cos \alpha) d$ ^{a)}	$(3 + \cos \alpha) d$
Spacing a_2 (perpendicular to grain)	$0^\circ \leq \alpha \leq 360^\circ$	$3d$	$3d$
Distance $a_{3,t}$ (loaded end)	$-90^\circ \leq \alpha \leq 90^\circ$	$\max(7d; 105 \text{ mm})$ ^{b)}	$\max(4d; 60 \text{ mm})$ ^{c)}
Distance $a_{3,c}$ (unloaded end)	$90^\circ \leq \alpha < 150^\circ$ $150^\circ \leq \alpha < 210^\circ$ $210^\circ \leq \alpha \leq 270^\circ$	$a_{3,t} \cos \alpha $ $3d$ $a_{3,t} \cos \alpha $	$(3 + \cos \alpha) d$
Distance $a_{4,t}$ (loaded edge)	$0^\circ \leq \alpha \leq 180^\circ$	$\max[(2 + 2 \sin \alpha) d; 3d]$	$\max[(2 + 2 \sin \alpha) d; 3d]$
Distance $a_{4,c}$ (unloaded edge)	$180^\circ \leq \alpha \leq 360^\circ$	$3d$	$3d$

a) minimum spacing a_1 may be reduced to $5d$ if $f_{h,0,k}$ is multiplied by $\sqrt{a_1 / (4 + 3 |\cos \alpha|) d}$

b) minimum end distance $a_{3,t}$ may be reduced to $7d$ for $d < 15 \text{ mm}$ if $f_{h,0,k}$ is multiplied by $a_{3,t} / 105 \text{ mm}$

c) minimum end distance $a_{3,t}$ may be reduced to $4d$ for $d < 15 \text{ mm}$ if $f_{h,0,k}$ is multiplied by $a_{3,t} / 60 \text{ mm}$

5.3 WOOD FAILURE MODES OF CONNECTIONS

5.3.1 Risk of splitting due to connection forces at an angle to the grain of face veneers

When a force in a connection acts at an angle to the grain (see Figure 5.8) the possibility of splitting caused by the tension force component, $F_{Ed} \cdot \sin \alpha$, perpendicular to the grain, shall be taken into account. For timber, glulam and LVL-P the following should be satisfied:

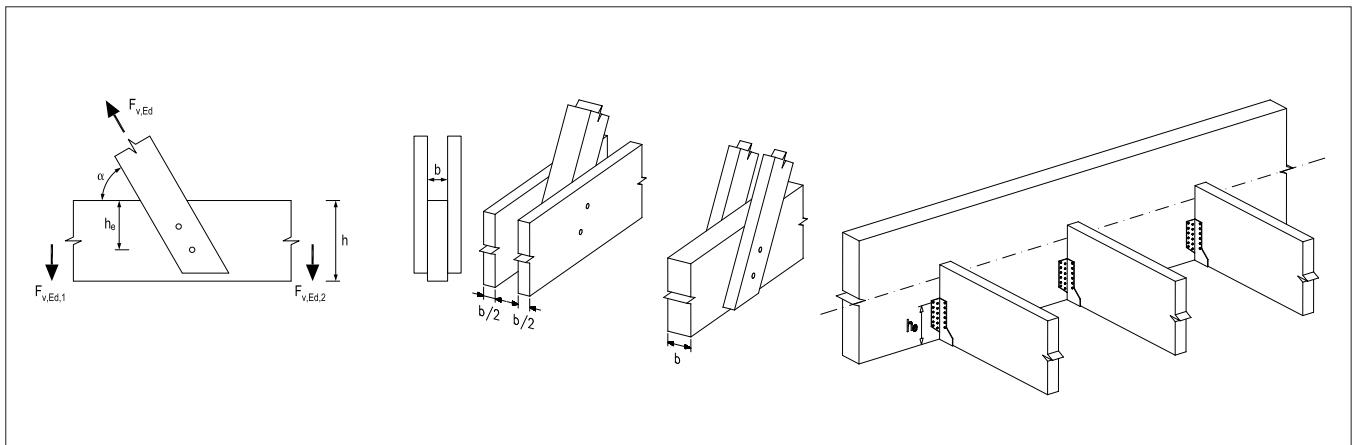
$$F_{v,Ed} \leq F_{90,Rd} \quad (5.1) \text{ (EC5 8.2)}$$

where

$F_{90,Rd}$ is the design splitting capacity;

$$F_{v,Ed} \leq \max \begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases} \quad (5.1) \text{ (EC5 8.3)}$$

$F_{v,Ed,1}$ and $F_{v,Ed,2}$ are the design shear forces on either side of the connection caused by the connection force component ($F_{Ed} \cdot \sin \alpha$) perpendicular to the grain.

**Figure 5.9.** Inclined force transmitted by a connection (modified from EC5 Figure 8.1)

5. STRUCTURAL DESIGN OF CONNECTIONS

Design splitting capacity is calculated from the characteristic splitting capacity according to equation (4.3), in subsection 4.1.6. For softwoods, the characteristic splitting capacity for the arrangement shown in Figure 5.9 should be taken as:

$$F_{90,k} = 14 \cdot b \sqrt{\frac{h_e}{(1 - \frac{h_e}{h})}} [N] \quad (5.3) \text{ (EC5 8.4)}$$

where

$F_{90,Rk}$ is the characteristic splitting capacity [N];
 h_e is the loaded edge distance to the centre of the most distant fastener [mm];
 h is the timber member height, [mm]; and
 b is the member thickness, however, not more than the penetration depth of the fasteners [mm].

Equation (5.3) does not need to be checked for wide face (flat-wise) LVL-C connections since it is not sensitive to splitting due to connection forces at an angle to the grain due to its cross veneers.

5.3.2 Effective number of fasteners to prevent splitting or row shear

An effective number of fasteners n_{ef} shall be used for bolt, dowel and $d > 12$ mm screw connections at tension-loaded ends of LVL members to prevent splitting or row shear failure mode. For one row of n_i fasteners parallel to the grain direction, the load-carrying capacity parallel to grain should be calculated using the effective number of fasteners n_{ef} according to the equation ³¹:

$$n_{ef} = \min \left\{ n_i, n_i^{0,9} \sqrt[4]{\frac{a \cdot t}{50 \cdot d^2}} \right\} \quad (5.4)$$

where

n_i is the number of fasteners in a row i ;
 d is the effective diameter of the fastener, for screws d_{ef} ;

$$a = \begin{cases} \min(a_1; a_3), & \text{when } n_i \geq 2 \\ a_3, & \text{when } n_i = 1 \end{cases} \quad (5.5)$$

a_1 is the spacing of fasteners in the grain direction;
 a_3 is the end distance of fasteners;

$$t = \begin{cases} \min(t_1; t_2) & \text{connection with timber only in outer members} \\ \min(2t_1; 2t_2; t_s) & \text{other two and multiple shear connection} \end{cases} \quad (5.6)$$

t_1 and t_2 are the thicknesses of outer timber members (disregarded if the outer member is not timber);

and

t_s is the thickness of the inner member of double shear connections or the smallest thickness of the inner member of a multiple shear connection.

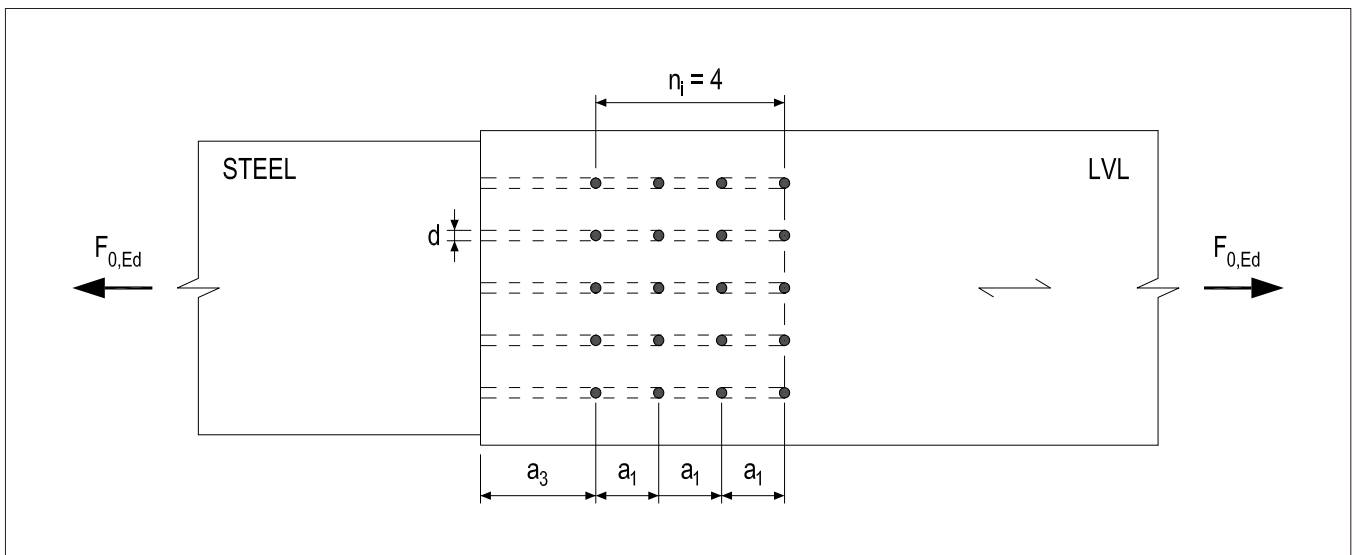


Figure 5.10. Row shear failure mode.

5.3.3 Block shear and plug shear failure modes at multiple dowel-type steel-to-timber connections

Block shear and plug shear failure modes shall be checked for steel-to-timber connections and tension-loaded member ends of double or multiple shear plane timber-to-timber connections. The wood failure capacity of the joint area can be calculated according to the method presented in the Finnish Handbook for EC5: RIL 205-1-2009, Section 8.2.4S³¹. In addition, the effective number of fasteners, n_{ef} , according to Section 5.3.2 are taken into account to prevent splitting and row shear failure mode. This method cannot be used for edgewise LVL connections.

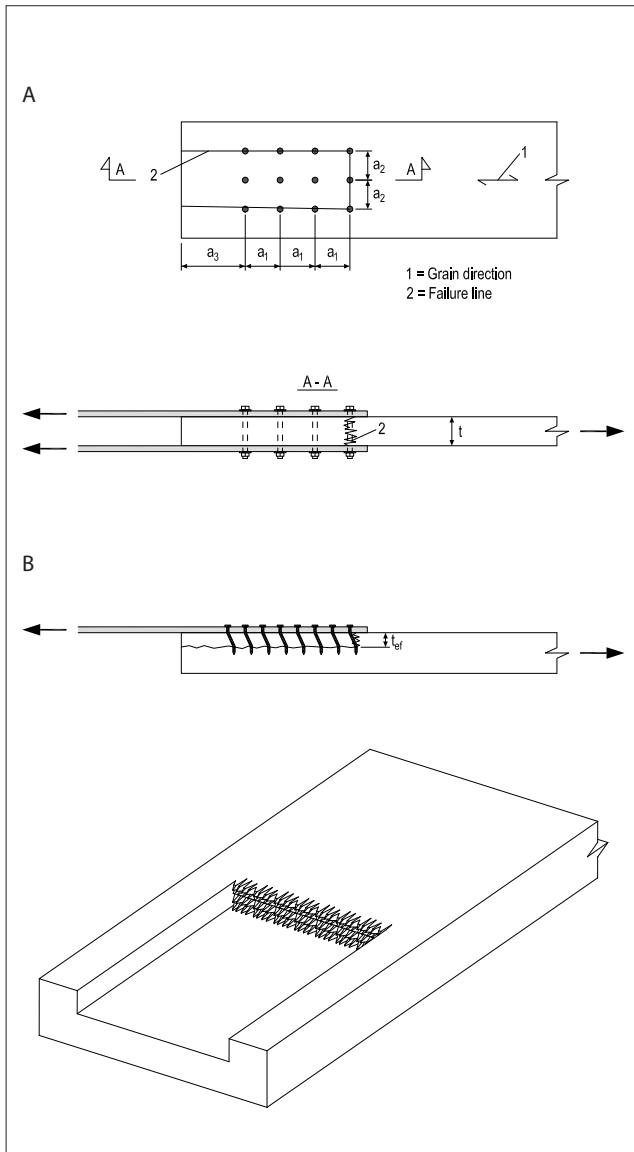


Figure 5.11. A) Block shear failure mode B) Plug shear failure mode³¹

Wood failure should be checked for at tension-loaded member ends for connection force components that are parallel to the grain. There are two types of timber failure mode: block shear and plug shear.

- Block shear needs to be checked for bolt and dowel connections and for screw connections when the centre member is screwed from both sides and the screws are overlapping.
- Plug shear failure mode must be checked for steel-to-timber connections with surface fasteners (nails, screws, nail plates and shear plates). Plug shear must be checked also for external lamellas in cases where the dowels are shorter than the overall thickness of the members in the connection. For LVL-C wide face bolt and dowel connections, both block and plug shear failure modes must be checked.
- Block and plug shear capacities are not checked for connections where all fasteners are in a single row parallel to the grain ($n_2 = 1$).
- If the timber member t_1 has fasteners from opposite sides and the effective thickness $t_{ef} \geq 0.5t_1$, the block shear capacity of steel-to-timber connections should also be checked.
- Block shear need not to be checked for bolt and dowel connections when:
 - the member thicknesses are $t_1 \geq 4d$, $t_s \geq 5d$ (inner member)
 - there are max. 4 fasteners in a row parallel to the grain, and
 - the distance perpendicular to grain between bolts $a_2 \geq 5d$ or between dowels $a_2 \geq 4d$, see Figure 5.11.

The characteristic plug shear capacity is calculated using the equation:

$$F_{ps,k} = L_{net,t} \cdot (t_{ef} \cdot f_{t,0,k} + (a_3 + (n_1 - 1) \cdot a_1 \cdot f_{v,0,k})) \quad (5.7)$$

where

$$L_{net,t} = (n_2 - 1) \cdot (a_2 - D) \quad (5.8)$$

$$t_{ef} = \frac{R_k}{d \cdot f_{h,0,k}} \quad (5.9)$$

$f_{h,0,k}$ is the embedment strength according to subsection 5.5.1 or 5.6;

n_1 is the number of fasteners in rows parallel to the grain;

n_2 is the number of fastener rows perpendicular to the grain;

a_1 is the fastener spacing parallel to the grain;

a_2 is the fastener spacing perpendicular to the grain;

a_3 is the fastener end distance;

D is the hole diameter;

$f_{t,0,k}$ is the tension strength of the timber member:

35 N/mm² for LVL 48 P and 19 N/mm² for LVL 36 C;

$f_{v,0,k}$ is the shear strength of the timber member: $f_{v,0,flat,k}$

is 2.3 N/mm² for LVL 48 P and 1.3 N/mm² for LVL 36 C in flatwise connections;

R_k is the characteristic load-carrying capacity per shear plane per fastener; and

d is the fastener diameter.

5. STRUCTURAL DESIGN OF CONNECTIONS

The characteristic block shear capacity of a timber member is calculated using the equation:

$$F_{bt,k} = L_{net,t} \cdot t_1 \cdot k_{bt} \cdot f_{t,0,k} \quad (5.10)$$

where

$L_{net,t}$ is according to equation (5.8)

k_{bt} is 1,50 for solid wood and glued laminated timber and 1,25 for LVL; and

t_1 is the thickness of the timber member ($t_1 \leq 2 t_{ef}$).

The characteristic block shear capacity of a LVL-C member is calculated using the equation:

$$F_{bt,k} = \max \left\{ \begin{array}{l} L_{net,t} \cdot t_1 \cdot f_{t,0,k} + 0,7 \cdot L_{net,v} \cdot t_1 \cdot f_{v,k} \\ L_{net,t} \cdot t_1 \cdot k_{bt} \cdot f_{t,0,k} \end{array} \right. \quad (5.11)$$

$$L_{net,v} = 2 \cdot (a_3 + (n_1 - 1) \cdot (a_1 - D)) \quad (5.12)$$

$f_{v,k}$ is edgewise shear strength, for LVL 36 C

$$f_{v,0,edge,k} = 4,5 \text{ N/mm}^2$$

LVL suppliers have also more advanced instructions in their technical documentation.

5.4 NAILED CONNECTIONS

Instructions for nailed connections are presented in Section 8.3 of Eurocode 5. In addition to those instructions, the following instructions shall also be taken into account:

- LVL members shall be predrilled when the diameter d of the nail exceeds 6 mm
- For nails with diameters up to 8 mm arranged perpendicular to the grain, the following characteristic embedment strengths apply³²:
 - without predrilled holes in softwood LVL or GLVL:

$$f_{h,k} = \frac{0,082 \cdot \rho_k \cdot d^{-0,3}}{k_C \cdot \cos^2 \beta + \sin^2 \beta} \quad (5.13)$$

- with predrilled holes in softwood LVL or GLVL

$$f_{h,k} = \frac{0,082 \cdot (1 - 0,01d) \cdot \rho_k}{k_C \cdot \cos^2 \beta + \sin^2 \beta} \quad (5.14)$$

where

ρ_k is the characteristic density, in kg/m^3 ;

β is the angle between nail axis and wide face; and

d is the nail diameter, in mm.

$$k_C \begin{cases} 1 & \text{for LVL - P and GLVL - P} \\ \min \left\{ \frac{d}{(d-2)} \right\} & \text{for LVL - C and GLVL - C} \end{cases} \quad (5.15)$$

- To prevent splitting failure mode, for one row of n nails parallel to the grain, unless the nails of that row are staggered perpendicular to grain by at least $1d$, the load-carrying capacity parallel to the grain (see EC5, clause 8.1.2(4)) should be calculated using the effective number of fasteners $n_{ef} = n^{k_{ef}}$. k_{ef} in Table 8.1 of Eurocode 5 applies to LVL-P wide face. For LVL-C wide face $k_{ef} = 1$ and for LVL or GLVL edge face:

$$k_{ef} = \min \left\{ 1 - 0,03(20 - a_1/d) \right. \quad (5.16)$$

- For smooth nails in predrilled holes in the edge face of LVL or GLVL the pointside penetration length should be at least $12d$
- LVL member thicknesses:

- LVL-P or GLVL-P with nails in the wide face should be pre-drilled when the thickness of the timber members is smaller than

$$t = \max \left\{ \begin{array}{l} 7d \\ (13d - 30) \frac{\rho_k}{400} \end{array} \right. \quad (5.17)$$

where

t is the minimum thickness of timber member to avoid pre-drilling [mm]

ρ_k is the characteristic timber density [kg/m^3]; and

d is the nail diameter [mm].

- Expression (5.17) may be disregarded for nails in the wide face of LVL-C or GLVL-C.
- LVL or GLVL with nails in the edge face should be pre-drilled when the thickness of the member in nailing direction is smaller than

$$t = \max \left\{ \begin{array}{l} 14d \\ (13d - 30) \frac{\rho_k}{200} \end{array} \right. \quad (5.18)$$

- Expression (5.17) may be replaced by expression (5.18) for edge distances $a_4 \geq 14d$
- In axially loaded nail connections for smooth nails without predrilled holes and with a pointside penetration of at least $12d$, the characteristic strength values of the withdrawal $f_{ax,k}$ and pull-through $f_{head,k}$ strengths [N/mm^2] should be found from the following expressions³²:

$$f_{ax,k} = 20 \cdot 10^{-6} \cdot \rho_k^2 \quad \text{for in the wide face of LVL and GLVL} \quad (5.19)$$

$$f_{ax,k} = 0,32 \cdot d + 0,8 \quad \text{for in the edge face of LVL and GLVL with } \rho_k \geq 480 \text{ kg/m}^3 \quad (5.20)$$

$$f_{head,k} = 70 \cdot 10^{-6} \cdot \rho_k^2 \quad (5.21)$$

where ρ_k is the characteristic density kg/m^3 . However, smooth nails shall not be used to resist permanent or long-term axial loading.

The following information should be taken from the nail supplier's DoP:

- Characteristic yield moment $M_{y,k}$ [Nmm]
- Characteristic withdrawal parameter $f_{ax,k}$ [N/mm²]
- Characteristic head out-through parameter $f_{head,k}$ [N/mm²]
- Characteristic tensile capacity $f_{tens,k}$ [kN]
- Nail diameter [mm]
- Nail head area [mm²]
- Nail length [mm]
- For threaded nails also length of threaded part (l_g) and length of point (l_p)

5.5 SCREWED CONNECTIONS

Design instructions for screwed connections are given in Eurocode 5, Section 8.7. The definitions provided in the current section introduce some differences to the Eurocode 5 instructions in order to improve the connection design of LVL. Screw suppliers also have their own design instructions for their fasteners documented in their ETAs and DoPs. These must be treated as separate supplier-specific instructions unless they make direct reference to Eurocode design.

The following information should be taken from the screw supplier's DoP:

- Characteristic yield moment $M_{y,k}$ [Nmm]
- Characteristic withdrawal parameter $f_{ax,k}$ [N/mm²]
- Characteristic head out-through parameter $f_{head,k}$ [N/mm²]
- Characteristic tensile capacity $f_{tens,k}$ [kN]
- Screw outer thread diameter d [mm]
- Screw inner thread diameter d_1 [mm]
- Screw head diameter d_h [mm]
- Screw length L [mm]
- Thread length L_G [mm]

5.5.1 Laterally loaded screws in LVL

The instructions specific to LVL presented in this subsection are based on the document *Design rules for LVL to Eurocode 5, Proposal for discussion in CEN/TC250/SC5*, Prof. Dr.-Ing. H.J. Bläß and Dr. -Ing.M.Flaig, Bläß & Eberhart GmbH, 30.6.2017, CEN/TC250/SC5/N0764³² and differ partially from EN1995-1-1:2004. The effect of the threaded part of the screw shall be taken into account in determining the load carrying capacity by using the yield moment capacity of the screw determined in accordance with EN 14592. The outer thread diameter d shall be used to determine the embedment strength, the spacing, edge and end distances and the effective number of screws.

Note: In EN 1995-1-1:2004 the embedment strength is defined based on an effective diameter d_{ef} instead of the outer thread diameter d .

For smooth shank screws, the yield moment capacity of the smooth shank may be used for plastic hinges occurring within the length of the smooth shank. Unless a detailed analysis is

made, the yield moment capacity of the smooth shank may be used, if the smooth shank penetrates into the member containing the point of the screw by not less than $4d$.

Unless otherwise stated in this section, the rules for bolts apply to screws with an outer thread diameter $d > 12$ mm in predrilled LVL / GLVL members. The embedding strength $f_{h,k}$ should be taken as:

$$f_{h,k} = \frac{0,082 \cdot (1 - 0,01d) \rho_k}{(k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha) \cdot (k_C \cdot \cos^2 \beta + \sin^2 \beta) \cdot (2,5 \cdot \cos^2 \varepsilon + \sin^2 \varepsilon)} \text{ N/mm}^2 \quad (5.22)$$

where

d is the outer thread diameter of the screw [mm];

ρ_k is the characteristic density [kg/m³];

α is the angle between load and grain direction, see Figure 5.7; for softwood LVL-C / GLVL-C and $\alpha > 45^\circ$, α may be assumed as 45° ;

β is the angle between screw axis and wide face, see Figure 5.7;

ε is the angle between screw axis and grain direction, see Figure 5.7;

k_{90} is 1,15 + 0,015 for softwood LVL / GLVL; and (5.23)

$$k_C = \max \left\{ \frac{d}{(d-2)}, 1,15 \right\} \text{ for softwood LVL} \quad (5.24)$$

Unless otherwise stated in this section, the rules for nails apply to screws with a diameter $d \leq 12$ mm or screws in non-predrilled timber or LVL / GLVL members. The embedding strength should be taken as:

- without predrilled holes in softwood LVL or GLVL

$$f_{h,k} = \frac{0,082 \cdot \rho_k \cdot d^{-0,3}}{(k_C \cdot \cos^2 \beta + \sin^2 \beta) \cdot (2,5 \cdot \cos^2 \varepsilon + \sin^2 \varepsilon)} \text{ N/mm}^2 \quad (5.25)$$

- with predrilled holes in softwood LVL or GLVL

$$f_{h,k} = \frac{0,082 \cdot (1 - 0,01d) \rho_k}{(k_C \cdot \cos^2 \beta + \sin^2 \beta) \cdot (2,5 \cdot \cos^2 \varepsilon + \sin^2 \varepsilon)} \text{ N/mm}^2 \quad (5.26)$$

where

$$k_C = \begin{cases} 1 & \text{for LVL - P and GLVL - P} \\ \min \left\{ \frac{d}{3}, \frac{d}{(d-2)} \right\} & \text{for LVL - C and GLVL - C} \end{cases} \quad (5.27)$$

5.5.2 Axially loaded screws

For connections in softwood timber or LVL/GLVL with $\varepsilon \geq 15^\circ$ of screws in accordance with EN 14592 with:

- $6 \text{ mm} \leq d \leq 12 \text{ mm}$
- $0,6 \leq d_1 / d \leq 0,75$

where

d is the outer thread diameter; and

d_1 is the inner thread diameter

The characteristic withdrawal capacity should be taken as ³²:

$$F_{ax,\varepsilon,Rk} = \frac{n_{ef} \cdot k_{ax} \cdot f_{ax,90,k} \cdot d \cdot l_{ef} \left(\frac{\rho_k}{\rho_a} \right)^{0,8}}{k_\beta} \quad (5.28)$$

where

$$k_{ax} = \begin{cases} 0,5 + \frac{0,5 \cdot \varepsilon}{45^\circ} & \text{for } 15^\circ \leq \varepsilon < 45^\circ \\ 1 & \text{for } 45^\circ \leq \varepsilon \leq 90^\circ \end{cases} \quad (5.29)$$

$$k_\beta = 1,5 \cdot \cos^2 \beta + \sin^2 \beta \quad (5.30)$$

$F_{ax,\varepsilon,Rk}$ is the characteristic withdrawal capacity of the connection at an angle ε to the grain [N];

$f_{ax,90,k}$ is the characteristic withdrawal strength perpendicular to the grain determined in accordance with EN 14592 for the associated density ρ_a [N/mm²];

n_{ef} is the effective number of screws, $n_{ef} = n^{0,9}$ where n is the number of screws acting together in a connection;

k_{ax} is a factor to consider the influence of the angle ε between screw axis and grain direction and the long-term behavior;

l_{ef} is the penetration length of the threaded part [mm];

ρ_k is the characteristic density [kg/m³];

ρ_a is the associated density for $f_{ax,k}$ [kg/m³];

k_β is a factor considering the influence of the angle β between the screw axis and the LVL's wide face;

ε is the angle between the screw axis and the grain direction, with $\varepsilon \geq 15^\circ$, see Figure 5.7; and

β is the angle between the screw axis and the LVL's wide face, with $0^\circ \leq \beta \leq 90^\circ$, see Figure 5.7.

Note: Failure modes in the steel or in the timber around the screw are brittle, i.e. with minimal ultimate deformation and therefore have a limited possibility for stress redistribution.

For screws in LVL, the characteristic withdrawal parameter may be assumed as $f_{ax,90,k} = 15 \text{ N/mm}^2$, when $\rho_a = 500 \text{ kg/m}^3$ and screws $6 \text{ mm} \leq d \leq 12 \text{ mm}$ in softwood LVL/GLVL.

The characteristic pull-through resistance of connections with axially loaded screws should be taken as:

$$F_{ax,\varepsilon,Rk} = n_{ef} \cdot f_{head,k} \cdot d_h^2 \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \quad (5.31)$$

where

$F_{ax,\varepsilon,Rk}$ is the characteristic pull-through capacity of the connection at an angle ε to the grain [N], with $\varepsilon \geq 30^\circ$

$f_{head,k}$ is the characteristic pull-through parameter of the screw determined in accordance with EN 14592 for the associated density ρ_a

d_h is the diameter of the screw head [mm]

5.5.3 Inclined screw connections

Inclined screwing is an efficient way to connect LVL members together or to other types of timber members. Although the connections transfer shear forces, the fasteners are axially loaded. The instructions in this subsection are based on the Finnish Handbook RIL205-1:2017 for Eurocode 5, Chapter 8.7.4S ³¹.

These rules concern the design of single shear connections according to Figure 5.11, where the screw inclination angle α should be between $30^\circ \dots 60^\circ$ in regard to the shear plane. The screws are axially loaded. The head side timber member (t_1) may be replaced with a steel plate if the screw head has a full bearing area on the steel plate for a Figure 5.12 (b) tension screw connection. The screws should be self-drilling and fully threaded or partly threaded with a smooth part diameter of $d_s \leq 0,8d$, where d is the outer thread diameter.

Different or supplementary connection types and screw specifications differing from eurocode 5 may be used according to their ETA.

Cross screw connection

The cross screw connection is built up from symmetrical screw pairs, see Figure 5.12 (a), in which one screw is under compression and the other under tension. The characteristic load-carrying capacity of the cross screw connection is calculated by the equation:

$$R_k = n_p^{0,9} (R_{C,k} + R_{T,k}) \cos \alpha \quad (5.32)$$

where

n_p is the number of screw pairs in the joint; and

α is the angle between screw axis and the shear plane ($30^\circ \leq \alpha \leq 60^\circ$), see Figure 5.12 (a)

The characteristic compression capacity of the screw is calculated by the equation:

$$R_{C,k} = \min \begin{cases} f_{ax,\varepsilon,1,k} d l_{g,1} \\ f_{ax,\varepsilon,2,k} d l_{g,2} \\ 0,8 f_{tens,k} \end{cases} \quad (5.33)$$

The characteristic withdrawal capacity of the screw is calculated by the equation:

$$R_{T,k} = \min \begin{cases} f_{ax,\varepsilon,1,k} d l_{g,1} + f_{head,k} d_h^2 \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \\ f_{ax,\varepsilon,2,k} d l_{g,2} \\ f_{tens,k} \end{cases} \quad (5.34)$$

where

- $f_{ax,\varepsilon,1,k}$ is the characteristic withdrawal strength parameter for a screw at the head side member of the connection at an angle ε to the grain direction [N/mm²];
- $f_{ax,\varepsilon,2,k}$ is the characteristic withdrawal strength parameter for a screw at the pointside member of the connection at an angle ε to the grain direction [N/mm²];
- d is the outer threaded diameter [mm];
- $l_{g,1}$ is the penetration length of the threaded part in the head side member [mm];
- $l_{g,2}$ is the penetration length of the threaded part in the pointside member [mm];
- $f_{tens,k}$ is the characteristic tensile capacity of the screw determined in accordance with EN 14592 [N];
- f_{head} is the characteristic pull-through parameter of the screw for the associated density ρ_a [N/mm²];
- d_h is the head diameter [mm];
- ρ_k is the characteristic density of LVL [kg/m³]; and
- ρ_a is the associated density for $f_{head,k}$ [kg/m³].

When the screwing direction in the beam is $\varepsilon = 90^\circ$ to the grain direction (even though the angle β is inclined between the edge face and the wide face), it is not allowed to add the tension capacity of the head to the withdrawal capacity of the threaded part in the beam. Therefore the characteristic withdrawal capacity $R_{T,k}$ of the screw is calculated by the equation:

$$R_{T,k} = \min \left\{ \begin{array}{l} \max \left(f_{ax,90,1,k} d l_{g,1} ; f_{head,k} d_h^2 \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \right) \\ f_{ax,\varepsilon,2,k} d l_{g,2} \\ f_{tens,k} \end{array} \right. \quad (5.35)$$

The withdrawal strength $f_{ax,\varepsilon,k}$ is determined by testing according to EN 14592 and EN 1382 or it can be determined at angle ε to the grain as follows:

$$f_{ax,\varepsilon,k} = \frac{k_{ax} \cdot f_{ax,90,k}}{1,5 \cos^2 \beta + \sin^2 \beta} \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \quad (5.36) \quad ^{32}$$

where

- ε is the angle between the screw axis and the grain direction, $\varepsilon \geq 15^\circ$, see Figure 5.12;
- β is the angle between the screw axis and the LVL's wide face, with $0^\circ \leq \beta \leq 90^\circ$, see Figure 5.7
- k_{ax} in according to the equation (5.29)
- ρ_k is the characteristic density of LVL [kg/m³];
- ρ_a is the associated density for $f_{ax,k}$ [kg/m³]; and
- $f_{ax,90,k}$ is the characteristic withdrawal strength parameter for a screw perpendicular to the grain direction [N/mm²]. It is determined by testing according to EN 14592 and EN 1382 or for screws in LVL, it may be assumed as $f_{ax,90,k} = 15 \text{ N/mm}^2$ for $\rho_a = 500 \text{ kg/m}^3$ and screws $6 \text{ mm} \leq d \leq 12 \text{ mm}$ in softwood LVL/GLVL ³².

Tension screwed connection

In a joint consisting of only screws in tension, contact between the wood members is required. Tension screw connection

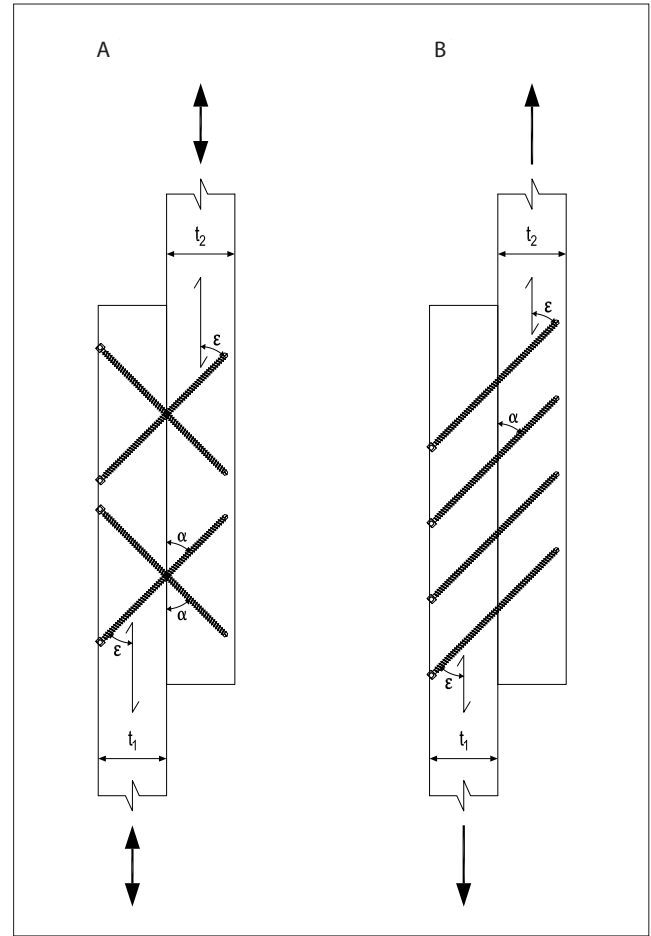


Figure 5.12. Inclined screwed connections (A) cross screw connection (B) tension screw connection

should not be used in conditions where wood drying could cause a gap of over $0.2d$. The gap is determined from the wood shrinkage at a thickness of the LVL members in the screw length ($L \cdot \sin \alpha$) ³¹.

The characteristic load-carrying capacity of the tension screw connection, see Figure 5.12 (b), is calculated by the equation:

$$R_k = n^{0,9} R_{T,k} (\cos \varepsilon + \mu \cdot \sin \varepsilon) \quad (5.37)$$

where

- n is the number of screws in the connection;
- $R_{T,k}$ is the characteristic withdrawal capacity, see (5.35);
- α is the angle between screw axis and the shear plane ($30^\circ \leq \alpha \leq 60^\circ$), see Figure 5.12 (b); and
- μ is the kinetic friction coefficient between the members, the following values may be used:
- 0.26 for untreated LVL edgewise or LVL to timber or timber-to-timber connections
 - 0.30 for steel-to-timber connections
 - 0.40 for untreated LVL flatwise connections

5. STRUCTURAL DESIGN OF CONNECTIONS

Connection detailing

Members should be pre-drilled when the diameter d of the screw exceeds 8 mm or the diameter of the smooth shank d_s exceeds 6 mm. The diameter of pre-drilled holes for non-self-drilling screws should be $D = 0.5d - 0.7d$, but not more than the inner diameter of the threaded part d_i .

The thickness of the member should be at least:

$$t = \max \left\{ \frac{5d}{(10d - 30) \frac{\rho_k}{400}} \right\} \quad (5.38)$$

where

ρ_k is the characteristic density [kg/m³]; and
 d is the screw diameter [mm].

General spacings and end and edge distances are presented in Table 5.1. These values are valid for cross screw connections when the compressed and tensioned fasteners of the screw pairs are placed in separate longitudinal rows parallel to the grain, so that the spacing between the rows a_2 is $4d$ and the staggered distance between the screw heads of a screw pair is not more than $3t_1$ parallel to the grain.

Different types or sizes of screws must not be combined in the same connection. All screws must be positioned at the same inclination angles ϵ and β in a member. The screws must be positioned centrally to the connection force and screwed deep enough so that the screw head is in full contact with the member surface. The minimum pointside penetration depth of the threaded part should be $6d$. The members should be compressed together so that no gaps are present ³¹.

LVL suppliers provide their own specific instructions for inclined screw connections with their products.

5.6 BOLTED AND DOWELED CONNECTIONS

For bolted and dowelled connections the design rules are defined for connections perpendicular to the grain of the face veneer. The design of LVL connections follows Eurocode 5 Sections 8.5 and 8.6 with the exception of the embedment calculation. The rules for connection geometry are given in Section 5.2 and the rules for wood failure modes in Section 5.3 of this handbook.

In laterally loaded connections of bolts and dowels up to 30 mm diameter arranged perpendicular to the grain, the following characteristic embedment strength values should be used for LVL ³²:

$$f_{h,k} = \frac{0,082 \cdot (1 - 0,01d)\rho_k}{(k_{90} \cdot \sin^2\alpha + \cos^2\alpha) \cdot (k_C \cdot \cos^2\beta + \sin^2\beta)} \quad (5.39)$$

where

ρ_k is the characteristic density [kg/m³];
 α is the angle between load and grain direction; for softwood LVL-C / GLVL-C and $\alpha > 45^\circ$, α may be assumed as 45° ;
 β is the angle between bolt axis and wide face;
 d is the bolt diameter [mm];
 k_{90} is $1,15 + 0,015$ for softwood LVL / GLVL; and

$$k_C = \max \left\{ \frac{d}{1,15}, \frac{d}{(d-2)} \right\} \quad \text{for softwood LVL / GLVL} \quad (5.40)$$

5.7 FASTENING LVL PANELS TO FRAMES

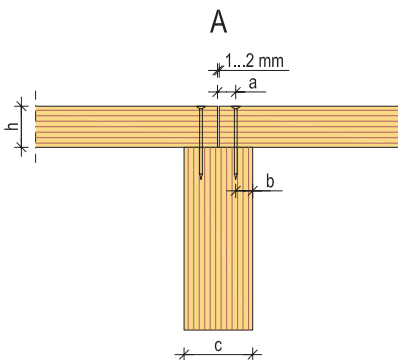
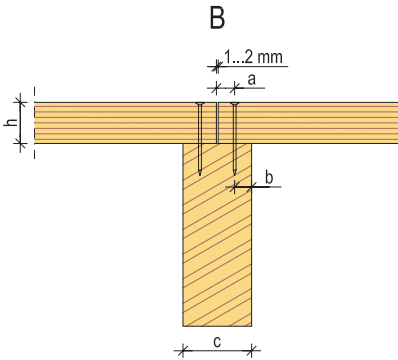
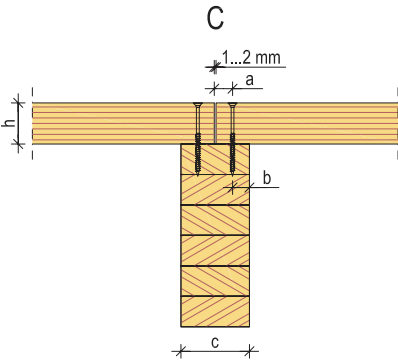
Connections of LVL-C panels can be designed according to Eurocode 5 instructions for timber-to-timber connections with commonly available fasteners. The connection capacities and minimum thickness of the frame depend on the panel, fastener and frame material combinations. Capacities and dimension for connections of LVL-C with LVL-P, glulam or solid timber frames are presented in Table 5.5.

5.8 SPECIAL CASES

Many nail plate manufacturers have tested the anchoring strength of their products for LVL-P members and the values are of similar levels to C30 structural timber. The values are included in their design software for trusses. LVL bottom cords are used in nail plate trusses to give additional stiffness for the floor part of attic trusses and fire resistance in cases where the rest of the trusses may be exposed to fire.

Usually the anchoring devices are fastened to bracing panels with laterally loaded connections to the wide face of the LVL panels. However, glued-in rod or glued-in screw connections on the edge face of LVL panels can be an efficient solution for anchoring large bracing panels. They however require separate type testing and production quality assurance and, in some countries, also separate product certification. Glued-in rod or glued-in screw connections at the edge face require a panel thickness of at least 66 mm to fulfil the edge distance requirement of the connection.

Table 5.5. Predesign table for the lateral load-carrying capacity of connection between an LVL 36 C panel and LVL-P, glulam GL30c or solid wood C24 frame; load-duration class between instantaneous and medium-term.

  						
A: LVL 36 C panels nailed with smooth round nails to LVL 48 P frame						
$h^{1)}$ [mm]	$d^{2)}$ [mm]	$L_{min}^{3)}$ [mm]	$\geq a^{4)}$ [mm]	$\geq b^{5)}$ [mm]	$\geq c^{6)}$ [mm]	$R_k^{7)}$ [N]
24	2,1	50	11	15	51	470
27	2,5	60	13	18	63	600
33	2,8	70	14	20	69	750
45	3,1	90	16	22	75	900
57	3,4	100	17	24	84	1000
69	4,2	125	21	30	102	1500
B: LVL 36 C panels nailed with smooth round nails to GL30c or C24 frame						
$h^{1)}$ [mm]	$d^{2)}$ [mm]	$L_{min}^{3)}$ [mm]	$\geq a^{4)}$ [mm]	$\geq b^{5)}$ [mm]	$\geq c^{6)}$ [mm]	$R_k^{7)}$ [N]
24	2,1	50	11	11	45	440
27	2,5	60	13	13	63	580
33	2,8	70	14	14	63	700
45	3,1	90	16	16	63	840
57	3,4	100	17	17	75	950
69	4,2	125	21	21	90	1350
C: LVL 36 C panels screwed to GL30c or C24 frame						
$h^{1)}$ [mm]	$d^{2)}$ [mm]	$L_{min}^{3)}$ [mm]	$\geq a^{4)}$ [mm]	$\geq b^{5)}$ [mm]	$\geq c^{6)}$ [mm]	$R_k^{7)}$ [N]
24	4,5	60	23	23	100	1100
27	5,0	70	25	25	115	1300
33	5,0	70	25	25	115	1300
45	6,0	90	30	30	140	1800
57	7,0	100	35	35	165	2400
69	8,0	120	40	40	165	3000

¹⁾ h [mm] is the thickness of the LVL 36 C panel

²⁾ d [mm] is the diameter of the nail or outer diameter of the threaded part of the screw

³⁾ L_{min} [mm] is the minimum length of the nail or screw

⁴⁾ a [mm] is the end distance ($5d$) of the panel, edge distance may be $3d$

⁵⁾ b [mm] is the edge distance ($7d$ for LVL edge face and $5d$ for solid wood or glulam) of the beam

⁶⁾ c [mm] is the minimum width of the beam

⁷⁾ R_k [N] is the characteristic lateral load-carrying capacity of the connection

6. Performance of LVL in fire

SP Trätek



Figure 6.1. Charring of wood.

6.1 WOOD AND FIRE

Wood and wood products, including LVL, are combustible materials. They start to burn at the surface at a temperature of 270 °C when exposed to flame. However, self-ignition does not occur at temperatures below 400 °C. According to structural fire design part of Eurocode 5 (EN1995-1-2), the start of charring is defined as the point at which the temperature of timber surface reaches 300 °C.

The resistance to fire behaviour of wood products is highly predictable and can be calculated according to the structural fire design specifications of Eurocode 5. Burning creates a char layer on the surface of wood products. The char layer acts as an insulation layer, which slows the burning and protects the rest of the cross section, see Figure 6.2. However, the high temperature prior to burning reduces the strength and stiffness properties of the wood even before charring, see Figure 6.3, which must be taken into account in structural fire design.

Reaction to fire requirements are specified for wood surfaces to control the risk of flame spread in buildings. They set boundary conditions for the use of visible wood in claddings and structures. In some cases fire retardant treatments or sprinkler systems can allow more visible wood structures to be used in architectural design.

6.2 REACTION TO FIRE

In the European classification system defined in EN 13501-1, the reaction to fire performance class of LVL is D-s2, d0. This class can be applied to LVL without further testing when the density is at least 400 kg/m³ and the product thickness is ≥ 18 mm (European Commission Delegated Regulation (EU) 2017/2293) ³⁷.

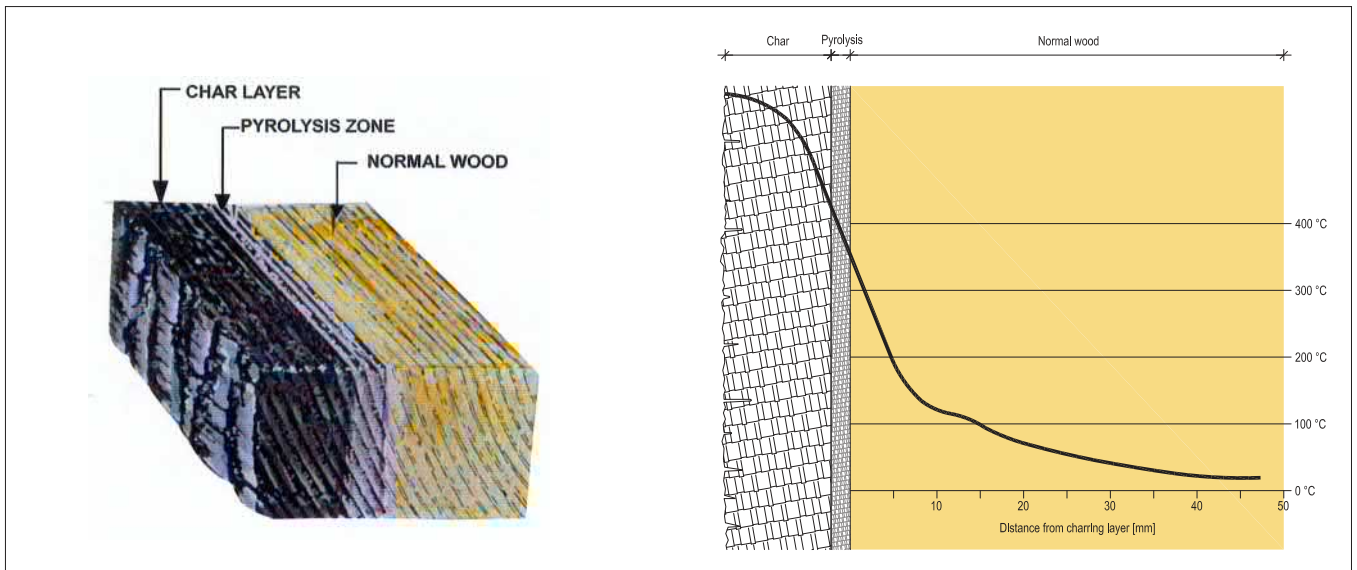


Figure 6.2. Temperature gradient in burning wood. The temperature drops significantly behind the charring layer. 15 mm from the charring zone the temperature is below 100 °C ³⁶.

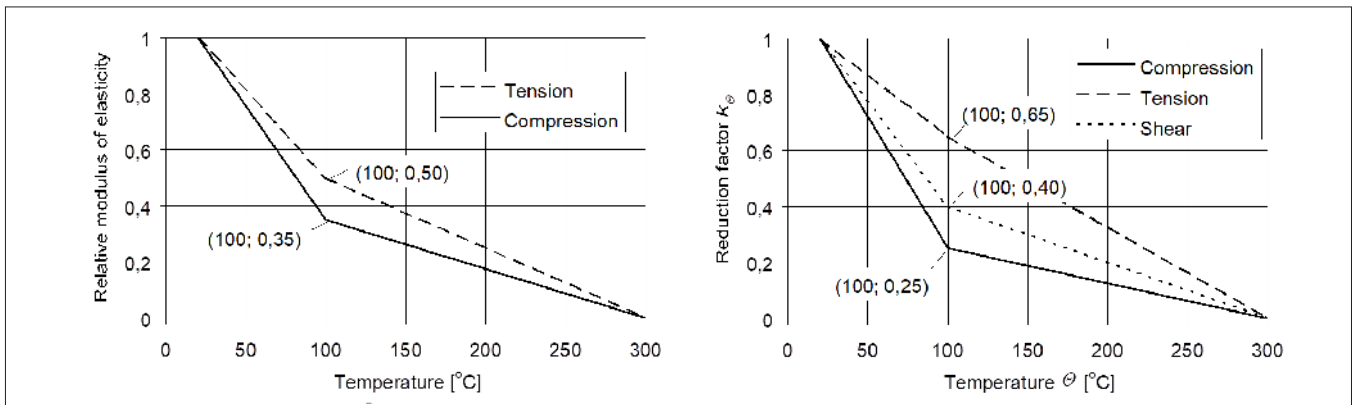


Figure 6.3. Influence of temperature on the mechanical properties of softwood. Left: Reduction of modulus of elasticity parallel to grain, Right: Reduction of strength parallel to grain (EN1995-1-2:2004, Figure B.4 and B.5).

(Schaffer, FPL, US 1967 and Brandsäkra trähus 3, SP Trä, Sweden 2012)

6. PERFORMANCE OF LVL IN FIRE

The requirements of reaction to fire classes for different applications are defined in national building regulations. In general, LVL panels and structures may be left exposed in a building under the same conditions as other solid wood products.

In the classification, the first letter A-E denotes combustibility, with D being typical for wood products. The second symbol, s1-s3, denotes smoke production and the third symbol, d0-d3, denotes the risk of flaming droplets. The class for LVL is similar to most untreated wood products. Some manufacturers specify for some applications a smoke production class of s1 based on separate testing, which is slightly better than the typical class s2. The flaming droplets class d0 assigned to wood-based products means that no flaming droplets or particles are produced. Although the European system has been in use since the beginning of 2000, national classification systems are still in parallel use in some countries.

The reaction to fire classification can be improved by fire retardant treatments or with inorganic surface laminates up to a class B-s1,d0, which is the highest class for combustible materials. The fire retardants are mainly salt-based chemicals that are usually hygroscopic, meaning that they absorb humidity from the surrounding air. It is therefore essential that the durability of the treatment is verified for the intended use class for the whole service life of the product.

LVL manufacturers provide fire retardant treatment as an additional service for their products, details of which can be found from their product information material.

6.3 FIRE PROTECTION ABILITY, K-CLASSES OF COVERINGS

The encapsulation requirements of fire protection ability classes K10 - K60 of coverings are used in building regulations to

protect underlying products against damage. The classes may be achieved by LVL panels, when the minimum thicknesses are according to the Table 6.1. However, in some applications only non-combustible materials are approved for the encapsulation.

Note: the same layer in a structure can give fire protection in accordance with the K-classes and at the same time be taken into account for fire resistance time of the structure.

6.4 FIRE RESISTANCE OF LVL STRUCTURES

The fire resistance of LVL structures can be calculated according to the structural fire design specification of Eurocode 5 (EN1995-1-2) and its National annexes. The model of the structural system adopted for design shall reflect the performance of the structure in the fire situation.

6.4.1 Fire resistance design process

The fire resistance design process has the following steps:

1. Determination of the charring depth

The charring depth is the distance between the outer surface of the original member and the position of the char line. It is calculated using the time of fire exposure and the relevant charring rate which depends on the material of the structure and possible additional protection layers on the structure. The original size reduced by the charring depth on exposed sides is defined as the residual cross section.

2. Determination of the effective cross section

Due to the increased temperature, the mechanical properties of wood reduce which is taken into consideration by

Table 6.1. Classes of fire protection ability performance of LVL – laminated veneer lumber according to EN14374. The table has been published as a draft annex of (EC Ref. Ares(2017)2463446 - 15/05/2017) pending formal issuing as a European Commission Delegated Regulation ³⁸.

Product ¹⁾	Product detail ²⁾	Minimum mean density [kg/m ³]	Minimum thickness of a board [mm]	Field of application	K Class
Laminated veneer lumber products covered by the harmonised standard EN 14374	With and without tongue and groove and a minimum layer thickness of 3 mm ⁴⁾	450	15	Wall and ceiling covering	K ₂ 10 ³⁾
	With and without tongue and groove and a minimum layer thickness of 3 mm ⁵⁾	450	26	Wall and ceiling covering	K ₂ 30
	With and without tongue and groove and a minimum layer thickness of 3 mm ⁶⁾	450	52	Wall and ceiling covering	K ₂ 60

¹⁾ Mounted directly on the substrate (particle board 680 kg/m³ according to EN 14135) without an airgap

²⁾ Joints with square edges or tongue and groove profile and with the same thickness as the wood product and without gaps.

³⁾ K₁ 10 for substrates ≥ 300 kg/m³

⁴⁾ Screw length minimum 30 mm and spacing maximum 200 mm

⁵⁾ Screw length minimum 50 mm and spacing maximum 200 mm

⁶⁾ Screw length minimum 75 mm and spacing maximum 200 mm

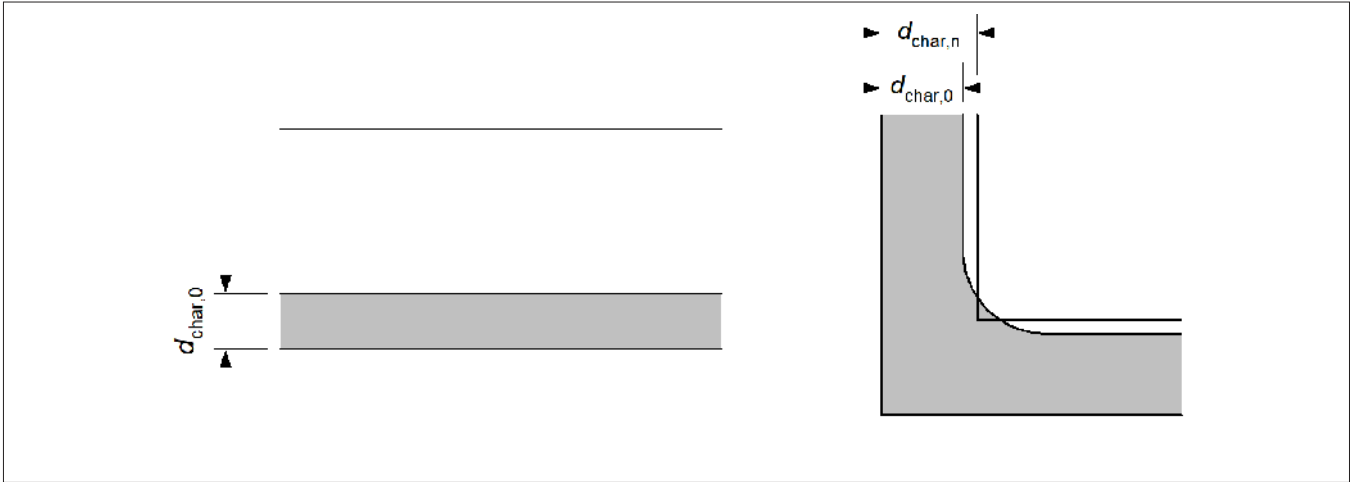


Figure 6.4. Left: One-dimensional charring of panel or wide cross section when fire exposure is below on one side, Right: Charring depth $d_{\text{char},0}$ for one-dimensional charring and notional charring depth $d_{\text{char},n}$ which takes into account the rounding of corners.

a zero strength layer. The remaining residual cross section beneath the zero strength layer on exposed sides is defined as the effective cross section. The effective cross section is calculated by subtracting the effective charring depth d_{ef} from the initial cross section, see Figure 6.5 and equation (6.1).

$$d_{\text{ef}} = d_{\text{char},n} + k_0 \cdot d_0 \quad (6.1) \text{ (EC5 4.1)}$$

where

d_0 is 7 mm;

$d_{\text{char},n}$ is the notional design charring depth, see equation (6.4); and

k_0 is in the case of unprotected surfaces $t/20$, when $t < 20$ min and 1,0 when $t > 20$ min. In the case of protected surfaces the value of k_0 is given in Figure 6.6.

For protected surfaces with a start of charring time of $t_{\text{ch}} > 20$ minutes, k_0 is assumed to vary linearly from 0 to 1 during the time interval from $t = 0$ to $t = t_{\text{ch}}$, see Figure 6.6 (b). For protected surfaces with $t_{\text{ch}} \leq 20$ minutes k_0 is $t / 20$.

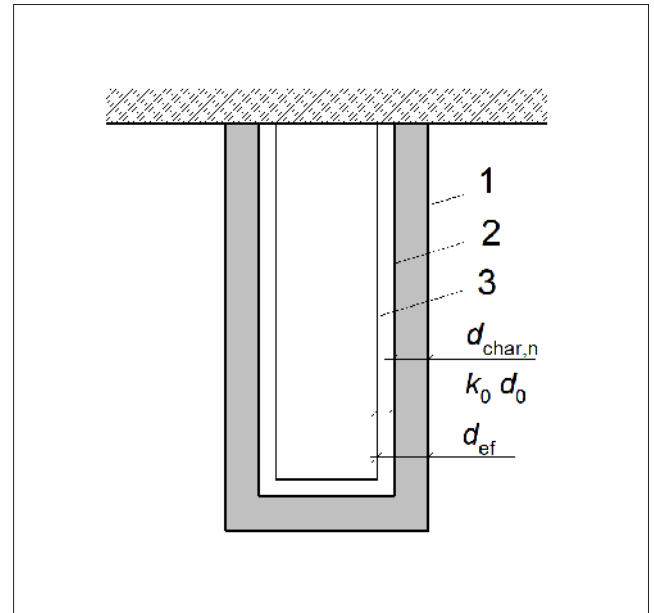


Figure 6.5. Definition of residual cross section and effective cross section. 1. Initial surface of member, 2. Border of residual cross section, 3. Border of effective cross section.

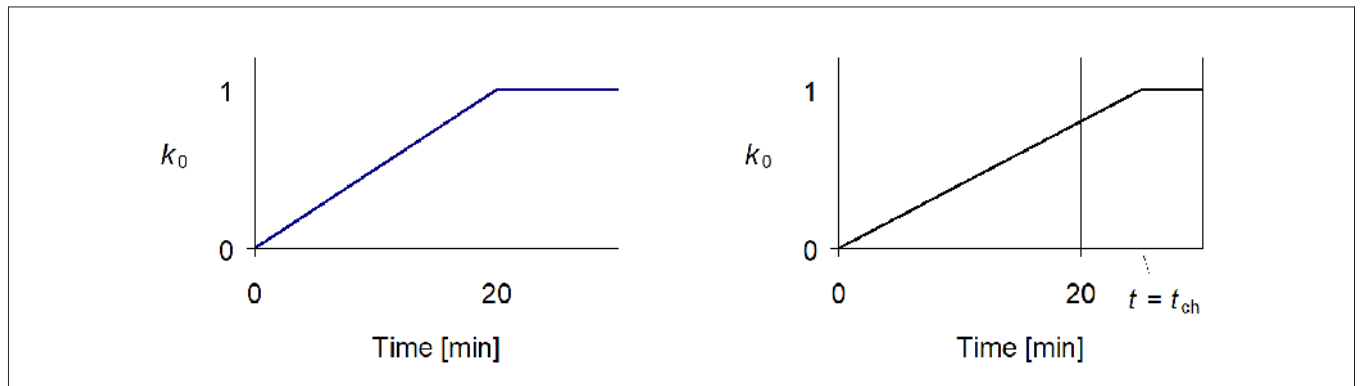


Figure 6.6. (a) Variation of k_0 for unprotected members and protected members where $t_{\text{ch}} \leq 20$ minutes and (b) for protected members where $t_{\text{ch}} > 20$ minutes.

6. PERFORMANCE OF LVL IN FIRE

For timber surfaces facing a void cavity in a floor or wall assembly (normally the wide sides of a stud or a joist), the following applies:

- Where the fire protective cladding consists of one or two layers of gypsum plasterboard type A, wood panelling or wood-based panels, at the time of failure t_f of the cladding, k_0 should be taken as 0,3. Thereafter k_0 should be assumed to increase linearly to 1,0 during the following 15 minutes;
- Where the fire protective cladding consists of one or two layers of gypsum plasterboard type F, at the time of start of charring t_{ch} , k_0 is 1. For times $t < t_{ch}$, linear interpolation should be applied, see Figure 6.6 (b).

The effective cross section should be used for the calculation of the stiffness and fire resistance of an LVL member.

Note: The effective cross section method is recommended. However, depending on the National Annex, the reduced properties method of Eurocode 5 may also be used.

3. Determination of design values of strength and stiffness

For the calculation of the design values of mechanical resistance $R_{d,fi}$ in a fire situation, the design values of strength properties shall be determined by the equation:

$$f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}} \quad (6.2) \text{ (EC5 2.1)}$$

where

- $f_{d,fi}$ is the design strength in fire;
- f_{20} is the 20 % fractile of a strength property at normal temperature. It can be calculated as $f_{20} = k_{fi} \cdot f_k$. For LVL k_{fi} is 1,1, so f_{20} is 1,1 times the characteristic strength f_k ;
- $k_{mod,fi}$ is the modification factor for fire. It replaces the modification factor for normal temperature design k_{mod} given in EN 1995-1-1. $k_{mod,fi}$ is 1,0 in most cases, except when the method of annex C of EN 1995-1-2 is used; and
- $\gamma_{M,fi}$ is the partial safety factor for timber in fire. The recommended factor for material properties in fire is $\gamma_{M,fi} = 1,0$. Information on national choice may be found in the national annex.

For example, the design value of bending strength for LVL-36 C:

$$f_{m,d,fi} = k_{mod,fi} \cdot \frac{k_{fi} \cdot f_{m,k}}{\gamma_{M,fi}} = 1,0 \cdot \frac{1,1 \cdot 36 \frac{\text{N}}{\text{mm}^2}}{1,0} = 39,6 \frac{\text{N}}{\text{mm}^2}$$

For stability calculations, the characteristic values of stiffness properties at normal temperature are used.

4. Determination of the design values of actions

The design effect of actions $E_{d,fi}$ for the fire situation is determined in accordance with EN 1991-1-2:2002, including the effects of thermal expansions and deformations. In typical cases of timber structures where the own weight is relatively low, the design values of actions $E_{d,fi}$ are 0,2-0,4 times the actions E_d in normal temperature design.

5. Verification that design resistance is larger than design action

It shall be verified for the required duration of fire exposure t that: $E_{d,fi} \leq R_{d,t,fi}$

According to Eurocode 5, clause 4.3 *Simplified rules for analysis of structural members and components*, compression perpendicular to the grain and shear resistance may be disregarded.

6.4.2 Charring rates of LVL

There are two different types of charring rates β_0 and β_n . For panels and wide cross sections one-dimensional charring rate β_0 is used in the calculations. This is also used as the basis value in some more advanced calculation methods. When the characteristic density of LVL is $\rho_k \geq 480 \text{ kg/m}^3$, the one-dimensional charring rate β_0 is **0,65 mm/min**.

The design charring depth for one-dimensional charring $d_{char,0}$ [mm] should be calculated as follows when the surface is unprotected throughout the time of fire exposure:

$$d_{char,0} = \beta_0 t \quad (6.3) \text{ (EC5 3.1)}$$

where t [min] is the time of fire exposure and β_0 [mm/min] is the one-dimensional charring rate.

For all other structures that are exposed from multiple sides, generally columns and beams, the notional charring rate β_0 is used in the calculations of the notional depth $d_{char,n}$. When the characteristic density of LVL is $\rho_k \geq 480 \text{ kg/m}^3$, the notional charring rate β_n is **0,70 mm/min**.

The design charring depth for notional charring $d_{char,n}$ should be calculated as follows when the surface is unprotected throughout the time of fire exposure:

$$d_{char,n} = \beta_n t \quad (6.4) \text{ (EC5 3.2)}$$

where t [min] is the time of fire exposure and β_n [mm/min] is the notional charring rate

In the test report VTT-S-04746-16 the one dimensional charring rate of different wood products was evaluated in 120min fire exposure according to a standardized time-temperature exposure curve (EN 1363-1:2012) ³⁹. According to the report, wood products behaved predictably and, for LVL, the one-dimensional charring rate $\beta_0 = 0,65 \text{ mm/min}$ can be used for an extended fire exposure. The results were similar in both face side and edge side exposure specimens. This gives the necessary information and confidence for fire designers in assessing

the resistance of LVL structures in demanding cases using, e.g., performance-based design methods.

Note: For special cases where more advanced design methods are used, the report VTT-S-04746-16 also has information on the charring rate β_0 in a test based on to a more stringent hydrocarbon (HC) time-temperature exposure curve (EN 1363-2:1999).

6.4.3 Design of unprotected beams and panels

Since LVL beams are typically slender structures with largest available beam thicknesses up to 75 mm without multiple gluing, unprotected LVL beams cannot be designed for higher than 15 min fire resistance time requirements. The zero strength layer ($k_0 \cdot d_0$) reduces the thickness of an effective cross section significantly, making the beam even more slender in lateral torsional buckling analysis.

Example: 63x300 mm LVL-P beam and 33 mm LVL-C panel in 15 minute fire exposure:

Beam:

$$d_{\text{ef,beam}} = \beta_n \cdot t + k_0 \cdot d_0 = 0,70 \frac{\text{mm}}{\text{min}} \cdot 15 \text{ min} + \frac{15 \text{ min}}{20 \text{ min}} \cdot 7 \text{ mm} = 15,75 \text{ mm}$$

Size of effective cross section of the beam in 3-side fire exposure:

$$\text{Width } b: \quad 63 \text{ mm} - 2 \cdot 15,75 \text{ mm} = 31,5 \text{ mm}$$

$$\text{Height } h: \quad 300 \text{ mm} - 15,75 \text{ mm} = 284 \text{ mm}$$

$$f_{\text{m,d,fi}} = k_{\text{mod,fi}} \cdot \frac{k_{\text{fi}} \cdot k_h \cdot f_{\text{m,k}}}{\gamma_{\text{M,fi}}} = 1,0 \cdot \frac{1,1 \cdot \left(\frac{300 \text{ mm}}{284 \text{ mm}}\right)^{0,15} \cdot 44 \frac{\text{N}}{\text{mm}^2}}{1,0} = 44,3 \frac{\text{N}}{\text{mm}^2}$$

Panel:

$$d_{\text{ef,panel}} = \beta_0 \cdot t + k_0 \cdot d_0 = 0,65 \frac{\text{mm}}{\text{min}} \cdot 15 \text{ min} + \frac{15 \text{ min}}{20 \text{ min}} \cdot 7 \text{ mm} = 15 \text{ mm}$$

Effective thickness of the panel $t_{\text{panel}}: 33 \text{ mm} - 15 \text{ mm} = 18 \text{ mm}$

Design value of bending strength for LVL 36 C:

$$f_{\text{m,d,fi}} = k_{\text{mod,fi}} \cdot \frac{k_{\text{fi}} \cdot f_{\text{m,k}}}{\gamma_{\text{M,fi}}} = 1,0 \cdot \frac{1,1 \cdot 36 \frac{\text{N}}{\text{mm}^2}}{1,0} = 39,6 \frac{\text{N}}{\text{mm}^2}$$

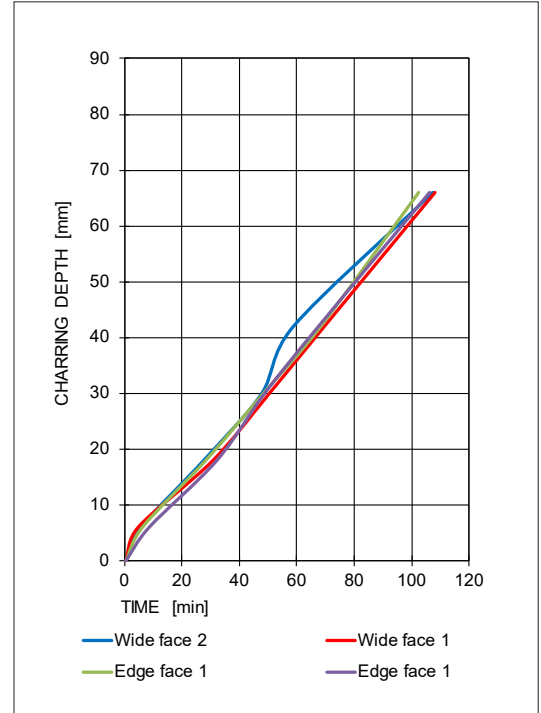


Figure 6.7. One-dimensional charring of LVL-C is linear in a 120 minutes fire exposure test according to the standardized time-temperature curve. Blue and red curves: exposure on the wide face of the specimens. Green and grey curves: exposure on the edge face of the specimens ³⁹.

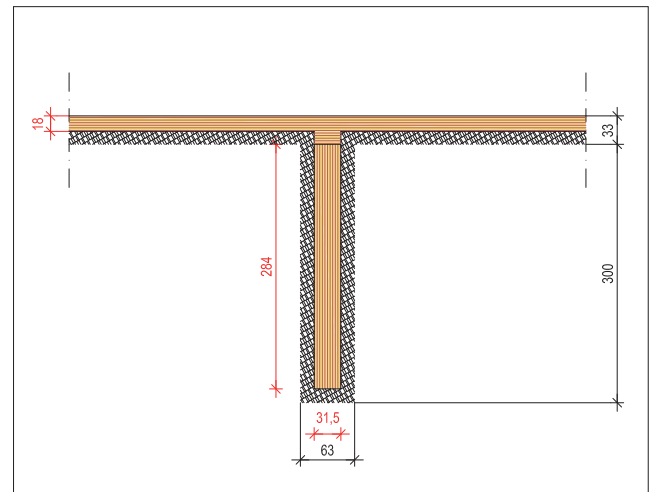


Figure 6.8. Effective cross section after 15 min fire exposure.

6. PERFORMANCE OF LVL IN FIRE

Example: 133x400 mm GLVL-P beam in 30 minutes fire exposure on all sides:

$$d_{\text{ef}} = \beta_n \cdot t + k_0 \cdot d_0 = 0,70 \frac{\text{mm}}{\text{min}} \cdot 30 \text{ min} + 1,0 \cdot 7 \text{ mm} = 28 \text{ mm}$$

Size of the effective cross section:

Width b: 133 mm - 2·28 mm = 77 mm

Height h: 400 mm - 2·28 mm = 344 mm

Design value of bending strength for LVL-48 P:

$$f_{\text{m,d,fi}} = k_{\text{mod,fi}} \cdot \frac{k_{\text{fi}} \cdot k_h \cdot f_{\text{m,k}}}{\gamma_{\text{M,fi}}} = 1,0 \cdot \frac{1,1 \cdot \left(\frac{300 \text{ mm}}{344 \text{ mm}}\right)^{0,15} \cdot 44 \frac{\text{N}}{\text{mm}^2}}{1,0} = 47,4 \frac{\text{N}}{\text{mm}^2}$$

When LVL-C panels are exposed to fire on one side, the one-dimensional charring rate β_0 is used for calculation of the effective panel thicknesses. Table 6.2 shows the thicknesses after 15, 30 and 60 min fire exposure. It is recommended that the effective thickness has at least one cross veneer. The values are thus shown for cases where the effective thickness is at least 9 mm.

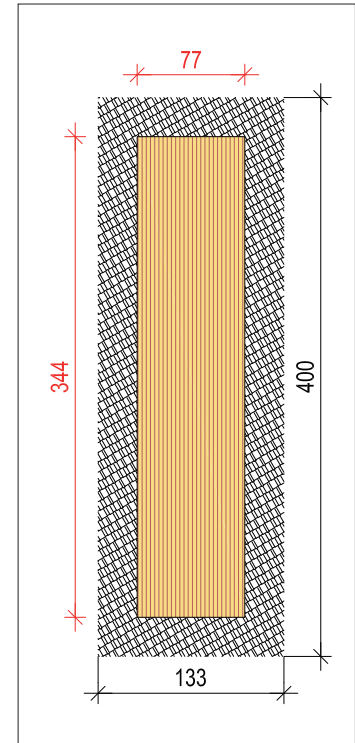


Figure 6.9. Effective cross section after 30 min fire exposure

Table 6.2. Effective thickness of LVL 36 C panels after 15 - 60 min fire exposure on one side.

LVL-C panel Original thickness [mm]	Effective thickness after 15 min [mm]	Effective thickness after 30 min [mm]	Effective thickness after 60 min [mm]
27	12	-	-
33	18	-	-
39	24	12	-
45	30	18	-
51	36	24	-
57	42	30	11
63	48	36	17
69	54	42	23
75	60	48	29

Table 6.3. Minimum thickness of LVL 36 C panel to give a fire exposure protection time 30 - 90 minutes for an underlying wooden structure.

Resistance to fire requirement R [min]	Minimum thickness h_{p1} of LVL 36 C panel protecting timber frame assemblies [mm]	Minimum thickness h_{p2} of LVL 36 C panel protection applied directly on beams or columns [mm]
30	29	27
45	39	36
60	49	46
75	58	56
90	68	66

6.4.4 LVL-C panel as a protection against fire exposure

When the LVL-C panel thickness is according to Table 6.3, the panel protects a wooden structure behind it for a certain fire resistance time t [min]. Other fire design calculations of the remaining wooden structure are not needed unless the panel is a part of the load-bearing system also in the structural fire design. In the case of timber frame assemblies, the LVL-C protection panel thickness h_{p1} [mm] is calculated for the required fire protection time t based on EN1995-1-2, equation (4.1) and (C.7) or (D.3):

$$\text{Minimum panel thickness } h_{p1} = \beta_0 \cdot (t + 4 \text{ min}) + 7 \text{ mm} \quad (6.5)$$

When the protection panel is applied directly on beams or columns, the protection panel thickness h_{p2} [mm] is calculated for the required protection time t based on EN1995-1-2 equations (4.1) and (3.10):

$$\text{Minimum panel thickness } h_{p2} = \beta_0 \cdot t + 7 \text{ mm} \quad (6.6)$$

6.4.5 Summary of LVL-C panels for fire protection

Table 6.10 specifies the minimum thickness h_p of LVL-C panel when used as a covering with fire protection ability for the underlying materials (column A), fire protection of the structures (column B) or when used as a ceiling structure which has a fire resistance requirement EI (column C). All of the different fire protection specifications also have requirements for the detailing of, e.g., joints between the panels.

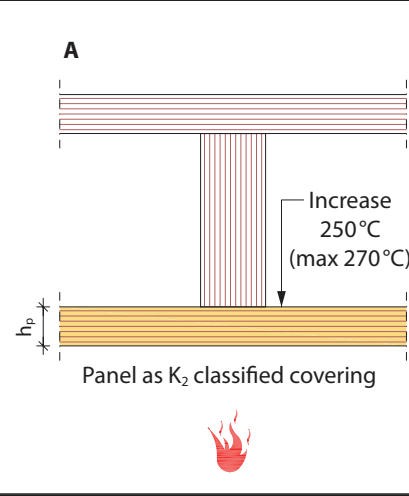
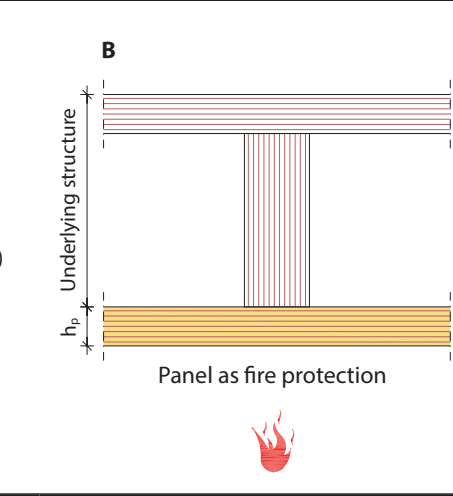
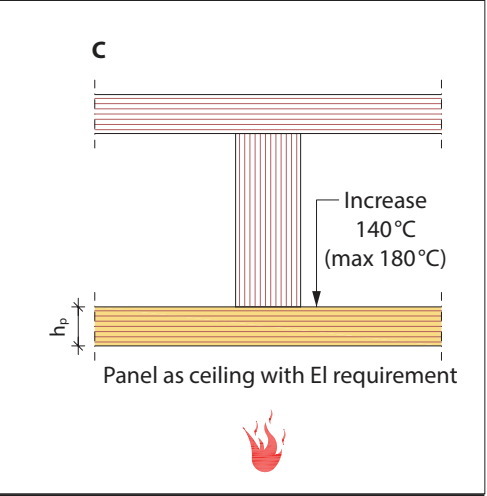
 <p>Panel as K_2 classified covering</p>	 <p>Panel as fire protection</p>	 <p>Panel as ceiling with EI requirement</p>
K_2 10, when $h_p \geq 15$ mm	Fire protection for 15 min, when $h_p \geq 18$ mm	Insulation time $t_{ins} = 15$ min, when $h_p \geq 18$ mm
K_2 30, when $h_p \geq 26$ mm	Fire protection for 30 min, when $h_p \geq 29$ mm	Insulation time $t_{ins} = 30$ min, when $h_p \geq 33$ mm
K_2 60, when $h_p \geq 52$ mm	Fire protection for 60 min, when $h_p \geq 49$ mm	Insulation time $t_{ins} = 60$ min, when $h_p \geq 66$ mm

Figure 6.10. Minimum LVL 36 C panel thicknesses in fire protection applications.

- A)** The fire protection ability K_2 class requirement for protecting the underlying material is that behind the protection panel the average temperature rise over the whole of the unexposed surface is limited to 250 °C and the maximum temperature rise at any point of that surface does not exceed 270 °C. Note that in some cases there can be additional requirements to use only non-combustible materials for the covering.
- B)** LVL panel as fire protection. When the panel thickness $h_p = d_{char,0} + k_0 \cdot 7$ mm, it protects the underlying structure for a required fire resistance time t and the effective cross-section in the resistance calculation is the entire underlying structure.
- C)** The separating function EI is assumed to be satisfied when integrity E and insulation I are maintained. When LVL-C panel is used as a ceiling structure and has a fire resistance requirement of EI , the required panel thicknesses h_p can be determined according to EN1995-1-2 Annex E based on the insulation time t_{ins} . In addition, the requirements for the detailing of joints shall be fulfilled. Note: An advanced calculation method for separating function is presented in the publication *Fire safety in timber buildings – Technical guideline for Europe*, chapter 5.5 ⁴⁰

6.5 FIRE RESISTANCE DESIGN OF FLOOR & WALL STRUCTURES WITH CAVITIES

The fire resistance of LVL wall and floor assemblies can be calculated according to Annexes C and D of EN1995-1-2 and its National Annexes. When the cavities between the joists or studs are filled with rock fibre insulation, Annex C shall be used. For void cavities Annex D shall be used.

In the case of filled cavities, the design process according to EN1995-1-2 Annex C is the following:

1. Verification that the structure fulfils the boundary conditions of the design method, note that it is valid for max 60 min fire resistance.
2. Protection time t_{ch} for the start of charring and t_f for failure of the panels are evaluated for the protection panel combination based on the equations in Annex C, and technical specifications of the panel suppliers. It is essential to use detailing that keeps the protective rock fibre insulation in place also after failure of the protection panels. One example for floor structure is fixing wooden battens to the joists to create a shelf to support the insulation, see Figure 6.11.
3. Residual cross section of the studs or joist is calculated with the assumption that they char only from the edge of the fire exposed side. Notional charring rate is calculated with adjustment factors which depend on the phase of the charring and the thickness of the joist or stud.
4. The resistance of the residual cross section is calculated with strength values $f_{d,fi}$, see equation (6.2). The reduction factor $k_{mod,fi}$ values are defined in section C.3 of the Annex C. The reduction depends on the member depth and the notional charring depth and the $k_{mod,fi}$ has a significant

influence on the resistance. In the case of wall structures, the same value of $k_{mod,fi}$ may be used both in the width and in the thickness direction of a stud.

In the case of void cavities, the design process according to EN1995-1-2 Annex D is the following:

1. Protection time t_{ch} for the start of charring and t_f for failure of the panels are evaluated for the protection panel combination based on the equations in Annex D, and technical specifications of the panel suppliers.
2. Residual cross section of the studs or joist is calculated with the assumption that they are charring from the sides and the edge of the fire exposed side. Charring depth is calculated with adjustment factors which depend on the phase of the charring, similarly to beams and columns that are initially protected from fire exposure in subsection 3.4.3.2. of EN1995-1-2.
3. The resistance is calculated for an effective cross section according to subsection 4.2.2 of EN1995-1-2 with strength values $f_{d,fi}$, see equation (6.2).

The resistance to fire of an LVL joist floor is mainly dependent on the properties of the protective panels. Since the thickness of LVL joists is usually 45–57 mm, their resistance reduces rapidly when their sides are exposed to fire.

Fire resistance of rib slabs and box slabs may be calculated with the same principles, but manufacturer-specific design instructions according to their ETAs shall be followed for, e.g., the conditions when the structural gluing may be utilized.

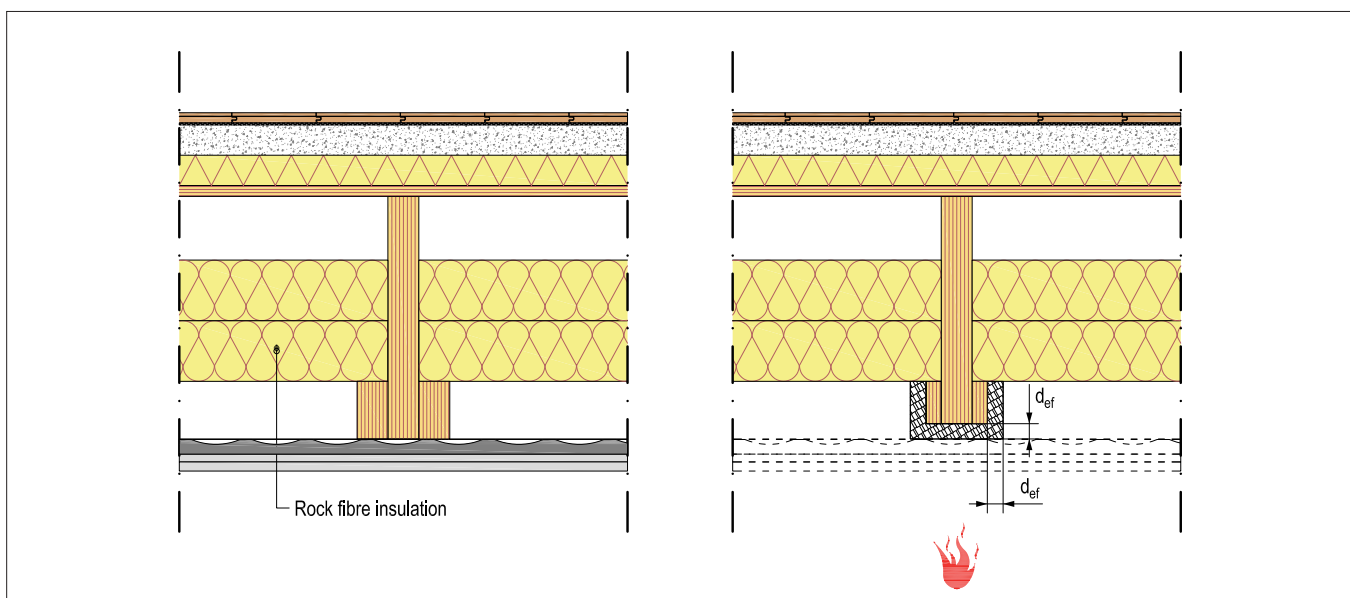


Figure 6.11. LVL joist floor structure with rock fibre filled cavities. Wooden battens fixed to the joists create a shelf that keeps the rock fibre insulation in place protecting the sides of the joists from charring.

7. Durability

Eurofins



Figure 7.1. Wooden pedestrian bridge, Matinpuron, Espoo, Finland.

7.1 SERVICE LIFE OF WOODEN BUILDINGS AND LVL STRUCTURES

Buildings are normally designed for a 50-year service life. When structural LVL is produced and tested according to its harmonized product standard EN14374, the durability of the adhesive bonding strength and the mechanical properties meet the requirements of the design service life. However, the structural protection of the building shall be designed, executed and maintained so that the structures remain in service class 1 or 2 conditions.

In remarkable buildings, the design service life is 100 years. This can be achieved with the same means as the default 50-year service life, but an increased safety factor for the design values of eventual climatic loads (wind and snow load) shall be used in the structural design. The safety factor is defined in national regulations, e.g. in Finland the design values of eventual climatic loads must be increased by factor 1,2 when the expected service life is above 100 years.

The report 100 years' service life of wood in service class 1 and 2 – dry and moderately humid conditions, Viitanen, VTT-R-04689-14⁴¹ summarizes that to achieve a 100-year service life the following should be ensured: use of dry and CE-marked wood material; use of correct glue type and glue class for engineering components; good detailing and design; good execution and protection against weathering, take into account the effect of natural loading; proper maintenance and provision of a maintenance manual for users; and guarantee proper conditions for materials in the building throughout its service life.

7.2 DURABILITY CLASSIFICATIONS OF STRUCTURAL LVL PRODUCTS

7.2.1 Durability of adhesive bonding strength of structural LVL

The adhesive and glue bond used for structural laminated veneer lumber (LVL) is suitable for service classes 1, 2 and 3 according to EN 1995-1-1:2004 (Eurocode 5). This is verified in a cleavage test defined in EN 14374:2004, Annex B. The test procedure for LVL specimens is as follows:

- Immersion for at least 4 h in boiling water
- Drying in a ventilated oven for at least 16h at 60°C
- Immersion for at least 4 h in boiling water again
- Cooling in room temperature water for at least 2h.
- After the boiling & drying cycles, the samples are cleaved at least at one glue line close to the middle of the thickness and the apparent cohesive wood failure percentage is determined according to EN 314-1.
- For each tested glue line the apparent cohesive wood failure percentage shall be at least 70%.

Adhesive type I (full exposure to the weather) according to EN 301 or EN 15425 (polyurethane adhesives) shall be used

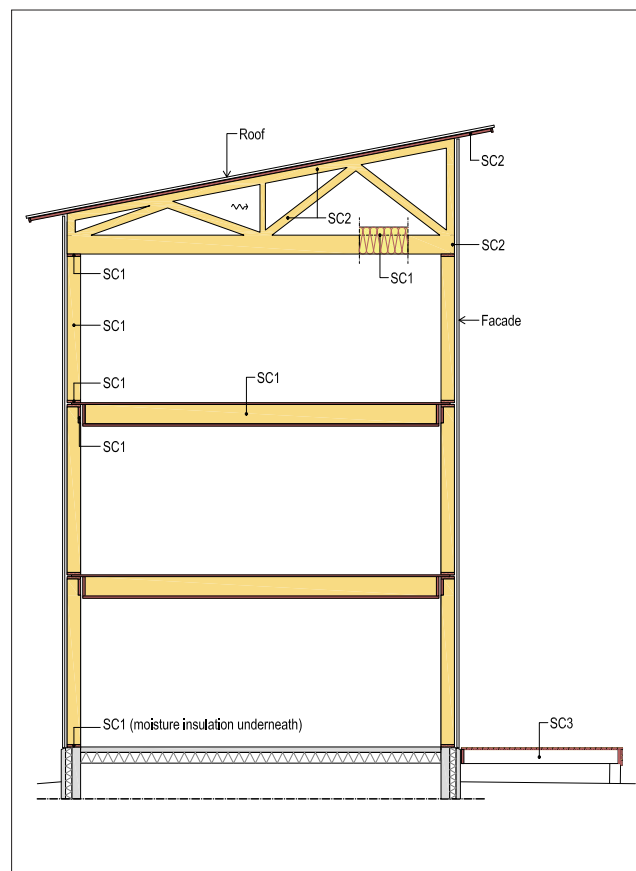


Figure 7.2. Roof overhangs and high foundation level are practical means of structural wood protection to keep wooden structures in service class 2 (use class 2) or dryer conditions. Note: Assignment of structures to service classes may be given in the National Annex to EN1995-1-1 for each country.

for structurally glued products are made from structural LVL components.

7.2.2 Biological durability of softwood LVL

LVL veneers always contain some sapwood. According to EN 350 sapwood is regarded as not durable unless otherwise proven by test data. The biological durability class of softwood LVL is therefore DC 5 (Not durable) according to this classification.

7.2.3 Structural LVL's suitability for different use classes and service classes

Structural LVL products can be used in service classes 1 and 2 as defined in Eurocode 5, which corresponds to the use classes 1 and 2 as defined in EN 335. Structural LVL products should not be used in service class 3 without additional protective treatment. Structurally glued products made from LVL components are not suitable for service class 3.



Figure 7.3. Moisture protection treatment on LVL beams.

7.2.4 Discussion of durability class definitions

The biological durability classification based on EN 350 is poorly applicable to LVL or other engineered wood products. The biological durability of LVL and other wood-based panels (uncoated or overlaid) is dependent on the end use conditions, and this is where the EN 335 use classes provide useful guidance.

EN 350 durability classes are defined for the heartwood of wood species in use class 4. However, there is no information on the suitability of different natural durability classes for different end uses. CEN/TS 1099 gives guidance for plywood, but does not provide information on the suitability of different natural durability classes for different end uses.

LVL and other wood-based panels contain sapwood. The fact that sapwood is always regarded as non-durable argues against using the EN 350 classification as, based on this, LVL and all wood-based panels are universally classed as 'not durable' (class 5).

EN 350 states that the durability classes are designed to indicate the expected level of performance of wood when used in contact with the ground (service conditions as described for use class 4 in EN 335) and that in other use classes the service conditions can result in wood performance that differs from that implied by the durability classification. Based on this, the standard may not be directly applicable to use class 3, 2 and 1 conditions and should therefore not be used as the biological durability criteria. The natural durability of LVL can differ from that of the wood species used in its manufacture. Additional factors contributing to durability include the thickness of the plies, the structure of the board, and the characteristics and quantity of the binder.

In conclusion, it is recommended to declare the product as durable in use class 1 and 2 (or service class 1 and 2 according to EN1995-1-1) conditions.

7.3 STRUCTURAL WOOD PROTECTION

The best way to manage the durability of LVL, as well as all other timber structures, is structural protection. This means, for example, long enough roof overhangs and high foundation levels and moisture isolation between the foundation and sole plates. Structural protection keeps the structures in service class 2 (use class 2) or dryer conditions, which wood products can resist well.

7.3.1 Resistance against temporary moisture exposure of structural LVL products

Exposure to rain, splashing as well as water convection from other structures shall be avoided. The designer must pay attention to the details of the construction to ensure that no water pockets will be formed. The product may be exposed to weather for a short period during installation. During the erection of a building, structural LVL products and elements, which are structurally glued from LVL components, have good resistance to temporary exposure to water without damage or decay. This requires, however, that it is ensured that the products can afterwards dry to the desired moisture content before the structural envelope is closed. The integrity of the glue bonding is maintained according to the assigned service class throughout the expected life of the structure ²⁸.

LVL products swell when the moisture content increases, and shrink when the moisture content decreases. A part of the swelling is permanent and the extent of these dimensional changes depends on the grain direction. Wetting can cause permanent deformations and impair the visual appearance of surface veneers, such as colour changes due to water staining, surface cracks and falling knots due to drying shrinkage after wetting ²⁸.

Joints with mechanical connectors, such as bolted connections, may become loose due to swelling and shrinking cycles. Drying shrinkage after severe wetting may cause cracking, which often reduces the load-bearing capacity of dowel-type connections, notched beams and beams with holes.

7.3.2 Resistance against UV radiation

Like all wood products, non-treated surfaces of LVL will slowly fade to grey due to the action of UV radiation from the sun. This greying does not affect the strength properties.

If this natural greying is not desired, an adequately pigmented coating system or a coating containing special additives must be applied. The higher the proportion of pigments, the greater the protective effect. The coating layer must be of sufficient thickness to fulfil the minimum protection requirement across the entire surface. More information is provided in the technical data sheets of the coating manufacturers ⁴².

7.3.3 Chemical durability

The main components of wood: cellulose and lignin, react in opposite ways to acids and alkalinity. Cellulose is not very

resistant to strong acids, but its resistance to bases is good. Lignin, on the other hand, is easily dissolved in bases, whereas it is resistant to most strong acids. For these reasons, wood is quite resistant to moderate chemical effects.

LVL has good resistance to mild acids and acid salt solutions. Alkalis, however, cause softening of the wood. Direct contact with oxidizing agents such as chlorine, hypochlorites and nitrates should be avoided.

Wood is generally quite resistant to organic substances. However, organic solvents such as acetone, benzene, alcohol etc. dissolve resins, fats and waxes, causing similar effects to water, i.e., producing swelling and a slight reduction in strength properties. Petroleum oils have no effect on strength properties but cause discolouration.

Chemical resistance can be improved with various types of coatings ⁴².

7.4 CHEMICAL WOOD PROTECTION

7.4.1 Surface treatment

LVL can be protected against temporary weather exposure by surface treatments that repel rainwater, and therefore reduce the amount of moisture absorbed by the product, but allow water vapour movement to and from the product. This improves dimensional stability and reduces swelling of the treated LVL product during the construction time.

Use in high relative humidity conditions may result in mould growth on the surface of structural LVL products. If the products are exposed to outdoor air humidity conditions (e.g. structures in unheated spaces) or wetting during the logistic chain and construction time, a brushable or sprayable treatment should be applied on the surface of the LVL components to reduce the risk of mould growth. In some cases LVL components are treated at the factory, but the default practice is for treatment to be done during offsite production of timber elements or on the construction site. If there is mould growth on the surface of LVL products, the mould must be removed, e.g. by sanding, before closing the structure.

The surface treatments do not affect the product's strength properties, but their compatibility with end finishes, such as paints, shall be verified separately ^{28, 43}.

7.4.2 Pressure impregnation

Pressure impregnation of spruce softwood LVL is possible when the impregnation process is tailored for it. Due to dimensional changes due to moisture and to achieve the required retention level of the water-based impregnation agent, it is recommended to use only LVL-C for impregnation. For beam-size products with a width of max 600 mm, a retention level similar to the AB class used in the Nordic countries can be achieved. It is also possible to impregnate panel-size products, although the retention level in the core area is lower.

Applications for pressure-impregnated LVL-C include terrace

and balcony structures, balustrades, stairs and piers where the products are not in direct ground contact (= Use class 3.1 conditions).

AB class impregnation agents are aggressively corrosive and therefore only stainless steel connectors should be used in structures made of impregnated LVL.

The surface of pressure-impregnated LVL is rougher than regular LVL. The impregnation and drying processes cause swelling and shrinkage, which open some of the peeling cracks in the surface veneers. Local cracking around knots and minor separation of scarf joints in small areas of the surface veneer might occur.

Impregnated LVL products swell in the process and their equilibrium moisture content and specific weight are a couple of per cent higher than untreated products. The increased weight shall be taken into account, but otherwise structural design shall be made with the nominal dimensions of the product with the strength and stiffness reduction factors according to service class 3 of Eurocode 5. In some countries additional reduction factors given by their national requirements shall be taken into account ⁴⁴.



Figure 7.4. Pressure-impregnated LVL-C beams.

8. Building physics

Puuinfo



Figure 8.1 Roof overhang protects the building against weather exposure and solar radiation, Kindergarten Vekara, Pukkila, Finland.

LVL beams, studs and panels can be used in structures similar to all other timber and engineered wood products. Building physics analyses of walls and roofs do not need any special methods and normal design tools are suitable for their thermal resistance calculation and dew point analyses. As LVL has fairly low thermal conductivity, the cold bridging effect of LVL beams or studs is minimal. In special cases LVL-C panels may be used alone as water vapour barriers with no separate plastic membranes required.

8.1 LVL AND MOISTURE

LVL is a hygroscopic material similar to other wood-based products. Therefore, the moisture content of LVL products is dependent on the relative humidity (RH%) and, more specifically, on the direction of moisture content change (drying / wetting). LVL products swell when their moisture content increases and shrink when their moisture content decreases. A part of the swelling is permanent and the extent of these dimensional changes depends on the grain direction. Wetting can cause permanent deformations, and impair the visual appearance of surface veneers, such as colour changes due to water staining, surface cracks and falling of knots due to drying shrinkage after wetting, see also subsection 7.3.1.

The hygroscopic surface of LVL has also advantages. If the surface is untreated or the surface treatment does not form a membrane on the surface, LVL can have a moisture buffering function absorbing humidity from the air when the RH% is high and releasing it when the RH% is low. This action levels out the peaks and can help to create pleasant indoor air conditions. In unheated storage spaces, the hygroscopicity prevents water condensation on cold surfaces and thus the risk of water dropping from, e.g., roof structures is smaller than in, e.g., steel structures.

8.1.1 Moisture content of LVL

When leaving the factory, the moisture content ω of the LVL product is approximately 8 to 10%. Due to changes in ambient temperature and relative humidity, the moisture content of the product will continuously change. In service class 1 the moisture content usually varies between 6 and 10%, while in service class 2 it usually varies between 10 and 16%. Thus, the LVL products are delivered from the factory at a moisture content that is close to the end use conditions.

Product moisture content ω is defined as follows:

$$\omega = \frac{m_{\omega} - m_0}{m_0} \quad (8.1)$$

where

m_{ω} is the product mass at moisture content ω ; and
 m_0 is the product dry mass.

The average equilibrium moisture content of LVL products in different relative humidity conditions (RH%) can be estimated using a sorption isotherm Figure 8.1. When wood is drying (desorption) its equilibrium moisture content is higher in the same relative humidity than when the wood is wetting (absorption). This phenomenon is called hysteresis.

8.1.2 Measuring the moisture content

Moisture meters based on electrical resistance give somewhat too high results for LVL due to the glue lines of LVL products. For exact determination of moisture content in a LVL sample, an oven drying test can be conducted according to EN 322.

Surface moisture meters (non-invasive) are recommended for measuring the moisture content of LVL products. The measurements should be taken perpendicular to the grain direction from the face veneers at an undamaged location. E.g., through sanded areas of the face veneers cannot be measured reliably. To obtain the most reliable results, the moisture meter should be calibrated using samples of known moisture content measured, for example, by oven drying.

Note: Examples of suitable moisture meters for measuring the moisture content of spruce LVL are the Delta 2000H (setup: H3 Spruce) and the Doser Messgerät HD5 (setup: material group 3) ¹⁸.

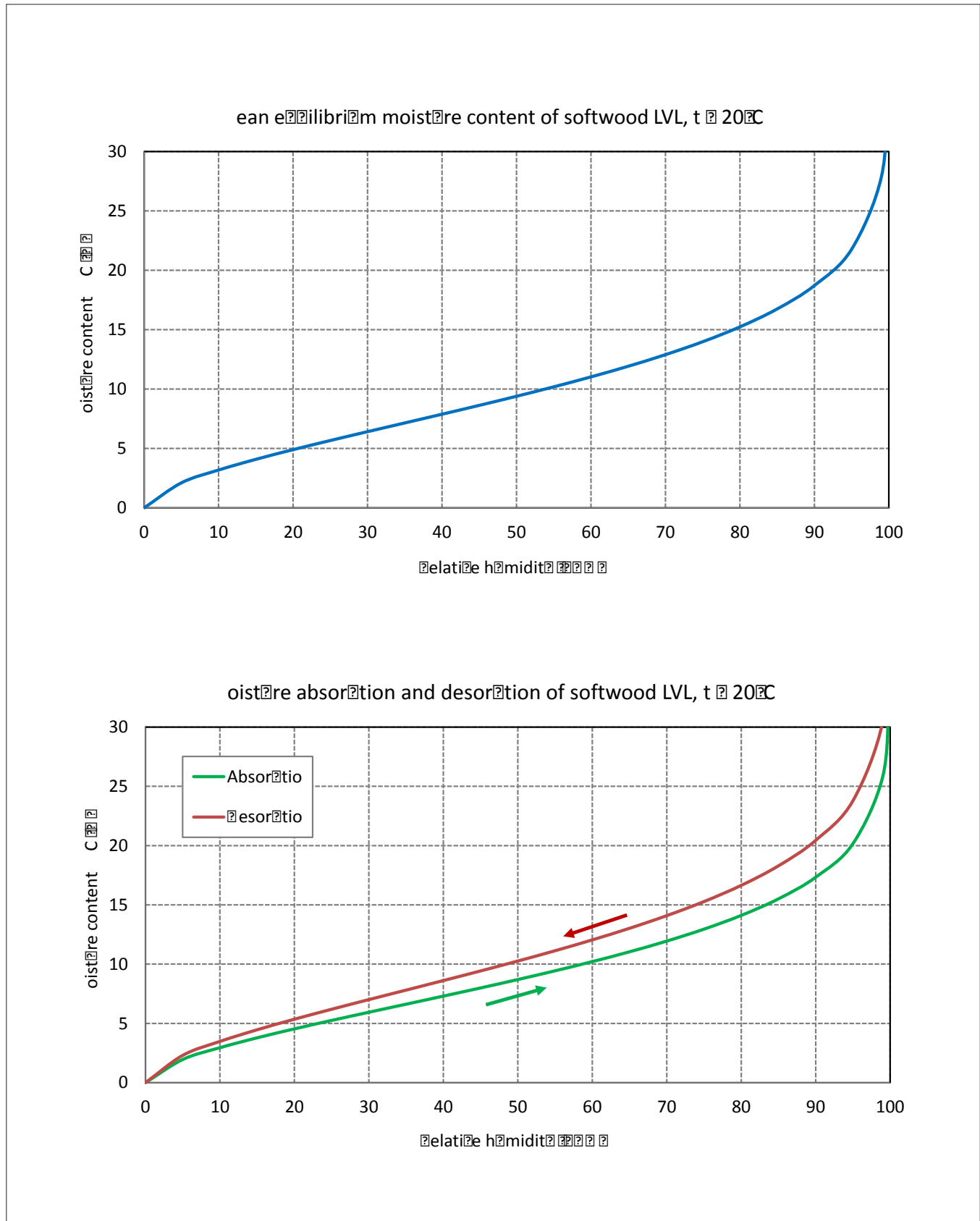


Figure 8.2. Left: Average equilibrium moisture content of softwood LVL in different relative humidity at 20 °C. Right: Absorption and desorption isotherms of softwood LVL at 20 °C based on weather cycling tests, RH 65% → 92% → 40%⁴⁵.

8.1.3 Dimensional changes due to moisture

LVL products are delivered from the factory at a moisture content that is close to the moisture content of a service class 1 end use. This is an advantage, as it significantly reduces dimensional changes due to moisture, provided that the members are protected against weather exposure during transport, storage and construction work.

Table 8.1. Swelling and shrinkage factor α_H for LVL products in % per 1% change in moisture content below fibre saturation point. Note: due to its cross band veneers, LVL-C undergoes much less dimensional change in the width direction than LVL-P (FprEN 14374:2018).

Dimension	LVL-P	LVL-C
Thickness t	0,32	0,32
Height h (or width of a panel)	0,32	0,03
Length l	0,01	0,01

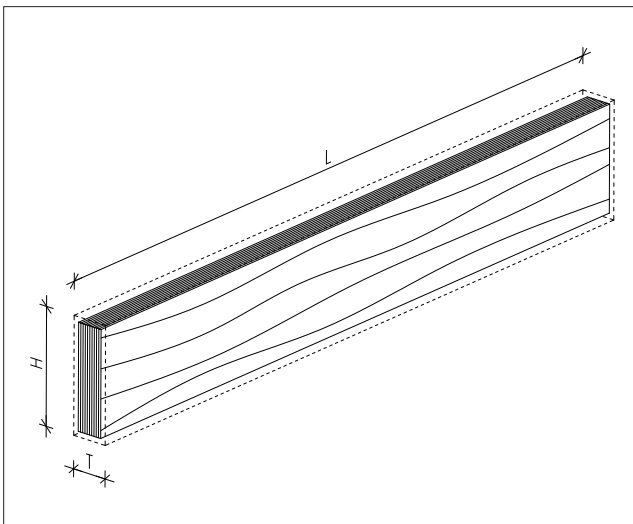


Figure 8.3. Directions of LVL dimensions

LVL products swell when their moisture content increases and shrink when their moisture content decreases. The extent of these dimensional changes depends on the grain direction. Due to the hot press production process, a proportion of the swelling in the thickness direction occurring upon first wetting of the product is permanent. However, despite the dimensional changes due to moisture, the nominal thickness of the product should be used in the structural capacity design of LVL members.

Dimensional change ΔL due to variation in moisture can be calculated as follows:

$$\Delta L = \Delta \omega \cdot \alpha_H / 100 \cdot L \quad (8.2)$$

where

$\Delta \omega$ is the change in product moisture content [%];

α_H is the product's dimensional variation coefficient, see Table 8.1 for values and Figure 8.3 for directions; and

L is the product dimension in the corresponding direction.

Notably, due to its cross band veneers LVL-C has a very low α_H factor in the member width direction: only 10% of the value specified for LVL-P products. This advantage can be utilized in structures that are sensitive to dimensional changes due to moisture.

LVL products can warp if the moisture content of opposite surfaces is not equal, for example if one surface is exposed to a higher relative humidity than the other. LVL-P products are more sensitive to such warping than LVL-C, especially if the height of the product is more than 8 times the thickness ($h > 8t$). Therefore, it is normally recommended to limit the slenderness of the LVL-P beams to this ratio. If careful moisture management of the components and structures can be assured throughout the logistic chain and construction process, e.g. in off-site element production, a h/t ratio of max ~ 12 may be considered.

Table 8.2. Example of dimensional changes due to moisture : If the relative humidity changes from 50% to 85%, the moisture content of a LVL beam increases by approximately 7%. The resulting effect on beam dimensions is as follows:

Product	Direction	Original dimension	Dimension after +7% increase of MC	Difference
LVL-P or LVL-C	Length l	4200 mm	$4200 + (7 \times 0.01/100 \times 4200) = 4203$ mm	+3,0 mm
LVL-P or LVL-C	Thickness t	57 mm	$57 + (7 \times 0.032/100 \times 57) = 58,3$ mm	+1,3 mm
LVL-P	Height h	260 mm	$260 + (7 \times 0.32/100 \times 260) = 266$ mm	+6,0 mm
LVL-C	Height h	260 mm	$260 + (7 \times 0.03/100 \times 260) = 260,6$ mm	+0,6 mm

Table 8.3. Water vapour resistance factor μ and water vapour diffusion coefficient in air δ_p of softwood LVL.

Density ρ_{mean}	Water vapour resistance factor μ [-]		Water vapour diffusion coefficient in air δ_p [kg/(Pa·s·m)]	
	Dry cup	Wet cup	Dry cup	Wet cup
440 kg/m ³	180	65	$0,73 \cdot 10^{-12}$	$2,3 \cdot 10^{-12}$
510 kg/m ³	200	70	$0,96 \cdot 10^{-12}$	$2,7 \cdot 10^{-12}$

The dry cup values are tested in 23°C - 0/50 RH % and apply when the mean relative humidity across the material is less than 70 %. The wet cup values are tested in 23°C - 50/93 RH % and apply when the mean relative humidity across the material is greater than or equal to 70 %.

8.1.4 Water vapour resistance

The water vapour resistance factor μ and water vapour diffusion coefficient in air δ_p defined in EN ISO 10456 may be used for LVL products in their thickness direction.

Thickness direction values are normally the essential values for building physics analyses. For special cases, however, the following estimates for the other directions may be used:

- Due to the wood cell structure, water vapour resistance in the length direction is only 5% of the water vapour resistance in the thickness direction
- For LVL-P the values are similar in both the thickness and height directions
- For LVL-C, due to the cross band veneers, resistance in the height direction is about 15% of the resistance in the thickness direction.

Note: the glue lines between the veneers do not have a significant influence on the water vapour resistance of LVL, which is of a similar level to solid sawn spruce or pine.

8.2 THERMAL PROPERTIES OF LVL

8.2.1 Influence of temperature on the mechanical properties of LVL

The characteristic values of the mechanical properties specified for LVL products can be used without any modification for temperatures below or equal to 50 °C for a prolonged period of time. LVL can be continually used in temperatures less than 100 °C and has a maximum short-term exposure temperature of 120 °C. Wood products resist cold better than heat and the minimum suitable temperature for LVL is -200 °C⁴⁶. In structural fire design, however, the decrease in strength and stiffness properties of LVL products due to high temperature must be taken into account, similar to other types of softwood members. The reduction factors may be evaluated from the Figures 6.3.

8.2.2 Thermal conductivity of LVL products

The design thermal conductivity λ of LVL products is 0,13 W/(m K), according to the tabulated values in EN ISO 10456 for a product density of 500 kg/m³, which is recommended to be used in the calculation of thermal insulation. The tabulated value is defined for 20 °C, RH 65% conditions. Lower density decreases the thermal conductivity and higher moisture content increases it. Within the practical range of density and moisture content, their influence on thermal conductivity λ may be $\pm 0,02$ W/(m K).

8.2.3 Temperature deformations

Since the dimensions of LVL products remain stable in normal temperature change conditions, it is usually not necessary to consider any effects of temperature variations on the structural design, unlike swelling and shrinkage due to moisture. The coefficient of thermal expansion in the direction of wood fibres is in the range from 3,5 to 5,0 10⁶/K.

Example: If the temperature changes from 5 °C to 30 °C, the length of a 10 000mm long LVL beam changes as follows:
 $10\,000\text{ mm} \rightarrow 10\,000\text{ mm} + (25\text{ °C} \cdot 4,0 \cdot 10^6/\text{°C} \cdot 10\,000\text{ mm})$
 $= 10\,001\text{ mm}$

In normal ambient temperatures the properties of LVL products are unaffected by temperature variations.

8.2.4 Heat combustion and specific heat capacity

The heat of combustion of LVL products is 17 MJ/kg²⁸. The specific heat capacity c_p is 1600 J/(kg K) according to EN ISO 10456.

8.2.5 Ignition temperature of LVL

Like all wood products, LVL is classified as a combustible material. The ignition temperature of LVL is about 270 °C, when exposed to flame. Spontaneous ignition does not occur in temperatures below 400 °C. The ignition temperature of wood is also dependent on the duration of exposure to high temperature. Ignition temperature decreases as the duration of exposure increases. After more than 20 hours of exposure, there is an ignition risk of wood products at temperatures above 120-150 °C ⁴⁶.

8.3 AIRTIGHTNESS

LVL-C panels are airtight beyond what can be measured. In the building physics design of structures, it is essential pay attention to the joints and seals between the panels and other structures to ensure the airtightness of the entire building envelope. This can be achieved, e.g., with careful installation of sealant tapes that are durable enough for the design service life of the building.

LVL-C panels have been utilized, e.g., in 3-layer CLT wall panels as the middle layer of the product to make them airtight.

9. Calculation examples of LVL structures

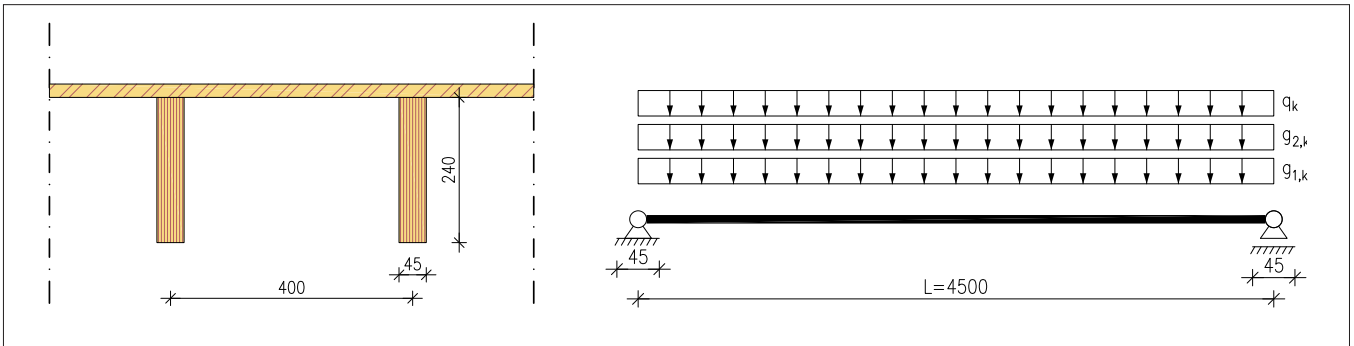


Figure 9.1. Wooden multi-storey building Lighthouse, Joensuu, Finland

These structural calculation examples for LVL are based on Eurocodes (EN1990, EN1991 and EN1995) and the additional instructions given in Chapters 4-5. Where information from National annexes is required, the Finnish annex or the default values of Eurocodes have been used. The calculations make references to the equation numbers of the Chapters 4 - 6. The examples are chosen to demonstrate the calculations methods. Therefore some of the component sizes may not necessarily ideal for practice and those cases have comments of possibly better suitable sizes at the end of each example.

9.1 LVL 48P JOIST FLOOR

Residential floor: span $L = 4500$ mm; width $b = 5000$ mm; 45x240 mm LVL 48P joists at spacing $s = 400$ mm; 22 mm chipboard decking. Support length 45 mm. Live load $q_k = 2,0$ kN/m²; partition load $g_{2,k} = 0,3$ kN/m² and self-weight $g_{1,k} = 0,6$ kN/m². Service class SC1.



Joist properties:

Bending strength edgewise $f_{m,0,edge,k}$	= 48 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,2 N/mm ²
Compression perpendicular to the grain edgewise $f_{c,90,edge,k}$	= 6 N/mm ²
Modulus of elasticity $E_{0,mean}$	= 13800 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 600 N/mm ²
Area of cross section $A = b \cdot h$	= 10800 mm ²
Section modulus $W = b \cdot h^2 / 6$	= 4,32 · 10 ⁵ mm ³
Moment of inertia $I = b \cdot h^3 / 12$	= 5,18 · 10 ⁷ mm ⁴
Moment stiffness of the joist $EI = 13800 \text{ N/mm}^2 \cdot 5,18 \cdot 10^7 \text{ mm}^4$	= 7,15 · 10 ¹¹ Nmm ²
Shear rigidity of the joist $GA = 600 \text{ N/mm}^2 \cdot 10800 \text{ mm}^2$	= 6,48 · 10 ⁶ N
Modification factor k_{mod} for medium-term, SC1	= 0,8
Modification factor k_{def} for SC1	= 0,6
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/240)^{0,15}$	= 1,034

Loading combinations

The most critical ultimate limit state (ULS) load combination:

$$E_{d,ULS} = \gamma_G \cdot (g_{1,k} + g_{2,k}) + \gamma_Q \cdot q_k \quad (4.1)$$

$$E_{d,ULS} = 1,15 \cdot (0,6 \text{ kN/m}^2 + 0,3 \text{ kN/m}^2) + 1,5 \cdot 2,0 \text{ kN/m}^2$$

$$E_{d,ULS} = 4,03 \text{ kN/m}^2$$

Note: Safety factors γ_G and γ_Q are according to Finnish National annex of Eurocode 0.

The most critical serviceability limit state (SLS) load combination:

$$E_{d,SLS} = \gamma_G \cdot (g_{1,k} + g_{2,k}) + \gamma_G \cdot q_k \quad (4.1)$$

$$E_{d,SLS} = 1,0 \cdot (0,6 \text{ kN/m}^2 + 0,3 \text{ kN/m}^2) + 1,0 \cdot 2,0 \text{ kN/m}^2$$

$$E_{d,SLS} = 2,9 \text{ kN/m}^2$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

ULS design

Bending moment resistance

$$M_d = E_{d,ULS} \cdot s \cdot L^2 / 8 = 4,03 \text{ kN/m}^2 \cdot 0,4 \text{ m} \cdot (4,5 \text{ m})^2 / 8 = 4,1 \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{4,1 \text{ kNm}}{4,32 \cdot 10^5 \text{ mm}^3} = 9,5 \text{ N/mm}^2$$

$$f_{m,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot f_{m,0,edge,k} \quad (4.3)$$

$$f_{m,0,edge,d} = \frac{0,8}{1,2} \cdot 1,034 \cdot 44 \frac{\text{N}}{\text{mm}^2} = 30,3 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq f_{m,0,edge,d} \rightarrow \text{OK}$$

Lateral torsional buckling is prevented by the fixing of the decking.

Shear resistance

$$V_d = E_{d,ULS} \cdot s \cdot L / 2 = 4,03 \text{ kN/m}^2 \cdot 0,4 \text{ m} \cdot 4,5 \text{ m} / 2 = 3,6 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_d}{2 \cdot A} = \frac{3 \cdot 3,6 \text{ kN}}{2 \cdot 10 \cdot 800 \text{ mm}^2} = 0,5 \text{ N/mm}^2$$

$$f_{v,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{v,0,edge,k} = \frac{0,8}{1,2} \cdot 4,2 \frac{\text{N}}{\text{mm}^2} = 2,8 \text{ N/mm}^2$$

$$\tau_{m,d} \leq f_{v,0,edge,d} \rightarrow \text{OK}$$

Compression perpendicular to the grain

$$F_{c,90,d} = V_d = 3,6 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{c,90,d}}{b \cdot (l_{support} + 15 \text{ mm})} \quad (4.14)$$

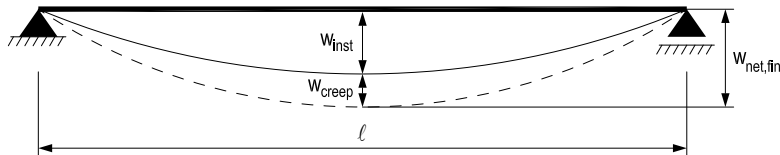
$$\sigma_{c,90,d} = \frac{3,6 \text{ kN}}{45 \text{ mm} \cdot (45 \text{ mm} + 15 \text{ mm})} = 1,35 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,edge,d} = k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,edge,k} \quad (4.13)$$

$$k_{c,90} \cdot f_{c,90,edge,d} = 1,0 \cdot \frac{0,8}{1,2} \cdot 6 \text{ N/mm}^2 = 4 \text{ N/mm}^2 \quad (k_{c,90} \text{ Table 4.7})$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,edge,d} \rightarrow \text{OK}$$

SLS design



Instantaneous deflection

$$(4.74)$$

$$w_{\text{inst,q}} = \frac{5 \cdot q_{\text{d,SLS}} \cdot s \cdot L^4}{384 \cdot E_{\text{mean}} \cdot I} + \frac{6/5 \cdot q_{\text{d,SLS}} \cdot s \cdot L^2}{8 \cdot G_{\text{mean}} A} = 5,97 \text{ mm} + 0,38 \text{ mm} = 6,35 \text{ mm}$$

$$w_{\text{inst}} = 2,86 \text{ mm} + 6,35 \text{ mm} = 9,2 \text{ mm}$$

$$\text{Requirement: } w_{\text{inst}} \leq \frac{L}{400}$$

Final deflection

$$w_{\text{net,fin}} = (1 + k_{\text{def}}) \cdot w_{\text{inst,g}} + (1 + \psi_2 \cdot k_{\text{def}}) \cdot w_{\text{inst,q}} \quad (4.73)$$

For the load category A: $\psi_2 = 0,3$

$$w_{\text{net,fin}} = (1 + 0,6) \cdot 2,86 \text{ mm} + (1 + 0,3 \cdot 0,6) \cdot 6,35 \text{ mm} = 12,1 \text{ mm}$$

Vibration design

Lowest natural frequency f_1

$$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_1}{m}} \quad (4.79)$$

$$m = g_1 + g_2 + 30 \text{ kg/m}^2 = 60 \text{ kg/m}^2 + 30 \text{ kg/m}^2 + 30 \text{ kg/m}^2 = 120 \text{ kg/m}^2$$

Note: In Finnish NA the share of live load q_k in the frequency calculation is 30 kg/m^2

$$(EI)_1 = EI \cdot (1000/s) = 7,15 \cdot 10^{11} \text{ Nmm}^2 \cdot (1000/400 \text{ mm})$$

$$(EI)_1 = 1,79 \cdot 10^6 \text{ Nm}^2/\text{m}$$

$$f_1 = \frac{\pi}{2 \cdot (4,5\text{m})^2} \sqrt{\frac{1,79 \cdot 10^6 \text{ Nm}^2/\text{m}}{120 \text{ kg/m}^2}} = 9,5 \text{ Hz} > 8 \text{ Hz} \rightarrow \text{OK}$$

→ The floor can be analyzed as a high frequency floor.

Floor stiffness perpendicular to the span direction based on 22 mm chipboard decking:

$$(EI)/m = 3500 \text{ N/mm}^2 \cdot 1000 \text{ mm} \cdot (22 \text{ mm})^3/12 = 3,11 \cdot 10^3 \text{ Nm}^2/\text{m}$$

For a rectangular floor with overall dimensions $b \times l$, simply supported along all four edges, the impulse velocity response v [m/Ns²] value may, as an approximation, be taken as:

$$v = \frac{4(0,4+0,6n_{40})}{mbl+200} \quad (4.80)$$

$$n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{b}{l} \right)^4 \frac{(EI)_1}{(EI)_b} \right\}^{0,25} \quad (4.81)$$

$$n_{40} = \left\{ \left(\left(\frac{40}{9,5 \text{ Hz}} \right)^2 - 1 \right) \cdot \left(\frac{5 \text{ m}}{4,5 \text{ m}} \right)^4 \cdot \frac{1,79 \cdot 10^6 \text{ Nm}^2/\text{m}}{3,11 \cdot 10^3 \text{ Nm}^2/\text{m}} \right\}^{0,25} = 11$$

$$v = \frac{4(0,4+0,6n_{40})}{mbl+200} = \frac{4(0,4+0,6 \cdot 11)}{120 \cdot 5 \cdot 4,5 + 200} = 0,010$$

When a high value $b = 150$ is chosen from the Figure 4.28 and a conservative damping value $\xi = 0,01$ is used, the requirement for v is

$$v \leq 150(f_1 \xi^{-1}) = 0,011 \rightarrow \text{OK} \quad (4.78)$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

The deflection under $F = 1$ kN point load

$$w = \min \left\{ \begin{array}{l} \frac{F \cdot l^2}{42 \cdot k_\delta \cdot (EI)_I} \\ \frac{F \cdot l^3}{48 \cdot s \cdot (EI)_I} \end{array} \right. \quad (4.82)$$

$$k_\delta = \sqrt[4]{\frac{(EI)_b}{(EI)_I}} = \sqrt[4]{\frac{3,11 \cdot 10^3}{1,79 \cdot 10^6}} = 0,2 \quad (4.84)$$

$$w = \min \left\{ \begin{array}{l} \frac{1 \text{ kN} \cdot (4,5 \text{ m})^2}{42 \cdot 0,2 \cdot 1,79 \cdot \frac{10^6 \text{ Nm}^2}{\text{m}}} \\ \frac{1 \text{ kN} \cdot (4,5 \text{ m})^3}{48 \cdot 0,4 \cdot 1,79 \cdot \frac{10^6 \text{ Nm}^2}{\text{m}}} \end{array} \right. = \min \left\{ \begin{array}{l} 1,3 \text{ mm} \\ 2,7 \text{ mm} \end{array} \right. = 1,3 \text{ mm}$$

According to the Finnish national Annex of EC5, for the span length $L = 4500$ mm the requirement is $w \leq 0,6$ mm and according to the Austrian national annex $w \leq 0,5$ mm which are not fulfilled by the structure.

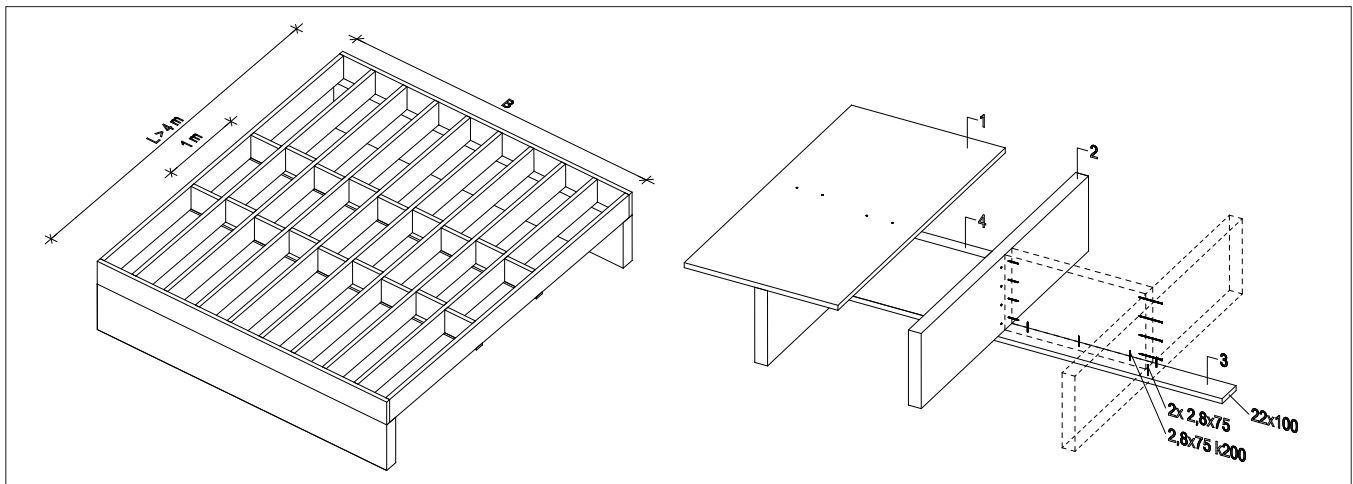
The performance can be improved by transverse bracing by 2 blockings lines with tension boards close to the centre of the span and cross battens 45×45 c/c 400 mm underneath the floor.

The stiffness $(EI)_{\text{bracing}}$ of the bracing lines can be estimated according to EN1995-1-1, Annex B Mechanically jointed beams. For simplification, also the top flange of the transverse bracing is modelled as a 22×100 C14 board and the stiffness of the 2 bracing lines are divided for the whole floor span length.

$$(EI)_{\text{bracing}} = n \cdot 2 \cdot E_{0,\text{mean}} \cdot \frac{\left(b_0 \cdot \frac{h_0^3}{12} + \gamma \cdot b_0 \cdot h_0 \cdot (h/2 + h_0/2)^2 \right)}{l_0}$$

where

- n = number of bracing lines in each span = 2
- $E_{0,\text{mean}}$ = mean modulus of elasticity of C14 flanges 7000 N/mm²
- b_0 = width of tension/compression chord = 100 mm
- h_0 = thickness of tension/compression chord = 22 mm
- h = depth of joist = 240 mm
- s_0 = Spacing of the nailing between blocking and flanges = 200 mm
- l_0 = span length of the bracing line = width of the floor = 5 m
- ρ_k = characteristic density (C14: 350 kg/m³)
- d = diameter of the nail 2.5 mm for nails 2,5x60 mm



1. Decking panel, 2. Floor joists, 3. Transverse tension board (C18, min 22x100) under the joists fixed with 2,8x75 nails to the joists and blockings, 4. Blocking

$$K_{\text{ser}} = \rho_k^{1,5} \cdot \frac{d^{0,8}}{30} = 454 \text{ N/mm}$$

$$\gamma = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot h_0 \cdot b_0 \cdot s_0}{K_{\text{ser}} \cdot L_0^2}} = \frac{1}{1 + \frac{\pi^2 \cdot 7000 \text{ N/mm}^2 \cdot 22 \text{ mm} \cdot 100 \text{ mm} \cdot 200 \text{ mm}}{454 \text{ N/mm} \cdot (5000 \text{ mm})^2}}$$

$$\gamma = 0,27$$

$$(EI)_{\text{bracing}} = 2 \cdot 2 \cdot \frac{7000 \text{ N}}{\text{mm}^2} \cdot \frac{\left(100 \text{ mm} \cdot \frac{(22 \text{ mm})^3}{12} + 0,27 \cdot 100 \text{ mm} \cdot 22 \text{ mm} \cdot \left(\frac{240 \text{ mm}}{2} + \frac{22 \text{ mm}}{2}\right)^2\right)}{4,5 \text{ m}}$$

$$(EI)_{\text{bracing}} = 6,44 \cdot 10^4 \text{ Nm}^2/\text{m}$$

The stiffness of the 45x45 mm cross battens C14 solid wood underneath the floor:

$$(EI)_{\text{cross battens}} = 7000 \frac{\text{N}}{\text{mm}^2} \cdot \frac{1000 \text{ mm}}{400 \text{ mm}} \cdot \frac{45 \text{ mm} \cdot (45 \text{ mm})^2}{12} = 5,98 \cdot 10^3 \text{ Nm}^2/\text{m}$$

$$(EI)_b = (EI)_{\text{decking}} + (EI)_{\text{bracings}} + (EI)_{\text{cross battens}}$$

$$(EI)_b = 3,11 \cdot 10^3 \text{ Nm}^2/\text{m} + 6,44 \cdot 10^4 \text{ Nm}^2/\text{m} + 5,98 \cdot 10^3 \text{ Nm}^2/\text{m}$$

$$(EI)_b = 7,35 \cdot 10^4 \text{ Nm}^2/\text{m}$$

$$k_{\delta,2} = \sqrt[4]{\frac{(EI)_b}{(EI)_1}} = \sqrt[4]{\frac{6,44 \cdot 10^4}{1,79 \cdot 10^6}} = 0,45$$

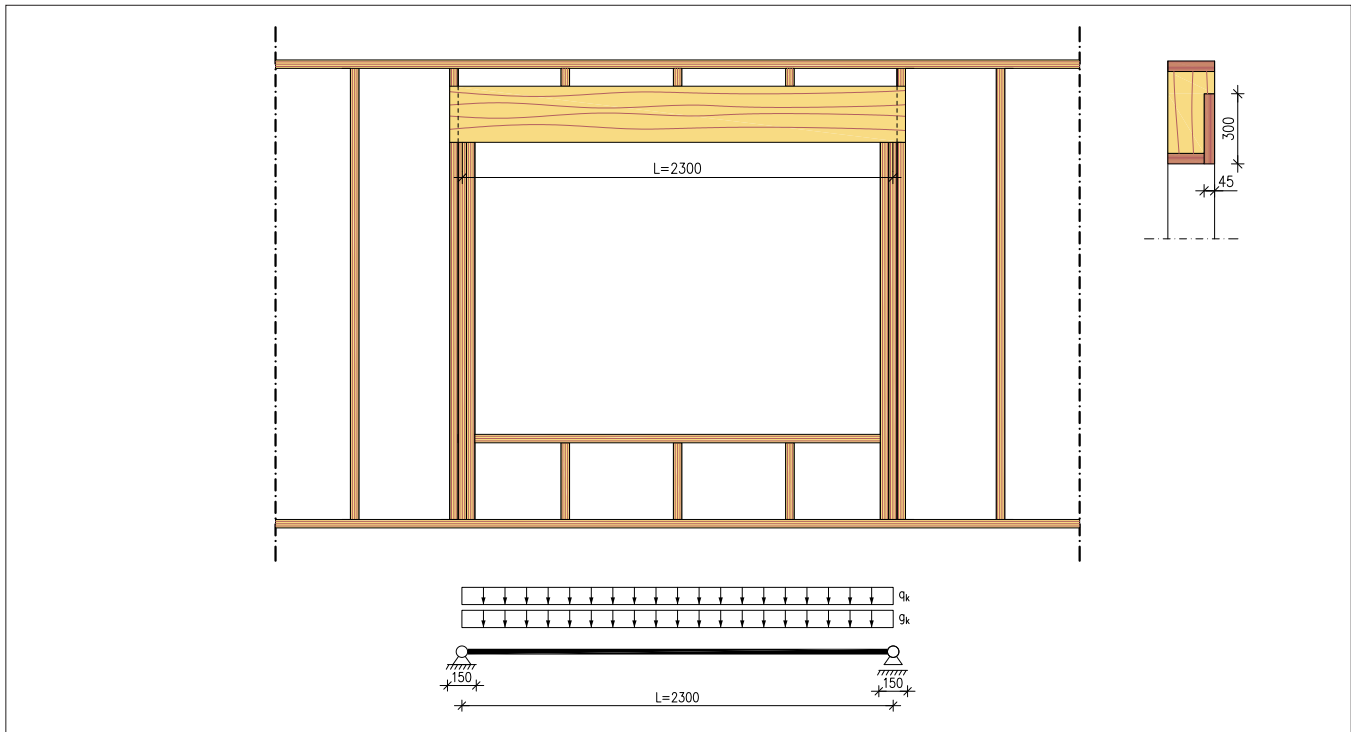
$$w_2 = \min \left\{ \begin{array}{l} \frac{1 \text{ kN} \cdot (4,5 \text{ m})^2}{42 \cdot 0,45 \cdot 1,79 \cdot \frac{10^6 \text{ Nm}^2}{\text{m}}} \\ \frac{1 \text{ kN} \cdot (4,5 \text{ m})^3}{48 \cdot 0,4 \cdot 1,79 \cdot \frac{10^6 \text{ Nm}^2}{\text{m}}} \end{array} \right\} = \min \left\{ \begin{array}{l} 0,6 \text{ mm} \\ 2,7 \text{ mm} \end{array} \right\} = 0,6 \text{ mm} \rightarrow \text{OK}$$

The transfer bracing lines and cross battens underneath the joists improve the floor stiffness under a 1kN point load significantly and the vibration design requirements are fulfilled. The deflection could be decreased ~0,2 mm more, by gluing the decking onto the joists.

9. CALCULATION EXAMPLES OF LVL STRUCTURES

9.2 LINTEL OVER A WINDOW OPENING

Single span lintel beam over a window opening in a one family house is LVL 48 P beam 45x300 mm. Span length is $L = 2300$ mm and the lintel is carrying load from 5000m wide roof area. Support length is 150mm. Snow load s_k is 2,75 kN/m² and own weight of the structure is 1,0 kN/m². Service class SC1.



Beam properties:

Bending strength edgewise $f_{m,0,edge,k}$	= 44 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,2 N/mm ²
Compression perpendicular to grain edgewise $f_{c,90,edge,k}$	= 6 N/mm ²
Modulus of elasticity $E_{0,k}$	= 11 600 N/mm ²
Modulus of elasticity $E_{0,mean}$	= 13 800 N/mm ²
Modulus of rigidity $G_{0,edge,k}$	= 600 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 400 N/mm ²
Area of cross section $A = b \cdot h$	= 13500 mm ²
Section modulus $W_y = b \cdot h^2 / 6$	= 6,75 · 10 ⁵ mm ³
Moment of inertia $I_y = b \cdot h^3 / 12$	= 1,01 · 10 ⁸ mm ⁴
Moment of inertia $I_z = h \cdot b^3 / 12$	= 2,28 · 10 ⁶ mm ⁴
Torsion moment of inertia $I_{tor} = 0,3 \cdot h \cdot b^3$	= 8,20 · 10 ⁶ mm ⁴
Moment stiffness of the joist $EI_y = 13800 \text{ N/mm}^2 \cdot 1,01 \cdot 10^8 \text{ mm}^4$	= 1,40 · 10 ¹² Nmm ²
Shear rigidity of the joist $GA = 600 \text{ N/mm}^2 \cdot 13500 \text{ mm}^2$	= 8,10 · 10 ⁶ N
Modification factor k_{mod} for medium-term, SC1	= 0,8
Modification factor k_{def} for SC1	= 0,6
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/300)^{0,15}$	= 1,00

Loading combinations

Snow load at roof level $q_k = \mu_1 \cdot C_e \cdot s_k$. Form factor $\mu_1 = 0,8$, when roof angle is less than 30° and in normal conditions $C_e = 1,0 \rightarrow q_k = 0,8 \cdot 1,0 \cdot 2,75 \text{ N/m}^2 = 2,2 \text{ kN/m}^2$.

The most critical ultimate limit state (ULS) load combination:

$$E_{d,ULS} = \gamma_G \cdot g_k + \gamma_Q \cdot q_k \quad (4.1)$$

$$E_{d,ULS} = 1,15 \cdot \left(5\text{m} \cdot 1,0 \frac{\text{kN}}{\text{m}^2} \right) + 1,5 \cdot 5\text{m} \cdot 2,2 \text{ kN/m}^2 = 22,3 \text{ kN/m}$$

Note: Safety factors γ_G and γ_Q are according to Finnish National annex of Eurocode 0.

The most critical serviceability limit state (SLS) load combination:

$$E_{d,SLS} = \gamma_G \cdot g_k + \gamma_Q \cdot q_k \quad (4.1)$$

$$E_{d,SLS} = 1,0 \cdot (5\text{m} \cdot 1,0 \text{ kN/m}^2 + 1,0 \cdot 5\text{m} \cdot 2,2 \text{ kN/m}^2) = 16,0 \text{ kN/m}$$

ULS design

Bending moment resistance

$$M_d = E_{d,ULS} \cdot s \cdot L^2 / 8 = 22,3 \text{ kN/m} \cdot (2,3\text{m})^2 / 8 = 14,7 \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{14,7 \text{ kNm}}{6,75 \cdot 10^6 \text{ mm}^3} = 21,8 \text{ N/mm}^2$$

$$f_{m,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot f_{m,0,edge,k} = \frac{0,8}{1,2} \cdot 1,00 \cdot 44 \frac{\text{N}}{\text{mm}^2} = 29,3 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq f_{m,0,edge,d} \rightarrow \text{OK}$$

Lateral torsional buckling

The lintel beam is laterally supported to wall studs in 600mm spacing and the load is applied via them. Therefore the effective length is $L_{ef} = 600\text{mm}$ (See table 4.9).

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{L_{ef} W_y} \quad (4.42)$$

$$\sigma_{m,crit} = \frac{\pi \sqrt{10600 \text{ N/mm}^2 \cdot 2,28 \cdot 10^6 \text{ mm}^4 \cdot \frac{400 \text{ N}}{\text{mm}^2} \cdot 8,20 \cdot 10^6 \cdot \text{mm}^4}}{600 \text{ mm} \cdot 6,75 \cdot 10^5 \text{ mm}^3}$$

$$\sigma_{m,crit} = 72,2 \text{ N/mm}^2$$

$$\lambda_{rel} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{44 \text{ N/mm}^2}{72,2 \text{ N/mm}^2}} = 0,78 \quad (4.41)$$

$$\text{when } 0,75 < \lambda_{rel,m} \leq 1,4, k_{crit} = 1,56 - 0,75 \cdot \lambda_{rel,m} = 1,56 - 0,75 \cdot 0,78 = 0,97$$

$$k_{crit} \cdot f_{m,d} = 0,97 \cdot 29,3 \text{ N/mm}^2 = 28,6 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq k_{crit} \cdot f_{m,d} \rightarrow \text{OK}$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

$$k_{crit} \cdot f_{(m,d)} = 0,97 \cdot 29,3 \text{ N/mm}^2 = 28,6 \text{ N/mm}^2$$

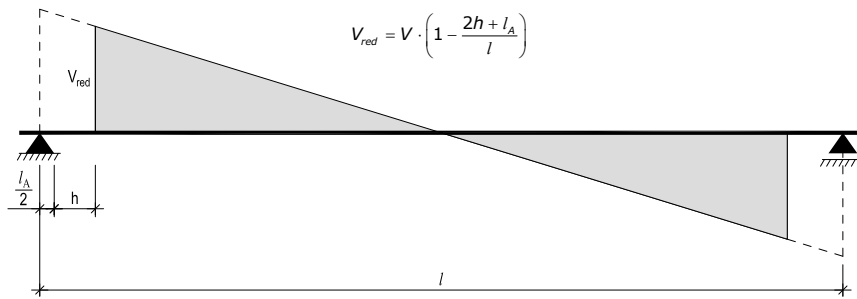
$$V_d = E_{d,ULS} \cdot s \cdot L/2 = 22,3 \text{ kN/m} \cdot 2,3 \text{ m} / 2 = 25,6 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_d}{2 \cdot A} = \frac{3 \cdot 25,6 \text{ kN}}{2 \cdot 13500 \text{ mm}^2} = 2,84 \text{ N/mm}^2$$

$$f_{v,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{v,0,edge,k} = \frac{0,8}{1,2} \cdot 4,2 \frac{\text{N}}{\text{mm}^2} = 2,8 \text{ N/mm}^2$$

$$\tau_{m,d} > f_{v,0,edge,d} \rightarrow \text{NOT OK}$$

The design shear force may be reduced by determining it from a distance of the beam height h from the support edge:



$$V_{red,d} = V_d \cdot \left(1 - \frac{2h + l_{support}}{l}\right) = 25,6 \text{ kN} \cdot \left(1 - \frac{2 \cdot 300 \text{ mm} + 150 \text{ mm}}{2300 \text{ mm}}\right)$$

$$V_{red,d} = 17,2 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_d}{2 \cdot A} = \frac{3 \cdot 17,2 \text{ kN}}{2 \cdot 13500 \text{ mm}^2} = 1,92 \text{ N/mm}^2$$

$$\tau_{m,d} < f_{v,0,edge,d} \rightarrow \text{OK}$$

Compression perpendicular to grain

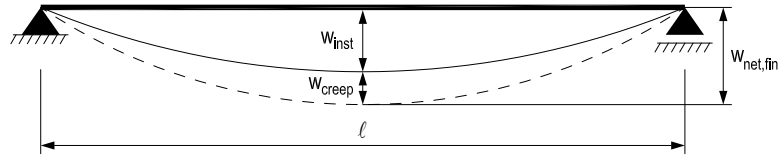
$$F_{c,90,d} = V_d = 25,6 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{c,90,d}}{b \cdot (l_{support} + 15 \text{ mm})} \quad (4.14)$$

$$\sigma_{c,90,d} = \frac{25,6 \text{ kN}}{45 \text{ mm} \cdot (150 \text{ mm} + 15 \text{ mm})} = 3,4 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,edge,d} = k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,edge,k} = 1,0 \cdot \frac{0,8}{1,2} \cdot 6 \text{ N/mm}^2 = 4 \text{ N/mm}^2$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,edge,d} \rightarrow \text{OK} \quad (4.13)$$

SLS design**Instantaneous deflection**

$$w_{inst} = w_{inst,g} + w_{inst,q}$$

$$w_{inst,g} = \frac{5 \cdot g_{d,SLS} \cdot s \cdot L^4}{384 \cdot E_{mean} \cdot I} + \frac{6/5 \cdot g_{d,SLS} \cdot s \cdot L^2}{8 \cdot G_{mean} A} = 1,30 \text{ mm} + 0,49 \text{ mm} = 1,79 \text{ mm} \quad (4.74)$$

$$w_{inst,g} = 1,30 \text{ mm} + 0,49 \text{ mm} = 1,79 \text{ mm}$$

$$w_{inst,q} = \frac{5 \cdot q_{d,SLS} \cdot s \cdot L^4}{384 \cdot E_{mean} \cdot I} + \frac{6/5 \cdot q_{d,SLS} \cdot s \cdot L^2}{8 \cdot G_{mean} A} = 2,87 \text{ mm} + 1,08 \text{ mm} = 3,95 \text{ mm}$$

$$w_{inst} = 1,79 \text{ mm} + 3,95 \text{ mm} = 5,5 \text{ mm}$$

Final deflection

$$w_{net,fin} = (1 + k_{def}) \cdot w_{inst,g} + (1 + \psi_2 \cdot k_{def}) \cdot w_{inst,q} \quad (4.73)$$

Note: For the snow load in Finnish National annex: $\psi_2 = 0,2$.

$$w_{net,fin} = (1 + 0,6) \cdot 1,79 \text{ mm} + (1 + 0,2 \cdot 0,6) \cdot 3,95 \text{ mm} = 7,3 \text{ mm}$$

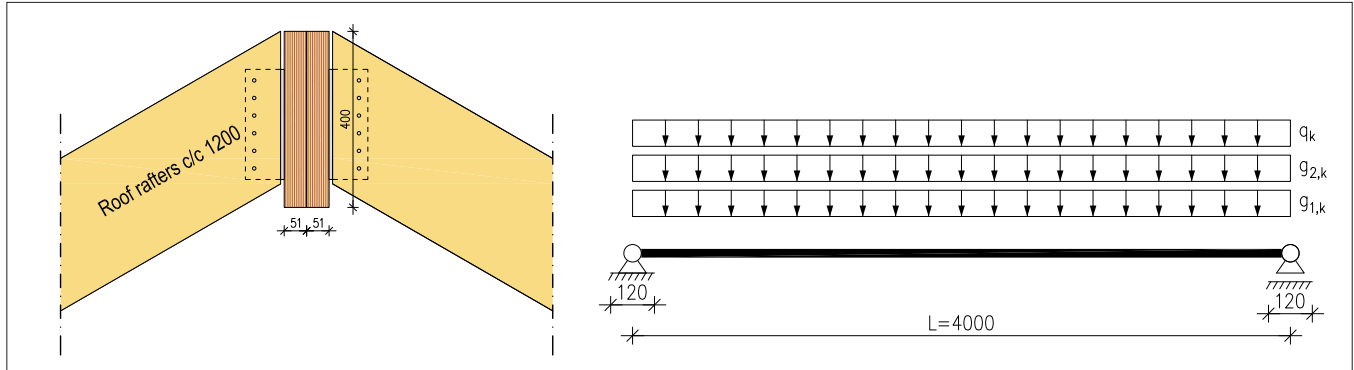
$$\text{When the requirement is } w_{net,fin} \leq \frac{L}{300}, \quad \frac{2300 \text{ mm}}{300} = 7,7 \text{ mm} \rightarrow \text{OK}$$

The lintel beam fulfils the design requirements. However, in practice the required support lengths are quite long and for windows a more strict deflection limit can be required. Therefore a double lintel 2x45x260 mm or a 69x300 mm lintel from LVL 36 C could be a more suitable choice.

9. CALCULATION EXAMPLES OF LVL STRUCTURES

9.3 DOUBLE LVL 48 P RIDGE BEAM FOR ROOF

Single-span ridge beam of the roof in a one family house is LVL 48 P double beam 2x51x400 mm. Span length is $L = 4000$ mm, width of the loading area 6000 mm and roof rafters connected to the sides of the beam at spacing $s = 1200$ mm. Support length is 120 mm. Snow load s_k is 2,5 kN/m², own weight of the roof structure is 1,0 kN/m² and own weight of the beam is 0,2 kN/m. Service class SC1.



Beam properties:

Bending strength edgewise $f_{m,0,edge,k}$	= 44 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,2 N/mm ²
Compression perpendicular to grain edgewise $f_{c,90,edge,k}$	= 6 N/mm ²
Modulus of elasticity $E_{0,k}$	= 11 600 N/mm ²
Modulus of elasticity $E_{0,mean}$	= 13 800 N/mm ²
Modulus of rigidity $G_{0,edge,k}$	= 600 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 400 N/mm ²
Area of cross section $A = 2 \times b \cdot h$	= 40800 mm ²
Section modulus $W_y = 2 \times b \cdot h^2/6$	= 2,72·10 ⁶ mm ³
Moment of inertia $I_y = 2 \times b \cdot h^3/12$	= 5,44·10 ⁸ mm ⁴
Moment of inertia $I_z = 2 \times h \cdot b^3/12$	= 8,84·10 ⁶ mm ⁴
Torsion moment of inertia $I_{tor} = 2 \times 0,3 \cdot h \cdot b^3$	= 3,18·10 ⁷ mm ⁴
Moment stiffness of the joist $E_{Iy} = 13800 \text{ N/mm}^2 \cdot 5,44 \cdot 10^8 \text{ mm}^4$	= 7,51·10 ¹² Nmm ²
Shear rigidity of the joist $GA = 600 \text{ N/mm}^2 \cdot 40800 \text{ mm}^2$	= 2,45·10 ⁷ N
Modification factor k_{mod} for medium-term, SC1	= 0,8
Modification factor k_{def} for SC1	= 0,6
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/400)^{0,15}$	= 0,96

Loading combinations

Snow load at roof level $q_k = \mu_1 \cdot C_e \cdot s_k$. Form factor $\mu_1 = 0,8$, when roof angle is less than 30° and in normal conditions $C_e = 1,0 \rightarrow q_k = 0,8 \cdot 1,0 \cdot 2,5 \text{ N/m}^2 = 2,0 \text{ kN/m}^2$.

Most critical ultimate limit state (ULS) load combination:

$$E_{d,ULS} = \gamma_G \cdot (g_{1,k} + g_{2,k}) + \gamma_Q \cdot q_k \quad (4.1)$$

$$E_{d,ULS} = 1,15 \cdot (6\text{m} \cdot 1,0 \text{ kN/m}^2 + 0,2 \text{ kN/m}) + 1,5 \cdot 6\text{m} \cdot 2,0 \text{ kN/m}^2$$

$$E_{d,ULS} = 25,1 \text{ kN/m}$$

Note: Safety factors γ_G and γ_Q are according to Finnish National annex of Eurocode 0.

The most critical serviceability limit state (SLS) load combination:

$$E_{d,SLS} = \gamma_G \cdot (g_{1,k} + g_{2,k}) + \gamma_Q \cdot q_k \quad (4.1)$$

$$E_{d,SLS} = 1,0 \cdot (6\text{m} \cdot 1,0 \text{ kN/m}^2 + 0,2 \text{ kN/m}) + 6,0 \cdot 1,0 \cdot 2,0 \text{ kN/m}^2$$

$$E_{d,SLS} = 18,2 \text{ kN/m}$$

ULS design

Bending moment resistance

$$M_d = E_{d,ULS} \cdot s \cdot L^2/8 = 25,1 \text{ kN/m} \cdot (4\text{m})^2/8 = 50,3 \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{50,3 \text{ kNm}}{2,72 \cdot 10^6 \text{ mm}^3} = 18,5 \text{ N/mm}^2$$

$$f_{m,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot f_{m,0,edge,k} = \frac{0,8}{1,2} \cdot 0,96 \cdot 44 \frac{\text{N}}{\text{mm}^2} = 28,1 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq f_{m,0,edge,d} \rightarrow \text{OK}$$

Lateral torsional buckling

The ridge beam is loaded by the roof rafters connected at the sides of the beam at 1200 mm spacing and they act as supports against lateral torsional buckling, so the effective length is $L_{ef} = 1200 \text{ mm}$.

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{l_{ef} W_y} \quad (4.42)$$

$$\sigma_{m,crit} = \frac{\pi \sqrt{10600 \text{ N/mm}^2 \cdot 8,84 \cdot 10^6 \text{ mm}^4 \cdot 400 \text{ N/mm}^2 \cdot 3,18 \cdot 10^7 \cdot \text{mm}^4}}{1200 \text{ mm} \cdot 2,72 \cdot 10^6 \text{ mm}^3}$$

$$\sigma_{m,crit} = 34,8 \text{ N/mm}^2$$

$$\lambda_{rel} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{44 \text{ N/mm}^2}{34,8 \text{ N/mm}^2}} = 1,12 \quad (4.41)$$

$$\text{when } 0,75 < \lambda_{rel,m} \leq 1,4, k_{crit} = 1,56 - 0,75 \cdot \lambda_{rel,m} = 1,56 - 0,75 \cdot 1,12 = 0,72$$

$$k_{crit} \cdot f_{m,d} = 0,72 \cdot 28,1 \text{ N/mm}^2 = 20,1 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq k_{crit} \cdot f_{m,d} \rightarrow \text{OK} \quad (4.38)$$

Shear resistance

$$V_d = E_{d,ULS} \cdot s \cdot L/2 = 25,1 \text{ kN/m} \cdot 4,0 \text{ m}/2 = 50,3 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_d}{2 \cdot A} = \frac{3 \cdot 50,3 \text{ kN}}{2 \cdot 40 \cdot 800 \text{ mm}^2} = 1,9 \text{ N/mm}^2$$

$$f_{v,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{v,0,edge,k} = \frac{0,8}{1,2} \cdot 4,2 \frac{\text{N}}{\text{mm}^2} = 2,8 \text{ N/mm}^2$$

$$\tau_{m,d} \leq f_{v,0,edge,d} \rightarrow \text{OK}$$

Compression perpendicular to grain

$$F_{c,90,d} = V_d = 50,3 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{c,90,d}}{b \cdot (l_{support} + 15 \text{ mm})} \quad (4.14)$$

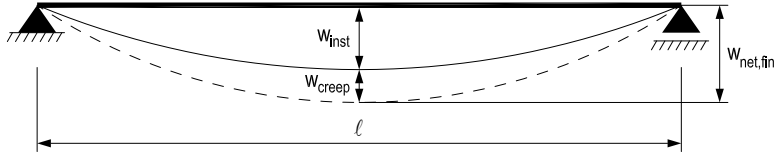
$$\sigma_{c,90,d} = \frac{50,3 \text{ kN}}{2 \cdot 51 \text{ mm} \cdot (120 \text{ mm} + 15 \text{ mm})} = 3,7 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,edge,d} = k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,edge,k} = 1,0 \cdot \frac{0,8}{1,2} \cdot 6 \text{ N/mm}^2 = 4 \text{ N/mm}^2$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,edge,d} \rightarrow \text{OK} \quad (4.13)$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

SLS design



Instantaneous deflection

$$w_{\text{inst}} = w_{\text{inst,g}} + w_{\text{inst,q}}$$

$$w_{\text{inst,g}} = \frac{5 \cdot g_{\text{d,SLS}} \cdot s \cdot L^4}{384 \cdot E_{\text{mean}} \cdot I} + \frac{6/5 \cdot g_{\text{d,SLS}} \cdot s \cdot L^2}{8 \cdot G_{\text{mean}} A} = 2,76 \text{ mm} + 0,61 \text{ mm} = 3,36 \text{ mm}$$

$$w_{\text{inst,q}} = \frac{5 \cdot q_{\text{d,SLS}} \cdot s \cdot L^4}{384 \cdot E_{\text{mean}} \cdot I} + \frac{6/5 \cdot q_{\text{d,SLS}} \cdot s \cdot L^2}{8 \cdot G_{\text{mean}} A} = 5,33 \text{ mm} + 1,18 \text{ mm} = 6,51 \text{ mm}$$

$$w_{\text{inst}} = 3,36 \text{ mm} + 6,51 \text{ mm} = 9,9 \text{ mm}$$

Final deflection

$$w_{\text{net,fin}} = (1 + k_{\text{def}}) \cdot w_{\text{inst,g}} + (1 + \psi_2 \cdot k_{\text{def}}) \cdot w_{\text{inst,q}} \quad (4.73)$$

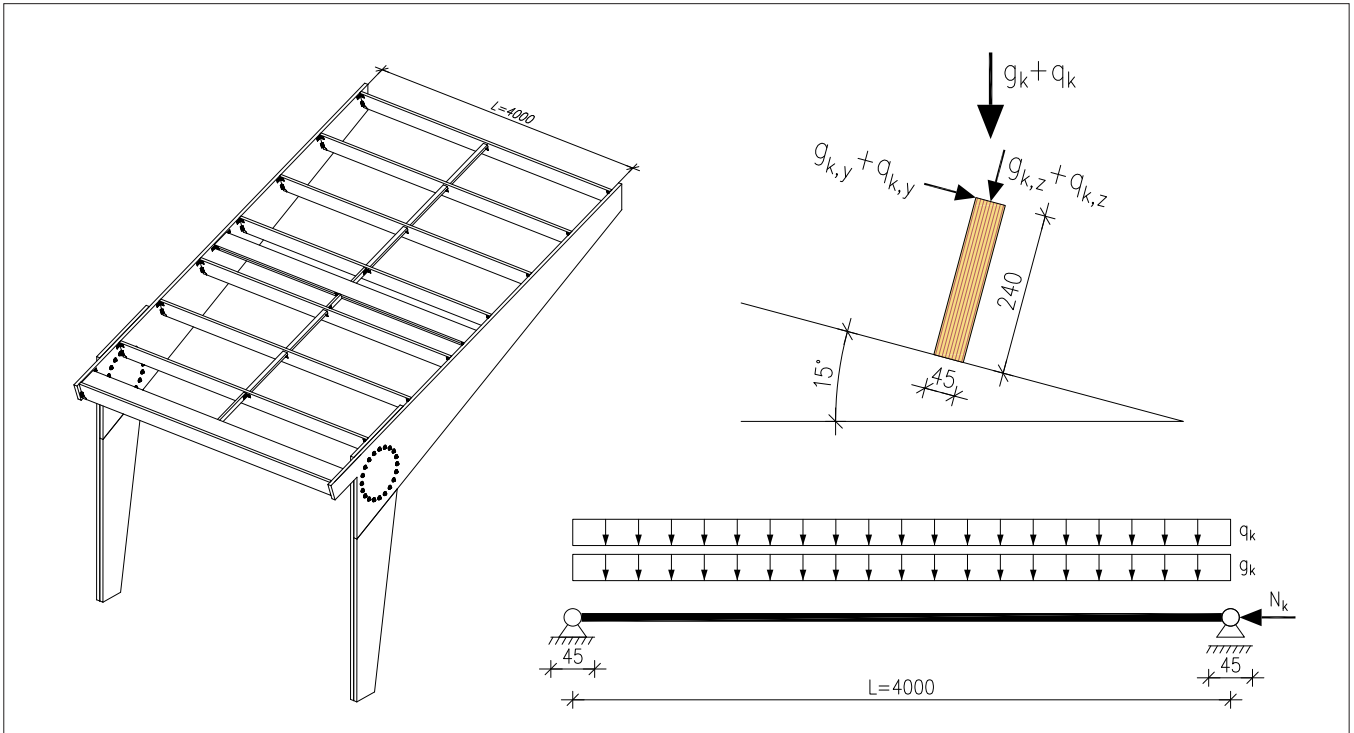
Note: For the snow load in Finnish National annex: $\psi_2 = 0,2$.

$$w_{\text{net,fin}} = (1 + 0,6) \cdot 3,36 \text{ mm} + (1 + 0,2 \cdot 0,6) \cdot 6,51 \text{ mm} = 12,7 \text{ mm}$$

When the requirement is $w_{\text{net,fin}} \leq \frac{L}{300}$, $\frac{4000 \text{ mm}}{300} = 13,3 \text{ mm} \rightarrow \text{OK}$

9.4 ROOF PURLIN

Single-span purlins of the roof of an unheated (service class 2) portal frame hall are 45x240 LVL 48 P beams. Roof angle is 15° , span length is $L = 4000$ mm, spacing $s = 900$ mm and the purlins are perpendicular to the roof plane having a side support line at the middle of the span. Support length is 45 mm. Snow load s_k is $2,5$ kN/m² and own weight of the roof structure is $0,30$ kN/m². For simplification there is no wind load in the example. The purlins are connected to the sides of the portal frame beams and they act also as lateral torsional buckling supports. Therefore the purlins have an axial load of $N_k = 3$ kN mainly from snow load. Service class SC2.



Beam properties:

Bending strength edgewise $f_{m,0,edge,k}$	= 44 N/mm ²
Bending strength flatwise $f_{m,0,flat,k}$	= 48 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,2 N/mm ²
Compression parallel to grain $f_{c,0,SC2,k}$	= 29 N/mm ²
Compression perpendicular to grain edgewise $f_{c,90,edge,k}$	= 6 N/mm ²
Modulus of elasticity $E_{0,k}$	= 11 600 N/mm ²
Modulus of elasticity $E_{0,mean}$	= 13 800 N/mm ²
Modulus of rigidity $G_{0,edge,k}$	= 600 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 400 N/mm ²
Area of cross section $A = b \cdot h$	= 10800 mm ²
Section modulus $W_y = b \cdot h^2/6$	= $4,32 \cdot 10^5$ mm ³
Section modulus $W_z = h \cdot b^2/6$	= $8,10 \cdot 10^4$ mm ³
Moment of inertia $I_y = b \cdot h^3/12$	= $5,18 \cdot 10^7$ mm ⁴
Moment of inertia $I_z = h \cdot b^3/12$	= $1,82 \cdot 10^6$ mm ⁴
Torsion moment of inertia $I_{tor} = 0,3 \cdot h \cdot b^3$	= $6,56 \cdot 10^6$ mm ⁴
Moment stiffness of the joist $EI_y = 13800 \text{ N/mm}^2 \cdot 6,56 \cdot 10^6 \text{ mm}^4$	= $7,15 \cdot 10^{11}$ Nmm ²
Shear rigidity of the joist $GA = 600 \text{ N/mm}^2 \cdot 40800 \text{ mm}^2$	= $6,48 \cdot 10^6$ N
Modification factor k_{mod} for medium-term, SC2	= 0,8
Modification factor k_{def} for SC2	= 0,8
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/240)^{0,15}$	= 1,034

9. CALCULATION EXAMPLES OF LVL STRUCTURES

Loading combinations

Own weight in z-direction $g_{k,z}: \cos 15^\circ \cdot 0,9\text{m} \cdot 0,3 \text{ kN/m}^2 = 0,26 \text{ kN/m}$

Own weight in y-direction $g_{k,y}: \sin 15^\circ \cdot 0,9\text{m} \cdot 0,4 \text{ kN/m}^2 = 0,07 \text{ kN/m}$.

Snow load at roof level $q_k = \mu_1 \cdot C_e \cdot s_k$ Form factor $\mu_1 = 0,8$, when roof angle is less than 30° and in normal conditions $C_e = 1,0$.

$q_k = 0,8 \cdot 1,0 \cdot 2,5 \text{ N/m}^2 = 2 \text{ kN/m}^2$ (horizontal projection).

$q_{k,z} = \cos 15^\circ \cdot \cos 15^\circ \cdot 2 \text{ kN/m} = 1,68 \text{ kN/m}$

$q_{k,y} = \cos 15^\circ \cdot \sin 15^\circ \cdot 2 \text{ kN/m} = 0,45 \text{ kN/m}$

The most critical ultimate limit state (ULS) load combination:

$E_{d,z,ULS} = \gamma_G \cdot g_{k,z} + \gamma_Q \cdot q_{k,z}$

$E_{d,z,ULS} = 1,15 \cdot 0,26 \text{ kN/m}^2 + 1,5 \cdot 1,68 \text{ kN/m}^2 = 2,82 \text{ kN/m}$

$E_{d,y,ULS} = \gamma_G \cdot g_{k,y} + \gamma_Q \cdot q_{k,y}$

$E_{d,y,ULS} = 1,15 \cdot 0,07 \text{ kN/m}^2 + 1,5 \cdot 0,45 \text{ kN/m}^2 = 0,76 \text{ kN/m}$

Axial compression $N_{c,d} = \gamma_Q \cdot N_{c,k} = 1,5 \cdot 3 \text{ kN/m}^2 = 4,5 \text{ kN}$

Note: Safety factors γ_G and γ_Q are according to Finnish national annex of Eurocode 0.

Most critical serviceability limit state (SLS) load combination:

$E_{d,z,SLS} = \gamma_G \cdot g_{k,z} + \gamma_Q \cdot q_{k,z}$

$E_{d,z,SLS} = 1,0 \cdot 0,26 \text{ kN/m}^2 + 1,0 \cdot 1,68 \text{ kN/m}^2 = 1,94 \text{ kN/m}$

ULS design

Bending moment resistance in y-direction

$$M_{d,z} = E_{d,z,ULS} \cdot L^2/8 = 2,82 \text{ kN/m} \cdot (4\text{m})^2/8 = 5,64 \text{ kNm}$$

$$\sigma_{m,y,d} = \frac{M_{d,z}}{W_y} = \frac{5,64 \text{ kNm}}{4,32 \cdot 10^5 \text{ mm}^3} = 13,1 \text{ N/mm}^2$$

$$f_{m,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot f_{m,0,edge,k} = \frac{0,8}{1,2} \cdot 1,034 \cdot 44 \frac{\text{N}}{\text{mm}^2} = 30,3 \text{ N/mm}^2$$

Bending moment resistance in z-direction at centre support of a 2-span beam

$$M_{d,y} = E_{d,y,ULS} \cdot (L/2)^2/8 = 0,76 \text{ kN/m} \cdot (4 \text{ m}/2)^2/8 = 0,38 \text{ kNm}$$

$$\sigma_{m,z,d} = \frac{M_{d,y}}{W_z} = \frac{0,38 \text{ kNm}}{8,10 \cdot 10^4 \text{ mm}^3} = 4,7 \text{ N/mm}^2$$

$$f_{m,0,flat,z,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{m,0,flat,z,k} = \frac{0,8}{1,2} \cdot 48 \frac{\text{N}}{\text{mm}^2} = 32,0 \text{ N/mm}^2$$

Lateral torsional buckling (LTB) is prevented at the middle of the span.

The purlin is loaded from the compression side and supported against torsion at the main supports and in the middle of the span. According to Table 6.1 of EN1995-1-1, for uniformly distributed load, the effective length is $L_{ef} = 2000 \text{ mm} + 2 \cdot 240 \text{ mm} = 2480 \text{ mm}$.

$$\sigma_{m,y,crit} = \frac{M_{z,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{L_{ef} W_y} \quad (4.42)$$

$$\sigma_{m,y,crit} = \frac{\pi \sqrt{10600 \text{ N/mm}^2 \cdot 1,82 \cdot 10^6 \text{ mm}^4 \cdot 400 \text{ N/mm}^2 \cdot 6,56 \cdot 10^6 \cdot \text{mm}^4}}{2480 \text{ mm} \cdot 4,32 \cdot 10^5 \text{ mm}^3}$$

$$\sigma_{m,y,crit} = 21,6 \text{ N/mm}^2$$

$$\lambda_{rel} = \sqrt{\frac{k_h \cdot f_{m,k}}{\sigma_{m,y,crit}}} = \sqrt{\frac{1,03 \cdot 44 \text{ N/mm}^2}{21,6 \text{ N/mm}^2}} = 1,45 \quad (4.41)$$

$$\text{when } 1,4 < \lambda_{\text{rel,m}}, k_{\text{crit}} = \frac{1}{\lambda_{\text{rel,m}}^2} = \frac{1}{1,45^2} = 0,48$$

$$k_{\text{crit}} \cdot f_{\text{m,y,d}} = 0,48 \cdot 30,3 \text{ N/mm}^2 = 14,4 \text{ N/mm}^2 \quad (4.40)$$

$$\sigma_{\text{m,y,d}} \leq k_{\text{crit}} \cdot f_{\text{m,y,d}} \rightarrow \text{OK} \quad (4.38)$$

Axial compression

$$\frac{N_{\text{c,d}}}{A} = \frac{4,5 \text{ kN}}{10800 \text{ mm}^2} = 0,42 \text{ N/mm}^2$$

$$f_{\text{c,0,d}} = \frac{k_{\text{mod}}}{\gamma_{\text{M}}} \cdot f_{\text{c,0,SC2,k}} = \frac{0,8}{1,2} \cdot 29 \frac{\text{N}}{\text{mm}^2} = 19,3 \text{ N/mm}^2$$

Buckling, buckling length $l_{\text{c}} = 2000 \text{ mm}$ in z-direction and 4000 mm in y-direction

$$\lambda_z = \sqrt{12} \left(\frac{l_{\text{c}}}{b} \right) = 3,46 \cdot \frac{2000 \text{ mm}}{45 \text{ mm}} = 154 \quad (4.37)$$

$$\lambda_y = \sqrt{12} \left(\frac{l_{\text{c}}}{h} \right) = 3,46 \cdot \frac{4000 \text{ mm}}{240 \text{ mm}} = 58$$

$$\lambda_{\text{rel,z}} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{\text{c,0,k}}}{E_{0,05}}} = \frac{154}{3,14} \cdot \sqrt{\frac{29 \text{ N/mm}^2}{11600 \text{ N/mm}^2}} = 2,45 \quad (4.36)$$

$$\lambda_{\text{rel,y}} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{\text{c,0,k}}}{E_{0,05}}} = \frac{58}{3,14} \cdot \sqrt{\frac{29 \text{ N/mm}^2}{11600 \text{ N/mm}^2}} = 0,92$$

$$k_z = 0,5 \cdot (1 + 0,1 \cdot (2,45 - 0,3) + (2,45)^2) = 3,61 \quad (4.34)$$

$$k_y = 0,5(1 + \beta_{\text{c}}(\lambda_{\text{rel,y}} - 0,3) + \lambda_{\text{rel,y}}^2)$$

$$k_y = 0,5 \cdot (1 + 0,1 \cdot (0,92 - 0,3) + (0,92)^2) = 0,95$$

$$k_{\text{c,z}} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel,z}}^2}} = \frac{1}{3,61 + \sqrt{3,61^2 - 2,45^2}} = 0,16 \quad (4.32)$$

$$k_{\text{c,y}} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel,y}}^2}} = \frac{1}{0,95 + \sqrt{0,95^2 - 0,92^2}} = 0,83$$

The following expressions shall be satisfied with $k_{\text{m}} = 0,7$ for rectangular cross sections:

$$\frac{\sigma_{\text{c,0,d}}}{k_{\text{c,z}} \cdot f_{\text{c,0,d}}} + k_{\text{m}} \cdot \frac{\sigma_{\text{m,y,d}}}{f_{\text{m,y,d}}} + \frac{\sigma_{\text{m,z,d}}}{f_{\text{m,z,d}}} \leq 1 \quad (4.29)$$

$$\frac{0,42 \frac{\text{N}}{\text{mm}^2}}{0,16 \cdot 19,3 \frac{\text{N}}{\text{mm}^2}} + 0,7 \cdot \frac{13,1 \frac{\text{N}}{\text{mm}^2}}{30,3 \frac{\text{N}}{\text{mm}^2}} + \frac{4,7 \frac{\text{N}}{\text{mm}^2}}{32,0 \frac{\text{N}}{\text{mm}^2}} = 0,14 + 0,30 + 0,15 = 0,59 \rightarrow \text{OK}$$

$$\frac{\sigma_{\text{c,0,d}}}{k_{\text{c,y}} \cdot f_{\text{c,0,d}}} + \frac{\sigma_{\text{m,y,d}}}{f_{\text{m,y,d}}} + k_{\text{m}} \cdot \frac{\sigma_{\text{m,z,d}}}{f_{\text{m,z,d}}} \leq 1 \quad (4.30)$$

$$\frac{0,42 \frac{\text{N}}{\text{mm}^2}}{0,83 \cdot 19,3 \frac{\text{N}}{\text{mm}^2}} + \frac{13,1 \frac{\text{N}}{\text{mm}^2}}{30,3 \frac{\text{N}}{\text{mm}^2}} + 0,7 \cdot \frac{4,7 \frac{\text{N}}{\text{mm}^2}}{32,0 \frac{\text{N}}{\text{mm}^2}} = 0,03 + 0,43 + 0,10 = 0,56 \rightarrow \text{OK}$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

$$\left(\frac{\sigma_{m,y,d}}{k_{crit} \cdot f_{m,o,edge}} \right)^2 + \frac{\sigma_{c,o,d}}{k_{c,z} \cdot f_{c,o,d}} \leq 1 \quad (4.39)$$

$$\left(\frac{13,1 \frac{\text{N}}{\text{mm}^2}}{0,48 \cdot 30,3 \frac{\text{N}}{\text{mm}^2}} \right)^2 + \frac{0,42 \frac{\text{N}}{\text{mm}^2}}{0,16 \cdot 19,3 \frac{\text{N}}{\text{mm}^2}} = 0,82 + 0,14 = 0,96 \rightarrow \text{OK}$$

Shear resistance

$$V_{d,y} = E_{d,ULS} \cdot s \cdot L/2 = 2,92 \text{ kN/m} \cdot 4 \text{ m}/2 = 6,2 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_{d,y}}{2 \cdot A} = \frac{3 \cdot 6,2 \text{ kN}}{2 \cdot 10\,800 \text{ mm}^2} = 0,9 \text{ N/mm}^2$$

$$f_{v,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{v,0,edge,k} = \frac{0,8}{1,2} \cdot 4,2 \frac{\text{N}}{\text{mm}^2} = 2,8 \text{ N/mm}^2$$

$$\tau_{m,d} \leq f_{v,0,edge,d} \rightarrow \text{OK}$$

Compression perpendicular to grain

$$F_{c,90,d} = V_{d,y} = 6,2 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{6,2 \text{ kN}}{45 \text{ mm} \cdot (15 \text{ mm} + 45 \text{ mm})} = 1,2 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,edge,d} = k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,edge,k} = 1,0 \cdot \frac{0,8}{1,2} \cdot 6,0 \text{ N/mm}^2 = 4 \text{ N/mm}^2$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,edge,d} \rightarrow \text{OK}$$

SLS design

Instantaneous deflection

$$w_{inst} = w_{inst,g} + w_{inst,q}$$

$$w_{inst,g} = \frac{5 \cdot g_{d,z,SLS} \cdot L^4}{384 \cdot E_{mean} \cdot I} + \frac{6/5 \cdot g_{d,z,SLS} \cdot L^2}{8 \cdot G_{mean} A} = 1,62 \text{ mm} + 0,13 \text{ mm} = 1,75 \text{ mm} \quad (4.74)$$

$$w_{inst} = 1,75 \text{ mm} + 8,45 \text{ mm} = 10,2 \text{ mm}$$

Final deflection

$$w_{net,fin} = (1 + k_{def}) \cdot w_{inst,g} + (1 + \psi_2 \cdot k_{def}) \cdot w_{inst,q} \quad (4.73)$$

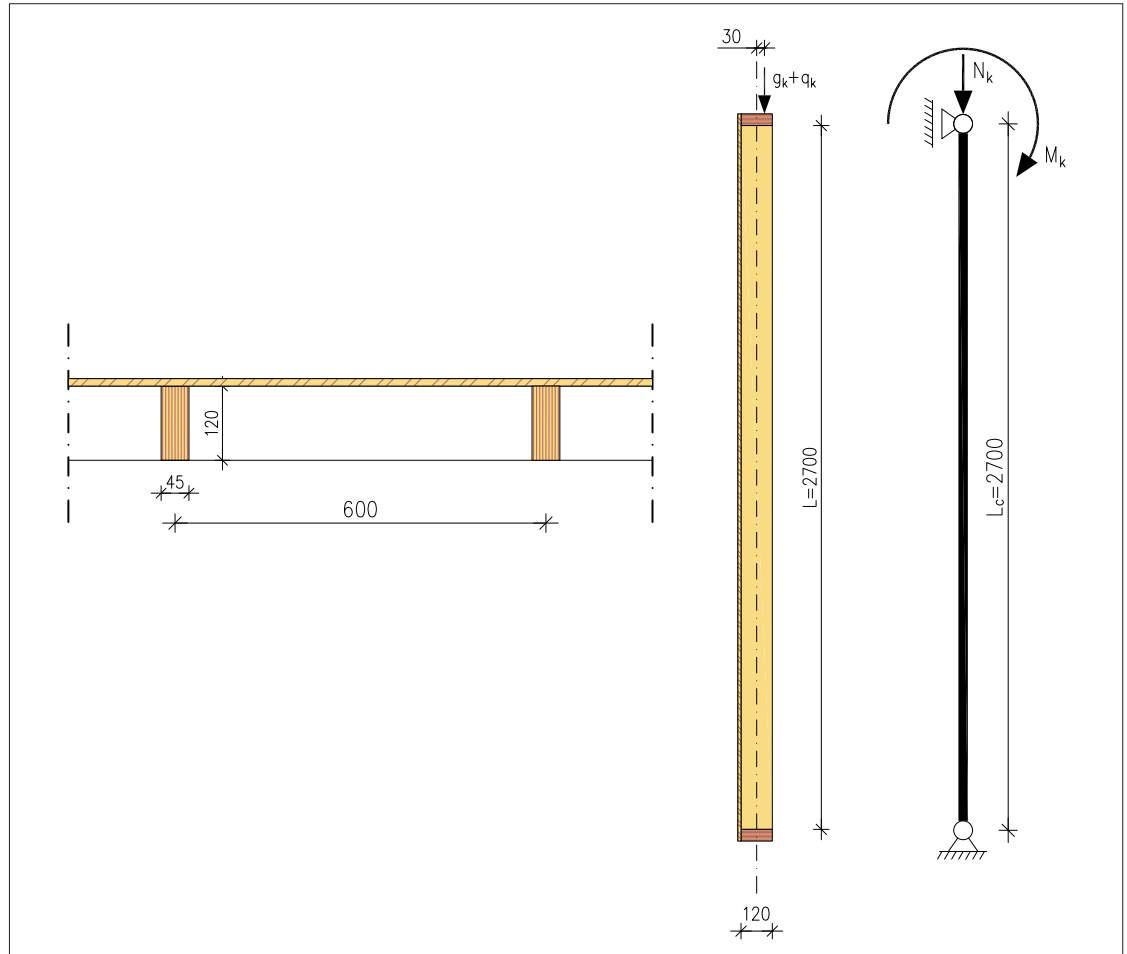
Note: For the snow load in Finnish national annex: $\psi_2 = 0,2$

$$w_{net,fin} = (1 + 0,6) \cdot 1,75 \text{ mm} + (1 + 0,2 \cdot 0,6) \cdot 8,45 \text{ mm} = 12,9 \text{ mm}$$

$$\text{Requirement: } w_{net,fin} \leq \frac{L}{250}, \frac{4000 \text{ mm}}{250} = 16 \text{ mm} \rightarrow \text{OK}$$

9.5 WALL STUD

Load-bearing internal wall is a centre support of an intermediate floor of a 2 storey one family house. 45x120mm LVL 32 P wall stud L is 2700mm and they are at $s = 600\text{mm}$ spacing. Each stud is loaded by the self-weight of the structure g_k is 5kN and imposed load q_k is 11kN. Eccentricity e_z of the loading is assumed to be $\frac{1}{4}$ of the stud width = $120\text{mm}/4 = 30\text{mm}$. Buckling is prevented by wall panelling in the weaker direction. Service class SC1.



Stud properties:

Bending strength edgewise $f_{m,0,edge,k}$	= 27 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4 N/mm ²
Compression parallel to grain $f_{c,0,k}$	= 26 N/mm ²
Modulus of elasticity $E_{0,k}$	= 9 600 N/mm ²
Modulus of elasticity $E_{0,mean}$	= 8 000 N/mm ²
Modulus of rigidity $G_{0,edge,k}$	= 600 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 400 N/mm ²
Area of cross section $A = b \cdot h$	= 5400 mm ²
Section modulus $W_y = b \cdot h^2/6$	= $1,08 \cdot 10^5 \text{ mm}^3$
Moment of inertia $I_y = b \cdot h^3/12$	= $6,48 \cdot 10^6 \text{ mm}^4$
Moment stiffness of the joist $EI_y = 13800 \text{ N/mm}^2 \cdot 1,08 \cdot 10^5 \text{ mm}^4$	= $8,94 \cdot 10^{10} \text{ Nmm}^2$
Shear rigidity of the joist $GA = 600 \text{ N/mm}^2 \cdot 5400 \text{ mm}^2$	= $3,24 \cdot 10^6 \text{ N}$
Modification factor k_{mod} for medium-term, SC1	= 0,8
Modification factor k_{def} for SC1	= 0,6
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/120)^{0,15}$	= 1,15

9. CALCULATION EXAMPLES OF LVL STRUCTURES

Loading combinations

The most critical ultimate limit state (ULS) load combination:

$$E_{d,ULS} = \gamma_G \cdot g_k + \gamma_Q \cdot q_k = 1,15 \cdot 5,0 \text{ kN} + 1,5 \cdot 11 \text{ kN} = 22,3 \text{ kN}$$

Note: Safety factors γ_G and γ_Q are according to Finnish National annex of eurocode 0.

ULS design

Axial compression

$$\frac{N_{c,d}}{A} = \frac{E_{d,ULS}}{A} = \frac{22,3 \text{ kN}}{5400 \text{ mm}^2} = 4,1 \text{ N/mm}^2$$

$$f_{c,0,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{c,0,SC1,k} = \frac{0,8}{1,2} \cdot 26 \frac{\text{N}}{\text{mm}^2} = 17,3 \text{ N/mm}^2$$

Buckling, buckling length $l_c = 2700 \text{ mm}$ in z-direction

$$\lambda_y = \sqrt{12} \left(\frac{l_c}{h} \right) = 3,46 \cdot \frac{2700 \text{ mm}}{120 \text{ mm}} = 78 \quad (4.37)$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{78}{3,14} \cdot \sqrt{\frac{26 \text{ N/mm}^2}{8000 \text{ N/mm}^2}} = 1,41 \quad (4.35)$$

$$k_y = 0,5(1 + \beta_c(\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2) = 0,5 \cdot (1 + 0,1 \cdot (1,412 - 0,3) + (1,41)^2) = 1,56 \quad (4.33)$$

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = \frac{1}{1,56 + \sqrt{1,56^2 - 1,41^2}} = 0,45 \quad (4.31)$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} = \frac{4,1 \frac{\text{N}}{\text{mm}^2}}{0,45 \cdot 17,3 \frac{\text{N}}{\text{mm}^2}} = 0,52 \leq 1,0 \rightarrow \text{OK}$$

Bending moment resistance

$$M_d = E_{d,ULS} \cdot e_z = 22,3 \text{ kN} \cdot 0,03 \text{ m} = 0,67 \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{0,67 \text{ kNm}}{1,08 \cdot 10^5 \text{ mm}^3} = 6,2 \text{ N/mm}^2$$

$$f_{m,0,edge,d} = \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot f_{m,0,edge,k} = \frac{0,8}{1,2} \cdot 1,15 \cdot 27 \frac{\text{N}}{\text{mm}^2} = 20,7 \text{ N/mm}^2$$

$$\sigma_{m,d} \leq f_{m,0,edge,d} \rightarrow \text{OK}$$

The following expressions shall be satisfied with $k_m = 0,7$ for rectangular cross-sections:

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \quad (4.30)$$

$$\frac{4,1 \frac{\text{N}}{\text{mm}^2}}{0,45 \cdot 17,3 \frac{\text{N}}{\text{mm}^2}} + 0,7 \cdot \frac{6,2 \frac{\text{N}}{\text{mm}^2}}{20,7 \frac{\text{N}}{\text{mm}^2}} = 0,52 + 0,7 \cdot 0,30 = 0,73 \rightarrow \text{OK}$$

The wall studs is installed on a 120 mm wide LVL 48P sole plate, which has compression perpendicular to the grain flatwise strength $f_{c,90,flat,k} = 2,2 \text{ N/mm}^2$. The resistance is

$$F_{c,90,d} = N_d = 22,3 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{22,3 \text{ kN}}{b \cdot (l_{\text{support}} + 2 \cdot 30 \text{ mm})} = \frac{22,3 \text{ kN}}{120 \text{ mm} \cdot (45 \text{ mm} + 2 \cdot 30 \text{ mm})}$$

$$\sigma_{c,90,d} = 1,77 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,flat,d} = k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,flat,k}$$

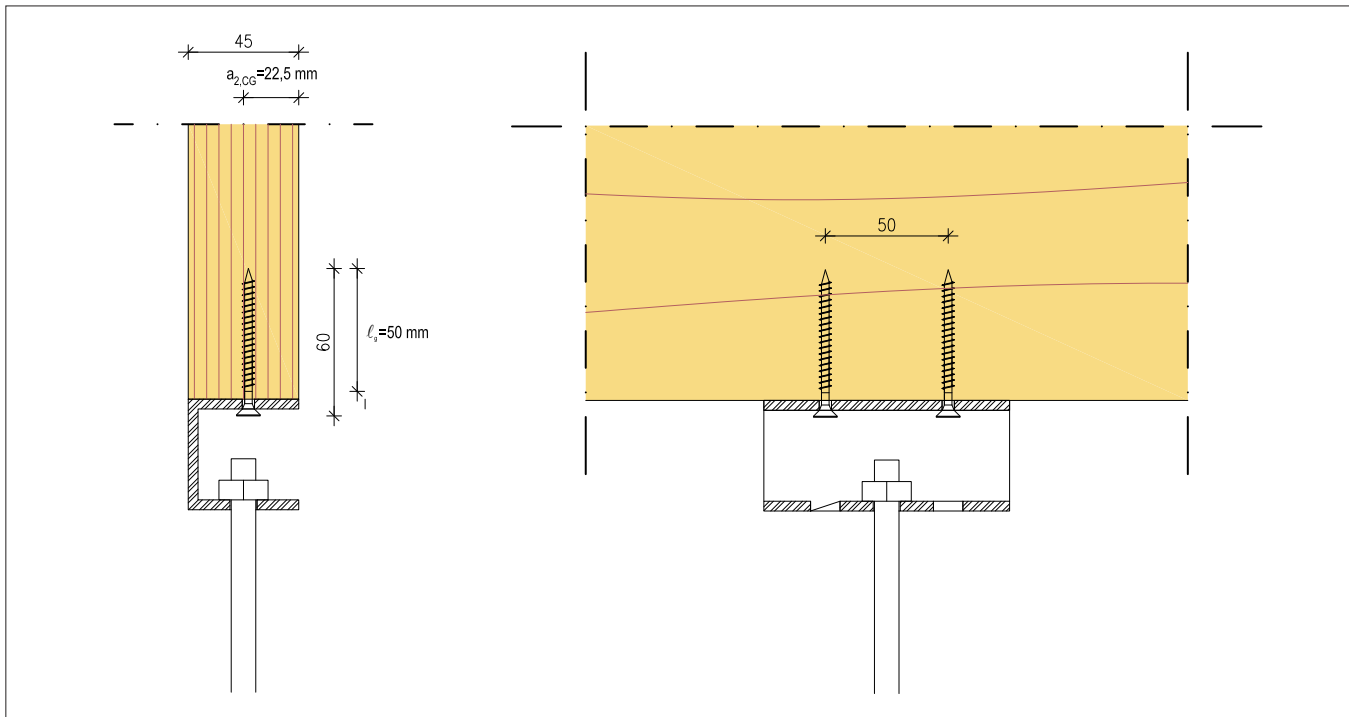
$$k_{c,90} \cdot f_{c,90,flat,d} = 1,4 \cdot \frac{0,8}{1,2} \cdot 2,2 \frac{\text{N}}{\text{mm}^2} = 2,05 \text{ N/mm}^2 \quad (k_{c,90} \text{ from Table 4.7})$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,flat,d} \rightarrow \text{OK}$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

9.6 AXIALLY LOADED SCREW CONNECTION AT THE EDGE FACE

An air duct is installed to the ceiling surface of roof element. Own-weight is 50 kg/m. Suspension connections are c/c 2.5 m crosswise to roof element and the connections are made to the edge face of a 45mm thick LVL 48 P beams. Service class SC1.



Loading combination

The most critical ultimate limit state (ULS) load combination for the connection:

$$E_{d,ULS} = s \cdot (\gamma_G \cdot g_k) = 2,5 \text{ m} \cdot (1,35 \cdot 0,5 \text{ kN/m}) = 1,7 \text{ kN}$$

Note: Safety factor γ_G for the case of only permanent loads according to Finnish National annex of Eurocode 0.

Screw properties

Size 5,0 x 60 mm screws, 2 screws / connection

Threaded length l_g = 50 mm

Head diameter d_h = 10 mm (Head pull-through capacity is not governing in steel-to-timber connection)

Tensile strength $t_{tens,k}$ = 7 kN determined in accordance with EN 14592. $t_{tens,k} > E_{d,ULS} \rightarrow \text{OK}$

Withdrawal strength $f_{ax,k}$ = 10 N/mm² at the edge face of LVL 48 P, determined in accordance with EN 14592

Modification factor k_{mod} for permanent load, SC1 = 0,6

Material safety factor γ_M for connections (default value in EC5) = 1,3

Geometry conditions:

Minimum distance to edge $a_{CG,2}$ in LVL edge $\geq 4d = 4 \cdot 5,0 \text{ mm} = 20 \text{ mm}$. Beam thickness $45 \text{ mm}/2 = 22,5 \text{ mm} \rightarrow$ Screw size 5,0x50 mm is OK for the beam.

Min. screw spacing $a_1 \geq 10d = 50 \text{ mm}$

Min. distance from the beam end $a_{1,CG} \geq 12d = 60 \text{ mm}$

Min. pointside penetration length of the threaded part $l_g \geq 6d = 30 \text{ mm}$, OK

Tension strength of the connection

$$R_k = n^{0,9} R_{T,k}$$

$$R_{T,k} = f_{ax,k} \cdot d \cdot l_{ef}$$

$$l_{ef} = l_g - d = 50 \text{ mm} - 5 \text{ mm} = 45 \text{ mm}$$

$$R_{T,k} = 10 \frac{\text{N}}{\text{mm}^2} \cdot 5,0 \text{ mm} \cdot 45 \text{ mm} = 2,25 \text{ kN}$$

$$R_d = \frac{k_{mod}}{\gamma_M} \cdot n^{0,9} \cdot R_{T,k} = \frac{0,6}{1,3} \cdot 2^{0,9} \cdot 2,25 \text{ kN} = 1,9 \text{ kN}$$

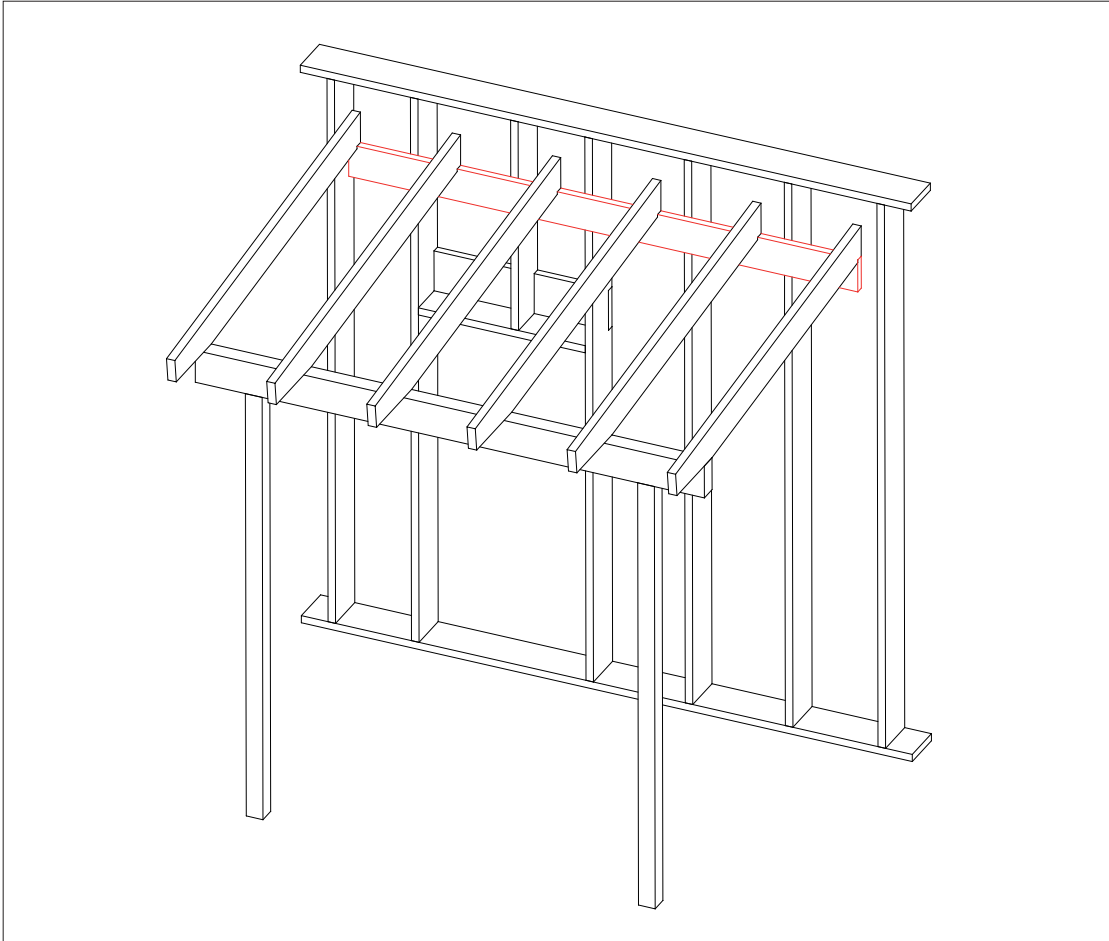
$$E_{d,ULS} \leq R_d \rightarrow \text{OK}$$

The axial loaded screw connection can be made with two 5,0 x 50 mm size screws/connection positioned at the middle of the edge face of the beam with 50 mm screw spacing. The screws shall not be closer than 60 mm to the beam end.

9. CALCULATION EXAMPLES OF LVL STRUCTURES

9.7 INCLINED SCREW CONNECTION

A canopy over the entrance of a one family house is planned to be supported to the external wall by a 51x200 mm LVL 48 P ledger beam. The beam is connected with inclined screws in 45° angle to the edges of 51mm thick LVL 32 P wall studs which have spacing $s = 600$ mm. Line load from own weight g_k is 0,3 kN/m and imposed load from snow q_k is 3,5 kN/m. Service class SC2.



Loading combinations

The most critical ultimate limit state (ULS) load combination for each connection between the beam and stud:

$$E_{d,ULS} = s \cdot (\gamma_G \cdot g_k + \gamma_Q \cdot q_k)$$

$$E_{d,ULS} = 0,6m \cdot (1,15 \cdot 0,3 \text{ kN/m} + 1,5 \cdot 3,5 \text{ kN/m}) = 3,4 \text{ kN}$$

Note: Safety factors γ_G and γ_Q are according to Finnish National annex of Eurocode 0.

Screw properties

Size 6,0x140mm full threaded screw

Threaded length l_g = 123 mm

Unthreaded length l_u = 17 mm

Head diameter d_h = 12 mm

Tensile strength $t_{\text{tens},k}$ = 10 kN, determined in accordance with EN 14592.

Head pull-through strength $f_{\text{head},k} = 13 \text{ N/mm}^2$, when $\rho_a = 350 \text{ kg/m}^3$

Modification factor k_{mod} for medium-term load, SC2 = 0,8

Material safety factor γ_m for connections (default value in EC5) = 1,3

Geometry conditions:

Minimum distance to edge $a_{2,CG}$ in stud $\geq 4d = 4 \cdot 6,0 \text{ mm} = 24 \text{ mm}$. Stud thickness $51 \text{ mm} / 2 = 25,5 \text{ mm}$

→ Screw size 6,0x140 mm is OK for the stud

Min. screw spacing a_1 in stud $\geq 10d = 60 \text{ mm}$

Min. screw spacing a_2 in beam $\geq 5d = 30 \text{ mm}$

→ a_1 in the stud is more critical

Distance to edge of the beam $a_{2,CG} \geq 4d = 24 \text{ mm}$. When the screwing angle is 45° , the beam thickness $t_1/2 = 25,5 \text{ mm}$ gives the minimum distance.

Maximum number of screws in the connection:

$$1 + \frac{(h_{\text{beam}} - 2 \cdot \min a_{2,CG})}{(\min a_{1,\text{stud}} / \sin 45^\circ)} = 1 + \frac{(200 \text{ mm} - 2 \cdot 25,5 \text{ mm})}{(60 \text{ mm} / \sin 45^\circ)} = 1 + \frac{149 \text{ mm}}{85 \text{ mm}} = 2,79$$

→ 2 screws are chosen for the connection, so that the heads of the screws are 20 mm and 110 mm from the bottom edge of the beam.

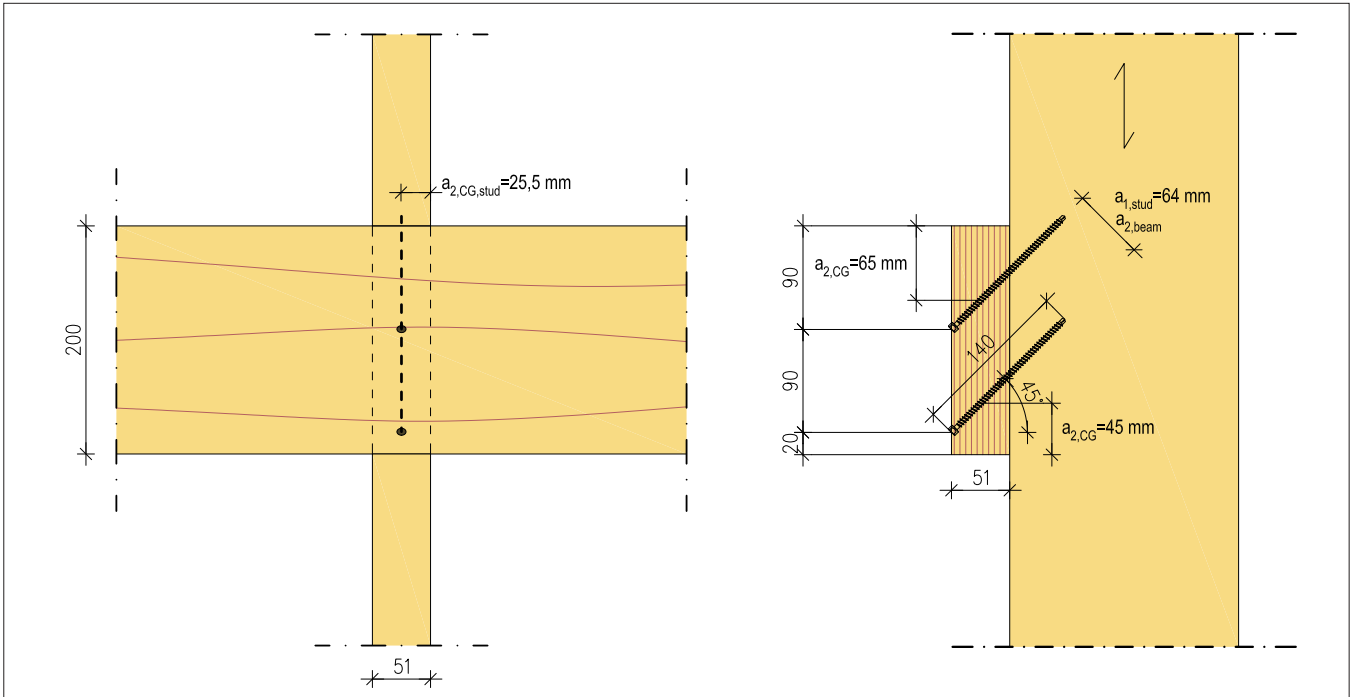
Minimum distance to the beam end $a_{1,CG} \geq 10d = 60 \text{ mm}$. Therefore the end of the beam shall exceed the stud edge.

Effective penetration length $l_{\text{ef},1}$ in ledger beam is

$$l_{\text{ef},1} = l_{g,1} = \frac{t_1}{\sin 45^\circ} - l_u = \frac{51 \text{ mm}}{\sin 45^\circ} - 17 \text{ mm} = 55 \text{ mm}$$

$$\text{Penetration length in wall stud } l_{g,2} = l - \frac{t_1}{\sin 45^\circ} = 140 \text{ mm} - \frac{51 \text{ mm}}{\sin 45^\circ} = 68 \text{ mm}$$

For the beam the angles in the connections are: $\alpha = 45^\circ$, $\beta = 45^\circ$ and $\varepsilon = 90^\circ$ and for the stud they are: $\alpha = 45^\circ$, $\beta = 0^\circ$ and $\varepsilon = 45^\circ$.


Connection capacity

The characteristic load-carrying capacity of the tension screw connection, see Figure 5.11(b), is calculated by the equation:

$$R_k = n^{0,9} R_{T,k} (\cos \alpha + \mu \sin \alpha) \quad (5.33)$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

Since the screwing direction ε in the beam is 90° to the grain direction, it is not allowed to add the tension capacity of the head to the withdrawal capacity of the treaded part in the beam. Therefore the characteristic withdrawal capacity $R_{T,k}$ of the screw is calculated by the equation:

$$R_{T,k} = \min \begin{cases} \max \left(f_{ax,90,1,k} d l_{g,1}; f_{head,k} d_h^2 \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \right) \\ f_{ax,\varepsilon,2,k} d l_{g,2} \\ f_{tens,k} \end{cases} \quad (5.31)$$

The withdrawal strength $f_{ax,\varepsilon,k}$ is determined by testing according to EN 14592 and according to EN 1382 or it can be determined at angle ε to the grain as follows:

$$f_{ax,\varepsilon,k} = \frac{k_{ax} \cdot f_{ax,90,k}}{1,5 \cos^2 \beta + \sin^2 \beta} \left(\frac{\rho_k}{\rho_a} \right)^{0,8} \quad (5.32)$$

The characteristic density ρ_k is 480 kg/m^3 for LVL 48 P and 410 kg/m^3 for LVL 32 P.

$f_{ax,90,k}$ is the characteristic withdrawal strength parameter for a screw perpendicular to the grain direction [N/mm²]. For screws in LVL, the characteristic withdrawal parameter may be assumed as $f_{ax,90,k} = 15 \text{ N/mm}^2$ for $\rho_a = 500 \text{ kg/m}^3$ and screws $6 \text{ mm} \leq d \leq 12 \text{ mm}$ in softwood LVL/GLVL.

$$f_{ax,90,1,k} = 15 \text{ N/mm}^2 \cdot \left(\frac{480 \text{ kg/m}^3}{500 \text{ kg/m}^3} \right)^{0,8} = 14,5 \text{ N/mm}^2$$

When $\varepsilon = 45^\circ$, $k_{ax} = 1$ and when $\beta = 0^\circ$,

$$f_{ax,45,2,k} = \frac{1 \cdot 15 \text{ N/mm}^2}{1,5 \cdot \cos^2 0^\circ + \sin^2 0^\circ} \left(\frac{410 \text{ kg/m}^3}{500 \text{ kg/m}^3} \right)^{0,8} = 8,5 \text{ N/mm}^2$$

The different conditions of the equation (5.31) give a characteristic capacity $R_{T,k}$:

$$1 : f_{ax,90,1,k} \cdot d \cdot l_{g,1} = 14,5 \frac{\text{N}}{\text{mm}^2} \cdot 6,0 \text{ mm} \cdot 55 \text{ mm} = 4,8 \text{ kN}$$

$$2 : f_{head,k} \cdot d_h^2 \cdot \left(\frac{\rho_k}{\rho_a} \right)^{0,8} = 13,0 \text{ N/mm}^2 \cdot (12 \text{ mm})^2 \cdot \left(\frac{480 \text{ kg/m}^3}{350 \text{ kg/m}^3} \right)^{0,8} = 2,4 \text{ kN}$$

$$3 : f_{ax,45,2,k} \cdot d \cdot l_{g,2} = 8,5 \frac{\text{N}}{\text{mm}^2} \cdot 6,0 \text{ mm} \cdot 68 \text{ mm} = 3,5 \text{ kN}$$

$$4 : f_{tens,k} = 10 \text{ kN}$$

$$R_{T,k} = \min \begin{cases} \max(4,8 \text{ kN}; 2,4 \text{ kN}) \\ 3,5 \text{ kN} \\ 10 \text{ kN} \end{cases} = 3,5 \text{ kN}$$

Design resistance of the connection:

$$R_d = \frac{k_{mod}}{\gamma_M} \cdot n^{0,9} \cdot R_{T,k} (\cos \alpha + \mu \sin \alpha)$$

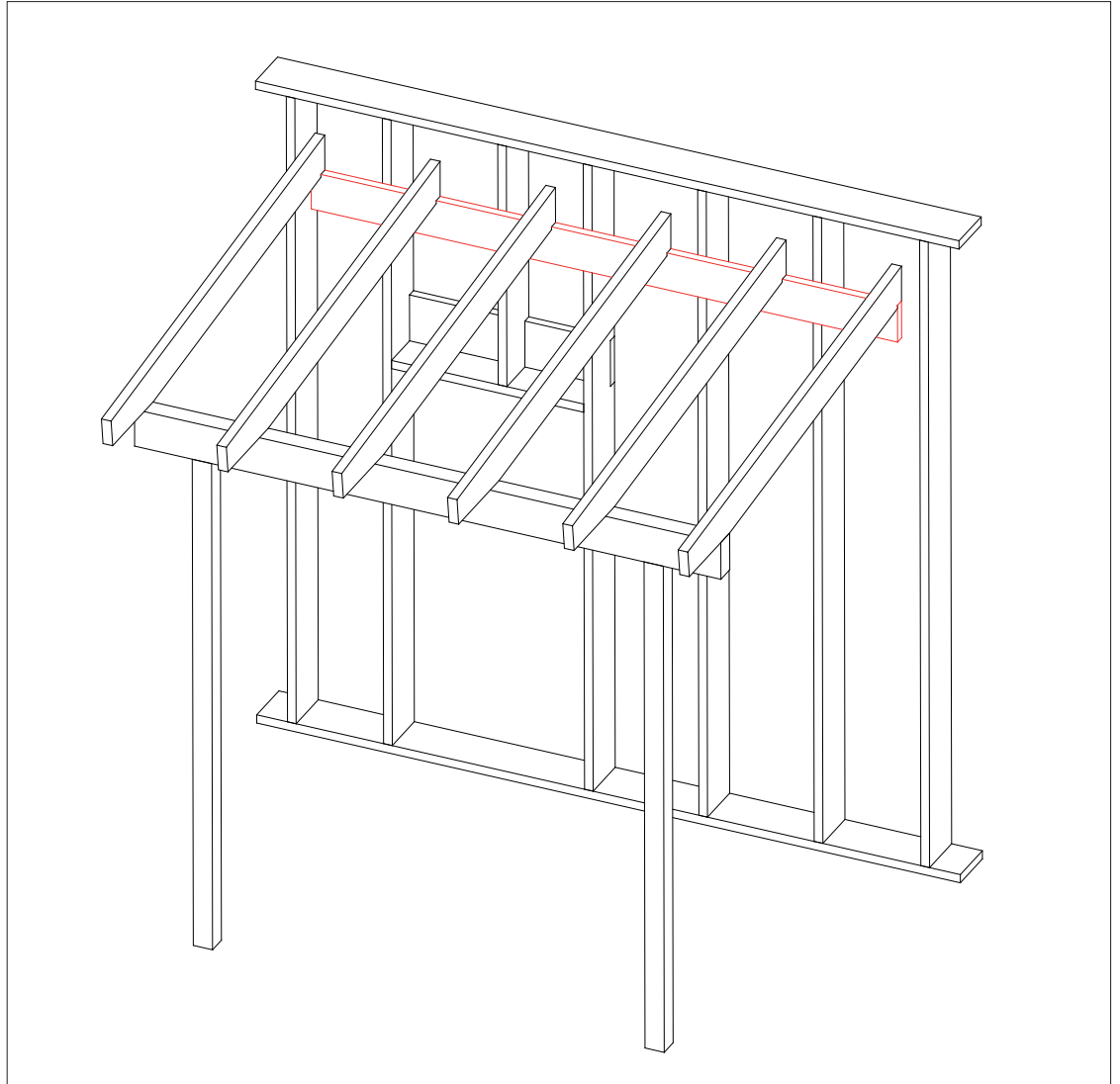
$$R_d = \frac{0,8}{1,3} \cdot 2^{0,9} \cdot 3,5 \text{ kN} \cdot (\cos 45^\circ + 0,26 \cdot \sin 45^\circ) = 3,6 \text{ kN}$$

$$E_{d,ULS} \leq R_d \rightarrow \text{OK}$$

The canopy can be supported on a 51x200 mm LVL 48P ledger beam which is connected to 51 mm LVL 32P wall studs with 2pcs 6,0x140 mm full threaded inclined screws. At the ends the ledger beam shall exceed the studs edges at least $60 \text{ mm} - 25,5 \text{ mm} = 34,5 \text{ mm}$.

9.8 LATERALLY LOADED NAIL CONNECTION

A canopy over the entrance of a one family house is supported to the external wall by a 51x300 mm LVL 48 P ledger beam. The beam is nailed to the edges of 45 mm thick LVL 32 P wall studs which have spacing $s = 600$ mm. Line load from own weight g_k is 0,3 kN/m and imposed load from snow q_k is 3 kN/m. Service class SC2.



Loading combinations

The most critical ultimate limit state (ULS) load combination for each connection between the beam and stud:

$$E_{d,ULS} = s \cdot (\gamma_G \cdot g_k + \gamma_Q \cdot q_k)$$

$$E_{d,ULS} = 0,6\text{m} \cdot (1,15 \cdot 0,3 \text{ kN/m} + 1,5 \cdot 3 \text{ kN/m}) = 2,91 \text{ kN}$$

Note: Safety factors γ_G and γ_Q are according to Finnish national annex of eurocode 0.

Nail properties

Size: 3,1x90 mm round nails

Tensile strength $f_u = 600 \text{ N/mm}^2$

Modification factor k_{mod} for medium-term, SC2 = 0,8

Material safety factor γ_M for connections (default value in EC5) = 1,3

9. CALCULATION EXAMPLES OF LVL STRUCTURES

Geometry conditions, Chapter 5, Table 5.1, Figures 5.4 and 5.5.

Ledger beam thickness $t_1 = 51 \text{ mm} = 16,4d > 7d$, so no predrilling is required.

Pointside penetration length $t_2 = 90 \text{ mm} - 51 \text{ mm} = 39 \text{ mm} = 12,6d$, OK

Note: Usually it is recommended to have $\geq 12d$ penetration length t_2 .

Minimum distance to unloaded edge $a_{4,c}$ in stud $\geq 7d = 7 \cdot 3,1 \text{ mm} = 21,7 \text{ mm}$. Stud thickness $45 \text{ mm}/2 = 22,5 \text{ mm} \rightarrow$ Nail size 3,1x90 mm is OK for the stud.

Min. nail spacing a_1 in stud $\geq [(7 + 8(\cos \alpha)) \cdot d] = 15d = 46,5 \text{ mm}$

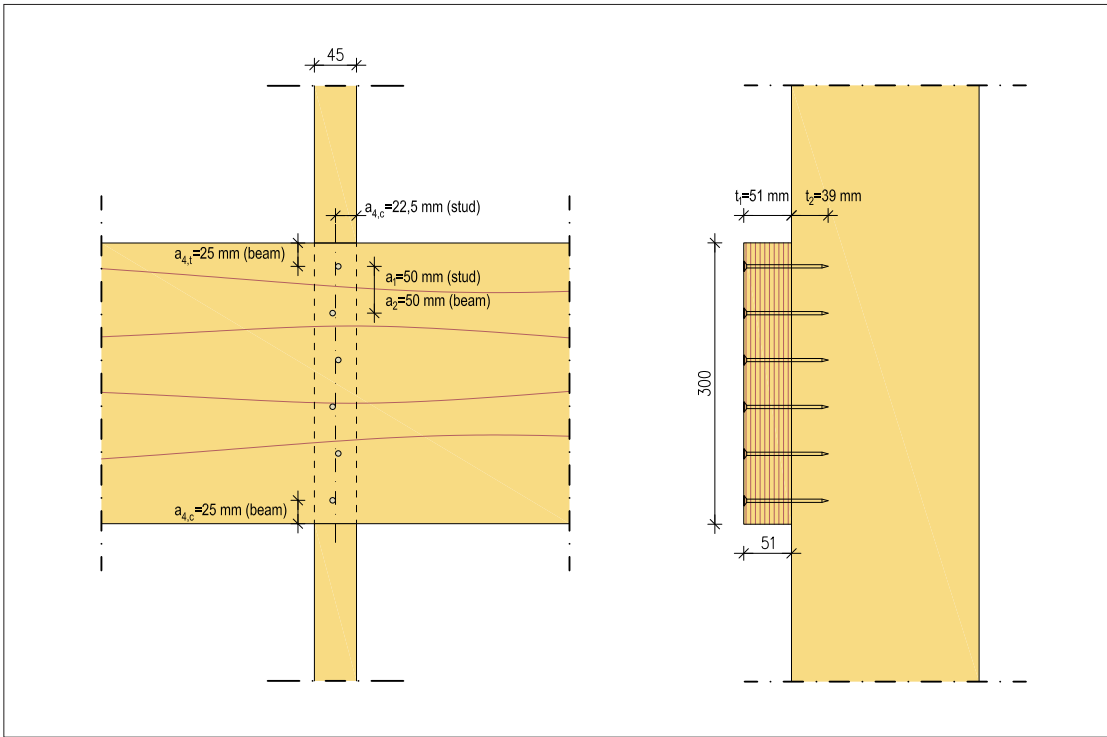
Min. distance to unloaded edge of the beam $a_{4,c} \geq 5d = 15,5 \text{ mm}$

Min. distance to loaded edge of the beam $a_{4,t} \geq (5 + 2 \cdot \sin \alpha) \cdot d = 7d = 21,7 \text{ mm}$

Maximum number of nails in the connection:

$$1 + \frac{(h - \min a_{4,c} - \min a_{4,t})}{\min a_1} = 1 + \frac{(300 \text{ mm} - 15,5 \text{ mm} - 21,7 \text{ mm})}{46,5 \text{ mm}} = 6,65$$

\rightarrow 6 nails are chosen for the connection, so that the edge distance $a_{4,c}$ and $a_{4,t}$ at the beam are 25 mm and the nail spacing a_1 in the stud (which is a_2 in the beam) is 50 mm.



Connection capacity

The embedment strength of 3,1x90 mm round nail in LVL 48P and LVL 32P

$$f_{h,LVL 48P,k} = 0,082 \cdot \rho_k \cdot d \cdot 0,3 = 0,082 \cdot 480 \cdot 2,5 \cdot 0,3 = 28,0 \text{ N/mm}^2$$

$$f_{h,LVL 32P,k} = 0,082 \cdot \rho_k \cdot d \cdot 0,3 = 0,082 \cdot 410 \cdot 2,5 \cdot 0,3 = 23,9 \text{ N/mm}^2$$

Note: the angle $\beta = 0^\circ$ for the beam and 90° for the stud, so the simpler equation of embedment strength can be used.

When the nails are produced from wire with tensile strength $f_u = 600 \text{ N/mm}^2$, the characteristic value of the yield moment $M_{y,k}$ for round nails is

$$M_{y,k} = 0,3 \cdot f_u \cdot d^2 \cdot 6 = 0,3 \cdot 600 \cdot (2,5)^2 \cdot 6 = 3410 \text{ Nmm}$$

Note: $M_{y,k}$ value should be checked from the D_{oP} of the nail supplier.

The influence of rope effect based on the axial withdrawal capacity $F_{ax,k}$ of round nails is negligible. With these properties $F_{V,nail,Rk}$ is as the minimum of EN 1995-1-1, equation 8.6 failure modes (a)-(f)

$$F_{V,nail,Rk} = \min \begin{cases} 4,43 \text{ (a)} \\ 2,89 \text{ (b)} \\ 1,55 \text{ (c)} \\ 1,58 \text{ (d)} \\ 1,13 \text{ (e)} \\ 0,85 \text{ (f)} \end{cases} = 0,85 \text{ kN}$$

Design resistance of the connection:

$$R_d = \frac{k_{mod}}{\gamma_M} \cdot n_{ef} \cdot F_{V,nail,Rk}$$

$$n_{ef} = n^{k_{ef}}$$

$k_{ef} = 1$, when a nail row staggered perpendicular to grain by at least $1d$. Without staggering in LVL edge face

$$k_{ef} = \min \left\{ 1 - 0,03 \left(20 - \frac{a_1}{d} \right) \right\} = 1 - 0,03 \left(20 - \frac{50\text{mm}}{3,1\text{mm}} \right) = 0,88$$

Nailing without staggering:

$$R_d = \frac{k_{mod}}{\gamma_M} \cdot n_{ef} \cdot F_{V,nail,Rk} = \frac{0,8}{1,3} \cdot 6^{0,88} \cdot 0,85 \text{ kN} = 0,62 \cdot 4,84 \cdot 0,85 \text{ kN} = 2,55 \text{ kN}$$

$$E_{d,ULS} > R_d \rightarrow \text{Not OK}$$

Nailing with staggering:

$$R_d = \frac{k_{mod}}{\gamma_M} \cdot n_{ef} \cdot F_{V,nail,Rk} = \frac{0,8}{1,3} \cdot 6 \cdot 0,85 \text{ kN} = 3,1 \text{ kN}$$

$$E_{d,ULS} \leq R_d \rightarrow \text{OK, staggering is required}$$

The canopy can be supported on a 51x300 mm LVL 48P ledger beam which is connected to 45 mm LVL 32P wall studs with 6pcs 3,1x90 mm round nails when the nail row is staggered perpendicular to grain by $1d$.

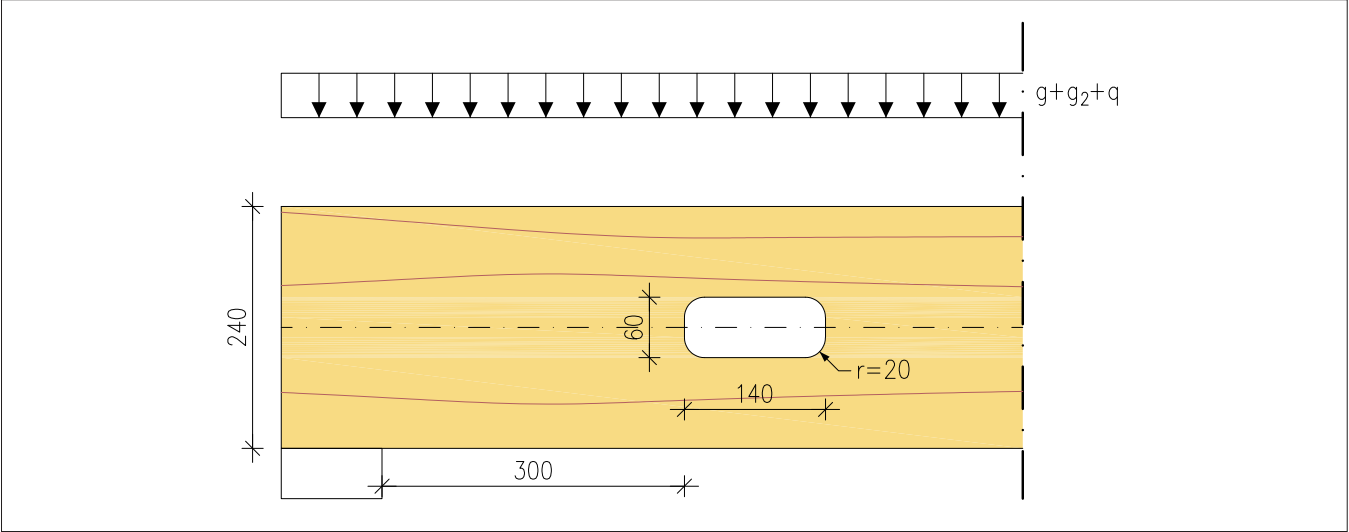
Discussion:

The inclined screws connection has 16% higher capacity than the laterally loaded nailed connection in the example 9.6 and the ledger beam depth is 100 mm smaller. However, the LVL 32P stud needs to be thicker due to the edge distance requirement $a_{2,CG} \geq 4d$ of the screws. A laterally loaded screws connection would not be possible for the combination of screw size and LVL beam stud thickness, since the edge distance $a_{4,c} \geq 7d$ at the LVL edge would not be fulfilled.

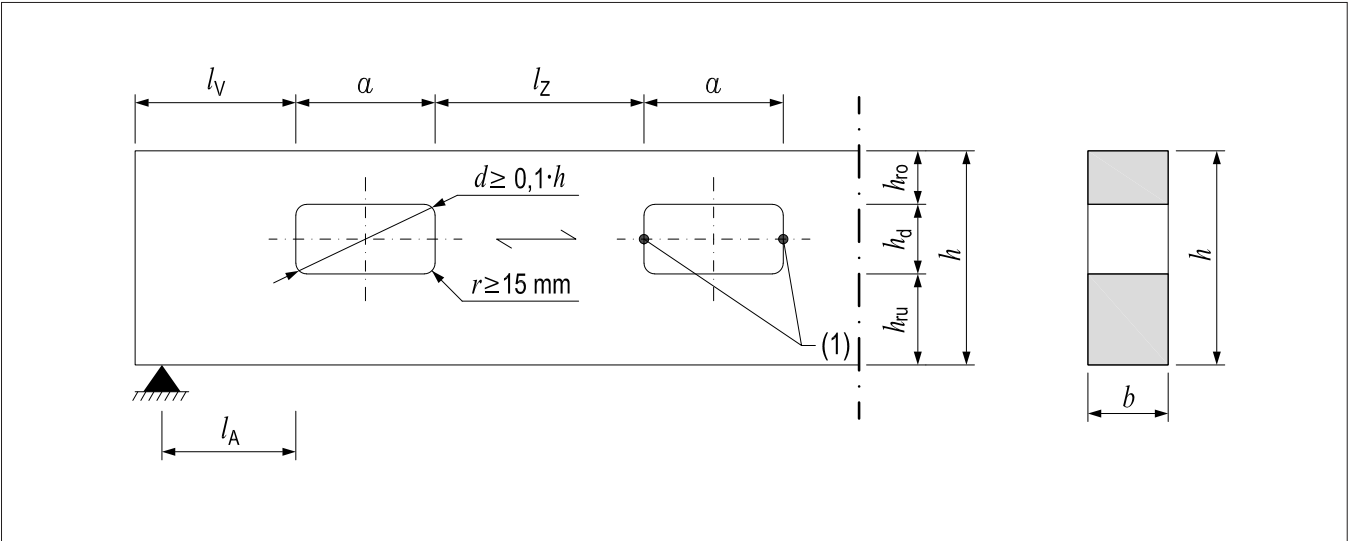
9. CALCULATION EXAMPLES OF LVL STRUCTURES

9.9 HOLE IN LVL BEAM

A residential floor joist of the example 9.1 needs a 60x140 mm rectangular hole for service installations located at the centreline of the cross-section of the joists 300 mm from the edge of the support. The hole corners have $r = 20$ mm rounding's.



Geometry requirements:



Product type	l_v	l_A	h_{ro} and h_{ru}	a	h_d
LVL-P	$300 \text{ mm} + 45 \text{ mm} \geq h$, OK	$300 \text{ mm} + 45 \text{ mm} / 2 \geq 0,5h$, OK	$(240 \text{ mm} - 60 \text{ mm}) / 2 \geq 0,35h$, OK	$140 \text{ mm} \leq 2,5h_d$, OK	$60 \text{ mm} > 0,15h$, Not OK
LVL-C	$300 \text{ mm} + 45 \text{ mm} \geq h$, OK	$300 \text{ mm} + 45 \text{ mm} / 2 \geq 0,5h$, OK	$(240 \text{ mm} - 60 \text{ mm}) / 2 \geq 0,25h$, OK	$140 \text{ mm} \leq 2,5h_d$, OK	$60 \text{ mm} \leq 0,4h$, OK

Corner roundings $r = 20 \text{ mm} > 15 \text{ mm}$, OK

→LVL-C type joist is needed to fulfil the requirement of the maximum hole depth h_d .

Properties of a LVL 36C joist:

Bending strength edgewise $f_{m,0,edge,k}$	= 32 N/mm ²
Tension in perpendicular to grain strength edgewise $f_{t,90,edge,k}$	= 5 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,5 N/mm ²
Modification factor k_{mod} for medium-term, SC1	= 0,8
Material safety factor γ_M (default value in EC5)	= 1,2
Size effect factor $k_h = (300/240)^{0,15}$	= 1,03

Tension stress perpendicular to the grain is verified by the equation

$$\sigma_{t,90,d} = \frac{F_{t,90,d}}{0,5 \cdot l_{t,90} \cdot b \cdot k_{t,90}} \leq f_{t,90,d} \quad (4.57)$$

where

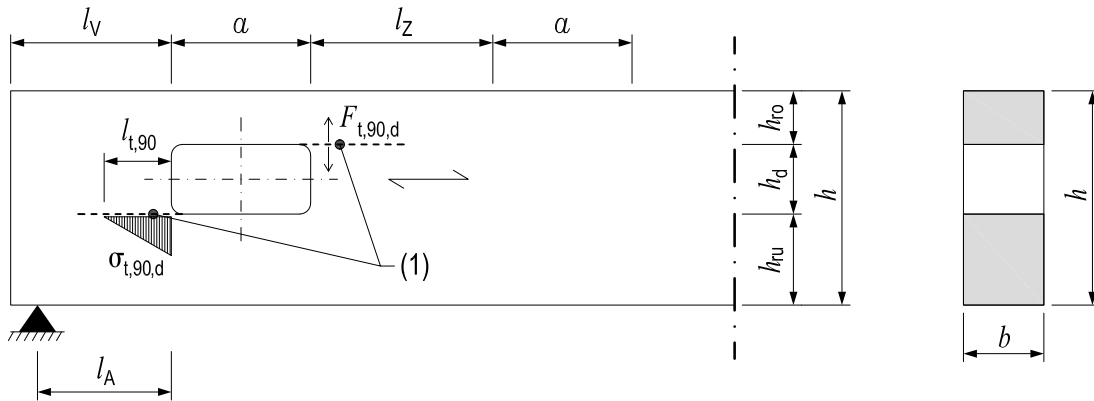
$$k_{t,90} = \min \left\{ \left(\frac{450}{h} \right)^{0,5} = 1,0 \right\} \quad (4.58)$$

$\sigma_{t,90,d}$ = design value of tension stress perpendicular to the grain [N/mm²]

$$l_{t,90} = 0,5 \cdot (h_d + h) = 0,5 \cdot (60 \text{ mm} + 240 \text{ mm}) = 150 \text{ mm} \quad (4.61)$$

$F_{t,90,d}$ = design value of tension force perpendicular to the grain [N]

$$f_{t,90,d} = \frac{k_{mod}}{\gamma_M} \cdot f_{t,90,edge,k} = \frac{0,8}{1,2} \cdot 5 \frac{\text{N}}{\text{mm}^2} = 3,3 \text{ N/mm}^2$$



Tension in perpendicular to grain force $F_{t,90,d}$ depends on the shear force V_d and bending moment M_d at the edge of the hole:

$$F_{t,90,d} = \frac{V_d \cdot h_d}{4 \cdot h} \cdot \left[3 - \left(\frac{h_d}{h} \right)^2 \right] + 0,008 \cdot \frac{M_d}{h_r} \quad (4.59)$$

where

$$h_r = 90 \text{ mm} \quad (4.60)$$

9. CALCULATION EXAMPLES OF LVL STRUCTURES

$$V_{d,(x=323\text{mm})} = 0,4 \text{ m} \cdot 4,03 \frac{\text{kN}}{\text{m}^2} \cdot \left(1 - \frac{0,3 \text{ m} + 0,045 \text{ m}/2}{4,5 \text{ m}}\right) = 3,13 \text{ kN}$$

$$M_{d,(x=323\text{mm})} = \frac{0,4 \text{ m} \cdot 4,03 \frac{\text{kN}}{\text{m}^2} \cdot (0,3 \text{ m} + 0,045 \text{ m}/2)}{2} \cdot \left(4,5 \text{ m} - \left(0,3 \text{ m} + \frac{0,045 \text{ m}}{2}\right)\right)$$

$$M_{d,(x=323\text{mm})} = 1,1 \text{ kNm}$$

$$F_{t,90,d} = \frac{3,13 \text{ kN} \cdot 0,06 \text{ m}}{4 \cdot 0,24 \text{ m}} \cdot \left[3 - \left(\frac{0,06 \text{ m}}{0,24 \text{ m}}\right)^2\right] + 0,008 \cdot \frac{1,1 \text{ kNm}}{0,09 \text{ m}} = 0,57 \text{ kN} + 0,10 \text{ kN} = 0,67 \text{ kN}$$

$$\sigma_{t,90,d} = \frac{F_{t,90,d}}{0,5 \cdot l_{t,90} \cdot b \cdot k_{t,90}} = \frac{0,67 \text{ kN}}{0,5 \cdot 0,15 \text{ m} \cdot 0,045 \text{ m} \cdot 1,0} = 0,20 \frac{\text{N}}{\text{mm}^2} \leq f_{t,90,d} \rightarrow \text{OK}$$

The verification of shear stress concentration at the hole edge shall fulfil the condition:

$$\tau_d = k_\tau \cdot \frac{1,5 \cdot V_d}{b \cdot (h - h_d)} \leq f_{v,d} \quad (4.62)$$

Where

$$k_\tau = 1,85 \cdot \left(1 + \frac{a}{h}\right) \cdot \left(\frac{h_d}{h}\right)^{0,2} = 1,85 \cdot \left(1 + \frac{0,14 \text{ m}}{0,24 \text{ m}}\right) \cdot \left(\frac{0,06 \text{ m}}{0,24 \text{ m}}\right)^{0,2} = 2,22 \quad (4.63)$$

$$\tau_d = k_\tau \cdot \frac{1,5 \cdot V_d}{b \cdot (h - h_d)} = 2,22 \cdot \frac{1,5 \cdot 3,13 \text{ kN}}{45 \text{ mm} \cdot (240 \text{ mm} - 60 \text{ mm})} = 1,3 \frac{\text{N}}{\text{mm}^2}$$

$$f_{v,0,\text{edge},d} = \frac{k_{\text{mod}}}{\gamma_M} \cdot f_{v,0,\text{edge},k} = \frac{0,8}{1,2} \cdot 4,5 \frac{\text{N}}{\text{mm}^2} = 3,0 \text{ N/mm}^2$$

$$\tau_d \leq f_{v,0,\text{edge},d} \rightarrow \text{OK}$$

Bending stress at the location of a rectangular hole is verified by the equations:

$$\frac{\frac{M_d + M_{o,d}}{W_n + W_o}}{f_{m,d}} \leq 1 \quad \text{and} \quad \frac{\frac{M_d + M_{u,d}}{W_n + W_u}}{f_{m,d}} \leq 1 \quad (4.64 \text{ and } 4.65)$$

Where

$$W_n = \frac{b \cdot (h^2 - h_d^2)}{6} = \frac{45 \text{ mm} \cdot ((240 \text{ mm})^2 - (60 \text{ mm})^2)}{6} = 4,05 \cdot 10^5 \text{ mm}^3 \quad (4.66)$$

$$M_{o,d} = \frac{A_o}{A_u + A_o} \cdot V_d \cdot \frac{a}{2} = \frac{b \cdot h_{ro}}{b \cdot h_{ro} + b \cdot h_{ru}} \cdot V_d \cdot \frac{a}{2} \quad (4.67)$$

$$M_{o,d} = \frac{45 \text{ mm} \cdot 90 \text{ mm}}{45 \text{ mm} \cdot 90 \text{ mm} + 45 \text{ mm} \cdot 90 \text{ mm}} \cdot 3,13 \text{ kN} \cdot \frac{140 \text{ mm}}{2} = 0,11 \text{ kNm}$$

$$W_o = \frac{b \cdot h_{ro}^2}{6} = \frac{45 \text{ mm} \cdot (90 \text{ mm})^2}{6} = 60750 \text{ mm}^3 \quad (4.69)$$

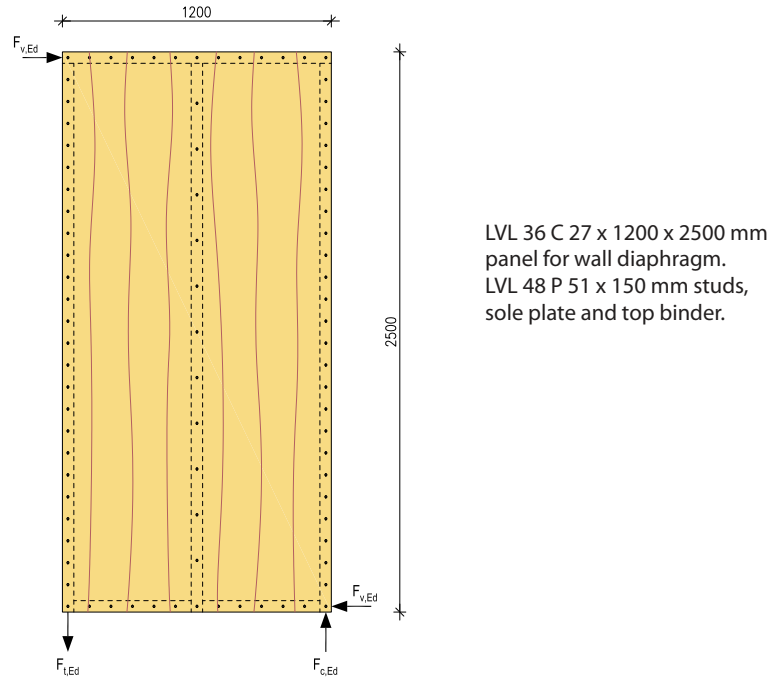
$$f_{m,0,d} = \frac{k_{\text{mod}}}{\gamma_M} \cdot k_h \cdot f_{m,0,k} = \frac{0,8}{1,2} \cdot 1,03 \cdot 32 \frac{\text{N}}{\text{mm}^2} = 22,1 \text{ N/mm}^2$$

$$\frac{\frac{M_d + M_{o,d}}{W_n + W_o}}{f_{m,d}} = \frac{\frac{1,1 \text{ kNm}}{4,05 \cdot 10^5 \text{ mm}^3} + \frac{0,11 \text{ kNm}}{60750 \text{ mm}^3}}{22,1 \text{ N/mm}^2} = \frac{4,5 \text{ N/mm}^2}{22,1 \text{ N/mm}^2} = 0,20 \leq 1 \rightarrow \text{OK}$$

Since the hole is at the centre line of the cross section, only one of the equations (4.64 and 4.65) needs to be verified. The 140x60 mm hole 300 mm from the support edge fulfils the requirements in LVL 36 C joist.

9.10 BRACING OF A STUD WALL

Define the maximum value of the horizontal force $F_{V,Ed}$ from instantaneous wind load when LVL 36 C panel 27 mm x 1200 mm x 2500 mm is used for wall diaphragm. The panel is nailed to the LVL 48P wall studs, sole plate and top binder with 2,5x60mm round nails. The studs are 51 x 150 mm in spacing $b_{net} = 600$ mm. The nail spacing is a constant 100 mm at the perimeters and 200 mm at the centre stud. Service class SC1.



The design load-carrying capacity $F_{V,Rd}$ (design racking resistance) under a horizontal force $F_{V,Ed}$ from wind load acting at the top of a cantilever panel secured against uplift is determined using the simplified method A of Eurocode 5 in section 9.2.4.2.

$$F_{V,Rd} = \frac{F_{f,Rd} \cdot b \cdot c}{s}$$

where

b = panels width 1200 mm

$$c = \begin{cases} 1, & \text{when } b \geq h/2 \\ \frac{b}{h/2}, & \text{when } b < h/2 \end{cases} \rightarrow c = \frac{2500 \text{ mm}}{1200 \text{ mm}} = 0,96$$

$F_{f,Rd}$ = lateral design capacity of an individual fastener $F_{V,nail,Rd}$ multiplied by 1,2 (EC5 9.2.4.2 (5)).

$F_{V,nail,Rd}$ is the minimum of failure modes (a) – (f) calculated from EC5 8.2.2, equation (8.6).

Geometry requirements of nail connections:

Pointside penetration length t_2 should be at least $8d = 8 \cdot 2,5 \text{ mm} = 20 \text{ mm}$. Nail length $l = 60 \text{ mm}$ – panels thickness 27 mm = 33 mm > 20 mm → OK.

The penetration length t_1 in LVL-C panel thickness should be at least $4d = 4 \cdot 2,5 \text{ mm} = 10 \text{ mm}$ → OK.

The unloaded edge distance $a_{4,C}$ should be at least $3d = 3 \cdot 2,5 \text{ mm} = 8 \text{ mm}$ in the face side of LVL-C panel and $7d = 7 \cdot 2,5 \text{ mm} = 18 \text{ mm}$ in the LVL-P edge face. The minimum stud thickness is $2 \cdot 7d = 36 \text{ mm}$ for a perimeter stud and $2 \cdot (7d + 3d) + 1 \text{ mm} = 51 \text{ mm}$ when the panel joint is at a stud location, see Table 5.5 of panel joints.

9. CALCULATION EXAMPLES OF LVL STRUCTURES

The embedment strength of 2,5x60 mm round nail in LVL 36 C and LVL 48P

$$f_{h,k} = 0,082 \cdot \rho_k \cdot d \cdot 0,3 = 0,082 \cdot 480 \cdot 2,5 \cdot 0,3 = 29,9 \text{ N/mm}^2$$

When the nails are produced from wire with tensile strength $f_u = 600 \text{ N/mm}^2$, the characteristic value of the yield moment $M_{y,k}$ for round nails is

$$M_{y,k} = 0,3 \cdot f_u \cdot d^{2,6} = 0,3 \cdot 600 \cdot (2,5)^{2,6} = 1949 \text{ Nmm}$$

The influence of rope effect based on the axial withdrawal capacity $F_{ax,k}$ of round nails is negligible. With these properties $F_{V,nail,Rk}$ is as the minimum of failure modes (a)-(f)

$$F_{V,nail,Rk} = \min \begin{cases} 2,02 \text{ (a)} \\ 2,47 \text{ (b)} \\ 0,94 \text{ (c)} \\ 0,78 \text{ (d)} \\ 0,92 \text{ (e)} \\ 0,62 \text{ (f)} \end{cases} = 0,62 \text{ kN}$$

$$F_{f,Rd} = \frac{k_{mod}}{\gamma_M} \cdot 1,2 \cdot F_{V,nail,Rk} = \frac{1,1}{1,3} \cdot 1,2 \cdot 0,62 \text{ kN} = 0,63 \text{ kN}$$

$k_{mod} = 1,1$ for instantaneous load (wind load) in service class 1

$\gamma_M = 1,3$ for connections (default value in EC5)

$$F_{V,Rd} = \frac{F_{f,Rd} \cdot b \cdot c}{s} = \frac{0,63 \text{ kN} \cdot 1200 \text{ mm} \cdot 0,96}{100 \text{ mm}} = 7,3 \text{ kN}$$

Shear buckling of the panel may be disregarded, when the stud spacing $b_{net} / t \leq 100$.

$$\frac{b_{net}}{t} = \frac{600 \text{ mm}}{27 \text{ mm}} = 22 \leq 100 \rightarrow \text{OK}$$

In order to with stand a horizontal force $F_{V,Ed} = 7,3 \text{ kN}$, the diaphragm panel shall be anchored at the bottom corners for the external forces

$$F_{t,Ed} = F_{c,Ed} = \frac{F_{V,Rd} \cdot h}{b} = \frac{7,3 \text{ kN} \cdot 2500 \text{ mm}}{1200 \text{ mm}} = 15,2 \text{ kN}$$

The contact area between perimeter stud and the end of the horizontal sole plate shall be verified for compression perpendicular to the grain.

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,d}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{c,90,d}}{l \cdot (b + 30 \text{ mm})} = \frac{15,2 \text{ kN}}{150 \text{ mm} \cdot (51 \text{ mm} + 30 \text{ mm})} = 1,3 \text{ N/mm}^2$$

$k_{c,90}$ is 1,4 for LVL-P flatwise and $f_{c,90,k}$ is 2,2 N/mm².

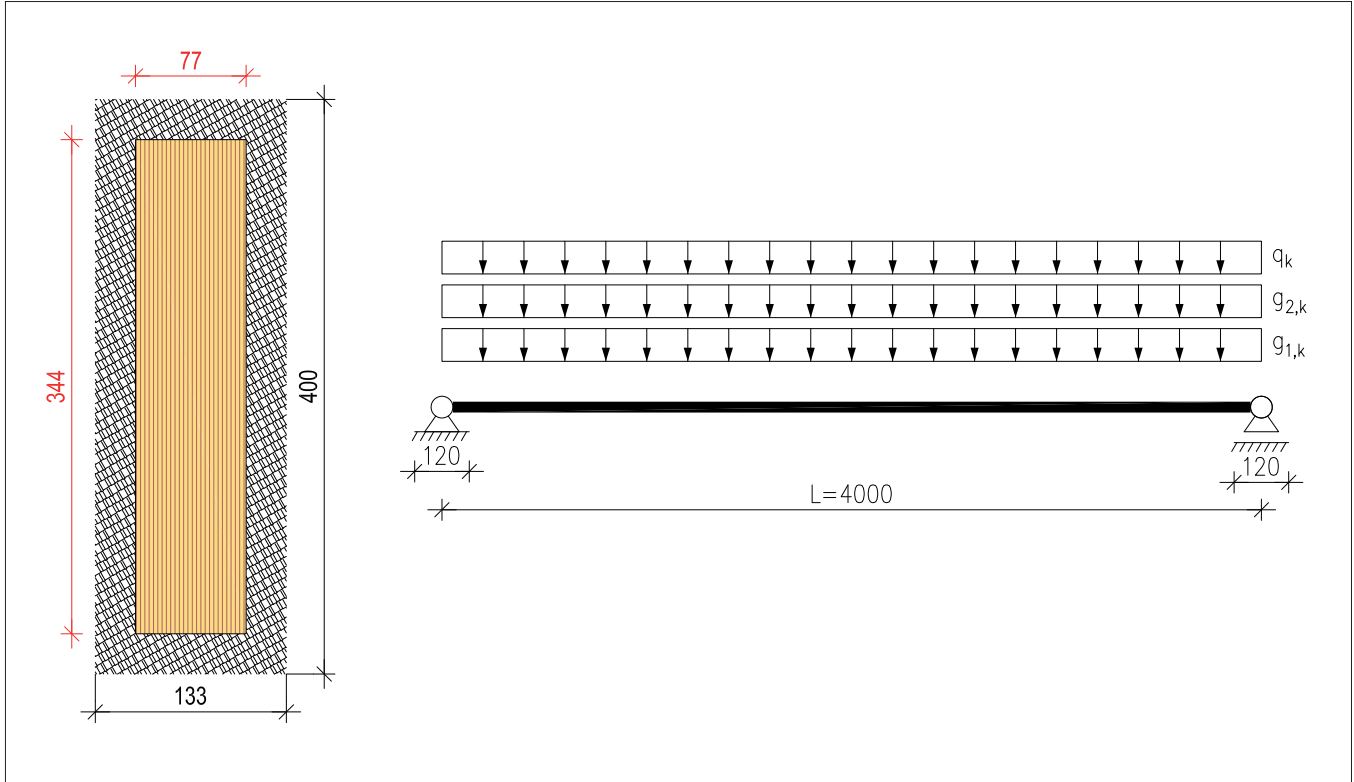
$\gamma_M = 1,2$ (default value in EC5)

$$k_{c,90} \cdot \frac{k_{mod}}{\gamma_M} \cdot f_{c,90,flat,k} = 1,4 \cdot \frac{1,1}{1,2} \cdot 2,2 \frac{\text{N}}{\text{mm}^2} = 2,8 \text{ N/mm}^2 > \sigma_{c,90,d} \rightarrow \text{OK}$$

Anchoring can be done with e.g. Rothoblaas WHT340 brackets for tension loads and Titan TFC200 brackets for shear loads.

9.11 MAIN BEAM OF ROOF STRUCTURE IN 30MIN FIRE EXPOSURE

Single-span main beam of a flat roof structure is GLVL 48 P beam 133x400 mm. Span length is $L = 4000$ mm, width of the loading area 8000 mm and roof purlins are on the top of the beam. Support length is 100 mm. Snow load s_k is 2,5 kN/m², own weight of the roof structure is 1,0 kN/m² and own weight of the beam is 0,2 kN/m. The structure fulfils the ULS and SLS requirements in normal temperature where $w_{net,fi} \leq L/300$ has been the most critical requirement. The fire resistance R30 of the beam needs to be verified for fire exposure on all sides of the beam.



Effective cross section size after 30 minutes fire exposure on all sides:

$$d_{ef} = \beta_n \cdot t + k_0 \cdot d_0 = 0,70 \frac{\text{mm}}{\text{min}} \cdot 30\text{min} + 1,0 \cdot 7\text{mm} = 28\text{mm}$$

$$\text{Width } b: 133 \text{ mm} - 2 \cdot 28 \text{ mm} = 77 \text{ mm}$$

$$\text{Height } h: 400 \text{ mm} - 2 \cdot 28 \text{ mm} = 344 \text{ mm}$$

Beam properties of the effective cross section after 30min fire exposure:

Bending strength edgewise $f_{m,0,edge,k}$	= 44 N/mm ²
Shear strength edgewise $f_{v,0,edge,k}$	= 4,2 N/mm ²
Compression perpendicular to grain edgewise $f_{c,90,edge,k}$	= 6 N/mm ²
Modulus of elasticity $E_{0,k}$	= 11 600 N/mm ²
Modulus of rigidity $G_{0,edge,mean}$	= 400 N/mm ²
Area of cross section $A = b \cdot h$	= 26488 mm ²
Section modulus $W_y = b \cdot h^2 / 6$	= 1,52 · 10 ⁶ mm ³
Moment of inertia $I_y = b \cdot h^3 / 12$	= 2,61 · 10 ⁷ mm ⁴
Moment of inertia $I_z = h \cdot b^3 / 12$	= 1,31 · 10 ⁷ mm ⁴
Torsion moment of inertia $I_{tor} = 0,3 \cdot h \cdot b^3$	= 4,71 · 10 ⁷ mm ⁴
Modification factor $k_{mod,fi}$	= 1,0
Modification factor k_{fi}	= 1,1
Material safety factor $\gamma_{M,fi}$ (default value in EC5)	= 1,0
Size effect factor $k_h = (300/344)^{0,15}$	= 0,98

9. CALCULATION EXAMPLES OF LVL STRUCTURES

Loading combinations

Snow load at roof level $q_k = \mu_1 \cdot C_e \cdot s_k$. Form factor $\mu_1 = 0,8$, when roof angle is less than 30° and in normal conditions $C_e = 1,0 \rightarrow q_k = 0,8 \cdot 1,0 \cdot 2,5 \text{ N/m}^2 = 2,0 \text{ kN/m}^2$.

Accidental load combination of fire in the ultimate limit state (ULS):

$$E_{d,ULS,fi} = \gamma_G \cdot (g_{1,k} + g_{2,k}) + \psi_1 \cdot \gamma_Q \cdot q_k$$

$$E_{d,ULS,fi} = 1,0 \cdot (8\text{m} \cdot 1,0 \text{ kN/m}^2 + 0,2 \text{ kN/m}) + 0,4 \cdot 1,0 \cdot 8\text{m} \cdot 2,0 \text{ kN/m}^2$$

$$E_{d,ULS,fi} = 14,6 \text{ kN/m}$$

Note: Safety factors γ_G , ψ_1 and γ_Q are according to Finnish National annex of Eurocode 0.

ULS design

Bending moment resistance

$$M_d = E_{d,ULS,fi} \cdot s \cdot L^2/8 = 14,6 \text{ kN/m} \cdot (4\text{m})^2/8 = 29,2 \text{ kNm}$$

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{29,2 \text{ kNm}}{1,52 \cdot 10^6 \text{ mm}^3} = 19,2 \text{ N/mm}^2$$

$$f_{m,d,fi} = \frac{k_{mod,fi} \cdot k_{fi} \cdot k_h}{\gamma_{M,fi}} \cdot f_{m,edge,k}$$

$$f_{m,d,fi} = \frac{1,0 \cdot 1,1 \cdot \left(\frac{300\text{mm}}{344\text{mm}}\right)^{0,15}}{1,0} \cdot 44 \frac{\text{N}}{\text{mm}^2} = 47,4 \frac{\text{N}}{\text{mm}^2}$$

$$\sigma_{m,d} \leq f_{m,d,fi} \rightarrow \text{OK}$$

Lateral torsional buckling

The beam is loaded on the top side and the purlins won't act as supports against lateral torsional buckling for 30min fire exposure. Therefore according to Table 4.9 and EN1995-1-2, clause 4.3.2 (1) and the effective length L_{ef} of the beam is

$$L_{ef} = 0,9 \cdot L + 2 \cdot h = 0,9 \cdot 4000\text{mm} + 2 \cdot 344\text{mm} = 4288\text{mm}.$$

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{l_{ef} W_y} \quad (4.42)$$

$$\sigma_{m,crit} = \frac{\pi \sqrt{10600 \text{ N/mm}^2 \cdot 1,31 \cdot 10^7 \text{ mm}^4 \cdot 400 \text{ N/mm}^2 \cdot 4,71 \cdot 10^7 \cdot \text{mm}^4}}{4288 \text{ mm} \cdot 1,52 \cdot 10^6 \text{ mm}^3}$$

$$\sigma_{m,crit} = 25,8 \text{ N/mm}^2$$

$$\lambda_{rel} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} = \sqrt{\frac{44 \text{ N/mm}^2}{25,8 \text{ N/mm}^2}} = 1,36 \quad (4.41)$$

$$\text{when } 0,75 < \lambda_{rel,m} \leq 1,4, k_{crit} = 1,56 - 0,75 \cdot \lambda_{rel,m} = 1,56 - 0,75 \cdot 1,36 = 0,58$$

$$k_{crit} \cdot f_{m,d,fi} = 0,58 \cdot 47,4 \text{ N/mm}^2 = 27,5 \text{ N/mm}^2$$

$$\sigma_{m,d} = 19,2 \text{ N/mm}^2 \leq k_{crit} \cdot f_{m,d} \rightarrow \text{OK} \quad (4.38)$$

Shear resistance

$$V_d = E_{d,ULS,fi} \cdot L/2 = 14,6 \text{ kN/m} \cdot 4,0\text{m}/2 = 29,2 \text{ kN}$$

$$\tau_{v,d} = \frac{3 \cdot V_d}{2 \cdot A} = \frac{3 \cdot 29,2 \text{ kN}}{2 \cdot 26488 \text{ mm}^2} = 1,7 \text{ N/mm}^2$$

$$f_{v,d,fi} = \frac{k_{mod,fi} \cdot k_{fi}}{\gamma_{M,fi}} \cdot f_{v,0,edge,k} = \frac{1,0 \cdot 1,1}{1,0} \cdot 4,2 \frac{\text{N}}{\text{mm}^2} = 4,6 \text{ N/mm}^2$$

$$\tau_{m,d} \leq f_{v,d,fi} \rightarrow \text{OK}$$

Compression perpendicular to grain

When the main beam is supported on a wooden column which has the notional charring rate $\beta_n = 0,70\text{mm/min}$, the support length becomes

$$\sigma_{c,90,d} = \frac{29,2\text{kN}}{77\text{mm} \cdot (72\text{mm} + 15\text{mm})} = 4,4 \text{ N/mm}^2$$

$$k_{c,90} \cdot f_{c,90,d,fi} = \frac{k_{c,90} \cdot k_{mod,fi} \cdot k_{fi}}{\gamma_{M,fi}} \cdot f_{c,90,edge,k} \quad (4.14)$$

$$k_{c,90} \cdot f_{c,90,d,fi} = \frac{1,0 \cdot 1,0 \cdot 1,1}{1,0} \cdot 6 \text{ N/mm}^2 = 6,6 \text{ N/mm}^2$$

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{m,0,d,fi} \rightarrow \text{OK}$$

$$l_{\text{support},fi} = 100\text{mm} - 0,70 \frac{\text{mm}}{\text{min}} \cdot 30\text{min} + 1,0 \cdot 7\text{mm} = 72\text{mm}$$

$$F_{c,90,d} = V_d = 29,2 \text{ kN}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{c,90,d}}{b \cdot (l_{\text{support},fi} + 15 \text{ mm})} \quad (4.13)$$

Discussion

According to EN1995-1-2:2004, clause 4.3.1 it is not necessary to verify compression perpendicular to the grain and shear resistance of a beam in the structural fire design. In this example they didn't become critical, but in the detailing it shall be verified that the beam is securely supported also when the support length becomes smaller due to charring of the supports.

10. References

Reference Documents

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LVL Handbook

EUROPE

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