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A guided bending strategy for the geometric and structural design of an elastically bent geodesic timber column

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Large elastic deformations provide an alternative form-giving strategy for complex double-curved timber structures, which are otherwise difficult to realize without extensive machining, gluing, or material-intensive milling. Geodesic curve networks offer particularly advantageous geometric properties for such applications, as they allow initially straight laths to be bent and stacked efficiently while remaining tangent to the design surface. This property helps overcome the structural limitations typically imposed by the material strength and bending curvature. This research presents the fundamental geometric principles necessary for accurately modeling geodesic grid structures and outlines the key design constraints associated with their implementation. It further introduces ongoing work on guided bending, an inverse-design approach that employs tailored incisions to weaken the cross section and steer the deformation of a lath into a prescribed spatial configuration. The presented project serves as a demonstrator for these methods. The paper focuses in particular on the geometric and structural development of its elastically bent geodesic timber column, a primary load-bearing element composed of mechanically laminated lamellae formed through guided elastic deformation. The project illustrates the structural viability of elastically bent compound beam assemblies and shows how guided bending can significantly broaden the feasible design space for such systems.

KEYWORDS

architectural geometry, computational design, elastic deformation, geodesic curves, guided bending, structural design, timber construction

1 Introduction

Regrowing natural materials and their applications in construction have recently emerged as a promising field of research, driven by the increasing demand for resource-efficient and sustainable building materials in the light of pressing climate challenges and the projected global population growth over the next fifty to 60 years [United Nations \(2024\)](#). Furthermore, the construction industry accounts for a substantial share of global resource depletion [United Nations Environment Programme \(2020\)](#), whereas, sustainably managed regrowing natural materials offer the potential for an abundant, self-regenerating resource [Svilans \(2020\)](#). However, naturally grown materials are generally highly inhomogeneous in size and shape [Sotayo et al. \(2020\)](#), and their mechanical properties are so inconsistent that their application in contemporary standardized construction becomes difficult without further processing. In timber manufacturing, these setbacks are typically overcome through the adhesive

bonding of multiple smaller sections of boards, veneers or fibers. This underlies the principles of engineered wood products, which is a way to turn natural wood into a reliable and predictable product, providing a valid alternative to steel and reinforced concrete [Svilans \(2020\)](#), [Sotayo et al. \(2020\)](#), [Ramage et al. \(2017\)](#). These engineering advancements have enabled the construction of numerous remarkable mass-timber buildings over the past decades, including the Yeosu Golf Club [Scheurer et al. \(2013\)](#) and the Centre Pompidou-Metz [Grinwald and Karen \(2010\)](#), among many others. While the development of contemporary mass-timber products has been essential to the realization of these projects, material innovation alone does not fully account for their development. Complementing these engineering advancements, the advent of digital tools and computational modeling has significantly facilitated the development of such complex architectural geometries. It is since long well known to designers and engineers that form and curvature play an important role in the design of material-efficient large-spanning structures. Yet, the fabrication of such geometrically intricate elements remains a challenging and resource-intensive task. Specifically for designed double-curved structural timber envelopes, those elements often require extensive molding, gluing, and machining, or result in substantial material waste through subtractive milling processes [Mayencourt et al. \(2025\)](#). In contrast, pre-rationalization, i.e., intervening in the early design phases of such complex geometries and considering relevant constructive properties, can enable alternative form-giving strategies for curved timber structures leveraging elastic bending [Montagne \(2025\)](#).

1.1 Related work on elastic bending in architecture

Chebyshev nets are a well-established but still active topic of research, particularly in the context of elastic gridshell structures. These structures are composed of equilateral quadrilateral grids that can be assembled flat and deployed into a spatial configuration. This transformation is enabled by the flexibility of the individual elements and the absence of rotational stiffness in the nodes. Final stability of these grids is achieved through the application of additional bracing after the erection process [Douthé et al. \(2017\)](#). This theory and technique has been perfectly demonstrated by the renowned gridshells of the Multihalle in Mannheim [Liddell \(2015\)](#) and the Downland gridshell [Kelly et al. \(2001\)](#). However, Chebyshev nets have a highly constrained design space [Garg et al. \(2014\)](#), which only opens up when deviations from the underlying design surface and singularities are allowed [Sageman-Furnas et al. \(2019\)](#). [Schling et al. \(2018a\)](#), [Schling et al. \(2018b\)](#) have explored gridshells based on asymptotic curves, characterized as curves with zero normal curvature. As a result, the members are oriented with their strong axis parallel to the surface normal, which allows for a flat assembly during which the members are elastically bent around their weak axis. The main limitation of this approach is that these curves only exist on surfaces with negative Gaussian curvature [Mesnil and Baverel \(2023\)](#). Another approach for leveraging large elastic deformations was proposed by [Lienhard \(2014\)](#), [Lienhard et al. \(2011\)](#) where individual elements are bent elastically and coupled together additively to create spatial

structures. However, a subsequent study [Lienhard and Knippers \(2013\)](#) demonstrated that large-scale applications of these structural systems is highly limited, as the material strength and the bending curvature limit the admissible bending stiffness (i.e., the cross section) of the individual elements. This, in turn, makes the systems sensitive to snap-through buckling, imposing structural constraints on their scalability. A strategy to address these scalability limitations is to stack multiple elements in superposed layers. A prominent example of this approach is the EXPO roofs [Natterer et al. \(2001\)](#), [Norbert et al. \(2000\)](#), realized for the World Exposition in Hannover in 2008. Their design is based on geodesic curves, which, unlike asymptotic curves, have zero geodesic curvature. Consequently, planks aligned with geodesic curves remain tangent to the surface, allowing them to be effectively stacked. In practice, however, an elastically bent element does not naturally adopt any geodesic configuration. Instead, it tends to deform into a state of minimal elastic energy under given boundary conditions, which is commonly known as the elastica shape [Levien \(2008\)](#). Thus, to make an elastic element deviate from the elastica external constraints are needed. However, when forcing an elastic element to follow another path than that of minimal elastic energy, geometrical discrepancies from the intended design shape will arise due to spring-back effects [Andersson Largueche et al. \(2024\)](#). With the aim to mitigate these effects and to expand the design space of elastic geodesic grid structures, researchers have sought to control the elastic response of elements by manipulating their material or mechanical properties, such that the element naturally assumes the intended target geometry. Parts of this ongoing research was applied in the large-scale demonstrator project Komorebi ([Figure 1](#)).

1.2 Komorebi

The Kronenplatz square in the city of Karlsruhe, southern Germany. Due to climate change the yearly average temperature in Karlsruhe has increased by 1.3 °C over the last 60 years [Die Städtische Wärmeinsel, 2025](#), and temperatures between 35–40 °C are not unusual in the summer months. This makes the Kronenplatz almost unusable during this period as shading on the square is almost non-existent. To tackle this challenge, the Karlsruhe Institute of Technology (KIT) initiated a research project as part of the study of shading structures for microclimatic comfort in urban settings. The project culminated in the design and realization of Komorebi, which is a structural solution that provides both shading and seating for public use. Komorebi was developed and realized by the Professorships Design of Structures (dos) and Digital Design and Fabrication (DDF) of the KIT, showcasing their on-going research on guided elastic bending and regrowing natural building materials, in collaboration with the project partner FibR GmbH.

The structure has a total height of 4.6 m and a maximum span of 7.6 m and consists of four main structural parts ([Figure 2](#)): steel foundations, steel substructure, fiber sail and timber column. A parking garage is located underneath the square, thus, any anchorage of the structure to the ground was not possible. The structure is instead supported by three steel blocks, large enough to distribute the loads on the ground slab and to provide enough additional weight, to prevent the structure from overturning or sliding under extreme wind load. Aware of the constraints set by the



FIGURE 1
Photograph of the completed Komorebi project at the Kronenplatz in Karlsruhe.

fabrication capabilities, the spatial steel substructure was conceived as a kit of parts that could easily be put together on site and be manufactured in a 3-axis laser cutting process. This vastly reduced production costs and erection time on site. The sail structure is constructed out of flax fibers and consists of twelve separate segments. Each segment is manufactured in a robotic winding process, in which the fiber strands are additively wound around winding pins attached to a steel frame, outlining the sail geometry, resulting in a material-efficient and lightweight structural component. Large elastic deformation was used as a form-giving strategy in the production of the timber column, where lamellae were subsequently bent in superposed layers and mechanically laminated using steel screws to create a near-to-solid beam compound. The resulting load-bearing timber element is entirely free of synthetic adhesives, enabling full end-of-life separability and preserving the material's recyclability and reuse potential.

1.3 Contribution

This article presents guided bending as an inverse-design method in which tailored incisions aligned with the conjugate direction locally weaken the lamella cross-section to steer its elastic deformation toward a prescribed target geometry. The method is validated through the geometric, structural, and constructive development of an elastically bent geodesic timber column, demonstrating the viability of such assemblies as

primary load-bearing elements and the potential of guided bending to expand their design space.

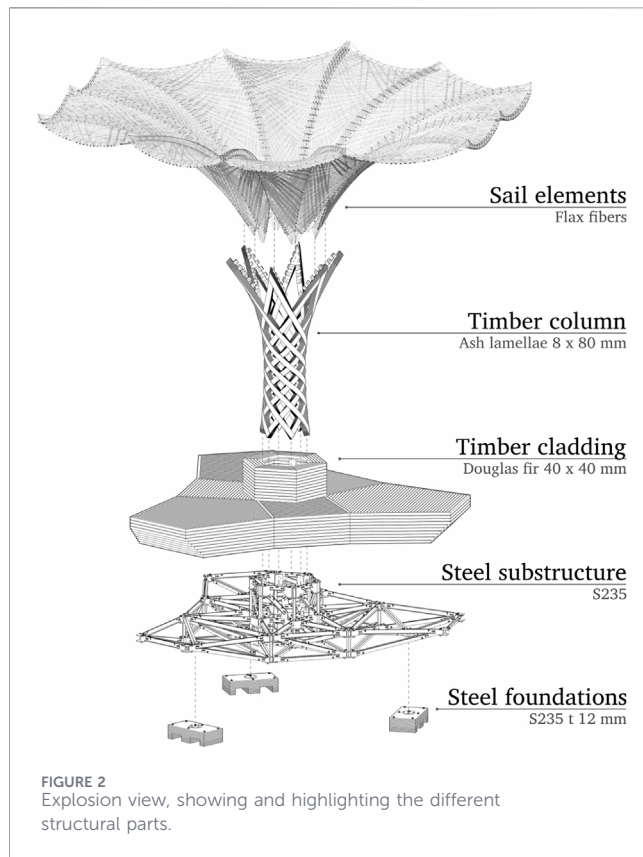
2 Geometrical design

2.1 Generation of the base surface

The column geometry is derived from a geodesic curve network generated on a base design surface. A key advantage of geodesic curves is their applicability to any surface, providing considerable flexibility in the design process. This characteristic was leveraged in the column design, where the surface generation was closely aligned with the project's overarching architectural intent. The resulting shape is a rotational surface defined by a generatrix controlled by five control points (Figure 3). The surface curvature influenced not only the bending radii of the individual planks, and therefore the embedded stress deriving from the bending process, but also the density of the geodesic network, as higher double curvature produced a correspondingly tighter net.

2.2 Generation of geodesic strips

To explain the generation of the geodesic strips, some fundamental concepts of differential geometry will briefly be reviewed in the following sections.



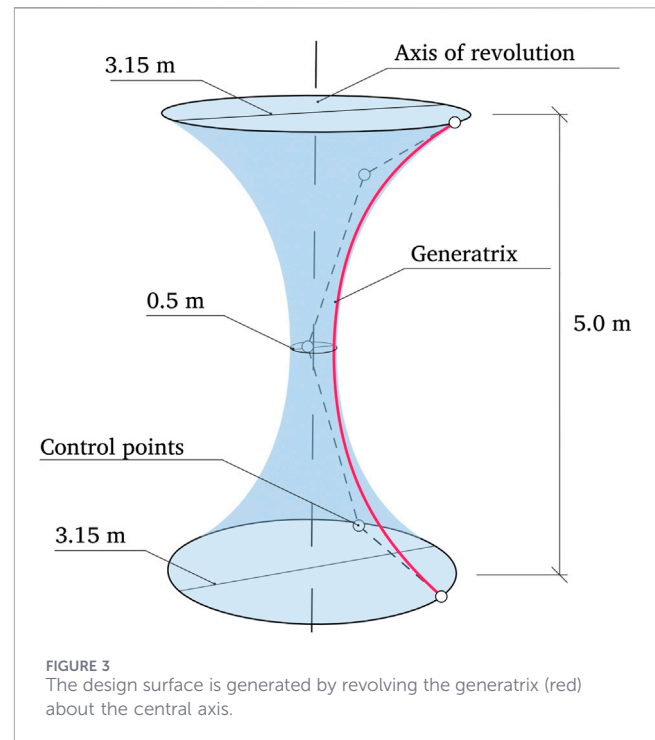
2.2.1 Curves on surfaces

The curve γ , parameterized by arclength s and constrained to a surface S , is described by the Darboux frame. This frame consists of the curve tangent vector $\mathbf{t}(s)$, the surface normal vector $\mathbf{n}(s)$ and their cross product $\mathbf{g}(s) = \mathbf{t}(s) \times \mathbf{n}(s)$, the tangent normal vector.

Using this frame and the arc lengths of γ , three fundamental curvature measures can be defined:

- the normal curvature $\kappa_n = \langle d\mathbf{t}/ds, \mathbf{n}(s) \rangle$, which is a measure of how a curve bends in the $\mathbf{t}(s) - \mathbf{n}(s)$ plane of the surface,
- the geodesic curvature $\kappa_g = \langle d\mathbf{t}/ds, \mathbf{g}(s) \rangle$, which is a measure of how a curve bends in the tangent plane of the surface,
- the geodesic torsion $\tau_g = \langle d\mathbf{n}/ds, \mathbf{g}(s) \rangle$, which is the rate at which the frame rotates around the tangent vector along the curve,

where $\langle \cdot, \cdot \rangle$ denotes the vector scalar product. These three properties are used to categorize different types of curves, where geodesic curves and asymptotic curves especially have found application as supporting structure in architecture [Schling et al. \(2022\)](#). This can primarily be attributed to their geometric properties, which allow them to be effectively modelled as strips with straight development, thereby facilitating fabrication and construction while providing favorable structural properties.



2.2.2 Geodesic curve network

Designers typically have limited control over the precise shape of geodesic curves on free-form surfaces [Pottmann et al. \(2010\)](#). Such curves are normally generated either as an initial value problem by a start point and a vector or as a boundary value problem by a start and an end point. For the column design, the intent was that the lamellae should fully wrap around the column, i.e., the start and end points of the geodesic curve were predetermined and positioned above one another on the same generatrix. However, as the geodesic curve is the shortest and straightest path between two points on a surface [Do Carmo \(2016\)](#), this would simply have reproduced the generatrix itself if the curve was generated as a boundary value problem. Generating the geodesic from a start point and an initial vector was also considered unsuitable, as this approach would not offer sufficient control over both the start and end positions. Instead, the geodesic curve forming the basis of the curve network for the column design was obtained by constrained relaxation. The underlying problem is a surface-constrained length minimization problem: for two prescribed points $\mathbf{p}_0$ and $\mathbf{p}_1$ on the design surface S , the geodesic is the curve

$$\min L[\gamma] = \int \|\gamma'(t)\| dt, \quad \text{subject to } \gamma(t) \in S, \gamma(0) = \mathbf{p}_0, \gamma(1) = \mathbf{p}_1. \quad (1)$$

In terms of the curvature measures introduced in [Section 2.2.1](#), the solution of [Equation 1](#) is the curve whose geodesic curvature vanishes along its entire length, $\kappa_g = 0$. In discrete form, γ is represented by a polyline whose end points are fixed and whose interior points are constrained to S , and the segment lengths are minimized. The process can be thought of as wrapping a rope

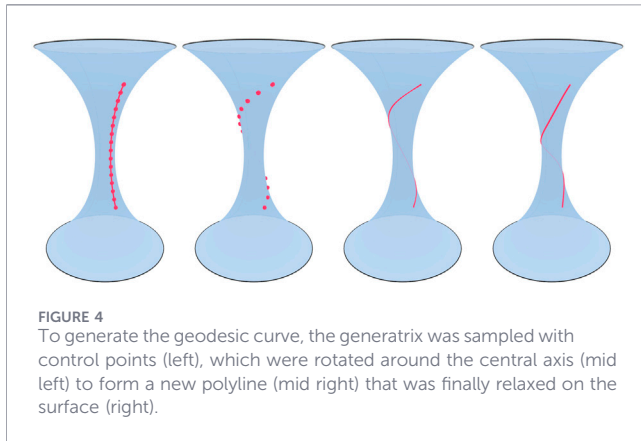


FIGURE 4
To generate the geodesic curve, the generatrix was sampled with control points (left), which were rotated around the central axis (mid left) to form a new polyline (mid right) that was finally relaxed on the surface (right).

around the surface and tightening the rope by pulling at each end. Concretely, the generatrix was sampled with control points (Figure 4 (left)), which were rotated around the central axis (Figure 4 (mid left)) to form the polyline (Figure 4 (mid right)) that was finally relaxed on the surface (Figure 4 (right)) using Kangaroo2, a live physics engine for interactive simulation, optimization, and form-finding [Piker \(2013\)](#). This method has proven to be a robust computational strategy for generating helix-like geodesic curves as a boundary value problem on complex surfaces without a clear mathematical derivation.

The resulting curve is only an approximation of a true geodesic, therefore, after relaxation, the extent of its deviation from a true geodesic was assessed by measuring the geodesic curvature along the curve. The resulting geodesic curvature κ_g is not strictly zero along the arc length of the curve. Nevertheless, since its magnitude remains well below 10 % of the normal curvature, this deviation was considered acceptable. The final curve network was formed by mirroring the base curve in the global xz -plane and rotating those curves five times around the surface with a rotational offset of 60°.

2.2.3 Materialization of geodesic curve network

An initially flat strip can, under pure bending and torsion, be deformed onto a curved surface. Such a deformation is called isometric, defined as a length-preserving and strain-free transformation [Audoly and Pomeau \(2010\)](#). Likewise, the spatial strip can be unrolled back into a flat strip on the plane, and that again, is an isometric deformation. A surface (or strip in this case) that allows such transformations is referred to as developable. As the construction principles of the column employ such transformations, it becomes apparent that the spatial strips defining the column geometry are developable.

Developable surfaces are typically defined as those with zero Gaussian curvature, or equivalently, as ruled surfaces with a constant normal direction for each ruling [Rabinovich et al. \(2018\)](#). Consequently, a developable surface is always a ruled surface (however, the inverse is not always true) [Audoly and Pomeau \(2010\)](#). Thus, a developable strip along a predefined center curve is defined through a discrete number of rulings along the curve. The ruling directions are often taken from the Darboux frame, which provides a convenient geometric reference. This choice, however, is

only true when the center curve lies in a plane. For spatial curves, aligning the rulings with the Darboux frame yields only an approximate developable strip under sufficiently fine discretization, as the resulting rulings are not strictly coplanar and therefore do not form a truly developable surface. To ensure developability, the rulings must instead be chosen as the vector conjugate to the tangent vector [Wallner \(2017\)](#). Graphically this is explained as the intersection line of two neighboring tangent planes (Figure 5).

In differential geometry, conjugate vectors are defined using the second fundamental form [Do Carmo \(2016\)](#), which is defined as follows:

$$II(\mathbf{x}, \mathbf{y}) = -\langle (d\mathbf{r})\mathbf{x}, (d\mathbf{n})\mathbf{y} \rangle. \quad (2)$$

Here, II is the matrix of the second fundamental form of the surface $\mathbf{r}$ and defined as follows:

$$II = \begin{bmatrix} L & M \\ M & N \end{bmatrix}, \quad (3)$$

where L , M and N are coefficients derived from the surface's second derivatives:

$$L = \langle \mathbf{n}, \mathbf{r}_{uu} \rangle, \quad M = \langle \mathbf{n}, \mathbf{r}_{uv} \rangle, \quad N = \langle \mathbf{n}, \mathbf{r}_{vv} \rangle. \quad (4)$$

Two vectors $\mathbf{u}$ and $\mathbf{v}$ are conjugate if and only if $\mathbf{u}^T II \mathbf{v} = 0$.

A curve network on a surface $\mathbf{r}$ where the tangent vectors in each intersection point are conjugate is called a conjugate net, which is a notion in differential geometry that is well known, especially from the design of planar quadrilateral (PQ) meshes. If the rows in a PQ mesh undergo refinement while the columns are kept unchanged (or the other way around), each column will converge to a smooth developable strip [Pottmann et al. \(2008\)](#), [Liu et al. \(2006\)](#). Thus, the rulings of a developable strip with its center line being the known curve γ , parameterized by arc-length s , are defined as the vector conjugate to the curve tangent vector $\mathbf{t}(s)$ in a discrete number of points along the curve, which can be solved for by replacing one of the vectors in Equation 1 with $\mathbf{t}(s)$

$$\mathbf{t}^T \begin{bmatrix} L & M \\ M & N \end{bmatrix} \mathbf{t}_{conj}(s) = 0, \quad \text{where} \quad \mathbf{t}(s) = \begin{bmatrix} \mathbf{r}_u & \mathbf{r}_v \end{bmatrix} \begin{bmatrix} du/ds \\ dv/ds \end{bmatrix}. \quad (5)$$

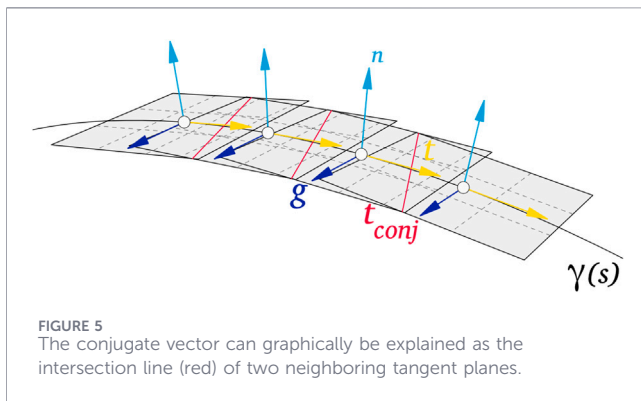
With

$$\begin{bmatrix} a & b \end{bmatrix} := \mathbf{t}^T(s) \begin{bmatrix} L & M \\ M & N \end{bmatrix} \quad (6)$$

Equation 5 becomes $\begin{bmatrix} a & b \end{bmatrix} \mathbf{t}_{conj}(s) = 0$. It follows directly that the solution is $\mathbf{t}_{conj}(s) = \begin{bmatrix} b/a & -1 \end{bmatrix}^T \rho$ where ρ is an arbitrary constant. Lastly, the conjugate vector $\mathbf{t}_{conj}(s)$ is calculated as follows

$$\mathbf{t}_{conj} = \mathbf{r}_u \cdot \frac{b}{a} - \mathbf{r}_v. \quad (7)$$

Because the rulings are, by definition, straight lines, they must lie in the tangent plane of the underlying surface. Determining their lengths, which correspond to the width of the resulting plank, then reduces to a straightforward geometric calculation. Specifically, the ruling length is given by $L = W/\sin(\alpha)$, where α denotes the angle between the curve's tangent vector and the ruling and W is the desired plank width.



2.2.4 Treatment of singularity

At a point in the upper region of the column, the curvature graph of the curve suddenly changes direction, from pointing out from the design surface to pointing inwards (Figure 6 (mid)), meaning that the curve normal rotates 180 degrees around the tangent vector. Such a rotation can occur only if the curvature passes zero, indicating a singularity. Since geodesics satisfy $\kappa = \kappa_n$, the normal curvature κ_n must vanish at that point, which is precisely the defining property of an asymptotic direction. Consequently, at the singularity the geodesic temporarily aligns with an asymptotic direction. And because asymptotic directions are, by definition, conjugate to themselves, the conjugate vector and the tangent vector align in that point. Recalling that the conjugate vector defines the axis around which the tangent plane rotates, it follows that at this point the strip is not bending but instead undergoes pure twist. This is also confirmed by the curvature graph in Figure 6 showing non-zero torsion in the point of zero curvature. For

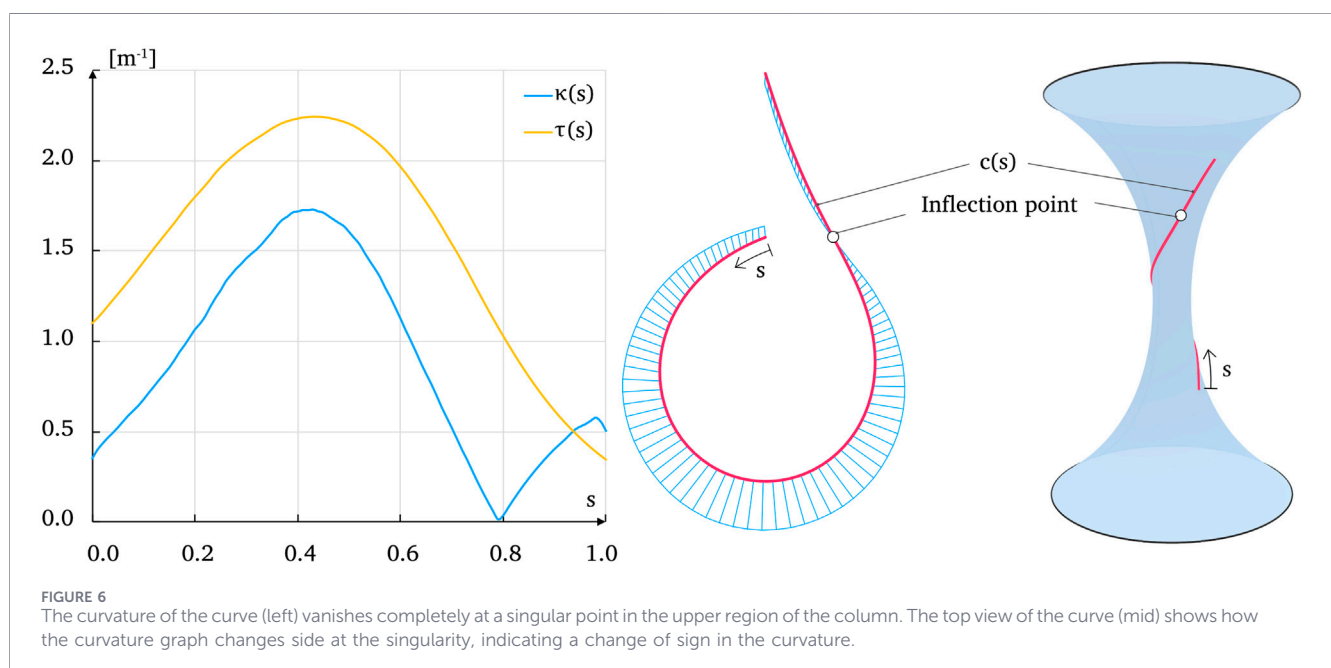
a strip to experience pure twist it must change in length. However, such a deformation is not isometric and consequently does not yield a developable surface. As in Schling et al. (2018a), a limited deviation from a true developable strip was tolerated to achieve the intended geometry. The resulting strain was computed to be below 1 %, and it was assumed that the material could accommodate for the corresponding stresses.

Figure 7 shows the rulings (in blue) around the singularity for four different plank widths. It can be observed that the ruling angle increases as the singularity is approached. When the ruling angle gets too sharp, it will intersect the neighboring rulings and consequently make the resulting surface non defined in that area. As the rulings get longer for increasing plank width, such ruling intersections start to occur at a longer distance away from the singularity, i.e., the non-developability of the singularity propagates to a larger area for wider planks. For the given plank width of 8 cm, this corresponded to a region of approximately 71 cm along the curve, representing less than 10 % of the total area of a 4 m plank.

3 Structural design

3.1 Guided bending

When an elastic element is forced to follow a predefined path that deviates from this minimum-energy state, additional internal residual stresses are introduced. These stresses can lead to geometric deviations from the intended design or even structural failure. To mitigate such effects and expand the range of achievable configurations, current research on controlled elastic bending was employed to guide the lamellae into their target shape.



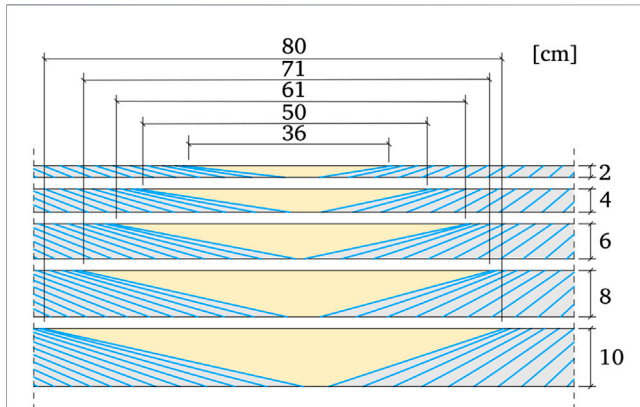


FIGURE 7
Propagation of non-developable area depending on the lamella width.

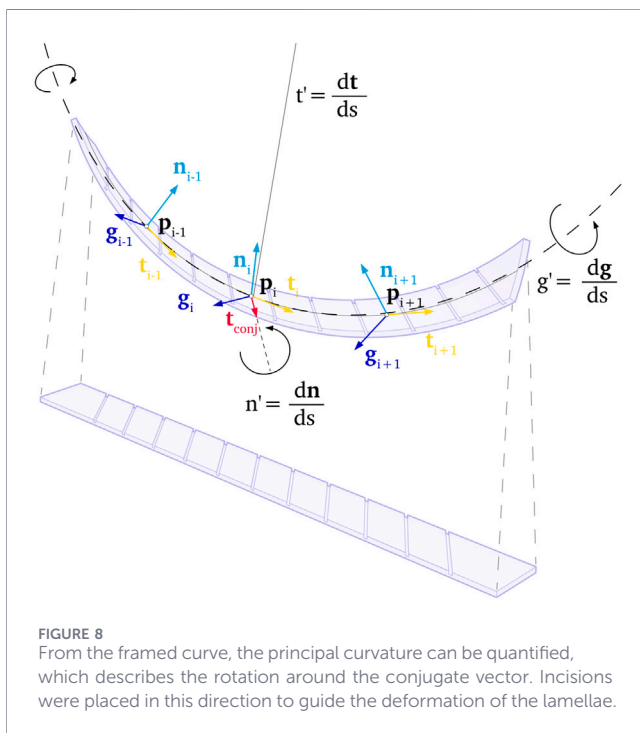


FIGURE 8
From the framed curve, the principal curvature can be quantified, which describes the rotation around the conjugate vector. Incisions were placed in this direction to guide the deformation of the lamellae.

3.1.1 Framed curves

Spatial deformation of a rod or lath is always a combination of bending and torsion [Charrondière et al. \(2024\)](#), [Hafner and Bickel \(2023\)](#). To be able to measure these quantities we attach a moving frame F to the curve ([Figure 8](#)). We recall that $\kappa^2 = \kappa_g^2 + \kappa_n^2$, however, as only geodesic curves are considered in this work $\kappa = \kappa_n$. Thus, the curvature and torsion of the curve can be expressed as:

$$\kappa_n = |\mathbf{t}'| = \frac{\arccos(\langle \mathbf{t}_{i-1}, \mathbf{t}_{i+1} \rangle)}{s_{i+1} - s_i}, \quad (8)$$

$$\tau = |\mathbf{g}'| = \frac{\arccos(\langle \mathbf{g}_{i-1}, \mathbf{g}_{i+1} \rangle)}{s_{i+1} - s_i}. \quad (9)$$

The total curvature of a spatial curve can be expressed as the derivative of the normal vector $\mathbf{n}$ with respect to the arc length s

$$\omega = |\mathbf{n}'| = \frac{\arccos(\langle \mathbf{n}_{i-1}, \mathbf{n}_{i+1} \rangle)}{s_{i+1} - s_i}, \quad (10)$$

or similarly as

$$\omega = \sqrt{\kappa^2 + \tau^2}. \quad (11)$$

Consequently, the total curvature is taking both curvature and torsion into consideration. In other words, rather than expressing the curve's motion as two successive rotations about two fixed frame axes, it can equivalently be described as a single rotation about one axis, whose orientation varies along the curve. This axis corresponds to the vector around which the moving frame F rotates as one progresses along the curve γ , commonly referred to as the Darboux vector [Hafner and Bickel \(2023\)](#), [Bergou et al. \(2008\)](#). For a geodesic curve the Darboux vector lies in the $t-g$ plane, since the geodesic curvature is zero, and coincides with the vector conjugate to the tangent vector (i.e., the ruling) [Prautzsch and Boehm \(1994\)](#). To guide the deformation of the lamellae, a series of incisions were introduced in the direction of the conjugate vector ([Figure 8](#)). These incisions extended halfway through the thickness of the lamella and were spaced 4 cm apart along the centerline. Acting as controlled hinges, the incisions directed the local bending at each point, thereby guiding the overall deformation of the lamellae toward their intended target shape.

3.2 Lamella layout

The applied construction methodology ([Figure 9](#)) follows the principles stemming from the developments of Julius [Natterer et al. \(2000\)](#), which has previously been applied in several prototypes and research demonstrators by the authors [Andersson Largueche and La Magna \(2024\)](#), [Andersson Largueche et al. \(2024\)](#). The structural compound was formed from five continuous ash lamellae, 80 mm wide and 8 mm thick, arranged in six axes in both directions and bolted together at their intersections, to secure them in their spatial arrangement. Shorter infill lamellae were inserted into the gaps created by the weave-like stacking of the two lamella directions. These infill elements were planned and manufactured with a controlled length tolerance, kept as small as possible to enhance the compound's load-bearing performance, yet sufficient to facilitate the insertion process. Finally, the entire assembly was mechanically laminated by screwing the layers together with 4 mm flange-head self-drilling timber screws, resulting in an overall compound depth of 72 mm, comprising five continuous and four infill layers.

3.3 Fabrication and erection

The straight-development property of the geodesic planks obviates the need to unroll their three-dimensional geometry. Instead, the necessary information, such as bolt hole positions

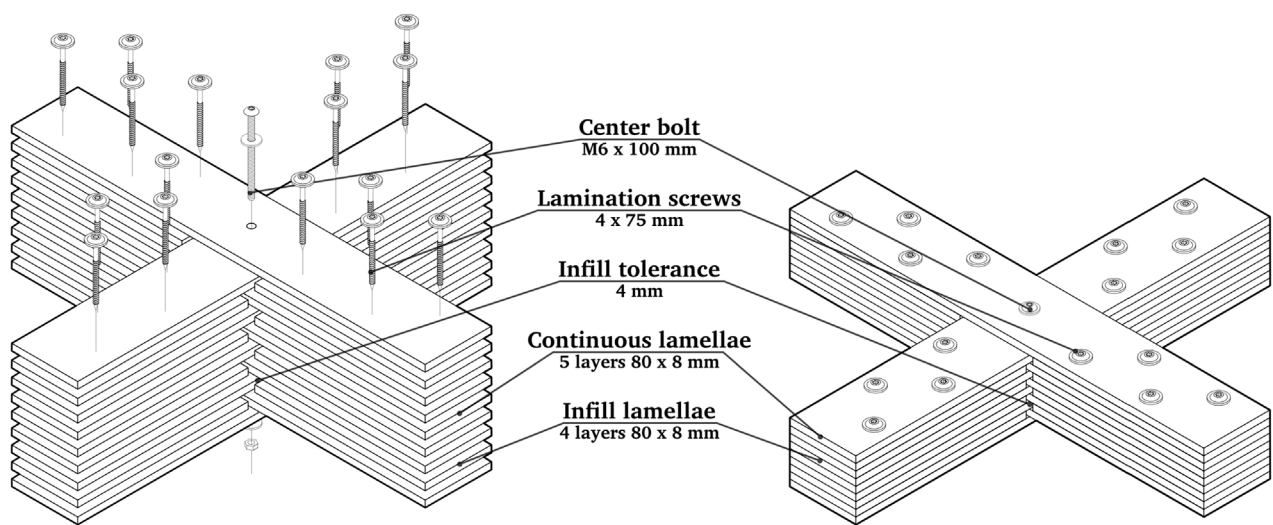


FIGURE 9
Basic construction principle of the lamellae layup.

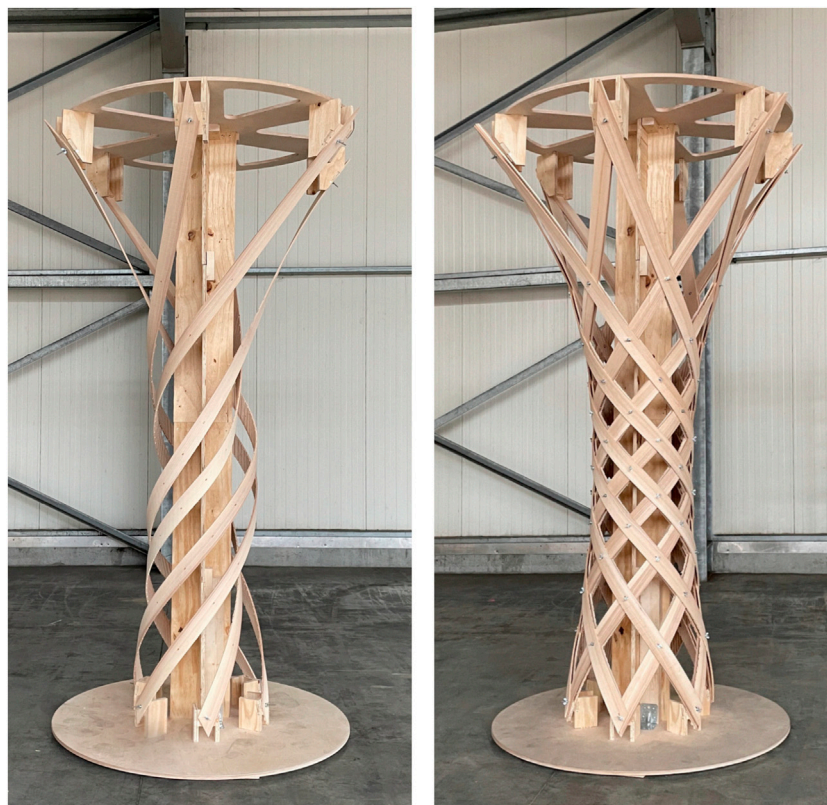


FIGURE 10
During the assembly of the first layer the lamellae sagged under their self-weight (left) however, after completing the second layer, the lamellae were locked into their final geometry (right).

and the position and angle of the incisions, can be analyzed on the 3D geometry and directly mapped onto a straight line on the plane representing the center axis of the spatial strip. This approach facilitates the generation of production data significantly.

A recent study [Naboni and Marino \(2021\)](#) demonstrated that, in wedge kerfing—a method in which the deformation of a batten is geometrically controlled through guiding cutouts (kerfs) and the insertion of wedges—the guiding cutouts should be placed in the

compression zone to improve structural capacity. Simple experimental tests confirmed this, demonstrating that lamellae with guiding incisions on the compression side achieved much smaller bending radii than those where the guiding incisions were placed on the tension side. As previously noted, the curvature of the geodesic curves changes direction at the singularity point, meaning the compression zone shifts from the inner to the outer side. Consequently, the guiding incisions in the upper part of the column would need to have been placed on the opposite side compared to those in the lower part. However, because the curvature in the upper region is relatively small, the corresponding shift of the guiding incisions to the opposite compression zone was omitted to avoid a disproportionate increase in production time.

The geometrical shape of the column was encoded in the planks during the fabrication process through the exact placement of the bolt holes and the guiding incisions, which were all planar machine operations. The simplicity of the manufacturing steps thus eliminated the need for advanced machinery, allowing the entire production to be carried out with a standard three-axis CNC mill.

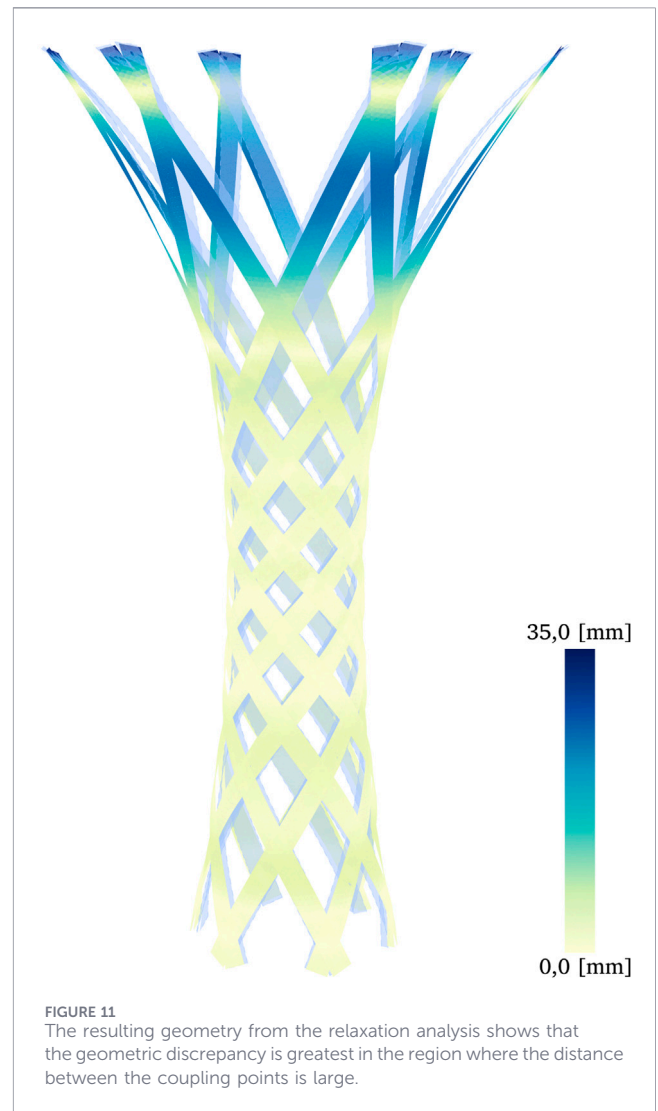
As the erection process was entirely guided by the overlap points, a reference scaffolding was required to accurately fix the position and orientation of each lamella's endpoints in space. To guarantee precise positioning and alignment of the connection bolts, the scaffolding was designed (Figure 10) as a prefabricated kit of parts with predefined notches and slots, eliminating the need for manual measurements that otherwise might have introduced unwanted tolerances during assembly. Furthermore, the design of the scaffolding components was developed with fabrication limitations in mind, as also they were produced using a three-axis CNC mill.

Due to the weave-like stacking, each layer had to be fully assembled (six lamellae, one for each beam axis) before the next layer could be installed. During the assembly of the first layer, the lamellae sagged under their self-weight (Figure 10 (left)), as no opposing lamellae were yet in place to stabilize the form. The column's shape began to emerge once the first lamella of the second layer was installed, and after completing the second layer, the lamellae were locked into their final geometry (Figure 10 (right)).

3.4 Relaxation of the geodesic strips

The column geometry served as input for the production frame for the fiber sails and it was therefore important that the column was modelled precisely and that it resembled the actual built geometry to ensure constructability. Since the guiding incisions in the upper part of the column were omitted, the spring-back effect in that region was considered to increase the geometric accuracy of the computational model, even though the curvature was considered relatively small in that region.

The relaxed geometry including spring back was generated in a numerical simulation performed in Kangaroo2. The planks of the two inner-most layers were modelled and discretized (Figure 11). The spring-back effect was included in the analysis as a constraint trying to force the angle between two neighboring elements equal to π , i.e., trying to make the planks flat. Furthermore, four rigid links were modelled at the four corners of the overlap polygon, to prevent penetration of the planks. This was done while the position of the

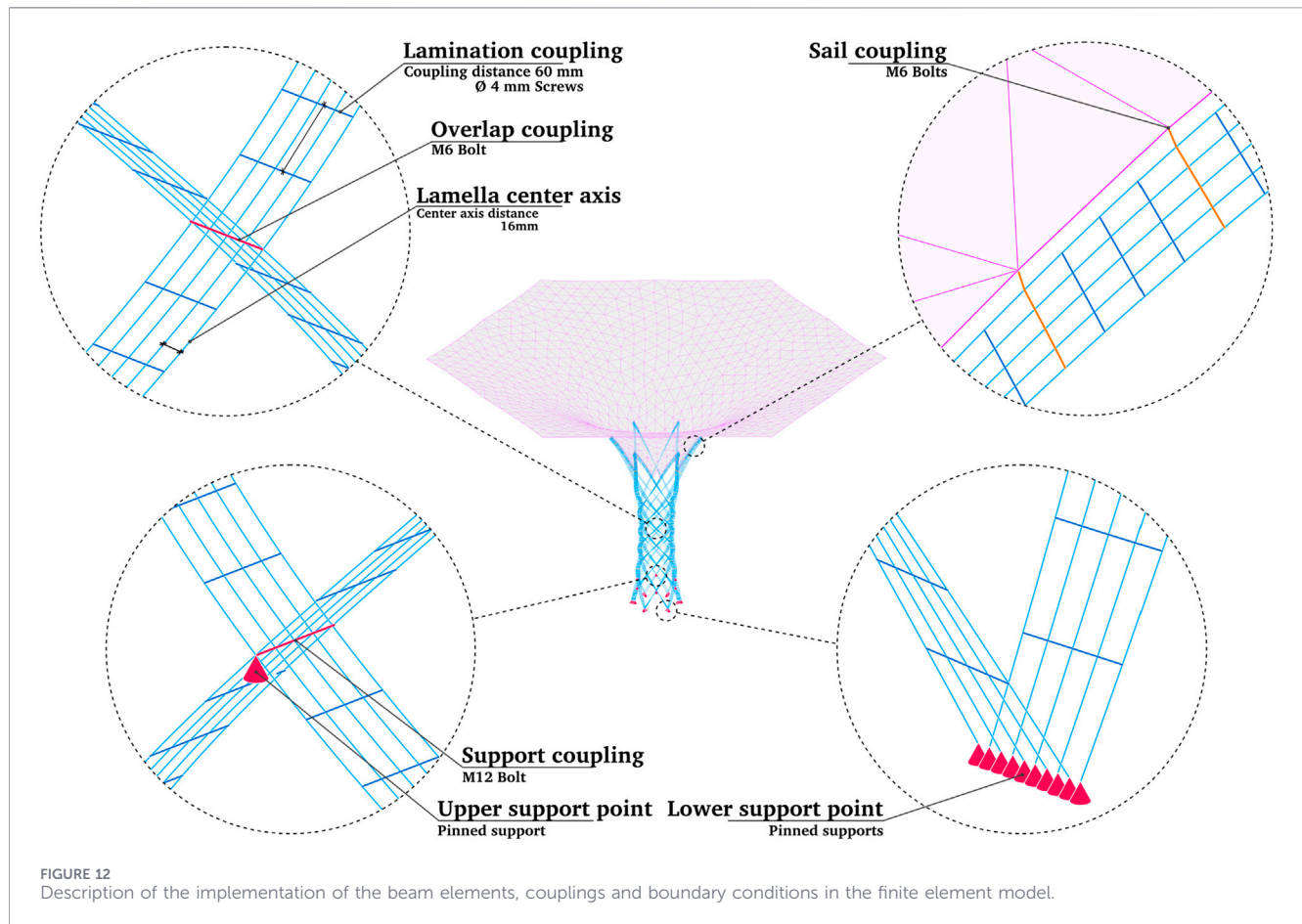


upper and lower coupling points were held fixed in space, as that would indeed be the case during construction.

In areas where the distance between the lamella overlaps was small, little spring back effect was seen, however, in the top part of the column, where the overlap distance increased, the resulting geometric discrepancy became significant, reaching a maximum deviation of roughly 33 mm.

3.5 Finite element implementation

For the finite-element (FE) analyses of the timber column, a modelling approach similar to that of Lara-Bocanegra et al. (2022) was adopted, implying modelling the lamellae using beam elements and continuously couple them with linear springs, in the position of the bolts at the overlaps and along each segment in the actual position of the screws (Figure 12 (top left)). A previous study has shown that for structures with compound beam sections, including the non-continuous layers in the numerical model only has limited influence on the structural capacity Andersson Largueche and La Magna (2024). The same is to be expected here on mechanical grounds.



The infill lamellae contribute to the stability of the compound, which depends on the bending stiffness of the assembly and on the restraint of the continuous lamellae. However, owing to its proportions, the column acts structurally as a shell rather than as a slender clamped column, so that the bending moments remain small and the lamellae are verified essentially for normal forces. And as the infill lamellae are discontinuous and cannot transfer normal forces along a beam axis they contribute only marginally to the structural verification. Only the continuous lamellae were therefore included in the model, and the material density assigned to them was scaled so that the total self-weight of the column corresponded to that of the built structure. The beam elements were subdivided manually to allow the definition of their local coordinate system according to the Darboux frame, ensuring the correct orientation of their cross-sections. The cross-section was defined with only half the real thickness, that is, 4 mm instead of 8 mm, to account for the weakened cross-section due to the incisions. This reduces the axial stiffness of the modelled lamellae to 50 % and their bending stiffness to 12.5 % of the gross section, and the assumption is therefore conservative with respect to the deformations, stresses and buckling capacity.

To ensure proper clamping conditions of the column, it was connected to the steel substructure at two levels: at the base, using a shoe-like connection, and at the third overlap point, where the usual M6 bolt joining the lamellae was replaced with an M12 bolt to secure the column to the steel frame. In the FE model, these support conditions were implemented by a pinned

support at the lower end of each lamella (Figure 12 (bottom right)) preventing all three translational degrees of freedom, and by an additional pinned support at the outermost lamella combined with a linear coupling spring with the stiffness of the M12 bolt at the third overlap point from the bottom (Figure 12 (bottom left)). The complete sail was included in the numerical model to allow for a more accurate load application and thus, a precise replication of the load transfer from the sail to the column. The sail was connected to the column by five steel brackets along each beam axis, which were modelled in the FE model as linear spring couplings positioned at the actual location of the brackets. A custom material was assigned to the sail elements, where the density was adjusted so that the weight of the modelled sail matched its actual total weight of 300 kg.

Experimental testing could not be accommodated within the scope and resources of the demonstrator project. The modeling approach adopted here therefore rests on the full-scale comparison reported by [Lara-Bocanegra et al. \(2022\)](#) for elastically bent timber grids with mechanically laminated compound sections.

3.5.1 Spring couplings

The couplings were defined according to the design recommendations in Eurocode 5 [Eur \(2014\)](#) for

TABLE 1 Definition of the spring couplings.

Coupling	d [mm]	ρ_m [kg/m ³]	k_x [kN/m]	$k_s = K_{ser}$ [kN/m]	$k_s = K_u$ [kN/m]	k_θ [kNm/rad]
Overlap	6	640	1.00E+06	4220	2810	0
Lamination	2x4	640	1.00E+06	5630	3750	0
Sail	6	640	1.00E+06	4220	2810	0
Support	12	640	1.00E+06	8440	5625	0

mechanically jointed beams. The slip modulus for dowel-type fasteners K_{ser} in N/mm per shear plane per fastener was calculated accordingly:

$$K_{ser} = \rho_m^{1.5} \frac{d}{23} \quad (12)$$

with d being the diameter of the fastener in mm and ρ_m being the mean density of the wood in kg/m³. The lamellae were connected at their overlaps with M6 bolts and continuously screwed together along their length with two rows of 4 mm screws spaced approximately 6 cm apart. The sail was attached to the column using M6 bolts, while the column itself was anchored to the steel substructure with M12 bolts. Consequently, four coupling types were defined (Table 1). The axial stiffness was assumed to be infinite, and the rotational stiffness was set to zero, as rotation around the spring axis was allowed. The values obtained from the expression above apply to the serviceability limit state. In accordance with EN 1995-1-1 Eur (2014) the ultimate limit state analyses were carried out with the reduced slip modulus $K_u = \frac{2}{3}K_{ser}$. Both values are given in Table 1 and were applied to the serviceability and ultimate limit state combinations respectively.

The expression for K_{ser} is calibrated on planar timber-to-timber joints, whereas the lamellae considered here are curved, elastically pre-stressed and weakened by incisions. Its transferability is supported by the work of Lara-Bocanegra et al. (2022), in which the same modelling approach, with slip moduli determined according to EN 1995-1-1 Eur (2014), was applied to elastically bent timber gridshells with mechanically laminated compound sections and compared against a full-scale test. Furthermore, the smallest radius of curvature along the beam axes is $R = 581$ mm (Figure 6), whereas an M6 fastener bears over a length of only $d = 6$ mm. Across that length the curved face departs from a plane by $d^2/(8R)$, which is less than 0.01 mm, far below the accuracy with which the holes can be drilled and the layers assembled. Thus the planar contact assumed in the expression for K_{ser} is realized locally, even though the member as a whole is curved. The incisions do not reduce the embedment of the fasteners either, since the lamination screws were positioned in the intact regions between successive incisions, where the full thickness of the lamella is available.

3.5.2 Definition of loading scenarios

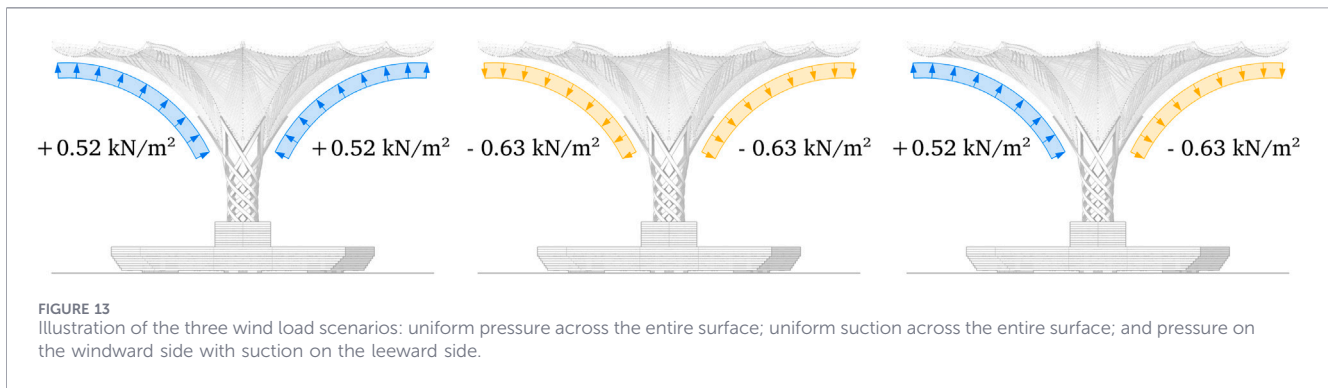
For the dimensioning of the timber column, wind and snow loads acting on the sail were considered. As the sail is not accessible, no live load was applied to it. The column itself,

however, is both accessible and climbable, therefore, a point load of 1 kN, representing the weight of a person climbing the column, was applied directly to it. Owing to the column's rotational symmetry, the number of analyzed point-load positions could be minimized. According to EN 1991-1-1 Eur (2005b), such imposed loads do not need to be combined with wind and snow loads and were therefore treated separately. As the site is located in an urban area, the terrain category, decisive for the determination of the wind load, was chosen as IV and as the structure is well below 16 m high, the characteristic velocity pressure could be determined as $q_p = 1.3 \cdot q_b$ with $q_b = 0.32$ kN/m² for wind zone 1 Eur (2005d). The pressure coefficients used to calculate the wind load on the sail were defined according to EN 1991-1-4 table 7. for duopitch canopies, where the structure was assumed to be completely open (i.e., $\theta = 0$) Eur (2005d). An average roof angle α of 30° was used to determine the force coefficient c_f , defined in accordance with Forster and Mollaert (2004) as the angle between the horizontal line and a straight line drawn from the perimeter to the apex of the sail. The final wind load values were calculated to +0.52 kN/m² and −0.63 kN/m². The robotic winding process used to fabricate the roof segments inherently produces a permeable structure, allowing for potential reduction of the wind loads. However, accurately determining this reduction factor would require wind tunnel testing, a process that is complex, time intensive and expensive. Furthermore, for this specific structure, such testing becomes even more challenging due to scaling issues associated with the dimension of the individual fiber strands, which may lead to inaccurate results. Consequently, the full wind load was applied in the structural assessment. Based on the structural symmetry, three wind load cases were defined: WLC1 with uniform pressure across the entire surface; WLC2 with uniform suction across the entire surface; and WLC3 with pressure on the windward side and suction on the leeward side (Figure 13).

According to EN 1991-1-3 Eur (2005c), the characteristic snow load s_k for the location, situated in snow zone 1 at an altitude of 123 m, was determined as 0.65 kN/m². To obtain the design snow load pressure, the shape coefficients for pitched roofs were applied. A detailed assessment of the roof angle was carried out, and the snow load was assigned accordingly.

As the wind load was shown to be the governing load case, no load combinations with snow load as primary load case were analyzed. The following load combinations were used in the structural analyses:

- LC101 (SLS) – 1.0 * DL + 1.0 * WLC1
- LC102 (SLS) – 1.0 * DL + 1.0 * WLC1 + 0.5 * SL
- LC103 (SLS) – 1.0 * DL + 1.0 * WLC2



- $LC104(SLS) - 1.0 * DL + 1.0 * WLC2 + 0.5 * SL$
- $LC105(SLS) - 1.0 * DL + 1.0 * WLC3$
- $LC106(SLS) - 1.0 * DL + 1.0 * WLC3 + 0.5 * SL$
- $LC1001(ULS) - 1.35 * DL + 1.5 * WLC1$
- $LC1002(ULS) - 1.35 * DL + 1.5 * WLC1 + 0.75 * SL$
- $LC1003(ULS) - 1.35 * DL + 1.5 * WLC2$
- $LC1004(ULS) - 1.35 * DL + 1.5 * WLC2 + 0.75 * SL$
- $LC1005(ULS) - 1.35 * DL + 1.5 * WLC3$
- $LC1006(ULS) - 1.35 * DL + 1.5 * WLC3 + 0.75 * SL$

3.5.3 Internal stresses due to elastic bending

Elastic bending generates internal stresses that should be considered when evaluating the structural capacity of an element. However, previous studies have investigated the effect of residual stresses caused by form-giving deformations in elastic structures and found their impact on structural capacity to be limited [La Magna and Knippers \(2017\)](#), [Lefevre et al. \(2015\)](#), [Mesnil et al. \(2015\)](#). More recent research has further shown that specifically for timber structures initial bending stresses decrease rapidly, indicating an “important structural reserve recovery” [Lara-Bocanegra et al. \(2018\)](#). For this reason, an initial form-finding step was omitted from the structural analyses in this work.

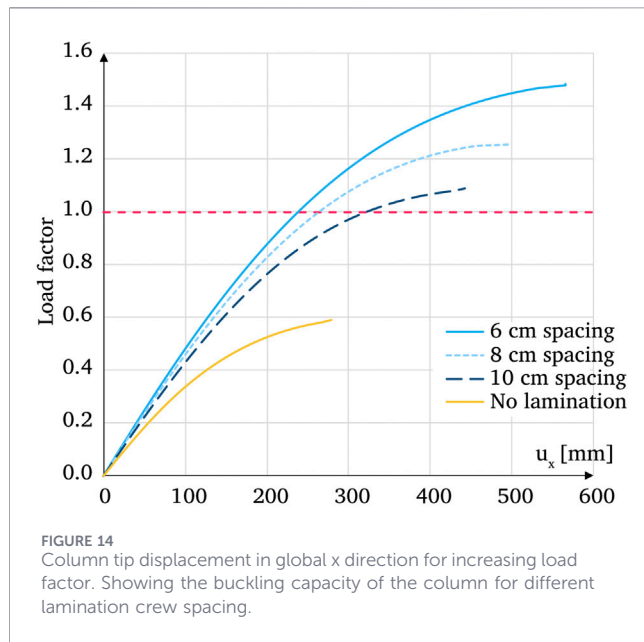
3.5.4 Influence of the infill lamellae

To examine the assumption made in [Section 3.5](#), a second model was built in which the infill lamellae were included as discrete beam elements, coupled to the adjacent continuous layers by the same lamination springs at the actual positions of the screws. They were released axially at their ends, so that no normal force is transferred between successive pieces. This reflects the controlled length tolerance with which the infill elements are inserted ([Section 3.2](#)), which means that end bearing cannot be assumed. The tolerance does not affect their remaining functions: the contact between the layers acts through the thickness of the compound, and each piece is connected to the adjacent continuous lamellae by the lamination screws along its length, so that the transfer of interlayer shear and the restraint against local buckling are retained irrespective of the fit at the ends. In both models the total self-weight of the column corresponds to that of the built

structure, so that the comparison reflects the structural contribution of the infill layers alone. The comparison was carried out for two limiting configurations of the lamination screws, chosen so that any spacing selected subsequently lies between them. The lower bound is the minimum connection that fixes the infill elements in position, i.e., two screws per element, one at each end. The upper bound is a spacing of 20 mm, the closest permitted spacing by EN 1995-1-1 [Eur \(2014\)](#) for the 4 mm fastener diameter. Including the infill layers raises the buckling capacity of the column from 113 % to 135 % of the design load in the first configuration and from 192 % to 238 % in the second, while the utilization of the lamellae gets reduced from 101 % to 98 % and from 86 % to 77 % respectively. The comparison therefore confirms the expectation set out in [Section 3.5](#). The influence of the infill elements is concentrated in the stability and stiffness of the compound and is marginal for the verification of the lamellae, which is consistent with the assumed load-bearing behavior. Their contribution to the stability is retained as a structural reserve rather than relied upon in the design. All analyses reported in the following sections were accordingly carried out with the simplified model, and are on the safe side in this respect.

3.5.5 Screw distance

The required lamination-screw distance was assessed in a buckling analysis for varying screw spacing ([Figure 14](#)). The analysis was performed in a geometrically non-linear limit load routine accounting for large deformations using the commercial FE-software [SOFiSTiK AG \(2025\)](#). The buckling capacity was determined for the ULS combination 1006 by measuring the column tip displacement while gradually increasing the applied load until the analysis diverged, indicating the limit load at which the numerical model became unstable. Without coupling the layers together, the column can carry only about 60 % of its design load. When the layers are connected the load-bearing capacity increases significantly. The capacity reaches approximately 110 % and 125 % for a screw spacing of 10 cm and 8 cm respectively. However, due to the complex overall structural behavior and capacity, additional reserves and redundancy in the structure was desired, therefore a screw spacing of 6 cm, corresponding to a buckling capacity of roughly 145 %, was selected.



3.5.6 Evaluation of FE-analysis

In accordance with Eurocode 1990 [Eur \(2005a\)](#) the horizontal displacement of the column top was calculated for the serviceability limit state combinations. The resulting maximum displacement was 157 mm ([Figure 15](#) (left)), for load combination LC 105, which corresponds to roughly $h/20$. Eurocode does not provide explicit regulations concerning the maximum permissible horizontal sway of clamped, umbrella-like structures, unlike the clearly defined limits on vertical deflections for beams. Instead, it requires that structural

deformations remain within bounds that do not compromise the structure's appearance, durability, nor usability. Although the maximum displacement of $h/20$ would generally be considered large, it was still regarded being within an allowable range, knowing that the modeling approach adapted in the FE-analysis underestimates the structural stiffness [Andersson Largueche and La Magna \(2024\)](#) and that the calculations were carried out using the full wind load, which could have potentially been reduced.

Umbrella-like structures are generally considered structurally inefficient, as their load-bearing behavior induces high clamping moments at the support. However, due to the column's proportions, with a height-to-width ratio of approximately 5, it cannot be classified as slender and therefore behaves structurally as a shell rather than a column clamped at a single point.

This is confirmed by the FE-analysis as bending moments are close to zero ([Figure 15](#) (mid right)) and only normal forces are acting in the lamellae ([Figure 15](#) (mid left)). This simplified the required structural checks as the column only needed to fulfill the two following inequalities at every point.

$$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{\gamma_M} > = \frac{F_c}{A}, \quad (13)$$

$$f_{t,0,d} = f_{t,0,k} * \frac{k_{mod}}{\gamma_M} > = \frac{F_t}{A}, \quad (14)$$

where $\gamma_M = 1.3$ and $k_{mod} = 0.9$ throughout all combinations, as the wind load has the shortest duration, in accordance with EN 1995-1-1 [Eur \(2014\)](#).

The material properties of the ash lamellae were provided by the supplier and can be seen in [Table 2](#).

The load combination generating the highest normal forces was LC1006. Here, the asymmetric wind load generates a rotational moment

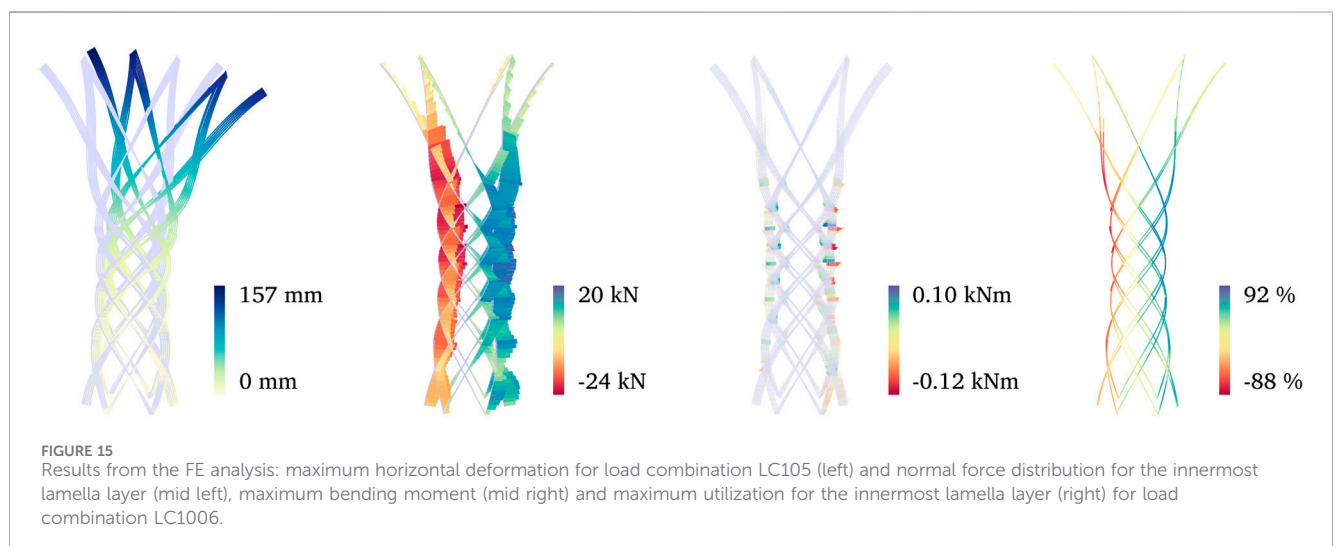


TABLE 2 Material properties.

Material	E_0 [N/mm ²]	E_{90} [N/mm ²]	ρ_m [kg/m ³]	$f_{t,0,k}$ [N/mm ²]	$f_{c,0,k}$ [N/mm ²]
Ash	14800	980	640	29	29.7

at the column head, which is resisted by compression and tension forces within the column. Due to the discrete coupling of the lamella layers, these forces are not evenly distributed across the compound. Additionally, as the lever arm is shorter for the inner layers, these lamellae experience the highest loads and were therefore decisive for the structural assessment of the entire column structure. The utilization of the inner lamella layer reached a maximum local value of 92 % (Figure 15 (right)).

4 Conclusion

The project demonstrates the potential of elastically bent compound beam structures as primary load-bearing elements, while highlighting how guided bending can significantly expand the feasible design space of such structural systems. The introduced incision patterns proved essential for achieving the required spatial geometry, as unmodified lamellae of identical thickness could not be bent around the column without structural failure.

Although the target geometry could theoretically have been achieved using thinner lamellae, similarly as in the production of glued laminated timber or laminated veneer lumber products. However, such an approach would have reduced the capacity of the mechanically laminated compound, as their structural performance strongly depends on the stiffness and load-transfer capacity of the metallic fasteners, and as the size of the metallic fasteners scale with the material thickness, the use of thicker lamellae was structurally advantageous.

In the current prototype, only the orientation of the incisions was theoretically derived, while their width, depth, and spacing were predefined and kept constant along each lamella. Ideally, however, these parameters should be adapted to the target geometry, as the mechanical influence of the incisions is directly related to the local curvature and torsion. This refinement is part of ongoing research into controlled bending (as opposed to guided bending). Furthermore, the incisions were not explicitly represented in the FE model. Instead, their influence was approximated through a simplified reduction of the lamella cross-section. Consequently, the precise effect of the incisions on the structural behaviour and load-bearing capacity remains to be quantified, and an appropriate numerical modelling strategy still needs to be developed.

More generally, the structural assessment presented in this work is based on numerical analysis only. The demonstrator was not instrumented, no load test was performed and the built geometry was not surveyed. An experimental validation of the elastically bent compound assembly therefore remains outstanding, and would also provide the basis against which such a modelling strategy could be validated and calibrated.

Although the fabrication process can be carried out using relatively simple machinery, it remains highly time-consuming. In the present project, nearly ten times more infill pieces than continuous lamellae were produced, making manual handling and assembly considerably more labor-intensive. Ongoing research is therefore focused on evaluating the structural contribution of the infill elements and on developing strategies to reduce or eliminate their necessity, thereby improving fabrication efficiency and scalability. In addition, the introduction of the guiding incisions significantly increased fabrication time. However, once the exact size and placement of the incisions are calibrated, the required number of incisions per lamella is expected to decrease substantially, further reducing overall production effort.

Overall, the presented approach demonstrates that guided bending can serve as a viable design and construction strategy for structurally efficient and informed timber systems, establishing a direct link between geometric design, fabrication logic, and structural performance.

Data availability statement

The original contributions presented in the study are included in the article/supplementary material, further inquiries can be directed to the corresponding author.

Author contributions

DA: Investigation, Conceptualization, Writing – original draft, Visualization, Project administration, Formal Analysis, Methodology. RL: Project administration, Conceptualization, Supervision, Funding acquisition, Writing – review and editing.

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Conflict of interest

The author(s) declared that this work was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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