Metropol Parasol in the Plaza de la Encarnación, Seville

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The planning for the redevelopment of the "Plaza de la Encarnación" (fig. 1) in the Old Town of Seville, Spain, began in 2004 when the Berlin-based architect Jürgen Mayer H. won the design contest together with engineers from Arup Berlin. The goal was to redevelop the square, as well as to connect the surrounding city districts, which were separated by a main road (fig. 2). The project included a museum for the Roman mosaics (fig. 3), which had been excavated about 5 m below the square, and shops and market stalls at street level, as well as a new design for the square at an elevation of approx. 5 m (reinforced concrete construction, fig. 3d to fig. 3g). The company Sacyr Vallehermoso, Madrid, acted as the general contractor.

Fig. 1 Aerial photograph of the Plaza de la Encarnación, Seville

Fig. 2 City plan (left) - design contest (right hand side)

Fig. 3 Excavations of Roman mosaics

Sunshade for the new meeting place to improve city life and tourism is provided by a wooden, tree-like construction, which is up to 28 m high and provides protection from the sun (fig. 4). This structure consists of 6 interlocking trees or "mushrooms", *parasoles* in Spanish, hence the project name "Metropol Parasol".
A restaurant, cafe and walkways under the parasols allow visitors to experience the free-form design up close and to enjoy the spectacular views of Seville's landmarked old town.

Fig. 4 View of the Metropol Parasol in the Plaza de la Encarnación, Seville

The parasol design is based on a framework construction or lattice consisting of LVL panels in an orthogonal grid measuring 1.50 m x 1.50 m. The timber structure is 120m long, has a width of about 45m and a maximum high of 24 m and is braced horizontally by diagonal steel bars.
Steel structures were used for the 21.50 m high platform of the restaurant and cafe area, as well as for the bridge over the main road (fig. 5). The restaurant and cafe area is covered by the timber lattice sealed with bitumen - this gives visitors the impression that they are inside a seashell (fig. 1, fig. 6).

Fig. 5 Steel load-bearing platform of the restaurant and cafe area (above)
Steel structure crossing the main road under traffic (below)

Fig. 6 Seashell design

1 The timber-frame mushroom structure

Geometry

The dimensions of the trunks of the parasols vary; they have a maximum diameter of 15 m. The trunks are made from glued LVL (Kerto-Q)-panels with a minimum thickness of 140 mm and are hollow on the inside allowing integration of the steel emergency staircase and reinforced concrete elevator shafts (fig. 7). The inclination of the trunks to the vertical also varies.

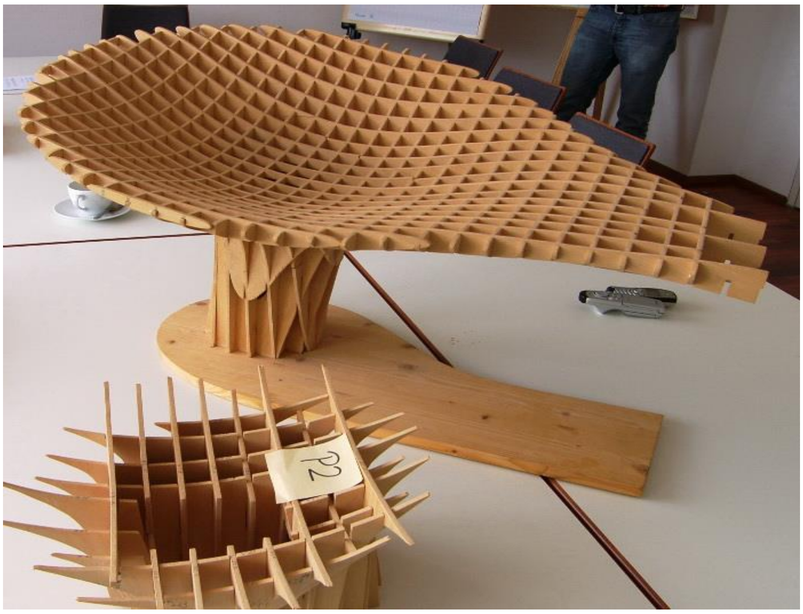
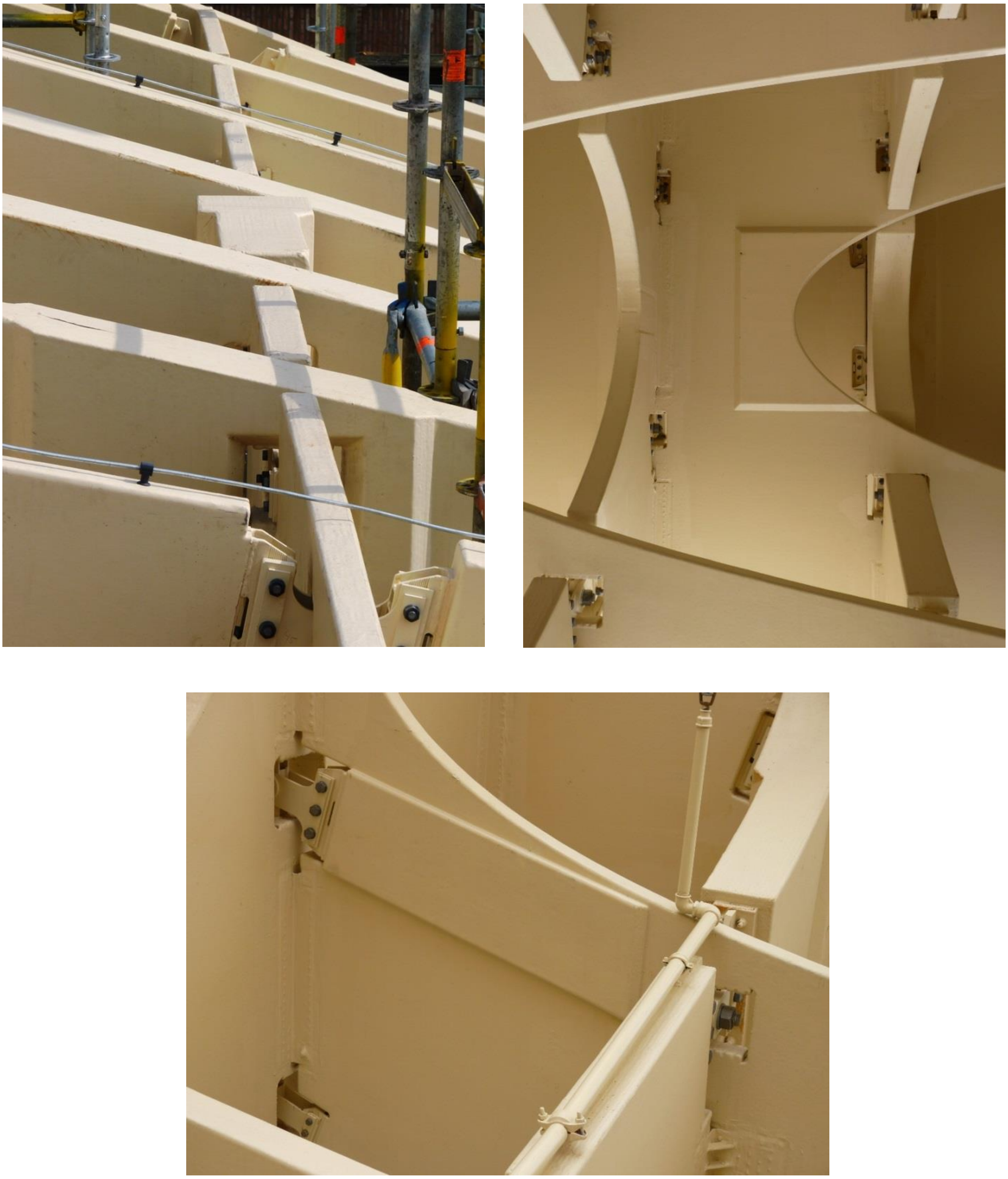


Fig. 7 Model of a caps and the cross section of a trunk

The elements of the parasol caps are between 1.50 m and 16.5 m long, with widths between 68 mm and 311 mm (fig. 8) and a maximum depth of about 3 m.



<u>Fig. 8</u> Different element widths, Widening of the web of the beam

Fig. 9 Component system of parasol caps

Basically the elements should be constructed with a span across two bays of the timber lattice jointed centrally on the continuous element (fig. 9) - similar to the previous Zollinger design. For reasons of geometry, there are many elements that pass through three or four lattice fields. The large cross-sections of the glued elements, vacuum pressed multiple times, are designed with reduced strength properties in accordance with an expertise by the MPA Stuttgart. The approximately 3.400 elements, with a gross total volume of 3.500 m^3, were produced by Finnforest Merk in Aichach (fig. 10).
The first natural frequencies of the walkable roof shell lie within the range of 1 to 2 Hz.

Fig. 10 Completion of the timber elements

Connections meet high standards

The connection principle of the more than 3.000 connection nodes that are found in the cap alone, are intended to offer easy installation, high load-carrying capacity, no protruding steel elements (ease of transport) and the possibility to compensate tolerances in three directions. In particular this concerns the 3 steel connections moment resisting, shear resisting and diagonal connections.
The 11.000 moment resisting connections, distributed over the top and bottom of the diaphragm elements (fig. 11) represent a special, standardized connection with inhibited torsional rotation. This can be closed quickly via a bolt during construction. The tabs are connected to the flange using a tooth-type interlocking with 3.5 mm separation and pre-tightened bolts type 10.9. The high connecting forces in the steel flanges are transferred to the timber by glued-in rods (fig. 12). The transfer of shear forces is transmitted through 12.000 large, individual steel angled plates (fig. 13); they were nailed subsequently on-site to the LVL (Kerto)-panels and can therefore easily adjust construction tolerances. The 2.000 diagonal bracing rods (fig. 14) are integrated into the angled steel connector plates.
The steel connections in the nodes are coated with a corrosion resistant paint and an UV-resistant powder coating. Thanks to its pronounced ductility and good adhesion to timber substrates, the spray coating can bridge possible cracks in the timber and is permeable to water vapour: with an s_d-value of about 1.8 m, it has about the same permeability to water vapour as 36 mm of solid timber or 12 mm of laminated veneer lumber.

Fig. 11 Moment connection

Fig. 12 Glued-in rods

Fig. 13 Angled connector plate to transfer shear forces

Fig. 14 Angled connector plate with connection for diagonal bracing

The threaded rods were glued into the panels with epoxy resin under normal workshop conditions. Subsequently, controlled heating in an unstressed state above 55°C enabled the glass transition temperature to be increased to over 80°C in a safe and controlled manner to withstand climates local conditions (another report of MPA Stuttgart).

A total of 700 tons of steel were fitted with connection weights between 3 kg and 70 kg (capacity: N_{Rd}= - 1363 kN (compression), N_{Rd}= 1251 kN (tension).

2 Special features of design and verification

Iterative calculation of internal forces

After all boundary conditions for the final calculation of the structure were determined, Finnforest was able to furnish a huge matrix that defined a connection type and the corresponding weight of the connection details for all angles of timber strut, any possible timber thickness, every grain angle and force-grain angles.
All other loads, such as the weight of the visitors, wind-loads, but also shrinkage and swelling of structural parts due to changes in temperature and moisture were determined by Arup essentially on the basis of EN 1991 and entered into the computer. Their three dimensional structural models provided the internal forces. The model structure was generated as a truss system for the caps (circumference of approx. 2 trunks) and partially with FE plate elements for the trunks. At the beginning of the project, the support structure could not be developed as a complete system in the computer.

The load bearing capacity of timber beams was then prooved by Harrer Ingenieure for these calculated loads and their dimensions were increased as necessary. Connections with load bearing capacity that were too small were replaced by larger connections.

Component and detailed verifications

For the verification of individual elements and details, Finnforest combined the global geometry data of the architects J. Mayer H. (outline) and the Arup engineers (static axis and inclination, node numbers) with data from their own element planning (initial and final component nodes, grain direction) and systemic detailed geometry (e.g. box distance from the edge of the component), and conducted a pre-selection of the connections on the basis of the internal forces provided by Arup. Then verification of the cross-sections and connections was performed largely by Harrer Engineers. Due to the large number of elements and compounds, self-programmed evaluation routines were used for this purpose.
In event of excessive utilization, Finnforest and Harrer Ingenieure performed adjustments to the detailed geometry and choice, until all verifications could be completed by Harrer Ingenieure.

Basis of verification of engineered timber panels

The characteristic strength properties specified in the German general technical approval for timber panels refers, generally, to the total cross-section under single plane loading. Use of these values, with combined state of stress for panels or pates, required further studies of interactions for localised, simultaneously acting tensions from multi-axial stress. As the necessary interaction rules are not found in the regulations, they were defined as design rules in accordance with the proposal of a study from TU Munich /1/.

/1/ KREUZINGER, H.; SCHOLZ, A.
Nachweis in Grenzzuständen der Tragfähigkeit bei Platten und Scheiben aus Holz und Holzwerkstoffen unter Spannungskombinationen.
Fraunhofer IRB Verlag 1999

Design of the trunks

Whilst the two trunks of parasol P3 and P4 under the cafe are modelled as strut and tie constructions, the other four trunks of P1, P2, P5, P6 are calculated in the model using FEM as panels. The verification of stresses from the FEM results that are provided as FEM internal forces, is performed on the basis of the aforementioned report /1/.

The rule of interaction used here is

$$\frac{\sigma_y}{f_y} \le 1 \quad \text{and} \quad \frac{\sigma_x}{f_x} + \frac{\tau_{xy}}{f_{v,xy}} \le 1 \qquad (1)$$

with

σ_y , σ_x Longitudinal stresses parallel and perpendicular to the grain from normal forces n_y, n_x in the panels and bending moments m_y, m_x

τ_{xy} Shear stress from shear force n_{xy} and torsional moment m_{xy}

This verification is provided in detail for the calculation according to EN 1995-1-1 as follows (see also fig. 15):

$$\frac{\frac{\gamma_F \cdot n_y}{d}}{f_{c,0,d}\ (f_{t,0,d})} + \frac{\frac{\gamma_F \cdot m_y}{d^2/6}}{f_{m,0,flat,d}} \le 1 \quad (1a) \qquad \text{and} \qquad \frac{\frac{\gamma_F \cdot n_x}{d}}{f_{c,90,edge,d}\ (f_{t,90,edge,d})} + \frac{\frac{\gamma_F \cdot m_x}{d^2/6}}{f_{m,90,flat,d}} + \frac{\frac{\gamma_F \cdot n_{xy}}{d} + \frac{\gamma_F \cdot m_{xy}}{0,3 \cdot d^2}}{f_{v,0,edge,d}} \le 1 \quad (1b)$$

with the design values

$f_{c,0,d}$, $f_{t,0,d}$ Compressive and tensile strength parallel to the grain

$f_{c,90,edge,d}$, $f_{t,90,edge,d}$ In-plane compressive and tensile strength perpendicular to the grain

$f_{m,0,flat,d}$ Panel bending strength parallel to the grain

$f_{m,90,flat,d}$ Panel bending strength perpendicular to the grain

$f_{v,0,edge,d}$ Panel shear strength

and

γ_F Safety factor

d Cross section thickness/depth

Internal forces and cross-section widths apply per metre.

This longitudinal forces and shear stresses together are greatest in the outer layer of the panel.

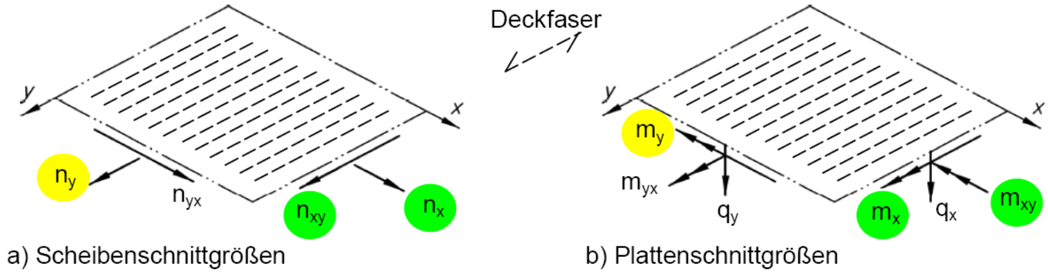


Fig. 15 Definition of internal forces of timber-based panels depending on the grain direction

Of the up to maximal 6 possible independent stress vectors in the three dimensional state of stress of an element, 3 tension vectors are used in this Eq. (1a) and (1b).

The 3 tension vectors not considered further here, are considered formally in the verification of shear stresses from out-of-plane loading Eq. (2), and in-plane-loading according to Eq. (3). In this case, these two verifications can be neglected because there is essentially an in-plane loading.

$$\frac{\tau_{yz}}{f_{v,yz}} \leq 1 \tag{2}$$

$$\frac{\sigma_x}{f_{Querzug/-druck}} + \frac{\sigma_z}{f_{Querzug/-druck}} + \frac{\tau_{xz}}{f_{Rollschub}} \leq 1 \tag{3}$$

For trunk P5 alone, it was necessary to consider some 10.000 finite elements with various maximum and minimum internal forces in the middle of an element. Both equations (1a) and (1b) needed to be performed and the results evaluated for all these. This is only possible in a very limited scope using tables; therefore a visual representation of the results was selected allowing areas with high utilization and/or singular points to be identified and evaluated quickly. Since no commercial program is available for this, it was necessary to develop a computer program to handle the task at hand.

Fig. 16 details the trunk-panel in axis x06 of trunk P1 together with the load capacity. To visually create the structure in EXCEL, first the node coordinates need to be entered, neighbouring nodes on edges connected with each other and then the lines outputted graphically. At the same time, these graphically generated triangles and squares, so-called drawing objects, were assigned program functions. This allows individual results to be retrieved quickly per mouse click so that, e.g. areas of high capacity can be better assessed. To achieve an overall picture of capacity distribution, the individual elements were filled with scaled gray tones according to their percentage utilization values.

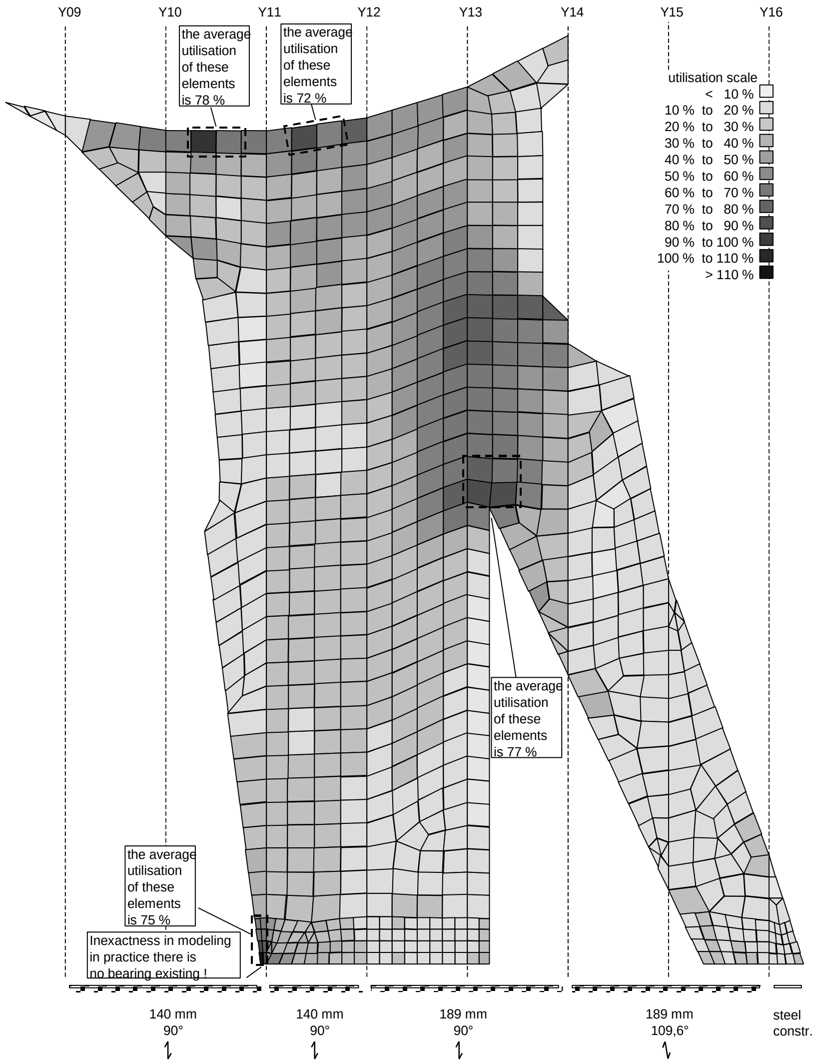


Fig. 16 Structure of the computational model of a trunk panel (example)

Connection elements between intersecting trunk panels were not defined within the model, meaning that cutting forces on these cut edges could not be outputted. These however are absolutely necessary when measuring the connections of the trunk panels. Given the large number of approximately 80 connecting joints per trunk, the self-developed program was also used here. For the verification of the fasteners this leads to the required horizontal in-plane normal forces n_x and the in-plane shear forces n_{xy} of neighbouring elements within a connection joint to be required in same order as in the building. Where appropriate, the internal forces must be transformed back in an intermediate step beforehand, if the grain direction of the trunk panel is not vertical.

Design of the caps

The over 3.000 cap nodes were combined in the model via truss elements forming a three dimensional structure, and the critical internal forces resulting from the different model calculations were delivered to their ends. The calculated internal forces needed to be split into individual components which followed the actual distribution of forces within the node. The node must be resolved so that all force components for the various types of connections lie within the node - without upsetting the equilibrium of forces.

Each node is designed for the extremal internal forces from different load combinations. There is also an additional retention level of approx. 15% for various aspects (e.g. strain during the assembly, any unwanted load redistribution when removing the scaffolding or temporary supports).
To keep track of the huge amount of data from the variety of loading case combinations, the loads are superimposed and simplified with a single partial safety factor of 1.5 on the load-side. The k_{mod} was harmonized at 0.8 on the resistance-side. This conservative simplification results in an additional reserve.

The 2 to 3 mm thick sprayed-on polyurethane-2K-coating with subsequent beige top coating (fig. 17) for UV protection allows the use of the service class 2 for the timber construction design. Verification of the critical internal forces was performed with the load duration “medium”.

Fig. 17 Sprayed-on Polyurethane-2K-coating with beige top coating

As already mentioned, joints have moment connectors that transfer moments as a couple of forces as well as nailed steel angle plates that transfer shear forces. The nailed steel angle plates also have a bracing function. Depending on the load, the steel angle plates are also reinforced to stabilize the timber components. If reinforcing diagonals also meet within a node, their horizontal force components are transmitted through the steel angle plates to the continuous timber parts, as well as parts fitted with the more rigid moment connectors. The vertical force components can only be transferred to the continuous timber parts because the system node is found there.

Due to the irregular geometry of the structure, side effects always occur for the different connection types when resolving forces. Moment connections, whose force couple are not parallel due to construction factors, always generate additional vertical forces in the steel angle plate. On the other hand, moment connections always have force components from longitudinal and lateral forces, as well as diagonal forces (if applicable), due to their higher stiffness compared to those of from nailed angled plates.

Fig. 18 shows an example of resolution of forces for a transmitted moment.

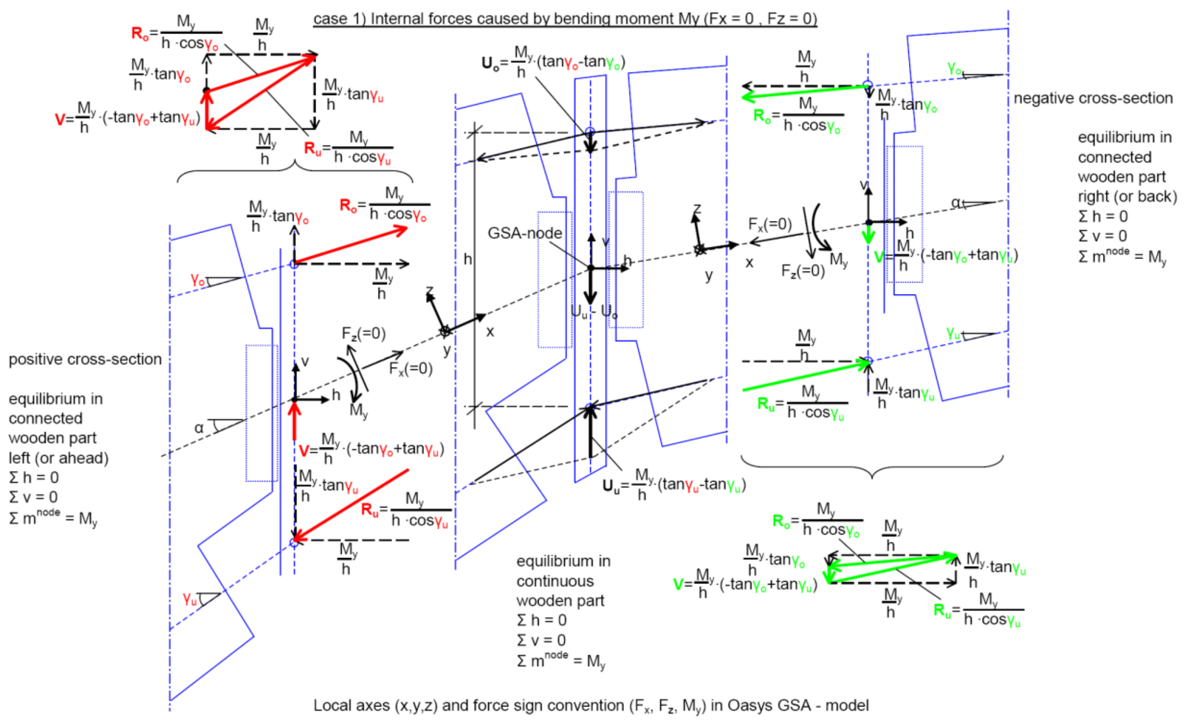


Fig. 18 View of a beam joint: resolution of forces for different connection types in the node (here: just moment)

The two variants of construction-related anchoring the diagonal rods at the nodes on site pose a particular difficulty. If diagonals are mainly anchored on the side facing away from the node, then in individual cases the anchors are mounted on the nearest side depending on construction, which leads to other rules of superposition. Because diagonals often don't meet at the intersection point due to the varying depth of the timber elements, additional fittings are required, which compensate for the resulting eccentricities (see also fig. 19).

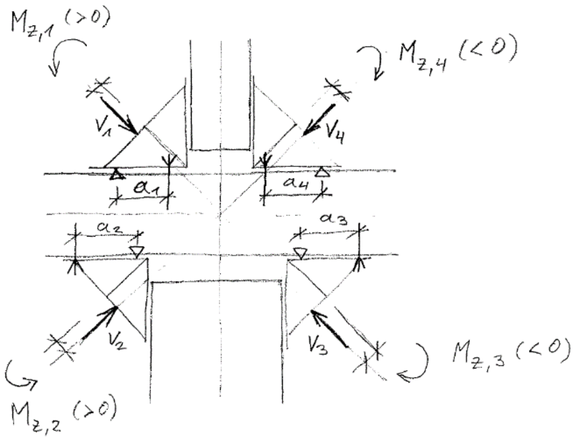


Fig. 19 Top view of girder:
additional fittings of unsymmetrical diagonal connections

Generally each connection type had to be as small as possible to prevent them obstructing each other and to take account of the weight-saving requirements. The transfer of shear forces via steel angle plates also reduce the negative influences of eccentricities in the connection. Here, too, a loop-based program was required for the iteration. Moments connections, which only receive subject to compressive loads, could be fitted with a significantly lower number of glued-in fully threaded rods as connections under tensile load. This reduced construction costs as well as weight.

An additional data management challenge is posed by the very high area loads, for which the component verifications must be performed. This leads to bending moments with values of up to about 2.000 kNm, similar to those more commonly found in bridge construction.

The connections of each node had to be examined and dimensioned individually. No two nodes are the same!

3 Logistics and installation

About 50 of the 100 truck-loads from Aichach to Seville corresponded to standard dimensions, the other half were special transports due to oversize or high height. The elements were coated with polyurethane in a temporary storage facility in Seville. First of all, the trunks of the parasols were assembled (fig. 20). Then the construction of the scaffolds for the caps followed (fig. 21). Afterwards the timber components were fixed in place, measured and connected (fig. 22). Platform scaffolds were created to assemble the free parts of the cap caps.
The technicians sometimes worked suspended from above, without a solid footing, in temperatures of up to 45°C.
When the load was transfered onto the nodes by removing the scaffolding the resulting deformation of the tooth-type interlocking was monitored to ensure an even distribution of the initial tension in the screws. The dimensional tolerances during construction were ± 1 cm; the permissable tolerance at the moment connection were ± 7 mm per connector plate.

On March 4, 2011, the last support was raised in the presence of project partners and politicians.

Fig. 20 Installing the trunks

Fig. 21 Construction of the scaffolding for the cap assembly

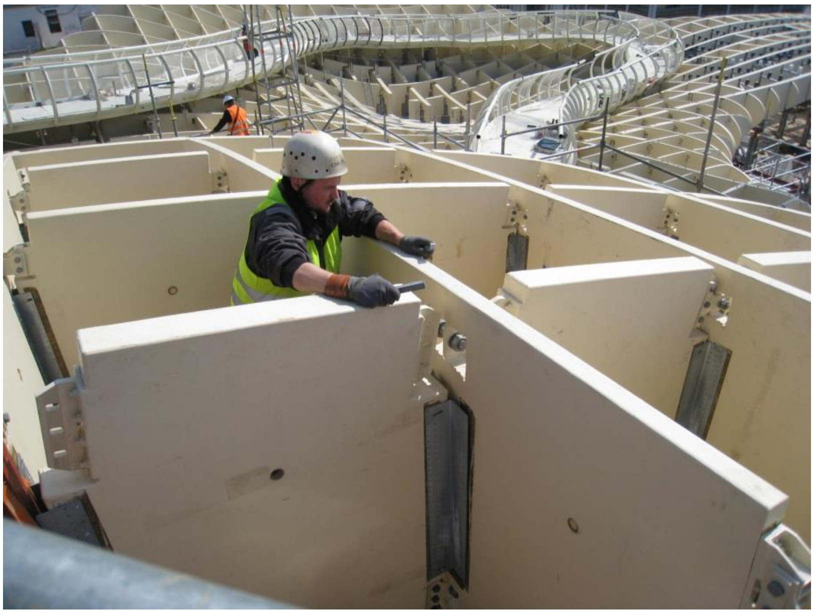


Fig. 22 Connecting the timber beams

4 Summary: technical highlight

The Metropol Parasol in the Plaza de la Encarnación, Seville (fig. 23), is an architectural and engineering highlight that illustrates the possibilities of modern timber construction. This timber construction milestone was built between June 2008 and March 2011 and includes some previously described, trend-setting solutions in timber construction.

Fig. 23 Architectural and engineering highlight

In particular, I would like to thank Mr Kunz of Finnforest Merk for the trust placed in us, my staff who undertook the hard work and my wife who supported me with her interpreting skills.

Finally on a personal note, please allow me to make two observations without wanting to diminish the size of the project and the achievements of those involved:

Lattice systems (fig. 24) are more commonly used in steel constructions. If such systems are used in timber constructions, larger grid dimensions should be used to reduce the number of steel connections.
And: With Europe's second largest cathedral, Calatrava's bridges or Easter Week Seville is definitely worth a visit.

Fig. 24 Lattice system