

# Cherry Blossom Palace

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**ABSTRACT:** The Cherry Blossom Palace, situated in the Jerte Valley, has been designed by the Spanish team of architects AMID [cero9]. The building is designed to hold an annual festivity related to the cherry tree blossom as well as for other social uses. It is basically made up of a concrete basement and a concrete ring which emerges from the ground and supports a metallic shell. The result is a great covered space drilled through by skylights which houses a multi purpose auditorium. The structure cap has been designed using tube profiles defining a triangulated geometry, using a second order analysis method, and achieving a great optimization of the structural material, as well as simplifying the joints.

## 1 INTRODUCTION

### 1.1 *Location*

The Jerte Valley is situated in Extremadura, Spain. It lies between two mountain ranges, being the Calvitero its highest point (its height is 2.410m) These two mountain ranges are the natural limits of the valley, and belong to the Sistema Central.

The main attraction of the Jerte Valley are the cherry trees which practically cover the whole area. In recent years, the Jerte cherry flowering has become a tourist attraction when the blossom becomes a natural spectacle that briefly transforms the landscape. A continuous blanket of white flowers covers the valley in early spring, attracting visitors amongst cherry trees in flower.



Figure 2. The Jerte Valley in blossom.

### 1.2 *Architects*

The architects of the Cherry Blossom Palace are Cristina Díaz Moreno and Efrén García Grinda. Their study, named AMID [cero9], is located in Madrid.

### 1.3 *The Cherry Blossom Festival*

Locals organize annual festivities when the trees are in blossom to celebrate the flowering season. The eleven townships take it in turns to hold this festivity. This event coincides with the main influx of visitors. The Cherry Blossom Festival is a traditional procession, a celebration of the fruits of the Earth.

### 1.4 *Architectural design idea*

For the Cherry Blossom Festival, the architects propose the construction of a modern chapel; a building

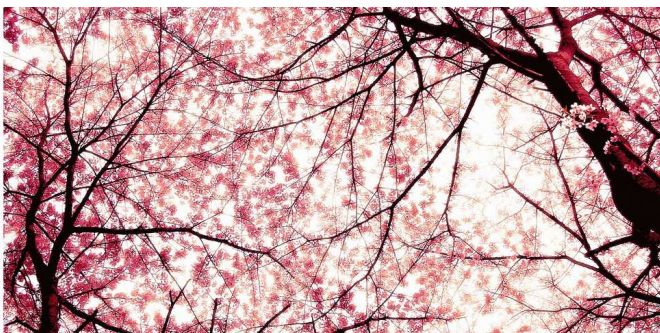


Figure 1. Cherry trees in blossom.

that creates an intense bond with the landscape through its presence, position, volume and material. The architects propose a hybrid between a cave drilled through by big holes where light enters and an interior space defined by its structure and light with biological and natural reminiscences, placed in the midst of a hand made landscape of terraces and cherry trees; a building that can remain closed for months.



Figure 3. The Cherry Blossom Palace in the Jerte Valley (photo collage)

It is a floating item amidst a landscape of cherry trees, oaks, stone terraces and fog. In this valley, where the colors change in the course of the year, they propose the construction of a building with a two-toned skin, tessellated, tense and continuous, which will overcome the fog with its sheen, contrasted with the chromatic changes in the landscape, from green to white via red.



Figure 4. Local population at the Cherry Blossom Festival (photo collage)

## 2 ARCHITECTURAL PROGRAM

### 2.1 Space definition

The building space is defined by a family of five elements:

- A concrete ring, facing the interior with a continuous convex surface
- A steel shell and mixed membrane shell, covering the concrete ring like a skirt
- A basement, composed of a series of underground rooms defined by a curved concrete wall

- Concrete ramps and stairs, which link the basement and the amphitheatre to the surrounding spaces
- A lightweight roof, fitted to a solid infrastructure built primarily in concrete. It is composed of a crinoline fabric made of slender steel rods in rhomboidal patterns, and a triple cross-ventilated membrane stretched across the metal structure.

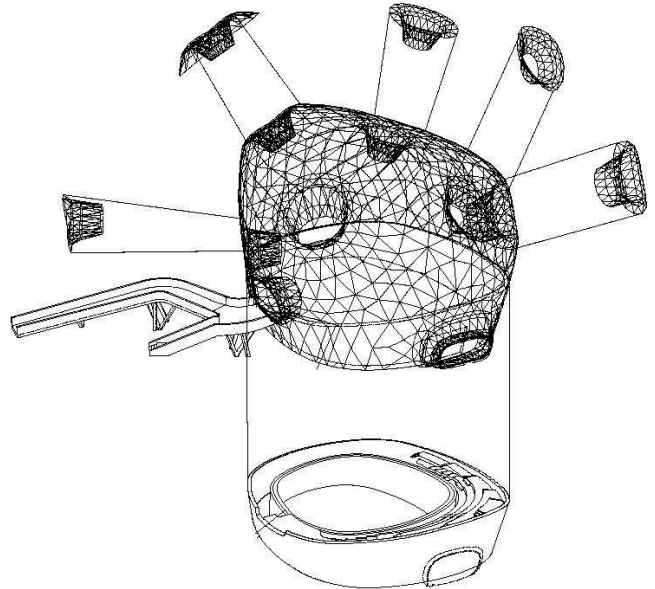


Figure 5. Axonometric view of the project

### 2.2 Shell definition

The shell is a three-dimensional structure made of slender interwoven steel components that form rhomboids, triangles and pentagons, which behaves like a dome. Its cladding is a continuous three-layer membrane which adapts to the initial geometry using differently sizes tessellates, depending on the curvature of the surface. Moving inside, it is replicated to form a faceted surface that facilitates the fine adjustment of the hall acoustics. Initially based on two tetrahedrons joined on one side, the dome surface is perforated at seven points for the entrance of visitors, light and views of the valley landscape.

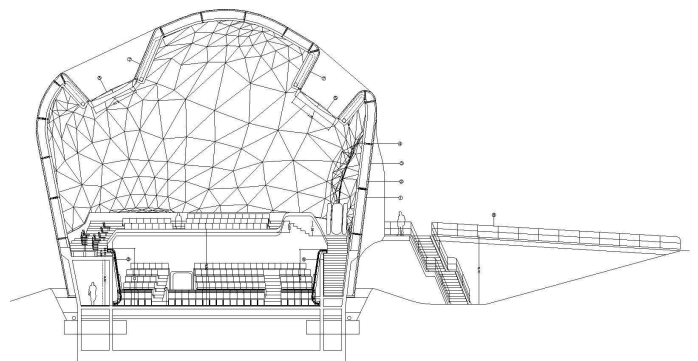


Figure 6. Section of the building

After a process of relaxing the surface geometry to optimise its structural performance, the tessellation geometry is warped by the inclusion of these discontinuities until we arrive at variable patterns, arranged in a strangely continuous way. At the points of discontinuity, the surface turns back towards the interior, thickening and forming large open vessels which reinforce the surface on the edge of the apertures. These large-format items, bathed in red paint on the inside are suspended from the space, feeding light and views of the valley into this camera obscura.



Figure 7. Shell interior

### 3 DESCRIPTION OF THE STRUCTURE

There are two structural types in this project. On the basement, we have a structural system consisting of reinforced concrete slabs and walls, which continue above ground as a concrete perimeter wall surrounding the auditorium. This wall supports the stands, and also the upper cap that closes the auditorium. This cap is formed by a large metal frame consisting of triangulated bars. A series of depressions appear on this cap as singular elements. In most cases, they serve as a natural entry of light into the auditorium,

as skylights.

As for the reinforced concrete structure, it is noteworthy that its geometry is not regular, but adapts to the curvilinear geometry of the project, becoming the most cases in concrete warped surfaces. The entrances to the Palace are reinforced concrete ramps and stairs. In some cases, being below ground they are surrounded by concrete retaining walls. As a unique feature, we highlight a number of V-shaped columns that support the ramp that goes directly to the first floor.

All the foundations of the building are done using concrete footings.

## 4 METALLIC SHELL

### 4.1 Regulations

Regulations used as reference are the Eurocodes and their national transcriptions (CTE- Actions and Structural Design) along with internationally recognized design guides.

### 4.2 Loads on structure

Permanent and variable loads are applied in this project, as defined in the DB SE-AE.

The following permanent loads have been considered:

- The dead load (PP), which takes into account only the weight of the structure.
- Permanent loads (CP), which take into account all the loads acting on the structure whose variation in time is negligible (textile and ceramics).

The following variable loads have been considered:

- Overload use (SU), generated by the weight its sole use, which is maintenance.
- Wind load (SV), produced by the impact of wind. The geometry of the shell has been simplified and assimilated to the composition of a cylinder and a hemisphere, to apply wind load.
- Snow overload (SN), produced by occasional snow accumulation on vertical surfaces.

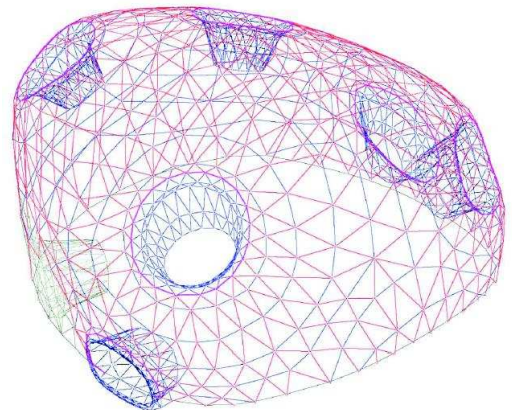


Figure 8. Shell modelling

### 4.3 Hypotheses

Diverse calculation hypotheses have been contemplated for the analysis of the structure presented here, mainly depending on the material of the element or structure. In this way we have the following hypotheses considered for Ultimate Limit States (ULS) and Service Limit States (SLS).

The considered hypotheses are those indicated by the DB-SE, "Basic Document SE Structural Safety" in article 4.2.2 and 4.3.2, as detailed below:

For Ultimate Limit States, the situations in the project have been approached using the following criteria:

- Persistent or transitory situations:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$$

- Accidental situations:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \gamma_A A_k + \gamma_{Q,1} \Psi_{1,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{2,i} Q_{k,i}$$

- Seismic situations:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \gamma_A A_{E,k} + \sum_{i \geq 1} \gamma_{Q,i} \Psi_{2,i} Q_{k,i}$$

For Service Limit States, the different situations of the project in general have been approached using the following criteria:

- Unlikely combination

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$$

- Frequent combination

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \gamma_{Q,1} \Psi_{1,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{2,i} Q_{k,i}$$

- Quasi-permanent combination

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \sum_{j \geq 1} \gamma_{G^*,j} G_{k,j}^* + \sum_{i > 1} \gamma_{Q,i} \Psi_{2,i} Q_{k,i}$$

Where:

$G_{k,j}$  characteristic value of the permanent actions,  
 $G_{k,j}$  characteristic value of the permanent actions with variable value,

$Q_{k,1}$  characteristic value of the determinant variable action,

$\Psi_{0,i} Q_{k,i}$  representative value of the combination of the concomitant variable actions,

$\Psi_{1,1} Q_{k,1}$  frequent representative value of the determinant variable action,

$\Psi_{2,i} Q_{k,i}$  quasi-permanent representative value of the variable actions with the determinant action or with the accidental action,

$A_k$  characteristic value of the accidental action

$A_{E,k}$  characteristic value of the seismic action.

The resulting hypotheses are:

ELU:  $1.35 \cdot pp + 1.35 \cdot cp + 1.50 \cdot su$

ELU (SV):  $1.35 \cdot pp + 1.35 \cdot cp + 1.50 \cdot sv + 0.75 \cdot sn$

ELS:  $1 \cdot pp + 1 \cdot cp + 1 \cdot su$

ELS (SV):  $1 \cdot pp + 1 \cdot cp + 1 \cdot sv + 1 \cdot sv$

### 4.4 Calculation methods for the metallic shell

Calculations of the metallic cap have been carried out following a 3-dimensional bar model. It has been studied in two different stages.

#### 4.4.1 Calculation. First Stage

On a first approach the calculation of the building's dome is made using the AGE program v3.2 [BOMA slp]: linear analysis of bar and sheet structures using the finite element method.

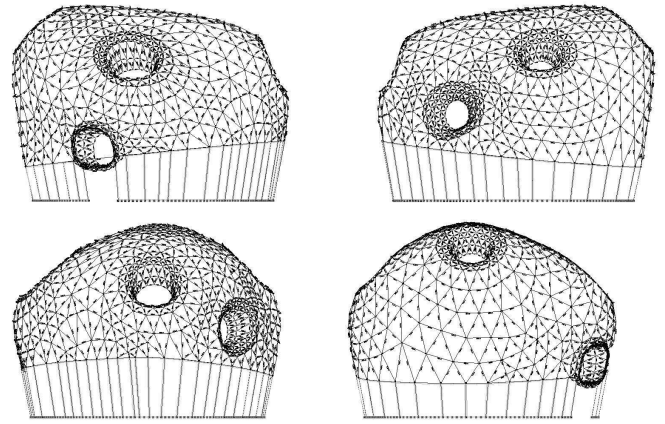


Figure 9. Different views of the modelling

We have performed a linear calculation by studying the individual stresses on each bar and affecting them by buckling effects. There are two families of bars, according to their types of support: fixed and pinned.

The stability of the dome is achieved by double curvature effect of the geometry, although in most flat areas main bars have fixed supports.

Even so, predominant efforts on tubular profiles are axial and bending moments are essentially irrelevant. Loads on the roof cover have been applied on all the rods' generatrix, because the PVC sheet will be supported on them.

There are two families of bars: the main and secondary. Main ones have fixed ends, and in most cases, they generate diamond shapes. Secondary bars have pinned ends, using connection plates. They close the diamond generated by the other family of bars, defining a triangular geometry. In both cases, circular tubes are 60x4mm and secondary rods have a rib-shaped trim to create a visual effect in the main hall. The nodes of the project have been defined as discs that receive the different tubular profiles (usually 6 elements) at different inclinations and orientations. The thickness of the discs allows the encoun-

ter in all situations and the diameter is the minimum necessary for the interference between profiles.

Discs are composed of a 275mm perimeter tube, two covers (top and bottom) and 6 internal stiffeners uniformly distributed which need not coincide with the tubes.

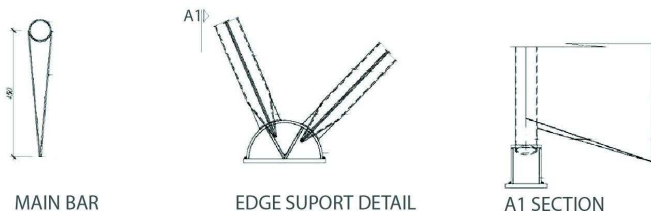


Figure 10. Design of the nodes

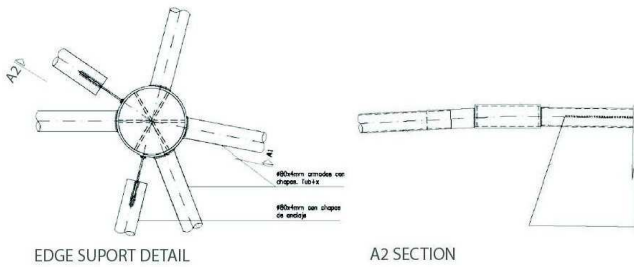


Figure 11. Design of the nodes

#### 4.4.2 Calculation. Second Stage

On a second stage, the structure has been studied using a nonlinear analysis (second-order P-Delta, which takes into account the change of bending rigidity according to axial forces, the additional perpendicular rigidity and limitations/constraints due to deflection). The method used is the "complete Newton-Raphson" where the K rigidity matrix is updated after every subdivision and after each iteration. The computer program verifies the convergence of the calculation automatically and stops when it reaches equilibrium.

In the starting model all bars are defined with the same section: 80.4 tubes without torsion inertia to prevent torsion stresses. Only the bars belonging to the perimeter of the circular depressions (holes on the structure) have a different section (219.8 profile tubes). According to the EC3 regulation, a geometric imperfection has been defined on each bar.

Main bars present fixed ends, whilst secondary bars have pinned ends.

Bars are fixed on the concrete wall on the base.

Once the calculation is done, the consequent stresses are studied in three different sections on each bar, and for each ELU scenario. That is, for each bar 6 checkings are carried out.

The analysis is performed only at section level, since according to the CTE DB-SE, a structure calculated in second order with geometric imperfections no bar analysis is needed.

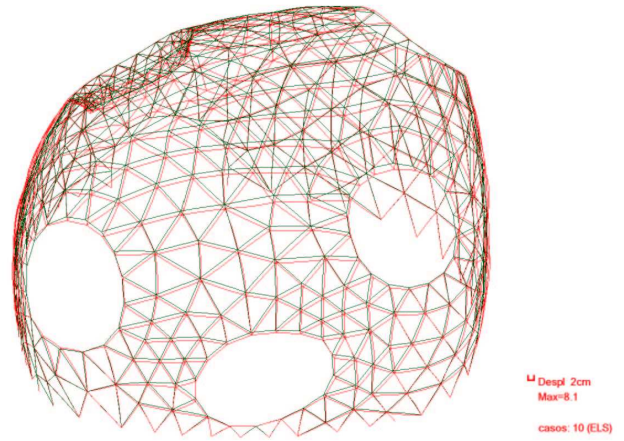


Figure 12. Model showing deflection

For each bar section, and for each scenario, the following stresses are obtained: Fx (axial force), Fy (shear force, y direction), Fz (shear force, z direction), Mx (torsion), My (bending moment, y direction) and Mz (bending moment, z direction).

With these values, sections have been verified according to the following formulas:

$$N, Ed / Nt, Rd + My, Ed / My, c, Rd < 1.00$$

(taken from the regulation: DB-SE steel, 6.2.8.(1))

$$Vz, Ed / Vz, pl, T, Rd < 1.00$$

(taken from the regulation: DB-SE Steel, 6.2.8(4))

For those bars where at least one section does not satisfy the previous formulas (that is, their results are >1), they are replaced by a larger section tube profile (the range used is: 100.6, 90.4, 80.4, 70.4, 60.4). For those whose results do not exceed 0.55, they are replaced by a tube of smaller section. Once these changes are carried out, the model is recalculated reiterating this process until all the bars meet the conditions described above.

## 5 CONCLUSIONS

The Project of Cherry Blossom Palace has been a very interesting exercise of geometry and sections optimization. It has also led to the confirmation that these types of structures develop the minimum strains. It has been possible to analyze the effects of double curvature as well as the equilibrium of the full model. We have analyzed the effect on geometry in case of failure of a particular area with very interesting results.

Working with tubular bars has allowed optimizing the sections and coming up with very simple joints, that could be generalized and thus simplify its construction.

Collaboration with the architects of the project has been intense and close, so that the result has been the best for the space and the building, as well as the most optimal for the structure.