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DOME AT THE ERNST & YOUNG NEW HQ BUILDING, LUXEMBOURG

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Introduction

The Kirchberg area, in the northeast of the city of Luxembourg, is the location of numerous banks, government and European Institutions. Here, at the wide Avenue John F. Kennedy the new headquarter of Ernst&Young Luxembourg was just opened. Sauerbruch Hutton Architects designed a H-shaped 24m high building with an open plaza which is marking the spacious public entrance to the headquarters.

The plaza is on three sides flanked by cafes and restaurants and needs to be covered by an appealing roof. In collaboration between the architects and Schlaich Bergermann Partner a glass roof was developed, which also fulfils all geometric requirements and boundary conditions.

Fig. 1. Ernst&Young Headquarters (Sauerbruch Hutton)
1. Conceptual design and structural detailing

The approx 35m x 41m large trapezoidal Plaza was to be covered by the new roof above the 4th floor at a height of about 20m. The major constraints for the roof design were the maximum allowable height of the roof structure of 3.8m above the ceiling of the 4th floor and the horizontal eaves on the front side of the plaza roof, which takes up the horizontal eaves line of the adjacent wing buildings. Furthermore, structural components such as trusses and columns below the ceiling of the 4th floor had to be avoided.

In the preliminary planning various roof alternatives were studied and discussed with Sauерbruch Hutton. Different types of structures, such as grid shells, membrane structures, cable supported grids, truss systems, etc. and numerous variations regarding orientation and arrangement of these structures were studied.

Finally, a flat grid shell roof proved to be the best solution regarding architectural design, transparency and economy. The roof structure consists of a biaxially curved steel grid structure which is supported by filigrane air props and cables in every second arch.

The dome structure of the roof was generated by a translation of a transverse arc along a perpendicular arc. The sides of the dome are intersected with inclined planes so that the eaves are circumferentially horizontal.

At the same time the transverse arc was stretched to fit between the upper lateral longitudinal edges. Thus, clippings of the square glass panes on the long edges could be avoided.

This geometrical modelling enables the cladding of the grid shell with planar quadrilateral glass panes. The heat strengthened laminated safety glass panes of the dome are continuously supported for downward loads and held by single clamping plates for upward wind suction loads. The resulting mesh size of the grid shell is between 1.7m x 1.7m and 1.7m x 0.8m. The inclined front facade glass panes have dimensions of up to 1.7m x 4.9m.
1.1 Conceptual design and structural detailing

The maximum allowable height of the steel structure from bottom edge to upper edge of the roof was about 3.8m. Usually a ratio between the arch rise and span width of 1/5 to a maximum of 1/10 is desirable for arch or shell structures. Resulting from the limited overall height, the span ratio was about 1/15 in this case and the structure need to be supported by additional structural members to avoid stability failures like buckling or a snap-through of the structure.

The compressed arch and form a trussed arch structure. These props are usually supported on a vertically curved tension element in which the inclination of the lower tension elements enables the transfer of the vertical prop forces to the outer bearings, where this system is still not so efficient and stiff as a 'real' truss with diagonals between the vertical members.

In general the total structural stiffness of a cable structure is defined by the sum of the elastic and geometric stiffness of the cables. The geometric stiffness is depending on the pre-stressing and the length of the cables, where the stiffness increases with increasing of the pre-stressing and with decreasing of the cable length. The elastic stiffness is depending on the Young’s Modulus $E$, the sectional area $A$, the cable length and the angle between the acting loads and the cables axis.

For the system shown in fig. 4 the elastic stiffness is dominating the overall stiffness. The smaller geometric stiffness could be

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**Fig. 3.** Arch with horizontal tension tie for equilibrate the horizontal arch forces

**Fig. 4.** Arch truss, elastic stiffness activated

**Fig. 5.** Hybrid system, geometric stiffness activated
improved by increasing the pre-stressing of the cables, but since the influence of the EA of the cables is in orders of magnitude greater than the pre-stressing force, this wouldn’t affect the overall stiffness significantly. Due to this fact, the geometric stiffness due to pre-stress of the structure becomes neglectable and the overall structural stiffness is more depending on the elastic stiffness.

For formal architectural reasons the lower tension cables couldn’t be curved in the case of the plaza roof and need to be strictly horizontal. Therefore a system as shown in fig. 5 was chosen to stabilize the shell structure.

For the dead weight configuration of this system, the air props are not loaded, since their support is only defined by the comparatively low geometric stiffness of the straight pre-stressed cables and almost no vertical forces can be transferred to the supports.

Here, the cable is orthogonally loaded and the angle between acting load and cable axis is perpendicular, what means the usually large elastic stiffness component drops to zero. Hence, the structural stiffness of an almost straight cable, is mostly defined by the geometric part.

If by additional loads the roof dome begins to deform, these deformations are transmitted via the props to the cables, where additional restraining forces are activated by the noticeable increasing of the elastic stiffness due to the change in shape. In sum, then the overall stiffness becomes large enough to stabilize the grid shell structure and prevents it from buckling.

1.2 Structural members

For the grid shell beams rectangular standard hollow sections with uniform dimension of 140x80mm and different wall thicknesses between 8mm and 14.2mm depending on the degree of utilisation were chosen. The nodes are solid machined steel nodes to which the beams are connected by full penetration butt welds. The circumferential stronger edge beams are made of standard hollow sections of size 250x150x16.

For the cables open galvanized spiral strands of diameter 24mm for the straight cables and 31mm for the polygonal cable on the front facade were used, which were attached to the structure via open swaged fittings. For stressing, one side was equipped with turnbuckles.

The very slender vertical air props have a diameter of only 24mm and are attached via Besista fork connectors to the steel structure. Most decisive for the design of the up to 2.4m long props were the stability checks. Special attention was paid on the design of the hinged connection points.

Large rotations of the air props could cause a locking between the hinged fork connectors and the connection plates, which would cause unwanted additional bending in the slender props and plates. The fork connectors of Besista where chosen, since they are designed to provide a larger rotation capacity than standard fork connectors. The required connector size was determined, taking the geometrical imperfections, unwanted rotations, construction tolerances and expected deflections of the system into account.

Fig. 6. Air props
The bearing concept of the roof was chosen so that the impact of constraining forces due to temperature expansions onto the adjacent buildings was minimized as much as possible. The roof is supported on the side wings of the building at a distance of approximately 3.30m and spans between 18m on the short side at main entrance of the Headquarter and about 41m on the open side at the Av. J.F. Kennedy. The supports are hinged and unidirectional sliding bearings with stainless steel pins and SKF filament wound bushings between the pins and the steel parts. These bushings have a very high load capacity, are insensitive to edge loading and misalignment, are maintenance free and reduce the friction forces between the individual components. The system, which was already successfully used in other projects by sbp, allows the roof to move along the edge beam axis without causing larger constraint forces due to temperature elongations. The fix point of the roof was placed very close to the open edge at the side of the street and not - as usually done- at the centre of the roof. Thus, the horizontal displacements due to temperature expansions of the roof structure were almost zero on the front side and a misalignment between the structure and the building facade could be avoided. For the corner points special hinged bearings were developed, which are only working for down and uplift forces and are horizontally in all directions free sliding.

Fig. 7. Bearing arrangement
1.5 Structural analysis

The global design was done with a three-dimensional geometric non-linear Finite Element analysis considering large deformations. Therefore, a detailed structural FE-model was used, which included the real support conditions and cross sections of the members.

The basic ULS design of the main structure was done according to the elastic-elastic-method with a equivalent stress checks of the members. Beside the stress checks the stability behaviour of the members was investigated. In general the structure must be checked for global stability failure of the main shell structure and local buckling of the single grid beams. Therefore, the local check of the single beam stability failure was proved according to the rules for uniform members in bending and axial compression of the Eurocode.

The global stability checks of the shell structure were done by performing a geometric nonlinear analysis with global imperfections. This way, the influence of the occurring second order effects could be taken into account in the stress analysis.

1.4 Front facade

The inclined front side of the roof is horizontally supported by horizontal struts and a polygonal cable which is attached to the corner points. The pre-stress and cable geometry was chosen to compensate the horizontal deflections of the arch shaped girder of the inclined roof front side. Without the horizontal support of the lower edge beam, the front facade would deform towards the plaza. Vertical props were attached on the polygonal cable to stabilize the upper front arch at the same time.
2. Production

The steel structure, comprising the central area and perimeter arches, was produced according to the pre-cambered geometry, provided by SBP, to compensate dead load deflection and thus obtaining the desired designed geometry after removal of the temporary props supported on the scaffold platform.

![Pre-cambered wireframe model of the steel structure](image)

The central grid shell is made of 140x80mm RHS S355 steel bars varying in wall thickness from 8mm to 14.2mm which are welded to high precision machined solid steel nodes. There are 453 different solid nodes as the grid shell has no lines of symmetry and the individual weight varies between 28 and 40 kg.

![Plasma cutting of the primitive node geometry](image)

The primitive geometry of the solid nodes was obtained by plasma cutting a 140mm thick S355 steel plate, as shown in figure [11], and then machined with a 5-axis milling machine to obtain the desired slope on each face, appropriate edge radius and an excellent quality of the finished surface. The high level of accuracy of the nodes allow the bars to be cut straight.

A central recess milled at the node faces left a perimeter ready to match the specific RHS in order to ensure a successful full penetration butt weld.

In figure [12] a 3D model of a typical node is shown. The light shaded geometry corresponds to the plasma cutting primitive and in green colour the final geometry after the complete milling process.

![3D CAD model of a typical node geometry](image)

All the nodes are pretty similar and in order to make them easily identifiable and correctly positioned inside the milling machine were notched at one of their four fingers during the plasma cutting and its individual reference was engraved on its top face.

To firmly position the nodes in the milling machine all nodes were bolted to the bench avoiding vibrations during the milling works. Two holes were drilled on the top face of the nodes for workshop and site surveying as it can be seen in figure [19]. Further threads were machined to some nodes to provide lifting points for prefabricated segments.

The arches member sections are 250x150x16mm RHS S355 at the perimeter and 140x80mm - with different wall thicknesses - forming the connecting members.
The connecting member for the large front façade arch are 180x100x12.5mm RHS.

Different to the central area, the perimeter arches were composed of 3D laser cut members.

3. Workshop pre-assembly

The structure was divided in 21 transportable segments ranging from 3.3x8.2 meters up to 5x20 meters. Figure [15] shows schematically the factory pre-assembled roof segments.
Fig. 21. Site assembly works. Workshop preassembled segments are identifiable

Following accurate positioning of segments, individual members were welded between them to build the final grid on site.

The site assembly works were carried out from September 2014 to April 2015. The strong wind and unfavourable climate conditions affected the progress of the welding and painting works. To minimize the weather impact and create local acceptable conditions for work some covers heaters and dehumidifiers were required.

Once the roof structure was completely welded, the cables were installed. When the cables were tightened, the structure slightly lifted and following this the temporary supports were removed.

Site paintworks were carried out after the cables installation. Special care was taken at the onsite welded parts.

Following completion of paintworks glass installation started. The central roof area and the perimeter arches were welded last and geometrical deviations were accommodated at the interface, i.e. site measurements were required to produce glass panels for this area accurately.

Fig. 22. Left and centre images, the pneumatic equipment used to tighten the cable. Right image, the cable once installed

Fig. 23. Roof glazing works

Fig. 24. Finished dome at the E&Y new headquarters in Luxembourg
7. Quality control

The following tests were carried out during the project to ensure the quality of materials/procedures:

a) Paintworks: Test adherence and coating thickness control both in workshop and on site.

b) Glass impact test: Soft body impact test on the laminated 8mm HS + 0.76PVB + 8mm HS according to NF P01-13 standard consisting in a 50 kg soft body mass dropped from 2.40 metres above the glass. No glass failure occurred.

c) Sliding test: A sliding test was performed according to DIN EN 10204 – 3.1B to check that no slip occurred at the cable clamp connection under design loads.

d) Non-destructive weld tests: Ultrasonic testing was done to more than 20% of the welds performed in the workshop. All on site welding was 100% Ultrasonic tested.

References