DESIGN AND EXECUTION OF THE EXHIBITION HALL 3 IN FRANKFURT/MAIN GERMANY

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The exhibition hall 3, with 38,000 m^2 of space on two levels, was built to expand the Frankfurt am Main trade fair location. An outstanding architectural feature is the 165 m wide self-supporting hall roof. After a general introduction, this article deals with the supporting structure of the hall roof and its static and dynamic calculation. After descriptions of the structural design, the production and assembly as well as the quality assurance of the tubular structure are described.

Keywords: Exhibition hall; long span roof; circular hollow sections; structural design, execution.

1 Introduction

In the course of the expansion of the trade fair location Frankfurt/Main, the construction of the new exhibition hall 3 was realized on the site of the former freight station. The London architect Nicholas Grimshaw was the designer of the approximately 220 m long and 125 m wide hall. Along with Norman Foster and Richard Rogers, he is regarded as the most important representative of British high-tech architecture. The new Hall 3 offers a total of 38,000 m^2 of exhibition space on two floors (Fig. 1).

An outstanding architectural feature is the construction of the 165 m free spanning roof. The supporting structure consists of a three-dimensionally shaped tubular steel construction, which is supported by so-called A-frames. The approximately 5 m high intermediate level of Hall 3 is used to integrate the building services and to access the upper exhibition floor by technical installations. A story-height steel truss structure and steel composite columns at a distance of 32.0 to 34.5 m support the intermediate level.

The hall opens to the north with a full-storey glass facade. The east, south and west sides of the hall are adjoined by 4-storey enclosing buildings which provide access to the exhibition levels. These contain office and conference rooms, restaurants and sanitary facilities. The façade of the east and west side is fully glazed and fitted with fixed sun protection slats (Fig. 1).

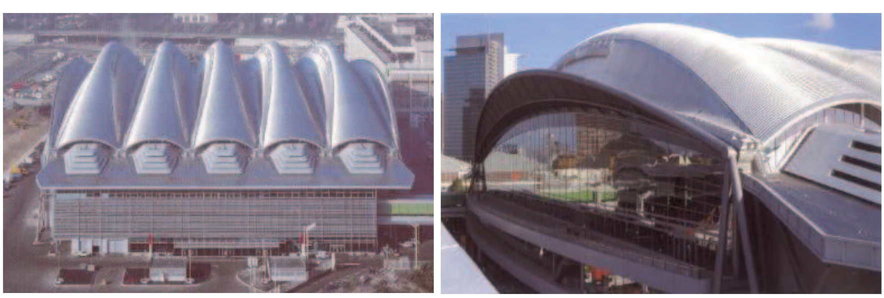

Figure 1. Exhibition hall 3 – view from the east, glass facade on the north side

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2 Bearing Structure of the Hall Roof

The bearing structure of the hall roof consists of a longitudinally spanning, curved framework of tubular steel sections (CHS). The members are welded together at the nodes in a rigid manner. The loads are transferred via 12 A-frames, which stand on a combined pile and slab foundation and are supported at a height of +10.37 m in the transverse direction of the building by reinforced concrete cores.

The vertical loads are transferred in the longitudinal direction of the hall via the compression arcs. Parts of the arc forces are short-circuited by the lower tension arcs within the roof. The remaining part is transferred to the building ground via the A-frames. The distribution of internal forces within the bearing structure is influenced by the stiffness ratios of the members and connections, and to a large extent also by the deformations of the foundations of the A-frames.

Each of the five inner compression regions of the roof structure consists of two main compression arcs with 914 mm and five secondary pressure arcs with 457 mm diameter. For the tension arcs and the transverse members, tubes with 419 and 323.9 mm diameter were used. The wall thicknesses are between 8 and 85 mm (Fig. 2). The transverse arcs are involved in the transmission of forces between the compression and tension regions and stabilise the compression arcs perpendicular to their plane. To absorb the resulting bending moments, the nodes of the main and secondary pressure arches are largely reinforced by thick-walled tube sections (Fig. 3).

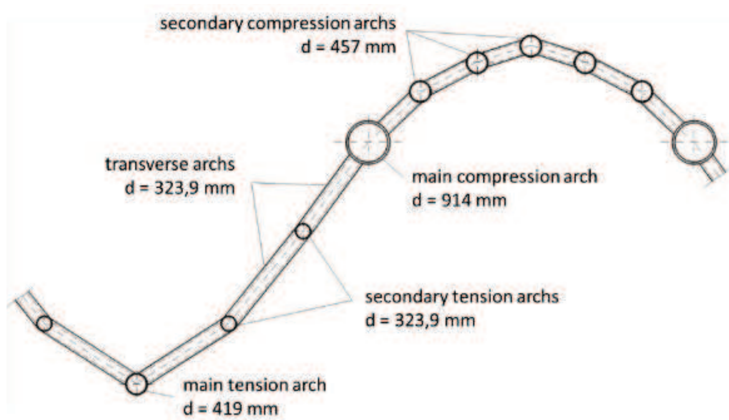

Figure 2. Section through the supporting structure of the hall roof

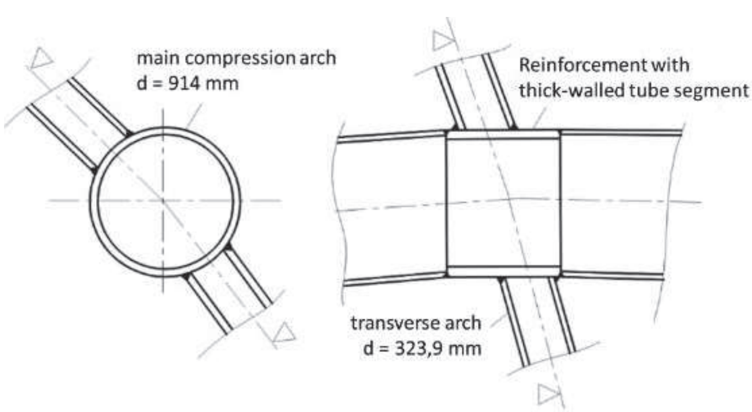

Figure 3. Nodes of the main compression arcs

3 Static and Dynamic Analysis of the Roof Structure

The roof construction was calculated geometrically nonlinear to investigate the global structural behavior as a spatial structure taking into account imperfections from the manufacturing process. Figure 4 shows the structural model that was used for the comparison calculation as part of the approval engineering. The basis of calculation included, in addition to the applicable DIN regulations appraisals for wind loads, building ground movements and structural fire protection.

The rigidity and stability of the main roof is strongly influenced by the rigidity of the connections of the transverse to the longitudinal arches. The clamping effect is achieved exclusively by the bending and membrane stiffness of the section walls in the nodes. In the comparative calculations, the flexibility was described by rotational spring elements at the ends of the member. The determination of the spring stiffness was carried out at reference nodes with FE shell models and with the calculation methods according to Wardenier (2008) and Dutta (2002) (Fig. 5). A distinction was made here between moments in and perpendicular to the nodal plane. The considerable influence is reflected in both the internal forces and bending moment distribution within the roof structure as well as in the buckling load factors and natural frequencies (Fig. 6).

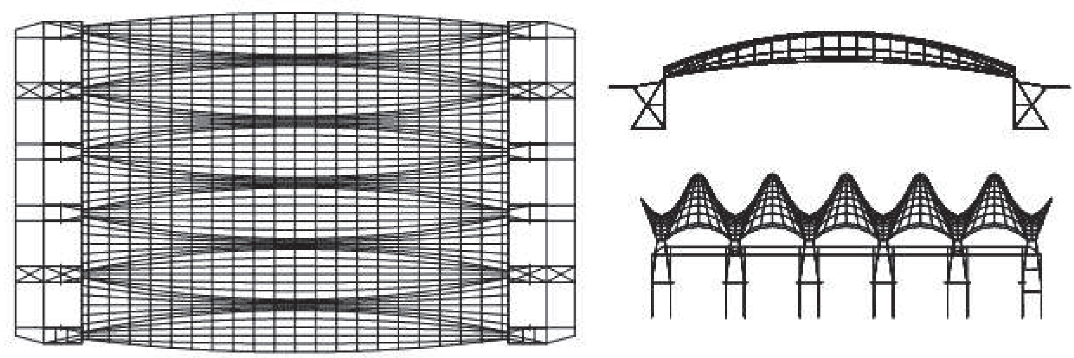

Figure 4. Modell for the structural analysis of the roof structure

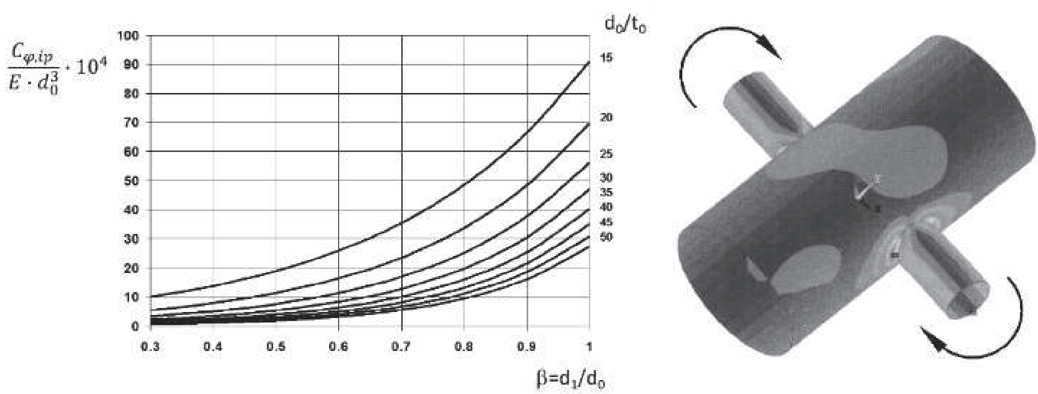

Figure 5. Determination of the rotational spring stiffness of the transverse tubes in the joint plane

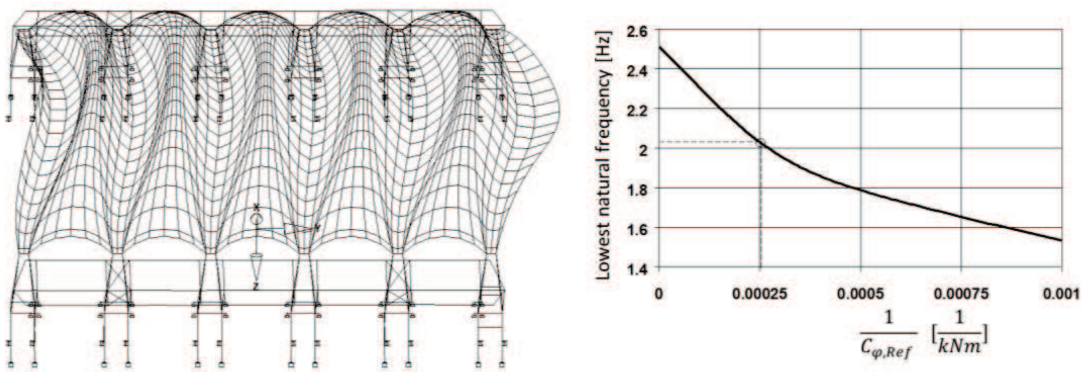

Figure 6. Dependence of the natural frequency on the connection stiffness of the transverse tubes

About 1430 joints of hollow sections had to be made for the roof structure of the exhibition hall 3. The sections are rigidly connected at the nodes, so that bending and torsion moments can be transmitted as well as normal and transverse forces. The internal forces Q_y, Q_z, M_T, which are transmitted by shear stresses, are of secondary importance for the load bearing capacity of the joints.

The tube segments of the longitudinal arches are connected by butt welds with V-seams. In the compression arches, the longitudinal tubes were reinforced to a large extent by short, thick-walled tube sections (Fig. 2) to increase the rigidity and load-bearing capacity of the nodes. The transverse tubes were placed on the longitudinal tubes and, depending on their position, are connected circumferentially with butt and fillet welds which merge continuously into one another.

Due to the partly very short tube segments for the reinforcements of the compression arches (Fig. 2) and the considerable wall thickness differences between the reinforcements and the adjacent sections, it was not possible to assume the validity of the load-bearing capacity formulas from CEDICT for all joint points. For highly utilized tube joints, where the welds of the transverse tubes reached over the reinforcements in certain areas, materially non-linear FE calculations were carried out on spatial structural models with solid elements.

Another essential design requirement for the application of the CEDICT formulas is that the hollow sections have to be connected with their full load-bearing capacity. Due to the different stiffness ratios within a tube joints, local stress peaks occur. If the load bearing capacity of the welded joints per unit length is higher than that of the tube wall, these can be relieved by plastic deformation and stress redistribution.

At the roof structure of Hall 3, all tube joints were connected with weld seam thicknesses $a \geq t$. For the purpose of quality insurance, visual inspections and surface crack tests as well as volume tests with ultrasonic measurements were carried out. The welds of the connections had to meet the requirements of evaluation group B according to EN 25817 (1992) (α_w=1,0).

The tube reinforcements at the nodes of the compression arches sometimes lead to considerable wall thickness jumps (Figure 3). The edges were not chamfered in favour of rational production. In order to check whether fatigue cracks may already occur at wind loads due to the strong notch effect, fatigue strength checks according to EC 3 were performed on the basis of local reference stresses. For this purpose, the related collective of gust stresses according to EN1991-1-4, Annex B (2005) for a service life of 50 years was used as a basis (Fig. 7). Periodic vibrations of the structure due to wind loads did not have to be taken into account. The fatigue proof was performed on the basis of the linear damage accumulation hypothesis according to Palmgren-Miner (Eq. 1). The continuous collective in Figure 7 was replaced by a staircase function,

$$\sum_i \frac{n_i}{N_i} \leq 1{,}0 \tag{1}$$

n_i = number of cycles with a stress range $\Delta\sigma_i$

N_i = number of stress cycles until fracture with a stress range $\gamma_{Mf} \cdot \Delta\sigma_i$ according to the fatigue strength curve for the relevant notch class

γ_{Mf} = partial factor for fatigue strength

N_g = number of gust loads during the service life of 50 years, at which the value ΔS_k of the wind load is reached or exceeded

S_k = gust loading with an annual occurrence probability of p=0,02.

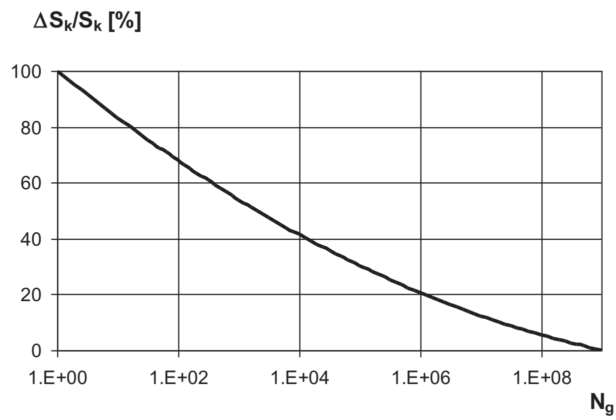

Figure 7. Normalized collective of gust loads for a service life of 50 years (EN1991-1-4, Annex B (2005))

The construction of the A-frame heads balances the arch forces of the roof with the post forces of the A-frames (Fig. 8). In the connecting point of the main compression arches to the A-frame posts, massive forgings are arranged, to which the tubes connect with a butt weld. Between the forgings, there are traverses out of welded box sections. The main tension arches are connected in the centre of the box sections by butt welds and flag plates. The traverses have openings with a diameter of 270 mm on both sides, through which the rainwater down pipes of the roof are led.

In the relevant design load case, the main tensile arches transmit tensile forces of approx. 7000 kN into the traverses. These are transmitted to the forgings of the A-frame heads via shear forces and bending moments. Due to the large transverse forces, considerable secondary bending moments occur in the area of the openings. The top and bottom flanges with a thickness of 30 mm were reinforced with additional plates with a thickness of up to 60 mm during construction. The structural analysis was carried out with material non-linear FE-calculations.

A minimum yield strength of 355 N/mm² was required for the forgings, as for the connecting steel sections. A quenched and tempered forged steel with a diameter of 920 mm, a length of 1000 mm and a weight of 5.22 t per unit was used.

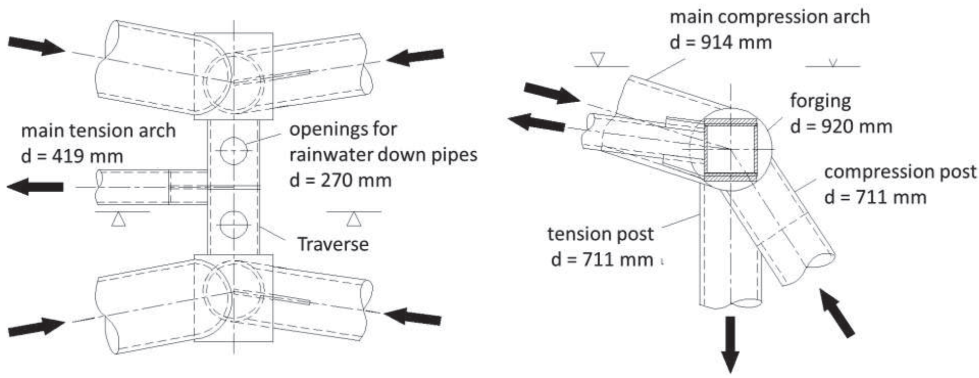

Figure 8. Construction of the A-frame heads with directions of acting forces from connected members

4 Manufacturing and Assembly

The assembly of the main supporting structure was carried out from autumn 2000 to spring 2001 in only 7 months. The large dimensions of the roof and the construction of the joints made it necessary to relocate a large part of the production to the construction site.

For the tubes, mainly hot finished hollow sections according to EN10210 made of S355J2H with a minimum yield strength of 355 N/mm² guaranteed for all material thicknesses were used. Due to the welded connections on the tube walls and the transmission of tensile forces perpendicular to the rolling direction, the use of Z-grades of the through-thickness ductility of up to 30 % was required to prevent lamellar tearing (EN10164, 2018). For tubes with wall thicknesses above 30 mm, weld-on bending tests based on the regulations of the German steel-iron test sheet 1390 were carried out (Stahl-Eisen-Prüfblatt 1390, 1998) to verify the brittle fracture tendency.

The material cutting and welding of components into transportable units (A-frame heads and posts, segments of the longitudinal arches, etc.) took place in the plants of the steel construction company. The cutting geometry and seam preparation at the ends of the transverse tubes were produced with a computer-controlled flame-cutting machine. The individual parts were delivered to the construction site with lengths of max. 25 m and unit weights of up to 19 t (Stroetmann 2003). The roof was divided into 4 x 11 = 44 segments for lifting assembly. These were assembled, stapled and welded in gauges on pre-assembly stations. Due to the symmetry of the roof and the periodic construction of the structure, the entire geometry could be produced with only four different pre-assembly gauges, two each for the compression and tension segments (Figs. 9 and 10).

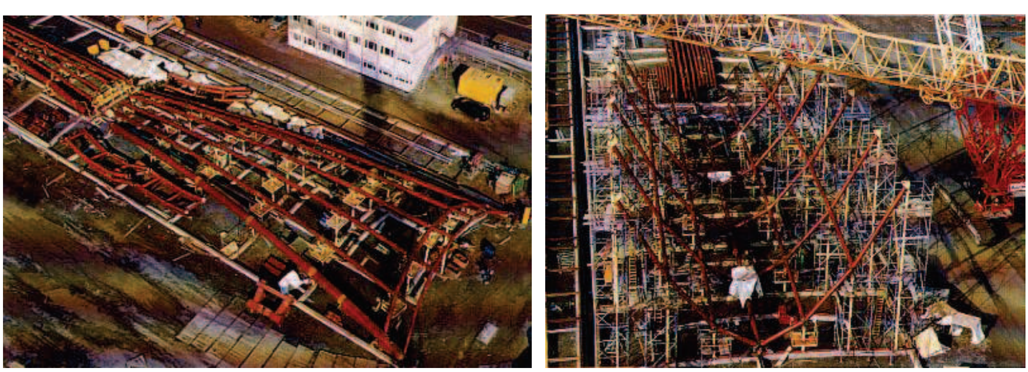

Figure 9. Prefabrication of the roof segments


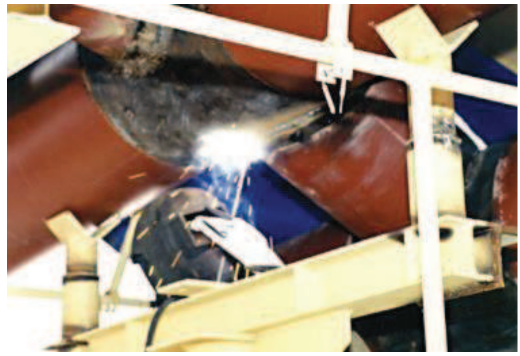

Figure 10. Welding of tube joints on the pre-assembly site

The roof assembly was carried out arch by arch, starting from the south side of the hall. Before the start of the lifting assembly, the A-frames at the ends and the assembly frames made of steel trusses were placed in the quarter points of the roof. The approximately 40 - 45 m long and up to 125 t heavy roof segments were transported by crawler cranes from the pre-assembly places to the installation site (Fig. 11). After welding the roof segments together, the roof was covered. For this purpose, diamond-shaped sandwich elements adapted to the special roof geometry were pre-assembled on the ground, lifted in with tower cranes and screwed to the supporting structure. The insulation, sealing and cladding could then be carried out with the bent aluminium standing seam plates. The roof was hydraulically lowered and the mounting frames dismantled. Subsequently, work could begin on the interior finishing of the upper hall level (Fig. 12).

The movements of the ground at the abutments have a considerable influence on the stress condition of the hall roof and the foundations. Since the settlement values could not be predicted with the accuracy required for the steel structure, it was decided to adjust the state of force in the main supporting structure by applying specific pre-stressing to the anchorages of the A-frame tension posts (Fig. 13). This process took place after the roof had been lowered from the auxiliary scaffolding and the immediate settlements had subsided. Due to interactions during the force regulation of the 12 A-frames and delay effects during the force transmission, the adjustment had to be carried out several times at intervals. When calculating the nominal values of the tension post forces, the current building temperature and the dismantling condition of the hall were taken into account. Due to the force setting, the influence of the building settlements on the expected residual settlements could be reduced in the static calculation. These amount to between 20 and 40 % of the total settlements in exhibition hall 3, depending on location and direction.


Figure 11. Assembly of the roof segments

Figure 12. Roof soffit, installation of the technical bridges

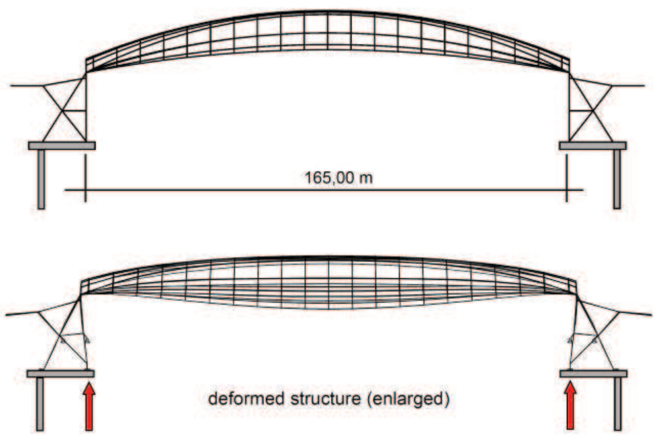

Figure 13. Pre-tensioning the roof by lifting the tensioning posts of the A-frames

5 Summary and Conclusion

The new exhibition hall 3 sets European standards with its size, functionality and design. The architecturally striking and technically sophisticated roof structure placed high demands on planning, construction and supervision. The construction of the project was carried out from April 2000 to August 2001 in the record time of only 16 months. At times, up to 1100 craftsmen were employed on the construction site at the same time.

Due to the very tight deadlines, the object, structural and assembly planning as well as the structural assessment and monitoring were an interactive construction-accompanying process that required intensive cooperation between all parties involved. Approx. 45,000 m^3 of concrete, 6,000 t of reinforcing steel and 10,000 t of sectional, flat and forged steel were used for the bearing structure of Hall 3. The exhibition hall passed its first test at the "Tendence" consumer-goods fair (which began on 24 August 2001).