Papp Laszlo Sportarena, Budapest

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Introduction

The multipurpose Papp Laszlo Sportarena situated near the centre of Budapest has quickly become the new symbol of the modern and dynamic Hungarian capital, capable of hosting a great variety of cultural and sporting events. At the very first glance, the visitor is captured by the smoothly rounded stone-like form of the building obtained by the architect from the revolution and subsequent distortion of a space curve in his computer model (*Fig. 1*). The outer shape is characterized by double curvature and symmetry along the longitudinal axis. Beyond this symmetry, every cross section of the model yields a different geometry, which was the main difficulty for the designers and builders.

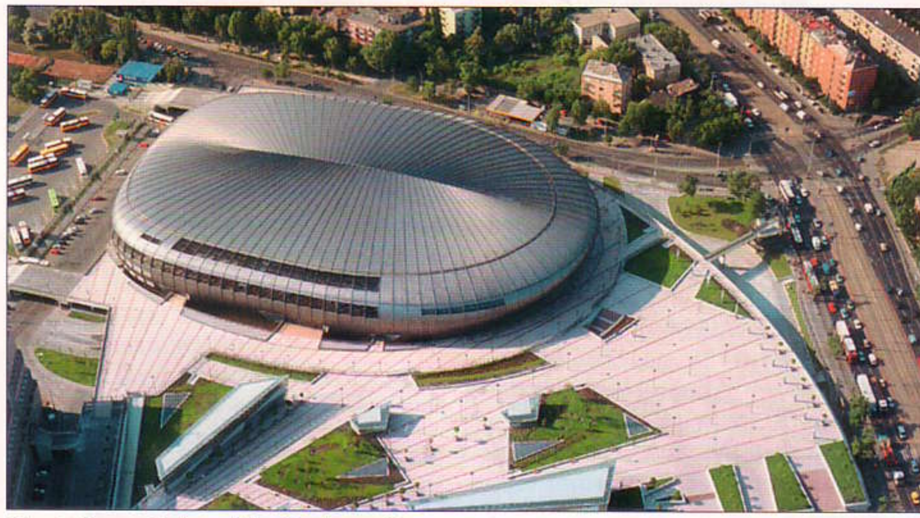

Fig. 1: General view of the Budapest Sportarena

Structural Solution

The 15 m high concrete superstructure around the arena is divided into seven movement blocks. The columns and shear walls are cast in situ, while the main beams of the grandstand and the seating elements are prefabricated. The slabs are made of prestressed precast hollow core planks. The main trusses of 90 m span are spaced every 10 m, supported by the concrete columns of the upper slab. Their height varies between 6,56 m and 10,62 m. The steel canopy on the east is cantilevered over the main entrance of the building. The west facade consists of a steel frame from ground level up to the roof.

The outer skin of the building consists of up to 40 m long continuous aluminium sheets on top of the thermal and acoustic insulation and waterproofing, radiating from the ridge of the roof down to the level of the elevated deck, without any apparent separation between roof and facade. Along the aluminium sheets, the movement joints are incorporated in the gutters. The main challenge in the structural design was to find the best way to coordinate the differential movements due to thermal expansion between the roof covering, the underlying steel structure and the concrete blocks below.

Evolution of the Roof Model

The first approach consisted of a quasi space frame behind the curved front of the roof (*Fig. 2*). The radiating truss beams followed the geometry of the roof ribs and were connected to the first main truss. Longitudinal trusses were provided between the main trusses in order to redistribute the higher load of the first truss to the inside of the building and to limit the vertical differential movements. These secondary trusses are linked together up to six bays along the main axis, decreasing gradually toward the edge of the roof. The same principle was adopted at the back of the building for the distribution of the loads from the west wall. The trusses were clamped at the supports, slightly cantilevered at the ends and connected by a strong perimeter beam. The detailed calculations showed that the redistribution was not efficient enough and the first truss still had to carry 1,7 times more load than the rest. The erection method of the secondary trusses presented some difficulties, which also ruled out this solution due to the very tight construction schedule.

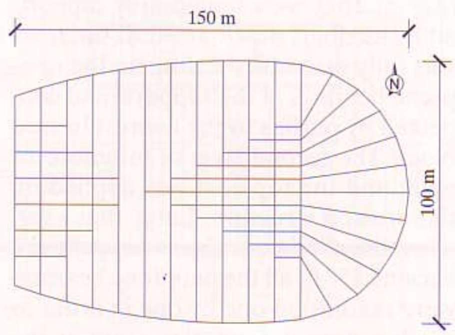

Fig. 2: Quasi space frame model

In the new model, the radial beams became continuous by adding columns, and also shorter due to three new trusses transversally on the curved part. Their setting out is slightly different from the path of the roof ribs (shown in red on *Fig. 3*), in order to distribute the loads more evenly on the first truss. Along the perimeter of the roof, the steel structure under the covering follows the orientation of the ribs, but above the arena the main trusses are set in an orthogonal grid perpendicular to the axis of the building (*Fig. 3*). The roof structure is divided into two movement blocks between gridlines 8 and 9. Double purlins with sliding connection on opposite end were used between these two trusses. The wind bracings are situated in the middle of each movement block.

Many support conditions of the trusses were studied for the loads and the movements. Some examples are shown above (*Fig. 4*). To avoid the creation of a movement joint in the roof at the supports of the trusses and to provide continuity of the metal sheeting, the bearings had to be identical at both ends of the trusses to respect the symmetry of movements. These conditions could be best satis-

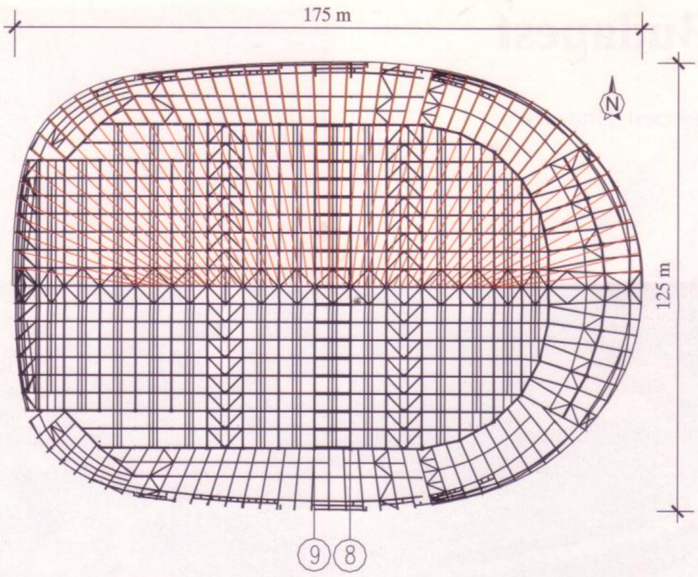

Fig. 3: The final roof model

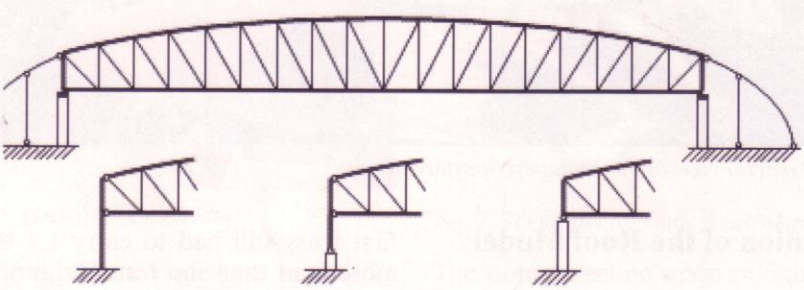

Fig. 4: Studied support conditions of main trusses

Fig. 5: Erection of the roof

fied by the use of neoprene bearings, which prevent vertical movement, allow rotation and horizontal movement and transfer only limited horizontal force on top of the cantilevered concrete columns. The steel roof is "floating" above the concrete superstructure. Accidental excessive movements of the trusses are prevented by a stopping mechanism on the top of the columns. The curved vertical perimeter beams are hinged both to the upper slab and to the truss ends (*Fig. 4*).

The top chords in the trusses are segmented to follow the rounded external shape. The top and bottom chords are made of hot rolled sections; the vertical and diagonal members are hot finished hollow sections. The bolts used in the connections are M20-M30 10,9 HSFG (High Strength Friction Grip) bolts.

The fire resistance was a top priority for the safety in the building. The 45 minutes fire rating of the roof trusses was provided by an intumescent paint tested in the laboratory. The main live load on the roof is the 600 kN of heavy equipment used for the shows, attached to the steel structure in different configurations and the 150 kN fixed scoreboard. An extensive wind tunnel test was carried out to validate the wind loads acting on the building. The three dimensional computer model of the steel structure was checked under a great number of load combinations according to Eurocode 3. The stability of critical members was also verified by a second order analysis.

Erection of the Steel Structure

The main trusses were pre-assembled in the factory before delivery to site. They were assembled on the ground into three 30 m long segments and the segments were erected one by one with the use of high capacity mobile cranes (*Fig. 5*). They were temporarily supported by auxiliary scaffolds until the truss was fully assembled sitting on the neoprene bearings at the supports and connected by purlins to the nearest braced block. The second layer of intumescent paint and the top coat was applied on the erected structure. Later that year, when the average ambient temperature became 15° C, all the neoprene bearings were readjusted one by one in order to reach a zero deformation state at the reference temperature.

Conclusions

The design development of the steel structure has been discussed. Construction started in August 2001 with the foundations. After a strong winter, the concrete and steel structural works finished in May 2002 and the opening ceremony was held in March 2003.

SEI Data Block

Owner:
Rendezvenycsarnok Ingatlanfejleszto es Kezelo Rt., Budapest, Hungary

Architects:
Gyorgy Skardelli (KOZTI), Peter Pottyondi (KOZTI), Budapest, Hungary

Structural Design:
Jozsef Almasi (CAEC), Budapest, Hungary (concrete)
Szalai Laszlo, Inokai Zsolt, Rezeli Csaba (RUTIN), Dombovar, Hungary (structural steel)

General Contractor:
Bouygues Batiment International, France, Bouygues Hungaria Kft., Hungary

Dimensions (L × W × H):
165 m × 120 m × 30 m

Capacity (person):	12 500
Total surface (m^2):	42 640
Concrete (m^3):	25 000
Steel (t):	1600
Total cost (EUR million):	100
Service date:	March 2003