Atrium of the Hotel Hesperia Tower, Barcelona, Spain

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Summary

The new Hotel Hesperia Tower has recently been inaugurated in Barcelona. This paper describes the design and construction of its glazed atrium, focusing on the prestressed bracing system which guarantees its stability. Several details are also described, emphasizing the possibilities of modern steel milling technologies to fabricate complex steel parts.

Introduction

The new Hesperia hotel in L’Hospitalet de Llobregat (Barcelona) is composed of two separate buildings: a tower measuring 112 m in height containing the guest rooms and all the services connected with the hotel, and a convention centre (Fig. 1).

The atrium is a 24 m high glazed volume covering an area of 30.5 x 20 m which connects the two buildings. This diaphanous space formed without a single column is used as a hall providing access to the conference rooms and the auditorium. The atrium is composed of two façades and a tilted roof which spans from the 6th floor of the tower to the roof of the convention centre.

Structure

The structure of the two façades and the roof of the atrium is a steel grid made of thin-walled rectangular hollow sections ranging from 60 x 60 x 3 mm to 250 x 150 x 8 mm. As no internal columns exist, the stability of both the roof and the façades relies on an internal bracing system (Fig. 2).

The bracing system consists of several tension-compression members connecting the façades and the roof to three rigid points (steel nodes) floating in space 12 m above the ground. A 139.7 x 10 mm CHS (circular hollow section) shaft of steel grade S355 (i.e. having a yield stress of 355 N/mm²) is used to join the three nodes. The tension-compression members are also circular hollow sections of steel grade S355 ranging from 139.7 x 6 mm to 219.3 x 7 mm. Four sets made up of three 39 mm diameter tension rods of steel grade S460 prestressed between 82 kN and 242 kN are used to keep the floating nodes in position. The lateral nodes fix one set each, while the other two sets are connected to the central node.

The design of the two lateral floating nodes was a challenge as they connect a number of members which must meet in the same spatial point without any eccentricity. The Architect also...
advised that the nodes should be as small as possible.

These conditions resulted in a design consisting of a front shield milled from a solid block of steel grade S355, on which nine façade bracing members are connected. The body of the node, which is built from a short piece of 273 × 12.5 mm CHS, is welded to the back of the shield. Three roof bracing members are connected to the upper surface of the body, while the shaft between the nodes is connected to the end cap of the body. Finally, the three tension rods used to hold and stiffen the node are connected to the body through fork terminals and steel plates (Fig. 3).

The nodes at both ends of the ridge also presented a considerable challenge. These two nodes connect a number of thin-walled rectangular tubes to two tension rods which might bear forces between 400 kN and 500 kN each under the most severe wind conditions. They were also milled from 450 Kg solid blocks of steel grade S355, and their design guarantees both a smooth transmission of forces from the tension rods to the façade and roof grids and a discreet appearance (Fig. 4).

**Construction**

The façade and the roof steel grids were divided into several modules which were built in the factory and then transported to the site. The typical façade module measured 6 × 2.5 m while the biggest module for the roof measured 7.9 × 3.8 m.

The modules were then assembled and welded on site while supported by a three-dimensional scaffolding which covered the whole area of the atrium and provided both temporary support to the structure and access for the assembly team. The steel grid was connected to the scaffolding in exactly the same points which would later be connected to the internal bracing system. The roof grid was assembled with a slight upward curvature in order to keep the roof as flat as possible for the time the glass cladding is installed.

The next step was to create the three rigid points which would support the façade and the roof grids. To do this, the floating nodes were placed in their prescribed positions and bolted to the 139.7 × 10 CHS shaft. The twelve tension rods were then installed and prestressed.

All tension rods had an in-line turn-buckle situated in their mid point to permit prestressing by means of hydraulic jacks. They were also instrumented with strain gages to control axial forces with the required accuracy.

An instrumented section situated at a distance of 1300 mm from the fork terminal fixing the rod to the floating nodes was chosen for its relatively easy access for both the installation of the electric wiring and inspection during the prestressing operation.

Two 90° double grid strain gages were installed on each instrumented section in order to form a six wire full Wheatstone bridge. This configuration makes the strain measure independent from the length of the electric wiring, temperature and any small flexure of the rod. An instrumented piece of rod 850 mm long was calibrated on a universal testing machine to determine the actual correlation between axial forces and measured strains.

The prestressing operation was carried out in six steps (Figs. 5 and 6):

**Preparation**

Assembly of the floating nodes and the tubular shaft between the nodes. The nodes were placed approximately in the position required in Step 1.

**Step 1**

Assembly of the six tension rods fixed to the lateral floating nodes. Progressive prestressing of these rods using a dynamometric wrench until the:

- desired ratio of prestresses between the three tension rods of each set was reached;

![Fig. 6: The prestressing operation](image-url)
- prestresses of the two sets of tension rods were symmetrical;
- most stressed rod of each set had reached a tension of approximately 40 kN and;
- nodes were in the required position P1.

Step 2

Prestressing of the two tension rods connecting the lateral floating nodes to the ridge by means of hydraulic jacks. Geometrically, the six tension rods achieved the desired prestresses and the floating nodes moved upwards until reaching position P2. The central floating node was lifted manually so that the shaft between nodes remained straight during the operation.

Step 3

Assembly of the six tension rods fixed to the central floating node. Progressive prestressing of these rods using a dynamometric wrench until the:
- desired ratio of prestresses between the three tension rods of each set was reached;
- prestresses of the two sets of tension rods were symmetrical;
- most stressed rod of each set had reached a tension of approximately 40 kN;
- axial forces in the rods prestressed in Steps 1 and 2 had not been altered and;
- central node was in the required position P3.

No significant displacement of the lateral nodes was detected.

Step 4

Prestressing of the two tension rods connecting the central floating node to the ridge by means of hydraulic jacks. Geometrically, the six tension rods achieved the desired prestresses and the central node moved upwards until reaching position P4.

Step 5

Assembly of the traction-compression members which connect the floating nodes to the façade and roof grids. The structure was then released from its connections to the scaffolding and it became self-supporting. The weight of the steel structure changed the distribution of forces in the tension rods and the floating nodes moved downwards to position P5.

Fig. 7: Internal view of the atrium

Step 6

Installation of the glass cladding. The weight of the cladding together with the weight of the steel structure compensated the initial deformation of the roof grid which became reasonably flat. The floating nodes moved downwards to their final position (Fig. 7).

The key stages of the prestressing process (Steps 1 to 4) were carried out on two consecutive days. The team started working early in the morning so that Steps 2 and 4 were finished by about 17 hrs, when the ambient temperature was close to 25°C (as considered in the calculation model) and the structure had been protected from the direct radiation of the sun for more than three hours thanks to the shadow cast by the tower.

Cladding

The cladding of the atrium roof consists of 698 m² of insulating glass panels measuring 3.2 × 2 m and composed of an external 10 mm toughened glass component with Heat Soak Test and a magnetronic coating on the inside face, a 12 mm air chamber and an internal 44.2 laminated glass component. The performance of the panels is defined by a solar factor of 37.7%, a visible light transmittance of 64.9% and a U-value of 1.466 W/m²K.

The cladding of the two façades consists of 640 m² of insulating glass panels measuring 2.6 × 2 m with a similar composition to the roof panels except for the external glass component which is 6 mm thick.

Both the roof and the façade glass panels are supported by a system of small aluminium profiles bolted to the steel structure at between 500 and 600 mm centres through 5 mm thick plastic spacers.

A set of textile sails is to be installed in the atrium in the near future. The position and orientation of the sails was carefully studied during the design stage to provide solar control and therefore the required comfort conditions.

Conclusion

This paper has presented a detailed description of the structural criteria applied in the atrium, and its construction procedure. Careful consideration of the construction procedure during design was crucial in order to successfully finish the work.

Special care was taken during the project to design and fabricate all supports and connections according to the calculation models. The floating nodes are good examples of this aim.

To conclude, the authors would like to make special mention of the possibilities of modern steel milling machinery combined with CAD/CAM/CAE tools, which offered in this case both accuracy and flexibility when milling the complex three-dimensional forms of both the floating and ridge nodes.

Acknowledgements

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