Atrium of the Hotel Hesperia Tower, Barcelona, Spain

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Summary

The new Hotel Hesperia Tower has recently been inaugurated in Barcelona (Spain). This paper describes the design and construction of its glazed atrium, focusing in the prestressed bracing system which guarantees its stability. Several details are also revised, 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 112m in height containing the guest rooms and all the services related to the hotel, and a convention centre which also contains the head office of the Hesperia chain (Fig. 1).

The atrium is a 24m tall glazed volume covering an area of 30.5x20m which connects the two buildings. This diaphanous space with not 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.

Fig. 1: Global view of the hotel during construction.

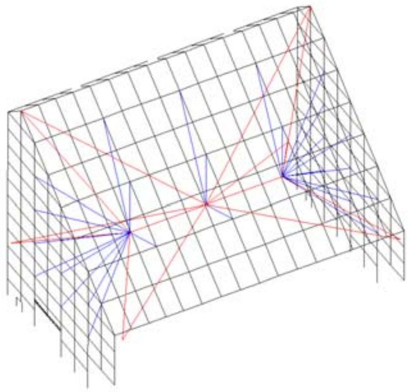

Fig. 2: Steel structure. Tension rods and central shaft in red. Tubular struts in blue.

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 60x60x3mm to 250x150x8mm. 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 in several tension-compression members connecting the façades and the roof to three stiff points (steel nodes) floating in the space 12m above ground. A Ø139.7x10mm tubular shaft of steel grade St-52 is used to join the three nodes. The tension-compression members are circular tubes of steel grade St-52 ranging from Ø139.7x6mm to Ø219.3x7mm. Four sets made up of three Ø39mm tension rods of steel grade S460 prestressed between 82kN and 242kN 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 gather a number of members which must meet in the same spatial point without any eccentricity. The Architect also advised the nodes to be as small as possible.

These conditions resulted in a design consisting in a front shield milled from a solid block of steel grade St-52, on which nine façade bracing members are connected. The body of the node, which is built from a Ø273x12.5mm short piece of steel tube, 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 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).

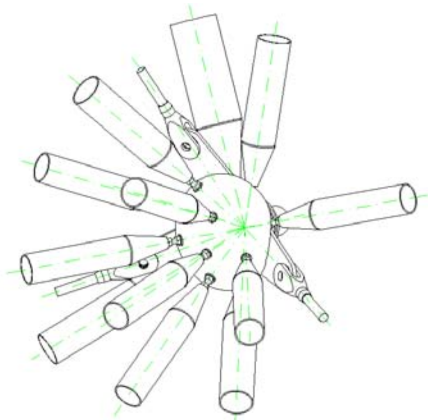

Fig. 3: Lateral floating node

The nodes at both ends of the ridge were also an important challenge.

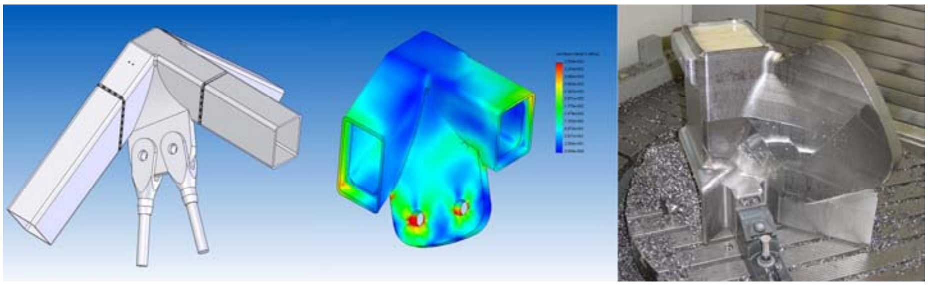

Fig. 4: Ridge node.

These two nodes connect some thin-walled rectangular tubes to two tension rods which might carry forces between 400kN and 500kN each under the worst wind conditions. They were also milled from 450 Kg. solid blocks of steel grade St-52, and their design guarantees both a smooth transmission of stresses from the tension rods to the façade and roof grids and a discrete 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 6x2.5m while the biggest module for the roof measured 7.9x3.8m.

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

The next step was to create the three stiff points which would support the façade and the roof grids. To do so, the floating nodes were placed in their prescribed positions and bolted to the Ø139.7x10mm tubular shaft. The twelve tension rods were then installed and prestressed.

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

An instrumented section situated at a distance of 1300mm 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.

Fig. 5: The atrium just before being released from its connections to the scaffolding.

Two 90º double grid strain gauges were installed on each instrumented section in order to form a 6 wire full Wheatstone bridge. This configuration makes the strain measure independent from the length of the electric wiring, temperature and any small flexion in the rod. An instrumented piece of rod 850mm long was calibrated in a universal testing machine to get the actual correlation between axial forces and measured strains.

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

Step 0:
Assembly of the floating nodes and the tubular shaft between nodes. The nodes were placed approximately in the position required in step 1.

Step 1:
Assembly of the 6 tension rods fixed to the lateral floating nodes. Progressive prestressing of these rods using a dynamometric wrench until (a) the desired ratio of prestresses between the 3 tension rods of each set was reached, (b) the prestresses of the two sets of tension rods were symmetrical, (c) the most stressed rod of each set had reached a tension of approx. 40 kN and (d) the 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 6 tension rods got 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 kept straight during the operation.

Step 3:
Assembly of the 6 tension rods fixed to the central floating node. Progressive prestressing of these rods using a dynamometric wrench until (a) the desired ratio of prestresses between the 3 tension rods of each set was reached, (b) the prestresses of the two sets of tension rods were symmetrical, (c) the most stressed rod of each set had reached a prestress of approx. 40 kN, (d) the axial forces in the rods prestressed in steps 1 & 2 had not been altered and (e) the central node was in the required position P3. No significant displacements of the lateral nodes were detected.

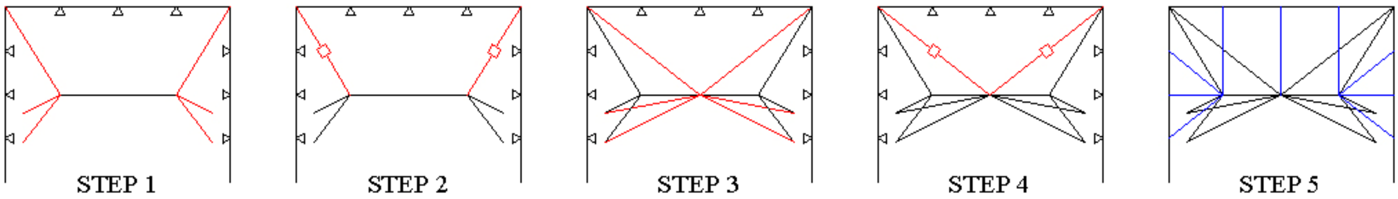

Fig. 6: The prestressing operation

Step 4:
Prestressing of the two tension rods connecting the central floating node to the ridge by means of hydraulic jacks. Geometrically, the 6 tension rods got 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 self-weight of the steel structure changed the distribution of stresses in the tension rods and the floating nodes moved downwards to position P5.

Step 6:
Installation of the glass cladding. The weight of the cladding together with the self-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 P6 (Fig. 7).

The key stages of the prestressing process (steps 1 to 4) were carried out in two consecutive days. The team started working early in the morning so that steps 2 and 4 were finished at about 5 p.m, when the ambient temperature was close to 25ºC (as considered in the calculation model) and the structure had been protected from the direct radiation of the sun for more than 3 hours thanks to the shadow projected by the tower.

Cladding

The cladding of the atrium's roof consists in 698m^2 of insulating glass panels measuring 3.2x2m and composed of an external 10mm thoughened glass component with Heat Soak Test and a magnetronic coating on side #2, a 12mm 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/m^{2}K.

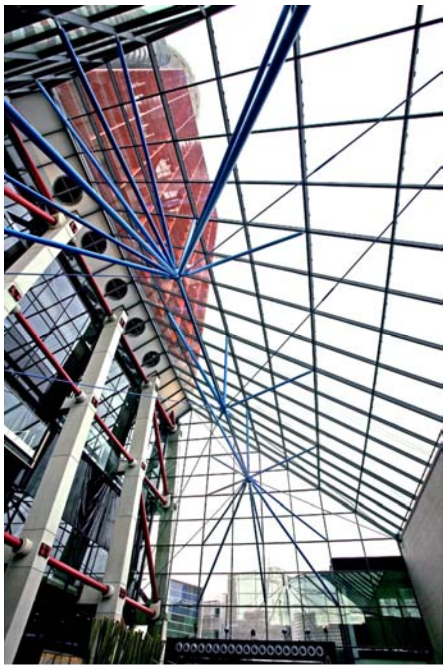

Fig. 7: Internal view of the atrium.

The cladding of the two façades consists in 640m^2 of insulating glass panels measuring 2.5x2m with a similar composition to the roof panels except the external glass component which is 6mm thick.

Both the roof and the façade glass panels are supported by a system of small aluminium profiles bolted to the steel structure every 500÷600mm through 5mm 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.

Conclusions

This article has presented a detailed description of the structural solutions applied in the atrium, and its construction procedure. A careful consideration of the construction procedure during design was crucial 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.

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

SEI Data Block	
Owner: Hoteles Hesperia	
Architect: Richard Rogers Partnership Alonso, Balaguer y Arq. Asociados	
Conceptual design: Buro Happold Brufau, Obiol, Moya y Asociados	
Engineering and construction: Bellapart S.A.U.	
Atrium steel (t):	40
Atrium glass (m^2):	1338
Service date:	February 2006