Seismic isolation for a wide span glazed roof

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Abstract

The roofing of the Crowne Plaza Hotel hall, located in Caserta (Italy), took into account the constraints given by the structural supports to be laid on the top of four existing buildings. The earthquake expected PGA is 0.25g. In order to avoid the important coupling of different structural parts while taking into account the dynamic behaviour of all them, the solution for correctly supporting the roof was found by seismically isolating the roof from the top of the existing buildings. The structural conceptual design included the construction method to be effected by incremental launching of the space frame with the glass panes already fixed. The results of the analysis, the detailing of the structure and the construction method are illustrated.
Keywords: seismic isolation, glazed roof, design, erection.

1 Structural conceptual design

The glazed roof features a sub square plan with side dimensions of 58.52 m and 58.42 m; because of architectural requirements, the roof was shaped as a shallow lamella dome with a maximum rise at the crown of 6.45 m; see fig 1.

The Owner is Progetto Industrie s.p.a, Caserta, the Architect was Gianmaria Beretta, SABA srl Milano, Italy and the Constructor was MAEG, Treviso, Italy.

The expected seismic design displacement of the top of the contour in the existing buildings is around ± 0.30 m and therefore the seismic isolation of the lamella dome was assumed as the conceptual way to overcome the problem of imposing such huge displacements on the superimposed structure.

A system of two way sliding bearings on the top of three of the contour buildings isolates the lamella dome from the relevant horizontal displacements, while a stiff connection to the fourth one resists the horizontal actions and the torque created by the inertial forces of the superimposed structure.

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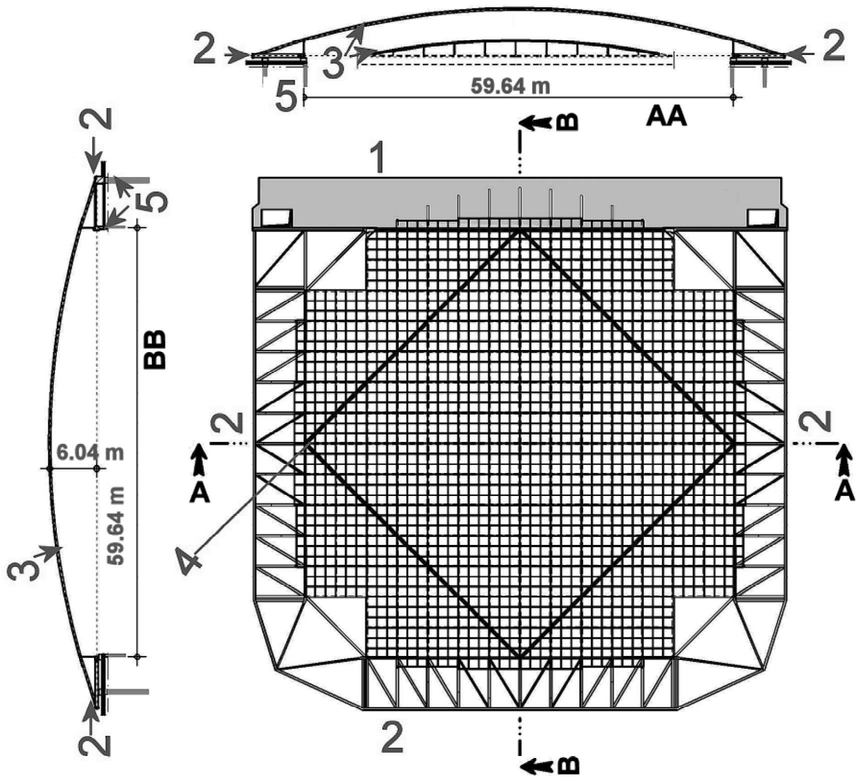

Figure 1: Dimensions and structural parts of the lamella dome. 1 – Pre stressed concrete beam over one of the existing buildings. 2 – Horizontal trussed frame over the existing buildings. 3 – Orthogonal grid of arches. 4 – Arch crown-connecting cables. 5 – Two way sliding seismic isolators.

In this case, contrary to the normal seismic isolation principles, the bearing system has to develop a negligible stiffness thus limiting to small friction forces the dynamic coupling of both the supporting and supported structures.

The low friction PTFE sliding bearings were designed for the total stroke of 0.60 m in every horizontal direction, see fig. 2.

Similar gaps were prescribed at the intersections of the dome structure with the walls and ducts protruding from the top of the existing buildings.

The dome geometry was shaped as a discrete surface composed of flat facets, suitable for bearing standardized glass panes.

The skeleton is composed of two groups of orthogonal segmental arches, stiffened by diagonal bars connecting the intersections; the tubular steel arches support a secondary structure laid at 1.413 m by 1.413 m centres which bear the safety stratified glass panes.

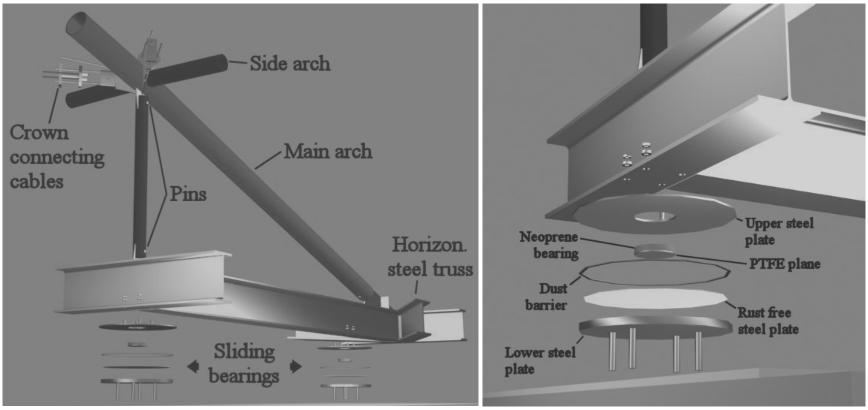

Figure 2: Scheme and detail of the sliding isolators.

A contour frame, having the same side dimension of the roof and laid in the horizontal plane, has the function of equilibrating the arch thrusts and of transferring, by means of two way sliding bearings, the vertical loads to the columns of the existing buildings.

This frame is constituted by trussed steel beams along three sides and of a prestressed concrete beam along the fourth one where the structure is anchored against the earthquake actions.

2 Structural analysis

The lattice dome had to be studied along with the whole building, which provides anchoring against inertial forces, but, on the other hand, also transmits the earthquake actions arising from ground motions. The relevant dynamic behaviour of the dome and of the anchoring building was properly detected by implementing a modal analysis based on the site acceleration spectra.

The horizontal friction forces arising at the position of the sliding bearings on the other buildings were disregarded because of their very limited values.

The P-delta effects were taken into account for the arches, which show important vertical deflections under the action of asymmetrical vertical loads.

Four post-tensioned cables, laid at 45°, connect the crowns of the side arches and increase the stiffness of the lattice dome.

The sensibility of the structure towards the stiffness of all the members was investigated and a limit to the size of the diagonal bars connecting all the arch intersections was prescribed accordingly, in order to avoid an overstressing of them originated by the arising of an alternative load carrying pattern set at 45° degrees with respect to the arches.

3 Structural detailing

The structural details of the lattice dome and the relevant engineering ones, like the fixing of the glass, could not be separated because of the limits of the allowable distortion of each pane, created by the loads and resulting from the stiffness of the continuous support on resilient pads along their edges [1].

A special attachment located at the corners prevents the pane sliding during the earthquake and the balancing of the wind suction pressures, see fig. 3.

Figure 3: Details of the earthquake resistant fixings of the glass panes.

An additional architectural requirement is the capability of the structure to support suspended loads, as for example bans, lamps and either a car (in this case attached to eight joints at the joints of the arches).

In order to achieve the necessary structural stiffness and to avoid poor architectural aspects, the joints of the arched pipes are completely welded, while the connections of the arches to the edge frame are constituted by pins, see fig.4. The diagonal stiffening bars are lightly stressed by applying a torque to force the threaded ends inside fixed nuts.

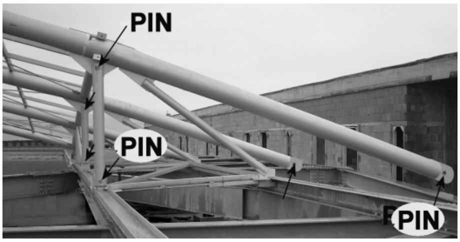

Figure 4: Arch end pin connections and welded crossings.

The steel members of the edge frame are connected by means of bolts; the joint of the chords and the diagonals to the pre stressed side of the frame is obtained by means of a direct connection to the end head of the post tensioned cable.

Figure 5: Aerial view of the dome during the launching.

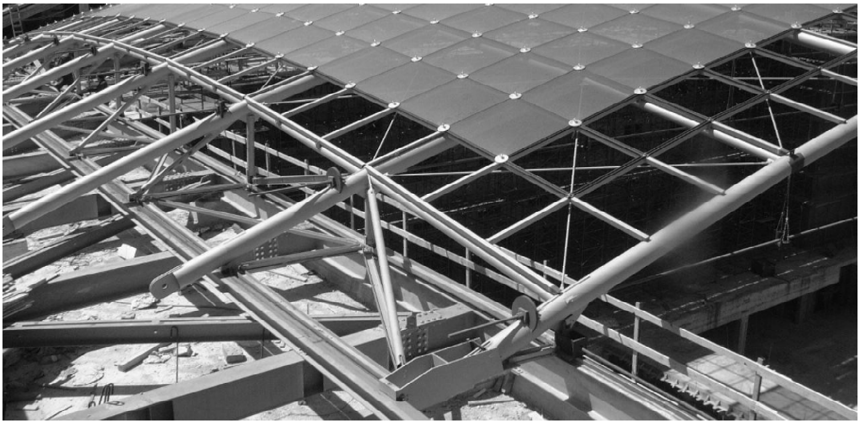

Figure 6: The dome launching in progress.

4 Erection system

The four existing buildings and the framing system, necessary on the top of them, prevented from assembling and hoisting the structure from the ground level.

The dome was designed for incremental construction to be effected by using the top of one of the buildings as a yard for assembling groups of arches capable of spanning the whole distance of around 60 m between supports and then launching these groups followed by the rear jointing of other ones and so on up to the completion of the roofing, fig 5.

During the launching the load bearing structure is constituted by single way arches and therefore the relevant horizontal thrust was balanced by provisional transverse ties; the relevant prestressing forces were tuned according to the subsequent evolution of the structural system, see fig.6.

The launching was terminated in front of the already built part of the roof standing on the anchor building [2].

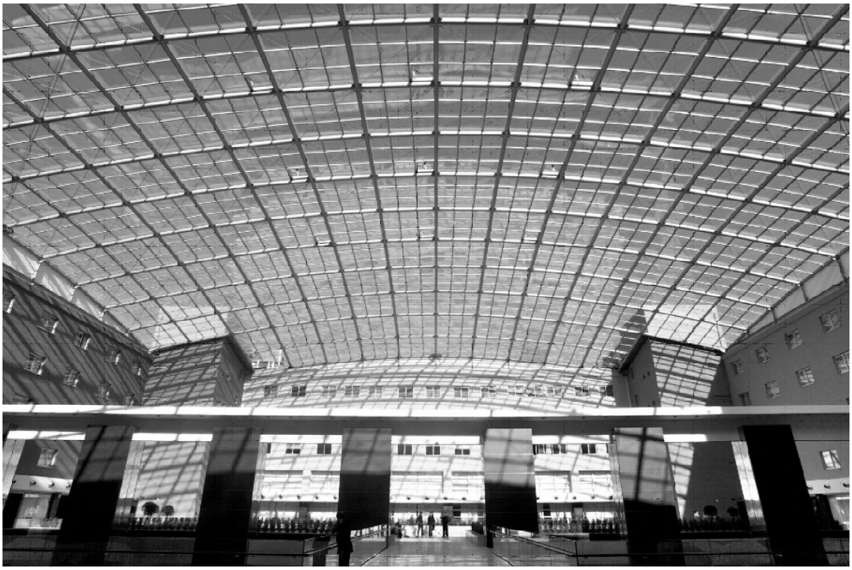

Figure 7: An interior view of the completed dome.

Figure 8: Another interior view of the completed dome.

The completed glazed dome is shown in the figures 7 and 8.

References

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