

## Structural Design of the New Football Stadium of Panathinaikos F.C. in Votanikos, Greece

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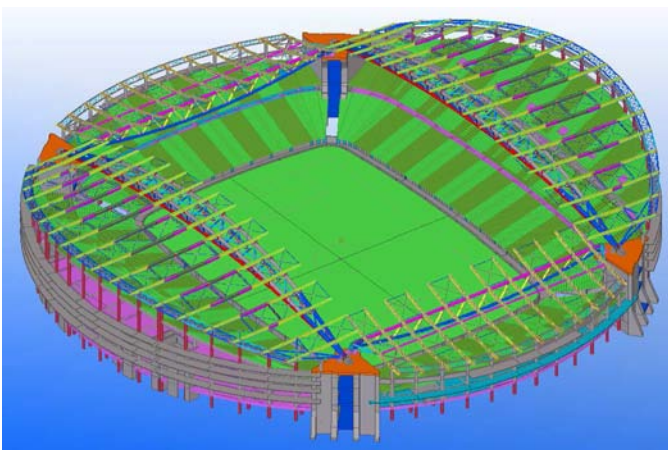
### Summary

In this paper the main structural elements of the new football stadium of Panathinaikos F.C. in Votanikos, Athens, Greece are presented, with particular emphasis on the steel roof and its interaction with the underlying reinforced concrete structures. The roof consists of four structurally independent parts, supported through main truss girders on reinforced concrete pylons and on the exterior peripheral reinforced concrete columns of the grandstands. Issues pertaining to optimization of geometry, type and size of cross-sections, supports and connections between members, in order to achieve satisfaction of architectural constraints in the most safe and cost-effective way are discussed. Appropriate decisions that had to be made at the conceptual design stage, in order to minimize the interaction of the steel roof with the pylons and the ten structurally independent grandstand structures during eventual seismic events, are described.

**Keywords:** football stadium, steel roof, conceptual design, seismic design.

### 1. Introduction

The new stadium of Panathinaikos F.C., a historic Greek football club, will be constructed in Votanikos, Athens, Greece and will have the ability to host approximately 40.000 spectators, with its grandstands being completely covered. The grandstand structures will be made of reinforced concrete and the roof of structural steel (Fig. 1). The stadium has a circular plan view with an



*Fig. 1: General layout of the stadium*

external diameter of 210m. The diagonals of the playing field divide the structure in four sectors, which are named accordingly North, South, East and West. In the East and West sectors and parallel to the playing field's longitudinal axis the "large grandstand areas" are thus defined, while in the North and South sectors, behind the goalposts, the "small grandstand areas" are defined. In each "large" area, three structurally independent buildings will be constructed, E1, E2 and E3 in the East sector and W1, W2 and W3 in the West sector. In each "small" area, two structurally independent buildings are foreseen, N1, N2 in the North sector and S1, S2 in the South sector (Fig. 2).

At the four corners of the stadium, four reinforced concrete pylons will be constructed, which are structurally independent from the grandstand buildings. The total roof structure consists of four independent parts, each of which is supported on a main space truss-girder and on the perimeter columns of the upper building levels. The four main trusses are simply supported on the four reinforced concrete pylons arrayed at the stadium corners. In accordance with the overall cylindrical

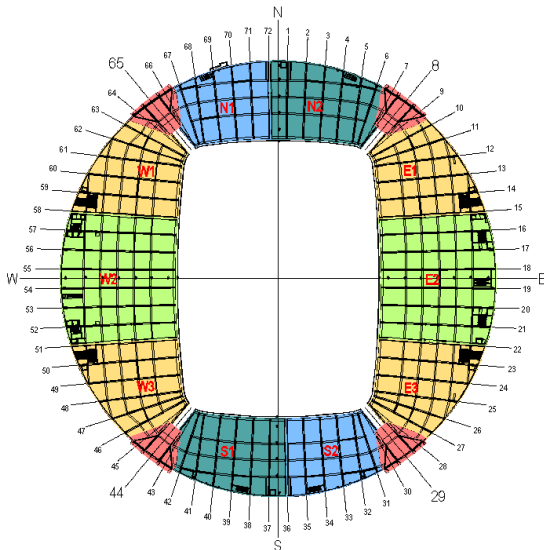


Fig. 2: Plan view and notation

shape of the roof, the two “large” main trusses are arch-shaped, while the two “small” ones have horizontal upper chords (Figs. 3 and 4). Due to the arch action of the “large” main trusses as well as the seismic loading situations, significant horizontal forces are applied from the steel roof at the pylons’ tops, thus large sections are required for the pylons’ walls, and a stiff pile foundation is provided for each pylon.

The “large” main truss-girders, spanning 160m, have a triangular section with one bottom chord and two upper chords, connected to each other with appropriate diagonal members. The distance between chords varies along the length, with maximum vertical and horizontal distance in the middle, equal to 9m and 8m, respectively, and all three chords converging towards one theoretical point at the two supports, in order to create simply-supported conditions. All main truss members have circular hollow sections with maximum sections for upper and lower chords CHS1250/28 and CHS1300/32,

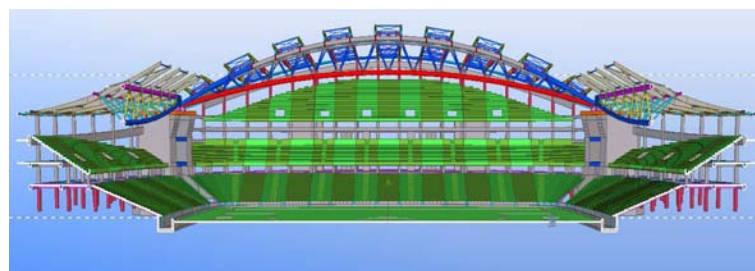


Fig. 3: Section along North-South axis

respectively. Secondary beams are supported on the upper chords of main trusses and on top of the outer perimeter columns of the grandstand buildings, arrayed parallel to the East-West axis of the stadium (perpendicular to the main truss). These beams are cantilevering on both ends, into the field and towards the stadium surrounding area. They are constructed with built-up sections with two webs, for providing increased torsional stiffness, and protruding flanges for ease of connections. The web and flange dimensions vary along the length, with maximum sections 1200/20 and 500/35, for web and flange, respectively. Purlins of standard I-sections, hinged on one end and free to slide in the longitudinal direction on the other, to avoid axial forces, are supported on the secondary

beams and carry the roof cladding. Secondary beams are also connected by appropriately located auxiliary vertical trusses and x-bracing that ensure their lateral stability. Rolling supports of secondary beams on the perimeter columns are configured, in order to minimize interaction between steel roof and underlying grandstand structures during earthquakes.

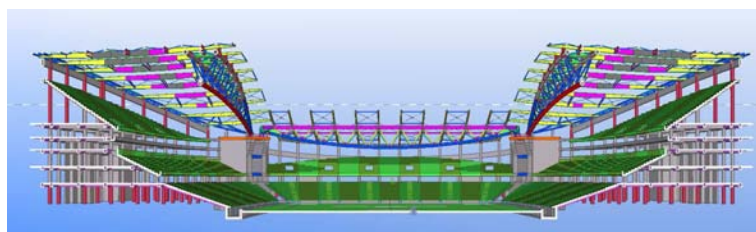


Fig. 4: Section along East-West axis

Likewise, each “small” roof is supported on a main space truss-girder with a triangular section, spanning 108m. The roof has an inclined surface towards the external part of the stadium and is slightly curving upwards. This main truss is also formed by inclined diagonals and transverse members on the upper chord. The secondary beams, the purlins and the bracings of the small roof structures are of similar arrangement as in the large ones. The supports of secondary beams on the perimeter columns are also shaped as simple rollers for minimizing seismic interactions. Although the small roof structures have a significantly smaller span than that of the large structures, their geometry, which is based on the architectural requirements, results in some different structural problems:

(a) The almost straight and horizontal axis of the main truss results in the

development of significant forces on the roof as well as on the pylons, due to temperature variations. This problem is avoided in the large roofs due to their curved shape.

(b) The length of the cantilever of the secondary beams is disproportionately large compared to the intermediate simply supported part, between the main truss and the perimeter columns.

## 2. Roof Structural System for Resisting Horizontal Loads

As already described, each “large” steel roof structure is supported by five structurally independent reinforced concrete structures, two pylons and three buildings. Similarly, each “small” steel roof structure is supported by four structurally independent reinforced concrete structures, two pylons and two buildings. In a region with significant seismic activity, such as Athens, this raised grave concerns about possible adverse interactions between steel and concrete parts during seismic events. Two issues were of particular concern in that regard:

- Due to the much higher mass of the grandstand structures with respect to the steel roof, the inertia of the reinforced concrete part might induce large actions on the steel part, in other words, during a seismic event the roof would not be supported on the buildings, but the other way around.
- Eventual out-of-phase motions of the buildings and pylons might also induce severe actions on the roof.

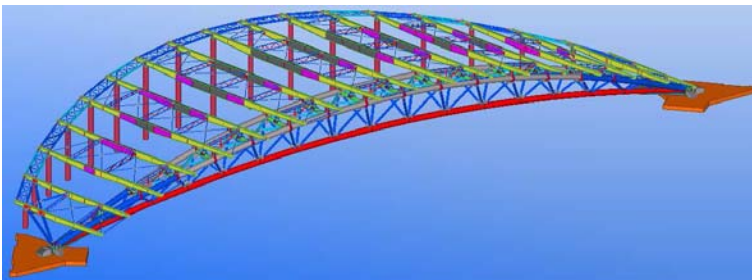


Fig. 5: Large steel roof

Of additional concern was the fact that such phenomena could not be easily and reliably captured by simulation models during structural design. Therefore, it was decided to minimize the interaction between structural parts, thus increasing confidence in the predictions of the structural models with respect to anticipated behavior. For that reason, supports of the roof on perimeter concrete columns were

configured as rollers, allowing free motion in both horizontal directions. Thus, seismic interaction was restricted to steel roof and concrete pylons, excluding grandstand buildings.

This way, issues (a) and (b) above are fully addressed as far as the grandstand buildings are concerned. Moreover, the curved shape of the roof acts beneficially in case of differential motion of the two pylons. The disadvantage of this solution is the reduced stiffness of the roof structure against horizontal loads. Taking into consideration this fact, it was decided to construct a horizontal “ring” peripheral truss at the back end of secondary beams. As shown in Figs. 5 and 6, this truss has a curved shape following the geometry of the stadium, in plan view. All truss members have circular hollow sections. This truss and the main space truss-girder towards the interior area of the stadium, constitute two large stiffness zones at the two sides of the roof. Considering the pairs of adjacent secondary beams connected by roof bracings to act as stiff beams in the plane of the roof, shown in green color in Fig. 6, a Vierendeel truss is created, providing sufficient stiffness against horizontal loads. A pair of stiff tubes connect the ends of the peripheral truss with each pylon, thus linking the two stiff zones and accomplishing proper transfer of forces due to horizontal actions to the pylons and through them to the foundation.

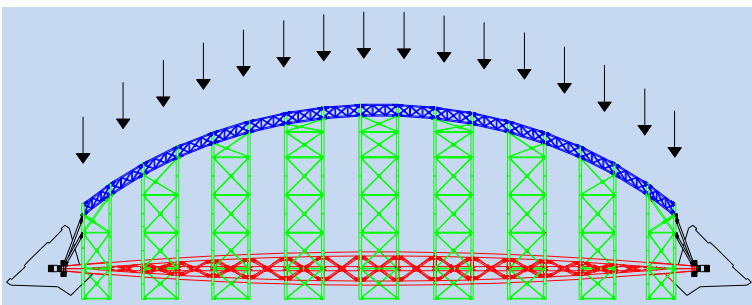


Fig. 6: Steel roof acting horizontally as a Vierendeel truss

Small roofs resist horizontal loads in a similar way, with one difference. Because of their rather flat shape, temperature variations as well as differential seismic motion of the pylons would create significant stresses in the steel roof. This was addressed by configuring one support of the main truss on the pylon as hinge and the other as roller in the longitudinal main truss direction, as opposed to the corresponding supports of main trusses in the large roofs, which have both been configured as hinges. This relieves axial stresses in the main truss due to the two loading situations mentioned above, at the expense of directing all seismic actions on small roofs in the East-West direction on only two among the four corner pylons.



### 3. Roof Structural System for Resisting Vertical Loads

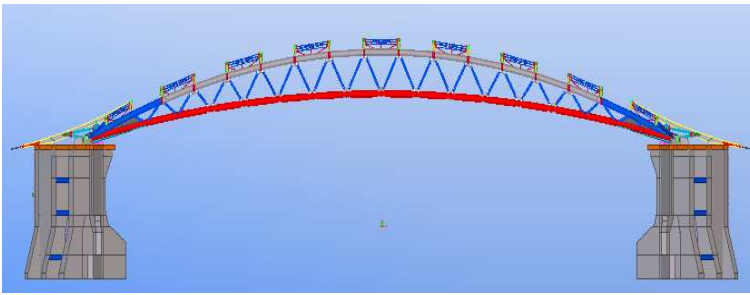


Fig. 7: Side view of large steel roof and pylons

In this section, the main issues of concern regarding the structural system against vertical loads are described. The behavior of the large roof structure depends heavily on the pylons' rigidity. Due to the arch action of the large main trusses, significant horizontal forces are exerted on the pylons' tops due to vertical loads (Fig. 7). For the arch action to be successful, the horizontal displacements of the pylons should be minimized. This

resulted in high stiffness requirements for the pylons, thus to rather thick pylon walls.

Nevertheless, the fact that a large percentage of horizontal forces on the pylons are due to dead loads, which act permanently, caused concerns of creep for the reinforced concrete pylons and for their pile foundation and surrounding soil. Therefore, it was decided to allow the main truss to roll freely on pylon tops during erection in the longitudinal direction, until all permanent loads were in-place, and then "lock" the supports, so that they act as hinges for subsequent live loads. The deformation of the large steel roof under permanent vertical loads is shown schematically in Fig. 8.

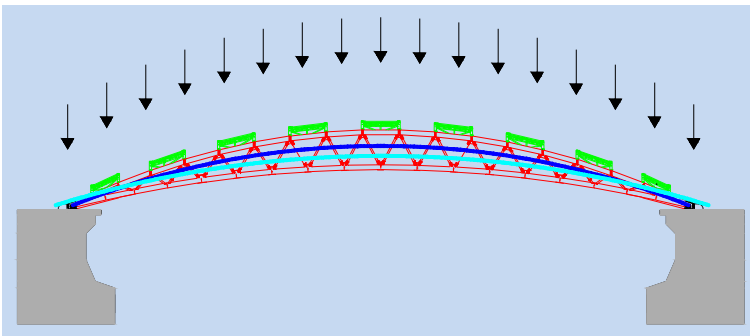


Fig. 8: Large steel roof deformation under vertical loads

The blue line corresponds to the center line of the main truss-girder in its undeformed shape, while the cyan line represents its deformed configuration. Live loads are not expected to create creep problems, as they remain in-place for a short period of time, during which they are also of a smaller magnitude than permanent loads. For materializing this conceptual design decision, significant difficulties were encountered with respect to connection detailing and erection procedure, some of which are outlined in section 4.

An additional uncertainty in the design process is related to the soil characteristics, particularly in the vicinity of pylon foundations. While more conventional strip foundation has been employed for the grandstand buildings, for each pylon 50 1.2m diameter piles have been used, driven to 25m depth below the bottom of the pylon (Fig. 9). As mentioned above, the arch action, which is essential for the proper structural behavior of the roof, depends a lot on the minimization of horizontal displacements at the top of the pylons. These displacements, in turn, result as a combination of pylon rigidity and rotation at the bottom of the pylon, induced by deformation of the

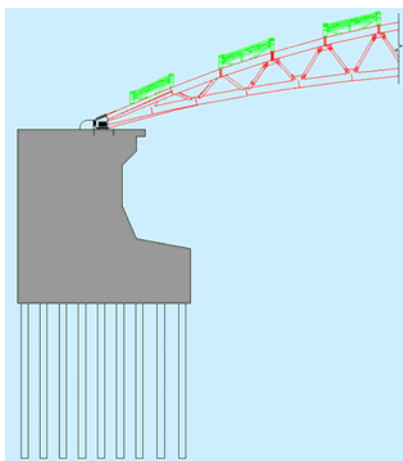


Fig. 9: A pylon and its piles

pile-soil system. Clearly, the primary factor affecting this rotation is the stiffness of vertical and horizontal soil springs, representing soil-structure interaction. In turn, this depends on soil properties from which soil spring stiffnesses are obtained.

An additional consideration is the fact that for the roof itself the worst case results from a soft pylon-piles-soil system that produces larger displacements on the top of the pylon, while the opposite is the case for the pylon and the piles. Although, the soil characteristics were provided by the geotechnical investigation, and the horizontal reaction forces on top of the pylon due to permanent loads were eliminated, as explained above, it was nevertheless decided to consider an envelope of possible soil stiffness, and decide on cross-sections and reinforcement of roof and pylons for the correspondingly worse cases.

In order to evaluate the influence of soil stiffness, and to decide

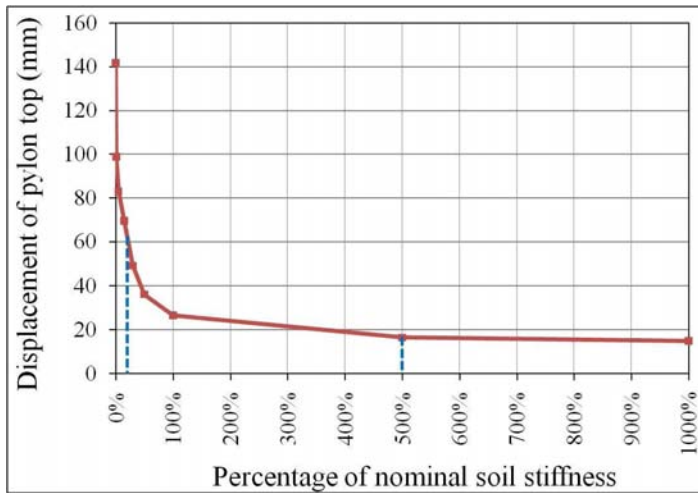


Fig. 10: Horizontal displacement of pylon's top for various values of soil stiffness

about upper and lower bounds to be used for structural design of roof, pylons and piles, a wide range of analyses on the structural model was carried out, for different values of the soil subgrade reaction modulus. The results of these analyses are shown in the chart of Figure 10, where the horizontal displacement of the top of the pylon is plotted on the vertical axis for a typical vertical loading case, while on the horizontal axis the soil stiffness is plotted with respect to the nominal one, obtained from the geotechnical investigation. Based on this figure it was decided to consider 20% of the nominal soil stiffness as a lower bound, for which the steel roof has been analyzed and dimensioned. Similarly, 500% of the nominal soil stiffness has been used as a

upper bound, for which the pylons and their pile foundation have been analyzed and dimensioned. Thus, it was ensured that the influence of uncertainty of soil properties in the behavior of the stadium structure to vertical loads is minimized.

#### 4. Basic Connections

In this section, some of the basic connections designed for satisfying the requirements outlined in the earlier sections are presented and discussed. Because of the complexity of the structure, several connections are not typical. Therefore, figures showing the connections in full as well as their parts are shown, in order to enable comprehension of the way in which the structure is erected, and for appreciating how the connections function in the desired way.

##### 4.1 Support of the space truss-girders on the pylons

As described above, the support of the main space truss-girders on the pylons is crucial for the overall behaviour of the roof. The three chords of the large main truss and the two chords of the small one, converging theoretically to a single point at the support, result in rather long lengths along which intersections between the tubes are encountered. Moreover, appropriate stiffener plates are placed, in order to guarantee monolithic behaviour near the support (Fig. 11). In compliance to this, moment transfer capability between the chords is allowed in the analysis models.

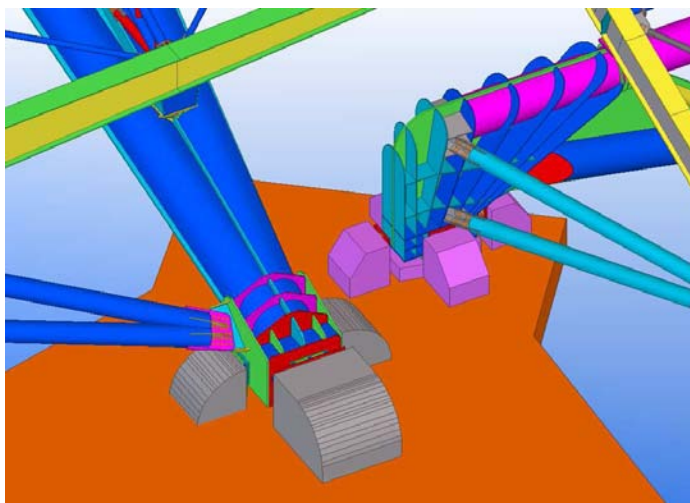


Fig. 11: Support of the large and small space truss-girders on the pylons.

The large main truss should be free to slide on the pylon in its longitudinal direction under permanent loads. The magnitude of anticipated horizontal displacement at the support points is in the order of 20cm, while the corresponding vertical deflection in the middle is approximately 60cm. Then, the horizontal degree-of-freedom at the two ends of the truss should be blocked and the support should behave as a hinge for live loads. This is achieved by the set-up shown in Fig. 12. A sliding surface is created between a base plate anchored on the pylon and a corresponding plate welded on the bottom of the converging main truss chords. Horizontal displacements in the transverse main truss direction are prevented by means of bearings attached to massive concrete blocks projecting from the pylon top on

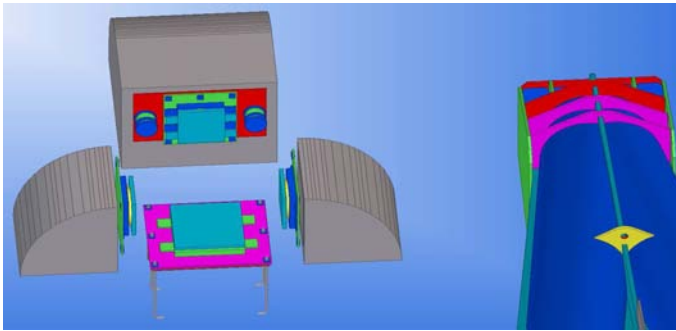


Fig. 12: Support of the large space truss-girder on the pylons (separate parts).

both sides of the main truss. These bearings also control rotation about the vertical axis and about the horizontal axis in the longitudinal direction of the truss. The same technique is employed for the longitudinal main truss direction after sliding due to permanent loads is completed. A concrete block and a corresponding bearing prevent further horizontal displacements and control rotation about the horizontal axis in the transverse direction of the truss. Jacks, located on the two sides of the bearing at both ends of the main truss ensure that sliding due to permanent loads is equally

divided between the two pylons, so that symmetry is retained and corresponding horizontal displacements of secondary beams at their supports on peripheral columns are as anticipated. A 3D finite element model of the entire roof, in which the area of the support has been simulated with a dense and detailed mesh of shell elements, connected with appropriate rigid elements with the rest of the roof, modelled with beam elements, has been used for estimating local stresses and verifying satisfactory local behaviour of tubes and stiffeners.

A similar arrangement has been used for the small main truss (Fig. 11). Due to geometry and constructional factors, a larger eccentricity between upper and lower chord than in the large truss was unavoidable. Therefore, a denser array of stiffeners has been used, to prevent local buckling and ensure monolithic behaviour. The supports have been configured as a hinge on one end and as a roller in the longitudinal direction on the other.

## 4.2 Truss-girder connections

In this section typical main truss connections are presented, one for the top chord of the truss and one for the bottom chord. At a typical joint of the top chord seven members meet, as shown in Fig.

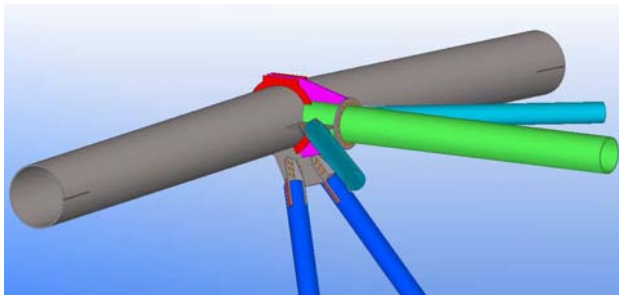


Fig. 13: Top chord connection of the truss-girder.

13, namely, the two tubes of the chord, one transverse and two diagonal members connecting the two top chords, and two diagonals connecting top and bottom chord. The two tubes of the top chord meet at an angle, in both the vertical and the horizontal plane, in order to follow the overall geometry of the truss. Their connection is achieved by means of an intermediate end plate, welded to both tubes. Diagonal members are bolted on gusset plates that are inserted into the top chord tubes through appropriately located grooves and are welded together and on the tubes.

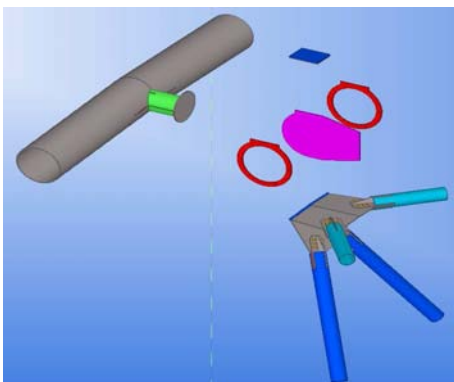


Fig. 14: Top chord connection of the truss-girder (separate parts).

Of interest is the fact that the gusset plates of the four diagonals are located in four different planes, to follow the overall geometry of the main truss. For the transverse strut between the two top chords a splice connection is employed, using end plates welded on the parts of the strut and bolted together. Two rings, located on both sides of the end plate between the two main tubes, create a nearly horizontal surface, on which the secondary beam will then "sit" (Fig. 14).

At a typical joint of the bottom chord six members meet, as shown in Fig. 15, namely, the two tubes of the chord and two pairs of diagonal members connecting the bottom chord with the two top chords. The two tubes of the bottom chord meet at an angle in the vertical plane. The diagonals are bolted to gusset plates inserted into the bottom chord tubes

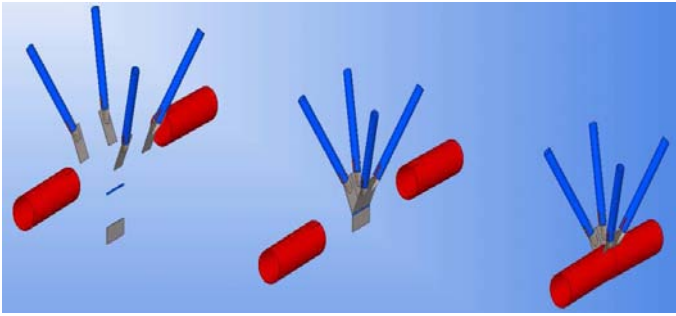


Fig. 15: Bottom chord connection of the truss-girder (separate parts).

through appropriately located grooves and welded together and on the tubes. An auxiliary rod along the axis of the bottom chords is utilized for welding the gusset plates to each other and to an additional vertical plate. The three plates also act as stiffeners, protecting the chord tubes from local buckling due to the concentrated forces exerted from the diagonal members. The two chord tubes are now directly welded to each other, without use of an intermediate end plate.

According to the proposed erection procedure, the large truss will be assembled on the ground in five parts, which will then be lifted by cranes and put into place by means of temporary towers, properly secured on the reinforced concrete structure of the grandstands. The temporary towers will be removed only after secondary beams, roof bracings and peripheral truss have been completed, which are necessary for lateral stability, due to the arch-shape of the main truss.

### 4.3 Support of secondary beams

In this section, the support of secondary beams of the steel roof on both the space truss-girder and the perimeter columns of the grandstand buildings will be described. Both connections are of pivotal importance for the correct function of the roof system during erection and in service conditions.

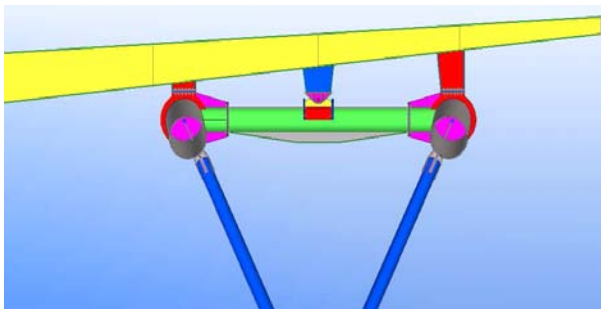


Fig. 16: Support of secondary beams on the truss-girder

As shown in Figure 16, each secondary beam is supported on the main truss at three locations, at the two joints of the top chords, and in the middle of the transverse strut. Among these connections, only the middle one is activated during the erection process, which is actually configured as a pin connection, so that the beam is free to rotate while the main truss slides horizontally on the pylons under permanent loads and deflects vertically. Thus, no moments can be transferred from the beam to the truss, which would induce undesirable torsion, rotation about the longitudinal axis of the truss, and loss of symmetry. After all

permanent loads have been applied, and the main truss has been blocked on the pylons, the two other connections are bolted (Fig. 17). Thus, the bending stiffness of the beam restricts torsional rotation of the truss, providing lateral stability and allowing removal of temporary towers. The final

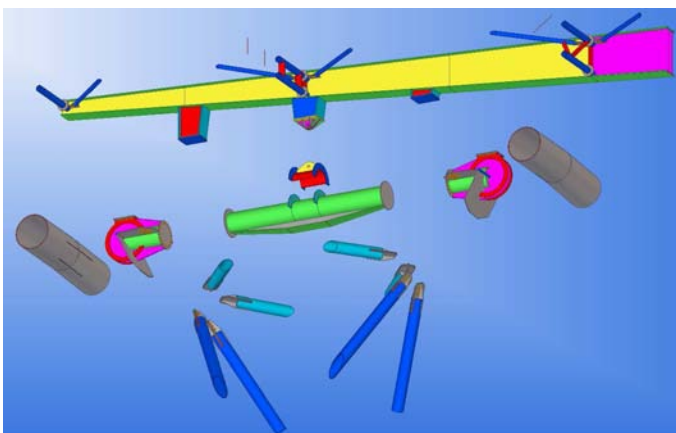
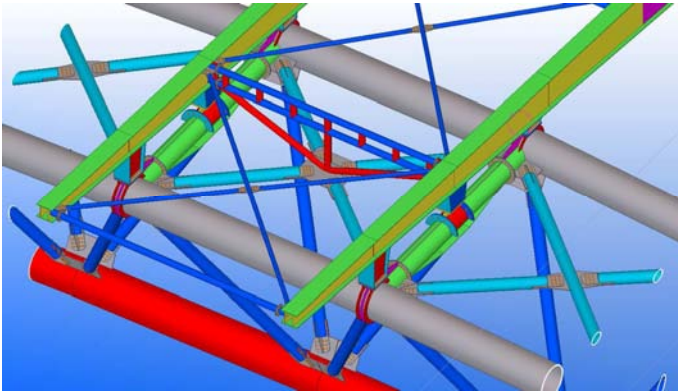


Fig. 17: Support of secondary beams on the truss-girder (separate parts)

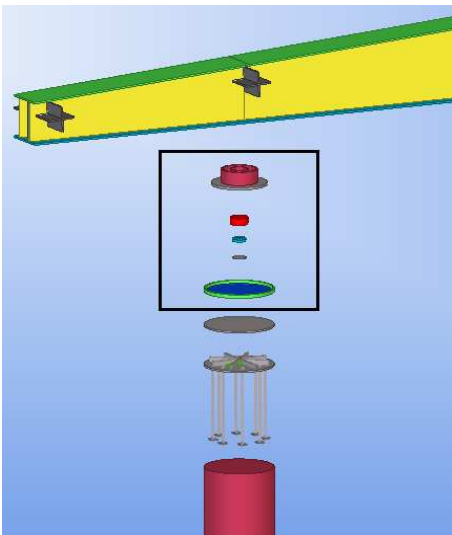
situation of two adjacent secondary beams supported on the truss-girder is illustrated in Fig. 18. Bracings as well as auxiliary members ensuring lateral stability of the secondary beams are also shown.

As described before, the support of secondary beams on the perimeter columns of the grandstand buildings should function as a roller, allowing displacements in both horizontal directions. This support is configured as shown in Fig. 19. Secondary beams are expected to displace laterally during erection under the action of permanent loads, with magnitudes varying between approximately 20cm near the pylons and diminishing towards the center of



*Fig. 18: Two adjacent secondary beams supported on the truss-girder against overall roof overturning.*

the large roof. To account for that pertinent eccentricities are given to these beams in the “assembly roof geometry”, so that no eccentricities will remain between beams and peripheral columns in the “locked” geometry. Horizontal freedom of motion is achieved by a spherical steel bearing moving in a concave surface. Even though no negative vertical reactions have been computed at these locations for any load combination, thus no danger of uplift is foreseen, additional auxiliary cables are provided between peripheral columns and secondary beams, with sufficient length to allow for horizontal deflections and rotations, but preventing uplift for increased safety



*Fig. 19: Support of secondary beams on the perimeter columns*

## 5. Summary and Conclusions

The main structural elements of the new football stadium of Panathinaikos F.C. in Votanikos, Athens, Greece have been presented, with emphasis on the steel roof and its interaction with the underlying reinforced concrete structures of grandstand buildings and corner pylons. Appropriate decisions that had to be made at the conceptual design stage, in order to minimize the interaction of the steel roof with the pylons and the ten structurally independent grandstand structures during eventual seismic events, have been described. The resulting mechanisms of resisting both horizontal and vertical loads have been outlined. The main connections between primary structural members, posing several challenges in order to comply with the previously mentioned structural function, have been schematically illustrated. Combining the above parameters, the structural design team has managed to satisfy the architectural requirements of a roof with a primarily cylindrical shape, and at the same time to meet structural safety concerns, in a region with significant seismic activity.

Preliminary, final and construction structural design for this project were performed by Design & Application Engineers S.A., Athens, Greece (Christos Gkologiannis, Charis Gantes, Alekos Athanasiadis). Consultants for construction structural design were Prof. Massimo Majowiecki, Venice, Italy, and Fotis Zoulas, Athens, Greece. Checking of construction structural design was carried out by Vienna Consulting Engineers, Vienna, Austria (Harald Schmidt).