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UPGRADING THE SPOKE WHEEL STADIUM ROOF CONCEPT

Massimo MAJOWIECKI[1], Stefano PINARDI[2], Giovanni BERTI[2] and Luca PATRUNO[1]

[1] DICAM, University of Bologna, [2] MWJ Structures, Casalecchio di Reno (Bologna)

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ABSTRACT

The roof structure of Olympic Stadium in Rome had been conceived, studied and realized between the end of 1988 and the April 1990, in total 16 months of work. This roof is the first example of spoke wheel roof structural system, that today is widely applied on many actual Stadiums around the world [1]. Wind tunnel tests on the roof structure have been carried out at the BLWT Lab. of Western Ontario University: at that time, there have been many discussions between the designer and wind tunnel researchers about the integration of wind tunnel tests data into the design process for sub-horizontal wide span roofs.
The conceptual design and the method of wind dynamic analysis has been updated for the Yaoundé roof Stadium, now under construction. The aerodynamic behavior of Yaoundé stadium roof has been tested in London RWDI wind tunnel. Innovative and recently proposed numerical techniques have been adopted to perform dynamic analyses under wind action.
Erection sequences prescribed for roofs of these two Stadiums are both remarkable and advanced: the big lift adopted procedure for the Olympic stadium roof in Rome and the new tie down pre tensioning operation, provided for the Yaoundé stadium cable roof.

Keywords: *Spoke wheel structure, wind tunnel tests, dynamic analyses, Proper Skin Modes, erection sequences, large span roof, stadium roof, non linear model*

1. ROOF OF OLYMPIC STADIUM IN ROME

1.1. History of the structural concept

The original stadium was built in the 1960 for the Olympic Games, in 1984 arose the idea of enlarging the structure needed in view of the 1990 World Football Championships.

The origin of the Rome Olympic Stadium roof covering is the result of a variant requested after the associations for the environment appealed to the regional court in January 1988 complaining against the visual impact of tendered project. In fact, the tendered solution provided for a cable suspended double layer space frame roof deck, anchored to eight concrete towers 50m high (Figure 1-a).

Under such circumstances and related time pressure, Majowiecki came up with the proposal of a flat and elegant cable ring structure (Figure 1-b): the spoke wheel stadium roof. The structural design concept was without precedent at the time being and it's a precursor of many other stadiums roof with the same or similar structural concept.

The qualities of the new covering are the following:

- Vertical bearing structures independent of those supporting the grandstands (only 12 peripheral columns);
- Very low lateral profile; important mitigating the visual impact (only 12m high);
- A closed ring structural system, without the use of structures outside the perimeter of the covering such as stays, tie – beams, counterbalances, etc..;
- Absence of horizontal forces in the supports due to vertical loads or thermal effects;
- Use a light covering system.

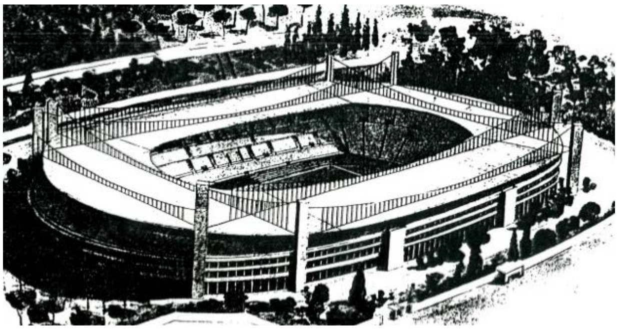

Figure 1-a: *Rome Olympic stadium new roof: tendered project*

Figure 1-b: *Rome Olympic stadium new roof: Prof. Majowiecki solution*

1.2. Description of the structure

The cable roofing system is formed mainly of [2]:

- a radial distribution of cable trusses (Figure 2-a);
- a polycentric inner tension cable ring;
- an outer anchorage system consisting of a space framed, reticular, polycentric ring.

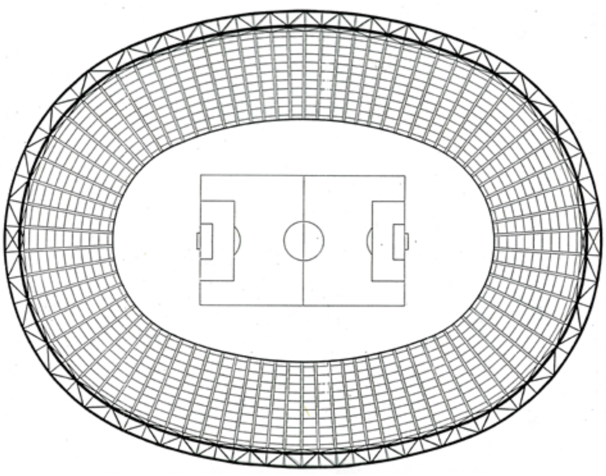

Figure 2-a: *Plan view of the stadium roof*

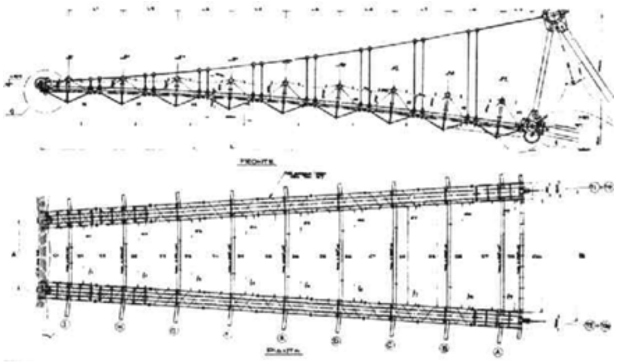

Figure 2-b: *Section of the stadium roof*

Cable trusses

The cable trusses are distributed in a radial direction from the two centers of the homothetic polycentric curve that generates all the stadium geometry. All cables are galvanized and full – lock or open spiral type, the main wires have an elementary strength of 1600 N/mm^2 (Figure 2-b).

Roof covering

The radial cable truss is combined with a secondary frame that supports the roof membrane panels: girders supported by a simple bearing system are suspended at the same levels as the stabilizing cables, forming a plane frame. The roof covering is made of fiberglass membrane coated with PTFE and is fastened to the extrados of the secondary girders.

Polycentric inner ring

The inner ring, whose main function is to balance horizontal stresses transmitted by the radial trusses within a closed local system, is geometrically shown on the plan as two circles with radii of 165.89 and 52.69 m respectively. The ring consists of 12 spiral, galvanized, full-lock coil cables, 087 mm, arranged on a horizontal plane at a height of +29 m.

Outer anchorage ring

The load bearing and stabilizing cables are anchored around the outside to a space framed reticular ring, appearing as a polycentric circle on the plan with maximum external dimensions of 307.94 m for the larger diameter, and 237.28 m for the smaller one.

The ring has a triangular section, composing an upper chord made of CHS steel tubes with a 1400 mm diameter and thickness of 70-60 mm, positioned at a height of +36.49 m, and of two lower joists at +23.99 m and 25.89 m, made of tubular steel with a 1000 mm diameter and thickness of 16-18 mm. The overall dimensions of the triangle, measured along the axes, are 10.50 m along the base and 12.50 m, in height.

1.3. Structural analysis

Structural static scheme

Cables structures show usually a significant geometrical nonlinearity; also in this case second order effects have been considered. A slight deflection of the inner ring causes the increase of the torque arm between the forces transmitted by the outer and inner rings. In this fact lies the concept of geometrical stiffness of the cable structures (Figure 3).

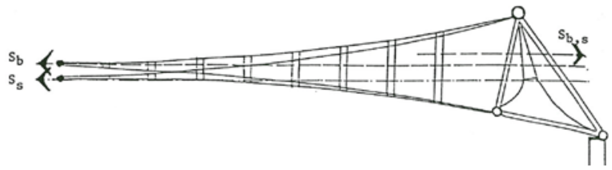

Figure 3: *Static scheme of the transversal section of the roof*

As well known, in all tensile structures the geometrical as well as the mechanical properties depend on the initial stress: in fact equilibrium position under dead loads depends on the tensile force in the cable truss. The tensile force, on the other hand, depends on the curvature radius of the horizontal projection of the two rings.

It can be easily shown that the outer ring is compressed by a force of the same intensity as the one of the tension ring. The stress transmission and the high resulting stiffness is obtained by means of the seventy-eight cable truss, thus working as the wheel spokes of a big bicycle.

Wind tunnel tests and aerodynamic behavior

For the Rome stadium, wind induced response of the cable-supporting roof has been analyzed by a non linear model and assisted by wind tunnel tests carried out at the BLWT Lab. of Western Ontario University. The integration of wind tunnel tests data into the design process presents significant problems for wide span sub-horizontal enclosures; at that time, the study of the Rome stadium drew attention to the inability of employed measuring system to provide data in a form that could readily be used as input for a dynamic numerical model developed by the designer and led to discussions between designer and wind tunnel researchers in order to examine alternate techniques that might be used in future projects [3].

Wind tunnel tests have been carried out on a model with 1:200 scale. The mean velocity of the flow was about 25 m/s with varying incoming direction and the following quantities have been measured by means of 170 pressure taps (44 on the underside and 126 on the topside), disposed on 8 panels of the roof (Figure 4-a):

- time histories of the local pressures for every 10° of incoming flow direction; the maximum, minimum and average values of the wind pressure have then been evaluated, as well as the root mean square of its fluctuating part;
- pressure coefficients (maxima, minima and average) for every 10° of incoming direction;
- auto and cross-spectra of the fluctuating pressure (averaged on every single panel) at the 8 instrumented panels, with flow direction varying from 0° to 360°.

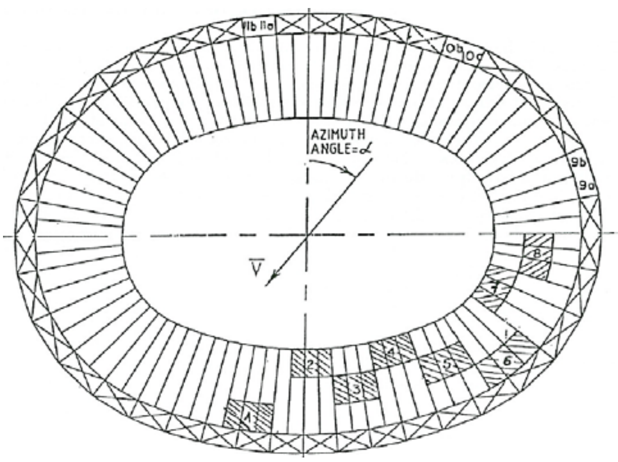

Figure 4-a: *Scheme of the instrumented panels*

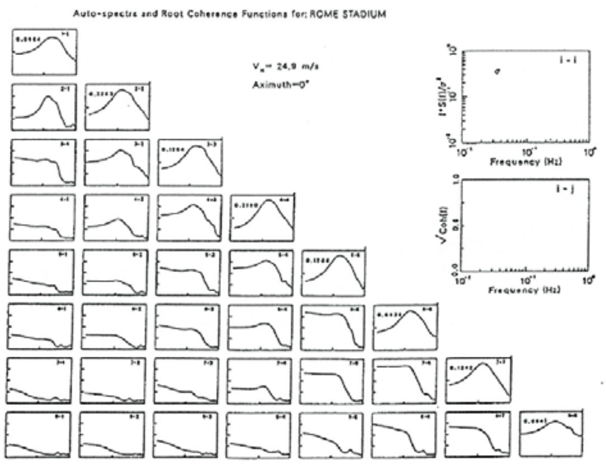

Figure 4-b: *PSDF of the measured pressures*

The aerodynamic behavior shows a clear shedding phenomenon. The external border (perimeter) of the structure, constituted of the trussed compression ring with triangular cross section and tubular elements and by the roofing of the upper part of the stands, disturbs the incoming horizontal flow in such a way so that vortex shedding is built up. This causes the roofing structure to be subjected to a set of vortices with a characteristic dominant frequency. This is confirmed by the resulting Power Spectral Density Function (PSDF) of the fluctuating pressures, which shows a peak at about 0.15 Hz, almost for all the pressure taps (Figure 4-b).

Numerical modeling of the structure

A nonlinear model of the overall structure has been developed which is composed by 796 beam elements for the outer compression ring, 2,320 cable elements for the radial cable trusses, 80 cable elements for the inner tension ring and 8 three-dimensional spring elements simulating the elastic supports at the boundary. Further 800 truss elements provide the stiffening of the roofing panels suspended on the radial cable trusses, giving a whole of 5,845 degrees of freedom (DOF).

The nonlinear dynamic analysis was carried out by means of a direct integration procedure following the implicit Newmark method; a duration length of 100 s was chosen, with a time step of 0.1 s.

Modeling of the wind action

Unfortunately, due to the fact that available data did not allow to rebuild completely the structure of the cross-correlation over the whole roof, a second set of the wind loading on the structure has been generated, starting from the actual PSDF for the whole system and from the same cross-correlation as assumed in the first run.

Due to the lack of the available data, it was not possible to rebuild a complete correlation field, so that, up to now, following assumptions have been made for the PSDFs:

- auto-spectral density functions according with the measured ones;
- cross-spectral density functions according to Vickery's model and Taylor's hypothesis.

The generation of the wind velocity field has been done simulating 91 cross-correlated time series of the wind velocity.

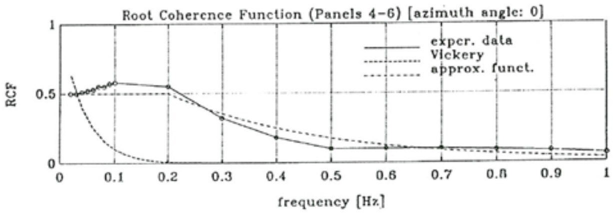

Figure 5: *Root Coherence Function: comparison between the law used in the 3rd run (approx.. funct.), the Vickery's law and the experimental data*

To analyze the sensitivity of the response to these values, a modified root coherence function for the wind velocity has been used, according to those reported in Figure 5.

1.4. Lifting and tensioning operations: the big lift

The tensioning stage was planned following the same methods used for the lifting operation, i.e. all of the intermediate geometrical and tensile states of the whole structure (structural steel-ring/cable trusses/cable ring) were analytically simulated before and then checked during the operation (Figure 6).

Figure 6-a: *After hoisting the Tension Ring via Upper Radial Cables, prestress not yet applied*

Figure 6-b: *Direct tensioning of anchorage cable sockets at the backside of the Compression Ring*

The main construction steps have been the following:

- positioning of 8 drawing teams at 8 symmetrically distributed points around the structural steel ring (load bearing and stabilizing cables);
- organization of a central control unit with a radio station and topographical instruments located in the center of the playing area;
- gradual tensioning, working alternately on the load-bearing then stabilizing cables, with clockwise movement of the work teams from one position to the next. The teams worked their way completely around the ring three times. The upper jacks were of a perforated type, with a 300 t capacity, used singly, while the lower ones were rod-type with a 250 t capacity, used in pair;
- Geometrical and pre-stressing control: this check involved measurement of the structure's geometrical characteristics and strain in the cable and ring system, and on line comparison with the values obtained through structural analysis.

2. YAOUNDE NEW STADIUM

The design of the new Paul Biya Stadium located in Yaounde has been started in 2016, the stadium is now under construction and when completed will host 60’000 spectators (Figure 7).

Figure 7-a: *Aerial view of the new Yaounde stadium*

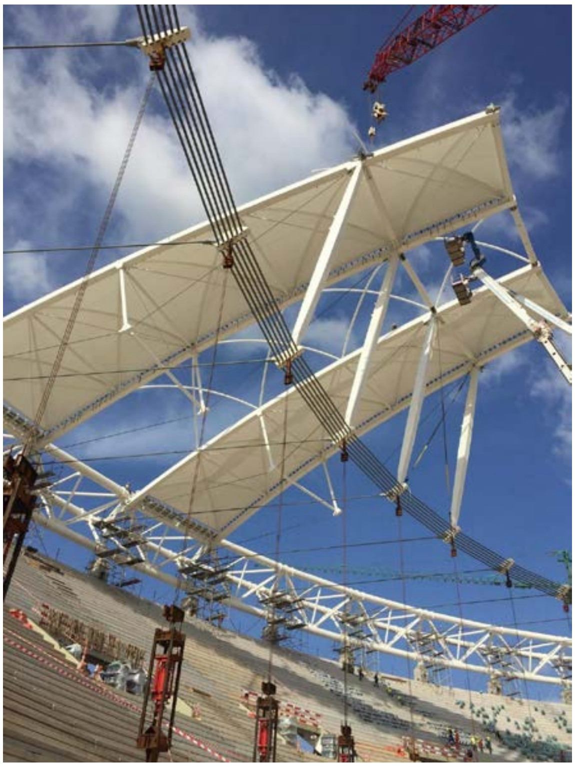

Figure 7-b: *Installation of the hybrid radial beams*

2.1. Description of the structure

The grand stands of the Stadium will be covered by a tension structure roof composed by one inner tension ring and one inner tension/compression ring (at top position) and a perimeter compression truss ring connected with radial bottom carrying cables and radial top secondary beams (Figure 8). Compressed sub-vertical flying masts (with composite cross section made by welded plates with cross layout) connect top compression inner ring with lower cable groups. The roof has plan dimensions about 300 m x 245 m and a height of about 46 m above ground.

Radial cables and outer extremities of secondary beams are connected to CR in the center of mass of its cross section and appended to CR top joints. Joints in the center mass of compression ring and flying mast top joints support radial hybrid string beam: each radial beam is composed by a couple of top radial beams (with arch shape) stiffened by 4 lower cables and 2 posts. Distance between support is about 40m.

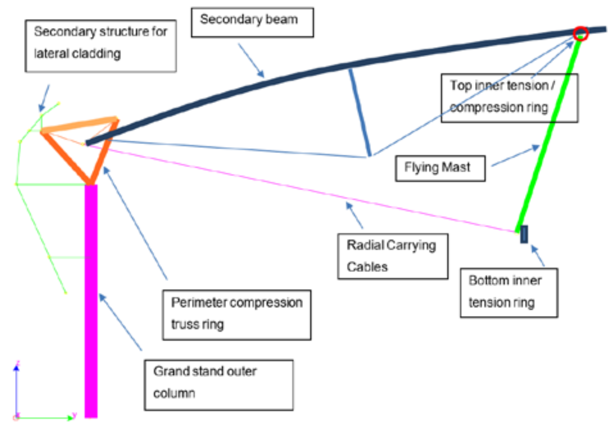

Figure 8-a: *Typical roof structural module: schematic description*

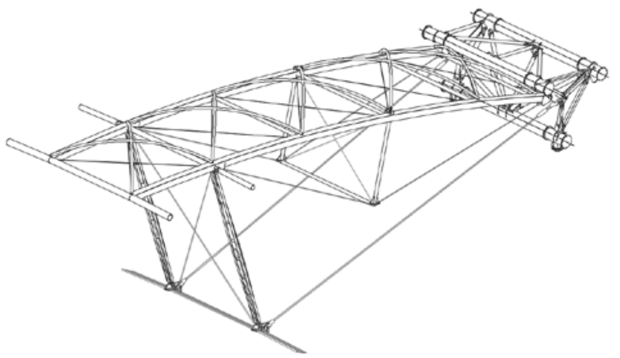

Figure 8-b: *Typical roof structural module: isometric view*

Compression Ring has 2 top chords and 1 bottom chord mutually spaced, triangulated by diagonals in lateral and top planes, connections between chords and diagonals are pinned. The ring is supported by the 68 heads of grand-stand columns, typical support lies on horizontal slide to permit horizontal displacements due to initial tension ring lifting phase. At the end of this phase slid will be closed and pendulum device of supports will permit horizontal "slow" displacements due to live loads and will ensure the "re-centering" action to reset them. At about 45° direction (plane view), 4 concrete grand stand sectors are arranged to restrain the roof respect to horizontal wind and seismic actions. For each sector two vertical supports are arranged with shock absorbers in order to fix horizontal radial displacements and the intermediate joint of lower chord is arranged with 2 shock absorbers in order to fix horizontal transverse displacements, each shock absorber includes spring for re-centering effect too.

The governing structural principle of the horizontal structure of the roof is that the equilibrium, geometry, static stiffness, dynamic stiffness, are obtained by an accurately planned and constructed distribution of forces in combination with an accurate geometry, in the components of the tension structure made by the cables, the connection nodes, the steel components and the perimeter compression truss ring as a whole integral system.

2.2. Structural analysis

Numerical modeling of the structure

The structural mathematical analysis has been carried out with the following steps:

- definition of the State 0 configuration of the Tension Structure roof under the application of the pretension forces and permanent loads;
- nonlinear elastic analysis of the global structure (taking in account of geometrical stiffness of cable elements) in order to evaluate the effects of variable design actions (gravitational live loads on roof, wind action, thermal action);
- linear elastic dynamic analysis in order to evaluate the dynamic characterization of the structure (natural frequencies and correlated modal shapes) and the effects of design earthquake actions (Figure 9);
- resistance and stability checks of steel structural elements.

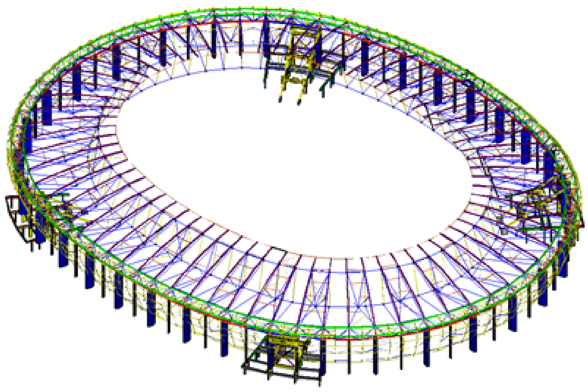

Figure 9-a: *Overview of the mathematical model of the structure*

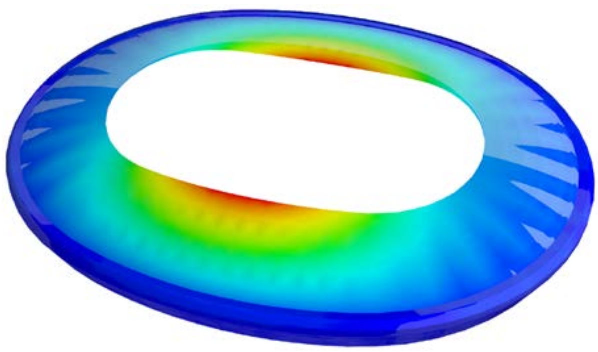

Figure 9-b: *Overview of the first mode of the roof*

Wind tunnel tests and aerodynamic behavior

Wind tunnel tests have been commissioned by the general contractor Piccini Group to RWDI. Preliminary studies based on the A rigid model of reduced scale equal to 1:300 have been built and tested in RWDI wind tunnel located in Milton Keynes.

Figure 10-a: *Scaled model in the wind tunnel*

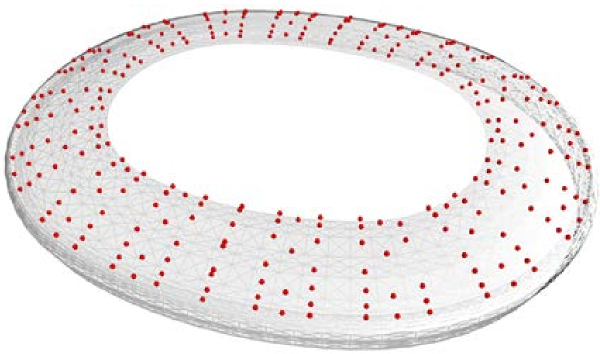

Figure 10-b: *Pressure taps locations*

A total of 36 angles of attack have been considered and pressures measured with a frequency of 512 Hz for 36 s. The reference velocity measured at 0.4 m from the wind tunnel floor was approximately 12 m/s for all the considered angle of attack. Pressures were synchronous measured in a total of 770 locations but only 317 of them will be used in the following analysis of the roof (Figure 10).

Buffeting analysis

For the buffeting analysis of the roof structure the first step is represented by the extraction of the Proper Skin Modes [4]. In particular, the mesh corresponding to the external surfaces of the building have been extracted and the eigenfunctions of the Laplace operator calculated, such eigenfunctions can be interpreted as pressure distributions and represent the PSMs. It can be clearly seen that their characteristic wavelength rapidly increases so that they can take into account variations in the pressure distributions between the inner and the outer part of the structure.

In the analysis the first 20 modes of the structure are considered which allow to span frequencies up to 10 Hz. Damping has been assigned by means of the Rayleigh method is such a way that the first and the last considered modes are characterized by a damping ratio equal to 1.5%. During the analyses normal forces in 681 sections of interest of the primary structures have been considered. In particular, the considered elements are that of both tension rings, the three chords of the compression ring and the flying masts.

2.3. Lifting and tensioning operations: the tie down

The lifting operations of the roof structural system have been planned following a tie down procedure that allow an easy installation of the secondary beams and to apply a pretension in the upper internal ring. In Figure 11 is shown the evolution of the working site in one year.

Figure 11: *Evolution of the working site*

The main steps of the erection sequence are the following:

- construction of the external compression ring;
- tension ring (inner lower ring + radial cables) lift using radial strain jacks;

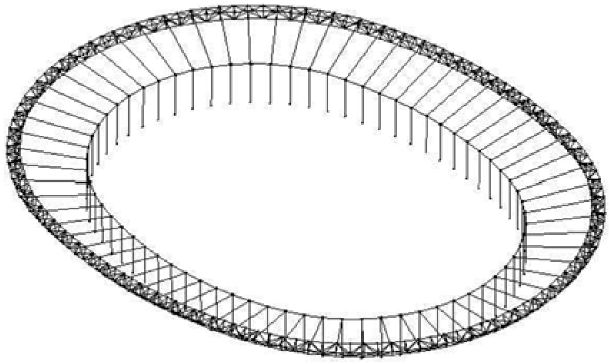

Figure 12-a: *Erection sequence: installation of tie down cables*

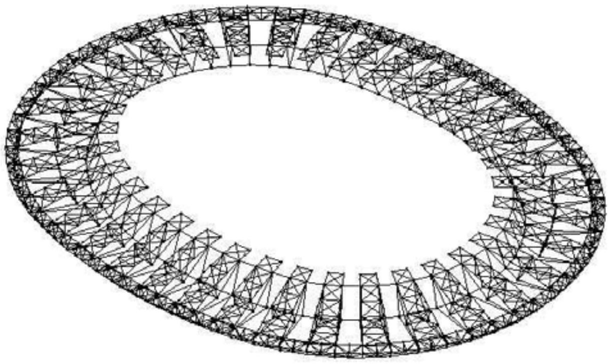

Figure 12-b: *Erection sequence: removing of tie down cable*

- installation and pre-stressing of tie down cables (Figure 12-a);
- installation of the secondary beams and closing of the upper internal ring;
- removing of the tie down cables, the residual tension force in the tie down cables generate the prestress in the inner upper ring (Figure 12-b).

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