New Braga Municipal Stadium, Braga

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Introduction

A dramatic state-of-the-art engineering and architectural stadium was built to receive the EURO 2004 football championship. The site where the client wanted a 30 000 seat stadium was a hill with a very good granite quarry on its top. The view of the valley from this site was beautiful. The Architect imagined one stadium with only two stands – one would be carved into the rock as in an amphitheatre; the other one would rise up from the ground (*Fig. 1*).

Fig. 1: General View of the Stadium – East Stand

The rock excavation and the roof design were the big challenges for the design team. The roof was to be compatible with the idea of integrating the Stadium into the environment. It was also to be as light and clean as possible. Therefore, arches, trusses, poles, cables and membranes could not fit well into the concept and were eliminated as solutions.

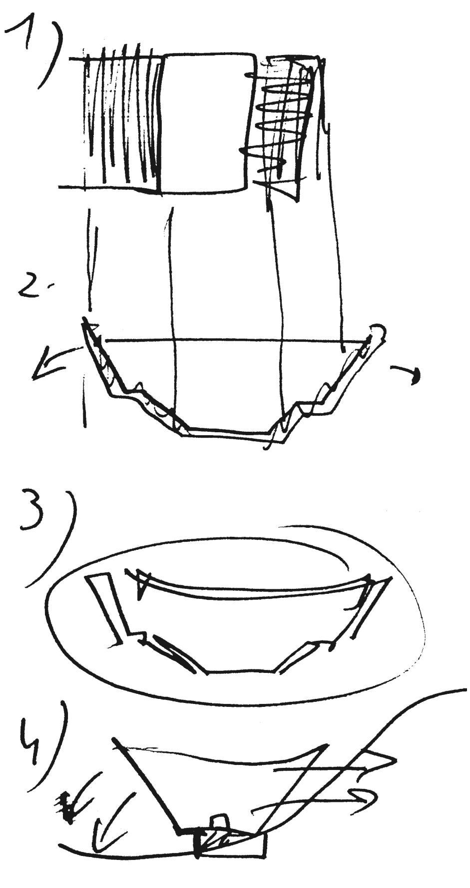

Fig. 2: Architect's sketches

A suspended roof such as was used in the Portuguese Pavilion in EXPO 98 came up as the natural solution. The rock existed there to anchor the cables and the reaction of the roof in the cantilevered stand would help in stabilising it (*Fig. 2*). However, there were reservations in the uncertainty of the dynamic behaviour of a 202 m-span roof and the fact that the roof would have to be built 50 m high. The Portuguese Pavilion roof has a 67,5 m span and with a lower height was built with a total propping system from the ground.

Research on similar structures and preliminary calculations showed the feasibility of the solution and that an appropriate geometry and slab weight could lead to a desirable balance of moments in the foundation for permanent load combinations. A structure of parallel "ribs" would provide the required stiffness for the stand and would allow for all the stairs, concourses, bars, etc, to fit in between.

Acting on the safe side, the designers were committed to the idea of a continuous roof. This solution, however, could not allow enough light in for the grass of the football field. Therefore, dividing the roof into two then had to be addressed and tested.

The initial studies showed that the solution with two separate roofs was possible but confirmed the need to develop the analysis of the dynamic interaction between the roof and the wind. The calculation of the main forces on the stands and on the foundations also showed the feasibility of the solution. Constructability was studied and a precast plank slab proved to be a feasible solution for the roof also considering the success in Dulles Airport in Washington that had been built in such a way 30 years ago. Cost estimates revealed that it could fit within the budget. The design concept was then fixed and detail design began.

Architecture and Engineering

The program was new to the whole team. With no preconceived ideas, the designers started from scratch. The architect's general approach is to allow the technical needs of the construction to rule the development of the design. Thus, outside of the definition of the spaces and the overall aesthetics, an intense, demanding and stimulating dialogue resulted between the architects and engineers whereby the search for design solutions only ended when both disciplines were satisfied.

Clear and rigorous criteria were then agreed upon to mold the solutions to the technical needs of the building. The resulting solution had to be simple in form and in detail. But simplicity is only achieved through a gradual and continuous process of successive modifications that often bring up solutions that are quite different from the original ideas. The architects and engineers worked closely together to achieve a common goal, which was not viewed as the exclusive territory of one or the other.

General description

The Braga Municipal Stadium is located in Dume, in the Braga Sports Complex which will also include a sports pavilion and an Olympic swimming pool. The most noticeable element of the stadium is without doubt its roof, which is made up of pairs of full locked coil cables, spaced 3,75 m apart from each other, supporting two concrete slabs that cover the two stands of the stadium. Its span of 202 m and the fact that the cables are free in the central zone were its main challenges. The roof is supported on two large beams at the top of both grandstands – east and west, where the cables are anchored.

The east grandstand is structurally made up of 50 m-high walls that are "punctured" by the slabs of the different foyer floors of the stadium. Its longitudinal stability is ensured by the slabs supporting the steps of the stands. The walls of the east stand, which are only 1 m thick, are extremely slender.

After the roof, the west grandstand is perhaps the most complex structural element, due to the diversity of the problems it involved; the vertical plates anchored into the rock, the functioning of the structure with the ground, compatibility between the structural functioning of elements with very different stiffness, and the laying of foundations in unstable embankments. Among the several solutions encountered it is difficult to pick out one worthy of special mention.

Below the whole area of the football field there is a building with two floors. It includes a parking garage, changing rooms and all the EURO 2004 backup services. It did not encounter particular structural difficulties apart from the control of the effects of shrinkage and temperature variations, which led to the installation of bearings between the columns and the top slab that needed to be especially well integrated into the architecture.

The excavation and stabilisation of embankments that preceded the construction of the stadium was in itself a huge task. In total 1 700 000 m^3 of hard rock and gravel were excavated (*Fig. 3*), which led to the need to retain enormous rock embankments, in which, unfortunately, the fractures were in an unfavourable direction. The embankment was retained with a net of anchors and rock bolts to guarantee its stability.

The Roof

The concept was to build a set of suspension cables suspended from the beams at the top of the rib walls, supporting the two independent concrete slabs that covered each stand. The geometry selected for the roof resulted from a compromise between the aim of the architect for an extremely subtle inverted arch and the value of the force produced on the structure by the horizontal component of the cable forces.

The selection of the type of cable was also extremely important as it would influence the definition of the shape and technological characteristics of the roof. Two cable options were possible: full locked coil strand and parallel-wire strand. After studying the different characteristics of the two solutions in terms of durability, anchoring devices and dimensioning, full locked coil cables were selected, which led to smaller diameters.

In the tender phase, given the vital role of the specialized technology of each cable manufacturer, it was decided that for the cable system a Design/Build contract would be used, where the contractor would be allowed to propose changes to the original design developed by the engineer. Responsibility for the roof project was therefore shared by the engineering project team and the contractor together with the manufacturer of the cables.

In addition to standardising the concrete slab height along the roof, the only significant modification from the initial project to the one that became the tender winner, consisted of grouping the cables into pairs, with a spacing between each pair of 3,75 m, two times the initially planned 1,875 m.

Suspended roofs always present special challenges for structural engineers, mainly due to the action of the wind and the construction process. To study the behaviour of this roof all the customary project procedures had to be rethought starting with the essentials, from establishing the values of the actions to be considered to the prediction of the structure response and the possible interaction between the two. It was therefore necessary to use physical models (tests on small-scale models) to complement the usual mathematical models (algorithms calculated using computerised simulations) which, in this case, no matter how sophisticated, were not sufficient on their own to simulate the behaviour of the structure, namely, regarding wind action.

Wind Analysis and Wind Effects

A modern and automatic meteorological station existed in the neighbourhood of the stadium, in Merelim, however, at the date of the project it had only been in service for thirty months. To obtain a description of the design wind at the construction site for a 100-year return period, data provided by the Merelim Meteorological Station was used and statistically processed and suitably corrected to take into account the "meagre" thirty months of records. The extreme value analysis of the anemometer data used the

Fig. 3: General View of the excavation

Leiblein Method on independent gust velocities. The analysis for mode and dispersion was done on the square of the velocities as this provides a better fit to the Fisher Tippet type-1 distribution of extremes.

To determine the effects of the surrounding topography on the average speed and on the intensity of turbulence arising from the airflow in the vicinity of the stadium, tests were undertaken in a wind tunnel on a rigid model in a scale 1:1500 (*Fig. 4*). The time history of dynamic pressures at several points on the roof, caused by the design wind for each of the directions considered, were obtained using wind tunnel tests on a rigid model at a scale 1:400. The roof of the model was monitored with pressure sensors on 200 points distributed over its upper and lower surface. The measurements were registered automatically, obtaining the respective time series for wind in 36 different directions spaced 10 degrees apart.

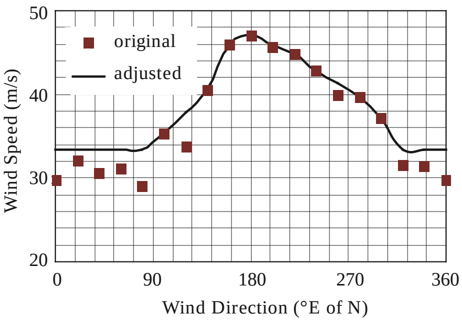

Fig. 4: Design wind speeds

The calculation of the response of the structure to the dynamic action of the wind described by the history of pressures previously obtained was done in two different ways. The first was a deterministic dynamic time history analysis, through direct step-by-step integration of the dynamic equilibrium equations of the structure based on a linear elastic analysis of a shell finite element model with a geometric stiffness matrix that was assumed as constant. The geometric stiffness matrix was obtained using the axial forces of the cables due to permanent loads. Through this analysis the dynamic response of the structure to the dynamic action of the wind could be calculated. The maximum displacement calculated was 47 cm.

The second method was a probabilistic dynamic analysis using the Orthogonal Decomposition Method. This method is based on the principle that an unsteady multi-variate pressure field can be usefully simplified by projecting it onto a space generated by the eigenvectors of the covariance matrix of the original field. This method has two big advantages. First, the new fields (pressure modes) are mutually uncorrelated. Second, the energy content of the complete multi-variate field is usually well represented by few components in the transformed space, allowing the representation of the effective pressure field by means of few pressure modes. This analysis enabled the estimation of the quasi-stationary component and the resonant component of the response of the structure in terms of stresses and deformations. These values were used in the design of several structural elements of the roof.

The mathematical modelling based on the results of the tests in the wind tunnel with the first physical models did not guarantee the non-occurrence of aeroelastic instability of the roof. Although it was agreed that a behaviour of this kind would be extremely unlikely, a decision was made to develop aeroelastic physical models to be tested in a wind tunnel. The aerodynamic stability of the initial solution was demonstrated through tests on a aeroelastic model at a scale 1:200. The absence of aeroelastic instability in the final solution was proven through tests on an aeroelastic model at a scale 1:70. Both models demonstrated the aeroelastic stability of the roof, and the measured maximum deflection was almost equal to the calculated movement.

Drainage of Rainwater

The rainwater is drained off the roof in the south-east embankment direction, with a one percent grade, which is achieved by varying the length of the pairs of cables that support the roof.

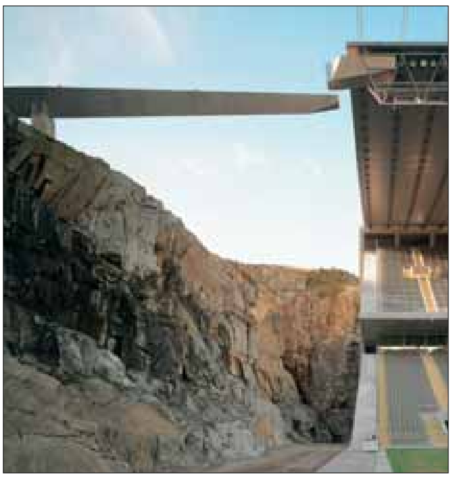

Fig. 5: Duplex stainless steel "aqueduct"

Two large spouts in stainless steel, suspended from the concrete slabs channel the water to the "aqueducts", made of duplex stainless steel, which lead to the rainwater network and the watercourse existing at the stadium site.

The aqueducts, approximately 40 m long, of which 27 m is overhanging (*Fig. 5*), are supported on the ledges of the embankment on columns with varying lengths. The dimensions of these two devices were established through a study of the water flow, taking into account the height of the fall, limited by the foreseeable movement of the roof due to wind.

Construction Process

The architect wanted to see the roof as a continuous surface standing over the cables. The slab is connected to the cables only in the normal direction allowing relative tangential movements between them (*Fig. 6*). This option has many advantages regarding the roof's response to thermal actions, shrinkage of the concrete roof slab and the dynamic wind load.

Fig. 6: Roof detail

The construction sequence of stress-ribbon bridges and the Dulles Airport building in Washington gave direction to using precast planks sliding over the cables with a thin layer of concrete poured on top to minimize problems arising from differential deflections. The reinforced concrete slab is 240 mm thick and was determined by balancing the need for a stabilising mass while minimising the weight. The prefabricated elements were assembled over the cables, on top of the stands. Each new piece is linked to the previous piece with bolts and the pieces were slid along the cables by gravity. When all the elements were in position the transversal and longitudinal joints between the panels were cast in concrete.

Control of Geometry and Monitoring

The behaviour of the structural elements of the stadium during assembly of the roof was monitored using the information collected by the instrumentation systems installed, namely: load cells in the roof cables; "internal" instrumentation of the concrete structure (strain gauges, tilt-meters and thermometers in the vertical ribs of the East Grandstand); instrumentation of the rock massifs and foundations; load cells in the ground anchors; and in-place inclinometers. The phase-by-phase response monitoring of the structure during assembly of the roof as well as its "mechanical interpretation" guaranteed that there would be no "surprises" in the final geometry of the system of cables that give this roof its unique form.

Currently, in addition to the static instrumentation sensors already mentioned above, both in the concrete structure and in the rock massifs, dynamic monitoring is also installed, made up of 6 triaxial accelerometers placed on points of the roof expecting the highest vibration amplitudes and by cells measuring the wind pressure at various points on the underside and top of the roof covers.

The East Grandstand

The East Grandstand is supported on sixteen vertical rib walls, all of which are 1 m thick, beginning at level +87,80 and reaching level +142,85 (*Fig. 7*). The hydraulic and electrical infrastructures are all placed inside the vertical ribs and the slabs, from the level of the foundation to the level of the roof. The grandstand has no structural joints. Therefore the set of 16 vertical ribs with the slabs of the floors and the stands stabilise the grandstand structure for both vertical and horizontal actions.

From the start of the design phase it was understood that a "suspended" roof between the two stands would give its main design challenge to the structure of the east side, where the cables are anchored more than 50 m above the foundation level. The structure of this stand should therefore adjust the geometry of the vertical ribs in order that the gravitational actions of the stand partially counteract the high forces transmitted by the roof to minimise the imbalance of moments at the foundation level. In an early phase this goal was partially achieved. With the evolution of the project and the incorporation of the functional demands of the stadium, equilibrium at the level of the foundation was not fully achieved, leading to a considerable modification at the end. On top of the walls is placed an extremely stiff beam following the direction of the roof cables, which guarantees the transition of forces between the roof cables and the vertical rib.

Regarding the foundations, there are three types of geological-geotechnical strata. This heterogeneity was especially important, and several analyses were carried out representing the global behaviour of the structure with ground-structure interaction. Taking into account the sensitivity of the structure to the subsistence shown in the numerical models, the option was to replace the earth of lesser quality with concrete, thus guaranteeing the whole structure is based on rock.

Fig. 7: East Stand during construction

The West Grandstand

The West Grandstand of the stadium is carved into a granite massif whose upper level is at the level of the square that provides access to it. The relation between circulation zones located under the stands with the striking rock embankment, characterised by irregular cuts and pronounced fractures, had to be emphasised in the project. Taking advantage of the fact that the upper square (west) is at the same level as the roof, the horizontal actions transmitted by the roof cables were stabilised by anchoring directly into the massif.

Eighteen vertical concrete plates were positioned in this transition zone, between the roof and the rock, all 1 m thick, which are adjusted in height to the irregularities of the embankment. The steps of the stands are prefabricated and are supported on prestressed and reinforced concrete angled beams.

Given the characteristics of the massif at the west foundation, footings directly bearing on the massif are used at most locations. For the columns and vertical plates in zones of fragmented massif and around the crest of the embankments, local stabilisation was provided using rock anchors. This highly hyperstatic structure is very sensitive to settlements of supports, which led to meticulous efforts to control possible settlements, not only with consideration of the ground/structure interaction in calculations, but also by improving the ground foundation behaviour by making it work together more uniformly.

The reactions of the roof and the horizontal components of the reaction of the stands determined the dimensions of the vertical plates. As this was an essential component of the global stability of the stadium, and as the vertical plates were founded in zones of differing geotechnical characteristics, models had to be developed representing the ground/structure interaction that allowed the design of the rock anchors to be confirmed, and the state of permanent compression between the vertical plates and the rock embankment to be verified. As such, it was possible to confirm the global equilibrium of the vertical plates in relation to the loads transmitted to them and proceed with their internal dimensioning, including the foundations.

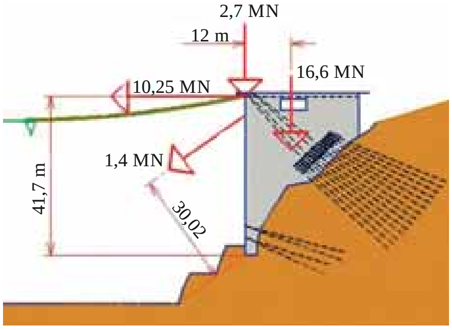

Fig. 8: West Stand – Equilibrium of forces

The upper level of the rock anchors transmits the majority of the roof cables' horizontal load almost directly to the rock massif (*Fig. 8*). The lower level of anchors play a crucial role in correcting the direction of the vertical plate reaction in its foundation, channelling the resultant of forces to the interior of the massif, which is necessary, given the possible fracturing of the rock under the foundation.

Conclusion

The variety and complexity of the technical problems involved in the design and construction of the "New Braga Municipal Stadium" constituted a great challenge for the design team and, at the same time, it was an an excellent opportunity to increase our know-how.

In this process we were aware of the risk that arises from constantly "pushing beyond the limits", and controlled it through an intensive research methodology. Risk assessments, redundancies, independent teams studying the same problem, cross-checking results, all allowed us to gradually drop the inevitable initial "extra safety margins".

During the 4 years of the project planning and execution, cost and time schedules lived side-by-side with the technical demands and aesthetic aspirations of the Project. It is rewarding to realise now that to talk about the stadium's structure is also to talk about its architecture, and that to explain its architecture is to tell the story of the engineering problems it brought up and how they were overcome.

Acknowledgement

The initial wind engineering services were provided by Ove Arup. The physical modelling and testing of the structure for wind was made by Rowan Williams Davies & Irwin Inc. (RWDI, Canada). Tests on the aeroelastic mathematical model for aerodynamic stability were made by Danish Maritime Institute (DMI, Denmark) for the initial solution and by Politecnico di Milano for the final solution.

SEI Data Block

Owner:
Braga Municipality, Braga, Portugal

Architect:
Eduardo Souto de Moura, Porto, Portugal

Structural design:
AFAssociados, Projectos de Engenharia, SA, Porto, Portugal

Contractors:
ACE Assoc. / Soares da Costa, Braga Portugal

Steel (t):	15 000
Concrete (m^3):	90 000
Total cost (USD million):	75
Service date:	December 2003