JOURNAL OF
JCIVILTECH
INNOVATIONS IN CIVIL ENGINNERING AND TECHNOLOGY

e-ISSN 2687-2129
jciviltech, 2019, 1(2), 75-97

Araştırma Makalesi / Research Article

Pretensioning Cable Roof Systems Besiktas Stadium Design Principles

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Geliş / Recieved: 17.12.2019; Kabul / Accepted: 28.12.2019

Abstract

In this study, design criteria of Besiktas Inonu Stadium, and the roof with a prestressed rope system, are included. Selection of systems in these structures, determination of the loads affecting the structure, analysis and design procedures of structural elements are explained. The geometry of the stadium structure, the assumptions made in the model for static and seismic analysis of the structure and the details of the analysis model are presented with this article. the necessity of building a rope roof system consists of a summary of mathematical models for the design criteria of the roof. For the roof design of this stadium which basically takes the mechanical working principle of a bicycle wheel as a starting point, the minimization and behavior modeling of the horizontal loads which can be transferred from the roof to the structure in a possible earthquake are summarized in this article including the wind tunnel test.

Keywords: *Cable, Prestressed steel, Ring beam, Membrane roof.*

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Bu makaleye atıf yapmak için
İz, S. (2019). Pretensioning cable roof systems Besiktas stadium design principles. *Journal of Innovations in Civil Engineering and Technology (JICIVILTECH), 1*(2), 75-97.

Öngerilmeli Halatlı Çatı Sistemleri Beşiktaş Stadı Tasarım Kriterleri

Öz

Bu çalışmada halatlı çatı sistemi ile inşa edilmiş Beşiktaş İnönü Stadı tasarım kriterlerine yer verilmiştir. Bu tür yapılarda taşıyıcı sistem seçimi, yapıya etkiyen yüklerin belirlenmesi ile yapısal elemanların analiz ve tasarım prosedürleri açıklanmıştır. Stadyum yapısının geometrisi Yapının statik ve depremli durum analizi için oluşturulan sonlu elemanlar modelinde yapılan kabuller ve analiz modeli ile ilgili detaylarda bu makale kapsamında sunulmuştur. Çatı geometrisinde uyulması gerekli kriterler halatlı çatı sistemi yapılması gerekliliği matematik modellerin özetinden oluşmaktadır. Bisiklet tekerleği mekanik çalışma prensibini örnek alan stadın çatı sistemi tasarımında öngerilmeli halatlı çatıların çatı ring kirişi ve üstten ankastre aşağıdan mafsallı kolon elemanları kullanılarak olası bir depremde çatıdan aktarılacak yatay yüklerin minimizasyonu ve davranış modellemesi özetlenmiştir. Ayrıca çatı ve üzerine oturduğu betonarme stad yapısı arasındaki dinamik karekteristikleri yansıtan çalışma rüzgar tünel testi sonuçları özet halinde verilmiştir.

Anahtar kelimeler: *Kablo, Öngerilmeli çelik, Halka kirişi, Membran çatı.*

1. Introduction

The Besiktas Vodafone Arena is located in Istanbul, Turkey. The general view and typical section of the stadium are shown in Figures 1 and 2. The building is oriented along two main axes and rotated by 36.4° to the cardinal directions. The primary building structure of the project consists of concrete grandstands to provide the support for a lightweight roof which covers the full perimeter of the stadium, as well as a part of the inner area. The footprint of the roof structure is approximately 216 m long and 161 m wide. The roof is divided to 42 transversal bays, whereas concrete work is divided into 86 axes, including 4 twin axes at the movement joints. The roof is a cabled structure which is completed with 1 central Lower Tension Ring, 3 central Upper Rings and 1 compression ring. Radially, the roof is sloped approx. 4° upwards. The roof will be covered primarily by a lightweight textile membrane made of Silicon coated glass fiber. TS500, TS498, Earthquake Regulation in the design of the stadium structure. Eurocode, AISC, ACI 318-02, NEHRP 2003, etc. have been used from many international regulations other than local regulations.

Figure 1. Stadium general view.

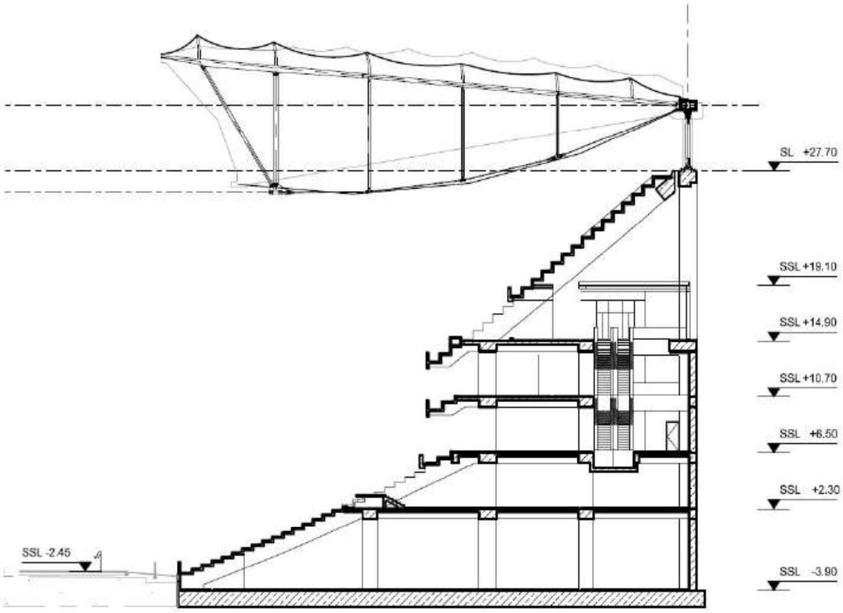


Figure 2. Stadium typical section

2. Geometry of Structural System

Structurally, the inner part of the Roof includes a T-structure made for the Lower Tension Ring, a Flying Column, and the last two elements of the Upper Radials, which holds the three Upper Rings, as well as an Inclined Strut. The T-Structure is pre-assembled before the Big Lift. The Compression Ring rests on 86 steel Columns, which are pinned radially to the concrete structure and bolted to the Compression Ring (Figure 3).

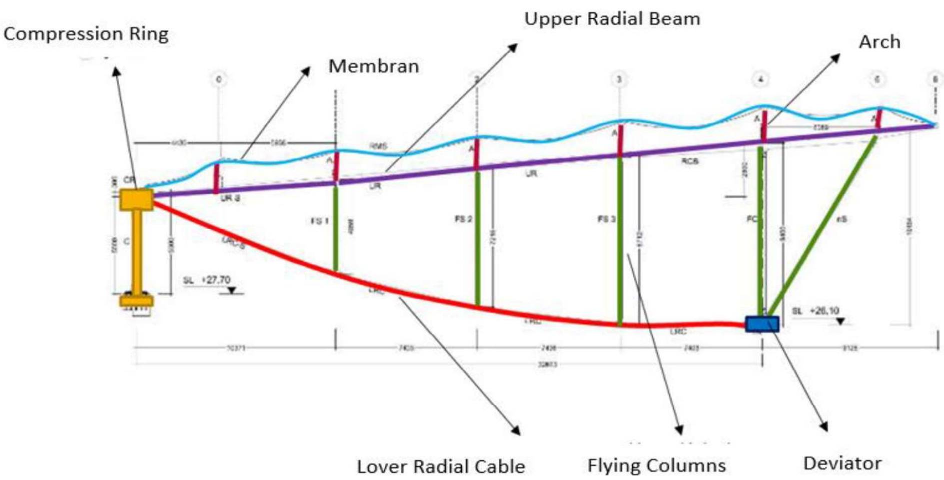


Figure 3. Besiktas stadium typical structure system.

The Compression Ring is approx. 630 m long and consists of three main parts, due to the oval shape of the roof. The central Rings (Upper and Lower) are tied to the Compression Ring by Radial elements, approx. 9 m long. As the columns are double than the number of bays, the Radial elements are split at the end. The inner Upper Ring is connected to the rest of the roof with cantilevering Upper Radials and to the Flying Columns. Due to the span of the cantilevers (approx. 9 m) intermediate supports are required between the flying columns and the tips (Inclined Struts). The Lower Tension Ring is connected to the Compression Ring with Lower Radial Cables, while the Upper Rings are connected with Upper Radial bending stiff elements. Upper and Lower radials re connected together via Flying Struts (length variable from 4 to 8 m) and Flying Columns, approx. 9 m long. Upper Radials support the membrane Arches, which are connected to the roof planes. The membrane has a typical span of ~10 m between arches and 10~15 m along arch span. At each axis, Lower Radial Cables have different curvatures, depending on the global in plane stiffness of the structure and in particular of the Compression Ring: where the Compression Ring has a small curvature because the system needs to become softer. The Lower Radial Cables have a bigger curvature, while the Compression Ring has a small curvature to ensure that the Lower Radials to become straight. The mathematical model of the stadium is shown in Figure 4.

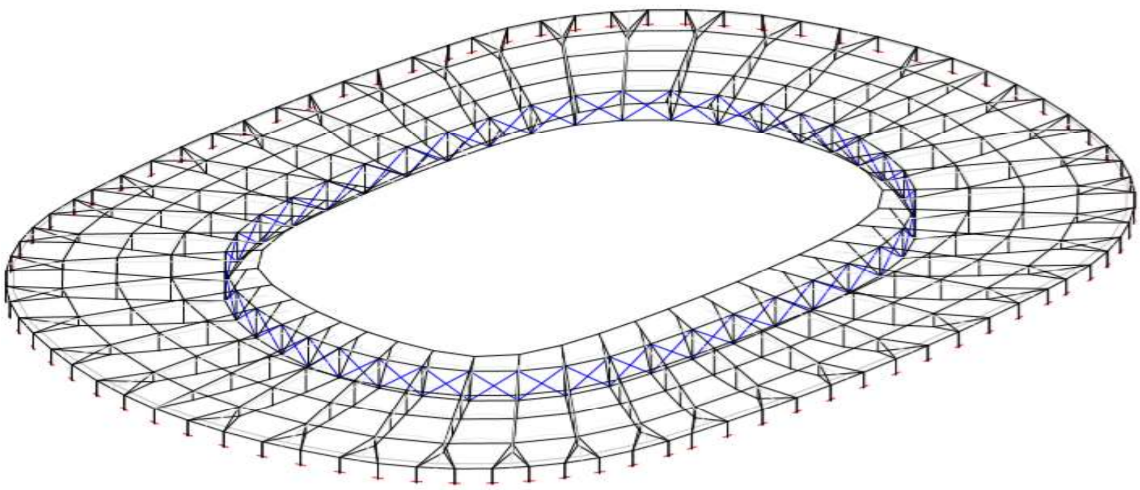


Figure 4. Besiktas Stadium mathematical model.

3. Principle of a Spoked Wheel System

It is known from engravings and old paintings that the roof covering similar to the first bicycle wheel technology known in history was used in Rome-Collosseum in ancient times (Figure 5). This system is thought to be the radial ropes that stretch the sail rope that cannot form the core and the sail cloth covering it (Masubuchi, 2012).

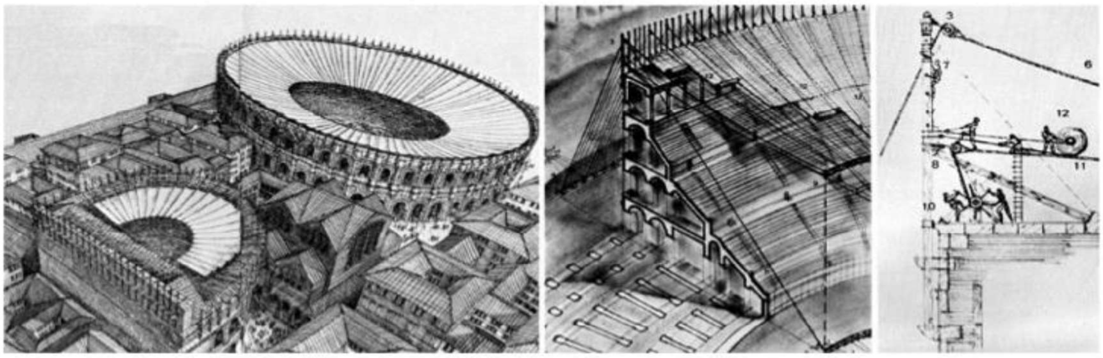


Figure 5. Engraving of the first application; Collosseum-Rome

In the system consisting of pressure and pull ring and radial ropes connecting them, the pull ring and the pressure ring forces are fixed to the pressure ring by radial ropes. This system can be briefly described as follows. The outer pressure ring or rings are fixed to the roof or the ground. With the radial ropes, the towing ring is tensioned and connected to the outer pressure ring.

The tensile force (V) in this radial cable is given in the following figure. (S) is the sum of the tensile force formed in the pull ring. If the vertical (dead + live) load (P) is applied to this structure, it is carried by the prestressing force in the cable. The load from the prestressing force must always be higher than the load (Bergermann & Chlaich, 1992).

In the world, one or two pressure rings are generally used in the applications of this system. A single pressure ring was used in Besiktas stadium.

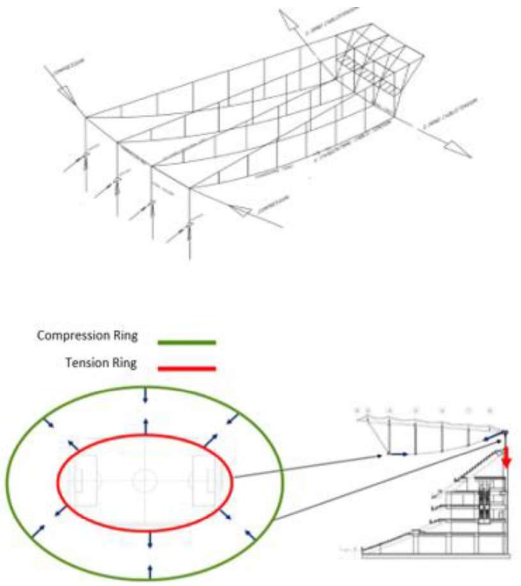


Figure 6. Typical load flow of besiktas stadium structural system.

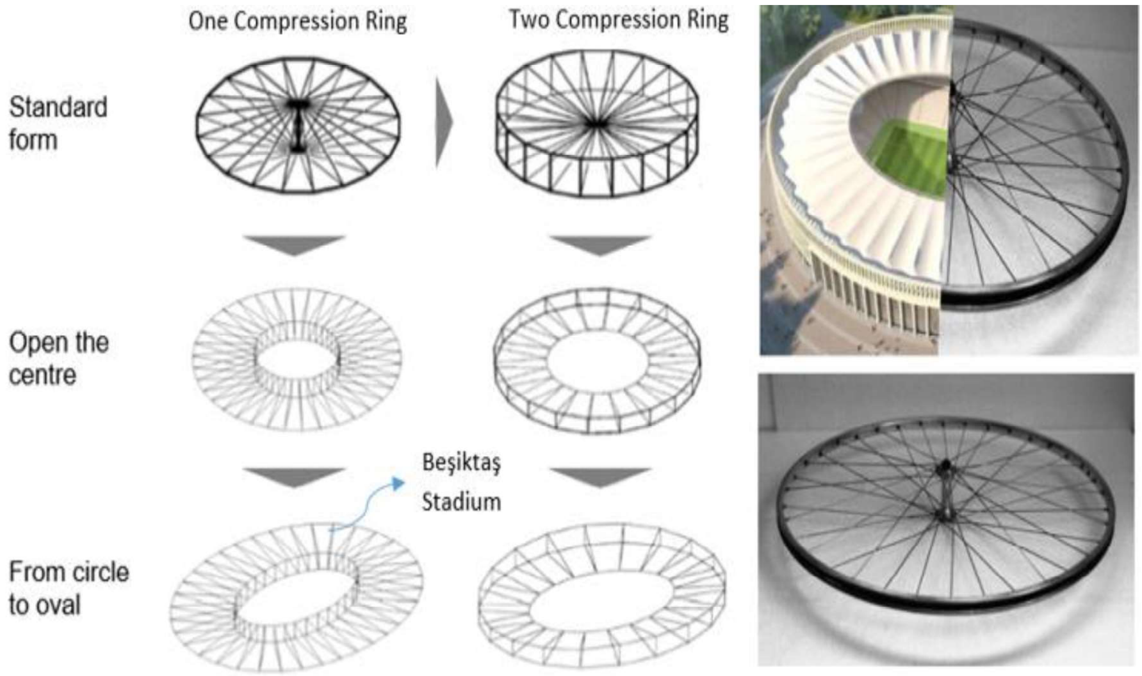


Figure 7. Typical bicycle wheel carrier system applications.

4.Methodology of Structural Modelling

4.1. Analysis Software

The study was carried out with non-linear analysis (GSRelax non-linear) of a model developed with software GSA8.7. It simulates a process of damped vibration in small time cycles. (Oasys Software, web source)

Usually the tension ring is single. However, it can be widely used in systems with double tension ring.

4.2. Numerical Model

4.2.1. Geometry

The geometry for the global model has been created parametrically, optimizing the Compression Ring curvature, Lower Radial variable curvature and bay divisions. All elements are located at the neutral axis of sections, eccentricity between cables work point and inner compression ring, are modeled. No global imperfection is introduced.

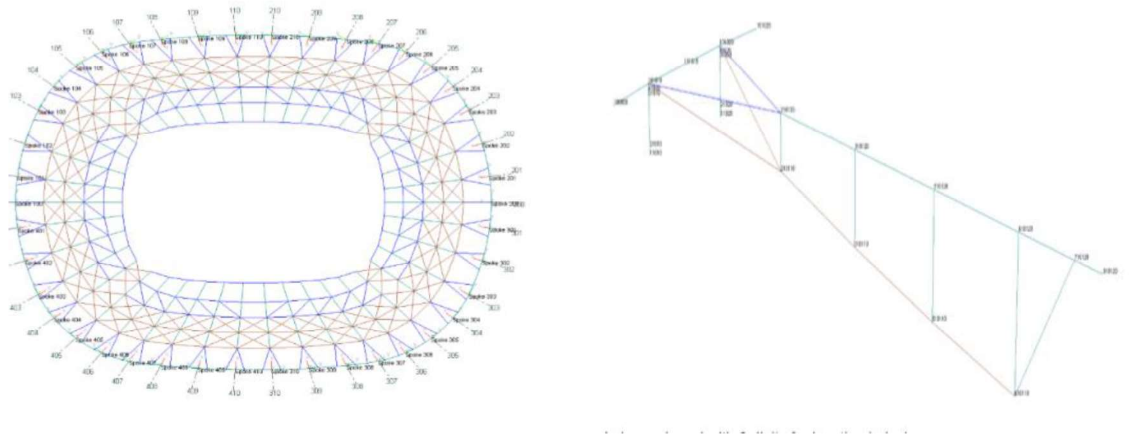


Figure 6. Mathematical model.

4.2.3. Sections and Properties

The listing below shows the section properties used for the global structure model (Table 1).

4.2.3.1. Materials

Table 1. Mechanical properties of the material

N⁰	Name	Type	Young's Modulus [N/mm²]	Poisson's Ratio	Shear Modulus [N/mm²]	Density [t/m³]	αT [1/°C]
M1	Fully Locked Cable	Elastic	155000	0.3	E/2(1+ν)	8	1.2e-5
M2	Steel S355	Elastic	210000	0.3	E/2(1+ν)	8	1.2e-5
M5	Steel S355	Elastic-Plastic	210000	0.3	E/2(1+ν)	8	1.2e-5
M6	Concrete Long Term	Elastic	14000	0.2	E/2(1+ν)	2.4	1.0e-5

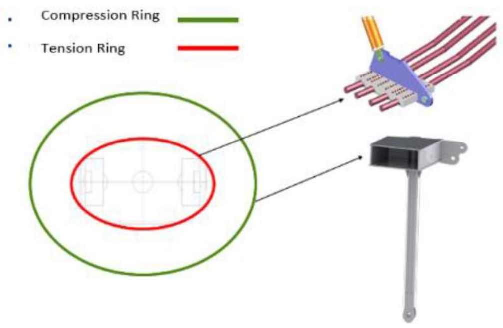


Figure 7. Tension and compression rings.

4.2.3.2. Sections

The following sections have been modeled, including sub-sections used to optimize steel weight:

Table 2. Section properties.

N⁰	Nomenclature	Name	Material	Section [mm]
1	C.R. 1	Compression Ring 1	Steel EC	RHS 600x1400x35x35
2	C.R. 2	Compression Ring 2	Steel EC	RHS 600x1400x35x35
11	F.S. 1	Flying Struts 1	Steel EC	CHS 139.7 x 10
12	F.S. 2	Flying Struts 2	Steel EC	CHS 168.3 x 10
13	F.S. 3	Flying Struts 3	Steel EC	CHS 168.3 x 10
14	D	Diagonals	Steel EC	C 18 (*)
21	F.C.	Flying Columns	Steel EC	CHS 355.6 x 12.5
31	In. S.	Inclined Struts	Steel EC	CHS 219.5 x 12
41	U.R.	Upper Radials	Steel EC	HE300A
42	U.R.	Upper Radials	Steel EC	HE300B
51	L.R. 1	Lower Radials 1	Cable LC	STD C 103 (*)
52	L.R. 2	Lower Radials 2	Cable LC	STD C 75 (*)
53	L.R. 3	Lower Radials 3	Cable LC	STD C 61 (*)
61	U.R.	Upper Radials	Steel EC	HE300B
62	U.R.	Upper Radials	Steel EC	HE300B
63	U.R.	Upper Radials	Steel EC	HE300A
71	U.R. 6	Upper Ring 6	Steel EC	CHS 168.3 x 10
72	U.R. 4	Upper Ring 4	Steel EC	CHS 355.6 x 22
73	U.R. 4	Upper Ring 4 and 3	Steel EC	CHS 355.6 x 12.5
73	U.R. 3	Upper Ring 3	Steel EC	CHS 323.9 x 12.5
81	AT1	Arch Ties 1	Steel EC	CHS 168.3 x 8
82	AT2	Arch Ties 2	Steel EC	CHS 168.3 x 8
83	M. Br.	Membrane Bracing	Cable LC	C 5 (**)
91	U.R.-S.	Upper Split	Steel EC	CHS 323.9 x 12.5
95	L.R.-S.1	Lower Split 1	Cable LC	STD C 75 (*)
96	L.R.-S.2	Lower Split 2	Cable LC	STD C 62 (*)
97	L.R.-S.3	Lower Split 3	Cable LC	STD C 42 (*)
101	L.T.R. 1	Lower Tension Ring 1	Cable LC	STD C 208 (*)
112	H.B.	Horizontal Bracing	Steel EC	CHS 323.9 x 12.5
121	V.B. 1	Flying Bracing 1	Cable LC	STD C 62 (*)
122	V.B. 2	Flying Bracing 2	Cable LC	STD C 62 (*)
151	C. 1	Cuter Columns Lower Part	Steel EC	RHS 280x250x20x50
152	C. 2	Cuter Columns	Steel (***)	CHS 355 x 19
153	C. 3	Columns Upper Part 1	Steel (***)	CHS 355 x 35 (*)
154	C. 4	Columns Upper Part 2	Steel EC	CHS 355 x 35 (*)
201	Mech. Low. 1	Lower Connection 1	Steel EC	C 50
202	Mech. Low. 2	Lower Connection 2	Steel EC	RHS 280x250x20x50

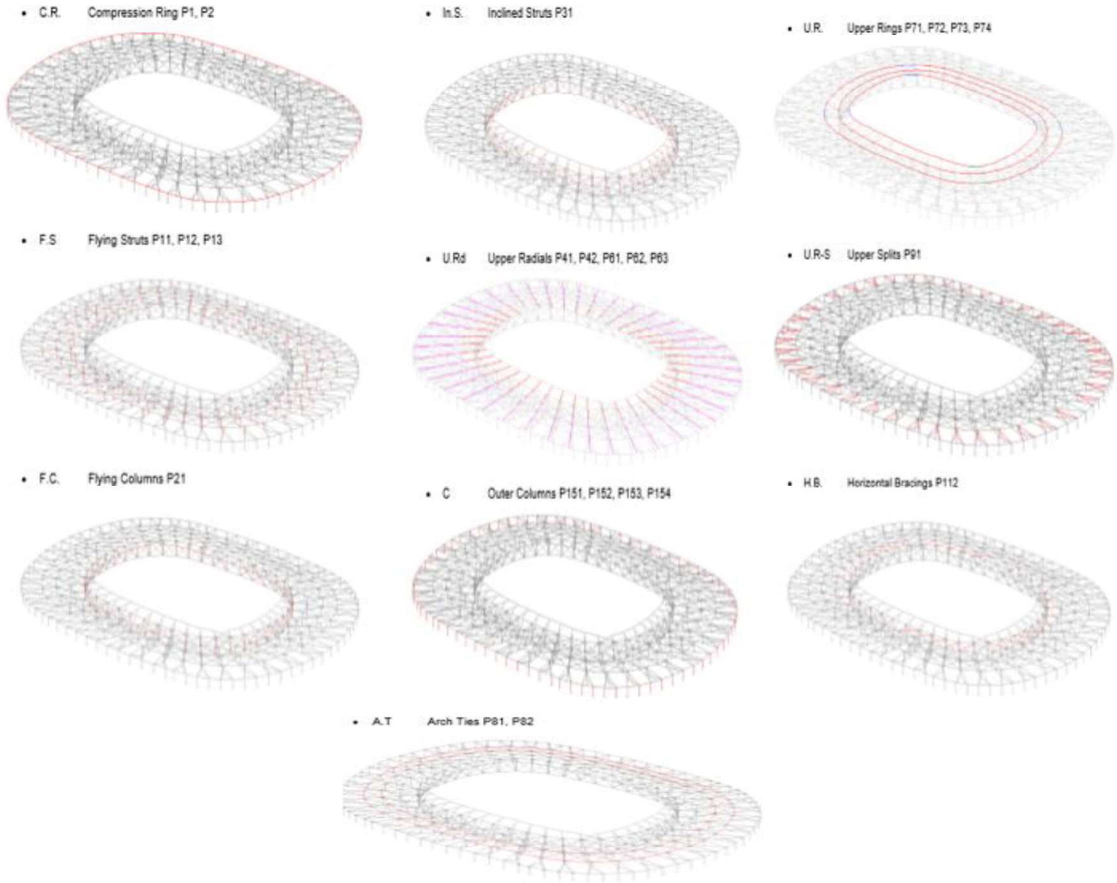


Figure 8. Steel frame elements.

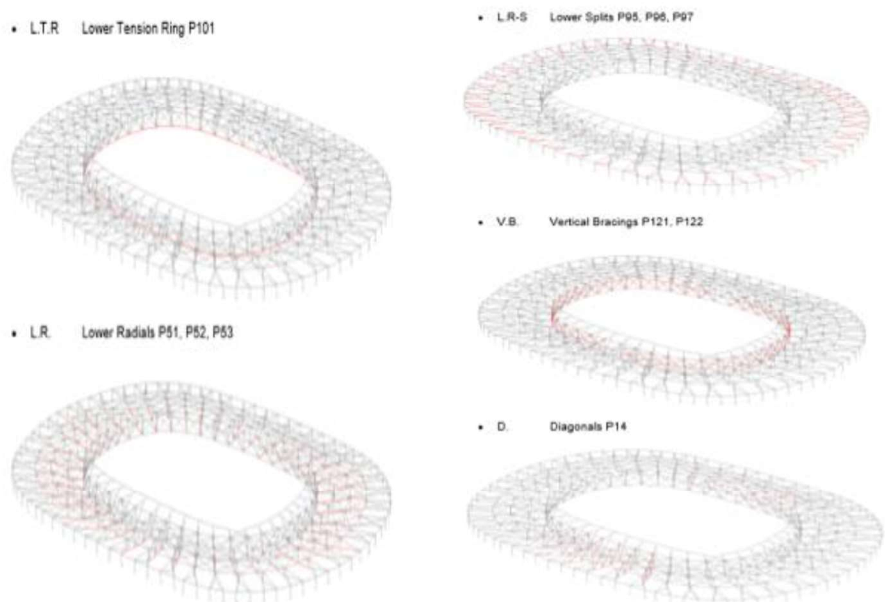


Figure 9. Cable elements.

5. Loads Hypotheses

5.1. Permanent Loads

Density values are in accordance with BS EN1991-1-1: Actions on structures – Densities, self-weight and imposed loads. Steel density is assumed 78.5 kN/m3. Roof cladding weights are described in 5.1.2. Gravity loads are self-weight loads and applied to all elements. Self-weight loads follow the global Z-axis vertically downwards.

5.1.1. Primary Steel Structure

All primary steel structure has been modeled in the structural calculation model, the gravity loads factor was enlarged to 1.1 to cover 10% additional weight from accessories and painting.

5.1.2. Roof Cladding

The stadium roof will be clad with a lightweight pre-stressed membrane; its weight including compression arches and necessary stiffening structure is assumed as 0.10 kN/m^2:
Membrane = 0.013 kN/m; Steel Arches, CHS 193.7x12.5 mm = 0.06 kN/m^2;
Steel Arch Ties, CHS 168 x 8 mm = 0.02 kN/m^2; Edge Cables and fixings = +10 %
No other surface loads (e.g. weight of acoustic or decorative panels) are applied.

5.1.3. Catwalk and Platform

A ring-catwalk along the lower central ring is considered with a permanent uniform load of 2.0 kN/m.
Two radial access bridges are considered with a permanent uniform load of 1.5 kN/m.

5.2. Imposed Loads

5.2.1. Service Loads

The roof structure allows at specific points and along specific axes to receive suspended (or externally applied) service loads. These include:

- Lighting total weight 230 kN, uniformly distributed along the lower central ring (total length 385m ~ 0,6 kN /m).
- Audio service, total weight 275 kN, uniformly distributed along the lower central ring (total length 385m ~ 0,6 kN/m).
- Further equipment and cable weight along catwalk and access walkway 1.5 kN/m
- Video screens, scoreboards 2 x 100 kN, one on each short side, connected to axes 101 201 301 and 401 of the lower central ring.

5.2.2. Additional Equipment Loads

The building user can apply additional technical or decorative equipment to the roof structure (lower central ring TR), e.g. rigging for concerts and shows, via point loads of maximal 5 kN at any of the 42 nodes, respectively as 0.04 kN/m^2. In conclusion, 3 different set of dead loads are used:

• Dead Load Min = Primary Steel Structure + Cladding + Catwalks

• Dead Load Max = Dead Load Min + Service Loads

• Dead Load Erection = Primary Steel Structure

5.3. Prestress Load

Prestress is a set of forces applied to the main elements (Compression Ring, Lower Tension Ring, Lower Radial Cables, Lower and Upper Splits, Upper Radials), which does not produce any deflection and small bending moment in the Compression Ring. This set of forces is the result of an iterative process:

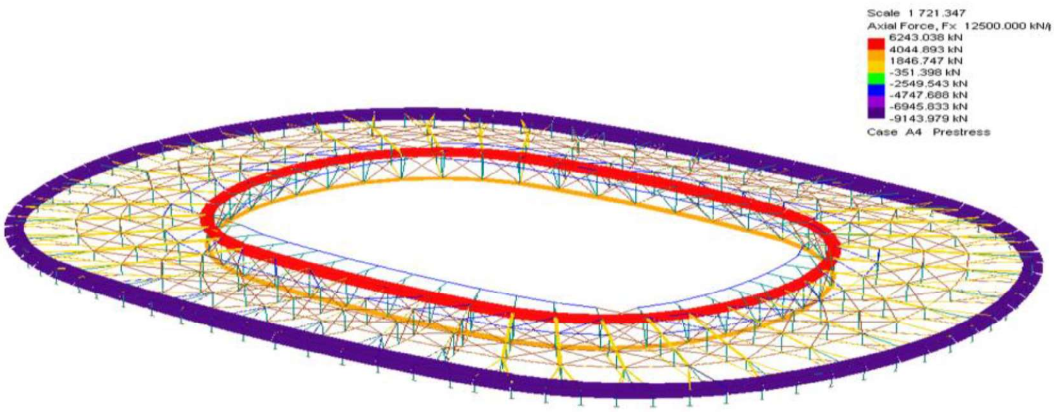


Figure 10. Prestress loads.

5.4. Rainwater Loads

For load and ponding studies, the intensity for 25years of return period is accounted for the design. A minimum slope of 1% at all points and under all circumstances is taken as criteria of acceptance for sufficient rainwater evacuation. Considered peak rain fall values are taken as 4.0 L/sec for 100m².

5.5. Snow Loads

Design ground snow load for Zone II (Istanbul, h<200m) Pk0 = 0.75 kPa (TSE 498)
Slope roof coefficient (angle α=5°) m = 1.00
Design roof snow load Pk = 0.75 kPa
An additional factor of ice build-up of 1.33 is considered, bringing the uniform roof snow load to 1.33 x 0.75 = 1.0kN/m²
Drifted snow shall be considered in the sizing of membrane and supporting elements (arches). Coefficient shall be taken as per Eurocode BS EN 1991-1-3.

For the membrane geometry an angle of max 15° is considered (height 1,5m width of membrane arch 12m) → $\mu 2$ $((\alpha 1+\alpha 2)/2)= 1.2$.

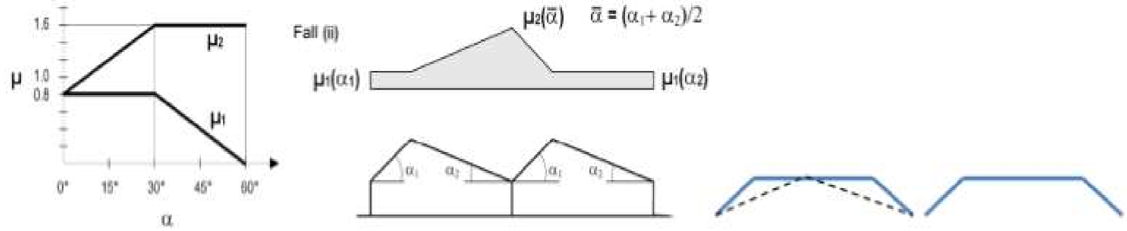


Figure 11. Snow loads on the roof.

The maximum local value of accumulated snow load is 1.2 x 1.33 x 0.75 = 1.2 kN/m².

5 different scenarios are taken into account for the structural calculation:

- Snow uniform and snow uniform exceptional

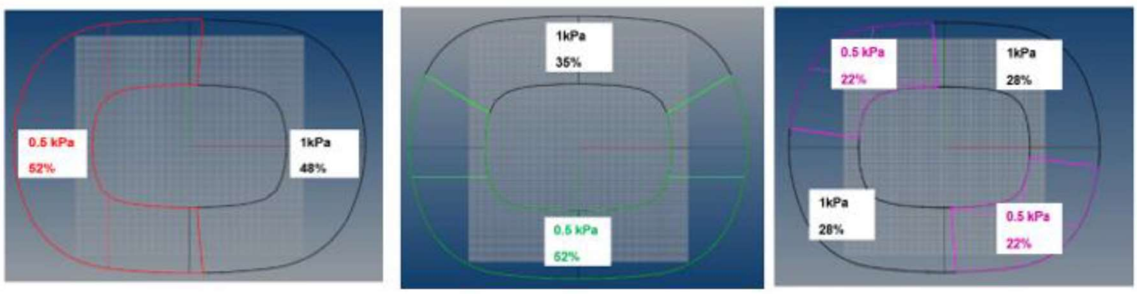


Figure 12. Unbalanced curve side-unbalanced long side-unbalanced unsymmetrical.

Pattern studies have been made to understand the critical unbalanced snow cases.

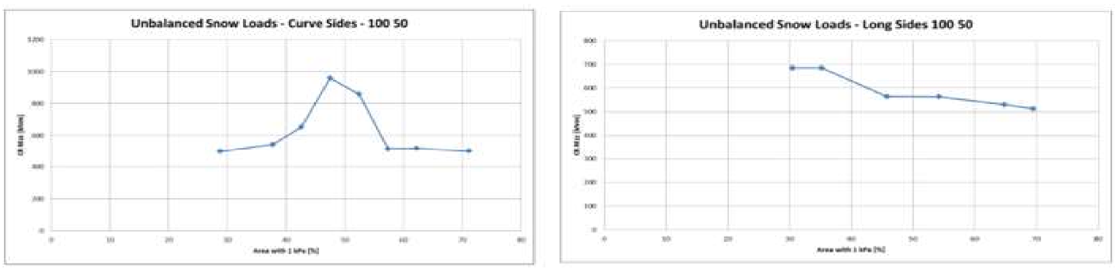


Figure 13. Snow unbalanced loads.

5.5. Thermal Actions

Maximal outside shaded air temperature, Tmax = 41 °C
Maximal outside temperature, taking into account of solar radiation effect, Tout = 41 + 30 = 71°C
Minimal outside shaded air temperature, Tmin = -7 °C
Temperature gradient, ΔT = 13 °C
The total range is 71-(-7) = 78°C. Therefore, in order to be approx. in the middle of this temperature range, the Reference Temperature is T0 = 30 °C. Steel and Cable Suppliers shall take into account workshop temperature during the cutting process of the elements.
4 scenarios of thermal loads are applied:
1. Uniform raise of 41°C on all structure elements.

2. Uniform fall of -37°C on all structure elements.

3. Temperature of all the parts not covered by the membrane (Flying columns) is 13°C higher than exposed steel structure.

4. Temperature of all the parts not covered by the membrane (Flying columns) is 13°C lower than exposed steel structure.

5.6. Supports Movements

The differential movement between adjacent primary concrete blocks due to settlement and expansion joint are considered as:

- ± 30mm in radial direction and tangential direction (60mm in total)
- No differential movement in vertical direction is considered

7 different critical patterns of settlements are considered, respecting the movement joints of the supporting concrete structure:

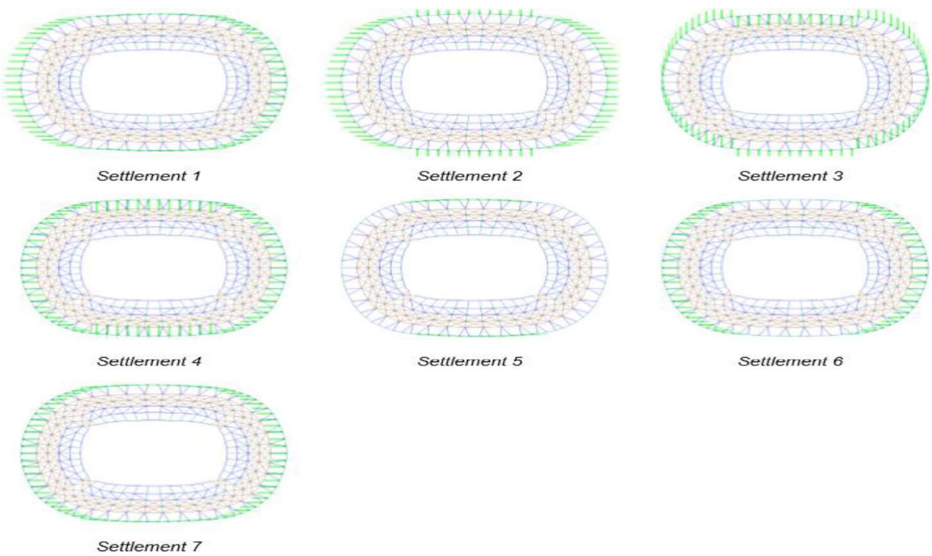


Figure 14. Support Movements.

5.7. Wind Loads

National and related international regulations were used to determine the wind loads affecting the structure. FLUENT fluid finite element analysis program was used for numerical wind tunnel test for the three-dimensional model of the structure since the wind effect is critical for the steel wire rope roof system and these loads are difficult to determine by theoretical methods. In addition, the wind tunnel test was carried out on a small-scale model in Wacker (2013) and the most unfavorable values of realistic wind effects were used in the calculations by checking the Model and Numerical analysis values. It was found.

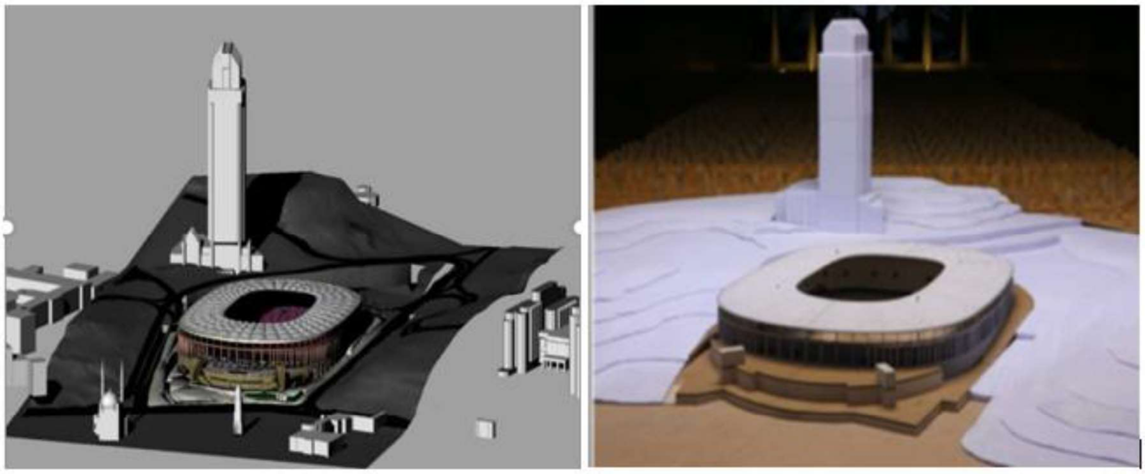


Figure 15. Three-dimensional model of the BJK Stadium and stadium model :1/350 scale.

5.7.1 Computational Fluid Dynamics

Computational Fluid Dynamics or CFD is a numerical algorithm used to solve the partial differential equation known as Navier-Stokes equation:

$$\rho \frac{Dv}{Dt} = -\nabla p + \nabla . T + f. \quad (1)$$

which is a differential form of Newton's 2nd law, incorporating also the law of continuity, i.e. the concept that mass is conserved throughout the volume as the fluid moves, is pressurized, and/or changes density. Here, v is the velocity vector, p is the pressure field, T is the stress tensor and f is the body-force field (such as gravity or electro-magnetic forces). For the simulation of stadium aerodynamics, the following considerations are included in the analysis. ANSYS CFX 15.0 is used to estimate the aerodynamic pressures and wall shear stresses resulting from wind loading at 160 km/h impinging the stadium in the longitudinal (x) and transverse (y) direction.

Air Properties:
Air at 25 C @ Sea Level
Isothermal (no heat transfer)
Turbulence model: scalable k-epsilon
Reference pressure 1 atm:

• ANSYS CFX 15.0 is used to estimate the aerodynamic pressures and wall shear stresses resulting from wind loading at 160 km/h impinging the stadium in the longitudinal direction.

• The geometry is simplified to include a smooth roof for preliminary results, with the structural block near its perimeter which can experience significant pressure from the wind.

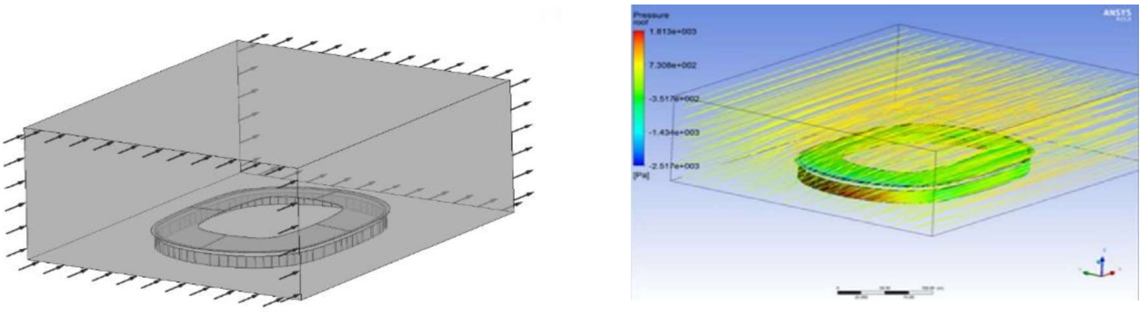


Figure 16. Roof Mesh for finite element methods.

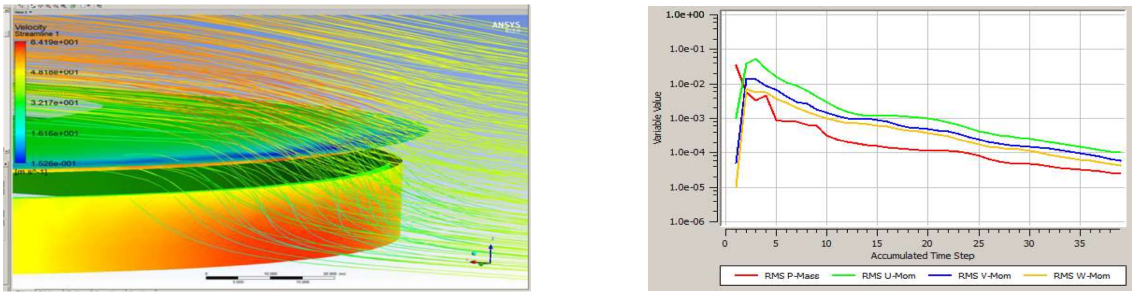


Figure 17. Solution residuals indictaing the error in different variables.

Because of the high-quality mesh, the solution occurs in fewer than 40 iterations. Inflation layers help resolve velocity gradients correctly so the residuals (errors) are reduced quickly and efficiently. The default normalized value of 1e-4 is used as a convergence criterion. (Tello, 2013).

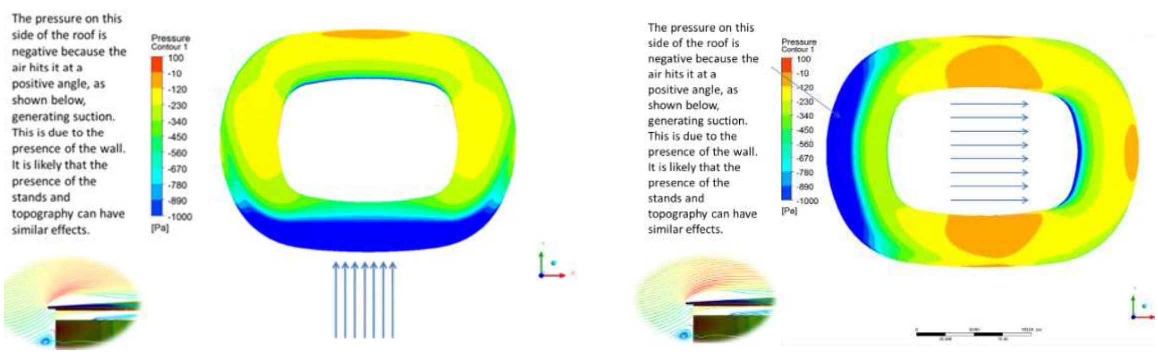


Figure 18. Stress contours on the roof.

5.7.2. Wind Tunnel Test

The extensions of the stadium are about 215 m along the longitudinal axis and about 160 m along the lateral axis. The maximum height of the roof is about 36 m (from the seaside) and about 23 m (landside) respectively. A safe and economic construction and design of the stadium roof of the proposed stadium against wind require the realistic assessment of the structural (area-averaged) peak wind pressures acting on the roof, where both static and dynamic wind loads have to be considered.

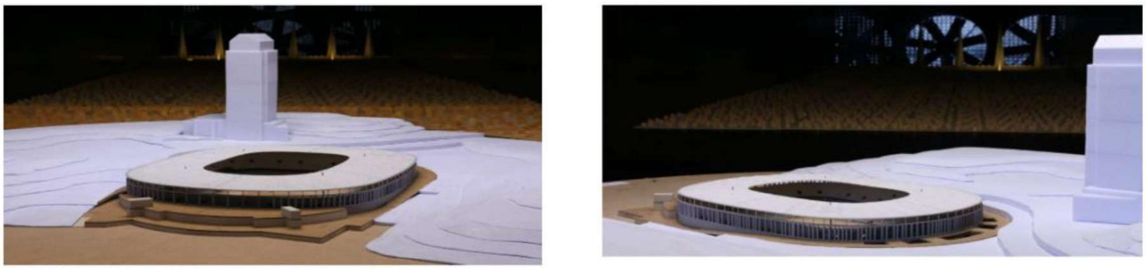


Figure 19. The 1:300 scale model of the stadium in the wind tunnel.

According to the Turkish Code (TS 498, 1997) the gust wind pressure for building heights in between 21 m to 100 m above ground is q = 1.1 kN/m², hence the reference gust wind pressure in stadium height H = 36 m or 23 m respectively would be calculated to : qwİND TSE (H = 36 m / 23 m) = 1.10 kN/m² .The fitted extreme value distribution of the data set leads to the following reference (mean) wind speed for the 50-year return period in 10 m height for open exposure: Vref, 50-years, data = 22.1 m/s. The reference wind speed was also determined as a function of the wind direction. Fig. 22 shows that the highest wind speeds occur at northern wind directions. For other wind directions the calculated reference wind speeds are lower allowing a reduction of the design wind speed (Wacker, 1995; Wacker, 2003).

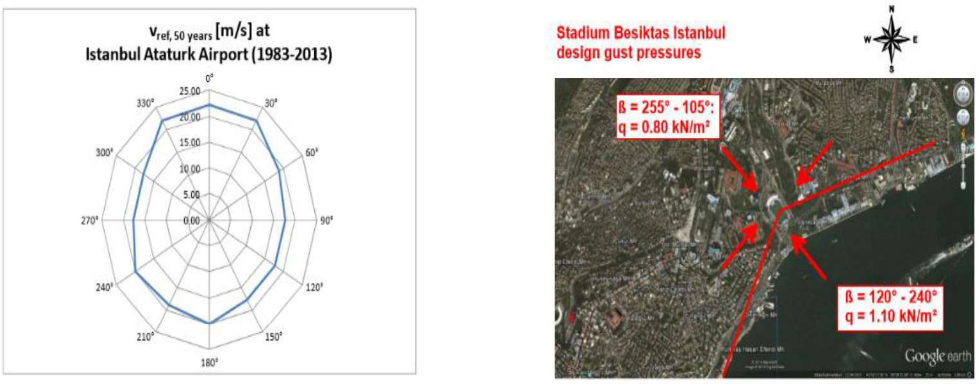


Figure 20. Reference wind speeds (10-min-mean, 10 m height, 50-year return period, open exposure).

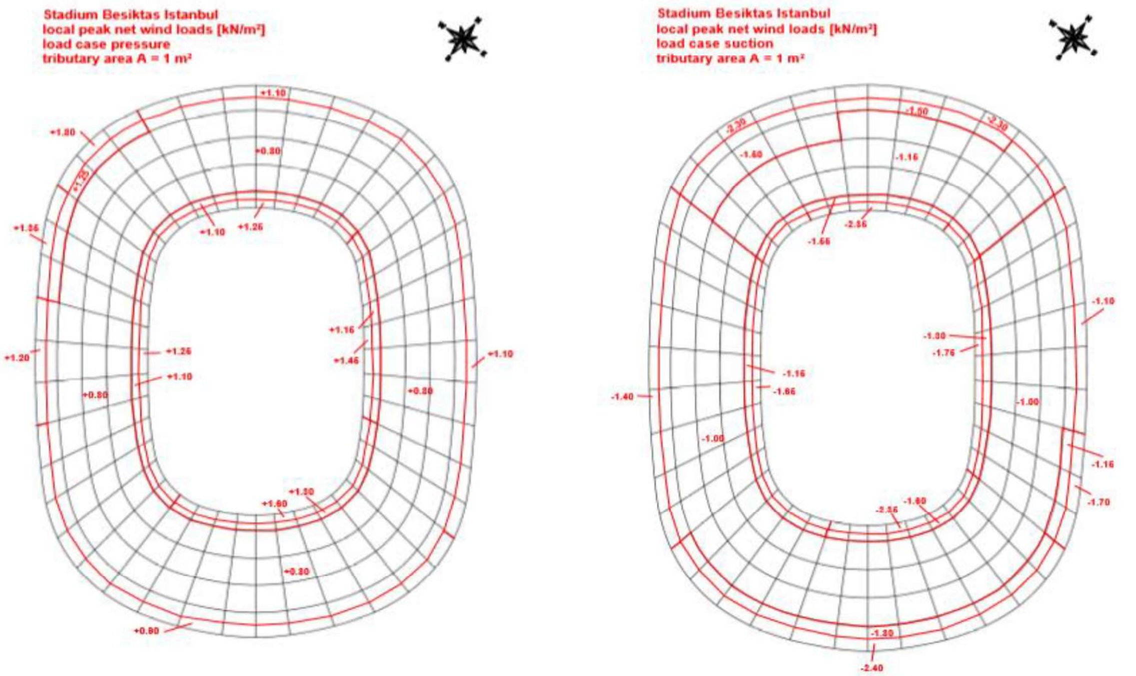


Figure 21. Local peak net wind loads for the design (suction and pressure).

5.8. Seismic Loads

5.8.1. Dynamics Analysis

Based on the information given by the complete model with the Concrete Structure and the Roof Steel Structure has been built:

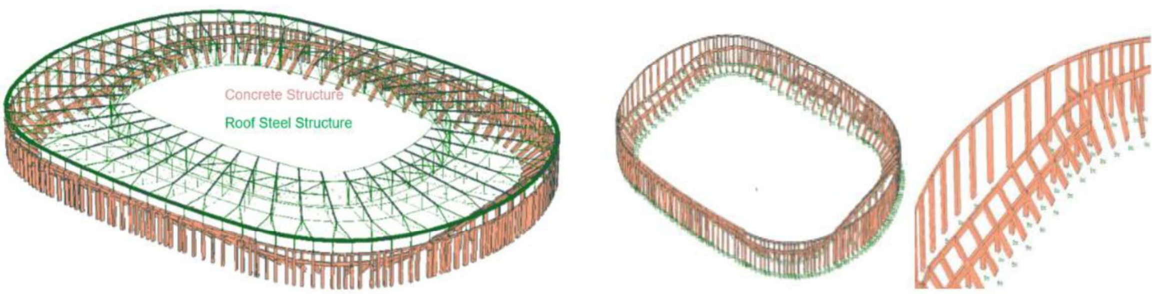


Figure 22. Concrete finite element model.

The following assumptions have been taken: - 119 Concrete Columns fixed at both ends - Concrete Structure fixed at the ends Concrete long term having Young Modulus E = 14000 Mpa

A Ritz P-Δ Modal Analysis has been used as a base for the Response Spectrum Analysis, to include geometric stiffness effects. The P-Δ load case chosen, according to the BS EN 1998-1 3.2.4: DeadLoad max + (Prestress+Geom Compensation) + 0.3 Snow

According to BS EN 1998-1 4.3.3.3 all modes with effective modal masses greater than 5% of the total mass must be taken into account. In order to satisfy this requirement, 100 modes for x, 100 modes for y and 50 modes for z have been considered, where horizontal forces along global X and global Y axes are applied and incremented, monitoring horizontal displacements of the structure. The compared models are:

Table 3. Restraints of the roof column system.

Model Name	Column Lower Connection Radially	Column Lower Connection Tangentially	Lower Mechanism Tangentially (Pin Play)	Column Upper Connection
Model f1	Pin	Fix	Yes	Fix
FIX FIX	Fix	Fix	No	Fix
FIX PIN	Pin	Fix	No	Fix
PIN PIN	Pin	Pin	No	Fix

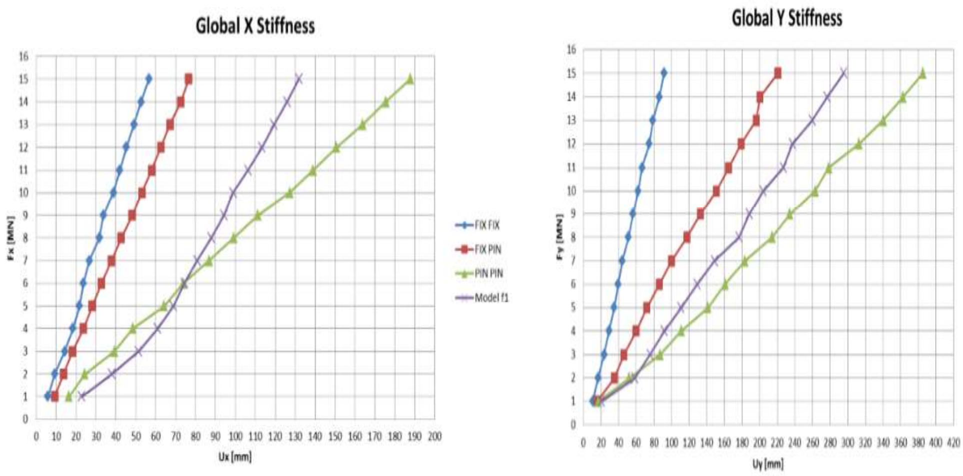


Figure 23. Two directional stiffness diagrams.

Results show that the actual behavior of the structure is between the model "FIX PIN" and "PIN PIN", with horizontal loads higher than 6MN along global X direction and 2MN along global Y direction. Based on this conclusion, the complete model (roof and concrete) includes columns modeled as Model FIX PIN. Being stiffer than Model f1, this approach is more conservative, leading to higher forces. The coupling effect due to the Concrete substructure has also been studied. A simplified model has been done: The frequency of the lower system has been modified, modifying the stiffness, while the upper system frequency has been kept constant:

Table 4. Summary of the modal values

	Model	M [t]	k [kN/m]	f [Hz]	T [s]
ROOF	A	132	9934	1.38	0.72
CONCRETE	1	13200	572700	1.05	0.95
	2	13200	1328000	1.60	0.63
	3	13200	2038000	1.98	0.51
	4	13200	3371000	2.54	0.39
	5	13200	4722000	3.01	0.33
	6	13200	150900	0.54	1.86
	7	13200	30020	0.24	4.17
	8	13200	1884	0.06	16.67
	9	13200	278300	0.73	1.37
	10	13200	820100	1.26	0.80
	11	13200	1655000	1.78	0.56
	12	13200	2644000	2.25	0.44
	13	13200	4000000	2.78	0.36
	14	13200	5530000	3.26	0.31
	15	13200	7080000	3.69	0.27
	16	13200	16520000	5.63	0.18
	17	13200	32480000	7.89	0.13

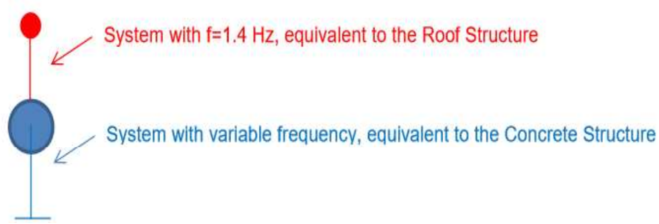


Figure 24. Concrete-Roof structure system frequencies.

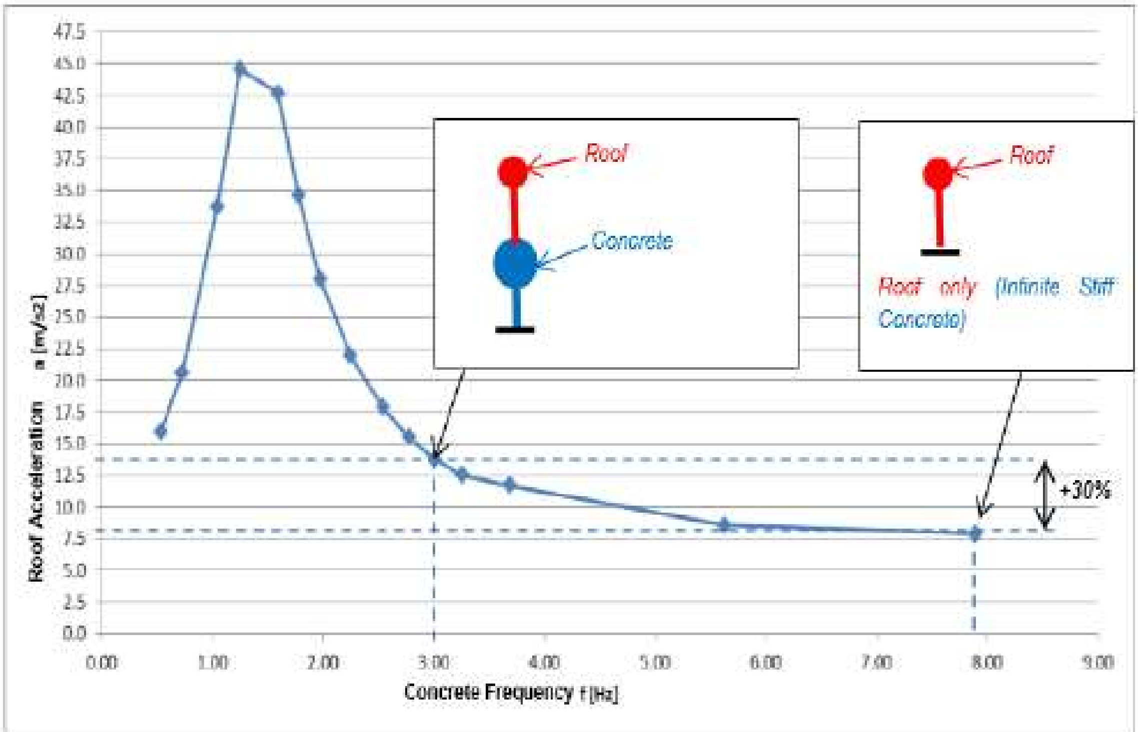


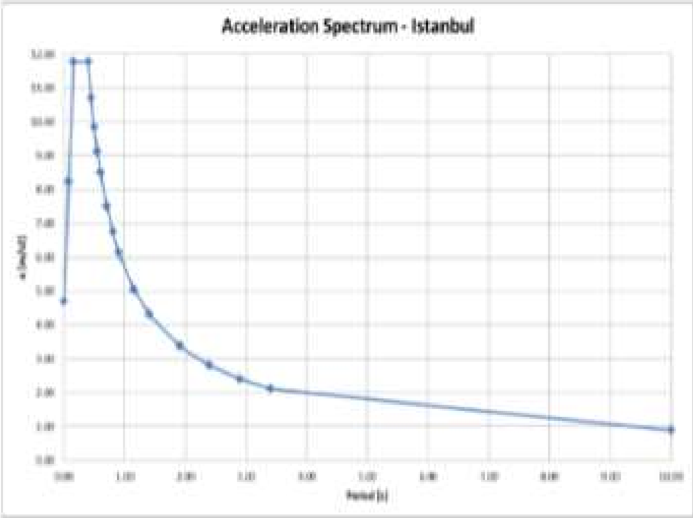
Figure 25. Roof acceleration diagram.

If the Roof Equivalent model is considered alone, i.e. infinite stiff Concrete substructure, the nodal acceleration is 7.4 m/s2: The "Coupling Effect" has an important impact when the ratio $f_{Concrete}/f_{Roof}$ is in the range 1 - 2.5 (the concrete substructure is normally stiffer than the roof structure, therefore its frequency is higher than the roof one). When the Concrete substructure has a frequency of 3 Hz the Roof acceleration increases Roof by 30%, compared to an infinite stiff Concrete substructure.

According to BS EN 1998-1, Table 5. the structure is assumed to be in DCL (Ductility Class Low), having a low dissipative structural behavior: The chosen behavior factor is q = 1.5.

5.8.2. Response Spectrum Analysis

Elastic Acceleration Spectrum is used:



Global Direction	V_{DR+C} [kN]	V_{DR} [kN]
X	11509	9410
Y	7455	7413
Z	5084	3958

Figure 26. Response spectrum curve and base shear force under the earthquake.

Table 5. Ductility behaviour.

Design Concept	Structural Ductility Class	Range of the reference values of the behaviour factor R
Concept a) Low dissipative structural behaviour	DCL	≤1.5-2
Concept b) Dissipative structural behaviour	DCL (Medium)	≤4
	DCL (High)	≤8

Comparing the global forces of the Roof Structure alone and the Roof structure with the Concrete (A B.Y.Y.H.Y., 2007).

5.9. Load Combinations

Load combinations turkey and Eurocode have been completed according to the most unfavorable value.

- Service Limit State (SLS) Combinations: SLS Characteristic combination is used for checking deflection criteria.
- Ultimate Limit State (ULS) Combinations: ULS combination is used for sizing of structural members.

6. Conclusion

The design criteria for Besiktas Stadium were explained and structural information about the stadium structure was given. The design of the stadium structure, including the construction phases, has been a process that requires the co-operation of architectural, design engineers, field engineers, academics and many disciplines in a coordinated manner. The stadium structural system consists of many different bearing elements and the performance of the structure depends on the correct modeling of the behavior of these elements. Steel construction superstructure Spoke Bicycle wheel technology Steel ring beam pressure ring and Rope Pull ring and main pressure ring on the 82 super-pillars are sitting on and in case of earthquake, a significant amount of horizontal load is transferred from the roof to the super-columns. The roof is 10m with very advanced technology. This type of gabari is a record in the stadium roofs.

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