The New Sport Centre in Thermi Thessaloniki: conceptual Design and Analysis of the Structural Steel Systems
for

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Abstract

This paper illustrates the three most important steel structural systems adopted for the Thermi sports facilities:

- The roofing system for the main Sport Hall;
- The roofing system for the Training Hall;
- The movable play ground,

involving, principally, aspects of conceptual design and theretical and experimental analysis of long span steel structures.
The conceptual design of long-span clear structures, in an holistic approach, must consider carefully the current state of the art and the related knowledge base in order to assure the required reliability level.
Due to the different scales and from the observations of the in-service performance of long-span structures, several special design aspects arise as (1):

- the snow distribution and accumulation on large covering areas as a function of statistically correlated wind direction and intensity;
- the wind pressure distribution on large areas considering theoretical and experimental power spectra densities or correlated time histories;
- rigid and aeroelastic response of large structural systems under the action of cross-correlated random wind action considering stati, quasi-static and resonant contributions;
- the time-dependent effect of coactive indirect actions as pre-stressing, short and long-term creeping and temperature effects;
- the local and global structural stability;
- the non linear geometric and material behaviour;
- the reliability and nominal callibrated safety factors of new hi-tech composite materials;
- the necessity to avoid and short-circuit the progressive failures and collapse of the structural system due to initial partial accidental damages;
- the compatibility of detail design with the structural model and response;

- the parametric sensibility of the structural system depending on the type and degree of static indeterminacy and hybrid collaboration between hardening and softening behaviour of substructures.

The long span structures of the main sport hall need special investigations concerning the actual intensity and distribution on large covering surfaces. National Building Codes and Eurocode1 are normally addressed only to small-medium-scale constructions. The uncertenties related to the random distribution of live loads on long-span structures imply a very careful loading analysis using special experimental investigations regarding the snow drift, accumulation factors and the dynamic action of wind loading. The results of the wind tunnel experimental analysis in turbolent flow are synthetically illustrated in the paper with the aim to extend the knowledge base by monitoring the structural system in order to control the subsequent performance of the structural behaviour and callibrate theoretical analysis.

1. Introduction

Mettere breve excursus storico e descrizione del progetto generale con planimetria e/o foto del modello.

Figure1. Global view of the new sport centre

Accennare alla utilizzazione degli Eurocodici e correlazione con la normativa locale.
Descrivere i materiali adottati quale scelta architettonico-strutturale.

2. Main Sport Hall roofing system

2.1 The conceptual design

The conceptual design of the roof structural system, taken place during the preliminary design phase, was mainly based on the following functional requirements and hyphotetical mechanical behaviour aspects:

* long span structures;
* strong seismic actions;
* seismic interaction between sub-structures;
* thermal - seismic correlation;
* roof shape - snow distribution correlation;
* roof shape - wind action correlation

The long span functional requirement and the seismic actions to be considered (related to the site conditions), strongly influenced the choice of the structural typology coming to the adoption of a light- weight prestressed cable hybrid structure..
The conceptual synthetical decition converged to a design idea adopting a long span cable structure for the main structural system to be longitudinally oriented, giving the possibility to separe the costruction into two symmetric sub-structures The sub-structuring of the construction permits: to obtain uniformity, subdividing the entire building into dynamically quasi-independent units (see Eurocode n.8), a better understanding of the structural response and, therefore, a simpler numerical simulation of the mathematical model.
The two sub-structures are formed by the concrete grand stand frames and a set of simple supported armed beam-cable roof structures .The transversal beams are supported, externally, by the concrete stand structures and ,internally, are suspended from the main central tensile structures by a special pendulum suspension detail. The pendulum suspension system have the property to elastically shortcircuit the two side substructures giving the possibility to obtain, a soft dynamical (t>2 sec.) transversal response of the main cable system, an uncorrelated thermal and seismic

behaviour between the two stiff and massive grand stand concrete construction (placed at a relative distance of 130m) and to avoid transmition of seismic forces and seismic displacements trought the roof lightweight structures.
For the seismic actions, directed along the longitudinal axis, a so-called seismic cable connects all the pendulum support points transfering the horizontal forces to a ground anchorage. A special viscoelastic spring-damping system will connect the seismic cable to the anchorage foundation in order to encrease the ductily of the structure response under seismic action.

2.2 Description of the roof structural system

The structural system is principally formed by:
* a longitudinal prestressed cable truss main system;
* a transversal prestressed cable beam system;
* a roof deck system.

2.2.1 Longitudinal cable truss system

Two 185 m. span prestressed cable structures, longitudinally oriented, separed parallely by a distance of 20 m., are the main carrying structure of the roof system, which is formed principally by:
- an upper carrying cable;
- a lower stabilizing cable-beam ;
- interconneting posts and diagonals;
- a column-stayed anchorage frame;
- a gravity anchorage and footing on piles foundation.

The upper carrying cable have an almost parabolic shape with a span of 185m. and a sag of 20 m. (10 % of the span) .The anchorage level of the cable is at 38.70 m.. The carrying cable is realized by 8 full locked class B zinc coated steel cables of 90 mm. diameter with open bridge sockets anchorage heads.
The lower stabilizing beam-cable have a span of 18? m. and a sag of 1?m., crossing the upper cable of opposite curvature at a level of ??m. and at ??m.. The section of the beam-cable element is formed by 4 HEAA 300 Fe 510 D steel, organized an connected according a Vierendell typology; the cable is formed by 8 galvanized 46 mm. full locked steel cables with open bridge sockets connected to the H profiles and adjustable cylindrical sockets, with internal and external thread, connected with the concrete anchorage foundation. The anchorage of this element is situated at 5 m. from the play ground level.
The upper and lower cables of the two tensile structures are interconnected each other, in plane and out of plane. In the plane of the structure, vertical posts realized by two crossed IPE 550 Fe 510 D profiles separe the upper and lower cables ; diagonal cable stays of ??mm. diameter X-brace the posts and stabilize geometrically the central part of the structure between the crossing points of the cables.
Out of plane of the cable structures,a stabilizing wind bracing system is placed in the cylindrical surface realized by the connection of the lower cables with horizontal box section steel beams and wind bracing spiral steel cables. In the central part of the cable trusses two trasversal wind bracing systems , placed in vertical planes, are provided.
The anchorage forces, developped by the cable trusses, are transmitted to an anchorage structure formed by a transversal wind braced frame and stay cables anchored to the ground. The columns of the frame have a similar section configuration to that of the Vierendell beam previously described ; they are connected , top and bottom , by a steel box section beam and the wind bracing is realized by pairs of prestressed steel cables. The columns are hinged in the plane of the structure and are framed out of plane.

The stays are formed by n. 8 full locked cables of 90 mm. diameter , fixed to the column by open bridge sockets and to the foundation, with adjustable cylindrical threaded sockets.
The forces, transmitted by the anchorage frame, are taken by the foundation system. The stay forces are equilibrated by a gravity foundation, similar to that of the suspended bridges; with a dimension of ??x?? in plan and ?? m. in elevation. The dead load of the anchorage foundation is supported by a pile foundation group, in order to minimize the differential settlements due to the particular soil conditions (?).
The foundation corresponding to the frame is realized by a discrete footing on piles with the dimensions of ??x??Xx??m. The piles of 1.20m. diameter, working principally by lateral friction, have a lenght of 27.50 m. and develop the necessary nominal carrying capacity with groups of ?? and ?? piles respectively for the anchorage and the footing foundation.

2.1.2 The transversal cable-beam structures

Along each side the longitudinal axis of the sport hall, two sets of beam-string structures, radially oriented, are internally suspended from the main central supporting cable structure and externally supported ,by the reinforced concrete structures of the grand stands.
The beam-strings (14 each side) acts as simple supported structure with an average separation of around 10m and a variable span from 55m to 12m.
The upper members, in compression and bending, are formed by H-beams from 1.40m to 0.50m high. The bottom members are realized by cables with an initial prestressing level addressed to optmize the bending distribution along the lenght.
The beam-string structures are self stibilized out of plane by a tringular separation of the uppr members in plan view. Trasversal rigid connections avoid twisting of those slender members.
The gravitational and seismic support reactions are transferred on the concrete structure by special devices in order to avoid parassitc end reactions.

2.1.3 The multicellular roof deck

In order to avoid secondary beams in the roof structure, a multicellular steel deck, made with steel corrugated metal sheets, was designed. As shown in the sketch (Fig.?), the deck is realized with two superimposed layers of steel metal sheets 160mm high and 0.88mm thick; the bottom layer lightened by microholes for acoustic reasons.
The two steel layers are connected by rivets in the neutral zone in order to transfer the shear stresses, realizing a compact composite structural section.
The multicellular steel sheets arrangement, allows for carrying the roof loading for a maximum span of 12m. between the main cable beam transversal structures.
The theoretical analysis has been verified according with the limit states methodology according with the Canadian Code related to “cold formed steel structural members”, CSA standard S136-94.
A special software programme was written in order to implement a non linear numerical procedure addressed to the definition of the “effective section”. As shown in Fig.? Is possible to observe that 5 non linear steps were required.
The gravitational loading patterns were determined according to Eurocode 1; the wind loading has been determined by experimental wind analysis considering, in function of the dimension of the loaded area, the local peak pressure coefficients acting on the roof panel.
The theoretical analysis has been integrated, according point8 of EC3 by experimental analysis. The results of the experimental investigations made in the DISTART Laboratory of the University of Bologna, gave very positive response for service and limit state conditions (Fig.?).

2.2 Some aspects concerning the structural analysis

The structural analysis has been elaborated according to the displacement method .
In function of the nature of the loading and the mechanical behaviour of the structure, several theoretical models have been analysed and addressed to the evaluation of the most significative scenario of the defomative and stress state of the structural system and structural elements.
Partial model analysis has been adopted in function of the weak mechanical interaction between sub-structures in order to produce a cross control of results between linear and non linear methods and partial and global models.

2.2.1 The non linear model

- the STATE “0”

The first step of the analysis consists in the definition of the geometric configuration of the structure, under a predetermined initial state of stress: the (STATE “0”).
For the determination of the state “0” a typical shape finding procedure has been applied, simulating the introduction of the required level of prestressing in two steps according an hyphotetical erection sequence:

-step 1 - erection of the cable structure,prestressing under the action of dead weight;

-step 2 - application of the global permanent loading and 2nd. level of prestressing evaluated as a inear extrapolation of the previous calculated variation under loading.

The incremental shape finding procedure, elaborated by the program RETE ,introduce an initial state of stress in presence of the global permanent loads, representing the service state of the building. The prestressing residual level, under the action of nominal live loads, must be sufficient in order to assure the static and dynamic stability .
The model consists in ?? joints, ?? cable elements and ?? truss elements as shown in fig.?
The main results are illustrated in table n.?.

(fig.? e tab.?)

- the variation of state

The typical geometric non linearity of the cable structures, under the action of the loading ,is taken into account by the program TENSO.
The loading transmitted from the roof sub-structures to the cable structures, according to the loading combinations previously defined, are applied, as concentrated forces, in correspondence of the suspension points, acting as boundary elements in the logic of the weak interaction between sub-structures as described above.
The output results, contained in enclosure n.2, are related to the following analysis:

- Static non linear analysis
- Service Limit State

All the loading combinations prescribed were considered and the most significative results are sinthetically illustrated in table?.

(tabella combinazioni di esercizio)

- Ultimate Limit State

Two limit full loading combinations, due to snow and wind actions, were considered for the investigation of the structural response under limit state loading.From the results shown in table n.? is possible to observe the important result that under incremental non linear behaviour the carrying capacity of the main structure remains almost unmodified and still in the elastic range.

This very important result is obtained by the action of the stabilizing cable acting as a pre-stressed spring softening the global response.

(tabella combinazioni ultime)

- Dynamic seismic analysis

The seismic action has been analysed by the modal superposition method considering the characteristic elastic design and a strong motion response spectra. Due to the inherent linear properties of the method only the linear part of the non linear model was considered but the geometric stiffnes were included in the analysis.

Considering that the statistic combination factors allow to reduce to a 30% the mass of the snow loading, the results of the seismic analysis are not relevant to the resistant verification of the elements and sections. The seismic analysis are ,therefore, addressed principally to the verification of the compatibility of the substructures differential displacements.

The results of the modal analysis were compared with the Ritz vector method in order to control the validity of the assumptions.

The most relevant results are synthetically illustrated in table n.?.

(mettere figure di configurazioni modali)

- Wind analysis

The response of the structural system under the wind action has been analysed integrating, into the design process, the wind tunnel experimental data obtained in the boundary layer wind tunnel laboratory of the University of Western Ontario. The model at a geometric scale of 1:250, was constructed in plexiglass and instrumented with 508 pressure taps.

The statistical wind climate for Thermi was developed from two meteorological data sets. The Mikpra site contains monthly and annual extremes from 1959-1992 and the Sedes site contains informations from 1959-1972. The building site is situated midway and the combined data sets give a 50 year surface wind speed of 30m/sec over open country (roghness lenght=0,03m.) consistent with Eurocode1 design values for similar latitudes. The reference height for this study is 40m and the stronger wind approach is from 290 degrees.

The prediction of the 50year return period local peak differential pressures and suctions has been measured in the central cable-supported structure, the two side domes that span from the main arch structure and in the lateral curtain wall surrounding the building. The largest suction reported on the arch is 1.61 kPa. The largest suction for the domes occurs on the southeast quadrant near the gap between central and lateral roofs. For the curtain wall a suction of 2.10 kPa occur in sothern side projection.

Drag, uplift and moments were calculated for the entire roof, and for the three substructures separately.

The dynamic analysis of the roof structure has been elaborated according the orthogonal decomposition method, Typical pressure mode shapes are shown on Fig.??? With a typical power spectrum of the first mode for a wind angle of 70 degrees. The peak factor g was estimated in 4 and the resonant amplification effects do not play a significant role considering that the bulk of energy is in a range much lower than the vibration frequencies of the cable structure and the side domes.

3. The movable play-ground

One of the main features of the design of the Thermi sport hall is represented by the adoption of a vertically movable play ground. This system gives a definitive solution

to the classical architectural problem related to the polyfunctional compatibility and optimization of different play-ground patterning.
The movable playground allows a vertical displacement of 4.10m. giving a double geometrical configuration, having the same visual slope, for the basket ball (level -2.05) and the indoor athletic activities (level 2.05).
The main dimensions of the movable floor system are 80x40m.(?), for an extension of 3200sqm..
The floor, rests on 96 hydraulic jacks acting as lifting columns, placed in a central rectangular grid of 6x6.50m. in plan view; but in order to fit the border line of the floor, the external mesh is somewhat irregular.

Fig.? Playground configuration and actuators arrangement

The structure of the floor is made by a composite steel-concrete slab with a standard beam grid patterning coincident with the above mentioned jacking arrangement
The reinforced concrete slab of 20 cm. thickness is connected to the H-steel profiles by Nelson typical welded studs. This system, gives the required vibration performance level of the floor structure dynamic response (1^{st} natural frequency over 6 Hz.) under rhytmic antropic events, according with the National Building Code of Canada and the recommendations of EC3.

Fig.? Typical floor slab section showing the hydraulic actuator

At -2.05m level the floor slab is directly supported by the foundation pile system (lower position); during the lifting operation the hydraulic actuators equilibrate the permanent loads and when the floor reaches the upper position, the actuator's rods shall be mechanically fixed by a locking device. In this condition the actuators shall be mantained at reservoir pressure and the actuator's rods will act as a steel column working as a truss member.
The nominal actuation time for a full stroke corresponding tothe basketfield position, both rising or descending, shall be 3600 sec., from operation starting to the final stop.
In case of system failure the actuation time shall be doubled and the floor speed shall never exceed 0.6mm/sec.
During the vertical displacement of the floor structure , having no horizzontal stiffness, is stabilized by 8 vertical sliding guides. After matching the two working static positions the lateral stabilization of the floor is obtained by horizontal simple effect viscoelastic actuators having a load-displacement constitutive law acting as a locking device under dynamic seismic response.
The design of the movable floor is elaborated under a high level of relaibility. Partial statistical failure modes has been numerically simulated in order to avoid long term out of service. The system is allowed by a monitoring system detecting differential displacemets and , by load cells applied between the actuators and the floor slab steel beams, wil be possible to control the actual loads acting on each actuator.

4. The roof structure of the Training Hall

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5. Conclusions

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Acknowledgements

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References

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