HS STEELS IN TENSION STRUCTURES

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ABSTRACT

Long span roofs are today widely applied for sport, social, industrial, ecological and other activities. The experience collected in last decades identified structural typologies as space structures, cable structures, membrane structures and new - under tension - efficient materials which combination deals with lightweight structural systems, as the state of art on long span structural design. In order to increase the reliability assessment of wide span structural systems a knowledge based synthetic conceptual design approach is recommended. Theoretical and experimental in scale analysis, combined with a monitoring control of the subsequent performance of the structural system, can calibrate mathematical modelling and evaluate long term sufficiency of design.

INTRODUCTION

Long span structures are today widely applied mainly for :

<u>Sport buildings</u>
- Stadia
- Sport halls
- Olympic swimming pools
- Ice tracks and skating rinks

<u>Social buildings</u>
- Fair pavilions
- Congress halls
- Auditorium and theatres
- Open air activities

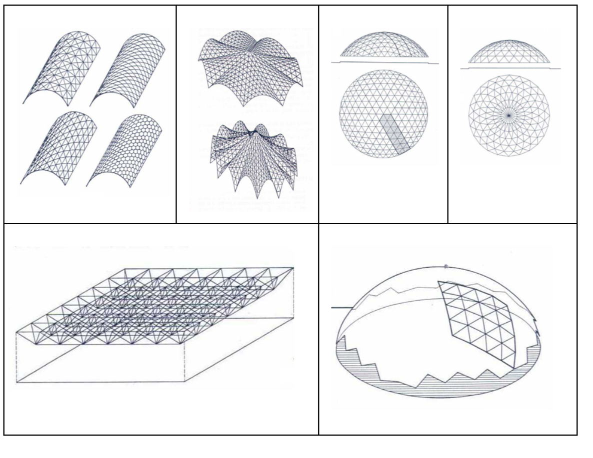
<u>Industrial buildings</u>
- Hangars
- Warehouses
- Airport terminals
- Waste material storage

The state of the art trend on widespan enclosures: the lightweight structures - from compression to tension.

According to the state of the art, the more frequently typologies and materials used for wide span enclosures are:

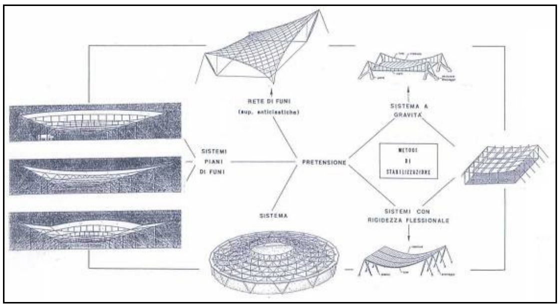
Space structures

- single layer grids
- double and multi layer grids
- single and double curvature space frames [1]



Cable structures

- cable stayed roofs
- suspended roofs
- cable trusses
- single and multilayer nets

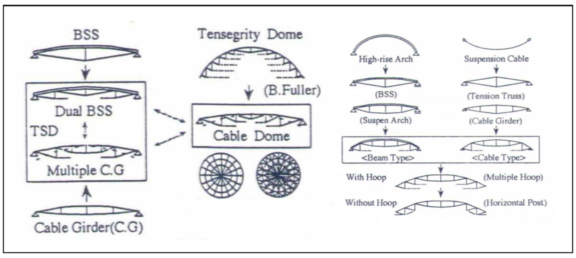


Membrane structures

- Pre-stressed anticlastic membranes
- pneumatic membranes

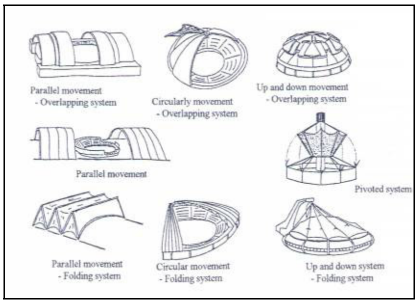
Hybrid structures

- tensegrity systems
- beam-cable systems



Convertible roofs

- overlapping sliding system
- pivoted system
- folding system [3]



The historical trend in the design and construction process of wide span enclosures was and is the minimization of the dead weight of the structure and , consequently, the ratio between dead and live loads (DL/LL).
From ancient massive structures (DL/LL>>1) to modern lightweight structures (DL/LL<<1), the DD/LL ratio was reduced more than 100 times due to the most effective exploitation of the properties of special high-strength materials, in combination with structural systems where tensile stresses are dominant (Tension structures). Due to the inherent stability of tension against compression, tension structures leads naturally to optimization of the system energy against structures which are subjected to bending moments or are stressed axially with the possibility of reversal from tension to compression, as is the case with grids and framed structures. Therefore, the actual trend on lightweight structural typologies is to combine, as far as possible, a dominant tension mechanical system and hi-strength materials.

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In Table 1, is possible to observe the exceptionally efficiency of conventional and HS steels and hi-tech materials observing the strength to weight ratio (K=σ/γ) in tension (Kt). Considering also the cost/weight ratio and the inherent reliability, steel remain the reference construction material for long span structures. The different mechanical behaviour of compression and tension structures can be illustrated by Fig.1 where, starting from a thin parabolic arch under uniform distributed load , it is possible to observe, during incremental loading, the following phases of the load displacement curve:

−Phase A: unloaded structure.

−Phase AB: compression phase; geometric softening; decrease of tangential stiffness, reduction in the positive value of the secondary term of the total potential energy $\delta^2\pi$.

−Phase BCE: unstable phase; dynamic displacement from B to E; liberation of kinetic energy (cross hatched area). Here, the secondary term of total potential energy is negative ($\delta^2\pi < 0$).

−Phase DEF: tension phase; geometric hardening increase in the tangent stiffness, branch of stable equilibrium with increasing value of secondary term of the total potential energy ($\delta^2\pi > 0$). Phase DEF is characteristic of the behaviour of tension structures. The non-linear geometric hardening results in a less than proportional increase of stresses in relation to increase external loads. This provides an increased nominal safety factor evaluated at ultimate limit state (β safety index).

MATERIALS	σ_t^R N/mm²	σ_c^R N/mm²	γ_k N/m^3 10^3	K_t m	Kc m
Bricks		3	18		166
Wood	85	37.5	5	21.250	9.375
Concrete		30	25		1.200
S 355	520		79.5	6.664	----
S 460	640		79.5	8.050	
S 690	860		79.5	10.080	
S 850	1050		79.5	13.376	----
Titanium	900		45	20.000	----

Composite materials hi-tech

Unidir. Carbon fibres	1400		15.5	90.000	
Textile carbon fibres	800		15.5	52.000	----
Unidir.Aramidic fibres	1600		13	123.000	----
Textile aramidic fibres (Kevlar)	750		13	58.000	----
Unidir. Glass fibres	1100		20	55.000	----
Textile glass fibres	450		20	22.500	----

Table 1. Mechanical properties of construction materials

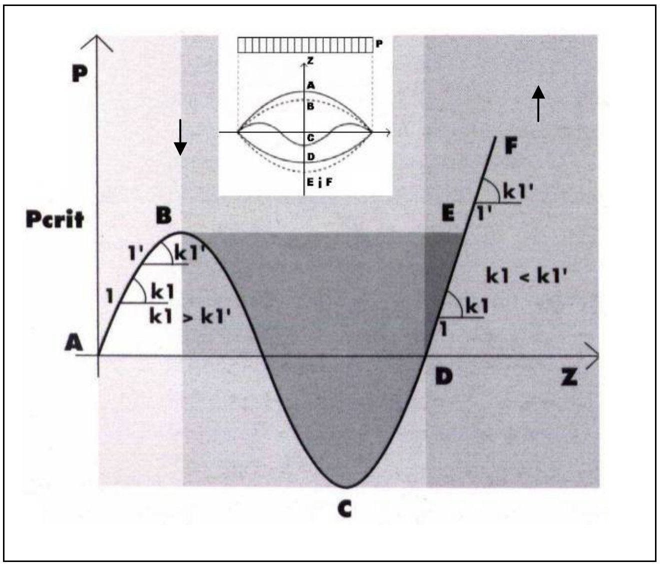

Fig.1 Mechanical behaviour from arch to cable

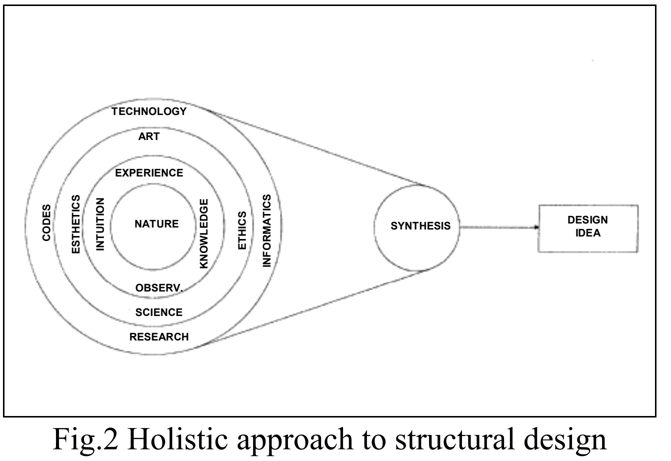

Fig.2 Holistic approach to structural design

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1. KNOWLEDGE BASED CONCEPTUAL DESIGN AND RELIABILITY LEVEL

The conceptual design is knowledge based and, basically, property of individual experts. Their involvement in early stages of design is equivalent, from the reliability point of view, to a human intervention strategy of checking and inspection and, from a statistical point of view, to a "filtering" action which can remove a significant part of “human errors”.
According to the design requirements, the conceptual design is defined by a knowledge expert synthetic approach based on the reliability intuition of the selected model which has to be confirmed by the results of the analysis phase. The conceptual design approach is holistic and directly depends on the skills and abilities of the design team members (Fig. 2).

1.1. Special aspects of conceptual design decisions on long span structures.

Considering the “scale effect” of long span structures several special design aspects arise as [2]:

−the snow distribution and accumulations on large covering areas in function of statistically correlated wind direction and intensity;

−the wind pressure distribution on large areas considering theoretical and experimental correlated power spectral densities or time histories;

−rigid and aeroelastic response of large structures under the action of cross-correlated random wind action considering static, 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 instability;

−the non linear geometric and material behaviour;

−reliability and safety factors of new hi-tech composite materials;

−the necessity to avoid and short-circuit progressive collapse of the structural system due to local secondary structural element and detail accidental failure;

−the compatibility of internal and external restrains and detail design, with the modelling hypothesis and real structural system 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.

−In the case of movable structures, the knowledge base concerns mainly the moving cranes and the related conceptual design process have to consider existing observations, tests and specifications regarding the behaviour of similar structural systems. In order to fill the gap, the IASS working group n°16 prepared a state of the art report on retractable roof structures including recommendations for structural design based on observations of malfunction and failures [3].

From the observations of the in service performance, damages and collapses of all or part of structural systems, we have received many information and teachings regarding the design and verification under the action of ultimate and serviceability limit states.
Long span coverings were subjected to partial and global failures as that of the Hartford Colisseum (1978), the Pontiac Stadium (1982) and the Milan Sport Hall (1985) due to snow storms, the Montreal Olympic Stadium due to wind excitations of the membrane roof (1988) and snow accumulation (1995), the Minnesota Metrodome (1983) air supported structure that deflated under water ponding, the steel and glass shell sporthall in Halstenbeck (2002), the acquapark in Moscow and the air terminal in Paris (2004). Those cases are lessons to be

learned from the structural failure mechanism in order to identify the design and construction uncertainties in reliability assessment. Many novel projects of long span structures attempt to extend the "state of the art". New forms of construction and design techniques generate phenomenological uncertainties about any aspect of the possible behaviour of the structure under construction service and extreme conditions.

Fortunately, structures rarely fail in a serious manner, but when they do it is often due to causes not directly related to the predicted nominal loading or strength probability distributions. Other factors as human error, negligence, poor workmanship or neglected loadings are most often involved. Uncertainties related to the design process are also identified in structural modelling which represents the ratio between the actual and the foreseen model's response.

According to Pugsley (1973), the main factors which may affect "proneness to structural accidents" are [4]:

- new or unusual materials;
- new or unusual methods of construction;
- new or unusual types of structure;
- experience and organization of design and construction teams;
- research and development background;
- financial climate;
- industrial climate;
- political climate.

Cause	%
Inadequate appreciation of loading conditions or structural behaviour	43
Random variations in loading, structure, materials, workmanship, etc.	10

Table 2 Prime causes of failure. Adapted from Walker (1981)

All these factors fit very well in the field of long span structures involving oftenly something "unusual" and clearly have an influence affecting human interaction.

In Table 2, the prime cause of failure gives 43% probability (Walker, 1981) to inadequate appreciation of loading conditions or structural behaviour. Apart from ignorance and negligence, it is possible to observe that the underestimation of influence and insufficient knowledge are the most probable factors in observed failure cases (Matousek & Schneider, 1976).

Performance and serviceability limit states violation are also directly related to structural reliability. Expertise in structural detail design, which is normally considered as a micro task in conventional design, have an important role in special long span structures: reducing the model and physical uncertainties and avoiding chain failures of the structural system.

According to the author, knowledge and experience are the main human intervention factors to filter gross and statistical errors in the normal processes of design, documentation, construction and use of structures.

The reliability of the design process in the field of special structures must be checked in the following three principal phases: the conceptual design, analysis, and working design phases.

Due to the lack of space, only some design & analysis illustrations of wide span enclosures, where the author was directly involved, will be included in the present paper with the intention to transmit some experiences, that today may be part of the knowledge base, specially addressed to loading analysis and structural behaviours.

Long span structures needs special investigations concerning the actual live load distribution and intensity on large covering surfaces. Building codes normally are addressed only to small-

medium scale projects. The uncertainties relate to the random distribution of live loads on long span structures imply very careful loading analysis using special experimental analysis.
From the direct author's experience in designing large coverings, the most important experimental investigation regarding live load distribution concerns the snow drift and accumulation factors and the dynamic action of wind loading.

2. DESIGN ASSISTED BY EXPERIMENTAL ANALYSIS

2.1. Snow loading experimental analysis on scale models

Olympic Stadium in Montreal. During the design of the new roof for the Montreal Olympic Stadium (Figure 3) a special analysis of snow loading was made considering three roof geometries varying the sag of the roof from 10 m, 11.5 m and 13 m, in order to find a minimization of snow accumulation.
Snow loads depend on many cumulative factors such as, snowfall intensity, redistribution of snow by the wind (speed and direction), geometry of the building and all surroundings affecting wind flow patterns, absorption of rain in the snowpack, and depletion of snow due to melting and subsequent runoff.
The experimental investigation was carried out by RWDI [5] to provide design snow according to FAE (Finite Area Element) method, representing up to day a state of the art on the matter.
The shape of the roof with a sag of more than 12m. gives separation of the air flow and turbulence in the wake increasing considerably the possibility of snow accumulations. The order of magnitude of the leopardized accumulations in the roof are of 4-15 kN; local overdimensioning was necessary in order to avoid progressive collapse of the structural system.

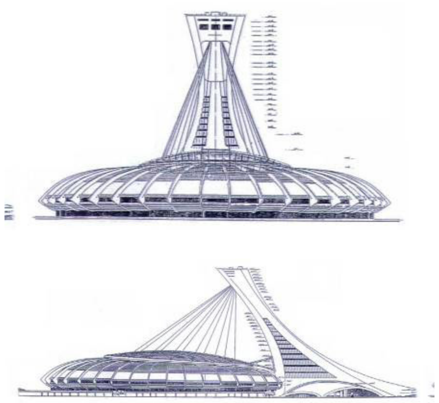

Figure 3. Montreal Olympic Stadium.
A cable stayed roof solution

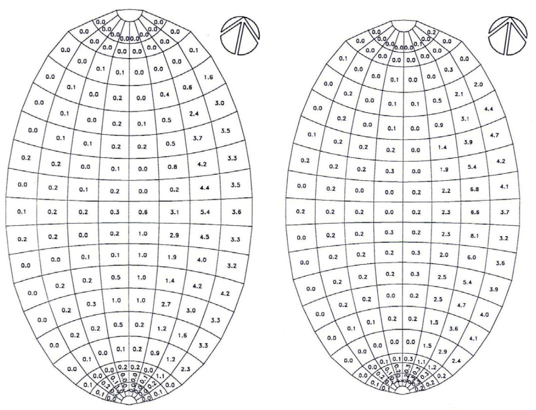

Figure 4. Comparative analysis of snow loading distribution in function of roof shape (10-13m)

2.2. Wind loading-experimental analysis on scale models: rigid structures-quasi static behaviour

The Cp factors: The Olympiakos Stadium in Athens

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The stadium is located near to the sea, as a consequence a "sea wind profile" with the parameters listed below and taken from literature and laboratory expertise, seems to be a good approximation of the wind profile in the area (Fig.6):

profile exponent	$\alpha = 0.15 \div 0.18$ (level ground, with few obstacles, sea),
roughness length	z0 = 5 ÷15 cm (cultivated fields),
integral length scale	LU = 50÷100 m.

Figure 5. 3D rendering of the Olympiakos
Stadium in Athens

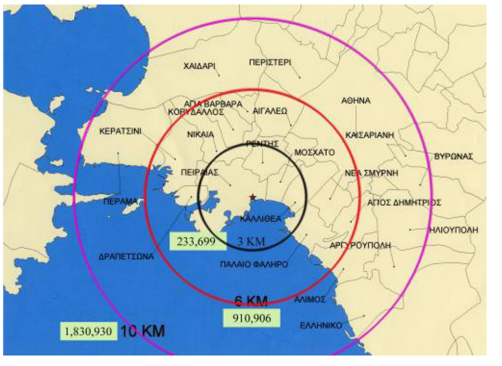


Figure 6. Geographic location of the stadium

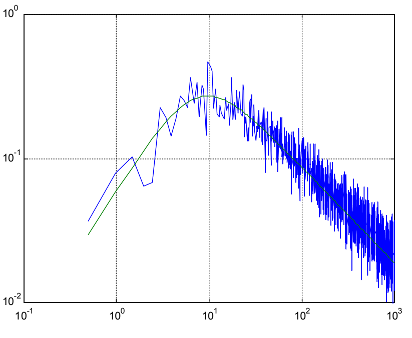


Figure 7. Spectral density of the longitudinal component of the wind velocity ("fitting" with Von Karmán spectral density).

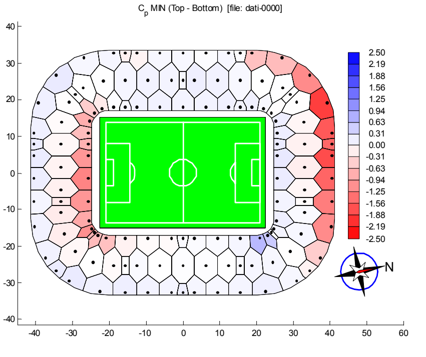


Figure 8. Maximum and minimum values of net pressure coefficients (wind direction: 0°).

The model has been made in a geometric scale of 1:250 and includes: the roofing, the stands, all the structures of the stadium, and other private and public buildings not far then 250 m (in full scale) (fig. 5) from the centre of the stadium. The geometric scale has been chosen in order to fulfil the similitude laws (fig. 7).
The roofing has been equipped with 252 pressure taps, of which 126 at the extrados and 126 at the intrados, in order to get the net pressures on the roofing. The location of the pressure taps has been chosen to cover the whole roofing surface according to the fig. 8, which shows also the influence area of each pressure tap.

The pressure measurements have been performed using piezoelectric transducers linked to the pressure taps through Teflon pipes.

Measurement and use of load time histories: *The Thessaloniki Olympic sport complex*

The integration of the wind tunnel data into the design process presents significant problems for wide span sub-horizontal enclosures; in contrast to buildings (high rise buildings) where knowledge of the base moment provides a sound basis for preliminary design, there is not single simple measure for the roof. The study of the Stadium of the Alpes and the Rome stadiums [6-7-8] drew attention to the inability of the measuring system employed to provide data in a form that could readily be based as input to the sophisticated dynamic numerical model developed by the designer and lead to discussion between the designer and the wind tunnel researchers to examine alternate techniques that might be used in future projects.The discussions centred on the use of high speed pressure scanning systems capable of producing essentially simultaneous pressure measurements at some 500 points at rates of perhaps 200 Hz per point. With such a system it would be possible to cover in excess of 200 panels and produce a complete description of the load. Such a system would produce roughly 1 to 2x106 observations for a single wind direction and it is clear that some compression of the data would be required. One possible approach would be to produce a set of load histories, Qj(t), such that:

$$Q_j(t) = \int_A p(x,y,t)\phi_j(x,y)dA \qquad (1)$$

where:

p(x,y,t) net load per unit area at position (x,y);

$\phi_j(x,y)$ weighting function.

For a series of pressure taps of the approximation to $\phi_j(t)$ would be:

$$Q_j(t) = \sum_{i=1}^{N} \overline{p}_i(\overline{x}_i, \overline{y}_i, t)A_i\phi_j(\overline{x}_i, \overline{y}_i) \qquad (2)$$

Ai area of ith panel;

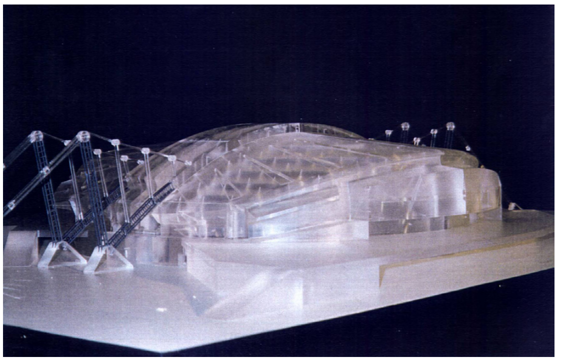
pi pneumatic average of pressure at the taps in the ith panel;

xi, yi geometric centre of the taps on the ith panel;

N number of panels.

In collaboration with the Boundary layer wind tunnel laboratory of the University of Western Ontario, a new very practical method to obtain the structural response under the random wind action and small displacements (linear response) has been applied under the name of the "orthogonal decomposition method" [7-9].

The experiment would involve the recording of the local histories $\phi_j(t)$ from which the model time histories could be constructed and the analysis conduced in either the time or frequency domain (fig. 9-10). For the type of structure under consideration resonant effects are small and the response is largely a quasi-static to a spatially varied load. The deflections induced are closely related to the imposed loads and their distribution differs significantly from the Gaussian form [6]. In such a case the time domain solution, which preserves the extreme value distribution, is to be preferred over a frequency domain approach.



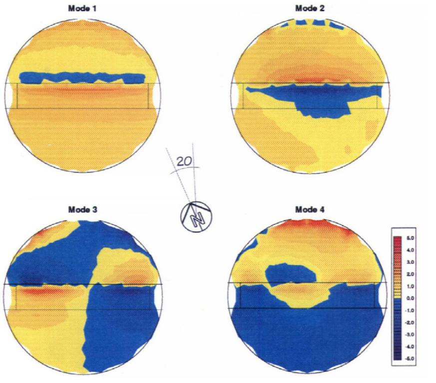


Figure 9. Views of pressure model of Thermis Sport Hall

Figure 10. Orthogonal decomposition: pressure mode shapes

2.3. Wind loading-experimental analysis on scale models : flexible structures-aerodynamic behaviour: The olympic stadium in Rome

The wind induced response of the cable supported stadium roof was analysed by a non linear model and a field of multicorrelated artificial generated wind loading time histories [7].Wind tunnel tests have been carried out at the BLWT Lab. of UWO on a model of 1:200 (fig. 11) scale determining:
- 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;
- presssure coefficients (maxima, minima and average) for every 10° of incoming direction;
- auto and cross-spectra of the fluctuating pressure (averaged on every single panel).

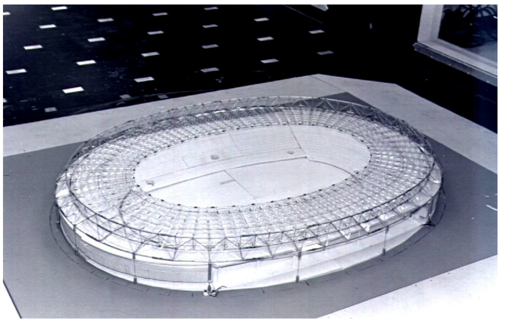


Figure 11. Aeroelastic model for Rome Olympic Stadium

Figure 12. Aeroelastic model for the Braga Stadium

The aerodynamic behaviour shows a clear shedding phenomenon. The external border of the structure, constituted of the trussed compression ring with triangular 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 frequency. This is confirmed by the resulting Power Spectra Density Function of the fluctuating pressures, which shows a peak at about 0.15Hz even if the values rapidly decrease with increasing distance (fig. 13).

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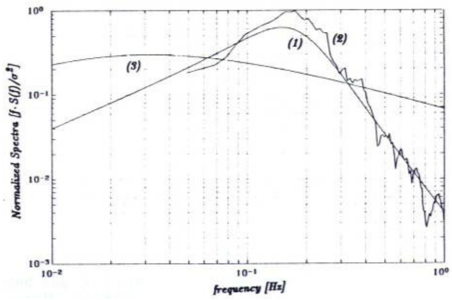

Figure 13. Target (1), simulated (2) and Kaimal's (3) normalized spectra of wind velocity

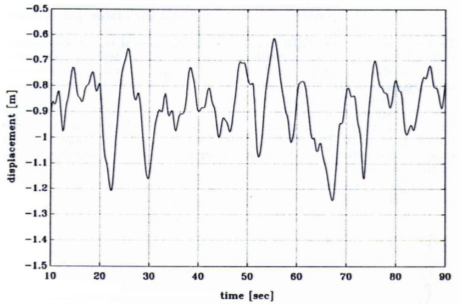

Figure 14. Time History of the displacement (leeward side at tension ring, run #2)

A fluid-interaction non linear analysis in time domain, made for the checking of La Plata stadium design [10] shows a better agreement between theoretical model and experimental values.

3. RELIABILITY ANALYSIS: THE SENSIBILITY ANALYSIS REGARDING THE NEW SUSPENDED CABLE ROOF OF BRAGA (PORTUGAL)

3.1. Reliability analysis of the roof structural system. Cable strain parametric sensibility.

Considering that in the basic solution the roof will be covered by a long span structural system with only uplift gravitational stabilization (fig. 17) it is essential to proceed to the analysis of the response of the structural system to loading patterns and wind induced oscillations.
The analytical process will be organized in order to be controlled by experimental investigations in reduced and full scale.
The reduced scale experimental analysis on rigid and aeroelastic models are concerned with the determination of the dynamic loading on the roof surface and of the stability of the structural system.
The full scale experimental investigations are addressed to check, by a monitoring program, the validity of the global analysis process.
The uncertainties on the elastic modulus of the cable, geometrical and elastic long term creeping, tolerances of fabrication and erection, differences with design prestress, non uniform distribution of temperature, non linear behaviour, created a sensitive response on the suspended roof hanging from a set of suspended cables. The sensibility analysis showed that the response is sensitive to the standard deviation of the cable strain ($\Delta\varepsilon$) variations. The failure probability is given by the probability that an outcome of the random variables ($\Delta\varepsilon$) belongs to the failure domain D. This probability is expressed by the following integral [11]:

$$P_f = \int_{D_f} f_{\Delta\varepsilon}(\Delta\varepsilon)\cdot d\Delta\varepsilon \qquad (3)$$

and the most probable failure mechanism will involve primarily the border cables.
The sensibility analysis was, therefore, extremely important to detect the weak points of the structural system and permits proper local dimensioning to prevent chain failure, as illustrated with the failure simulation of same sensitive cable elements.
The roof is composed by a structural concrete plate sustained by n prestress cables. In the analysis the roof, the bending moments at m points will be considered. For a particular load

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combination, the n cables have computed strains given by the vector ε. Considering that these effects are represented by the vector of random variables Δε with mean values μ and standard variations σ, the problem is to estimate the probability, Pf, that the generated random bending moments M will be larger than the plate ultimate resistance moments, Mu, at any of the m points of the structural plates system.

Figure 15.The new suspended cable roof of Braga Stadium (Portugal)

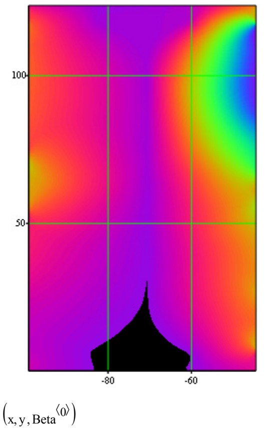

Figure 16. β-Safety Index distribution, evidencing SLU sensibility on black region (β=3.798)

3.2. Results and conclusions

All the load cases were analysed and the following preliminary conclusions are described as follows.
In order to identify the most dangerous load case the minimum reliability index β for each load cases were calculated for a standard deviation σ =0.5 x 10-3 for Δε of all cables. The following table (Table 3) summarizes the index β (computed with σ =0.5 x 10-3).
The load cases 7, 9 and 10 have the lowers β, i.e., the higher failure probability, and therefore they are the critical load condition. Particularly critical is the load case 7.

Load Case	Beta	Phi(-Beta)
1	5.8739	2.14E-09
2	5.7957	3.42E-09
3	5.9555	1.31E-09
4	5.5733	1.26E-08
5	4.1218	1.87E-05
6	4.8436	6.41E-07
7	1.6658	4.79E-02
8	5.7281	5.11E-09
9	5.5396	1.53E-08
10	2.6269	4.31E-03
11	2.3812	8.63E-03
12	4.3046	8.37E-06
13	4.3045	8.37E-06
14	5.8201	2.96E-09
15	5.7479	4.55E-09
16	5.8415	2.61E-09

Table 3. Reliability index β

3.3. Failure probability and sensibility analysis

The figure 17 shows the failure probability for load combination 7 as a function of the standard deviation, σ, of the cable strain variations, Δε.

a. The problem is extremely sensitive to the standard deviation, σ, of the cable strain variations, Δε. For example for load case 7, if σ is increased from 2x10-4 to 3x10-4, Pf is increased from 2x10-5 to 480x10-5.
b. Cable standard deviation, σ, should be maintained below 2x10-4 for the designed ultimate bending moment.

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c. Larger cable standard deviation, σ, could be allowed increased the slab reinforcement along x-direction in the critical roof zone.
The figure 16 shows the most probable values of Δε (x10-3) in each cable at failure for load combination 7.

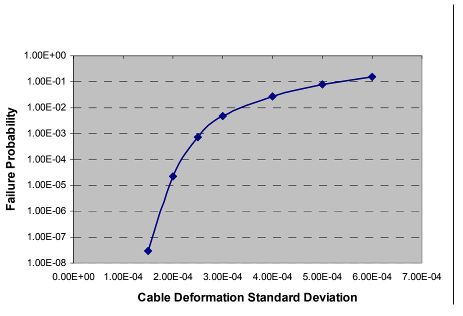

Figure 17. Failure probability in function of cable deformation standard deviation

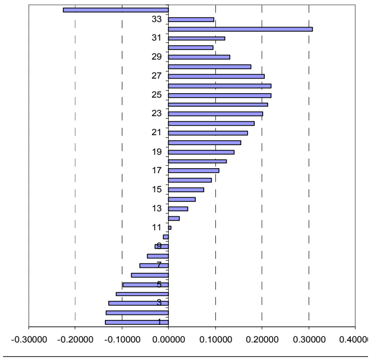

Figure 18. Most probable Δε in each cable at failure for load comb. 7

4. CONCLUSIONS

It has been noted the influence of knowledge base on conceptual design in removing gross human intervention errors from initial design statements.
Design assisted by experimental investigation is a useful integration of the design process of wide span structures.
Sensibility analysis is an extremely powerful tool to determine the influence of parametric design uncertainties for unusual long span structural systems.
In the last full pages figures some designs and realizations, where the writer was involved as structural designer ,are illustrated.

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