EXPERIENCES IN DESIGN AND REALIZATION OF WIDE SPAN STRUCTURES

M. Majowiecki
IUAV – Dipartimento di Costruzioni dell'Architettura. University of Venice
Massimo.majowiecki@majowiecki.com

In actual realizations, today we see free formal expressiveness originating architectural objects such as leaning towers, sculptured bridges, free-form enclosures and the like, whose shape sometimes has no connection whatsoever with structural principles. In line with the technical and scientific philosophy expressed by Torroja, Nervi and others, they are no more than structural forgeries. On the contrary, many of these new architectural objects marvelled us and are appreciated in the name of the very definition of the word architecture, as an intellectual and technical exercise directed at adapting our physical environment to the needs of social life. It cannot be denied that some works achieve the level of architectural and sculptural art and the role played by structures is merely to support architectural design.

Conversely, these new architectural realities essentially based on individual artistic capabilities can be didactically deviant. A structural forgery may induce students and professionals to elaborate design imitations, with the introduction of dangerous unbalanced structural systems and/or morphological sculptured shapes making any prismatic configuration building look outdated.

1.Structural Architecture

The disciplinary correlation between architecture and structures, conceived as an integrated design language, may be stated as non-existent or false in many modern constructional realities affected by new painting and sculpture, scenic and cartoonist design variables. Modern examples of structural architecture are no longer correlated in disciplinary terms as in the past. Even though Spinoza states that ethics change in time because substances the intellect perceives obviously change, the introduction of architectural and structural ethical issues, according to the principle of technological ethical responsibility introduced by Hans Jonas[1], could prevent some technological and structural stereotypes, such as London's Millennium Bridge where structural stability was sacrificed to generating technological astonishment for instance, as well as false conceptual design statements, didactically deviating, such the Seville Amarillo Bridge, where successful design as a landmark was associated with the hypothesis that the bridge inclined tower weight was enough to counterbalance the bridge deck with stays, while most of the material used for the bridge function was, in actual fact, structurally useless but addressed to obtain a sculpture. Ethics may help to obtain a more reliable information from design actors and realizations process and, consequently, prevent, at least, design imitations based on false statements.

Ethics must also not be considered as a limit to creativity in searching for a design idea .

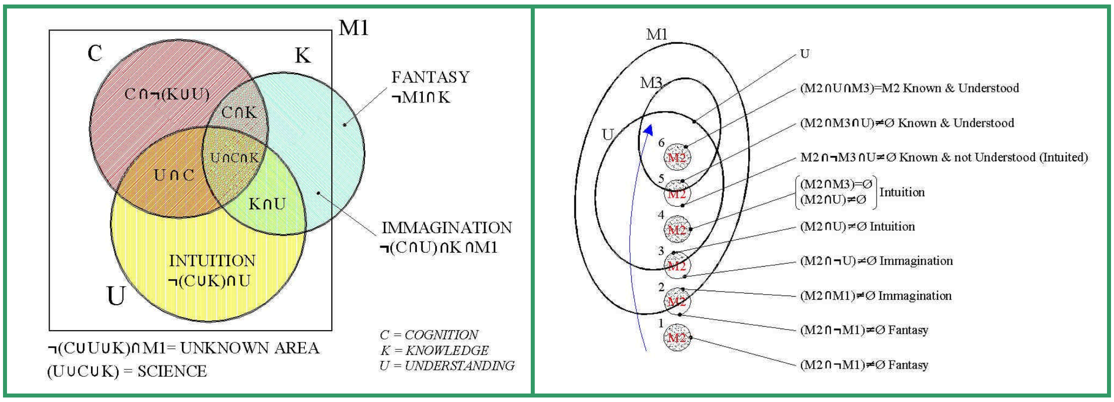


Figure 1. Creativity – extending the Knowledge

In particular, according to Bignoli[2], the power of human mind as knowledge, understanding, wisdom, fantasy, imagination and intuition allow a phenomenological uncertainty level, where to extend creativity matches up with creating a new state of the art *(fig.1)*.

Going against the technical and scientific philosophy taken from Eiffel, Torroja, Nervi and others, who designed by looking first and foremost at the construction, quite sure that observing the laws of static engineering would be seen, per se, as a guarantee of aesthetic results achieved, many contemporaries observe the laws dictated by new design trends as:

- the prevalence of aesthetics over static rationality;
- stringent search for structural efficiency to solve a more complex issue than reality, in order to achieve an original solution;
- the categorical rhetoric of structural actions that translate into design languages;
- the structure as a sculpture;
- mechanistic impressionism;
- the metaphorical transposition, into architecture, of Nature and other foreign elements;
- the rhytmic and monotonous repetition of an architectural motif;
- the emphatic representation of a typical element's details, to identify the overall scale;
- the introduction of auxiliary IT resources.

Some design errors originating from the lack of interaction between architecture and structural engineering under the new design trends and circumstances, or non-compliance with ethical standards according to the principle of responsibility, have been in the past and still are today the cause of serious unsuccessful design ensuing legal proceedings as well as structural malfunctioning and even collapse. Considering that modern designing is a complex, holistic, trans-multi and inter-disciplinary process, that must achieve a required reliability level observing general hypotheses and feasibility constraints, Structural Architecture (SA) presents as a methodology, a reflective knowledge, productive of proper design approaches, within the framework of technological civilisation responsibility ethics.

2 Interaction between Ethics and Structural Architecture

Let's consider the classical ethics of contemporaneity, the ethics of sustainability (called by Hans Jonas ethics of responsibility) and the consensus ethics, based on statistics, which in fact is a lack of Ethics, the non ethics.

Classical engineering is a possession (P) and a state (S), State means "state of art S in time $t_0 = S(t_0)$.

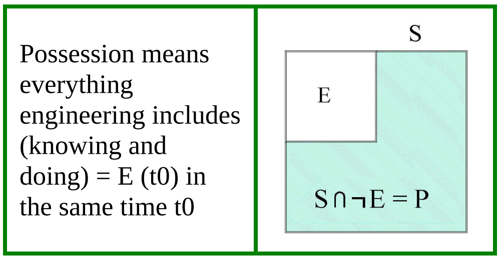


Figure 2. The object is S = E. To be up to date with engineering (almost constant in time) Progress:
$P = S \cap \neg E$ until $S \cap \neg E$ becomes $\varnothing$.

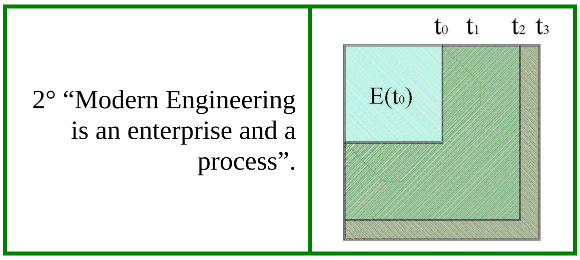


Figure 3. The enterprise is to carry out the process:
$E(t0) \subseteq E(t1) \subseteq E(t2)$…The object is the continuous progress (inevitable according to Jonas):
$Pi = Ei \cap \neg Ei - 1 \neq \varnothing$ when i = 1; 2; 3..

SA interacts with ethics by the scientific+technological contents of the components : structural engineering + architecture *(fig.4)*.

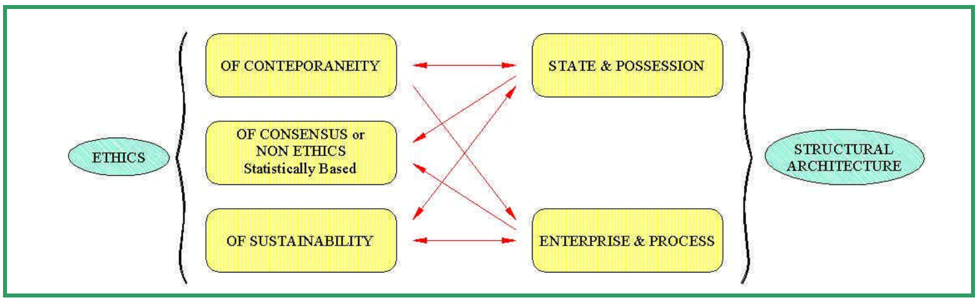


Figure 4. Interactions between SA and ethics

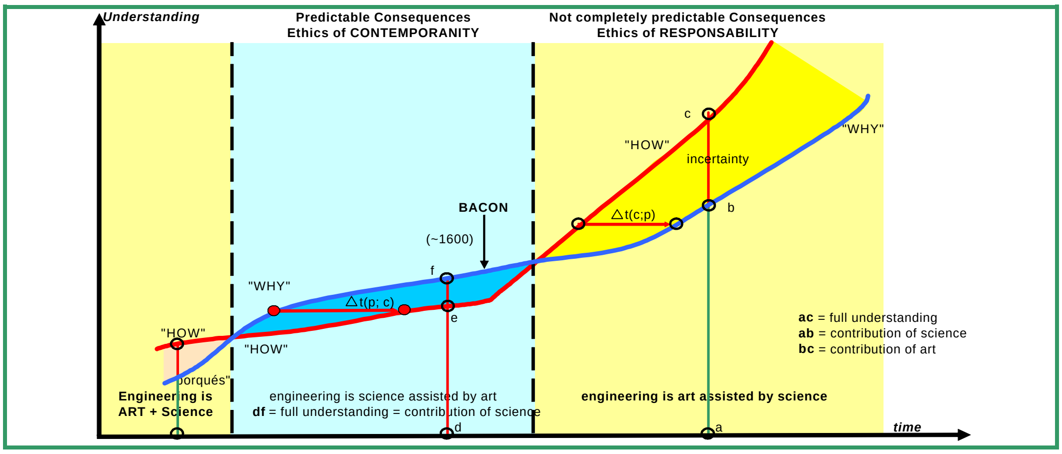


Figure 5. Growth of know how and know why in time

The History of Technique refers to a lot of Hows resulting from Whys, known just as Bacon wanted, but in many occasions, the Hows anticipated the Whys. There have also been some What's like Watt's steam engine, that worked for less than a century with a known How (the invention) and ignored Why, (revealed only when Sadi Carnot stated the 2° Principle of Thermodynamics).

Sustainability Ethics establishes progressively narrower limits in time, to the Know How exceeding the Know Why. The Know How grows, in function of time, quicker than the Know Why, with the consequent increase of uncertainty *(fig.5)*; it follows that it is not possible to foresee the long-term consequences of an act, which you know how to accomplish but not why it occurs and its consequences.

3. Realizations: What-Why-How or What-How-Why ?

David I. Blockley said "To do you must know, and to know you must do". Translating it to the language of this paper, we would say: To accomplish the Hows you must know the Whys and to know the hows, you must have accomplished many Hows (expertise). We have seen that HOWS may come earlier than Whys, in fact, it has happened. Therefore the previous expression is just a wishful one or an ideal situation, difficult to fulfil in reality, impossible for just one man but possible for further generations of caring diligent engineers.

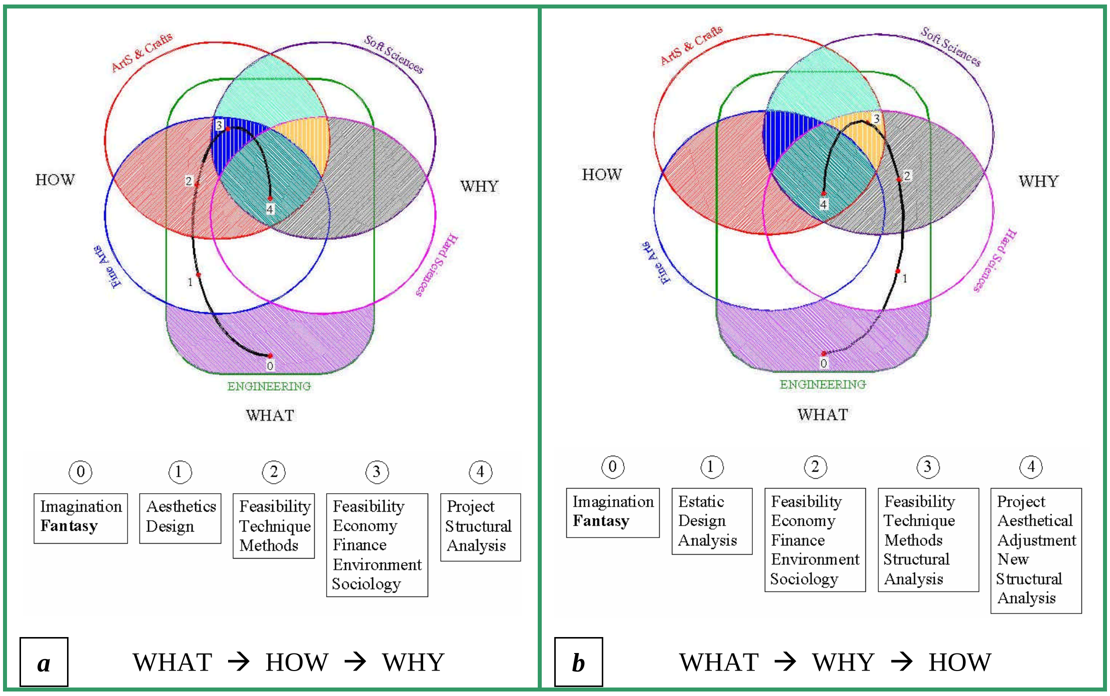


Figure 6.***a***_SA is an Art (technique) assisted by Science;
6.***b***_SA is a Science assisted by Art (technique).

Among the two ways to reach the ”perfect engineering” Qn Cn P (point 4) to reach simultaneously **the know how and the know why** is a privilege of wise experimented architects&engineers *(fig.6)*.

If it is also considered that present progressive engineering is an enterprise that accomplishes a process and this process means that “possession” and “state” grow in function of time, then this process might change the ethics of sustainability, making it increasingly complex in the event of the state growing larger than possession, which is also possible. However, it is easy to predict that the growth of possession P (t) implies the possibility of changes that would generate new acts and that Sustainability Ethics could consider ethically unacceptable *(fig.7)*. This wouldn’t be so if changes (the growth of the Know How) implied progress; in other words, if changes were always beneficial as they should be. Thus: Known objectives, that have been always pursued, may find a better satisfaction through new techniques brought by those objectives. But things also happens the other way round, more typically every day, that new techniques may inspire, produce or even force new objectives never thought of before, only because they are possible now.

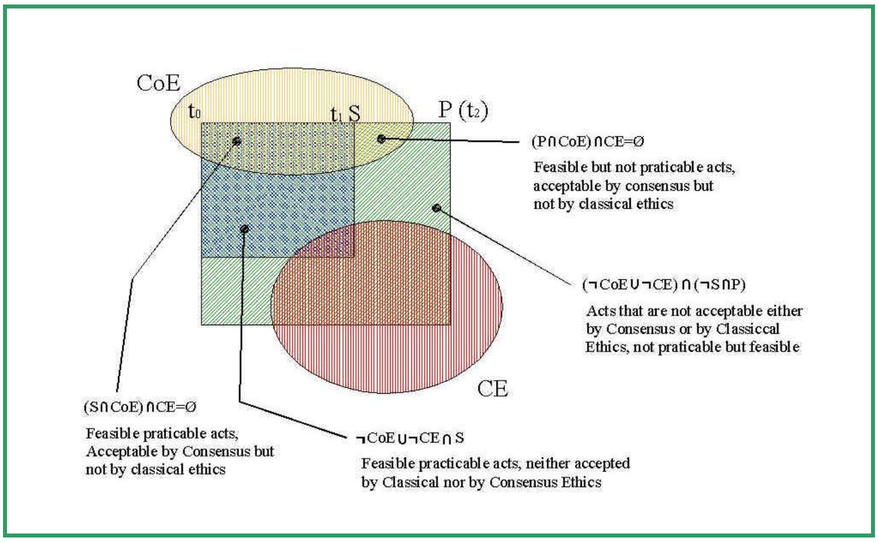


Figure 7.Realization & Ethic acceptability

4. Widespan roof structures: a contribution to knowledge

Wide span structures 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 synthetical 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.

Considering the statistical results of [3] , the unusual typologies, new materials and the “scale effect” of long span structures, several special design aspects arise as:

- 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 modeling 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 [4] including recommendations for structural design based on observations of malfunction and failures.

From the observations of the in service performance, damages and collapses of all or part of structural systems, we have received many informations 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, etc. 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 behavior 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:

- 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 1 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 1, 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 structual 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.

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.
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.

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.

4.1 Design assisted by experimental analysis

4.1.1 Snow loading experimental analysis on scale models

Olympic Stadium in Montreal. During the design of the new roof for the Montreal Olympic Stadium (Figure 8) 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.

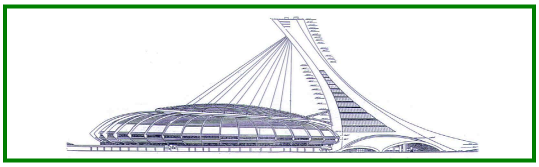


Figure 8.Montreal Olympic Stadium. A cable stayed roof solution

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.

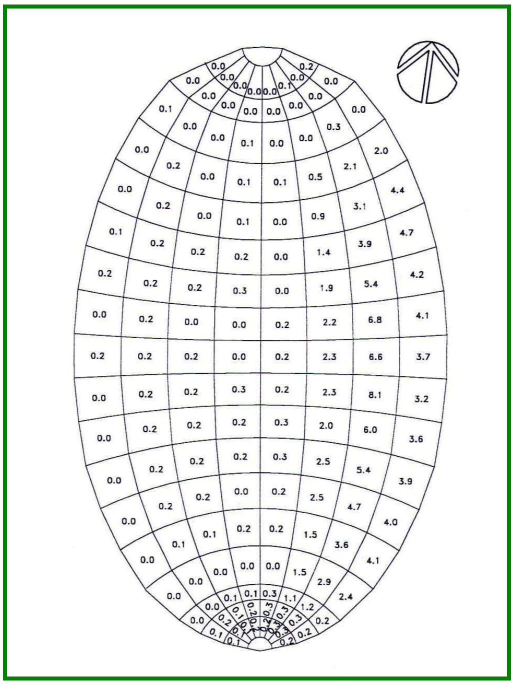


Figure 9. Comparative analysis of snow loading distribution in function of roof shape (10-13m)

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.

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

4.1.2.1 The Cp factors: ***The Olympiakos Stadium in Athens***

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. 5):

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

Figure 10. 3D rendering of the Olympiakos Stadium in Athens

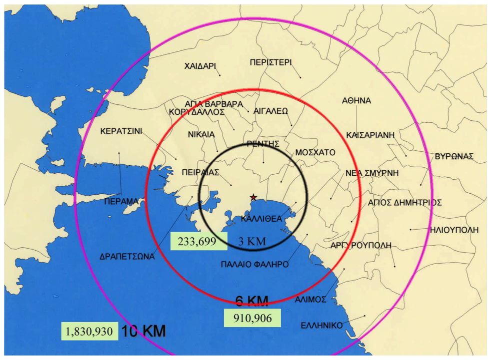


Figure 11. Geographic location of the stadium

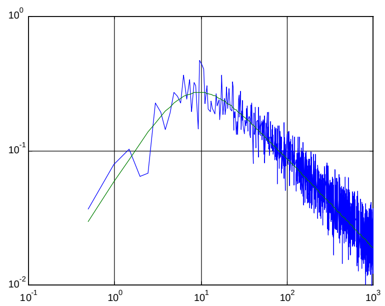


Figure 12. Spectral density of the longitudinal component of the wind velocity (“fitting” with Von Karmán spectral density)

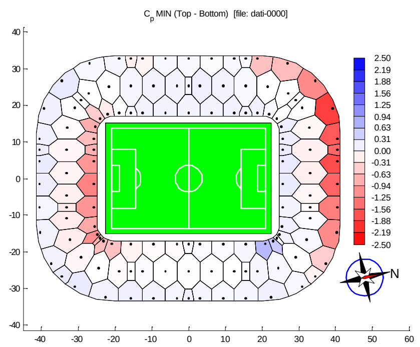


Figure 13. Maximum and minimum values of net pressure coefficients (wind direction: 0°).

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. 10)* from the centre of the stadium. The geometric scale has been chosen in order to fulfil the similitude laws *(fig. 12)*.

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. 13*, 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.

4.1.2.2 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-horizzontal 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 [9].

The discussions centered 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:

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

where:

p(x,y,t)	nett load per unit area at position (x,y);
$f_j(x,y)$	weighting function.

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

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

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" [10].

The experiment would involve the recording of the local histories $f_j(t)$ from which the model time histories could be constructed and the analysis conducted in either the time or frequency domain *(fig. 14-15)*. 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 [9]. In such a case the time domain solution, which preserves the extreme value distribution, is to be preferred over a frequency domain approach.

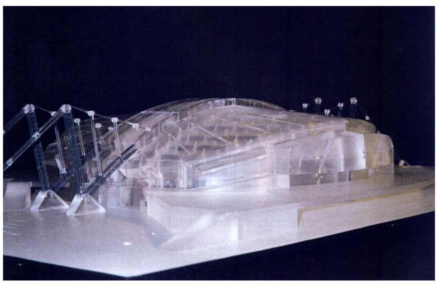


Figure 14. Views of pressure model of Thermis Sport Hall

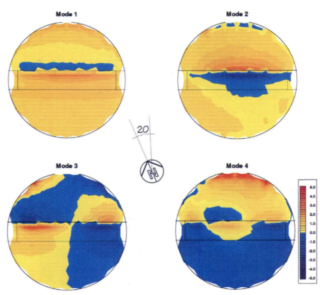


Figure 15. Orthogonal decomposition: pressure mode shapes

4.1.3 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 [8].Wind tunnel tests have been carried out at the BLWT Lab. of UWO on a model of 1:200 *(fig. 16)* scale determining:

- time histories of the local pressures for every 10° of incoming flow direction;the maximun,minimun 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).

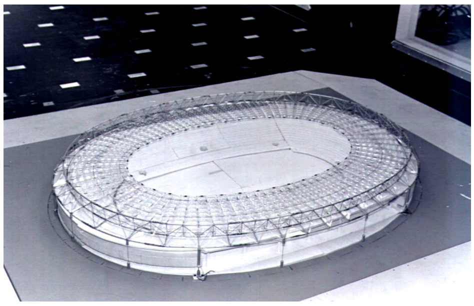


Figure 16. Aeroelastic model for Rome Olympic Stadium

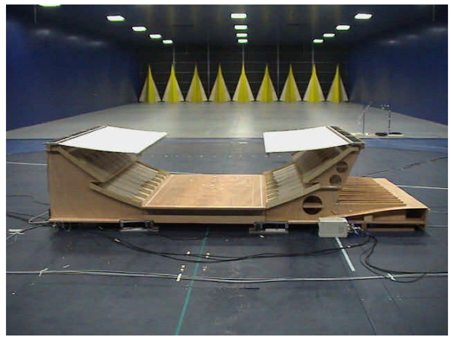


Figure 17. Aeroelastic model for the Braga Stadium

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. 18)*.

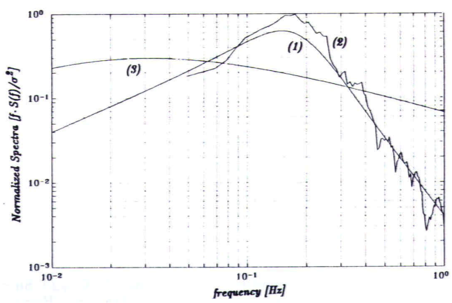


Figure 18. Target (1), simulated (2) and Kaimal's (3) normalized spectra of wind velocity

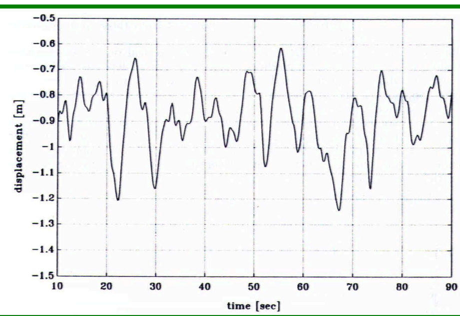


Figure 19. Time History of the displacement (leeward side at tension ring, run #2)

A fluid-interaction non linear analysis in time domain, made for the checking of La Plata stadium design [11] shows a better agreement between theoretical model and experimental values.

4.2 RELIABILITY ANALYSIS: *the sensibility analysis regarding the new suspended cable roof of Braga (Portugal)*

4.2.1 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. 20)* 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 (Δε) variations. The failure probability is given by the probability that an outcome of the random variables (Δε) belongs to the failure domain D. This probability is expressed by the following integral [12]:

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

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 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.

4.2.2 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 s =0.5 x 10-3 for Δε of all cables. The following table (Table 2) summarizes the index β (computed with s =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 2. Reliability index β

4.2.3 Failure probability and sensibility analysis

The figure 20 shows the failure probability for load combination 7 as a function of the standard deviation, s, of the cable strain variations, Δε.

a. The problem is extremely sensitive to the standard deviation, s, of the cable strain variations, ? e. For example for load case 7, if s is increased from 2x10-4 to 3x10-4, Pf is increased from 2x10-5 to 480x10-5.
b. Cable standard deviation, s, should be maintained below 2x10-4 for the designed ultimate bending moment.
c. Larger cable standard deviation, s, could be allowed increased the slab reinforcement along x-direction in the critical roof zone.

The figure 21 shows the most probable values of ? e (x10-3) in each cable at failure for load combination 7.

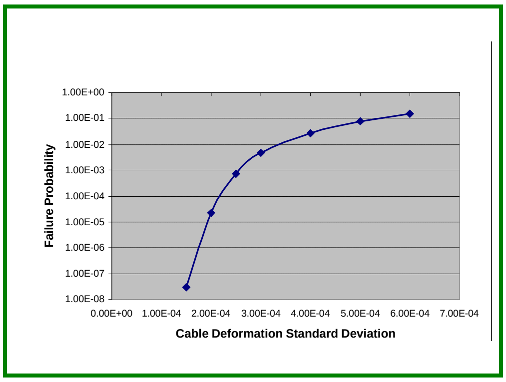


Figure 20. Failure probability in function of cable deformation standard deviation

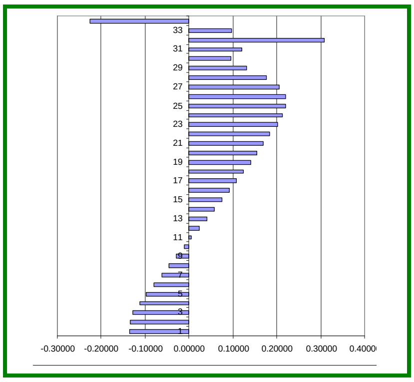


Figure 21. Most probable Δε meach cable at failure for load comb. 7

4.3 MONITORING

The roof structures of the Torino Stadium have been built in 1990 *(fig.22-23)*. According to the quality control and maintenance program, the in service subsequent performance of the structure has been controlled by site inspection, experimental measurement and spot monitoring of representative structural parameters. The anchorage forces in the cable stays have been controlled during 1992 and considerable differences in average and peak values of pre-stressing have been observed between experimental and expected theoretical values. The authors proceeded, with a computer simulation of the actual observed anchorage force values, to determine a new pre-stressing sequence in order to fit the original design cable force. Actually the structure is under normal monitoring observation according to a special maintenance program.

Figure 22. Stadium of the Alpes – Aereal view….

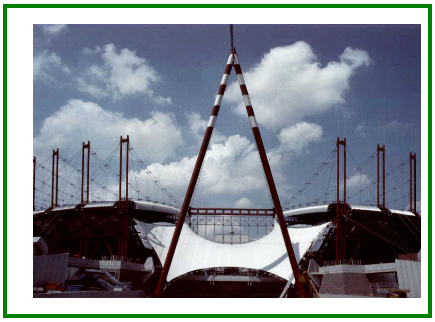


Figure 23. Stadium of the Alpes – North view

4.3.1 Displacement and force control.

The first tensional controlling operations were performed on the cable anchorage forces in 08.05.1992. The geometrical control of the central ring was performed during and after the completion of the operations, dated 13.05.1992. On the 19.05.1992 the designers were requested to conceive an intervention plan to eliminate the detected errors related to the intensity and distribution of the state of initial stress.

Therefore, on the 3.08.1992, it was presented a proposal related to a theoretical simulation of the experimentally observed stressing state and to the definition of a correct new pre-stressing. The comparative analysis between the results, referred to the actual service state anchorage forces (whose permanent loads were updated according to the final consumptive analysis) and the experimental data, revealed average and peak values' differences bigger than the allowable ones. The average prestressing level is reduced of 30%, with peak differences of 40-50% *(fig. 24)*.

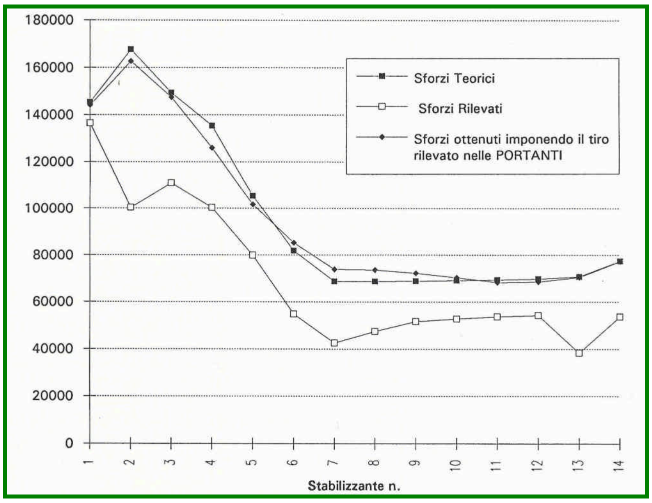


Figure 24. Stabilizing cable. Anchorage forces. Computed actual values. Measured values. Restored values.

4.3.2 Measurements and monitoring.

A special monitoring program was adopted during transitory time in order to control the displacements of some points of the roof.

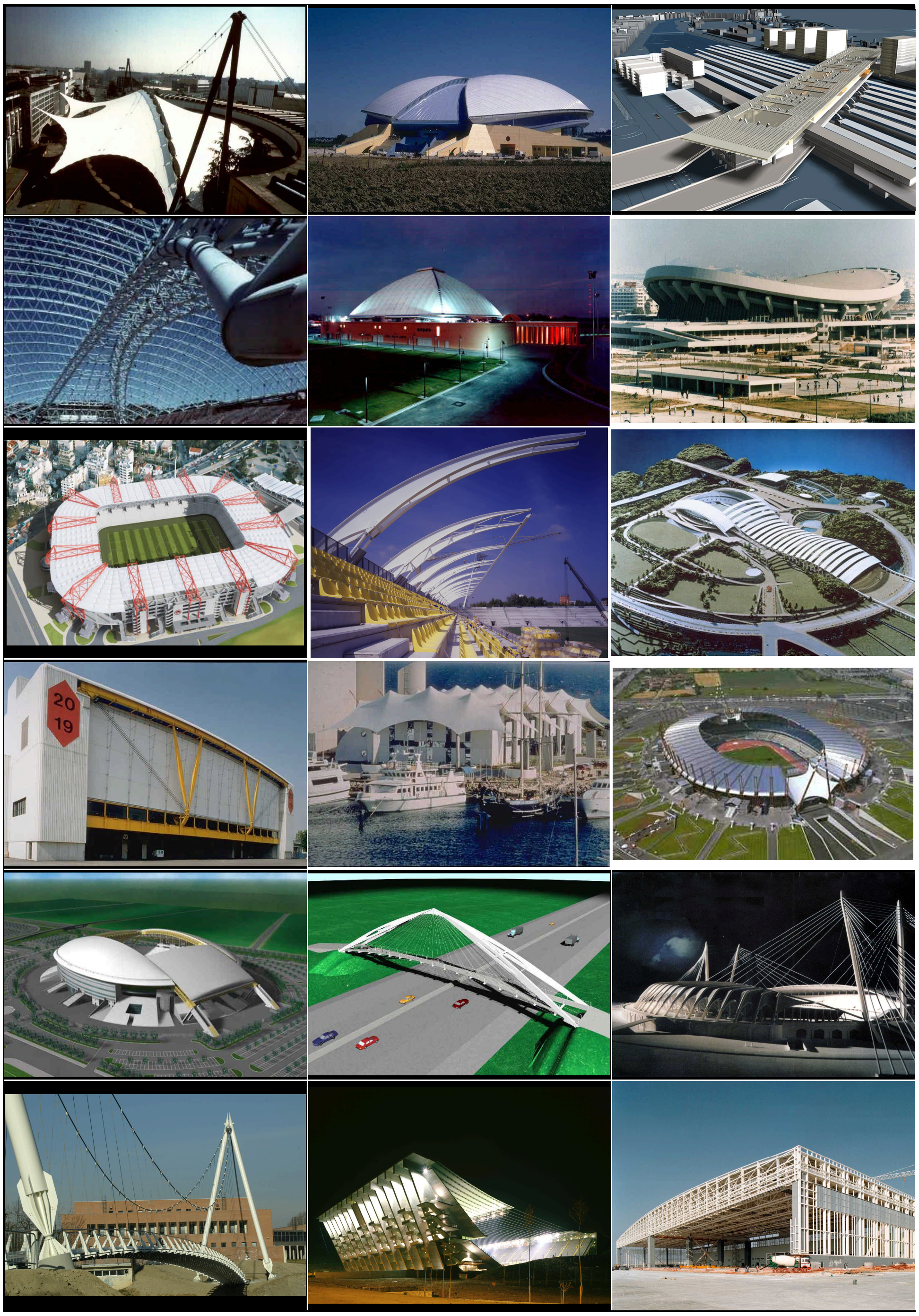
Before and after the pre-stressing procedure a dynamic load was applied to the structure and, with the use of a special accelerometer giving efficient possibility to integrate accelerations in range of low frequencies (< 10-1 Hz) , power spectral densities of response could be plotted. With the frequencies and associated mode shapes it was possible to observe the increase of geometrical stiffness of the structural system by the new pre-stressing level. Accelerations and displacements, before and after prestressing, are shown in Table 3.

	Acceleration m/sec2	
	Before	After
A3	0.93 - 1.19	0.86 - 0.96
A4	0.50 - 1.00	0.32 - 0.44
	Displacements mm	
	Before	After
A3	7.8 - 10.0	6.8 - 7.5
A4	6.8 - 9.2	5.2 - 7.1

Table 3. Accelerations and displacements under dynamic load before and after pre-stressing.

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.



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