NEW MATERIALS AND THEORETICAL & EXPERIMENTAL TECHNOLOGIES IN THE FIELD OF LIGHTWEIGHT STRUCTURES

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ABSTRACT
Lightweight 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. Some special remarks concerning the influence on the reliability level of detail design, are given at the end of the paper.

INTRODUCTION

Lightweight structures are today widely applied for:

Sport buildings
- Stadia
- Sport halls
- Olympic swimming pools
- Ice tracks and skating rinks
- Indoor athletics

Social buildings
- Fair pavillions
- Congress halls
- Auditorium and theatres
- Open air activities

Industrial buildings
- Hangars
- Warehouses
- Airport terminals

Ecology buildings
- Waste material storage
- Pollution isolation

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

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

Cable structures
- cable stayed roofs
- suspended roofs
- cable trusses
- singleand multilayer nets

Membrane structures
- prestressed anticlastic membranes
- pneumatic membranes

Hybrid structures
- tensegrity systems
- beam-cable systems

Convertible roofs
- overlapping sliding system
- pivoted system
- folding system

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.

In Table 1, is possible to observe the exceptionally high strength to weight ratio ($K=\sigma/\gamma$) in tension (Kt) of hi-tech composite materials, important components of lightweight structural concept.

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 with 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 ($\delta^2\pi$). 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).

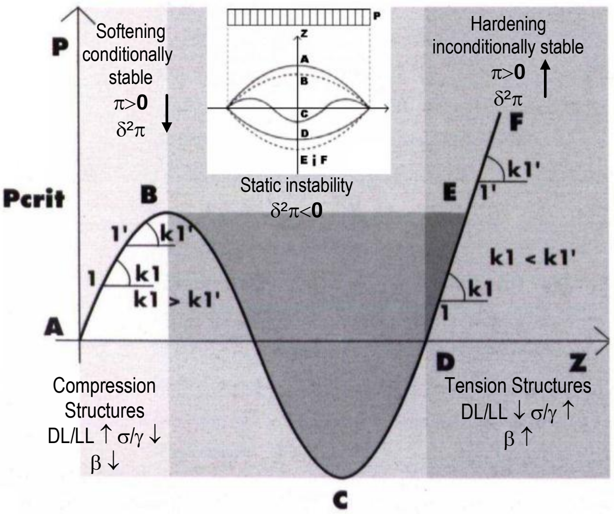


Figure 1 Mechanical behaviour from arch to cable.

MATERIALS	σ_t^R N/mm²	σ_c^R N/mm²	γ_k N/m³ 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
Steel 52	520		79.5	6.664	----
Steel 105	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.

Special aspects of conceptual design decisions on long span structures.

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. Limit state violation for engineered structures have lead to spectacular collapses as the Tay (1879) and Tacoma bridges (1940). Sometimes an apparently "unimaginable" phenomenon occurs to cause structural failure. The Tacoma Narrows Bridge previously cited was apparently one such a case. It was also a design which departed considerably from earlier suspension bridge design.

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 under snow accumulation (1999), 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 [1]. 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.

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 oftenly 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.

Cause	%
Inadequate appreciation of loading conditions or structural behaviour	43
Mistakes in drawings or calculations	7
Inadequate information in contract documents or instructions	4
Contravention of requirements in contract documents or instructions	9
Inadequate execution of erection procedure	13
Unforeseeable misuse, abuse and/or sabotage, catastrophe, deteriora tion (partly "unimaginable"?)	7
Random variations in loading, structure, materials, workmanship, etc.	10
Others	7

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

Considering the statistical results of table 2, 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 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 [2] including recommendations for structural design based on observations of malfunction and failures.

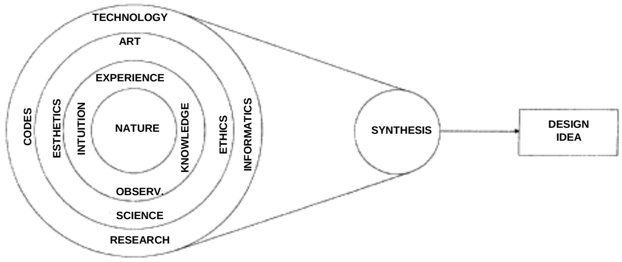


Figure 2 Holistic approach to structural design.

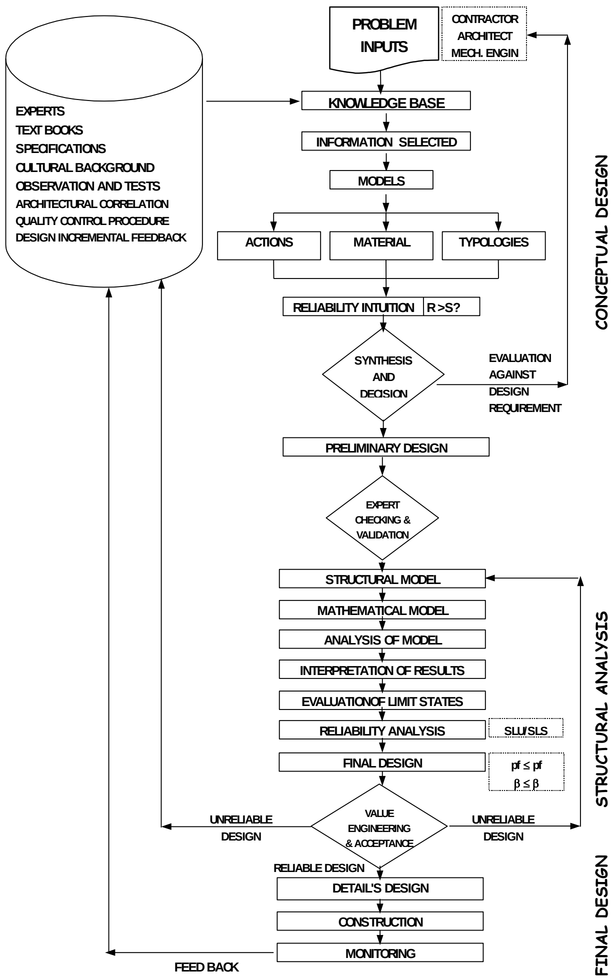


Figure 3 Conceptual design and analysis of structures.

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 knowledged expert synthetical approach [3] 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 (see Fig.2 Srivastava [5]) and directly depends on the skills and abilities of the design team members.

This concept is now included in some national building codes, which are normally addressed only to conventional structural systems.

As far as innovative designs are concerned, as in the case of most of the realized long span structures, only few comments are dedicated as, for instance, in the National Building Code of Canada (1990), point A-4.2.4.1: "It is important that innovative designs be carried out by a person especially qualified in the specific method applied...".

Eurocode no. 1 intends to guarantee the level of safety and performance by a quality assurance (QA) strategy (point 2) and control procedures of the design process (point 8) in order to minimize human errors.

Formalized methods of QA considers the need to achieve, by the institution of a "safety plan" the requirements of structural safety, serviceability and durability. A real danger is that excessive formalization of QA , born for tangible manufactured articles and not suitable for intangible conceptual control procedures, could lead to unacceptable and self-defeating degeneration of the design process, in a certain kind of Kafkian bureaucratic engineering and management.Notice about this phenomena is given by Carper (1996)in (Construction Pathology in the United States) [4]: “many repetitive problems and accidents occur, not from a lack of technical information, but due to procedural errors and failure to communicate and use available information”. An important contribution concerning the matter was given by the International Symposium on “Conceptual design of Structures” organized by IASS [5].

Furthermore, it would be necessary to have adequate and systematic feedback on the response of the design by monitoring the subsequent performance of such structures so that the long term sufficiency of the design can be evaluated.

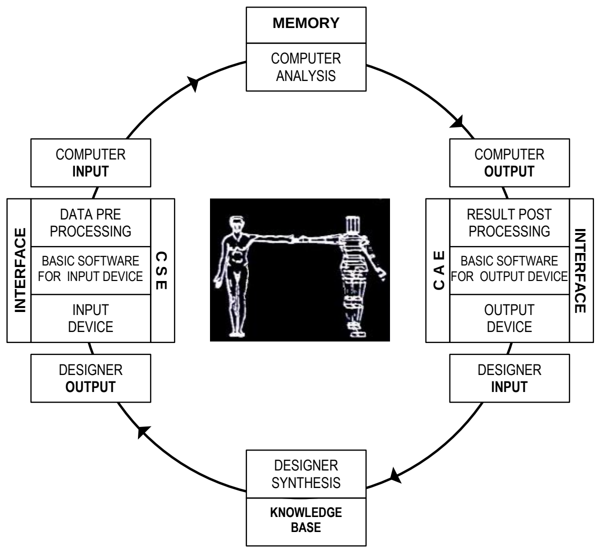
To assure a required reliability level, in the field of special structures, the design process must be checked in the following three principal phases: the conceptual design, the analytical model, and the working design phases as shown in Fig.3.

COMPUTER AIDED DESIGN & ANALYSIS

Conceptual errors are very hard to remove in the subsequent phase of structural analysis. In this phase the human intervention strategies as education, work environment, complexity reduction, self-checking and external checking and inspections are today assisted by new interactive computer aided design and analysis techniques. Specially the interactive graphic language will be very effective in obviating the effects of gross human errors during the structural modelling.

Hardware and software interfaces make it possible to generate an useful interactive design & analysis cycle (Fig. 4). The computer aided design methodology semplifies complex tasks and increases the reliability level, if the ergonomy and the logical flow of the interactive computer assistance is organized as follows:

- the interactive design methodology is not substitutive but rather integrates the creative aspects of the traditional design process (conceptual design).
- By means of an interactive graphic language (pre-processing and post-processing software), the electronic computer becomes the useful mental and operating extension of the designer, while considerably increasing his capacity, speed and decision-making abilities;



- design optimization is a logical consequence of the interactive methodology, allowing very fast data modification and evaluation of consequences (what if) according to the classical step by step iterative procedure of trial and error (and trial and success) based on the experience of the designer, who is able to synthesize a considerable mass of data that is difficult to exppress as a mathematical problem.

Figure 4 Interactive design process. C.S.E.: Control of Synthetical Elaboration. C.A.E.: Control of Analytical Elaboration.

At present time it seems to be very difficult to introduce aspects of artificial intelligence by a semantic software language inside the process of design of lightweight structures (expert system 3rd level software).
The structural design, analysis & monitoring phases are today functionally linked and logically integrated, with other design components (architectural, mechanical, project management, etc), with a common topological and geometrical 3-D identification model, trought an hardware and software network (Figure 5).
<u>Interactive graphic language addressed to the structural design of lightweight structures</u>.: from architectural to mathematical modelling
The development of the lightweight structural concept is historically correlated with the research in CAD technology. From the the initial empiric research made by Frei Otto, the theoretical and experimental investigation in the world concerning cable and membrane structures started in early 70s. In the Department of Structural Engineering of the University of Bologna, for instance, a research concerning an integrated computer aided analysis and design of lightweight structures produced a first interactive computer-aided shape-finding program that ran, in 1973, on an IBM mainframe with a video Console 2250. Nowadays, the interactive programs, written in C++ are object-oriented (OLE) under Windows 98 and Windows NT platform [6] (Figure 6).

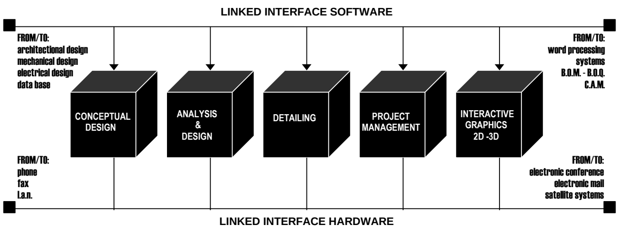
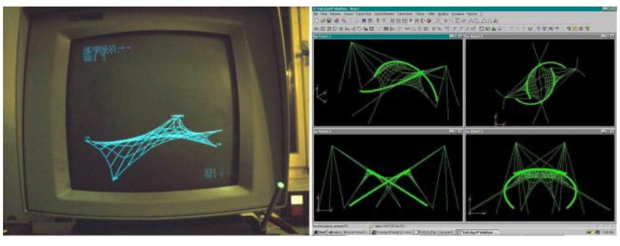


Figure 5 Hardware and software network system.



1973 - IBM 2250 2000 - PC WINDOWS NT

Figure 6 Light weight structural software

The interactive software for analysis and design of special structural systems, as normally involved in wide span enclosures requires, more than general pourpose programs, addressed software to assist on many aspects of theoretical analysis as:

- state '0' form-finding analysis, for the shape-finding of cable, membrane and pneumatic structures;
- non linear material analysis for elastic, anelastic and plasticity including short and long term creeping;
- non linear geometrical analysis; for the static and dynamic analysis under large displacements;
- stocastic dynamic analysis in frequency domain for the buffeting response under the random wind action assisted by the experimental identification, on scale rigid models, of cross-correlated power spectral densities (PSD) of the internal and external pressures on large enclosures;
- stocastic dynamic analysis in time domain for the control of the aerodynamic stability of wide and flexible structural systems under wind excitation, assisted by the experimental identification, on aeroelastic scale models, of the cross-correlated time histories, considering fluid interactions;
- application of the optimization techniques to the structural design [7];
- parametric stocastic sensibility & reliability analysis.

The advantages offered by the informatic and automation has been very important in the field of structural design in general and particularly essential in the case of long span lightweight structural systems. It was possible to examine more rigorous theoretical models avoiding, on the one hand, excessive simplifications that deprive the theoretical model, as a schematic reduction of the reality, of all significance and, on the other, that exhausting calculations lead to the loss of facts with a true influence, with the consequent discouragement of the designer from making efforts towards trying out different structural solutions.
Under those apparently favourable circumnstances, many documented structural failures has been detected where mistakes in the inadequate appreciation of structural behaviour was caused by unreliable man-machine interaction and the illusion that the computers, as powerfull instrument of analysis, could replace conceptual design. For this purpose, IABSE have set up a special commission for the control of automation in structural design [8] . Documented.FEM modelling errors are illustrated in the First International Conference on computational Structures Technology [9].

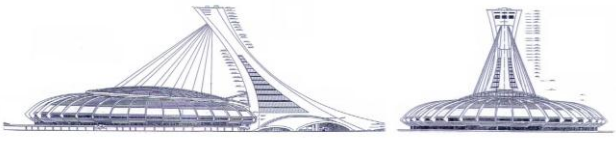


Figure 7 Montreal Olympic Stadium - A cable stayed roof solution

SOME WIDE SPAN ENCLOSURES
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.

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.

Snow loading experimental analysis on scale models

Olympic Stadium in Montreal. During the design of the new roof for the Montreal Olympic Stadium Figure 7 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.

The experimental investigation was carried out by RWDI [10] to provide design snow according to FAE (Finite Area Element) method, representing up to day a state of the art on the matter.

The FAE method uses a combination of wind tunnel tests on a scale model and computer simulation to provide the most accurate assessment possible to estimate 30 year snow loads.

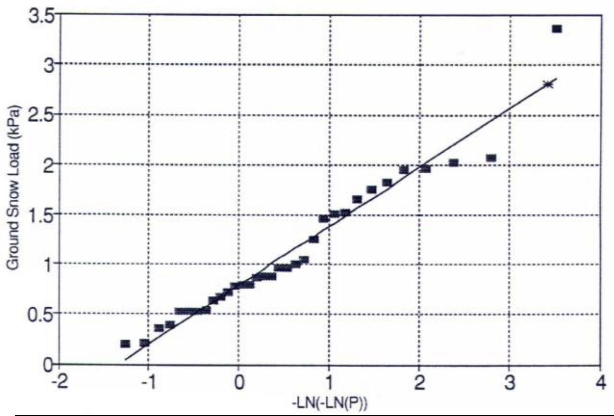


Figure 8 Fisher-Typett Type 1 extreme values plot

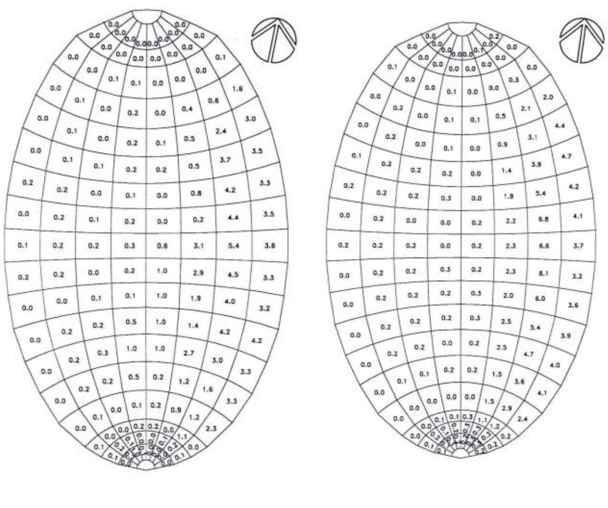


Figure 9 Comparative analysis of snow loading

distribution in function of roof shape (10-13m)

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 current NBCC (National Building Code of Canada) provides minimum design loads for roofs which are based primarily on field observations made on a variety of roofs and on a statistical analysis of ground snow load data. There are, however, numerous situations where the geometry of the roof being studied and the particulars of the site are not well covered by the general provisions of the code. In these situations, a special study, using analytical, computational and model test methods, can be very beneficial since it allows the specific building geometry, site particulars and local climatic factors to all be taken into account. The National Building Code allows these types of studies through its "equivalency" clause and various references to special studies in its commentary.

The model of the three new roof shapes were each constructed at 1:400 scale for the wind tunnel tests. The three model roof designs were each instrumented with 90^{o} directional surface wind velocity vector sensors covering the surface. On the console roof, an additional 90 sensors were installed. Measurements of the local wind speed and direction, at an equivalent full-scale height of 1 m above the roof surface, were taken for 16 wind directions. The wind speed measurements were then converted to ratios of wind speed at the roof surface to the reference wind speed measured at a height equivalent at full scale to 600 m.

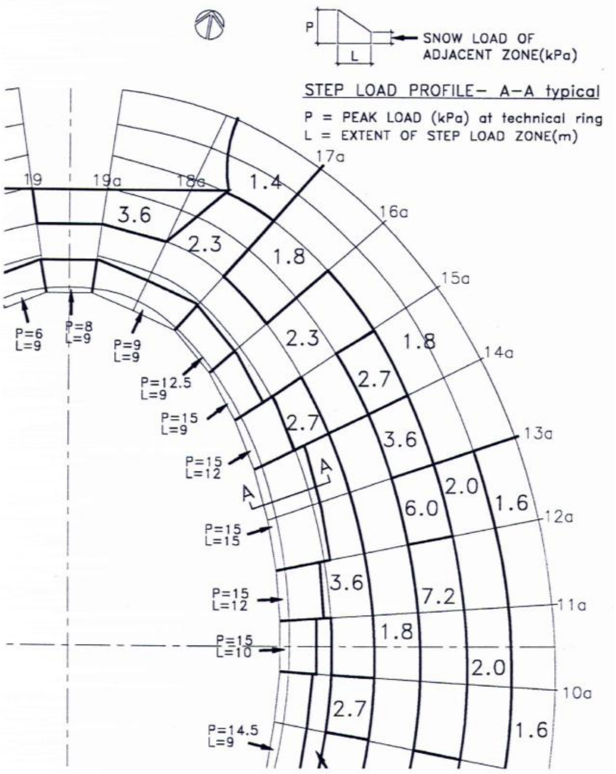


Figure 10 Sliding and wind snow accumulations step

loads

The plot shown in Figure 8, obtained by interpolation of the data using the Fisher-Typett type I extreme value distribution method, predicts the 30 year ground snow load, including both snow and rain (S_s+ S_r), to be 2.8 kPa, which is in agreement with the code value.

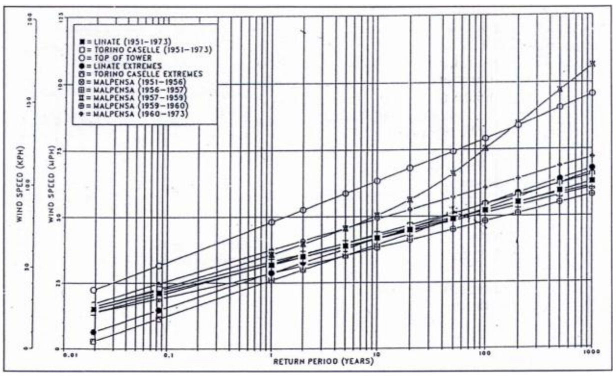


Figure 11 Statistical investigation for the reference 50 years return period wind speed

Results of structural load cases and local peak loading, not to be considered as acting over the roof simultaneously are shown in Fig. 9-10. 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.

<u>Wind loading-experimental analysis on scale models: rigid structures-quasi static behaviour.</u>

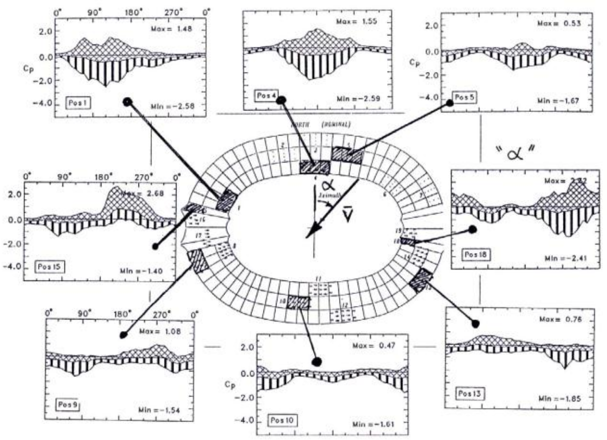


Figure 12 Panel loading as a function of wind direction

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 Turin and Rome stadiums [11-12-13] 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 [12].

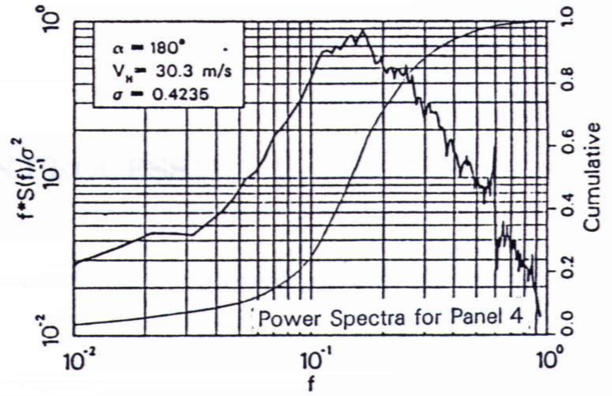


Figure 13 Typical spectra of panel loads

In that case preliminary estimates of the resonant response were obtained from the panel spectra using interpolation to estimate magnitudes and assuming no correlation (at the natural frequencies) between panels Fig.12.

These estimates proved to be significantly larger than those observed on the aeroelastic model due to significant aerodynamic damping effects not included in the prediction process (see Figures 13-14).

The situation is further complicated by the inability of the instrumentation system to provide a complete description of the loading. Only seven of about 60 panels were instrumented and the data obtained must be interpolated to provide estimates of the overall loading, the interpolation required is concerned not only with the magnitude of the panel loads but also the spectra and cross-spectra.

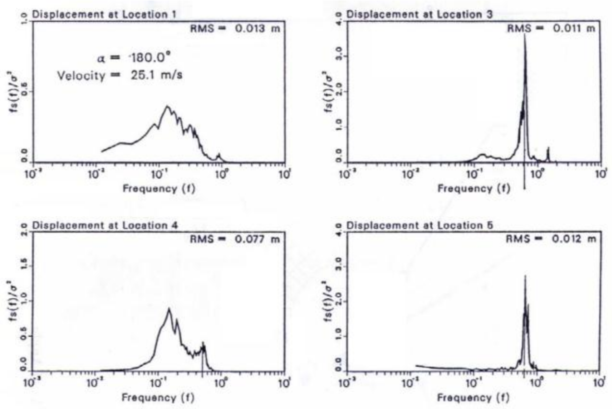


Figure 14 Selected spectra of roof deflections

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 $2x10^6$ 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, $Q_j(t)$, such that:

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

where:

$p(x,y,t)$	nett 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:

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

A_i	area of ith panel;
p_i	pneumatic average of pressure at the taps in the i^{th} panel;
x_i, y_i	geometric centre of the taps on the i^{th} panel;
N	number of panels.

The requirements of a system designed to produce the load histories, $\phi_j(t)$, is discussed in the following section.

The Thessaloniki Olympic sport complex: measurement and use of load time histories.-

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”.

If the weighting functions, $\phi_j(t)$, are chosen as mode shapes then $\phi_j(t)$ is a modal load and its use in conjunction with a dynamic model is clear; either as a set of time histories or a set of modal force spectra and cross-spectra. In the initial stages of a design the roof shape is probably known with reasonable accuracy but mode shapes not so. In such cases it might be appropriate to choose a suitable set of ϕ_j from which modal loads corresponding to shapes ψ_i can be estimated when the design is more advanced. In such a case we can approximate ψ_j as:

$$\psi_j \cong \psi_j^i \sum_i^M a_{ij}\phi_j \qquad (3)$$

the values of a_{ij} can be evaluated by minimizing the discrepancy between ψ_j and ψ_j, ie:

$$\frac{\partial}{\partial a_{ij}} \int \left(\psi_j - \sum_i a_{ij}\phi_i \right)^2 dA = 0 \qquad (4)$$

$$i = 1, M$$

If the functions ϕ_i are chosen as a set of orthogonal shapes $\int \phi_i\phi_j dA = 0; i \neq j$ then the coefficients are given as

$$a_{ij} = \frac{\int \phi_i\phi_j dA}{\int \phi_i^2\phi_j dA} \qquad (5)$$

For a finite panel sizes the corresponding relationship is:

$$a_{ij} = \frac{\sum_k^N \phi_i(\overline{x}_k,\overline{y}_k)\phi_j(\overline{x}_k,\overline{y}_k)A_k}{\sum_k \phi_i^2(\overline{x}_k,\overline{y}_k)A_k} \qquad (6)$$

where:

$$\sum_k^N \phi_i(\overline{x}_k,\overline{y}_k)\phi_j(\overline{x}_k,\overline{y}_k)A_k = 0$$

$$i \neq j$$

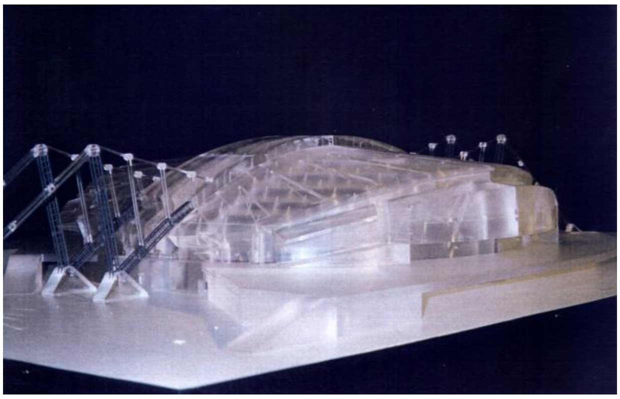


Figure 15 Views of pressure model

The experiment would involve the recording of the local histories $\psi_j(t)$ from which the model time histories could be constructed and the analysis conduced in either the time or frequency domain Figures 15-18). 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 [12]. In such a case the time domain solution, which preserves the extreme value distribution, is to be preferred over a frequency domain approach.

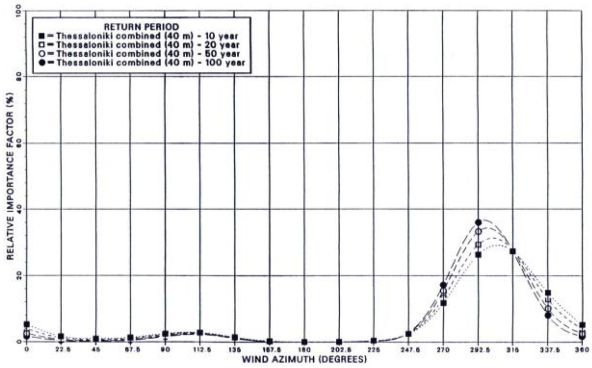


Figure 16 Relative contribution of Azimuthal Direction to the exceedance probability of various return period wind speeds for Thermi, Thessaloniki, Greece

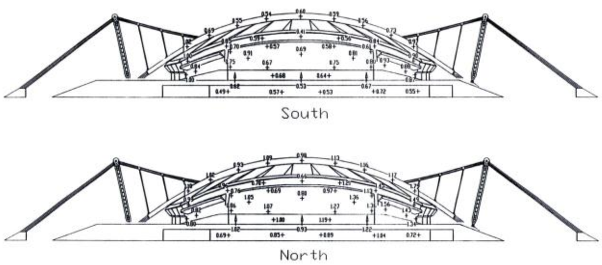


Figure 17 Predicted 50 year return period peak differential pressures

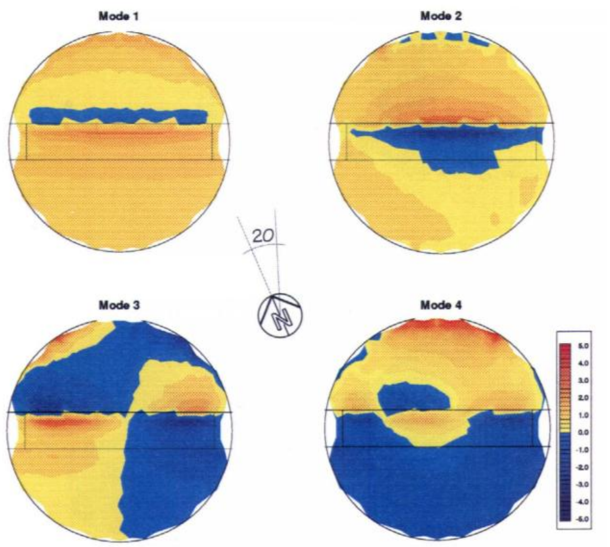


Figure 18 Orthogonal decomposition: pressure mode shapes

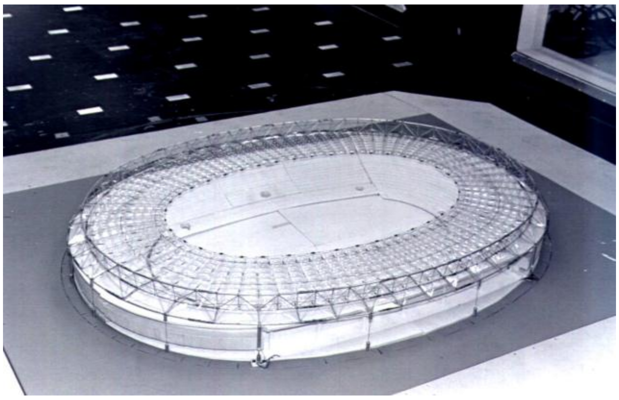


Figure 19 Aeroelastic model for Rome Olympic Stadium

<u>Wind loading-experimental analysis on scale models : flexible structures-aerodynamic behaviour</u>

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 [14-15].Wind tunnel tests have been carried out at the BLWT Lab. of UWO on a model of 1:200 Fig. 19 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).

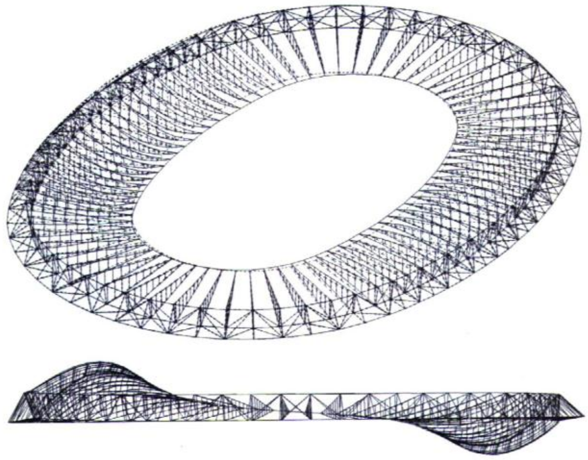


Figure 20 1st modal shape (T=1.78 s)

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 horizzontal 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. 21.

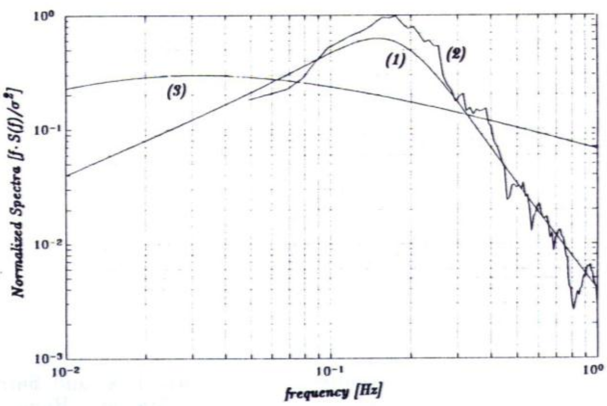


Figure 21 Target (1), simulated (2) and Kaimal's (3) normalized spectra of wind velocity

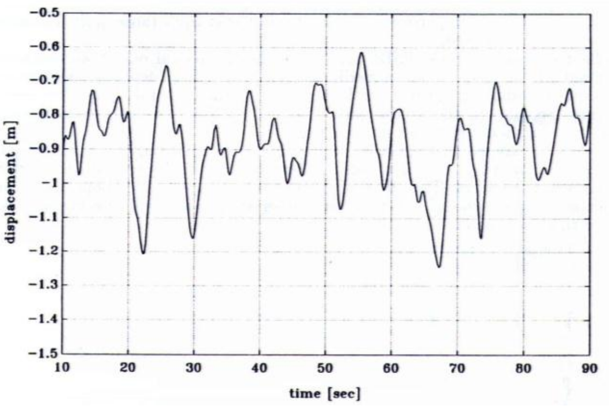


Figure 22 Time History of the displacement (leeward side at tension ring, run #2)

A fluid-interaction non linear analysis in time domain, made for the checking of La Plata stadium design [16-17] 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.9) 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 [10]:

$$P_f = \int_{D_f} f_{\Delta\varepsilon}(\Delta\varepsilon)\cdot d\Delta\varepsilon \tag{7}$$

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 $\boldsymbol{\varepsilon}$. Considering that these effects are represented by the vector of random variables $\boldsymbol{\Delta\varepsilon}$ with mean values $\boldsymbol{\mu}$ and standard variations $\boldsymbol{\sigma}$, the problem is to estimate the probability, P_f, that the generated random bending moments $\boldsymbol{M}$ will be larger than the plate ultimate resistance moments, $\boldsymbol{M_u}$, at any of the ***m*** points of the structural plates system.

3.2 Roof structural system data

The following probabilistic description was considered for the random variables $\boldsymbol{\Delta\varepsilon}$.

$\boldsymbol{\mu}$ = Vector of mean values of $\boldsymbol{\Delta\varepsilon = 0}$ (i.e., all possible actions on the cables are considered by the load combination itself).

$\boldsymbol{\sigma}$ = Vector of standard deviations of $\boldsymbol{\Delta\varepsilon = 0}$. The $\boldsymbol{\sigma}$ values were varied from $0.5\text{x}10^{-3}$ to $0.1\text{x}10^{-3}$ so that the sensibility of the system can be studied. These values were selected to cover the range of failure probabilities of practical significance.

$\boldsymbol{f_{\Delta\varepsilon}(\Delta\varepsilon)}$ = Probability density function = Normal distribution with parameters $\boldsymbol{\mu}$ and $\boldsymbol{\sigma}$.

3.3 Failure condition

For load case *"i"* the bending moments, M_x, M_y y M_{xy} in the 130 points of the plate can be computed as follow:

$$M_x = M_{Gx_i} + \sum_{j=1}^{34} A_{x_{i,j}}.\Delta\varepsilon_j$$

$$M_y = M_{Gy_i} + \sum_{j=1}^{34} A_{y_{i,j}}.\Delta\varepsilon_j$$

$$M_{xy} = M_{Gxy_i} + \sum_{j=1}^{34} A_{xy_{i,j}}.\Delta\varepsilon_j \qquad (8)$$

Considering the bending moments in each direction, the failure functions at each point of the plate ($1 \le r \le 130$), $G_r(\Delta\boldsymbol{\varepsilon})$, are the following hyperplanes,

$$M_{Upx} - (M_{Gx_i} + \sum_{j=1}^{34} A_{x_{i,j}}.\Delta\varepsilon_j) < 0$$

$$M_{Upy} - (M_{Gy_i} + \sum_{j=1}^{34} A_{y_{i,j}}.\Delta\varepsilon_j) < 0$$

(9)

$$M_{Unx} - Abs(M_{Gx_i} + \sum_{j=1}^{34} A_{x_{i,j}}.\Delta\varepsilon_j) < 0$$

$$M_{Uny} - Abs(M_{Gy_i} + \sum_{j=1}^{34} A_{y_{i,j}}.\Delta\varepsilon_j) < 0$$

(10)

$$M_{Uxy} - Abs(M_{Gxy_i} + \sum_{j=1}^{34} A_{xy_{i,j}}.\Delta\varepsilon_j) < 0$$

(11)

where $G_r \le 0$ is failure and M_{Uxy} is computed from the Johanssen Theory as the smallest of the following expressions

$$M_{Uxy} = (M_{Upx} + M_{Upy})/2$$

$$M_{Uxy} = (M_{Unx} + M_{Uny})/2$$

(12)

In these formulas, M_{Upx}, M_{Upy}, M_{Unx}, M_{Uny} and M_{Uxy} are considered always positive.

The failure condition is obtained when failure is reached at any point of the plate, i.e., the structural failure can be defined as

$$(G_1 \le 0) \cup (G_2 \le 0) \cup \_ \cup (G_{130} \le 0)$$

(13)

3.4 Solution method

Since a closed form solution is not possible for the integral in (7) the failure domain defined by equations above, Montecarlo Simulation must be used. By Montecarlo Simulation, the failure probability is obtained by computing $\boldsymbol{G_r(\Delta\varepsilon)}$ for several values of $\boldsymbol{\Delta\varepsilon}$ generated with normal distribution. An approximation to the failure probability is obtained by counting the number of times that $\boldsymbol{\Delta\varepsilon}$ belong to the $\boldsymbol{D_f}$ with respect to the total number of simulations. For small failure probabilities, however, direct application of Montecarlo Simulation is not possible because of the large number of needed iterations to get enough accuracy. To avoid this problem, the Orientated Simulation Method was used in this report. A complete description of the method can be found in the paper [10

3.5 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 $\sigma = 0.5 \times 10^{-3}$ for $\boldsymbol{\Delta\varepsilon}$ of all cables. In Appendix III are shown the distribution of Mx, My, Mxy and β for each load case and for that standard deviation. The following table summarizes the index β (computed with $\sigma = 0.5 \times 10^{-3}$).

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.

3.5 Failure probability and sensibility analysis

The figure 12, shows the failure probability for load combination 7 as a function of the standard deviation, σ, of the cable strain variations, $\boldsymbol{\Delta\varepsilon}$.

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 $2\text{x}10^{-4}$ to $3\text{x}10^{-4}$, Pf is increased from $2\text{x}10^{-5}$ to $480\text{x}10^{-5}$.

b. Cable standard deviation, σ, should be maintained below $2\text{x}10^{-4}$ for the designed ultimate bending moment.

c. Larger cable standard deviation, σ, could be allowed increased the slab reinforcement along x-direction in the critical roof zone.

The figure 13, shows the most probable values of Δε ($\text{x}10^{-3}$) in each cable at failure for load combination 7.

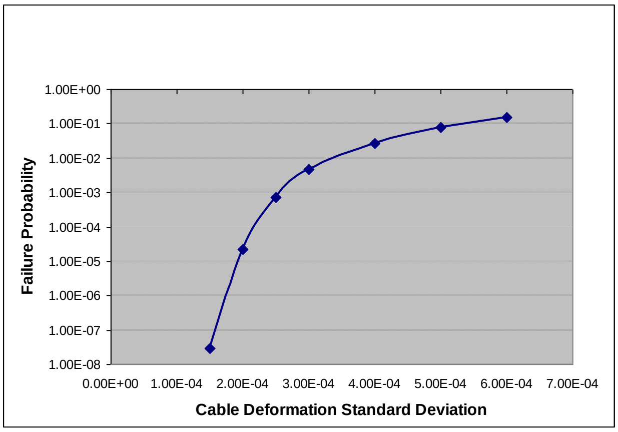


Figure 12 – Failure probability in function of cable deformation standard deviation

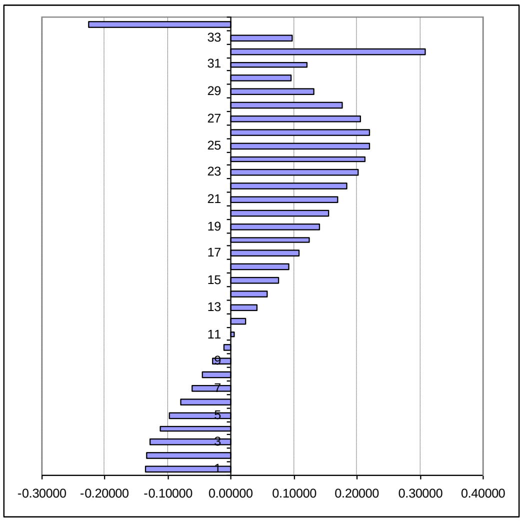


Figure 13 - Predicted 50 year return period peak differential pressures

The following comments can be done.

a. The most probable values of Δε are practically independent of the standard deviation σ. In other words, the configuration at failure is constant. This configuration is reached with more probability as the standard deviation of Δε increases.

b. The most probable configuration at failure is mainly due to variations in the strains of cables 32 and 34. Since elongations of cables can be computed as ΔL=L Δε, the elongation at failure of cables 32 and 34 are approximately ΔL_{32}=210m x (-0.2x10-3) = 4.2 cm and ΔL_{34}= 210m x (0.3 x10-3) = 6.3 cm.

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