CONCEPTUAL DESIGN, EXPERIMENTAL AND THEORETICAL ANALYSIS, DETAILING AND PERFORMANCE OF LONG SPAN LIGHTWEIGHT STRUCTURAL SYSTEMS

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ABSTRACT: Long 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. In order to increase the reliability assessment of long span lightweight structural system a knowledge based synthetical conceptual design approach is recommended. Experimental and theoretical 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.

1. INTRODUCTION

Long span structures are today widely applied for:

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

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

According to the state of the art, the lightweight structural tipologies and materials more frequently used for long span structural systems are:

Space structures
- single layer
- double layer
- double curvature
- single curvature

Cable structures
- cable stayed roofs
- suspended roofs
- cable trustes
- single layer nets

Membrane structures
- double curvature prestressed membrane
- pneumatic membrane

Hybrid structures (materials)
- steel and alluminium
- structural glass
- carbon fibres
- fiber glass and PTFE
- aramidic fibers (KEVLAR)
- ceramic materials
- smart materials

Hybrid structures (typology)
- tensegrity system
- beam-cable system

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

Due to the different scale 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 power spectral densities or correlated 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 failure;
- the compatibility of detail design with the modelling hypothesis;
- 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.

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), 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 (Ref 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 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.

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 1 Prime causes of failure. Adapted from Walker (1981).

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.

2. KNOWLEDGE BASED CONCEPTUAL DESIGN AND RELIABILITY LEVEL

The conceptual design (Figure 1) 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 errors. Gross errors may be removed, also informally, as a result of the observation: "something is wrong".

In the conceptual design phase the structural expert contributes in finding a design solution together with other specialized professionals (architects, project managers, mechanical engineers, etc.). According to the design requirements the conceptual design is defined by a knowledged expert

Figure 1 Conceptual design and analysis of structural systems.

synthetical approach based on a reliability intuition of the selected model which has to be confirmed by the results of the analysis phase. The conceptual design phase 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 is intended 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 consider the need to achieve, by the institution of a "safety plan" the requirements of structural safety, serviceability and durability.

QA procedures include:

a) proper definition of functions;
b) definition of tasks, responsibilities, duties;
c) adequate information flow;
d) control plans and check lists;
e) documentation of accepted risks and supervision plan;
f) inspection and maintenance plan;
g) user instructions.

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. The real danger is that excessive formalization of QA procedures will lead to unacceptable and self-defeating degenerations, in a certain kind of Kafkian bureaucratic engineering and management.

2.1 Knowledge based on lightweight structures: from compression to tension.

The most effective exploitation of the properties of special high-strength materials is achieved with a structural system which is tensioned throughout. This leads naturally to exploitation of its possible economies and 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 framed structures. Therefore, lightweight structural typologies must combine a dominant tension mechanical system and hi-strength materials.

The mechanism of resistance of tension structures can be illustrated as shown in Figure 2 where, for a thin parabolic arch loaded uniformly, 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 liberaration of kinetic energy (cross hatched area). Here, the secondary term of total potential energy is negative ($\delta^2\pi < 0$).

Figure 2 Mechanical behaviour from arch to cable.

– 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 a limit state of deformation.

In Table 2 it is possible to observe the exceptionally high strength to weight ratio in tension (K_t) hi-tech materials, important component of lightweight structural concept.

Figure 3 Statistical analysis of ground snow intensity.

Materials	σ_t^R N/mm^2	σ_c^R N/mm^2	γ_k N/m^3 10^3	K_t m	Kc m
Bricks		3	18		0.166
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 2 Mechanical properties of constructions materials.

3. EXPERIMENTAL ANALYSIS OF SCALE MODELS (CASE STUDIES)

Long span structures needs special investigations concerning the actual 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 investigations.

From the direct author's experience in designing large coverings, the most important investigation regards the snow drift and accumulation factors and the dynamic action of wind loading.

3.1 Snow loading.

Stadium of the Alpes (Torino). During the design phase of the Torino Stadium under the lack of definition of basic loadings by the National Building Code, it was necessary to proceed to the statistical investigation of the characteristic intensity value for a specified return period. In Figure 3 the characteristic ground snow intensity for different values of T_o are illustrated.

From the basic uniform distributed valued with To = 175 years an experimental intestigations was conducted by CEBTP in order to determine the point, linear and surface snow accumulation indices ia,s. The method adopted was that of the BEAN (Banc d'essais d'accumulation de la Neige due au vent) using "simil snow" resin 400 µ particles uniformly distributed for a reference thickness of e_m = 5 mm. The particles are dragged by the correlated wind actions directed every 30º form main axis. In Figure 4 and Figure 5 it is possible to observe the accumulations shapes of snow distribution for wind direction along main and transversal axis of the Stadium.

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

The experimental investigation was carried out by RWDI (Ref 2) 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.

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 side 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.

Figure 4 Snow accumulation factor (wind direction 0º)

Figure 5 Snow accumulation factor (wind direction 90º)

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 90º directional surface wind velocity vector sensors covering the surface (Figure 6). 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 6 Montreal Olympic Stadium - New roof.

Figure 7 30 years return period snow loads (kPa).

FAE Computer Simulation. The roof of the building was initially divided into orthogonal elemental areas by an orthogonal grid system (Figure 8). The accumulation of snow in each area element is dependent upon the quantity of snow that falls into the element and the amount that drifts into and out the element. Increases in snow loads can occur when rainfall is absorbed by the snowpack and decreases can occur as a result of: melting and runoff from above

freezing temperatures; solar radiation; and/or heat loss through the roof surface. By stepping through meteorological data for Montreal/Dorval International Airport in one-hour interval, the snow loads were evaluated for all elements of the grid. Critical load conditions, such as maximum overall snow load on the roof or maximum snow load on a particular smaller zone such as along the roof edge, were then examined in detail.

Heat loss through the roof of a building is defined by the thermal resistance of the building envelope (R-value). For the new roof shapes an R-value of 3.0 m^2 $^oC/W$ was used. A value of 3.5 m^2 $^oC/W$ was used for the console roof. It was assumed that an indoor air temperature of 20 ^oC would be maintained at all times.

Thirty-three years of meteorological data from the Montreal/Dorval International Airport were used to create hourly records of mean gradient wind speed, wind direction, air temperature, snowfall, rainfall, cloud cover and cloud opacity for this study.

When the meteorological data, obtained from the local airport, is processed though the computer simulation, the approach is taken that the ground snow load produced by the simulation should be consistent with the value of the ground snow load fiven by the 1990 NBCC, which is based on extensive research by the Atmospheric Environment Service. For the Olympic site in Montreal, the 1990 NBCC specifies a 30 year ground snow load

Figure 8 Location of wind velocity vector sensors and grid system.

of 2.4 kPa with a rain load of 0.4 Pa. The plot shown in Figure 9, 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 9 Fisher-Typett Type 1 extreme values plot ground snow load prediction

Results of structural load cases and local peak loading, not to be considered as acting over the roof simultaneously are shown in Figure 7 and Figure 10.

3.2 Wind loading-test data and design processes.

The integration of the wind tunnel data into the design process presents significant problems for a structure of this type; in contrast to buildings where knowledge of the base moment provides a sound for preliminary design there is not single simple measure for the roof. 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 10 Structural loads on console roof (kPa).

In the present 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. 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.

The study of the Turin and Rome stadiums 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 (Ref 3).

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 ith panel;
x_i, y_i	geometric centre of the taps on the ith 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 measurement and use of load time histories. 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:

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

$$i \neq j$$

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. For the type of structure under cosideration 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 Gaussiaan form (Ref 4). In such a case the time domain solution which preserves the extreme value distribution is to be preferred over a frequency domain approach.

In Figure 11 it is possible to observe a) the statistical analysis of average wind velocity for Torino Stadium, b) the panel wind pressure coefficients, c) selected spectra of roof displacements, and d) pressure mode shape for Montreal Stadium according the orthogonal decomposition method (Ref 5).

4. COMPUTER AIDED DESIGN AND 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 a useful interactive design cycle (Figure 12). The computer aided design methodology semplifies complex tasks and increases the reliability level.

The interactive computer assistance must be organized as follows:

- the interactive design methodology is not substitutive but rather integrates the creative aspects of the traditional design process. By means of an interactive
- 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 according to the classical step by step iterative procedure of trial and error 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 12 Interactive design process.

Figure 13 Hardware and software network system.

At present time it seems to be very difficult to introduce aspects of artificial intelligence inside the process of design of light weight structures as illustrated in Figure 15 (expert system 3^{rd} level software). The design, analysis, management, monitoring, etc. phases are today linked as a hardware and software network (Figure 13).

Interactive graphic language addressed to the structural design of lightweight structures. The development of the lightweight structural concept is historically correlated with the research in CAD technology. For instance, research concerning an integrated computer aided analysis and design of membrane structures started in the Department of Structural Engineering of the University of Bologna in eraly of 1970. The first interactive computer-aided shape-finding program ran on an IBM mainframe with a video Console 2250, supplemented with a CDC 6600 with a Tektronik 4010 video display. Nowadays, the interactive programs available in this centre run on mini-computers and PC under Windows 3.1, Windows NT and Windows 95 (Ref 6). The interactive software for analysis and design of membrane structures developed to date have the following names:

RETE	for the shape-finding of cable and membrane structures;
PNEUS	for the shape-finding of pneumatic structures;
TENSO	for the statical non-linear analysis of cable/membrane structures;
TENSO-TEL	version of TENSO, for the analysis of cable membrane structures which interact with anchorage structural systems;
TENSO-DIN	for the dynamical non-linear analysis of cable and membranes structures;
STRIP	for interactive cutting pattern on geodetic surface lines.

Many aspects of theoretical analysis has been developed in the techincal literature which cannot be illustrated in this paper as:

- state '0' form-finding analysis;
- analysis under strong material non linearities including short and long term creeping;
- analysis under strong geometrical non linearities;

- response analysis under random wind excitation - dynamic stability - fluid interactions;
- parametric sensibility reliability analysis.

Due to the lack of space, only an interesting illustrations concerning the sensibility analysis regarding the new cable stayed roof of the Olympic Stadium in Montreal can be included in the present paper.

4.1 Reliability analysis. Equilibrium and compatibility.

Many times, during the initial stages of the architectural design, structural engineering's intervention is almost absent. In this situation the geometry, typologies and materials are defined in preliminary design under poor structural knowledge base.

Oftenly happens that, under particular political and financial conditions, in a final design phase the inertia of the design process does not allow any changement of an original unsatisfactory conceptual design. When such a situation is created, structural engineers are forced to find solutions under very strict boundary conditions. A very typical case was that of the retractable membrane roof of the Montreal Olympic Stadium (Ref 7), built in 1985 under design conditions determined in a first construction phase related to the preparations of the olympic games in 1976. Several local roof failures under non extreme wind conditions appeared in 1988 and 1991.

A new cable stayed spatial steel framework is actually under final design in order to replace the damaged membrane roof. This design collected all the previous problems related to the difficult boundary conditions, determined by the existing structure. The new roof of around 20000 m^2 will be suspended from the inclined tower by 28 stay cables. Due to the particular boundary conditions and low ratio between dead loads and live loads the structural system, globally determined by the equilibrium conditions, it appears to be sensitive to internal compatibility between space frame ring and cable supporting substructure. In this case it was necessary to proceed to an elastic and anelastic response of the roof system according to an oriented statistical simulations' sensibility analysis in order to determine the probability of failure and the β-index in function of parametric variations related to random uncertainties of the stiffness of cable stays. This research will be published extensively elsewhere (Ref 9).

Figure 14 Computer Aided Design plotted output and results.

Figure 15 Flow chart expert system.

Reliability analysis of the roof structural system. Cable strain parameric sensibility.

The uncertainties on the elastic modulus of the cable, geometrical and elastic long term creeping, tollerances of erections, non linear behaviour, created a sensitive response on the rigid space frame hanging from a set of 28 stay cables. The sensibility analysis showed that the response is sensitive to the standard deviation of the cable strain variations and the most probable failure mechanism will involve primarily the longest cables (Figure 16).

Figure 16 Failure probability for load Case 1.

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 breaking simulation of same sensitive cable elements (Figure 17).

5. DETAIL DESIGN

Conceptual detail design plays a significant role in the real response of long span structures under the influence of direct or indirect actions.

The design of a Bologna Fair pavillon of 90 m span (Figure 18) provides the end restrains of the steel main structure allowing temperature displacements and, at the same time, transmiting forces due to wind and earthquake. The use of a special visco-elastic device will allow low displacements due to snow and temperature.

Under service conditions the joints return to the original position. When dynamic displacements are produced the detail reacts viscously damped. The new Sport Hall in Pesaro (Figure 18) adopts a special flange detail, in order to avoid parassitic high local bending and eventual welding cracks under tightening the bolted connections.

Figure 17 Breaking simulation of senitive cables.

5.1 Avoiding chain collapse.

Many modern buildings' codes introduced the concept of preventing to avoid chain collapses in large structural system. Under the conceptual design phase of the Olympic Stadium in Rome (Ref 8) special attention has been paid to the incremental failure analysis of part of the structure. The membrane elements are uncorrelated with the cable structure in order to avoid fragile rupture propagation and facilitate eventual repair and/or substitutions of membrane panels (Figure 18). A chain failure simulation analysis shows that the structural system allows for complete stress re-distribution under partial collapse of more than 4 rope trusses. The tension inner ring is formed by 12 cables minimizing the effect of individual failure.

The project of another complex structural system as the Stadium of the Alpes (Ref 10) includes a special detail design avoiding collaborations between column and cable and eliminating unnecessary and dangerous interaction. This was determined by the high sensibility of the column stability to parametric local bending (Figure 18).

Figure 18 Detail design.

6. MONITORING

The roof structures of the Torino Stadium have been built in 1990. According to the quality control and maintenanceprogram, 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.

6.1 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. 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% (Figure 19).

6.2 Interpretation of results.

In order to have a better scenario of the pre-stressing level and distribution, a new control of all anchorage forces under uniform temperature condition (night-time) had been produced. The results confirmed the spot control measurements. Considering the main characteristics of the structural typology adopted or the stadium the variation of the pre-stressing state can be related principally to:

- non uniform temperature variation;
- elastic an anelastic foundations' settlements and soil interactions;
- random errors in cutting or marking the cables;
- geometrical, elastic or anelastic short and long term creeping of cables;
- errors in pre-stressing procedure.

The theoretical simulations, which include the compsuntive updating of the permanent loads, provide to the eliminations of the uncertainties related to the loading variation's influence. The soil conditions and measurements of settlements during 4 years permit to remove any consideration of influence of soil interaction in variation of pre-stressing level. Measurements taken during night-time show no influence for temperature variations.

Considering that peak values are random distributed it is also possible to disregard the influence of errors in cutting and marking of cables, normally respecting the symmetry of construction. The logical answer, therefore, confirmed by a parallel experience on the structural system of the Olympic Stadium in Rome, is that the variation noted in pre-stressing level of the system can be related to a combinations of creeping and to uncertainties concerning the pre-stressing procedure.

"Geometrical" equivalent creeping of spiral strands is supposed to be removed by 50% breaking load shop prestressing cycles. Many cable structures showed creeping in short and long term. Creeping of cables is not well documented in technical literature. Some fabricators give 3‰ as "possible values" of anelastic creeping which imply the impossibility to guarantee any value of E-modules (3‰ of E imply a variation of 500 N/mm^2 !).

Measurements of the geometry of the central ring (Figure 20) show a separation between expected theoretical and measured values, but the experimental Z-configuration of the ring had only small variations in time during 3-year observations.

The consequence of the above mentioned observations is that the changements observed in the pre-stressing state must be related principally to the procedure of pre-stressing under high gradient of daily temperature variations. The tensostructure was prestressed by an iterative "step by step" sequence. The number of steps have to be determined in function of the characteristics of the structural system as the parametric sensibility, the correlation degree between incremental values and the verification of the level of the transitory sequential overstressing. The reliability of the process is increased by the number of iterations, allowing to calibrate and automatically control the pre-stressing according with a predictor-corrector

Figure 19 Stabilizing cable. Anchorage forces. Computed actual values. Measured values. Restored values.

Figure 20 Z coordinate of the inner ring according to experimental measurement.

techniques.

As a matter of fact, probably in order to reduce the time needed for the operation, it was decided to make the pretensioning in just one step. This choice allowed the minimization of the reliability of the process even though the geometric, mechanic, thermical variations along with those related to the mathematical modeling of the pretensioning simulation could contribute to increase, effectively, the varying between the project's values and the ones related to the expected forces, actually induced. The designers recommended the functional restore of the in service state by bringing the pre-stressing values back to the average expected ones.

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.

6.3 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/sec^2	
	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.

7. CONCLUSIONS

It has been noted the influence of knowledge base on conceptual design in removing gross human intervention errors from initial design statements.

Computer assisted methods of design and analysis allow to generate an interactive design cycle which optimizes the user synthetical capacities and the analytical computer elaboration power. Sensibility's analysis is an extremely powerful tool to determine the influence of parametric design uncertainties for unusual long span structural systems.

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