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1 EXTENDED ABSTRACT

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 system, as the state of art on long span structural design. In order to increase the reliability assessment of wide span structural system a knowledge based synthetic conceptual design approach is recommended. Theoretical and experimental in scale analysis, combined with a monitoring control of the subsequent performance of the structural system, can calibrate mathematical modeling and evaluate long term sufficiency of design.

For the direct author's experience of a comparative analysis with experimental investigation on scale model of Rome Olympic Stadium roof (1990), the theoretical model related to the random dynamical analysis of large quasi-horizontal coverings appeared to be not completely consistent. The description of pressure distribution on the covering surface cannot disregard the transversal component of the wind and also is impossible to give the vectorial role to pressure coefficients c_p , independent to time, to change the direction from along wind to normal pressure distribution. As shown in Figure 1 experimental data of wind DSP differ from theoretic values in amplitude and for a phase displacement.

Figure 1: Comparative analysis of the frequency of the wind DSP in direction u and the DSP measured experimentally

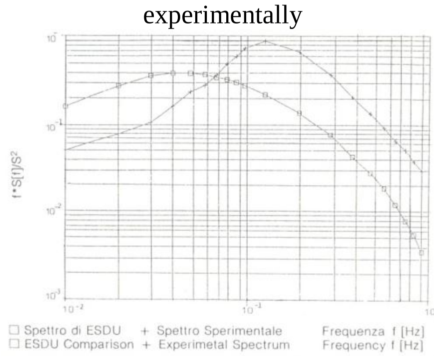
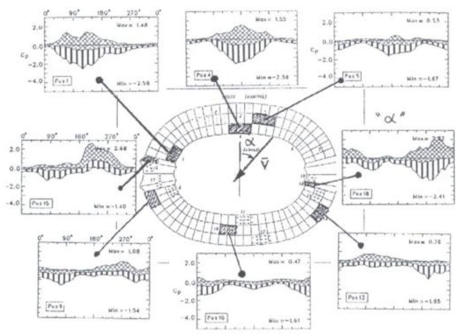


Figure 2: Panel load as a function of wind direction



Results of structural load cases and local peack loading not to be considered as acting over the roof simultaneously because the shape of the roof gives separations of the air flow and turbolence and so the values of the c_p coefficient are similar in uplift or downlift conditions as shown in Figure 2.

In substance 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 [1-2-3] 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.

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. 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 situation in 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.

To produce a complete description of the load it's necessary an high speed pressure scanning system capable of producing essentially simultaneous pressure measurements at some 500 points at rates of 200 Hz per point. 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:

Equation 1

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

where:

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

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

A new very practical method to produce the load histories $\phi_j(t)$, and so to obtain the linear structural response under the random wind action for small displacements, has been applied under the name of the "orthogonal decomposition method". This method has been applied in wind tunnel tests made in collaboration with the Boundary layer wind tunnel laboratory of the University of Western Ontario on the model of the Thermis Sport Hall.

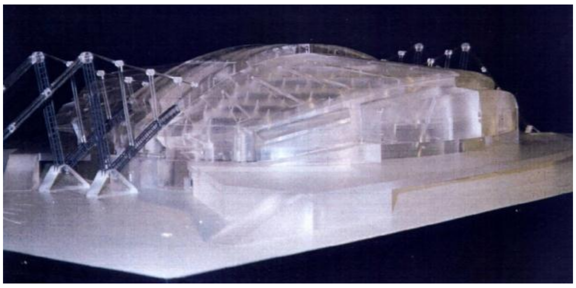


Figure 3: Views of pressure model of Thermis Sport Hall

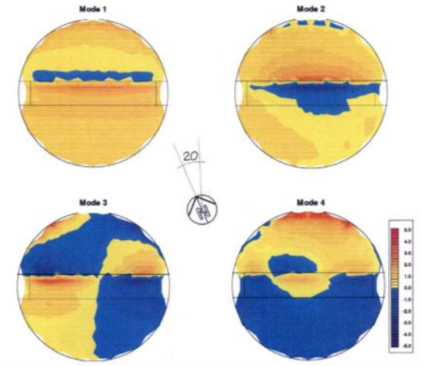


Figure 4: Orthogonal decomposition: pressure mode shapes

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_j^i can be estimated when the design is more advanced. In such a case we can approximate ψ_j as:

Equation 2

$$\psi_j \cong \psi_j^i \sum_i^M a_{ij}\phi_j$$

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

Equation 3

$$\frac{\partial}{\partial \alpha_{ij}} \int \left(\psi_j - \sum_i a_{ij}\phi_i \right)^2 dA = 0 \Rightarrow a_{ij} = \frac{\int \phi_i \phi_j \, dA}{\int \phi_i^2 \phi_j \, dA}$$

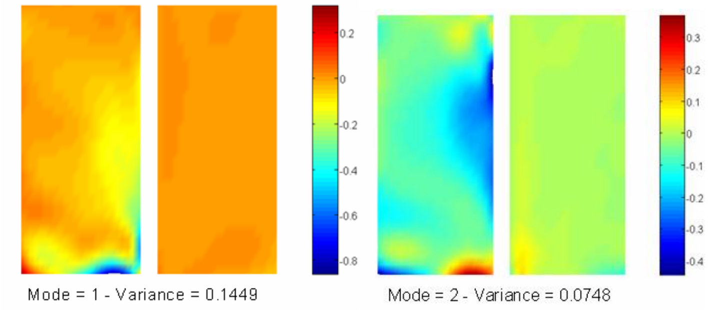
The Proper Orthogonal Decomposition techniques have also been adopted in the wind tunnel test on an aeroelastic model of the Braga Stadium at "Politecnico di Milano", to check the aerodynamic stabilities and the effectiveness of a possible external damping system.

Hence, the wind pressures were derived from the tests on a rigid model. Since the pressure time histories were simultaneously measured at different points, within the upper and the lower sides of the roof panels, the instantaneous pressure fields were available.

As a matter of fact, due to the different spatial distribution of upper and lower pressure taps, preliminary interpolation was required to obtain the differential pressures, which represent the actual load on the structure. Despite the apparent simplicity of this operation, the instantaneous interpolation gave rise to uncertainties and numerical difficulties.

Figure 5: a) Aeroelastic model of the Braga Stadium, b) Proper orthogonal decomposition of the measured wind pressure field

a



b

The analytical procedure is synthesized in the following expression, while a typical result is summarized in Table 1. The Equation 4 resume the procedure to decompose (POD) a pressure field (Figure 5a), the Equation 5 determine the mean and the quasi-static response and the Equation 6 are relative to the resonant contribution.

Equation 4

$$\mathrm{C}_p=\sum_{k=1}^{m}\phi_k\phi_k^T\lambda_k;\qquad \phi_i\phi_j^T=\delta_{ij};\qquad \phi_i^T\mathrm{C}_p\phi_j=\delta_{ij}\lambda_j;\qquad p(t)=\sum_{k=1}^{m}\phi_k x_k(t)$$

Equation 5

$$R(t)=\sum_{k=1}^{s}x_k(t)R_k;\qquad \sigma_{R,qs}^2=\sum_{k=1}^{s}\sigma_{x_k}^2R_k^2;\qquad R_{qs}=\bar{R}\pm g_{qs}\sigma_{R,qs}$$

Equation 6

$$R_{qs+res}=\bar{R}\pm\sqrt{g_{qs}^2\sigma_{R,qs}^2+g_{res}^2\sigma_{R,res}^2};\qquad g_{res}^2\sigma_{R,res}^2=\sum_{h=1}^{n}g_h^2\sigma_{R_h}^2;\qquad g_h=\sqrt{2\ln(f_hT)}+\frac{0.577}{\sqrt{2\ln(f_hT)}};$$

$$M_h=\int_A m(x,y)\psi_h^2(x,y)dA;\qquad Q_h(t)=\int_A p(x,y,t)\psi_h(x,y)dA;\qquad \sigma_{R_h}^2=R_h^2\frac{\pi}{4\xi_h}\frac{f_hS_{Q_h}(f_h)}{(2\pi f_h)^4M_h^2}$$

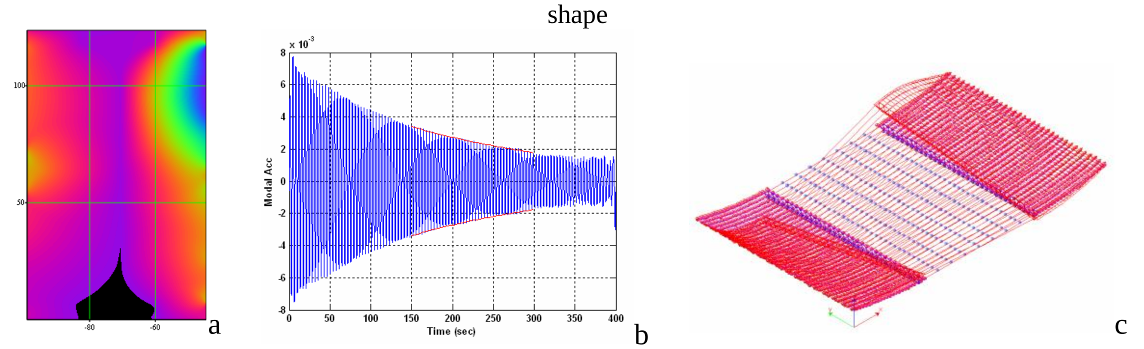
The resonant contribution is proportional to the square of the inherent structural damping ratio. Since this contribute was the most important in the case of the Braga Stadium (see Table 1), a more appropriate evaluation of the structural damping was necessary (full scale dynamic characterization) and an experimental check of the resonant and aeroelastic effects was made (aeroelastic model). The aeroelastic model was also tested by placing (at the roof corners) linear viscous dissipative devices (C = 100-150 kN s/m at full scale). Damping ratios up to 7-8% were reached. The response was subsequently reduced of about 50%, so confirming the analytical estimations.

Table 1: Comparison of mean, quasi-steady and resonant contributions: vertical displacement, wind 270° from north

Node	$\bar{R}$	$g_{qs}^2\sigma_{R,qs}^2$	$\bar{R}\pm\sqrt{g_{qs}^2\sigma_{R,qs}^2}$	$g_{res}^2\sigma_{R,res}^2$	$\bar{R}\pm\sqrt{g_{qs}^2\sigma_{R,qs}^2+g_{res}^2\sigma_{R,res}^2}$
A	0,1550	0,0250	0,3131	0,3027	0,7275
B	-0,0837	0,0115	-0,1910	0,3076	-0,6486
C	-0,0441	0,0014	-0,0812	0,0433	-0,2555
D	0,0930	0,0052	0,1652	0,0419	0,3101
E	0,0621	0,0092	0,1582	0,2040	0,5238
F	-0,0198	0,0016	-0,0599	0,2018	-0,4709

A sensibility analysis of the failure probability to the spatial distribution of wind loads was performed too. This analysis allowed to determine the wind direction which corresponds to the higher failure probability, to recognize the structure regions which have the higher failure probability (Figure 6 a, where the black region show the lowest β values: 3.798) and to determine the wind loads spatial distribution which leads to the higher failure probability. An advantage of this technique with respect to the POD is that it considers both the wind stochastic structure and the structural response.

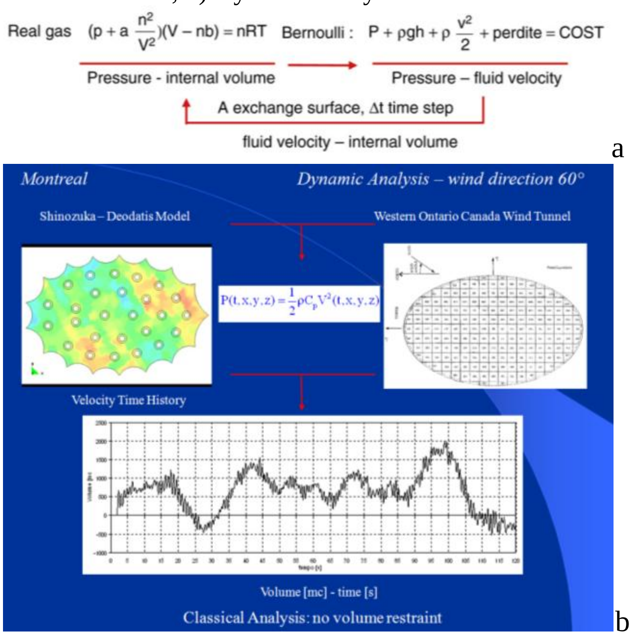
Figure 6: a)Distribution of β values along the slab; b) typical free motion record; c)first natural mode shape



Since January 2004, the structure was equipped with a permanent monitoring system, to measure different parameters (such as pressures, accelerations, wind velocity, cable stresses ecc.) at different significant point of the structure.
As it was found numerically and confirmed by the aeroelastic model, the resonant response is dominant and the structural inherent damping was a very important parameter in determining the response. The dynamic characterization was performed to determine the actual value of the damping ratio for the firsts modes. To excite the system, a cable was linked to the edge steel girder by mean of a pre-tensioned spring and it was sinusoidally moved at the other end by an electrical engine. To measure the structural dynamic response under the applied dynamic loads and the free vibrations decay after the loading excitation stop, 6 accelerometers were placed on the roof slab.

In order to evaluate the natural modal shapes and the corresponding modal damping, the free vibration decay time series, recorded after the resonant harmonic excitation stop, have been used (Figure 6 b). The modal shapes were determined by orthogonal decomposing (POD) the recorded signals (Figure 6 c).

Figure 7: a) Interaction model between structure and internal volume; b) Dynamic analysis



The structural response of wide-span roofing such as the structure over the Montreal Stadium is particularly difficult to assess, essentially because of the geometrically non-linear behaviour this type of structure typically features and complexity of any bearing load simulation.

Due to the flexibility of the roof of the Montreal Olympic Stadium, fluid acts on the directly exposed outer surface of roof as well as on its internal surface depending on the internal volume changes. The roof encloses a fluid volume that can be classified as an appendage of external fluid where the connection is due to exchange surfaces, such as stand access openings, etc. In case of large openings, the fluid field produces point-to-point variable internal pressures, acting on the internal side, which are rather small but not negligible when measured against external pressure level.
Studies carried out on the roof enable identifying and better understanding its structural behaviour, emphasising the problems, and help to understand the reasons of the local failure occurred for a wind load value less than the usual value assumed for the design of these structures.

2 REFERENCES

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