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Wind induced response of a cable supported stadium roof

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Abstract

The first section of the paper describes the results of a wind tunnel investigation of the behaviour of wind of the cable supported roof of the "Stadio delle Alpi", Turin, Italy. The results presented and discussed include local and area-averaged pressures on the roof and the motions observed on a 1:200 aeroelastic model. It is shown that the major part of the loading is dynamic in nature and is due to vortices formed in the wake of the upstream sector of the roof and producing vertical velocity fluctuations in the flow approaching the leading edge of the downstream sector.

The second section of the paper is concerned with the difficulties of presenting the experimental data to the designer in a form which could be readily brought into the design and analysis process and to the discussion of improved experimental techniques which might surmount those difficulties in future studies of similar structures. The improved approach involves the use of a multi-point high speed pressure scanning system and the formation of a suitable set of load pattern time histories.

1. INTRODUCTION

Wind tunnel studies conducted for the cable supported roof systems of both the Turin and Rome stadiums are described in detail by Vickery, Steckley and Ho [1] and by Vickery et al [2]. The design of the two stadia are described in some detail by Majowiecki & Ossola {3} and by Majowiecki and Finzi [4]. Both stadia have an essentially elliptical planform and a very low aspect ratio in elevation. The test programs for the stadia were similar in nature and, in a qualitative sense, the observed behaviour was essentially the same. For this reason, attention is concentrated on the Turin study.

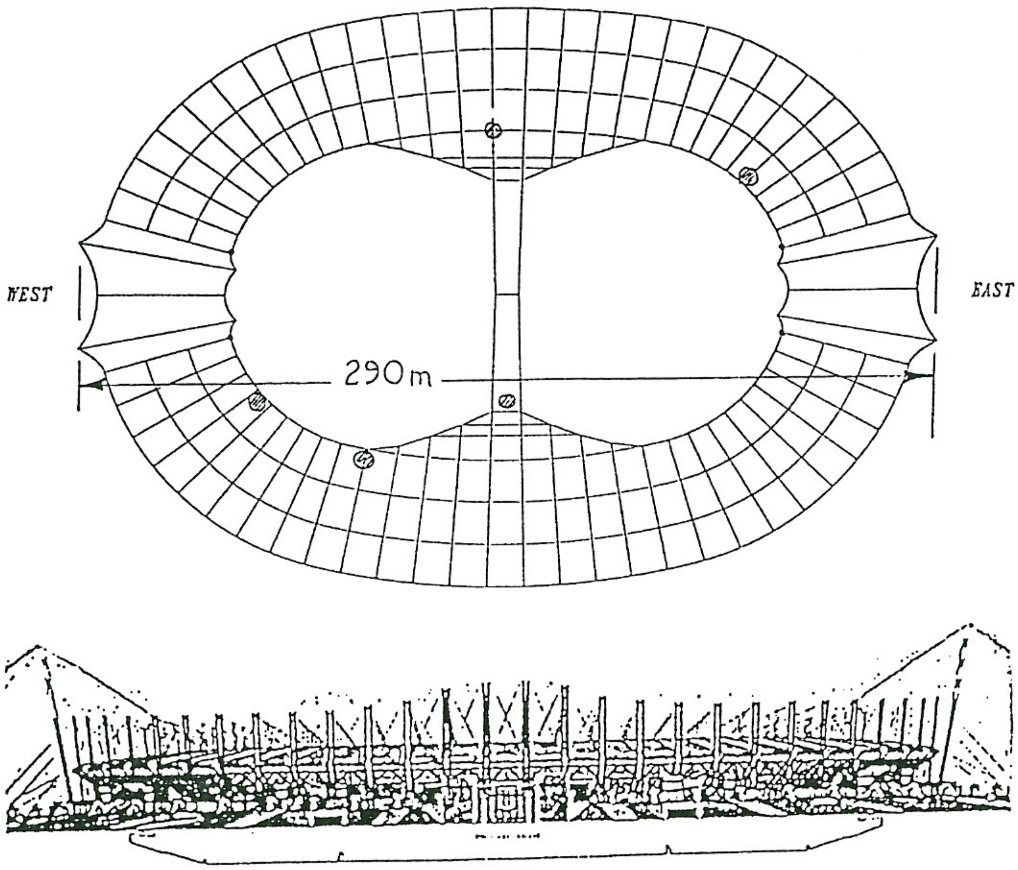

Fig. 1 Roof Plan and Stadium Elevation of the "Stadio delle Alpi" , Turin

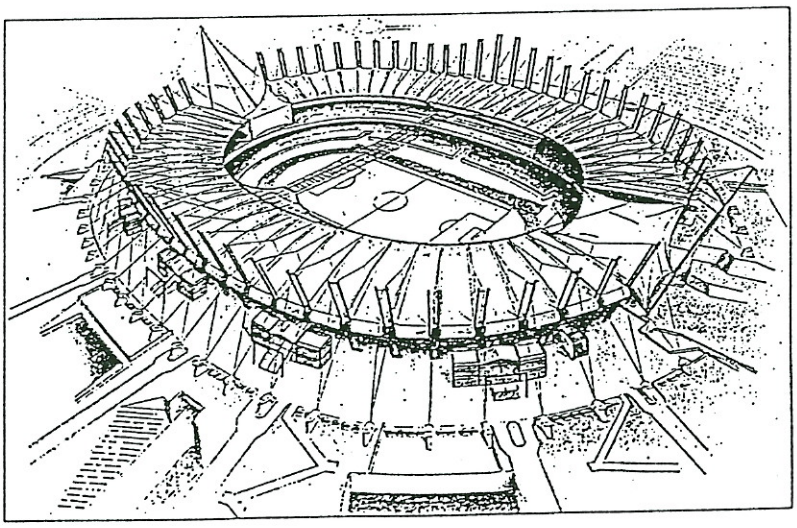

Fig. 2 Perspective View of Stadium

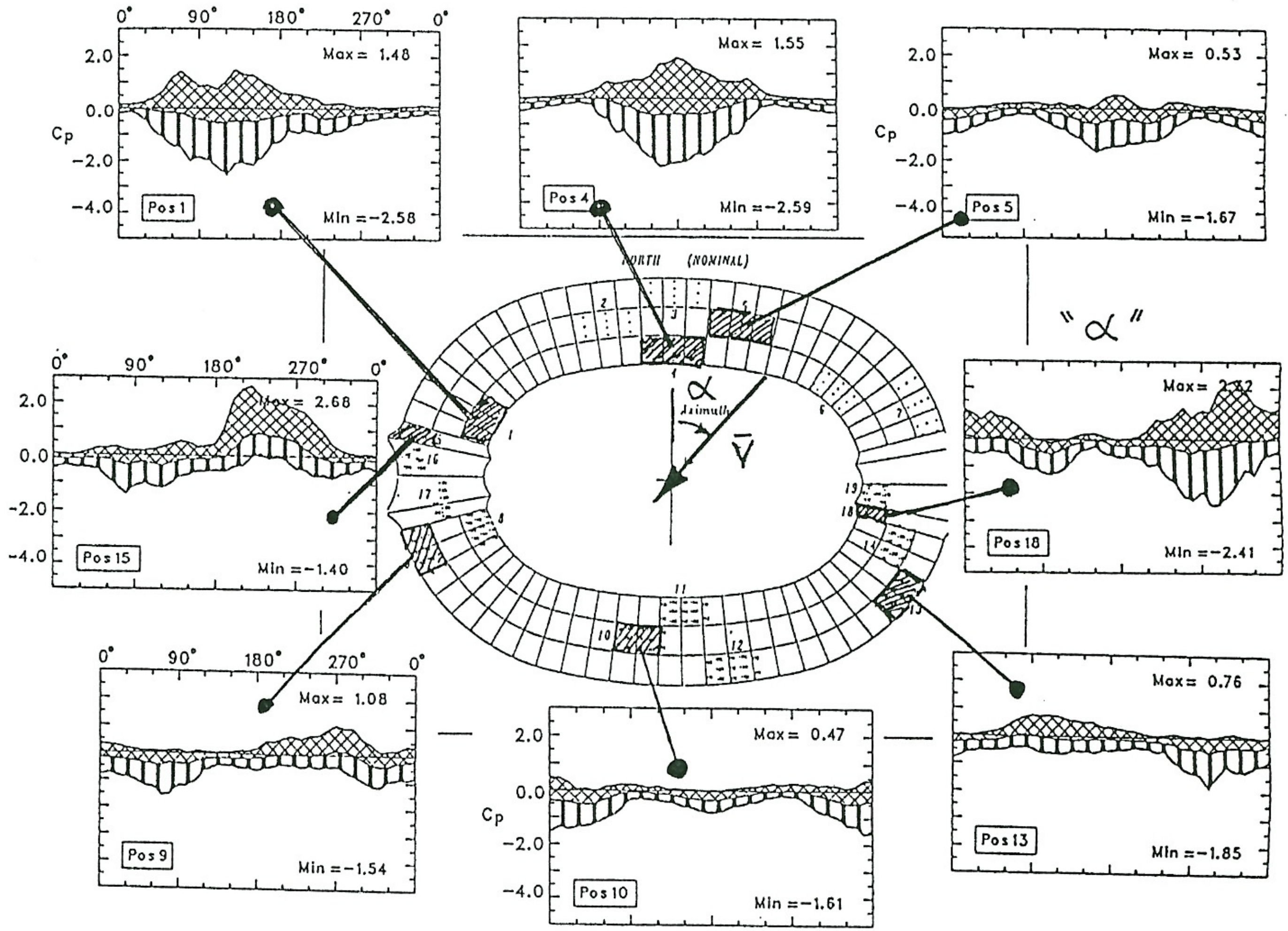

Fig. 3 Panel Loading as a Function of Wind Direction

The plan of the roof and the elevation of the stadium is shown in Fig. 1 and a perspective view is presented in Fig. 2. The cable trusses are tensioned by back-stays from the 56 towers and by the inner ring cable. The only mechanism for load transfer between the two sides of the stadium is through the diametric cables. The Rome structure is broadly similar but the cable tensions are transferred to an outer compressive ring truss rather than to ground.

The test program included an essentially rigid model on which pressures were measured and a 1:200 aeroelastic model constructed to match the Froude Numbers of model and prototype. The pressure study included local point pressures for the design of minor structural members and fixings and a set of panel pressures obtained by pneumatic averaging. Sixteen transducers were employed and this permitted the simultaneous measurement of the nett load (upper-lower surface loads) on eight panels and the corresponding matrix of spectra and cross-spectra. The measurements on the aeroelastic model were limited to the deflection at five locations and the tensions in two of the back-stays.

2. EXPERIMENTAL RESULTS

Selected panel loads are shown in Fig. 3. The loads are presented in coefficient form where;

$$C_p = (P_u - P_L) / \frac{1}{2}\rho V_R^2 . A$$

where; P_u = load on upper surface (+ve down)

P_L = load on lower surface (+ve up)

A = panel area

V_R = mean speed at a height of 30m in the approach flow.

The results presented in Fig. 3include the maximum and minimum values and the mean value which divides the two hatched areas. There are a number of features deserving of comment;

(i) the mean loads coefficients are generally small with maximum values of about 0.5.

(ii) the dynamic component is dominant and the peak load for the more important cases is five times the mean.

(iii) leading edge panels are the most heavily loaded and, very roughly, the load is proportional to the cosine of the angle between the wind and the normal to the leading edge.

The load spectrum for a more important load case is shown in Fig. 4. Velocity fluctuations in the approach flow would lead to the expectation of a spectral peak near 0.03 Hz but it is clear that the bulk of the variance is at much higher frequencies in a broad spectral peak centred on 0.16 Hz (with an approach speed of 30 m/s). This peak can be attributed to vertical velocity fluctuations produced by vortices shed from the upstream

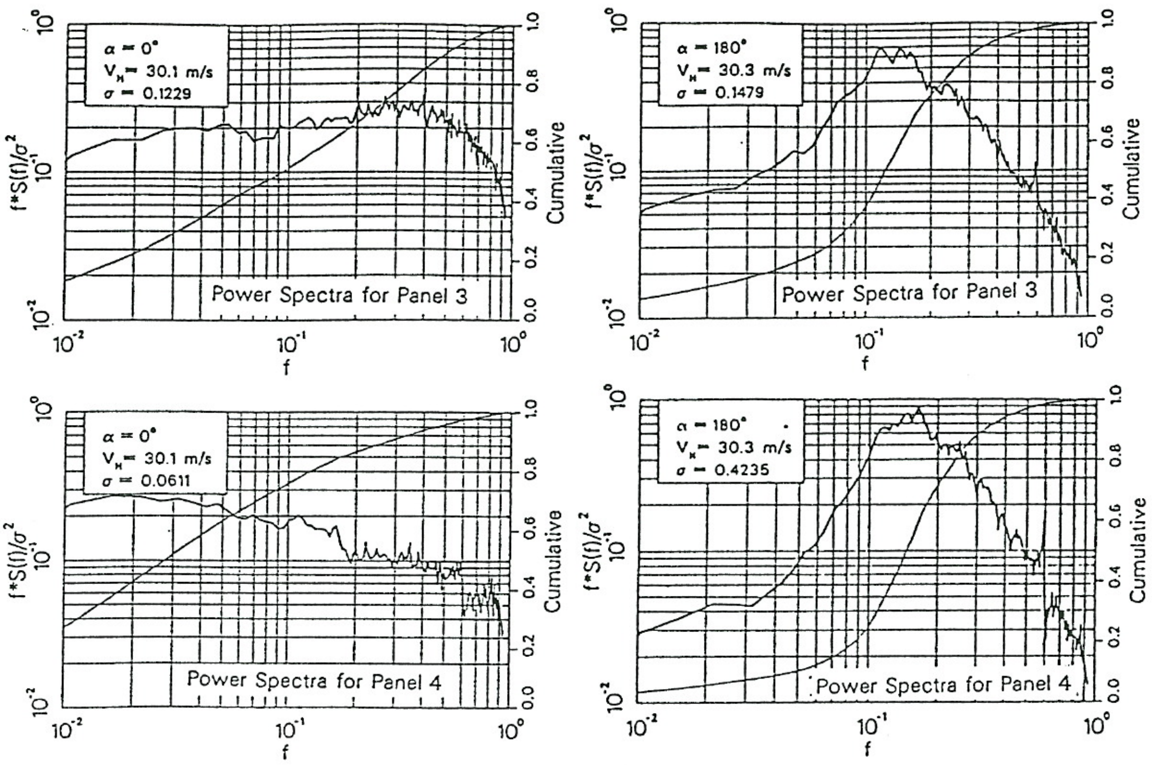

Fig. 4 Typical Spectra of Panel Loads

sector of the stadium. The Strouhal Number corresponding to the peak is;

$$S = f_s h / \bar{u}(h) \cong 0.12$$

where f_s is the central shedding frequency, h the roof height above upstream ground and *u (h)* the mean speed at this height. The lowest natural frequency of the roof is about 0.6 Hz and sufficiently removed from the peak to avoid excessive resonant response.

The spectra in Fig. 4 shows a discontinuity near $f = 0.5$ Hz. Although the pressure model was stiff it was very lightly damped sheet metal with a frequency corresponding to 0.5 Hz at prototype scale. The spectral distortion is due to motion induced pressures which, relative to the forcing pressures, change phase by 180° as the forcing component frequency is below or above resonance. The distortion is severe but limited to a very narrow frequency band and can be readily removed. The cumulative or integrated spectra also shown in Fig. 4 have no visible discontinuity at 0.5 Hz.

The correlation of panel loads is given by the cross-spectra density matrix shown pictorially in Fig. 5. Of note is the very weak correlation (root coherence), even between nearly adjacent panels, at the natural frequency. Near the shedding frequency the correlation is strong (typically 80%) over the most heavily loaded section of the roof ie; the central section containing panels 2,3,4 and 5. The cross-spectral matrix for a wind angle of 0°

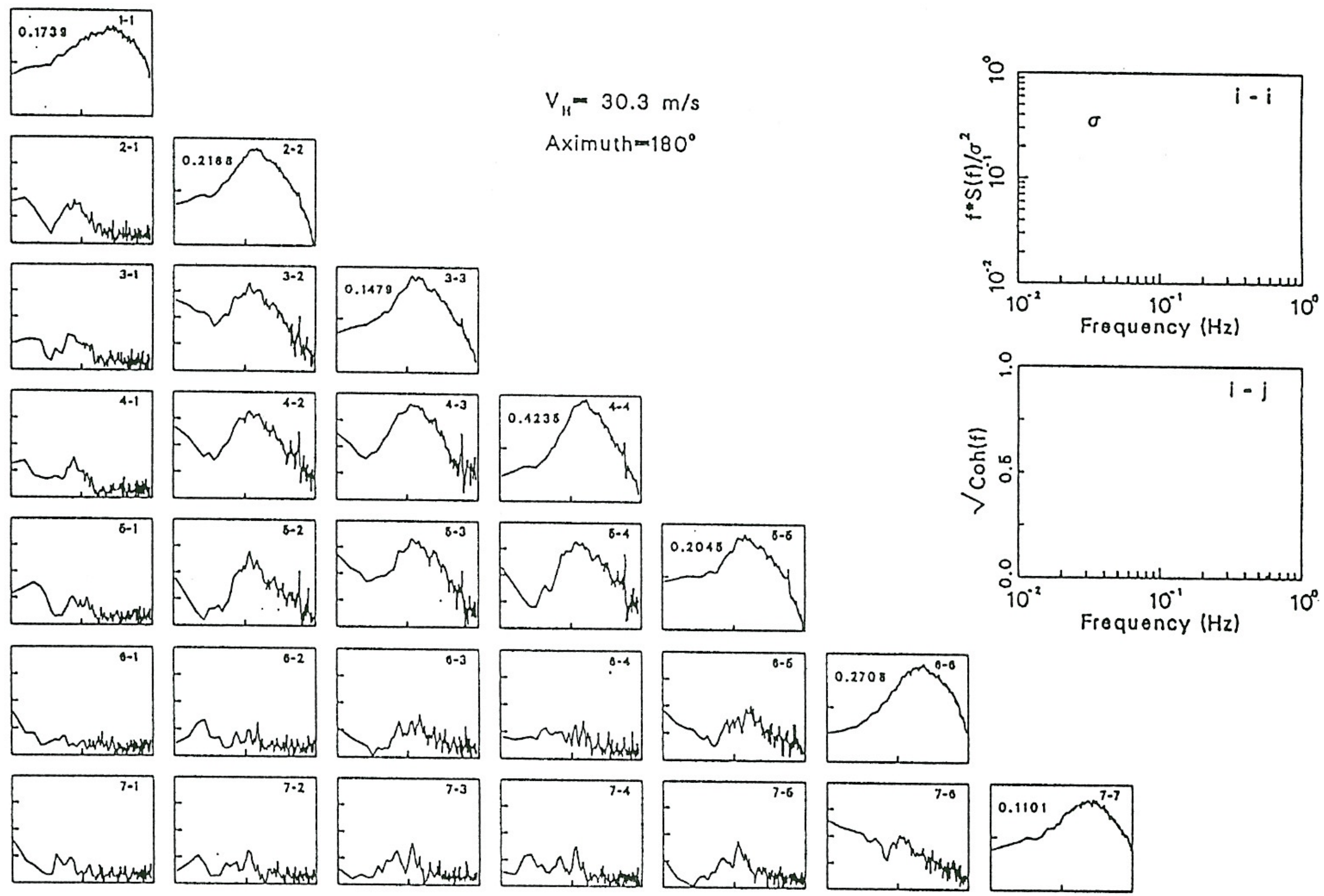

Fig. 5 Auto-spectra and Root-Coherence Functions of Panel Loads for a Reference Speed of 30 m/s and a Wind Angle of 180°

(instrumented roof upstream) is shown in Fig. 6. The influence of shedding is less pronounced in this case and the correlation over the central part of the roof is significantly weaker; typically 50% for panels 2,3,4 and 5.

The results from the aeroelastic test are shown in Figs. 7 and 8. Fig. 7 shows the roof deflections as a function of wind direction while typical spectra are presented in Fig. 8. The deflections mirror the panel loads with small mean values and large dynamic motions when the wind is normal to the leading edge of the roof. For the largest deflections, eg; location 4 @ 180°, the bulk of variance is associated with shedding with a comparatively minor contribution due to resonance at the natural frequency. The resonant response is far more significant (relative to the total variance) when the upstream section of the roof (eg; locations 3 and 5 @ 180°) but is nevertheless very much smaller than the vortex forced motions when the segment is on the downstream part of the roof.

3. TEST DATA AND THE DESIGN PROCESS

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 basis 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 panesl 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.

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 used as input to the sophisticated dynamic numerical model developed by the designer and lead to discussions between the designer and the wind tunnel researchers to examine alternate techniques that might be used in future projects. The discussions centred on the use of high speed pressure scanning systems capable of producing essentially simultaneous pressure measurements at some 500 points at rates of perhaps 200 Hz per point. With such a system it would be possible to cover in excess of 200 panels and produce a complete description of the load. Such a system would produce roughly 1 to 2 x 10^6 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;

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

where $p(x,y,t)$ = nett load per unit area at position (x,y)

$\phi_j(x,y)$ = a weighting function

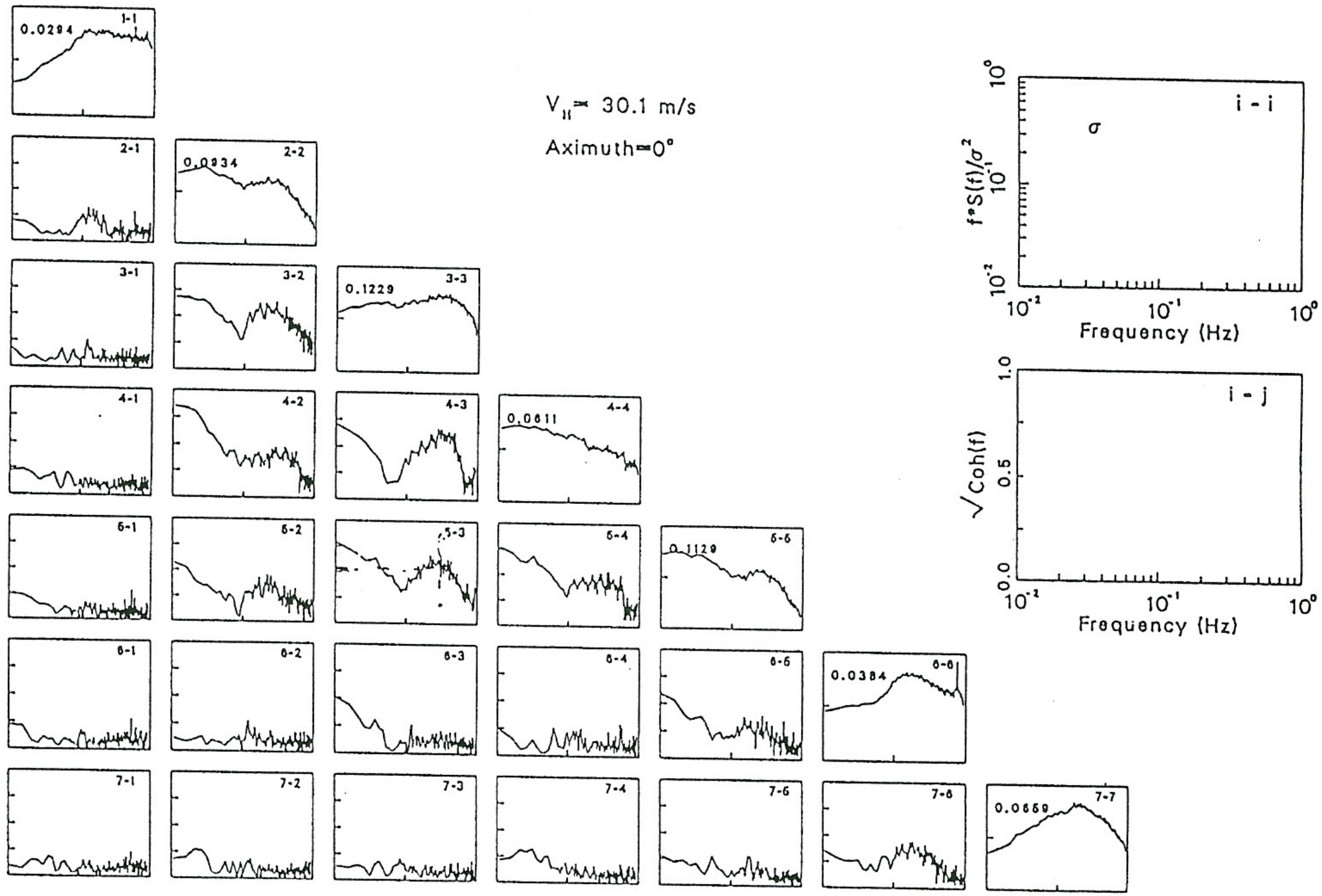

Fig. 6 Auto-spectra and Root-Coherence, Functions of Panel Loads for a Reference Speed of 30 m/s and a Wind Angle of 0°

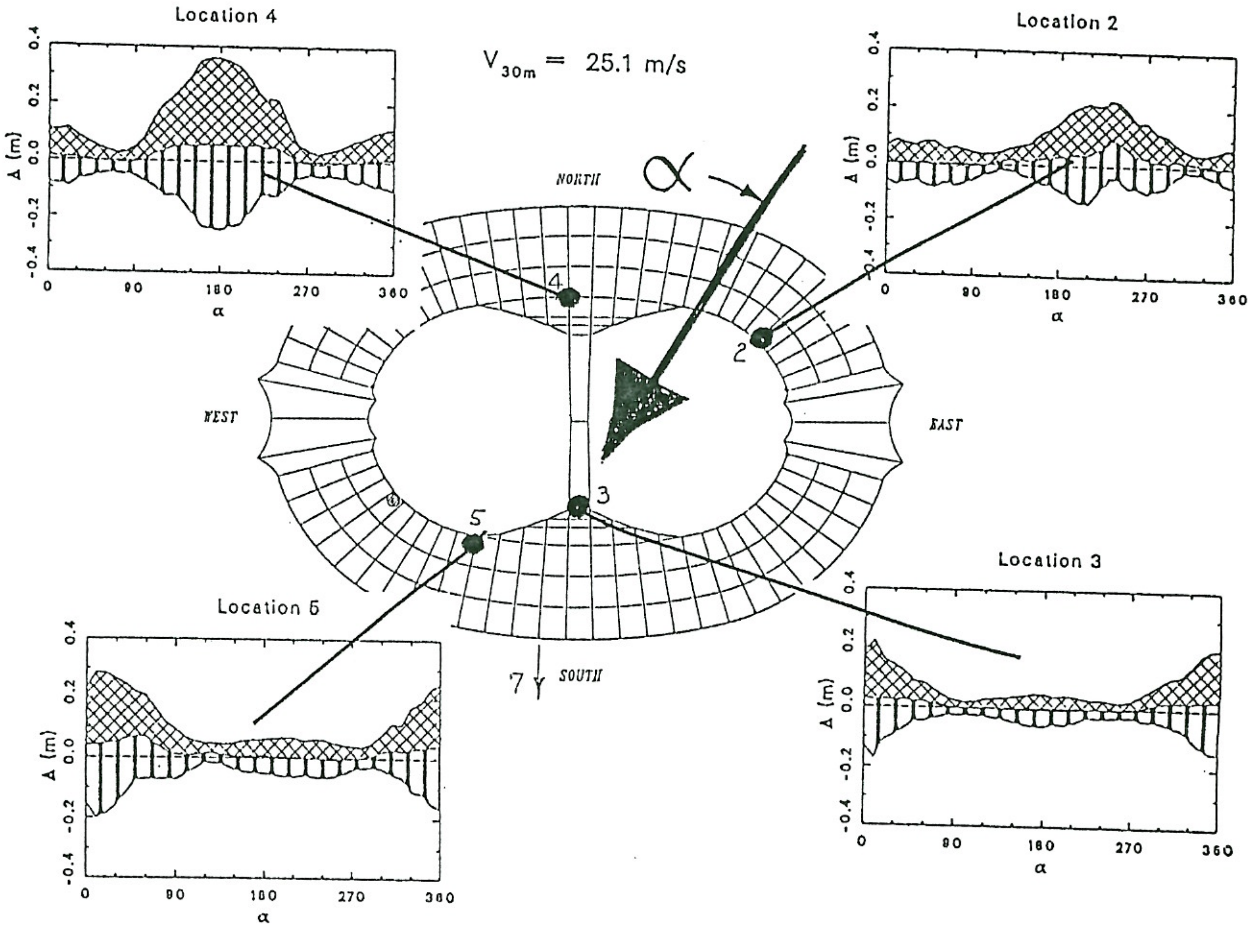

Fig. 7 Roof Deflections as a Function of Wind Angle

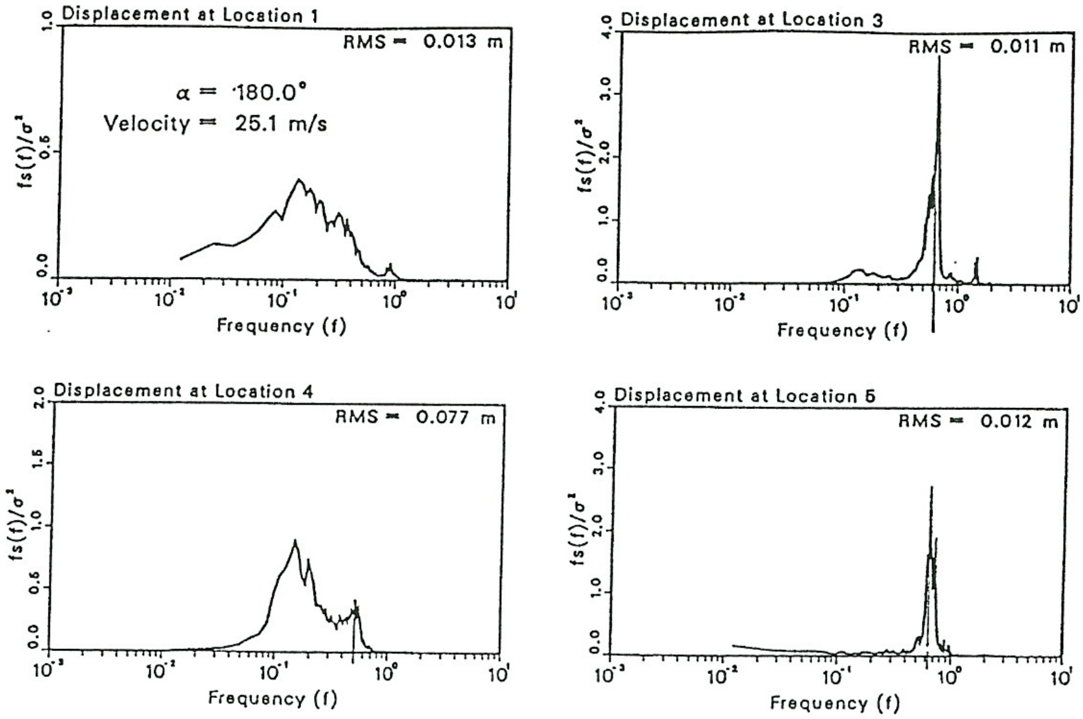

Fig. 8 Selected Spectra of Roof Deflections

For a series of pressure taps the approximation to ϕ_j (t) would be;

$$Q_j(t) = \sum_{i=1}^{N} \bar{p}_i(\bar{x}_i\bar{y}_it).A_i\phi_j(\bar{x}_i,\bar{y}_i)$$

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, ϕ_j *(t)*, is discussed in the following section.

4. THE MEASUREMENT AND USE OF LOAD TIME HISTORIES

If the weighting functions, ϕ_j, are chosen as mode shapes then ϕ_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 can be estimated when the design is more advanced. In such a case we can approximate ψ_j as:

$$\psi_j \cong \psi_j^1 = \sum_1^M a_{ij}\phi_i$$

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

$$\frac{\partial}{\partial a_{ij}} \cdot \int (\psi_j - \sum_i a_{ij}\phi_i)^2 \, dA = 0; i = 1, M$$

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; $a_{ij} = \dfrac{\int \phi_i \phi_j dA}{\int \phi_i^2 dA}$

For a finite panel sizes the corresponding relationship is

$$a_{ij} = \frac{\sum_k \phi_i(\bar{x}_k, \bar{y}_k)\phi_j(\bar{x}_k, \bar{y}_k)A_k}{\sum_k \phi_i^2(\bar{x}_k, \bar{y}_k)A_k} \quad \text{where,} \quad \sum_k \phi_i(\bar{x}_k, \bar{y}_k).\phi_j(\bar{x}_k, \bar{y}_k).A_k = 0 \quad fori \neq j$$

The experiment would involve the recording of the local histories ψ_j (t) from which the modal time histories could be constructed and the analysis conduced in either the time or frequency domain. 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. In such a case the time domain solution which preserves the extreme value distribution is to be preferred over a frequency domain approach.

Questions which must be faced in the design of the experiment include the spacing of individual taps, the panel shape and size and the number of shape functions to be employed. The question of tap spacing has been addressed by Letchford [5]. The tap spacing is determined primarily by the wavelength $\lambda_m = u/f_m$ where f_m is the maximum frequency of interest; for an adequate estimate of the load spectrum at f_m the spacing should not exceed the correlation length associated with pressure field components with a wavelength λ_m and hence a spacing of about $\lambda_m/10$ is appropriate.

The choice of panel size and the number of shape functions is dependent upon the rapidity of the spatial rate of change of pressures over the roof and the spatial rate of change of the influence surface for the particular structural action under consideration. The panel size problem is analagous to estimating the errors involved in numerical integration by the "trapezoidal rule"; these depend primarily on the magnitude of the second derivatives of the integrand.

Selection of panel size and the number of functions requires some prior knowledge of general nature of the pressure field and of the structural actions in the roof and a large measure of judgement. In the present case, a study of the measured pressures and the influence surfaces for major structural actions (eg; the tension in a backstay) leads to the conclusion that a satisfactory arrangement would involve (on one half of the roof) a total of about 500 uniformly distributed taps over about 100 panels with some reduction if the number of taps are reduced and/or the panel size increased near the fixed boundaries. The required number of shape functions is estimated as three radially and five circumferentially or 15 in total. Thus, the experimental data would be made available as fifteen time histories. in contrast to the 500 measuring points. This data condensation yields a manageable package that can be readily accepted into the design and analysis process.

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