DYNAMIC ASPECTS OF THE NEW BRAGA STADIUM LARGE SPAN ROOF

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Summary

The wind induced structural response is sensitive to the modal shapes, frequencies and, especially, damping; hence, the knowledge of the actual dynamic properties becomes essential to confirm the design assumptions, based on theoretical calculations and wind tunnel tests, and/or to introduce mitigation devices. In order assess the dynamic behavior of the New Braga Stadium large span suspension roof, the dynamic characterization of the real structure has been performed. Two test types were carried out: classical sudden load release, used to identify the frequencies of the first vibration modes, and harmonic excitation, at different structure points, with subsequent measurement of the free vibration decay. The low values of the first modal frequencies required the set up of a special excitation system, capable to induce appreciable structural oscillations at very low frequencies. The test results were analyzed to recognize the modal shapes and frequencies, by comparing them with the theoretical ones, and to determine the inherent structural damping of the excited vibration modes. The dynamic testing is aimed to improve the reliability of the entire design process and to calibrate a external damping system. In fact, the analysis of wind effects showed that (due to the very low inherent structural damping ratio, as measured by the cited tests) the wind induced vibrations can reach very high amplitudes. Hence, it will be opportune to add an external damping system which guaranties an acceptable level of wind induced oscillations. In this paper, a simplified method to evaluate the optimal properties of such a damping system is also presented and the responses of the structure with and without the dampers are compared.

Keywords: long span roof; wind induced vibration; dynamic tests; dynamic characterization; non-classical damping; damping optimization; proper orthogonal decomposition; monitoring systems.

1. Introduction

This paper describes the experimental dynamic investigation carried out on the new Braga (Portugal) stadium roof. The dynamic characterization was required by the structural complexity, increased by the high flexibility of the suspension system, and by the uncertainties of some structural parameters, as the structural damping of the complex composite system.

The testing program on the complete roof structure is part of a design and verification strategy, according to the Eurocode - Basis of Structural Design - that gives general indications about the Design Assisted by Testing; in the specific case, the full scale testing has been performed to reduce the uncertainties on the dynamic damping parameters and to check the vibration mode shapes and frequencies, as indicated in the Annex D of the cited Eurocode.

The experimental evaluation of the modal damping ratio values - which are difficult to be foreseen in absence of full scale data on very similar structures - allows to increase the reliability of the theoretically calculated response to dynamic loads and, in particular to the wind action. In addition, the measured natural modal shapes and frequencies allow to validate and to calibrate the numerical models which have been arranged during the design stage, for their use during the monitoring and the maintenance phases. This is an essential step in the evaluation of the structural subsequent performances.

The high flexibility and the large roof mass, consisting of two huge concrete slabs supported by suspension cables 202 m free span length, result in low frequency values for the first vibration modes. Hence, the structure testing, under harmonic excitation, required the setup of a special excitation system, capable to induce appreciable oscillations of the structure in the range 0,2 to 1,0 Hz.

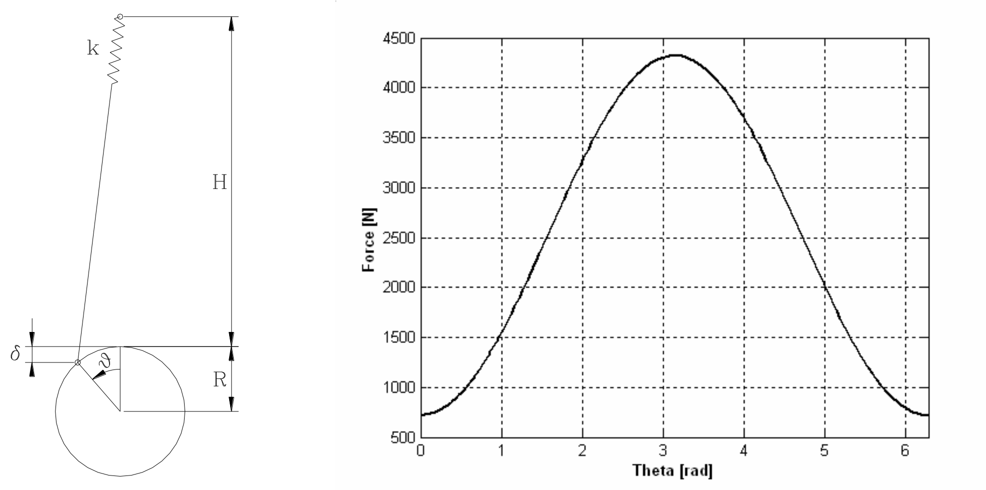

Figure 1. Scheme of the harmonic exciting system (a); spring force vs engine angular position (b).

The tests have been performed in May (Test Series 1) and July 2004 (Test Series 2). They substantially consisted in measuring the dynamic response of the structure to impulsive and to harmonic loads. The response was measured in terms of acceleration in some significant points of the structure itself. The recorded accelerations were analyzed to recognize the excited modal shapes and to determine the corresponding modal damping ratios. The experimentally recognized modal shapes are compared to the numerical model derived ones. The ascertained values of the damping ratios - whose magnitude was variable, depending also on the stiffness type of the excited mode - have been used to confirm or to review the response predictions derived from numerical analyses and wind tunnel tests [1-6].

The analysis of wind effects on the roof (as performed during the design stage - with an assumed damping ξ = 1%) showed that the wind induced vibrations can reach very high amplitudes. As it will be shown, the measured damping is much lower than 1%; thus, vibration are even larger. Since vibration amplitudes are very sensitive to the inherent structural damping ratio (approximately inversely proportional to $\sqrt{\xi}$), it seems important to add a damping system which guaranties an acceptable level of wind induced oscillations. In the following, the optimal properties of such a damping system have been preliminary evaluated by mean of a simplified but powerful method.

2. Experimental setup

2.1 Exciting system

Two different exciting systems were used. During the Test Series 1, the structure was excited by impulsive loads, by releasing an approximately 5 tons mass, preliminary suspended to the roof edge steel girder by mean of a cable.

Within the Test Series 2, the structure was excited by periodic, approximately harmonic loads, at different frequencies. The forces were provided by a cable, linked to the edge steel girder by mean of a pre-tensioned spring, sinusoidally moved at the other end. Since the spring was sufficiently flexible, the roof movement was negligible in comparison to the cable one; hence, the force was nearly sinusoidal, as shown in Figure 1. The cable movement was provided by an electric engine, through an eccentric link (Fig. 2).

The actual induced force was measured by mean of a force transducer located between the roof girder and the spring. The force transducer output was sampled and recorded at 10 Hz.

In order to excite the first significant vibration modes, both the impulsive and the harmonic loads were applied in two different significant points of the structure: an inner roof corner and a roof border steel girder mid span. The exciting positions are schematically shown in Figure 3, where they are named as EXC A and EXC B respectively.

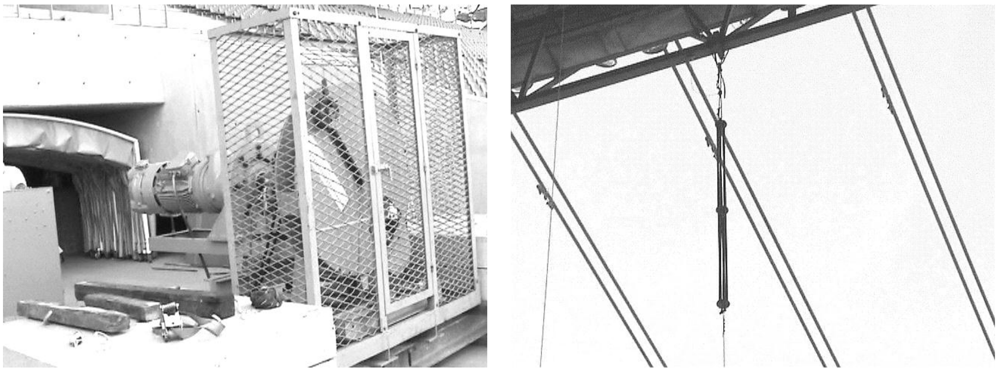

Figure 2. View of the electric engine with the eccentric link (a) and of the girder linked spring (b).

2.2 Measuring arrangement

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. The sensor positions and their order number (ACC 1 to ACC 6) are schematically shown in Figure 3.

The accelerometers are part of the structural monitoring system which is installed in the stadium. They are Tri-axial Force Balance Accelerometer type, consisting in a mass-spring resonator with optical mass pickoff sensing, working within a frequency range of 0 to 60 Hz.

The vertical acceleration component only has been taken into account, in the calculations for the present work. In fact, it is sufficient to describe the behavior of the more flexible vibration modes, which are substantially the wind excited ones, the vertical component being prevailing for these modes. The accelerations at the 6 sensors were simultaneously acquired and recorded during the tests at 250 Hz frequency sampling.

3. Testing program

3.1 Impulsive tests

During the Test Series 1 the structure was excited by impulsive loads, by releasing an approximately 5 tons mass preliminary suspended to the roof. The main purpose of these tests was to identify the main natural frequencies of the structure.

This first analysis allowed to simplify the successive harmonic excitation, which could be focused around the recognized natural frequencies. On the other hand, the impulsive force had to be - for safety reasons - relatively small (if compared to the involved inertial forces). Hence, the relative structural response was not sufficiently wide and clear to be used for the modal shapes and damping identification. The Figure 4 shows a typical acceleration record and the corresponding identified frequencies for one of the impulsive tests.

3.2 Harmonic loading tests

Once the main natural frequencies of the roof were identified by mean of the Test Series 1, the harmonic excitations were focused around those frequencies. The purpose of this second test series (namely the Test Series 2) was to significantly excite the main natural vibration modes, by mean of resonant harmonic forces. At this purpose, different excitation frequencies were tried, close to the preliminary identified ones, until the best fitting of the resonant frequency was reached for each mode. Figure 5 shows a summary of the main forcing and response parameters involved in this test series. The plot of the A/F values is shown to point out the resonant conditions; Freq is the forcing frequency (Hz); the ratio A/F is representative of the dynamic magnification due to the resonant effects, being A the roof acceleration half-amplitude (g) in correspondence of the exciting system, at the frequency Freq and F the force half-amplitude (kN) at the same frequency (after a frequency filtering which isolates the only components around Freq).

In order to have a complete information, the free vibrations after the harmonic load stop were also recorded. All the time series have been preliminary frequency filtered to retain the interesting frequencies (assumed between 0.05 and 30.00 Hz, in the present case) and to cut the other ones, which are affected by the sensor distortions and by different noises.

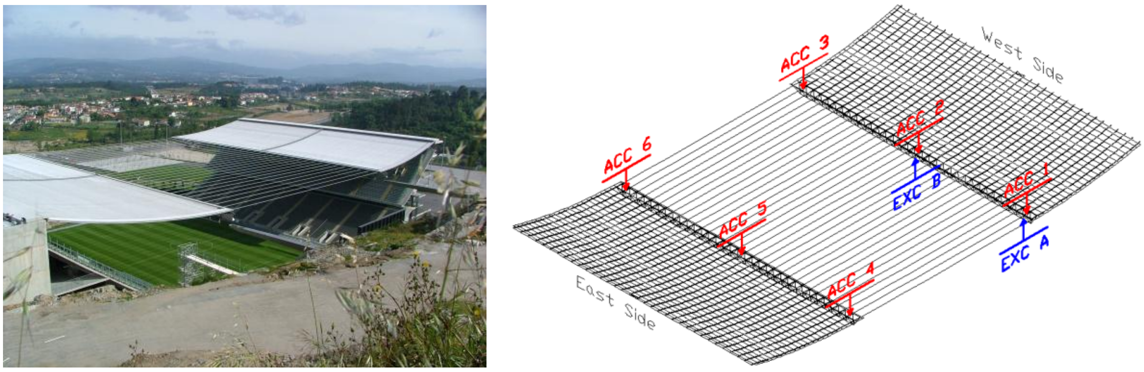

Figure 3. Stadium roof view (a); accelerometers and exciting system position (b).

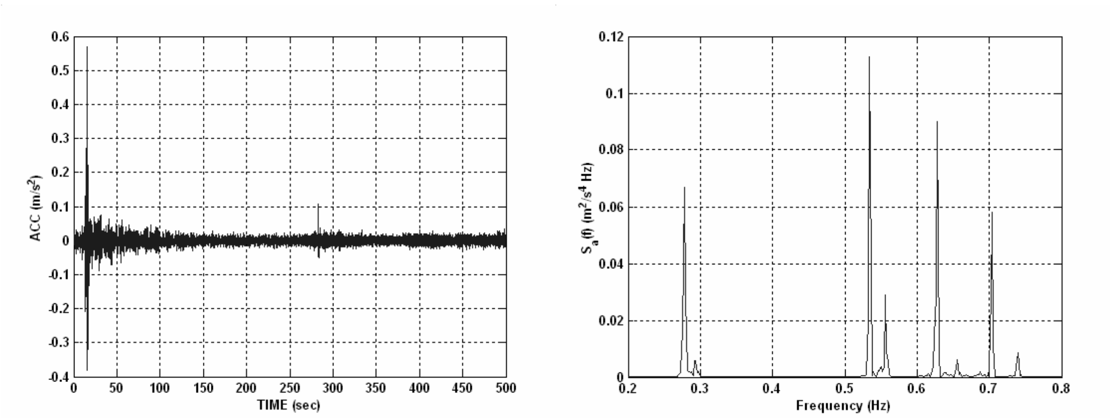

Figure 4. Acceleration time history (a) and PDSF (b): impulse in EXC B - sensor ACC 3.

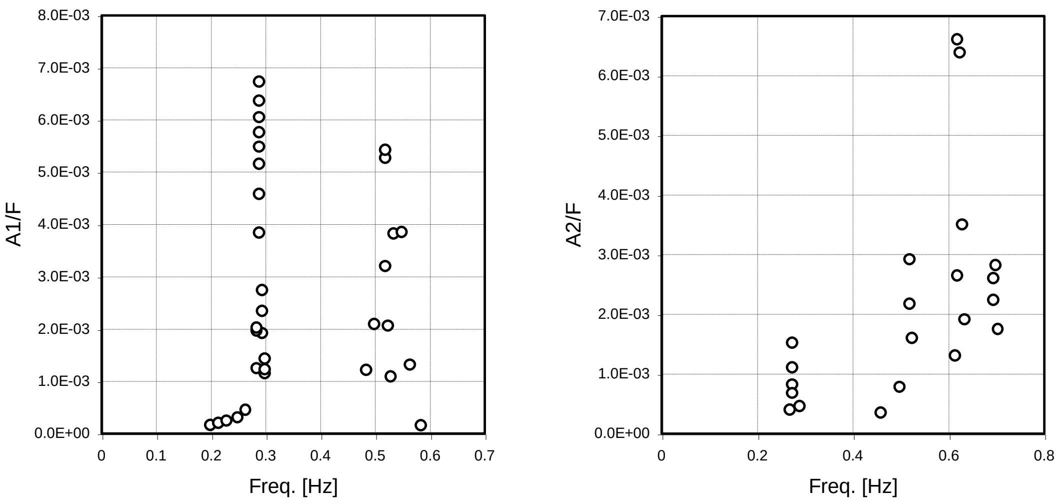

Figure 5. Dynamic response ratio A/F vs excitation frequency: force in EXC A (a) and EXC B (b).

4. The dynamic characterization

4.1 The identification methodology

The free vibration decay time series, recorded after the resonant harmonic excitation stop, are among the most appropriate data to evaluate the natural modal shapes and the corresponding modal damping. For this purpose, the measured acceleration fields have been numerically treated. Firstly, a frequency filtering has been performed to isolate the frequency range involved by the given vibration mode. Then, the obtained signals have been projected on the covariance matrix eigenvector generated space (the POD - Proper Orthogonal Decomposition - has been performed).

This procedure points out the dominant modal shape, within the considered frequency range, and the corresponding time history. This latter, whenever it includes free vibration intervals, allows to evaluate the logarithmic decrement and, thus, to determine the modal damping ratio. In addition, if the free vibration time history is sufficiently long to follow the motion from large to small amplitudes, the logarithmic decrement can be evaluated within different intervals and the damping ratio can be separately obtained for different vibration amplitudes.

4.2 The identified dynamic properties

Figure 6 shows the results of the above described analysis and, in particular: (a) the modal acceleration time history within the free vibration interval, with an indication of the natural frequency - including an outline of the logarithmic decrement curve for intermediate vibration amplitudes; (b) the theoretical corresponding mode (from the same FEM model as the one used to evaluate the wind induced response).

Similarly, different main modes have been experimentally identified. Due to the limited number of excitation positions, other modes have not been excited during the test campaign. In addition, some of the theoretical modal frequencies are very close each to one other: they are substantially dual modes and the excited one depends on and is very sensitive to the excitation position. On the other hand, the identified modes are sufficient to characterize the dynamic behavior of the structure in terms of modal damping ratio and modal shapes and frequencies to be compared to the theoretical ones. Table 1 resumes the main features of the identified modes.

The comparison between the theoretical and the experimentally recognized mode shapes and frequencies, pointed out that the anti-symmetric modes (with reference to a cable plane section) are very accurately represented by the FEM model, while the symmetric ones are lightly stiffer than the theoretical predicted ones and the respective frequencies are slightly higher.

It seems important to observe that the symmetric modes heavily involve the cables elongation, the stiffness of the substructure and the geometric stiffness, while the anti-symmetric ones are substantially determined by the geometric stiffness matrix only.

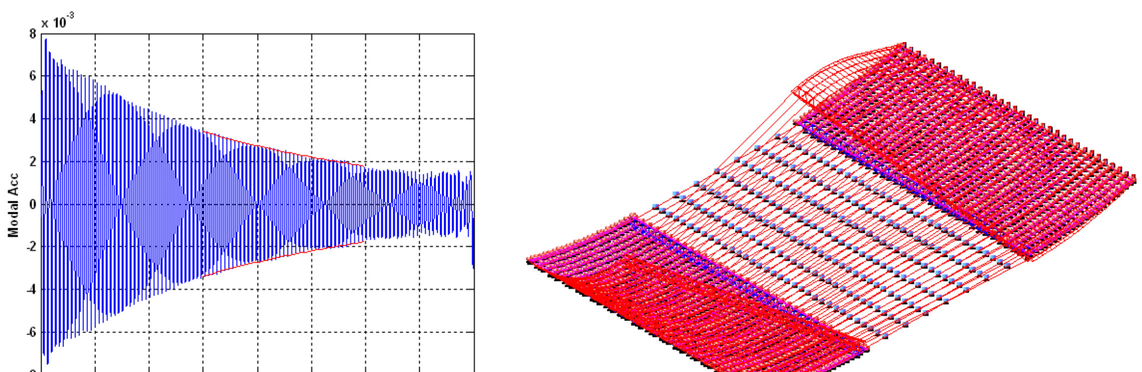

Figure 6. First mode dynamic properties. Test n. EXCB06; experimental frequency f_{exp} = 0,275 Hz; experimental damping ratios: $\xi_{large\ amplitudes}$ = 3,0‰ - $\xi_{mean\ amplitudes}$ = 2,5‰ - $\xi_{small\ amplitudes}$ = 2,3‰; FEM theoretical frequency f_{theor} = 0,274 Hz.

Table 1. Dynamic properties of the main identified structural vibration modes.

Test #	Exp. freq. [Hz]	$\xi_{\text{large amplitudes}}$ [‰]	$\xi_{\text{mean amplitudes}}$ [‰]	$\xi_{\text{small amplitudes}}$ [‰]	FEM mode #	FEM theor. freq. [Hz]
EXCB06	0.275	3.0	2.5	2.3	1	0.274
EXCA22	0.291	3.2	2.5	1.8	2	0.288
EXCA33	0.519	2.7	2.1	2.0	3	0.399
EXCB17	0.627	4.5	3.7	3.1	5	0.540
EXCB21	0.698	2.2	1.6	1.3	10	0.678

Regarding the determined modal damping ratios, it can be observed that, as it can be expected, for large oscillation amplitudes the damping values are higher than for small amplitudes. The largest excited amplitudes give rise to damping ratios of 3‰, as order of magnitude, for approximately all the excited modes.

The modes which involve the bending deformation of the steel border girder, partially collaborating with the concrete slab, are characterized by slightly higher values of the damping ratio. These modes are quite stiff and, hence, they are not particularly significant in determining the wind induced response.

On the other hand, opposite to the expected behavior, the flexural deformation of the reinforced concrete slab does not increase the damping value; this is probably due to the small thickness-width ratio (smaller than 5/1000 in the shorter direction) and the subsequent relatively small deformations of the concrete slab, when involved in roof oscillations.

5. The dampers location

The presence of a stiff beam at the inner borders of the roof (from points 1 to 3 and 4 to 6 in Figure 7a) induces the firsts natural modes, which are the most excited by the wind turbulence, to have the largest amplitudes at the ends of the beams themselves. Thus, the beam ends (points 1, 3, 4 and 6 of Figure 7a) are among the most quoted to locate external dampers. These are also among the few locations compatible with the architectural requirements.

Hence, the analyzed damping system consists of four linear viscous dampers located at the inner vertices of the roof; in Figure 7, C represents the damping coefficients of such a dampers. As a matter of fact, the dampers will be located at the ground level and connected to the roof by mean of tensile strands. Thus, a mass or a spring will be necessary to avoid compression in the cables. Nevertheless, in the present preliminary analysis the only presence of a simple linear viscous damper has been considered, without taking into account the added mass or stiffness, the mean value of the cable tensile force, etc.

A deeper stage of the design should evaluate the specific problems as, for instance, the opportunity to realize a friction damping system rather than a viscous one, the devices to avoid the cable slackening, the support induced cable excitation, etc.

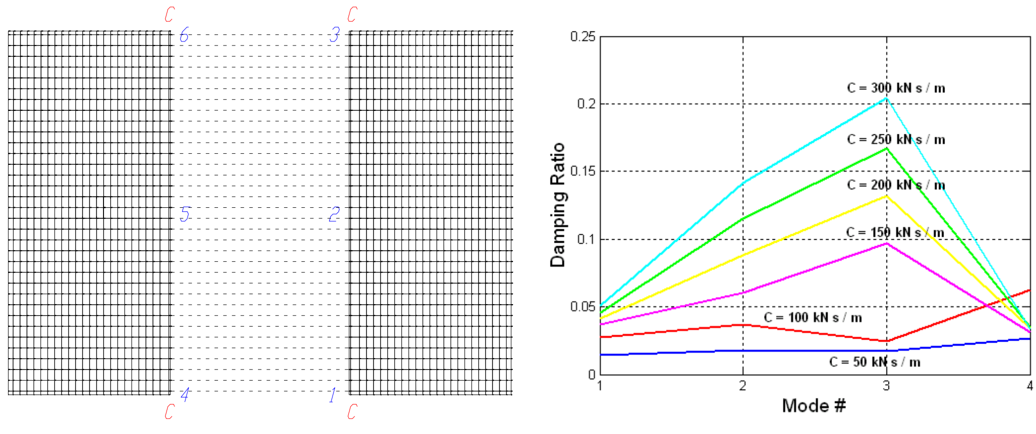

Figure 7. Dampers location and monitored points (a); damping ratio of complex modes (b).

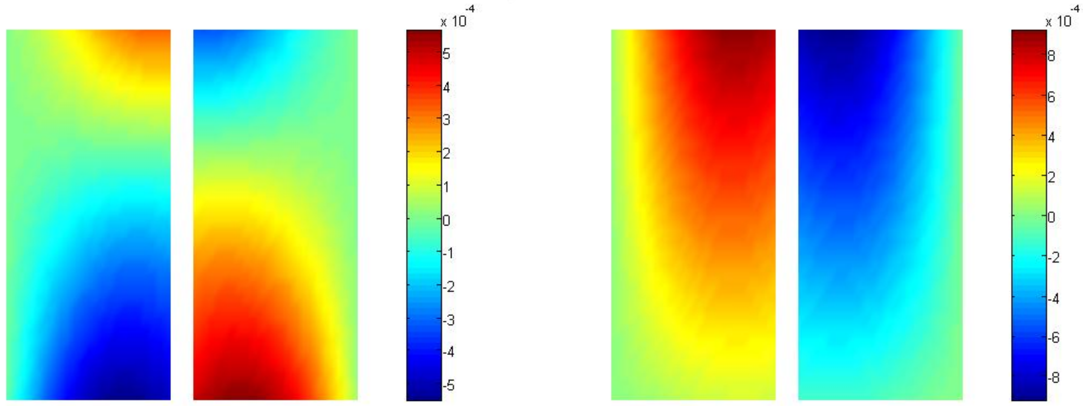

Figure 8. Real and imaginary parts of the 1st complex mode eigenvector - f_1=0,28 Hz - ξ_1=3,7%.

6. Estimation of the added damping ratio through complex eigenvalue analysis

A preliminary estimation of the damping system efficiency has been carried out by mean of the eigenvalue analysis of the non-classical damped system [7]. The eigenproblem is formulated by considering the first 10 modes of the undamped structure and the 4 added linear viscous dampers. The first 4 complex modes have been analyzed (the upper ones requiring a more refined model). Figure 7b shows the damping ratio associated to such modes, for different values of the damping coefficient. Figure 8 shows the real and the imaginary parts of the 1st eigenvector.

The results obtained by this simplified procedure have been confirmed by some time history analysis of the full system (without modal projection) subjected to impulsive loads.

7. Effects of the damping system on the wind induced vibrations

In order to evaluate the realistic behavior of the damping system and the maximum force produced by the dampers, different time history analysis of the structure subjected to the pressure fields measured in the wind tunnel have been carried out. Different external damping coefficients (C=0 to 300 kN s/m) and inherent modal damping ratios (ξ_i=0 to 10‰ associated to the basic classical modes) have been considered. Table 2 resumes the main results in terms of vertical displacement stochastic properties and dampers “working” force (wind pressure time series are derived from RWDI wind tunnel tests at 270° from the true north).

It can be observed that a significant reduction of the oscillation amplitudes (see standard deviations) can be obtained by using viscous dampers with a damping coefficient C=150 kN s/m. The subsequent damping force will reach an amplitude of about 60 kN, as order of magnitude.

Table 2. Vertical displacement stochastic properties, with and without external dampers.

	ξ_i = 3 ‰ - C = 0				ξ_i = 3 ‰ - C = 150 kN s/m				
	z_{min} [cm]	z_{max} [cm]	μ_z [cm]	σ_z [cm]	z_{min} [cm]	z_{max} [cm]	μ_z [cm]	σ_z [cm]	F_d [kN]
P1	-62	76	-6	20	-24	10	-6	4.9	56
P2	-26	34	-10	12	-26	2	-10	3.9	0
P3	-66	60	-15	29	-38	3	-15	6.8	51
P4	-55	57	2	19	-11	14	2	3.8	41
P5	-25	33	4	10	-4	13	4	2.7	0
P6	-62	81	8	30	-13	31	8	6	62

8. Conclusions

Different vibration modes have been experimentally identified. Since they involve all the main de-formation mechanisms of the structure, they are sufficient to characterize the dynamic behavior of the structure and its damping properties, particularly for the wind action excited modes.

The comparison between the theoretical and the experimentally recognized mode shapes and frequencies, points out that the anti-symmetric modes are very well represented by the FEM model, while the symmetric ones are lightly stiffer than the theoretical predicted ones (the sym-metric modes involve the cables elongation, the restraint and the geometric stiffness while the anti-symmetric ones are substantially determined by the geometric stiffness matrix).

The measured modal damping ratios were found to be about 3‰, as order of magnitude, for approximately all the excited modes and, in particular, for the more flexible ones, which are the major responsible of the wind induced resonant response.

The measured damping ratios, together with the confirmation of the main modal frequencies and shapes, involved in the wind loading dynamics, have then been used to review the analysis based on numerical models, with improved confidence in the results of the design process.

Since the actual inherent damping ratio is not sufficient to guaranty an acceptable level of wind induced oscillations, it seems opportune the realization of an external damping system. In the present paper, a preliminary evaluation of the damping system features has been presented.

The proposed damping system consists of four linear viscous dampers located at the inner vertices of the roof. The dampers will be located at the ground level and connected to the roof by mean of tensile strands. In the present preliminary analysis, the only presence of a simple linear viscous damper has been considered, without taking into account added masses or stiffness, the mean value of the cable tensile forces, etc. A deeper stage of the design should evaluate the specific problems as, for instance, the opportunity to consider a friction damping system rather than a viscous one, the devices to avoid the cable slackening, the support induced cable excitation, etc.

The analysis showed that an additional damping ratio of 4 to 10% on the fist three complex modes and a significant reduction of the oscillation amplitudes on the first non-classical modes can be obtained by using viscous dampers with a damping coefficient C=150 kN s/m. The subsequent damping force will reach an amplitude of about 60 kN, as order of magnitude (the wind pressures being derived by the RWDI wind tunnel tests).

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