

## Design Assisted By Wind Tunnel Testing

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### Summary

According to principle of the “design assisted by testing”, contemplated by the Eurocode, the combination of tests and calculations is increasingly used. The motivation of this increasing interest-demand are various: from the sophisticated architectural shapes to the absence of corresponding reliable fluid-dynamics models, from the increasing flexibility of structures to the necessity of checking complex aeroelastic phenomena.

The purpose of the paper is to outline the potentiality of wind tunnel testing and the main problems encountered in their use. These aspects have to be accurately evaluated in defining the tests specifications. The use of some data analysis techniques (such as the proper orthogonal decomposition, the sensibility analysis, etc.), in keeping the essential design parameters, is presented too.

**Keywords:** design assisted by testing, wind tunnel testing, test specification, aerodynamic behaviour, aeroelastic behaviour.

### 1. Introduction

The paper presents some of the recent experiences in wind tunnel testing, made by the Authors within the design stage of steel structures. According to principle of the “design assisted by testing”, contemplated by the Eurocode, the combination of tests and calculations is increasingly used. Multiples are the motivation of this increasing interest/demand. The sophisticated architectural shapes (culminating in the “free form design”), the absence of corresponding reliable fluid-dynamics models and the increasing flexibility of structures are probably the main ones. Not less important is the opportunity/necessity to confirm by control checks the assumptions made in the design, specially with respect to complex aeroelastic phenomena, such as the lock-in, the flutter, the rain-wind induced vibration, etc..

The main purpose of the paper is to outline the potentiality of the presented tests and the main problems encountered in their use. These aspects have to be accurately evaluated in defining the tests specifications. The use of some data analysis techniques (such as the proper orthogonal decomposition, the sensibility analysis, etc.), in keeping the essential design parameters, is presented too.

The presented experiences are relative to the tests performed during the design of: a cable stayed bridge on the Adige river (on the A31 motorway - Italy); the New Braga Stadium (Braga - Portugal); the new Unipol headquarter Tower (Bologna - Italy); the new Bologna town-hall (Bologna - Italy).

### 2. The Adige River Bridge

The design of the Adige river cable stayed bridge was assisted by wind tunnel tests aimed to check the aerodynamic and the aeroelastic behaviour of the deck and of the tower. The tests were carried out at the “Politecnico di Milano” boundary layer wind tunnel (Fig. 1).



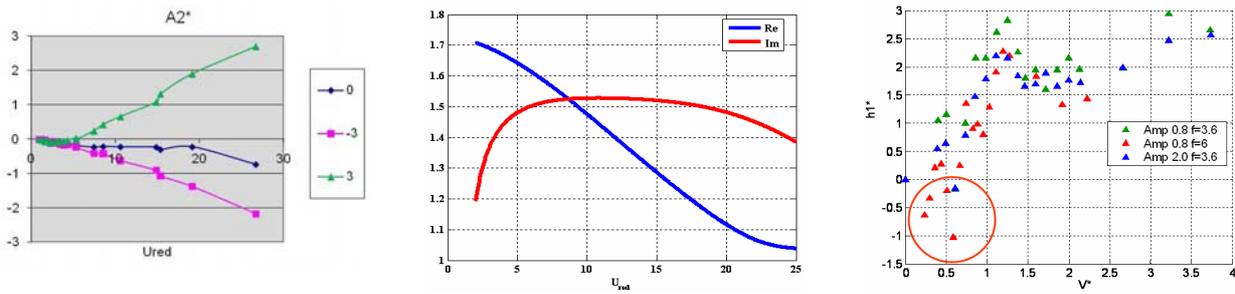


Fig. 2: Flutter derivatives  $A2^*$ , for the attack angles  $0^\circ$ ,  $3^\circ$ ,  $-3^\circ$  (a); real and imaginary parts of the flutter equation solution. Coupling of modes 1 and 3 (b); flutter derivatives  $h1^*$  at low reduced velocity (c)

Of course, this is an approximate procedure, because vortex shedding is a strongly non-linear phenomenon; it is here treated by a linear approach, but this allows to estimate the order of magnitude for the minimum structural damping which is necessary to protect the structure from vortex-shedding.

### 3. The New Braga Stadium

The tests for the design of the new Braga Stadium suspended roof were carried out, independently, at the RWDI and at the “Politecnico di Milano” wind tunnel. RWDI tests were performed on rigid models, to measure the pressure field on the roof. At the “Politecnico di Milano”, was tested a flexible (aeroelastic) model, 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 (Fig. 3). 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.

The design wind speeds at the stadium site were derived from 1:1500 scaled wind tunnel tests (including a wide stadium surroundings), while the pressure coefficients were measured in 1:400 model. Both the wind speed profiles and turbulence presented different features in 1:400 tests with respect to 1:1500 ones. In order to match the peak gust velocities between the two scale models at the 25 and 50 m (roof position) heights, an equivalent design wind speed was calculated by RWDI. The “double model” was a source of further uncertainties and data treatment problems. It also outlined a more general problem which often occurs in defining the model reference height: due to the imperfect reproduction of the wind profile, significant errors can befall when wind tunnel dimensionless coefficients are used with reference to standard codes design wind speed.

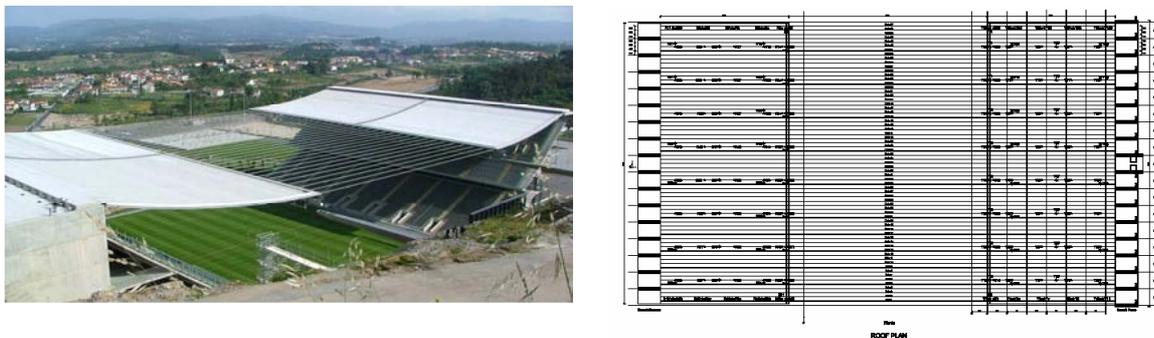


Fig. 3: The New Braga Stadium and the RWDI pressure taps arrangement

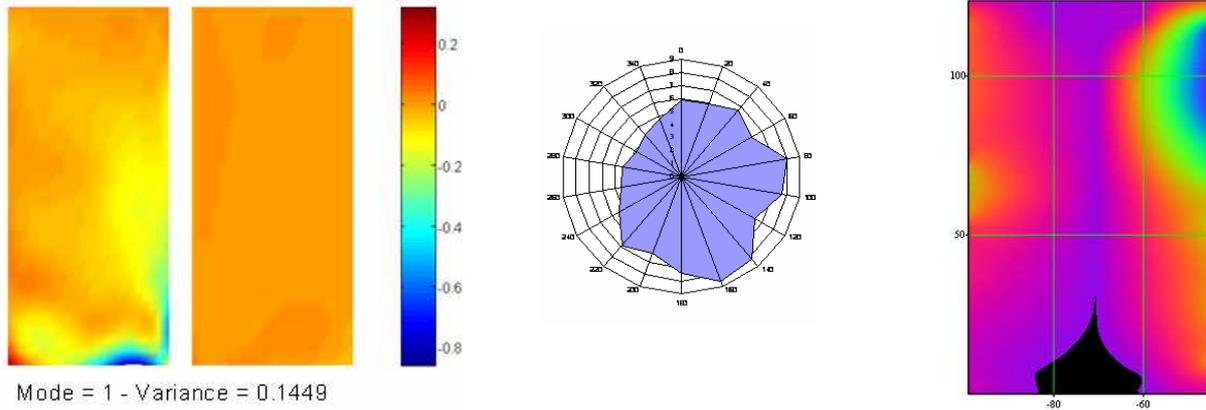


Fig. 4: Proper orthogonal decomposition of the measured wind pressure fields (a); minimum reliability index  $\beta$  for each direction (b) and distribution of  $\beta$  values along the slab (c).

The orthogonal decomposition techniques have been adopted to simplify the pressure representation, and to reduce the computational effort. The structural response was determined by separately evaluating the mean, the quasi-steady and the resonant response. The first term simply takes into account the mean pressure distributions. The second one is obtained by performing a classical covariance proper orthogonal decomposition (POD). The third one takes into account the dynamic amplification of a suitable number of structural vibration modes, each one being excited by the unsteady wind pressures.

The analytical procedure is synthesised in the following expression, while a typical result is summarised in Table 1. The Eqs. (3) resume the procedure to decompose (POD) a pressure field (Fig. 4a), the Eqs. (4) determine the mean and the quasi-static response and the Eqs. (5) are relative to the resonant contribution.

$$\mathbf{C}_p = \sum_{k=1}^m \phi_k \phi_k^T \lambda_k; \quad \phi_i \phi_j^T = \delta_{ij}; \quad \phi_i^T \mathbf{C}_p \phi_j = \delta_{ij} \lambda_j; \quad \mathbf{p}(t) = \sum_{k=1}^m \phi_k x_k(t). \quad (3)$$

$$R(t) = \sum_{k=1}^s x_k(t) \cdot R_k; \quad \sigma_{R,qs}^2 = \sum_{k=1}^s \sigma_{x_k}^2 R_k^2; \quad R_{qs} = \bar{R} \pm g_{qs} \sigma_{R,qs}. \quad (4)$$

$$R_{qs+res} = \bar{R} \pm \sqrt{g_{qs}^2 \sigma_{R,qs}^2 + g_{res}^2 \sigma_{R,res}^2}; \quad g_{res}^2 \sigma_{R,res}^2 = \sum_{h=1}^n g_h^2 \sigma_{R_h}^2;$$

$$g_h = \sqrt{2 \ln(f_h T)} + \frac{0.577}{\sqrt{2 \ln(f_h T)}}; \quad M_h = \int_A m(x, y) \cdot \psi_h^2(x, y) \cdot dA; \quad (5)$$

$$Q_h(t) = \int_A p(x, y, t) \cdot \psi_h(x, y) \cdot dA; \quad \sigma_{R_h}^2 = R_h^2 \frac{\pi}{4 \cdot \xi_h} \frac{f_h \cdot S_{Q_h}(f_h)}{(2 \cdot \pi \cdot f_h)^4 M_h^2}.$$

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).

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 (Fig. 4b), to recognize the structure regions which have the higher failure probability (Fig. 4c, where the black region show the lowest  $\beta$  values – 3,798) and to determine the wind loads spatial distribution which leads to the higher failure probability.

Table 1: Typical response contribution, in terms of vertical displacement at some roof locations: mean (col. 2), quasi-static (col. 4) and total response (col. 6)

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

An advantage of this technique with respect to the POD is that it considers both the wind stochastic structure and the structural response.

The aeroelastic model (1:70) was tested at the “Politecnico di Milano” BLWT in both laminar flow (without external environmental reconstruction) and in turbulent flow (with external environmental reconstruction). The aeroelastic stability was checked up to 58 m/s (full scale). In turbulent wind conditions, oscillation up to 40-50 cm full scale, as order of magnitude, were recorded. 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.

Since January 2004, the structure was equipped with a permanent monitoring system, to measure different parameters (such as pressures, accelerations, wind velocity, cable stresses, etc.) 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.

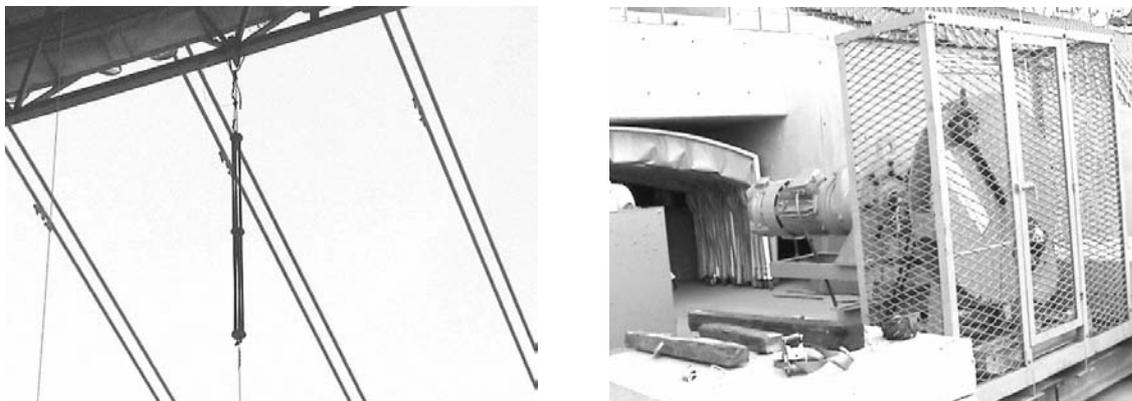


Fig. 5: Images of the exciting system

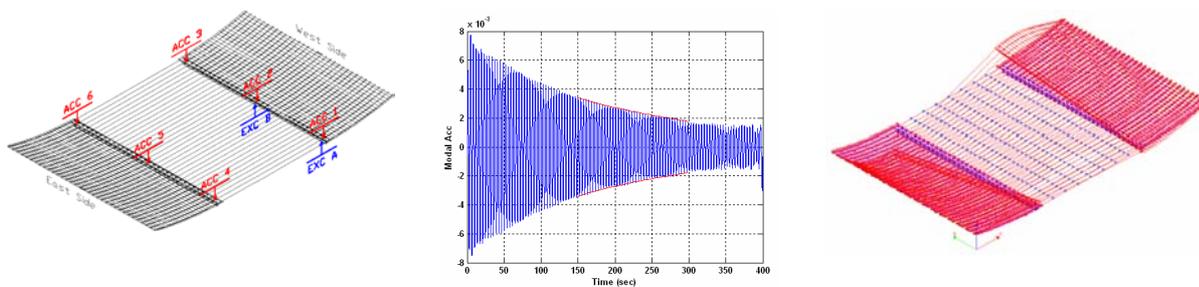


Fig. 6: Exciting-measuring positions (a), typical free motion record (b) and 1st natural mode (c)



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 (Fig. 5a) and it was sinusoidally moved at the other end by an electrical engine (Fig. 5b). 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 (Fig. 6a).

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 (Fig. 6b). The modal shapes were determined by orthogonal decomposing (POD) the recorded signals (Fig. 6c).

#### 4. The Unipol Tower

The design of a tall building (123 m) was assisted by wind tunnel tests carried out at the CRIACIV (Prato, Italy) boundary layer wind tunnel on a rigid model 1:350 scaled and equipped with 125 pressure taps (Fig. 7).

The aim of the tests was to determine the aerodynamic forces by both a simultaneous measurement of pressures at 125 point on facades and a direct measurement of the base global forces by mean of a dynamometric balance. In addition, pressures and flow within a double skin facade, which covers part of the building, were required to be checked.

Different problems were encountered to use wind tunnel data in the design evaluations. Firstly, the global forces and moments (obtained by integration of the pressure fields) were provided in a “wind based” local reference system (drag-lift, Fig. 8a,b). Independent elaboration was necessary to define the design loads in the global-structure reference system.

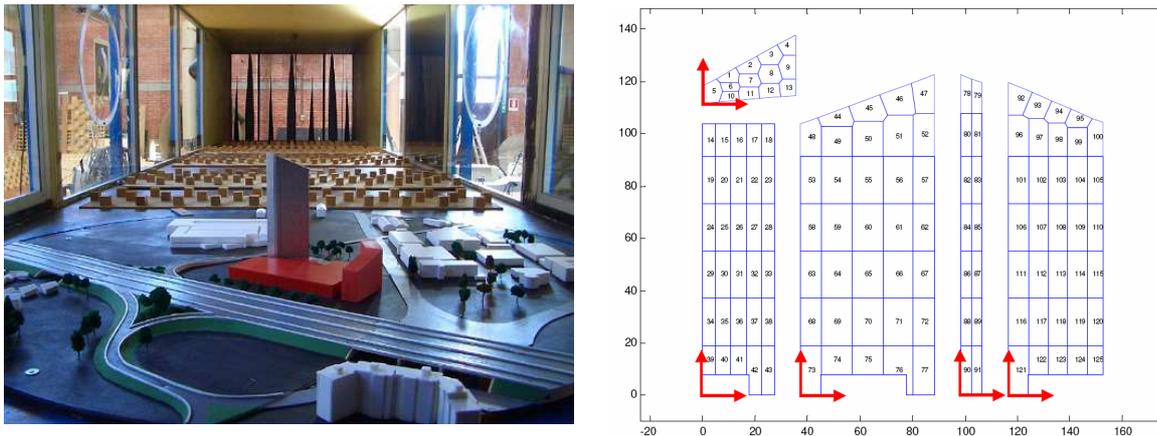


Fig. 7: The Unipol Tower wind tunnel model (a) and the pressure taps locations (b)

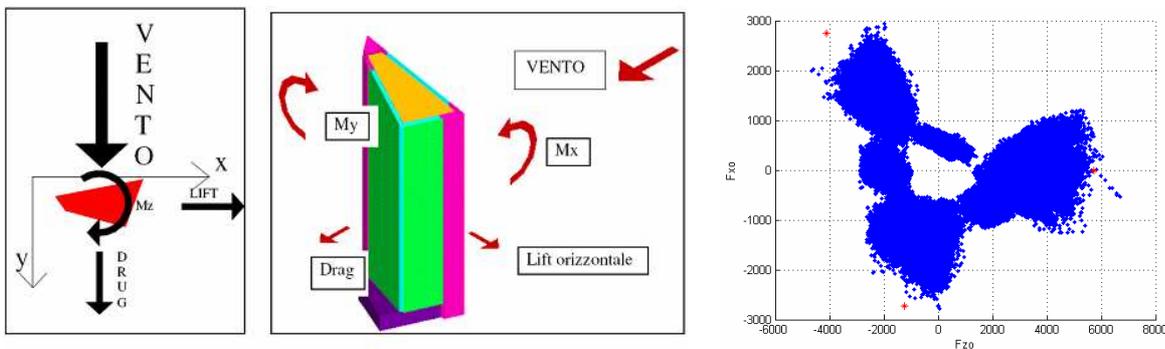


Fig. 8: Wind tunnel reference system (a,b) and correlation locus of longitudinal-transversal force (c)

Secondly, data were given without any information about the statistical correlation between components. Hence, the designer cannot define confident load combinations without an independent elaboration of the digital data.

Thirdly, the direct measure of global forces (dynamometric balance) did not agree with the global forces obtained as the integration of the pressure fields. As a matter of fact, while the mean values were quite similar, the fluctuating part was considered unreliable; in fact, the independent elaboration of the recorded data showed that this fluctuating part was very similar to a white noise.

Once the preliminary treatment and reliability problems were overcome, the simultaneous pressure fields were retained only. The design forces on the building were evaluated at different levels, by taking into account the correlation between the components. Due to the asymmetry of the structure, the significant load combinations were determined by mean of the longitudinal-transversal force locus (Fig. 8c), by including all the tested wind directions. In addition, information on local pressures were also available. The measurement of simultaneous pressure fields is quite “reach” of information, even if the comparison with directly measured global forces is suggested.

## 5. Bologna Town Hall

The Bologna town hall structural design was assisted by wind tunnel tests performed at the “Politecnico di Milano” BLWT. Two different models were arranged: a 1:50 model of the whole structure (Fig. 9a), for the measurement of loads on roofs and facades; a 1:1 model of a roof tube array (Fig. 10a), to check the vortex shedding sensibility.

The 1:50 model of the whole structure was used to measure the forces on the roof (by mean of dynamometric balances) and the pressures on facades (by mean of appropriate distribution of pressure taps).

Loads on the roofs were given in terms of force coefficients, while pressures on facades were given in terms of pressure coefficient diagrams and global maps (Fig 9b).

The building roof is covered by a series of tube arrays, regularly spaced. The length of tubes varies between 7 and 14 m. The length-diameter ratio (up to 39), the light weight (aluminium) and the end connection type (which arises a very low damping ratio, approximately  $\xi = 0,2\%$ ) make the tubes susceptible of dynamic excitation. Since the Scruton number is approximately 2,3, it is expected that the tubes are sensitive to vortex shedding excitation.

In the case of isolated cylinder, this Scruton number value gives rise to oscillations of about 0.3 diameters. As a matter of fact, the array effect is also tested in the wind tunnel. The results substantially confirm the previsions made for the single tube, the recorded displacement being 0,25 D as order of magnitude.

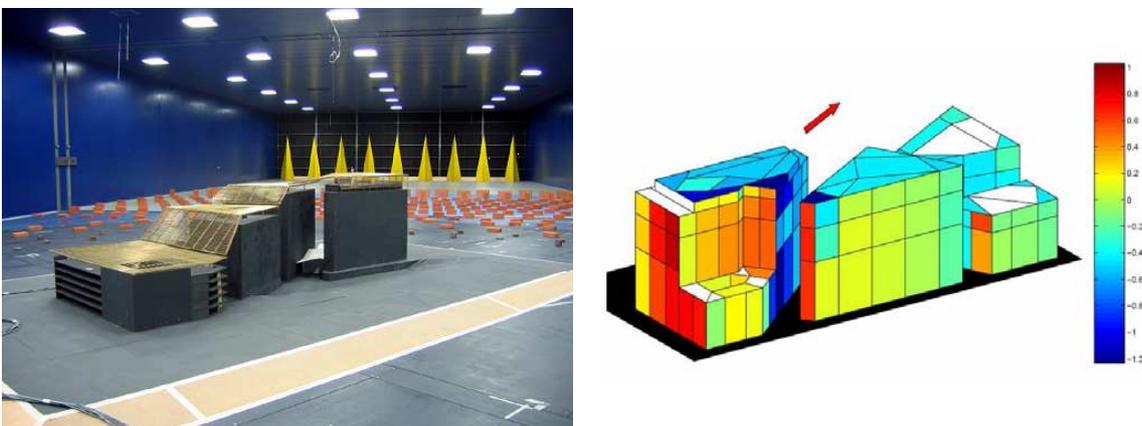


Fig. 9: Bologna Town Hall model (a) and typical pressure map (b)

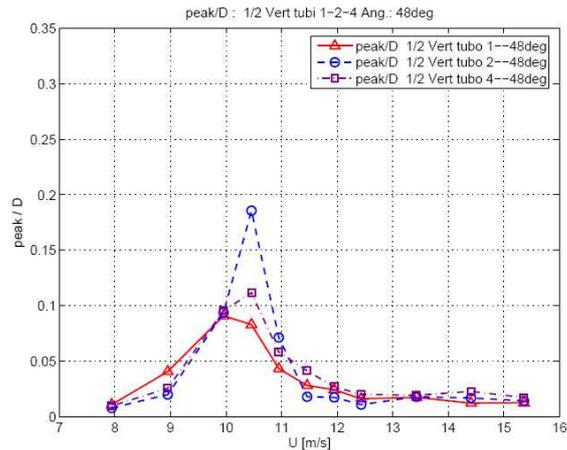


Fig. 10: Wind tunnel model (a) and spectral density function of the wind speed in the wake for different angle of the array setup (b)

In addition, wind tunnel tests showed that the tubes oscillation can reach amplitudes which can induce damage on the tubes themselves and on the connection and can induce appreciable oscillation on the whole roof. The wind “look-in” speed which is clearly in the range of the frequently expected wind speed at the structure site (Fig. 10b).

Wind tunnel tests suggested to implement design solution to avoid or reduce the vortex shedding sensibility. Possible ways to do so are aerodynamic solutions (as, for instance, placing spires on the tubes) and/or adding mechanical damping to the tubes. The final designer-owner decision was to realize the roof without any countermeasure, to monitor it and to evaluate “on site” the opportunity of acting on the tubes (a case of design assisted by testing and monitoring).

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