



Building occupants' comfort assessment in the PBWE framework

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Abstract

This research deals with the problem of the comfort assessment of high-rise building occupants under wind action. Also if the problem has been studied by the researchers and by the civil engineering industry during last thirty years, appropriate methods to handling the design of high-rise buildings in order to avoid wind-induced occupant discomfort has not been defined yet, mainly due to the high uncertainties involved in the determination of both the demand and the sensitivity of the building occupants to wind-induced vibrations. The main issues related with this problem are first summarized, then the growing, pioneering performance-based wind engineering (PBWE) approach is proposed as tool to handle the problem. The required analyses are presented and discussed on both the conceptual and operational point of view. A case-study is then presented in order to demonstrate the effectiveness of the proposed approach. In the PBWE view, the contribution of the work is focused on the procedural step identified as “damage analysis”, something that, in authors' knowledge, has not been yet developed in the literature.

1 Introduction

Performance-Based Wind Engineering (PBWE) is a new branch of performance-based engineering gaining interest in the last years (Ciampoli et al. 2011, Spence and Giofrè 2012, Pozzuoli et al. 2013). As defined in Ciampoli et al. (2011), the central objective is the assessment of the adequacy of the structure through the probabilistic description of a set of Decision Variables (DVs). Each decision variable is a measurable attribute that represents a specific structural performance, which can be defined in terms of the interest of the users or the society. The problem of risk assessment is addressed by means of the resolution of the integral shown in Eq. (1). Thus, the risk is disaggregated into different elements: site and structure-specific hazard analysis; structural characterization; interaction analysis; structural analysis; damage analysis; and loss analysis.

$$G(DV) = \int \int \int \int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM, IP, SP) \cdot f(IP|IM, SP) \cdot f(IM) \cdot f(SP) \cdot dDM \cdot dEDP \cdot dIM \cdot dIP \cdot dSP \quad (1)$$

In Eq. (1), $G(\bullet)$ is the complementary cumulative distribution function and $G(\bullet|\bullet)$ the conditional complementary cumulative distribution function; $f(\bullet)$ is the probability density function, and $f(\bullet|\bullet)$ the conditional probability density function; DM is a proper damage measure; EDP is an engineering demand parameter; the basic parameters characterizing the Aeolian hazard, the interaction phenomena and the structural systems and non-environmental actions are described respectively to IM, IP and SP. In Equation 1, IM and SP are assumed as uncorrelated and independent on IP, while IP is dependent on both IM and SP. The flowchart of the PBWE is shown in Figure 1.

Considering the last two risk assessment elements (damage and loss analysis), even though there is a great deal of literature on Performance-Based Design (PBD) in earthquake engineering, but also, for example, in hurricane engineering, these elements have not been investigated thoroughly in the PBWE for non-hurricane winds. Especially with regard to the comfort assessment of high-rise buildings under wind, additional research efforts are needed in order to complete the PBWE procedure.

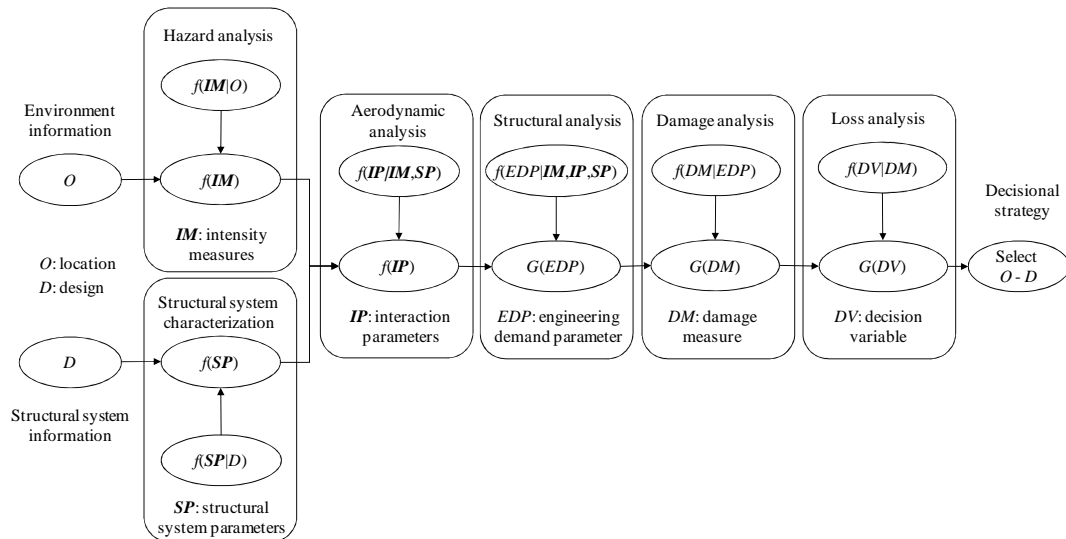


Figure 1. Flowchart of the PBWE procedure

In a first step, that in the PBWE framework can be identified as damage analysis, appropriate criteria need to be defined in order to characterize the uncertainty affecting the response thresholds assumed for the evaluation of the serviceability levels. This is particularly difficult in the evaluation of the building residents comfort (Bashor et al. 2005). In addition, in conducting the loss analysis, the monetary quantification of the loss of serviceability is required. In the authors' knowledge this aspect is, at the moment, poorly treated in literature on tall building serviceability under wind action.

2 Case study building

The damage analysis is carried out considering the results obtained by Ciampoli and Petrini (2012). The peak accelerations (the engineering demand parameter for the serviceability) are provided for a case-study tall building, and two structural configurations of the case-study tall building are considered: i) the original design and ii) the modified design by a Tuned Mass Damper (TMD).

The case-study building (Fig. 2) has a square plan, the side is 50 m long and the total height is 305 m; the number of floors is 74. The structural system is composed by both a central core (a 3D frame with 16 columns) and a 3D frame (composed by 28 columns) on the external perimeter. The two substructures are connected at three levels (at 100 m, 200 m and 300 m) by stiffening systems extended for 3 or 2 floors. The columns have a hollow square section, with dimensions and thickness varying with the height (between 1.20 m and 0.50 m, and 0.06 m and 0.025 m respectively). The beams are double-T steel beams and the beam-column joints are considered as being rigid in the structural model. The bracing system is composed by hollow square struts. Figure 2 shows both the finite element model and the three subsystems of the case-study tall building.

In Ciampoli and Petrini (2012) the model of the wind actions is calibrated on the basis of experimental results consisting in the time series of the global floor forces available by tests on a 1:500 scale rigid model. These tests have been carried out at the Boundary Layer Wind Tunnel of CRIACIV (Inter-University Research Centre on Buildings Aerodynamics and Wind Engineering) in Prato, Italy.

By the mentioned experimental test it was been assessed that the energy content of the global force spectra for various directions of the wind velocity increases due to the vortex shedding generated at the sharp edges of the building. By the analytical model of the lift force spectrum in across-wind direction this phenomenon is taken into account summing up an ad-hoc triangular function

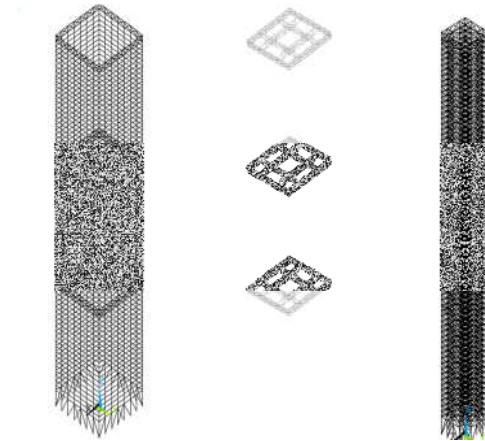


Figure 2: Finite element model of the structural system: the 3D frame on the external perimeter (left), the bracing system (central), the central core (right)

Figure 3 summarizes the response of the building to different wind intensities (represented by the 10-mins mean wind velocity at ten meters height V_{10}) in terms of the peak acceleration both in the original and modified design configuration. The study on the damage analysis about the serviceability takes place by the results of Figure 3.

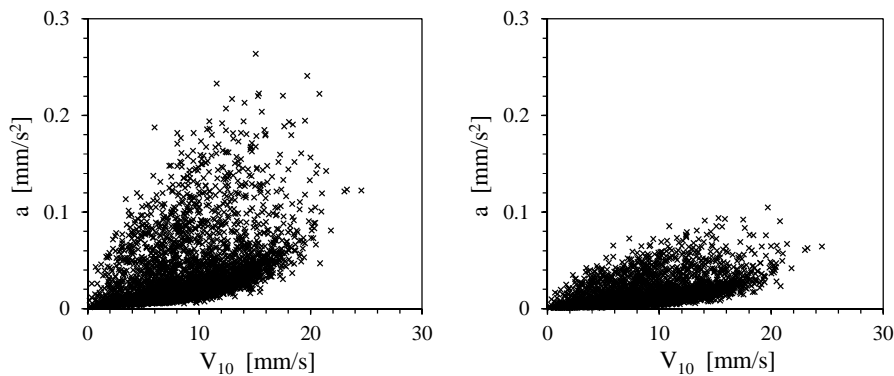


Figure 3. Samples of the peak acceleration (Monte Carlo simulation: 5000 runs) as a function of V_{10} provided by Petrini and Ciampoli (2012). Original design (left), modified design (right)

Three probability density functions are considered for fitting the peak acceleration samples shown in Figure 3: the linear piecewise (kernel probability density function in Matlab[®]), the log-normal, and the Wald probability density function. The probability of exceeding the defined serviceability limit states is computed by these three density functions and for the original and modified design of the case-study tall building. The damage analysis consists in the definition of the fragility in terms of damage measure, namely $G(DM \geq DM^* | EDP)$, being DM^* a threshold value for DM . In this case the fragility can be specialized as $G(N_p \geq n_p | a)$, where N_p is the number of people perceiving the motion (in percentage with respect to the total building users), and n_p is an acceptable number of people perceiving the motion (then identifying a limit state).

For computing the $G(N_p \geq n_p | a)$ the results of several experimental tests as taken from the literature are considered. Each of these results is given by a number of curves representing the motion perception threshold (in terms of floor acceleration a) as a function of the fundamental natural frequency of the building. Each curve is associated with a value of N_p , and it can be assumed as the determinist threshold of the acceleration for which N_p people perceive the motion. Here, in order to model the uncertainties affecting these threshold values, N_p is assumed as a stochastic variable having a lognormal distribution, the dispersion and the mean value of this distribution are evaluated, as a function of the acceleration a , on the basis of the above-mentioned experimental results. Then, for a fixed a value n_p , and a fixed value of a , the fragility function can be simply evaluated as the ratio

between the number of experimental results being greater than a (for that n_p) and the total number of experimental results available in literature for that n_p (see Fig. 4). The fragility curves are shown in Figure 5 referring to the 74th floor of the building, both for the original and the modified design configurations.

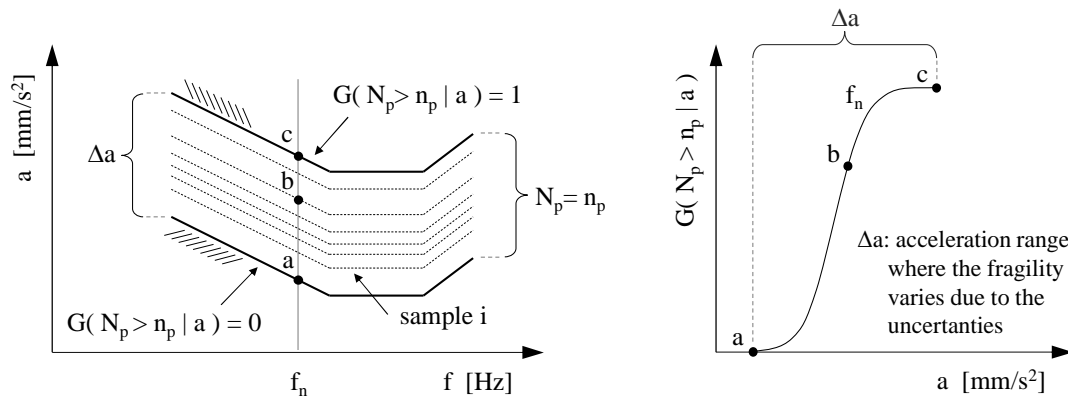


Figure 4. Extrapolation of the fragility fragility curves from experimental results available in literature. Threshold curves taken from the literature for a certain value of n_p (left), example of fragility curve (right).

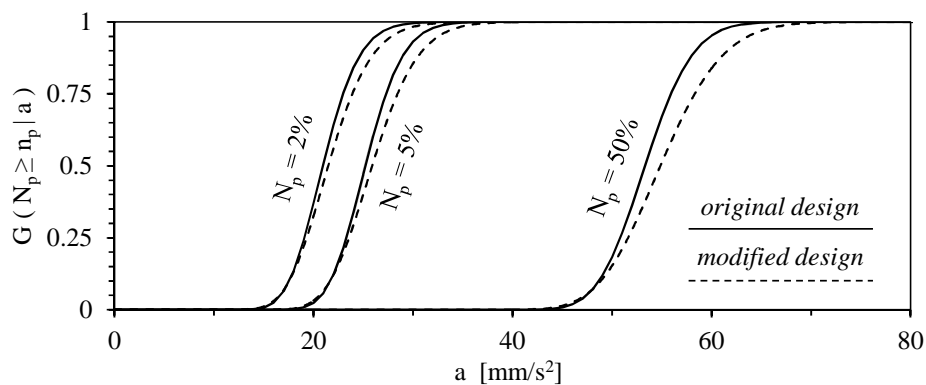


Figure 5. Fragility curves obtained from the analysis.

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