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Probabilistic serviceability-performance assessment of tall mass-timber buildings subjected to stochastic wind loads: Part II - structural reliability analysis

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ABSTRACT

In this paper, the second of two companion papers, the serviceability-performance of a tall mass-timber building is evaluated probabilistically. For the assessment, the Alan G. Davenport Wind Loading Chain is adapted to a probabilistic Performance-Based Wind Engineering (PBWE) framework. The framework allows incorporation of uncertainties at each step of the wind loading chain, i.e., local wind climate and exposure, aerodynamics, dynamic effects, and criteria. As a case study, the framework is applied to quantify the serviceability-performance of a 102-m tall mass-timber building. Initially, parametric analyses are carried out to study the influence of critical damping ratio, wind direction, and local turbulence intensity on the response of the case study tall mass-timber building. Results showed the dependence of story level structural responses on the studied parameters and hence the need to consider the role of uncertainties. Therefore, uncertainties in various parts of the Wind Load Chain are explicitly modeled using fifteen random variables. Structural reliability analysis using Monte Carlo sampling is performed to propagate the uncertainties through the Wind Loading Chain. The results from reliability analysis are used to develop fragility curves for wind vulnerability estimations. Based on the results, design recommendations based on building functionality are forwarded.

1. Introduction

Over the past two decades, urban sustainability requirements are partly influencing urban designers and architects towards the utilization of timber for tall buildings. This revolution in the current urban form is due to two main reasons: (1) wood is a major renewable, and recyclable material which requires less production energy as compared to concrete and steel, and (2) tall buildings can be a solution to the ever-increasing land scarcity in cities and reduce urban sprawl. To this end, there is a global effort to increase the height of timber based buildings with significant research attention devoted to developing seismic design guidelines and performance evaluation of sub-assemblies in timber based structures, for example (Chen and Chui, 2017; Pei et al., 2017; Bezabeh et al., 2017; Bezabeh et al., 2016; Tesfamariam et al., 2015; Gavric et al., 2015; Popovski et al., 2014; Karacabeyli and Lum, 2014; SOM, 2013; Rinaldin et al., 2013; Green and Karsh, 2012; Fragiacomo et al., 2011). In addition, in the past decade considerable research has been carried out on the wind performance of low-rise wood-frame structures, to name a few (Zisis and Stathopoulos 2009, 2013; Zisis et al., 2011; Morrison and Kopp, 2011; van de Lindt and Dao 2009, 2011; van de Lindt et al., 2007; Li and Ellingwood, 2006, 2005; Ellingwood et al., 2004; Rosowsky and Ellingwood, 2002).

As compared to buildings made of concrete and steel, tall mass-timber buildings have low weight and lateral stiffness. The low lateral stiffness of tall mass-timber buildings could make them vulnerable to excessive along- and across-wind vibrations (Bezabeh et al., 2018b; Chen and Chui, 2017; Wilbur and Bartlett, 2016; Feldmann et al., 2016; Popovski et al., 2014; Fairhurst, 2014; Reynolds et al., 2011). In general, frequent

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exposure to excessive wind-induced vibrations can cause occupant discomfort (possible inhabitability) and unserviceability of this kind of emerging buildings. Therefore, the design of tall mass-timber buildings should consider the wind serviceability and ultimate limit state requirements. While there are considerable studies in the earthquake performance assessment of mass-timber structures and wind assessment of low-rise timber buildings, studies on wind response of tall mass-timber buildings with explicit consideration of uncertainties are scarce.

The performance assessment of wind-excited tall buildings requires information on the wind hazard, building aerodynamics, structural properties, criteria, and associated consequences. In the wind engineering community, the literature is rich in wind micro-climate analysis, building aerodynamic studies (using wind tunnel techniques) and establishing performance criteria (using full-scale measurement techniques and experiments). However, the information from many of the above techniques is subjected to aleatory and epistemic uncertainties. Several researchers have attempted to quantify the uncertainties associated with wind performance assessment process (e.g Fu and Li, 2018; Elshaer et al., 2017; Hong, 2016; Spence et al., 2016; Cui and Caracoglia, 2015; Spence and Kareem, 2014; Hong et al., 2014; Bernardini et al., 2014; Bernardini et al., 2013; Li and Hu, 2014; Warsido and Bitsuamlak, 2015; Spence and Gioffre, 2012; He and Hong, 2012; Warsido, 2013; Smith and Caracoglia, 2011; Ciampoli et al., 2011; Pozos-Estrada et al., 2011; Merrick and Bitsuamlak, 2009; Bashor and Kareem, 2009; Tamura et al., 2006; Diniz and Simiu, 2005; Bashor et al., 2005; Hong et al., 2001; Minciarelli et al., 2001; Pagnini and Solari, 1998; Solari, 1997, 1996; Pagnini, 1996; Griffis, 1993; Kanda, 1983; Davenport et al., 1985; Davenport, 1983; Davenport, 1981; Davenport, 1964). For building serviceability assessment, Pagnini and Solari (1998) and Kareem and Sun 1990 demonstrated that lack of knowledge related to structural damping could make the assessment results unreliable. In practice, even though structural damping is the most uncertain parameter, it is often considered as a deterministic parameter. Most building codes and standards suggest the use of 1–1.5% of critical damping for steel buildings and up to 2% for reinforced concrete uncracked sections for wind design and performance assessment of tall buildings. Limited studies used probabilistic methods to quantify and assess the impact of wind and structural related uncertainties on the response of the structures (e.g. Carassale and Solari (2006); Cui and Caracoglia 2015; Zhang and Chen 2015; Bernardini et al., 2014; Hong et al., 2014; Hong et al. 2013; Yang et al., 2013; Petrini and Ciampoli 2012; Pozos-Estrada et al., 2011; Ciampoli et al., 2011; Smith and Caracoglia 2011; Pagnini, 2010; Masters et al., 2010; Kareem 2008; Zhang et al., 2008; Carassale and Solari, 2006; Bashor et al., 2005; Pagnini and Solari 1998; Kareem and Gurley 1996; Shinozuka et al., 1990; Kareem and Sun 1990; Riera et al., 1977).

The state of development in defining the source and mechanism of structural damping in mass-timber structures is poor. Based on limited ambient vibration test data, Karacabeyli and Lum (2014) suggested the use of 3% of critical damping ratio for wind design of mass-timber buildings. As compared to other structural systems, timber connections expected to dissipate more energy at service level wind loads. Johansson et al. (2016) presented a critical damping ratio database for timber-based structures. The reported damping levels are highly scattered. Fairhurst (2014) used 2% of critical damping for wind analysis of a 30-story FFTT tall hybrid-timber building. Based on the Eurocode 1 recommendation for timber bridges, Tjernberg (2015) used ~1% of critical damping to analyze a 22-story timber building. Reynolds et al. (2011) studied the effects of structural damping on the along-wind response of structures.

Another major challenge in performance assessment of wind-excited mass-timber buildings is the variations in properties of wood materials, which could make the lateral stiffness of mass-timber buildings uncertain. The uncertainties in structural properties of mass-timber compounded with the wind hazard and aerodynamic uncertainties prompted the use of a probabilistic wind performance assessment framework in this paper. This paper, based on the transient load information from the wind tunnel tests of the companion paper, evaluated the serviceability performance of the case study tall mass-timber building using a probabilistic framework. In the companion paper (Bezabeh et al., 2018a), the framework is discussed, and conceptual considerations are outlined to adapt the *Wind Loading Chain* as a viable PBWE framework. Moreover, in the companion paper, a complete structural design procedure is outlined and applied to design a case study 102-m tall mass-timber building using 1-in-50-year design wind speed. The results of aerodynamic wind tunnel tests are also presented.

In this paper, initially, functional relationships in the frequency domain are developed to obtain the dynamic responses of the case study mass-timber building using random vibration theory that accounts for 3D mode shapes of buildings and statistical correlations between modal responses. After the formulation of functional relationships, parametric studies are performed to study the influence of critical damping ratio, wind direction, and local longitudinal turbulence intensity. Subsequently, uncertainties in various part of the *Wind Loading Chain* are modeled, and two serviceability limit states (habitability and excessive deflection) are defined using 10- and 50-year return period wind speeds. The final part of the *Wind Loading Chain*, checking with performance criteria, is performed using structural reliability method by calculating the probability of exceeding the limits of the NBCC 2010 (NRCC, 2010). The results from reliability analysis are used to develop fragility curves for wind vulnerability estimations.

2. Functional relationships in frequency domain

For the performance evaluation of the case study building, functional relationships are needed in the *Wind Loading Chain* to quantify building responses under turbulent wind load. This can be achieved by using random vibration theory (e.g., Davenport, 1964, 1967). In random vibration theory, dynamic response of the buildings can be evaluated using linear modal analysis in the frequency domain. The governing equation of motion of a building with n number of floors under turbulent wind load can be written as:

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = F(t) \tag{1}$$

in which:

$$[M] = \begin{bmatrix} M_x & 0 & 0 \\ 0 & M_y & 0 \\ 0 & 0 & M_\theta \end{bmatrix}; \quad [C] = \begin{bmatrix} C_{xx} & 0 & C_{x\theta} \\ 0 & C_{yy} & C_{y\theta} \\ C_{x\theta}^T & C_{y\theta}^T & C_{\theta\theta} \end{bmatrix}; \quad [K] = \begin{bmatrix} K_{xx} & 0 & K_{x\theta} \\ 0 & K_{yy} & K_{y\theta} \\ K_{x\theta}^T & K_{y\theta}^T & K_{\theta\theta} \end{bmatrix}$$

where:

$\ddot{u}(t) = \{\ddot{u}_x, \ddot{u}_y, \ddot{u}_\theta\}^T$	: $3n \times 1$ acceleration vector
$\dot{u}(t) = \{\dot{u}_x, \dot{u}_y, \dot{u}_\theta\}^T$	: $3n \times 1$ velocity vector
$u(t) = \{u_x, u_y, u_\theta\}^T$	: $3n \times 1$ displacement vector
M_x, M_y, M_θ	: $n \times n$ floor mass sub-matrices
C_{xx}, C_{yy}, $C_{\theta\theta}$, $C_{x\theta}$, $C_{y\theta}$	: $n \times n$ damping sub-matrices
K_{xx}, K_{yy}, $K_{\theta\theta}$, $K_{x\theta}$, $K_{y\theta}$	: $n \times n$ stiffness sub-matrices
$F(t) = \{F_x, F_y, F_\theta\}^T$	: $3n \times 1$ wind force vector from the pressure integration of wind tunnel data presented in the companion paper (Bezabeh et al., 2018a)

The wind force vectors at each floor of the building are calculated from the full-scale reference mean wind velocity at the building height, assigned tributary area of tap (A_i), direction cosines of building surface at each pressure tap (n_x and n_y) and moment arm from the considered tap to the geometric center of the plan of the building (r_i).