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RISK-BASED WIND DESIGN OF TALL MASS-TIMBER BUILDINGS

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Abstract: The rapid growth of urban population and the associated environmental concerns are partly influencing city planners and construction stakeholders to consider "*Sustainable Urbanization*" alternatives. In this regard, recent urban design strategies are entertaining the use of "*tall timber buildings*". Generally, tall mass-timber buildings (MTBs) utilize pre-engineered wood panels to form their main gravity and lateral load resisting systems, which makes them lighter and more flexible than buildings made from concrete, masonry or steel. As a result, frequent exposure to excessive wind induced vibrations can cause occupant discomfort and possible inhabitability of the buildings. This paper attempts to apply a risk-based procedure to design a 102 meter tall MTB by adapting and extending the *Alan G. Davenport Wind Loading Chain* as a probabilistic performance-based wind engineering framework. The structural systems of the study building are composed of Cross Laminated Timber (CLT) shear walls, CLT floors, glulam columns and reinforced-concrete link beams. Initially, aerodynamic wind tunnel tests were carried out at the *Boundary Layer Wind Tunnel Laboratory* of *Western University* on the 1:200 scale MTB model to obtain transient wind loads. Subsequently, using the wind tunnel data, the study MTB was structurally designed. In the risk-based performance assessment, uncertainties were incorporated at each step of the *Wind Loading Chain*, i.e., local wind, exposure, aerodynamics, dynamic effects, and criteria. These uncertainties were explicitly modeled as random variables. Dynamic structural analyses were carried out in the frequency-domain to include the amplification due to the resonance component of the excitation. Structural reliability analysis through Monte Carlo sampling was used to propagate the uncertainties through the *Wind Loading Chain* to quantify the risk of inhabitability and excessive deflection. The results of reliability analysis were used to develop fragility curves for wind vulnerability estimations. Based on the results, the effects of various uncertainties are discussed, and risk-based design decisions are forwarded.

1 INTRODUCTION

The rapid growth of urban population and the associated environmental concerns are partly influencing city planners and construction stakeholders to consider "*Sustainable Urbanization*" alternatives. Now days, sustainable urbanization has emerged as a viable solution towards smart and livable cities that are more resilient, and environmental friendly. In this regard, recent urban design strategies are entertaining the use of "*tall timber buildings*". Recent studies on the life cycle assessment of buildings indicated that wood is an efficient construction material in terms of embodied energy and greenhouse gas emissions (FPInnovations 2010). To this end, latest design guidelines and standards in US and Canada are considering the use of multi-story mass-timber buildings (MTBs). For example, the *2015 American National Design Specification (NDS) for Wood Construction* (ANSI/AWC NDS-2015) and the *2016 supplement of the Canadian National*

Standard for Engineering Design in Wood (CSA O86) included chapters dedicated to the design of CLT structural elements. In 2018, the *International Building Code (IBC) Ad Hoc Committee on Tall Wood Buildings approved the application of "heavy-timber" structural elements for constructions beyond Type IV* (Baldassarra 2017). In addition to the inclusions in the building codes and standards, over the past 15 years, with the use of CLT, several tall timber-based buildings were constructed. In 2017, the University of British Columbia (UBC) finalized the construction of 18 story (53 m) tall hybrid mass-timber residential building. Currently, the building is the tallest standing timber-based building in the world. The success of existing timber-based buildings in meeting their design objectives enhances the confidence of architects and developers towards pushing the height limits of timber buildings beyond 50 meters.

To make wooden skyscrapers possible, research on timber structures is getting momentum. For example, in Canada, FPInnovations released the first comprehensive design and construction guideline of tall timber buildings (Karacabeyli and Lum 2014). Researchers at the University British Columbia and FPInnovations also developed both, force- and displacement-based design guidelines for timber-steel hybrid system (Tesfamariam et al. 2015, Bezabeh et al. 2016, Bezabeh et al. 2017). In the USA, the "NHERI Tall wood Project" was launched in 2016 to develop a resilience-based seismic design guideline for tall timber buildings (Pei et al. 2017). Moreover, several research projects are currently underway in different parts of the world. While most of the studies focused on material science, fire, and seismic performance evaluations, wind performance of high-rise timber buildings has scarcely been studied. Generally, tall MTBs are lightweight and more flexible than buildings made from steel, concrete or masonry. The increased flexibility limits their lateral stiffness, thus making them vulnerable to excessive along- and across-wind vibrations (Reynold et al. 2011, Popovski et al. 2014). As a result, frequent exposure to excessive wind induced vibrations can cause occupant discomfort and possible inhabitability and unserviceability of this kind of emerging buildings. Therefore, in this paper, a risk- based wind design procedure is applied to design a 102 meters tall MTB by adapting and extending the *Alan G. Davenport Wind Loading Chain* as a performance-based wind engineering (PBWE) framework.

2 ADAPTING THE WIND LOADING CHAIN AS A PROBABILISTIC PBWE FRAMEWORK

Current prescriptive building design approaches consider designing for single limit state by accounting uncertainties through safety coefficients to achieve minimum safety and acceptable serviceability levels. Efforts to improve designs for fixed limit state started decades ago in the earthquake engineering community after an enormous amount of monetary losses due to the 1989 Loma Prieta and 1994 Northridge Earthquakes in California. Performance based engineering considers a range of limit states (performances objectives) throughout the lifetime of the structures to make risk-based design decisions. In wind engineering, suggestions to design structures for different limit states was first introduced by Davenport (1970). The identified limit states are ultimate strength, permanent deformation, excessive acceleration (occupant discomfort), and integrity of the cladding and finishing materials.

Performance based engineering requires accurate models of hazard, hazard-structure interaction, structural properties, criteria, and consequences. In 1961, the late Alan G. Davenport introduced a mathematical, somewhat philosophical, model to evaluate the wind load on structures similar to the current context of performance-based engineering. He coined his thought process as interconnected chains and named the performance evaluation framework as "*Wind Loading Chain*" (Davenport 1982). The weakest link in the chain determines the final response and reliability of the system. Davenport's *Wind Loading Chain* laid the foundation for the modern wind engineering and provides a theoretical basis for many building codes and standards. In 2011, the International Association of Wind Engineering (IAWE) recognized the *Alan G Davenport Wind Loading Chain* as an official wind engineering terminology (Isyumov 2012). For the performance assessment and design of structures, the *Wind Loading Chain* (Figure 1) starts by modeling the local micro-climate of a target site to predict the design wind speeds. Statistical analysis of historical wind speed data or computer simulations can be used to determine the design wind speeds (Davenport 1970). The approaching wind flow around the building site is affected by the terrain roughness and local topography. These effects are accounted through the second element of the Wind Loading Chain. Building aerodynamics (or simply building shape effects) is also another factor that significantly affects the wind loads and responses of structures. Wind tunnel tests and computational fluid dynamics can be used to study bluff body aerodynamics (Davenport 1970, Bitsuamlak 2006, and Bitsuamlak and Simiu 2010, Irwin et al. 2013, Dagnew & Bitsuamlak 2014). Davenport (1967) developed a framework to quantify the dynamic response of structures to wind using random vibration theory. In the framework, he suggested an approach to linearize the dynamic wind force equation and to perform dynamic analysis under wind in the frequency domain. Finally, the performance of the structure could be judged by comparing the peak response demands with criteria from codes and standards.

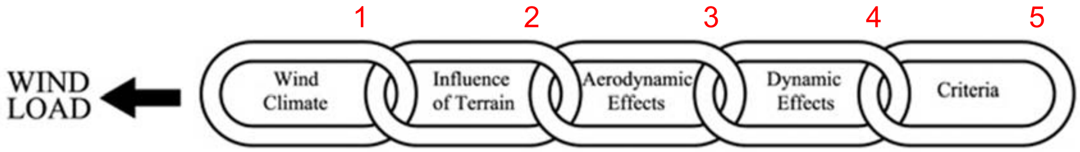
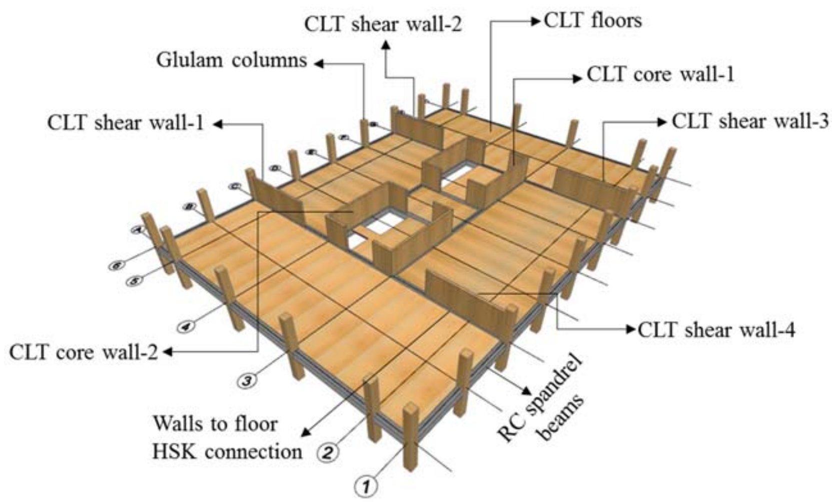


Figure 1: The Alan G. Davenport *Wind Loading Chain*

Even though, the Alan G. Davenport Wind Loading Chain framework can be used to design and assess tall buildings for different limit states, the current wind design codes and standards only consider the linear-elastic capacity of structural systems. This is mainly due to the non-load reversal nature of wind load (uni-directional mean component), issues with damage accumulation and duration of wind storms. To accurately assess the performance of structures under wind load, studies should aim at understanding the non-linear behaviour under wind load and developing wind damage states, fragility and consequence curves related to wind hazard. Towards this, a collaborative research is currently underway between the University of British Columbia (UBC), Western University, and FPInnovations to develop a unified PBWE framework by adapting the Wind Loading Chain. The research program includes several wind tunnel tests of tall MTB models subjected to both synoptic and non-synoptic (tornadoes) wind loads. Different performance and damage limit states are being developed using both linear and non-linear aeroelastic wind tunnel tests. The first part of the collaborative research, i.e., the application of risk-based design procedure to design tall MTBs subjected to random wind loads is presented in this paper. For this purpose, the outcome of the probabilistic serviceability performance assessment (probability of failure or vulnerability) is used as a risk measure. This paper is organized as follows. Initially, we introduce the case study 102 meters (30 story) MTB. Next, the details of aerodynamic wind tunnel tests and the obtained results are presented. Subsequently, the structural design process of the building, carried out using wind load information from wind tunnel tests, is presented with the design results. This is followed by uncertainty modeling, development of two serviceability limit states using 10- and 50-year return period wind speeds, and uncertainty propagation using structural reliability analysis. The results from reliability analysis are used to develop fragility curves for wind vulnerability estimations. Based on the results, the effects of various uncertainties are discussed, and risk-based design decisions are forwarded.

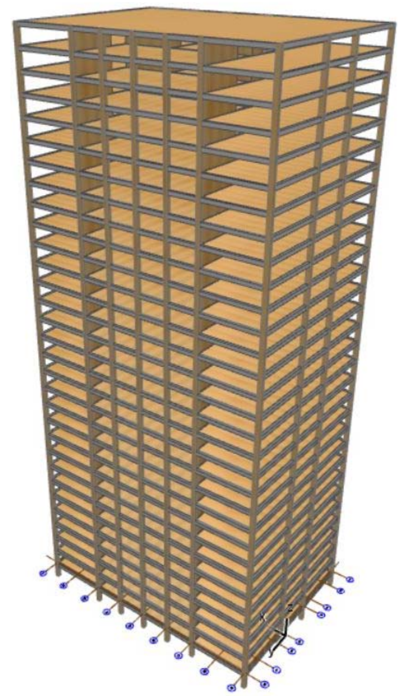
3 DESCRIPTION AND DESIGN DETAILS OF THE STUDY MASS-TIMBER BUILDING

In this paper, a 30-story mass-timber building is considered as a case study prototype building. The conceptual layout of the building was introduced in 2013 by Skidmore, Owings and Merrill (SOM LLP, 2013). The building has a typical floor plan dimensions of 30m x 42m and floor-to-floor height of 3.4m. Heavy timber products such as Glulam and CLT are used to construct both the gravity and LLRS of the building. Figures 2 (a and b) show a typical floor and 3D views of the case study building.



a)

Figure 2: The studied 30-story MTB: (a) 3D perspective view; (b) Typical floor perspective view



b)

As shown in the figure, the shear walls and floor systems are made from CLT panels. Concrete link beams are used to couple the movements of CLT shear walls resulting a global "H" shaped Lateral Load Resisting System (LLRS). As shown in Figure 2a, in addition to the two coupled CLT core walls, four single CLT shear walls provide uplift resistance due the wind load from the wider face of the building. Glulam columns are only used at the perimeter of the building to increase the gravity load share of the LLRS. In this way, it is possible to increase the uplift resistance of the building. However, the absence of the interior columns would require larger span floor systems due to increased floor deflection demands. Therefore, end-rotational restraint is provided by concrete spandrel perimeter beams. Concrete elements are extended from the RC link beams to the end spandrel beams to act as a concrete joint. A modified Holz-Stahl-Komposit (HSK) system (Zhang 2017) is used to connect the CLT walls and floor systems to the concrete joints. The modified HSK system is designed and used as shear and hold-down connectors to resist storey level wind forces and net uplift forces, respectively. Additional epoxy coated vertical reinforcement bars are also provided at the boundary elements of the CLT shear and core walls to increase their in-plane bending moment resistance.

3.1 Wind tunnel testing

Wind loads were quantified through aerodynamic wind tunnel tests at the Boundary Layer Wind Tunnel Laboratory (BLWTL) of The University of Western Ontario (UWO). To account for the effect of the terrain and shape of study building (second and third elements of the *Wind Loading Chain*), a study building was tested using a boundary layer flow corresponding to an open country exposure condition at 1:200 geometric scale. The aerodynamic rigid building model and test setup inside the wind tunnel are depicted in Figures 3 (a and b). Simultaneous time series of pressure fluctuations were measured using 495 taps installed on the models. The tests were carried out for 36 wind angles of attack (AOA) in 10 degrees increment and digitalized at the rate of 400 Hz. The time series of pressure fluctuations were converted to non-dimensional pressure coefficients (*Cp*) to scale the model pressures to full scale pressure. Equation 1 was used to calculate the time series of pressure coefficients. Average dynamic pressure (q_p) at the building height is used to normalize the measured pressure readings.

a) b)

Figure 3: a) Aerodynamic model of a 30-storey MTB; b) Wind tunnel test setup at BLWTL

[1] $$C_p = \frac{P(x,y,z,t) - P_o}{\frac{1}{T}\int_o^T q_p dt}$$

where *P(x, y, z, t)* is a pressure reading on the surface of the aerodynamic model at time (t) using a tap located at *(x, y, z)* from the origin (centre of the roof) , P_o is a static pressure, *T* is test duration, q_p is a dynamic pressure measured at the height of the building using the Pitot tube.

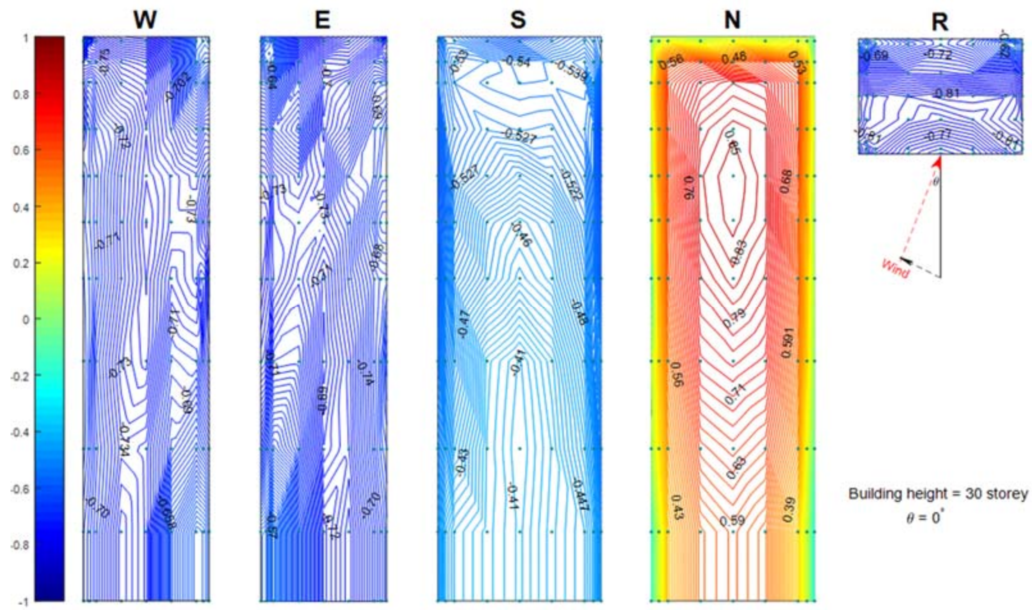


Figure 4: Mean *Cp* distribution over the surface of the building for zero-degree wind angle of attack

The mean Cp contours over the surface of the building are presented in Figure 4 for zero-degree AOA i.e., when the wider face of the model is orthogonal to the incoming wind. As shown in figure, the windward (N) wall is under positive pressure with a stagnation point around 2/3 of its height. The maximum of the mean Cp value on the windward wall is 0.88. However, all the other sides are under suction (negative pressure). The east (E) and west (W) walls (sidewalls) show a symmetric C_p distribution. To perform dynamic structural analysis and design of the study building, pressure integration over the surface of the building model was performed to compute the aerodynamic wind force time series in two sway, and torsional directions.

3.2 Structural design of the study mass-timber building

Initially, to perform the design of the structural systems of the prototype MTB, the design gravity and wind loads were quantified using NBCC 2010 (NRC 2010) and wind tunnel results, respectively. Preliminary dynamic analyses were performed to quantify the equivalent static wind loads (ESWLs) which include the inertial contribution (resonance). The details of the dynamic analysis procedure are provided in subsequent sections. For full scale wind load calculation, the building is assumed to be in the city of Chicago, USA. The design of the study building considered the axial compression, in-plane-shear, and in-plane and out-of-plane bending moment demands, and their interactions due to gravity and wind loads. Section capacities were obtained from CSA O86-14 (CSA 2014) and Structurlam CLT manufacturer catalogue. The process to design the gravity and main lateral load resisting system of the study building is outlined in the following steps. Tables 1 and 2 summarize the final design sections of the study tall MTB.

Step-1: Design of floor systems

Step-2: Design of Glulam columns

Step-3: Preliminary sizing of CLT shear and core walls for factored gravity loading

Step-4: Check the factored in-plane shear capacity of CLT walls against the wind load demand

Step-5: Check the coupled axial and out-of-plane bending of CLT shear walls

Step-6: Check the coupled axial and bending of CLT core walls

Step-7: Design of HSK connections

Step-8: Design of RC link-beams

Step-9: Design of RC spandrel beams

Table 1: Final design section of the prototype MTB

Structural System	Structural member	Design checks	Design specification
Gravity load resisting system	CLT floor system	-	7 layers-245E-E1M5
	Glulam column	axial	945 mm x 945 mm
	Spandrel beam	-	400 mm x 500 mm
Lateral load resisting system	CLT shear walls	axial in-plane shear axial + out-of-plane bending	9 layers-315E-E1M5
	RC link-beam	Shear stress limit	405 mm x 650 mm
	CLT core walls	axial axial + out-of-plane bending	9 layers-315E-E1M5

Table 2. Details of the designed shear connectors for the first floor the building

Structural system	Shear connecters profile			
	Total number of SL	Number of shear connectors	Number of plates inside CLT	SL per plate
CLT shear wall-1	900	3	3	100
CLT shear wall-2	900	3	3	100
CLT shear wall-3	900	3	3	100
CLT shear wall-4	900	3	3	100
CLT core wall-1	2100	8	3	90
CLT core wall-2	2100	8	3	90

4 DYNAMIC STRUCTURAL ANALYSIS IN FREQUENCY DOMAIN

To quantify the wind load demands, functional relationships are needed to connect the elements of the *Wind Loading Chain*. For this purpose, random vibration theory was used to quantify the response of buildings subjected to stochastic wind load. In random vibration theory, the dynamic response of buildings can be evaluated using linear modal analysis in the frequency domain. In the analysis, by simplifying the case study building as uncoupled stick-mass model, full scale wind force time series were applied at each floor in two sway and torsional directions. During the analysis, the dynamic properties were represented by the mechanical admittance factor at each floor. Modal responses were combined by considering the intermodal cross correlation due to statistical coupling effects between modes. Peak responses were computed using up-crossing rate concept by assuming the responses are Gaussian. Peak vector resultant responses were calculated from the sway mode responses using mean square addition approach. The statistical correlation between the sway modal responses was accounted by the joint action factor.

5 UNCERTAINTY MODELING

Several researches showed the influence of other uncertainties in the wind performance assessment of tall buildings (Minciarelli et al. 2001, Diniz and Simiu 2005, Bashor et al. 2005 and Bernardini et al. 2014). In this section the details of the final step of the *Wind Loading Chain,* i.e., criteria are presented in a probabilistic framework. Unlike the current prescriptive building code approaches, PBWE requires accurate models of uncertainties that affect the final performance of the structure. In PBWE of tall buildings, the stochastic nature of the wind field, wind tunnel test procedures, structural properties, and human perception of motion are the main sources of uncertainties. In this paper, 15 random variables are used to model these uncertainties. The uncertainties in the wind field were introduced by three random variables (ζ_1, ζ_2, ζ_3, ζ_7) in the power-law wind profile equation. Aerodynamic uncertainties in the wind tunnel tests were modeled as non-dimensional random variables (ζ_4, ζ_5, ζ_6). Uncertainties in structural properties were represented by modeling the first 3 building frequencies (ω_1, ω_2, ω_3) and structural damping values (ξ_1, ξ_2, ξ_3) as random variables using lognormal probability distributions. Fundamental frequencies from Eigenvalue analysis were used as a mean value with Coefficient of Variation (COV) equals to 0.01. Table 3 summarizes the parameters and type of probability distributions to model the considered uncertainties.

6 STRUCTURAL RELIABILITY ANALYSIS

Structural reliability analysis through sampling was used to propagate the uncertainties through the *Wind Loading Chain* to quantify the probability of exceeding the habitability and deflection criteria of the NBCC (2010). The resultant horizontal (PFA), a_r and lateral displacement (u_r) responses are considered as engineering demand parameters. Two limit states (performance functions) are defined as follows for vector of random variables (X) as follows:

[2] $$g\left(x, a_r(x)\right) = HC - a_r(x)$$

[3] $$g\left(x, u_r(x)\right) = DC - u_r(x)$$

Equation 2 represents the habitability limit state as a function of the criteria and the horizontal resultant PFA under 10-year return period mean wind speed. The excessive deflection limit state is given in Equation 3 as a difference between the uncertain criteria and 50-year peak resultant lateral displacement. Since the first two modes of the structure are translation, both the horizontal PFA and peak lateral displacement occur at the top floor of the building. The probability of exceeding the limit states, probability of failure, (P_f) in Equation 2 and 3 can be obtained from the joint probability density function of all random variables $f_X(x)$ using the multi-fold integral as follows.

[4] $$P_f = \int \Lambda \int_{D_f = \{x:\, g(x, a_r(x)) \le 0\}} f_X(x) dx$$

Equation 4 can be solved using both analytical approximations and/or simulation techniques. Simulation techniques are attractive for problems that involve many random variables and offer relatively accurate results at the expense of computational cost (Kareem 1988). Therefore, in this paper structural reliability analysis uses a mean centered Monte Carlo sampling technique to quantify the exceedance probability (P_f). Mean centered Monte Carlo simulations were carried out for two principal wind directions (0 and 90 degrees) and six mean critical damping ratios, ξ, (0.25%, 0.5%, 1%, 1.5%, 3%, and 5%), and mean wind velocity at the building height in the range of 10-65 m/s.

Table 3. Parameters of random variables

	Source of uncertainty	Random variable	Mean	COV	Distribution type	Citation
Wind hazard uncertainties	Observation and analysis of wind speed	ζ_1	1	0.025	Truncated normal	Minciarelli et al. (2001)
	Conversion factors between different averaging times	ζ_2	1	0.05	Truncated normal	Minciarelli et al. (2001)
	Characterization of mean wind speed profile	ζ_3	1	0.05	Truncated normal	Bashor et al. (2005)
	Wind speed from ASCE 7-10	ζ_7	ASCE 7-10 wind speed map	0.07	Normal	Bernardini et al. (2014)
Aerodynamic uncertainties	Aerodynamic errors in during wind tunnel testing	ζ_4	1	0.05	Truncated normal	Minciarelli et al. (2001)
	Approximations during pressure integration	ζ_5	1	0.05	Truncated normal	-
	Accuracy of similitude concept in the wind tunnel testing	ζ_6	1	0.05	Truncated normal	Minciarelli et al. (2001)
Structural properties	Building frequency	ω_1, ω_2, ω_3	Eigenvalue analysis	0.01	Lognormal	Bashor et al. (2005)
	Critical damping ratio	ξ_1, ξ_2, ξ_3	0.25%,0.5%, 1%1.5%, 3%, 5%	0.3	Lognormal	Bashor et al. (2005)
Criteria	Habitability criteria	HC	25milli-g	0.2	Lognormal	Bashor et al. (2005) and NBCC (2010)
	Deflection criteria	DC	(Building height)/500	0.1	Lognormal	NBCC 2010

each analysis, 2.5E5 random samples were utilized. The results of the Monte Carlo simulations were used to develop fragility curves, i.e., plots that show the probability of exceeding the serviceability limit states conditioned on wind velocity. These curves are important to estimate the vulnerability and associated financial costs of the case-study MTB. Figures 5 (a and b) present the fragility curves for habitability limit state for wind AOA 0 and 90 degrees, respectively. In general, the effect of the critical damping ratio is also observed in the fragility curves where vulnerability is higher for low critical damping ratio. For example, for mean wind speed at the building height (V_H) of 30 m/s and critical damping ratio of 1.5%, the failure probabilities of 0^o and 90^o wind directions are 66.7% and 45%, while for 5% critical damping ratio the failure probabilities are 40% and 10%, respectively. According to Bashor and Kareem (2009), major complaints might occur when failure probabilities exceed 54%. Therefore, by increasing the critical damping from 1.5% to 5%, it is possible to avoid occupant complaints due to excessive building's motion.

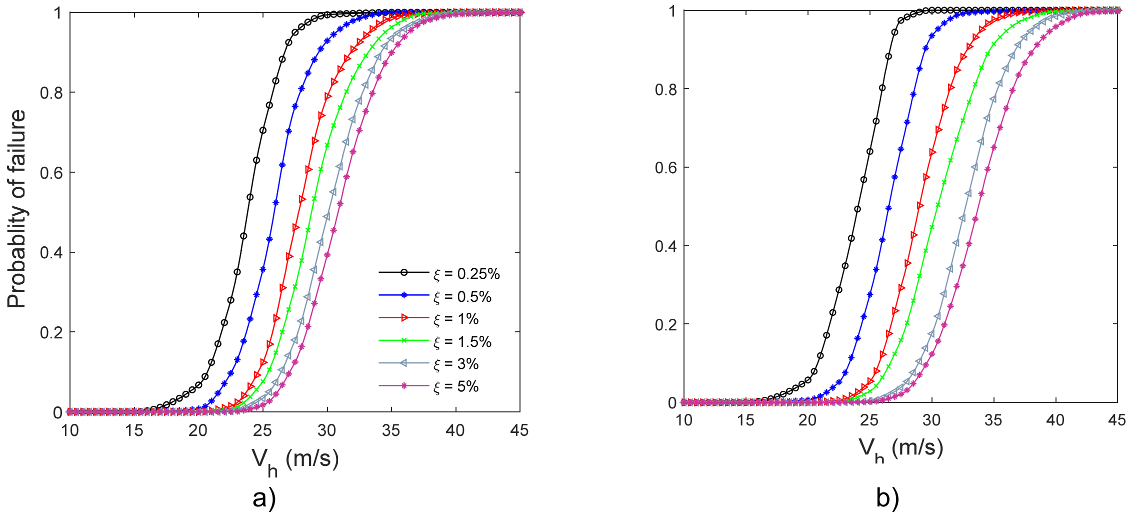


Figure 5: Fragility curves for habitability limit state: a) wind AOA = 0^o, b) wind AOA = 90^o

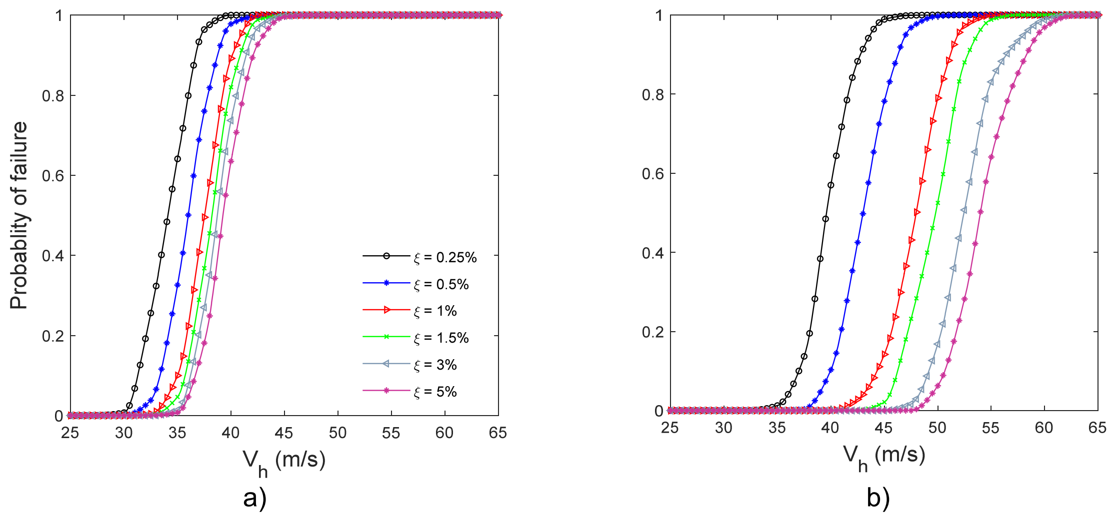


Figure 6: Fragility curves for deflection limit state: a) wind AOA = 0^o, b) wind AOA = 90^o

Fragility curves were also developed for deflection limit state. Figures 6 (a and b) show the variation of failure probability with critical damping ratio (ξ) and wind AOA. As expected the probability of failure is higher for wind AOA = 0^o, as the worst wind direction matches with the weak axis of the building. The fragility curves for wind AOA = 90 degrees are flat and in general vulnerability is smaller as compared to wind AOA = 0 degrees. The obtained results from Monte Carlo simulations consistently show the dependence of wind vulnerability on critical damping ratio and wind direction.

Irrespective of the wind angle of attack, under the 1-in-50 -year wind storm, the probability of exceeding the H/500 drift limit of NBCC 2010 (NRC 2010) is very small. Therefore, the performance of the study building can be enhanced by using damping mitigations instead of increasing the lateral stiffness. For example, it is possible to reduce the failure chance by ~27% by increasing the critical damping ratio from 1.5% to 5%. This kind of damping enhancements can be achieved by using both passive and active supplemental damping systems. To reduce excessive wind induced vibrations, tuned-mass dampers (TMDs) successfully applied in tall buildings (such as Taipei-101 in Taiwan, Citicorp Center in New York, and John Hancock Tower in Boston). Similar external damping systems can be designed and optimized using the results of this paper to reduce the human discomfort risk in MTBs. The presented risk-based design and analysis framework can be used to make risk informed decision during the design of tall MTBs. By considering the reliability of the building as decision criteria, different hybridization techniques and external damping systems can be analyzed, optimized and compared using the proposed framework.

7 CONCLUDING REMARKS

In this paper, a risk- based wind design procedure is applied to design a 102 meters tall MTB by adapting and extending the *Alan G. Davenport Wind Loading Chain* as a probabilistic PBWE framework. Initially, aerodynamic wind tunnel tests were carried out to obtain transient wind load information. Subsequently, using the wind tunnel data, the study MTB was structurally designed. In the risk-based performance assessment, two limit states were considered, i.e., habitability and deflection. The 1-in-10-year resultant horizontal (PFA) and 50–year lateral deflection responses were considered as engineering demand parameters. These limit states were established based on the habitability criteria and deflection limit of the *2010 National Building Code of Canada*. Uncertainties related to the nature of the wind field (hazard), wind tunnel tests, structural properties, and human perception of motion were explicitly modeled as random variables using probability distributions. Structural reliability approach was used to propagate the considered uncertainties through the *Wind Loading Chain* to quantify the probability of exceeding the NBCC 2010 criteria. Mean centered Monte Carlo simulations were carried out for six levels of mean critical damping ratios and two principal wind directions. In total, 9E06 simulations were carried out to calculate the probability of exceeding the criteria (failure probabilities). The results revealed the dependence of the exceedance probability on the wind direction and critical damping ratio. For both limit states, wind vulnerability increases when the critical damping ratio decreases.

The results of the structural reliability analyses indicated the possibility of reducing the risk of unserviceability (possible complaints of occupants) by adding supplemental structural damping to the study tall MTB. Irrespective of the wind angle of attack, under the 1-in-50 -year wind storm, the probability of exceeding the *H/500* drift limit of NBCC 2010 (NRC 2010) is small. Therefore, the performance of the study building can be enhanced by using damping mitigations instead of increasing lateral stiffness. For example, it is possible to reduce the failure chance by ~27% by increasing the critical damping ratio from 1.5% to 5%. This can be achieved by using both, passive and active supplemental damping systems such as TMDs. It should be noted that due to the strong dependence of the wind response of structures on shape (aerodynamics), structural properties, surroundings, and hazard level, the obtained results are case specific. However, the presented risk-based design and analysis procedure can be adopted to make risk informed decisions during the design of other types of mid- and high-rise buildings.

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