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LONG-TERM MONITORING OF TALL MASS TIMBER BUILDINGS – EVALUATION OF DYNAMICS PROPERTIES

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ABSTRACT: Due to the lightweight nature of wood construction, wind excitation induces vibration with a larger amplitude when compared to buildings built with heavier materials, such as steel and concrete. The design method provided in the National Building Code of Canada requires buildings' natural frequencies and damping ratios as input. However, there is little data available for mid-to high-rise timber buildings. Therefore, FPInnovations launched a research project to measure mid-to high-rise timber buildings' natural frequencies and damping ratios in attempt to expand the database and validate or adapt the existing equations to estimate the natural frequencies. Two high-rise timber buildings were equipped with an anemometer and accelerometers and are constantly monitored to study how the wind excites high-rise timber buildings. The measurements suggest that the Arbora building and the Origine building have a modal damping ratio of about 2.5%.

KEYWORDS: Tall wood building, Wind-induced vibration, Damping ratio, Empirical mode decomposition (EMD), ambient-vibration test (AVT)

1 INTRODUCTION

Mass timber construction is known to be lightweight and has lower lateral stiffness in comparison to steel or concrete construction. These properties make the building more susceptible to wind-induced vibration in high-rise construction [1, 2]. Excessive vibration of buildings can cause occupants discomfort and damage to finish materials, services, and equipment in the building [3, 4]. These kinds of vibrations are not safety-related issues but may affect the market acceptance of mid- to high-rise timber buildings. Despite the challenge that comes with designing high-rise mass timber (MT) buildings, there is an international interest in building with wood, as it is more sustainable compared to concrete or steel [5]. However, there are still some unknowns regarding the dynamic properties of tall MT buildings under wind-induced vibration.

To evaluate the building response to wind loads, Davenport [3] has developed a process named wind load chain, illustrated in Figure 1, which laid the foundation of modern wind engineering and provides a theoretical basis for many building codes and standards.

Figure 1: *Wind loading chain of Davenport [3]*

As shown in Figure 1, the first step is to study the climate of the target site to define the design wind speed. This design wind speed could be directly taken from the climatic table in the National Building Code (NBC) in Canada [6], or it could be estimated from reliable anemometers. The wind flow around the building site is influenced by the terrain which is, in general, characterized by the roughness (e.g., vegetation, buildings, etc.) and the local topography. The third element of the wind load chain is the aerodynamic effect which is directly influenced by the building shape. This element is generally studied in boundary layers wind tunnels or with computational fluid dynamics (CFD). CFD can only be used for pre-design or for research since the NBC directly states that CFD cannot be used independently of boundary layers wind tunnel procedures. The fourth element of the wind load chain is the dynamic effect. This element is influenced by the dynamic characteristic of the wind and the dynamic properties of the structures. Since tall MT structures are a relatively new type of structure, there are several unknowns regarding its dynamic properties, especially the modal damping ratio [2]. The last element is the criteria which are generally defined by codes and standards.

The study of Bezabeh et al. [2] illustrated the impact of different modal damping ratios on the top floor acceleration caused by induced wind vibration. Several building heights from 10 to 40 storeys were studied with damping ratios from 1% to 10% in different exposure conditions, such as open country, suburban, and urban. The results of this study have shown the importance of the damping ratio on the evaluation of the top floor acceleration.

To fill the knowledge gap in the modal damping ratio of high-rise MT buildings, several European universities and

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institutions launched the DynaTTB initiative in attempt to quantify these properties [7]. In Canada, FPInnovations started a similar research project aiming at quantifying the acceleration and damping ratio under different wind conditions of two existing high-rise MT buildings. As such, two high-rise MT buildings have been equipped with an anemometer and accelerometers to study how the wind excites the buildings. The first building is named Origine and is located in Québec City, and the second is named Arbora and is located in Montreal, both in the province of Québec.

2 BUILDING DESCRIPTION

2.1 ORIGINE BUILDING

Origine is a 13-storeys with a height of 40.0 metres completed in 2018 and designed by Nordic Structures. It is located in Quebec City in Canada and its near surroundings are a green park on the north side, a river on the west side, and low-rise buildings or no buildings on the south and east side, as shown in Figure 2.

The first level consists of a concrete podium, while the twelve other levels are made of MT elements (glulam and CLT). The gravity system consists of a post-and-beam glulam frame and 175 mm 5-ply CLT floor slabs. The typical clear span of the floor is 5.5 m. This type of floor can be considered as a semi-rigid diaphragm.

The lateral load resisting system (LLRS) consists of balloon framed CLT shear walls with vertical glulam elements at the ends of the shear wall. CLT and glulam extend from the concrete podium floor to the roof. The CLTs are either 9-ply 291 mm, 7-ply 244 mm, or 5-ply 175 mm panels, depending on the location and magnitude of the lateral force. For that type of LLRS, the lateral load is essentially taken by the CLT, and the uplift load is resisted by the glulam. The hold-down connectors consist of steel plates with dowels fastened into the wood. The horizontal joints between the CLT panels are connected with steel plates and dowels fastened into the wood. The vertical joints are connected with shear keys, as shown in Figure 3. These connectors are stiffer and stronger compared to traditional fasteners, such as nails, screws, or dowels. To ensure good contact between the two panels, self-tapping screws were inserted diagonally in addition to the shear keys. Several architectural and gravity CLT panels were also installed in the building which certainly contributes to stiffening the building laterally.

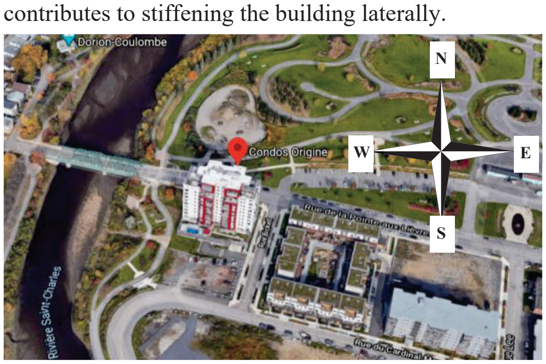

Figure 2: *Surrounding of the Origine building*

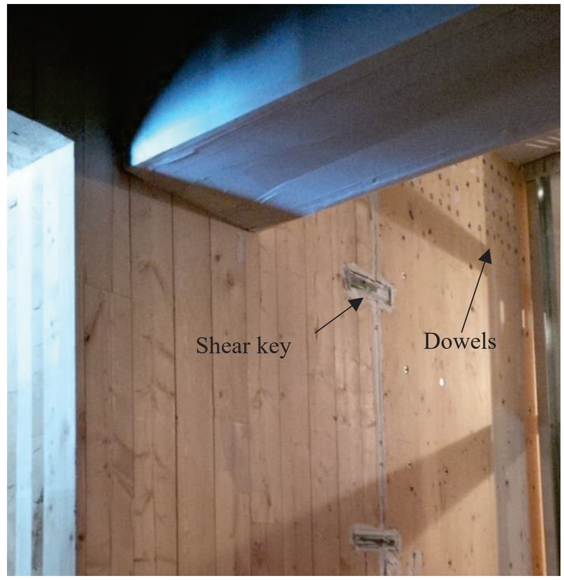

Figure 3: *Shear wall with shear keys and a hold-down with dowels*

2.2 ARBORA BUILDING

Completed in 2017, Arbora building is 8 storeys tall with a height of 24.5 metres and designed by Nordic Structures. It is located downtown Montreal in Canada and its near surroundings are buildings with a similar height except on its south side where it is a large one storey industrial building as shown in Figure 4.

The first level is a concrete podium, and the seven subsequent levels are made of MT (Glulam and CLT). The gravity system consists of a post-and-beam glulam frame and 175 mm CLT floor slabs. The typical floor clear span is 5.5 m. As with Origine, this type of floor creates a semi-rigid diaphragm effect. The LLRS are balloon framed CLT shear walls and elevators cores that extend from the concrete podium floor to the roof. The CLT shear walls are made of either 7-ply 244 mm or 5-ply 175 mm panels, depending on the location and magnitude of the lateral force. The uplift force is resisted by steel plates anchored to the CLT using ring shank nails, as shown in Figure 5. The vertical and horizontal joints between panels were designed with steel plates with ring shank nails. This kind of connection forms a flexible connection between CLT panels.

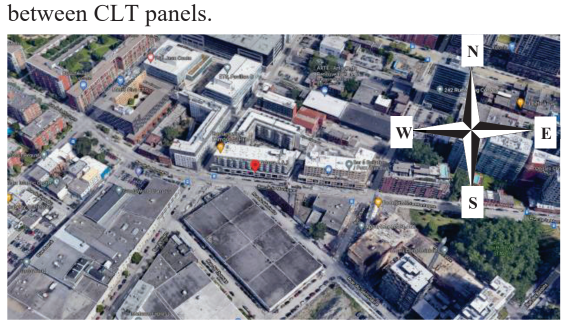

Figure 4: *Surrounding of the Arbora building*

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Figure 5: Shear wall with hold-down connected to the concrete mezzanine

3 APPROACHES

3.1 Position and location of the devices

Both buildings are being constantly monitored with a set of accelerometers fixed on the ceiling of the top storey and an anemometer located on the roof of the building to roughly estimate the wind speed experienced by the buildings. The monitoring started in 2018 and is still ongoing.

The anemometer on both buildings is from Young Company, and the model used is 05103VK, which offer an output signal voltage output from 0 to 5V for wind speed from 0 km/h to 200 km/h (0 m/s to 55.5 m/s). The anemometer also has a mechanical range of 360° for the wind direction and an output signal of 0 V to 5 V. The location of the anemometer is shown on Figures 6 to 8. It must be understood that the wind flow around a building can be greatly influenced by the structure itself [8, 9]. This means the wind speed measurement cannot be taken as a precise value of the wind speed experienced by the building, namely if the anemometer is in a turbulent region. The analysis of the acceleration responses of the building with respect to the wind speed and direction needs to be made with precautions. Thus, the value measured by the anemometer only gives an indication of how much wind the building experiences. In attempt to limit the turbulence effect, the anemometers were positioned at about 2.5 m above the roof stair entrance (Figure 8).

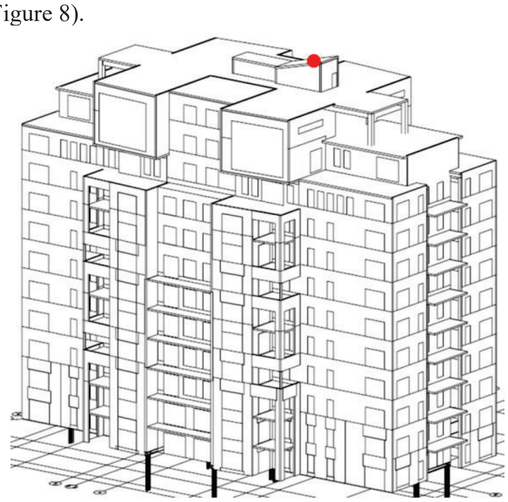

Figure 6: Location of the anemometer on the Origine building

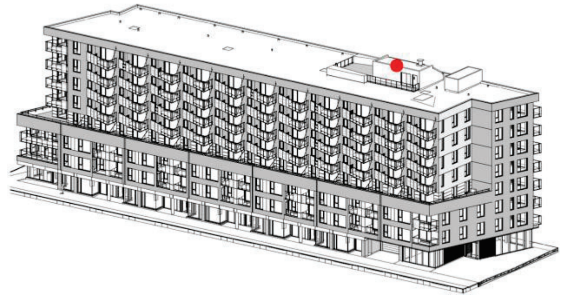

Figure 7: Location of the anemometer on the Arbora building

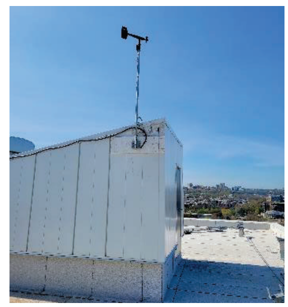

Figure 8: Photo of the anemometer installed on the Origine building

The accelerometers used to monitor the building acceleration are model 626B02 from PCB Piezotronics that uses the ICP® signal technology. The main benefit of an ICP® signal in this project is the low output impedance (<100 Ohms) which allows signals to be transmitted over long cables through harsh environments with minimum loss in signal quality [10]. The model 626B02 has a low frequency sensitivity ranging from 0.2 Hz to 6000 Hz and a measuring acceleration range of ± 10 g (± 98 m/s^2). The output signal of the accelerometer range is ± 5 V which gives a sensitivity of 500 mV/g (51 mVs^2/m).

Figures 9 and 10 show the plan locations where a pair of accelerometers have been fixed to measure the acceleration in the x and y directions. They have been fixed on the ceiling of the top floor. Ideally, these accelerometers should have been installed at the corners of the building, but this location would have been within a unit, and if for any reasons these accelerometers would have needed to be accessed, it would have been impossible. Therefore, the accelerometers were installed in a common area hidden in the ceiling.

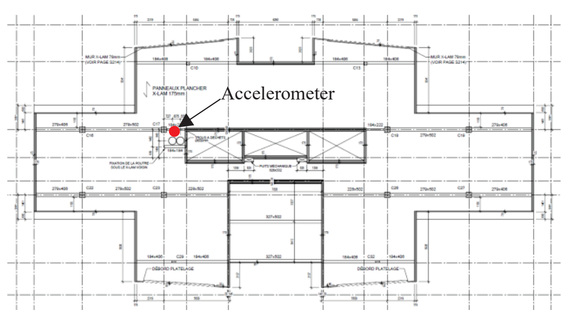

Figure 9: Location of the accelerometers on the ceiling of the top floor for the Origine building

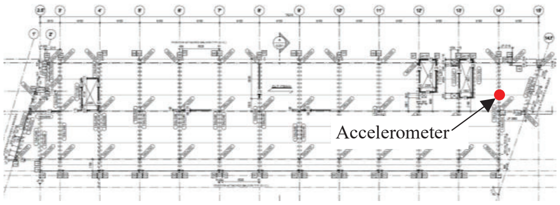

Figure 10: *Location of the accelerometers on the ceiling of the top floor for the Arbora building*

3.2 Data analysis

The data from the accelerometers and the anemometers are acquired at a frequency of 100 Hz permanently. The data are saved in a binary format in blocks of 6 hours. All the acquired data are then screened with a python script to extract meaningful 2-minute data. The trigger to extract the 2-minute window data is based on the 3 seconds average wind speed. The following rules are followed to extract the data for the Origine buildings:

1. If the maximum 3 seconds average wind speed of the 6 hours data block is below 10 m/s, no data are extracted;
2. If the maximum 3 seconds average wind speed of the 6-hour data block is between 10 and 15 m/s, one 2-minute data block is extracted;
3. If the maximum 3 seconds average wind speed of the 6-hour data block is between 15 and 20 m/s, three 2-minute data blocks are extracted unless if the maximum 3 seconds average wind speed of the subsequent data blocks is lower than 15 m/s for which no block is extracted;
4. If the maximum 3 seconds average wind speed of the 6-hour data block is higher than 20 m/s, five 2-minute data blocks are extracted unless if the maximum 3 seconds average wind speed of the subsequent data blocks is lower than 20 m/s for which only three blocks are extracted, or if the maximum 3 seconds average wind speed is lower than 15 m/s for which no data block is extracted.

For the Arbora building, the following rules are used:

1. If the maximum 3 seconds average wind speed of the 6-hour data block is below 10 m/s, no data are extracted;
2. If the maximum 3 seconds average wind speed of the 6-hour data block is between 10 and 15 m/s, two 2-minute data blocks are extracted;
3. If the maximum 3 seconds average wind speed of the 6-hour data block is between 15 and 20 m/s, ten 2-minute data blocks are extracted unless if the maximum 3 seconds average wind speed of the subsequent data blocks is lower than 15 m/s for which only two blocks are extracted;
4. If the maximum 3 seconds average wind speed of the 6-hour data block is higher than 20 m/s, 2-minute data blocks are extracted with no limit on the quantity until the maximum 3-seconds average wind speed of the subsequent data blocks is lower than 20 m/s for which the previous rule applies.

Two different extraction rules were used to have sufficient data for both buildings. The Arbora building is less exposed to wind compared to the Origine building. Therefore, less data during high wind speed events is available.

As described in the previous subsection, the sensitivity of the accelerometers used for the acquisition is 500 mV/g. These accelerometers are considered very sensitive but much more sensitive accelerometers exist at 10,000 mV/g. With the current accelerometers, if the maximum acceleration experienced by the building is the limit prescribed by the NBC for the residential building, 15 milli-g, the actual measurement over the measuring capacity will only be 0.15%. Therefore, noise in the measurement is almost inevitable in this case. In order to extract reliable dynamic properties of the building, the method to analyse the data needs to be carefully chosen.

The load experienced by the building is unknown, thus the modal identification technique used to process the signal of the accelerometer needs to be able to analyse output data only. Over the past decades, different time-frequency methods, including wavelet transform (WT) [11, 12], blind source separation (BSS) [13], and empirical mode decomposition (EMD) [14] have been used as modal identification techniques for large-scale civil infrastructure. Of all methods, EMD has the capability of using only a single channel of measurement.

There are different derivatives of the EMD method, such as the variational mode decomposition (VMD) [15], the multivariate empirical mode decomposition (MEMD) [16], and the time varying filtering-based EMD (TVF-EMD) [17]. This study utilizes the TVF-EMD method, since it can be used with only a single-channel measurement without causing mode-mixing or end effects under the presence of closely spaced modes or measurement noise. This is an important property of this technique, since the signal-to-noise ratio is small for the measurement performed in this study. Additionally, each channel is analysed independently, since the two accelerometers at the top of the building are placed orthogonally and the accelerometers at the ground are not considered during the analysis as the excitation from the ground is negligible in comparison to wind.

The EMD method decomposes the signal into a set of oscillatory waveforms known as intrinsic mode functions (IMFs). Then, the time varying filtering-based EMD (TVF-EMD) [17, 18] performs local cut-off filtering where the signal is successively filtered into local high-pass and low-pass components to decompose it into narrowband signal components (i.e., IMFs). Each IMF has a specific energy and frequency. To determine the modal damping ratio of the structure, an IMF with high energy (high RMS value) around the natural frequencies of the structure is chosen, which is represented by a red "x" and a red circle in Figures 11 and 12. This IMF is then analysed into the time domain by performing an autocorrelation to extract the damping ratio, as shown in Figure 13. The equation shown in Figure 13 is of the following form:

$$E(\tau) = Ae^{-\zeta f_n 2\pi\tau} \tag{1}$$

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where ζ is the modal damping ratio, f_n the natural frequency of the IMF, τ is the time lag, A is the amplitude and $E(\tau)$ is the envelope of the autocorrelation.

This method is performed for every 2-minute extracted data. As such, a statistical analysis of the evaluated modal damping ratio for each extracted IMFs can be performed. When extracting modal damping ratio with ambient load, the level of confidence with the modal damping ratio is generally quite low since this is a parameter difficult to evaluate [19]. The level of confidence for every single data of modal damping ratio in this study is in the same order. However, one of the originalities of this study is to have performed that analysis more than 1000 times to obtain a statistical distribution of the modal damping ratio. Therefore, the modal damping ratio of the monitored structure could be based on the average, the median, or the mode in which we have a much better degree of confidence due to the quantity of data.

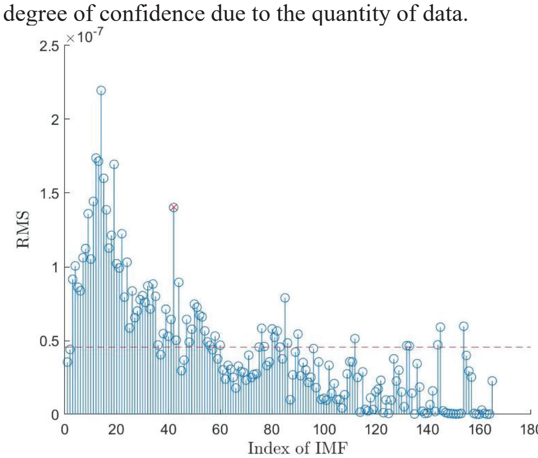

Figure 11: *RMS value of the IMFs*

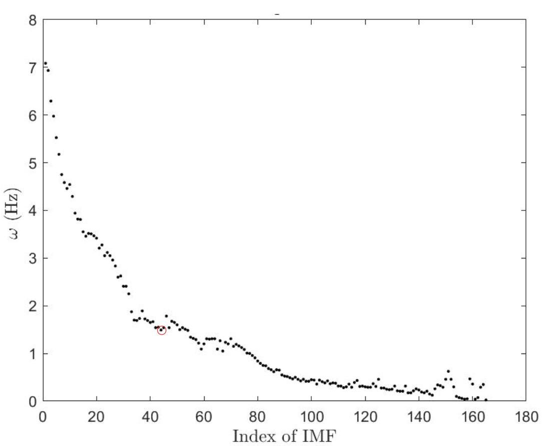

Figure 12: *Frequencies of the IMFs*

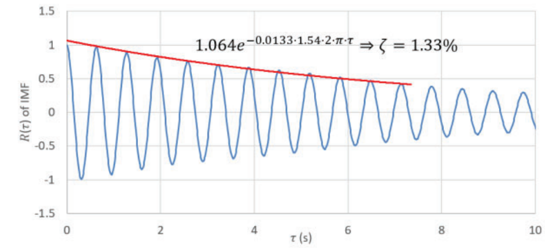

Figure 13: *Autocorrelation of the chosen IMF to evaluate the damping ratio*

4 RESULTS AND DISCUSSION

This section presents the results from the statistical analysis of the extracted modal damping ratio. The monitored buildings are equipped with two uniaxial accelerometers fixed on the ceiling of the top floor. One accelerometer is measuring the acceleration along the short axis of the building, while the other is measuring the acceleration in the orthogonal direction, i.e., along the long axis.

From the monitored data, several 2-minute window data are analysed to extract the damping ratio with a TVF-EMD algorithm [17]. Figures 14 to 17 show the probability density of the extracted damping ratio. It should be noted that, since evaluating the modal damping ratio under ambient loading is very difficult, the reliability of a single measurement for the modal damping ratio is relatively low [19] and therefore, using this method in which there is a statistical distribution of the modal damping ratios is recommended. Analysing more data and then performing statistical analysis of these data is a way to increase the reliability of the damping ratio value assigned to the building, as long as there is no systematic error in the method employed to extract the damping ratio.

It can be observed from Figures 14 to 17 that the lognormal distribution fits the measured damping ratio distribution very well. As a property of lognormal distribution, the probability density below the mode, i.e., highest probability density, decreases rapidly. It means that a very low modal damping ratio is less probable than a high damping ratio. Therefore, the average and the median are always higher than the mode. Tables 1 and 2 provide these estimated values for all the data points and for high wind events. The threshold for high wind events has been chosen only to have enough 2-minute window data under relatively high wind events. Since Origine is more exposed to wind, there is more data available for this building under these wind events. Consequently, threshold value is 18 m/s for Origine and 15 m/s for Arbora. From those tables, it can be observed that the extracted modal damping ratio under high wind speed is similar to the damping ratio extracted under wind speed higher than 10 m/s. This is an important observation, since it indicates that the building is still experiencing a linear behaviour under different wind conditions. Buildings are designed to remain elastic under wind load, but the connections between panels generally exhibit non-linear behaviour even under low load. The non-linear behaviour

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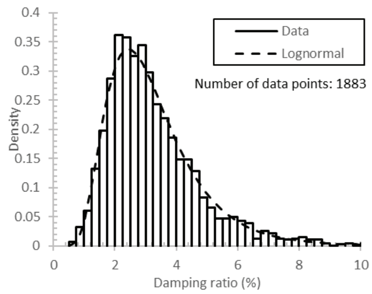
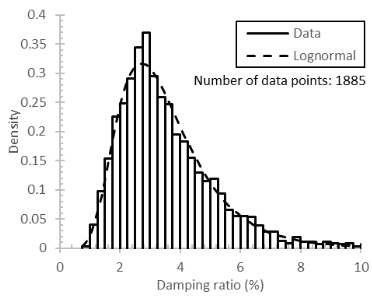
of these connections does not seem to influence the damping ratio of the structures under wind loads noticeably.

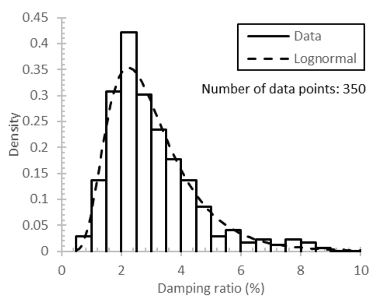


a) Along the short side of the building

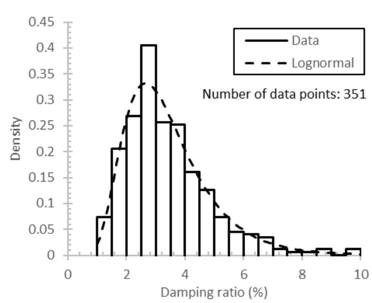


b) Along the long side of the building

Figure 14: *Statistical distribution of the modal damping ratio for wind speed higher than 10 m/s for the Origine Building*

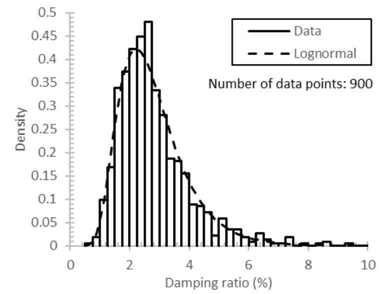


a) Along the short side of the building

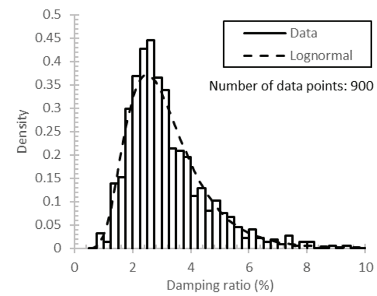


b) Along the long side of the building

Figure 15: *Statistical distribution of the modal damping ratio for wind speed higher than 18 m/s for the Origine Building*

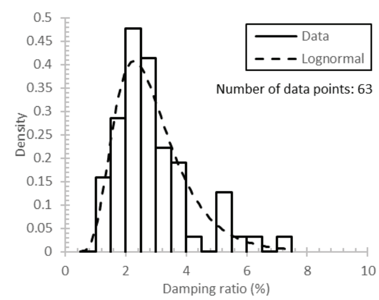


a) Along the short side of the building

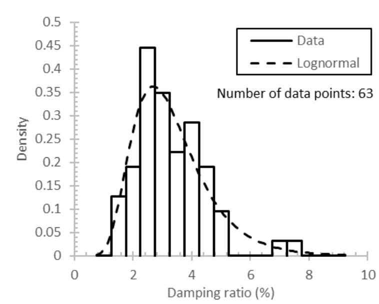


b) Along the long side of the building

Figure 16: *Statistical distribution of the modal damping ratio for wind speed higher than 10 m/s for the Arbora Building*



a) Along the short side of the building



b) Along the long side of the building

Figure 17: *Statistical distribution of the modal damping ratio for wind speed higher than 15 m/s for the Arbora Building*

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As mentioned in commentary A of the NBC when defining the stiffness of new material, such value should be based on a 50% exclusion, i.e., the median. Material stiffness is generally used for serviceability limit state verification. Top floor acceleration limitation is a serviceability limit state verification. Therefore, it is believed that the damping ratio of the Arbora and Origine building could be considered as 2.5% which is found to be slightly lower than the median under high wind speed for the fundamental frequency of both buildings. Table 1 and Table 2 present the statistical parameter of the modal damping ratio for both buildings.

Arbora and Origine have similar construction systems. The LLRS are made with balloon frame CLT, and the floors are made with CLT. However, the shape of the buildings is different. Origine has more a tower shape by being almost square, while Arbora is rectangular, and its length is much longer than its height. Since these two buildings have a similar construction material but different shapes, it seems that MT buildings with CLT balloon-framed shear walls have a damping ratio of about 2.5%.

Table 1: *Statistical parameter of the modal damping ratio for the two first natural fundamental frequency for the Origine building*

Statistical Parameter	Wind > 10 m/s f_1	Wind > 10 m/s f_2	Wind > 18 m/s f_1	Wind > 18 m/s f_2
Number of data	1883	1885	350	351
Average	3.26	3.56	3.05	3.44
Median	2.96	3.25	2.75	3.15
Mode	2.44	2.71	2.23	2.65

Table 2: *Statistical parameter of the modal damping ratio for the two first natural fundamental frequency for the Arbora building*

Statistical Parameter	Wind > 10 m/s f_1	Wind > 10 m/s f_2	Wind > 15 m/s f_1	Wind > 15 m/s f_2
Number of data	900	900	63	63
Average	2.80	3.15	2.88	3.34
Median	2.59	2.91	2.66	3.11
Mode	2.22	2.48	2.27	2.69

5 CONCLUSIONS

This study investigated the modal damping ratio value of the Origine and Arbora mass timber buildings under wind-induced vibration. The Origine building is 13 storeys with a roof height of 40.0 m, and the Arbora building is 8 storeys with a roof height of 24.5 m. These buildings were designed through the development of alternative solutions conforming to the National Building Code of Canada. Although the height of these buildings is not enough to raise serious concerns regarding the wind-induced vibration behaviour of the buildings, they can give relevant and critical data, such as modal damping ratio, that can only be evaluated on real-size buildings. This variable is critical when evaluating the dynamic behaviour of the building under wind-induced vibration.

To extract this dynamic property of the Origine and Arbora buildings, they were equipped with accelerometers fixed on the ceiling of the top floor and an anemometer fixed on the roof to almost permanently monitor the acceleration experienced by the buildings. These data were then screened to extract meaningful 2-minute data windows. These 2-minute data windows were then analysed with a time varying filtering-based empirical mode decomposition (TVF-EMD) algorithm to extract the modal damping ratio of the analysed data. A statistical analysis of the damping ratio was then performed to increase the reliability of the extracted damping ratio, since evaluating such parameters under ambient load with a single measurement has limited reliability. From these analyses it was found that the Origine and Arbora buildings have a damping ratio of about 2.5% for their fundamental natural frequencies. Therefore, for future high-rise mass-timber (MT) buildings with lateral load resisting systems (LLRS) made of cross-laminated timber (CLT), the damping ratio could be assumed around 2.5% or below. Currently, damping ratio for wind-induced vibration is usually assumed to be 1% for MT buildings. Such value is now believed to be overly conservative for high-rise MT buildings with CLT shear walls.

The next steps of this research project will be to estimate the top floor acceleration of different designs of high-rise MT buildings and then to iterate on different structural parameters to optimize the structure for reducing the wind-induced vibration. Usually, for very tall buildings such as skyscrapers, this optimization is made along with shape optimization. However, going through a shape optimization process for high-rise MT buildings may not be the most efficient and economical way, since the height in comparison to conventional steel or concrete skyscrapers is still small and only structural optimization may be sufficient. Such a study will allow defining the best strategies to reduce wind-induced vibration for MT buildings.

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REFERENCES

[1] M. Johansson, A. Linderholt, B. Åsa, K. Jarnerö and T. Reynolds, "Building higher with light-weight timber structures - the effect of wind induced vibrations," in *inter.noise*, San Francisco, California, USA, 2015.

[2] M. A. Bezabeh, G. T. Bitsuamlak, M. Popovski and S. Tesfamariam, "Dynamic Response of Tall Mass-Timber Buildings to Wind Excitation," *Journal of Structural Engineering,* vol. 146, no. 10, 2020.

[3] M. Bezabeh, G. Bitsuamlak, M. Popovski and S. Tesfamariam, "Probabilistic servability-performance assessment of tall mass-timberm buildings subjected to stochastic wind loads: Part I - structural design and wind tunnel testing," *Journal of Wind Engineering & Industrial Aerodynamics,* vol. 181, pp. 85-103, 2018.

[4] M. Popovski, M. Mohammad, C. Ni, M. Rezai, K. Below, R. Malczyk and J. Sherstobitoff, "Design and construction of tall wood buildings: Input data, testing and advanced analysis," *World Conference on Timber Engineering,* 10-14 August 2014.

[5] B. Grann, "A Comparative Life Cycle Assessment of Two Multistory Residential Buildings. Cross-Laminated vs. concrete slab and Column and Light Gauge Steel Walls," FPInnovations, Vancouver, 2013.

[6] Canadian Commission on Building and Fire Codes, "National Building Code of Canada: 2020," National Research Council of Canada, 2022.

[7] DynaTTB consortium, "DynaTTB - Dynamic Response of Tall Timber Buildings under Service Load," 2019. [Online]. Available: https://www.dynattb.com/. [Accessed 21 10 2021].

[8] A. Elshaer, H. Aboshosha, G. Bitsuamlak, A. E. Damatty and A. Dagnew, "LES evaluation of wind-induced responses for an isolated and a surrounded tall building," *Engineering structures,* vol. 115, pp. 179-195, 2016.

[9] G. Bitsuamlak, A. K. Dagnew and J. Erwin, "Evaluation of wind loads on solar panel modules using CFD," *The Fifth International Symposium on Computational Wind Engineering,* 23-27 06 2010.

[10] PCB Piezotronics, "Signal conditioning basics," [Online]. Available: https://www.pcb.com/resources/technical-information/signal-conditioning-basics. [Accessed 20 12 2021].

[11] Z. Hou, M. Noori and R. Amand, "Wavelet-based approach for structural damage detection," *Journal of Engineering Mechanics,* vol. 126, no. 7, pp. 677-683, 2000.

[12] Y. M., S. A. and L. K., "Condition assessment of structure with tuned mass damper using empirical wavelet transform," *Journal of Vibration and Control,* vol. 24, no. 20, pp. 4850-4867, 2018.

[13] A. Sadhu, S. Narasimhan and J. Antoni, "A review of output-only structural mode identification literature employing blind source separation methods," *Mechanical Systems and Signal Processing,* vol. 94, p. 415431, 2017.

[14] N. E. Huang, Z. Shen, S. R. Long, M. C. Wu, H. H. Shih, Q. Zheng, N.-C. Yen, C. C. Tung and H. H. Liu, "The empirical mode decomposition and the Hilbert spectrum for nonlinear and non-stationary time series analysis," *Proc. R. Soc. Lond. A,* vol. 454, pp. 903-995, 8 3 1998.

[15] A. Bagheri, O. E. Ozbulut and D. K. Harris, "Structural system identification based on variational mode decomposition," *Journal of Sound and Vibration,* vol. 417, pp. 182-197, 2018.

[16] N. U. Rehman, S. Ehsan, S. M. U. Abdullah, M. J. Akhtar, D. P. Mandic and K. D. McDonald-Maier, "Multi-Scale Pixel-Based Image Fusion Using Multivariate Empirical Mode Decomposition," *Sensors,* vol. 15, no. 5, pp. 10923-10947, 2015.

[17] M. Lazhari and A. Sadhu, "Decentralized modal identification of structures using an adaptive empirical mode decomposition method," *Journal of Sound and Vibration,* vol. 447, pp. 20-41, 2019.

[18] L. Heng, L. Zhi and W. Mo, "A time varying filter approach for empirical mode decomposition," *Signal Processing,* vol. 138, pp. 146-158, 2017.

[19] F. Magalhães, Á. Cunha, E. Caetano and R. Brincker, "Damping estimation using free decays and ambient vibration tests," *Mechanical Systems and Signal Processing,* vol. 24, no. 5, pp. 1274-1290, 2010.

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