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Establishing the Fundamental Period of Light-Frame Wood Buildings on the Basis of Ambient Vibration Tests

Ghazanfarah Hafeez¹, Ghasan Doudak², Ghyslaine McClure³

ABSTRACT

This research project deals with dynamic field testing of light-frame wood buildings with wood-based shear walls. The primary objective of the investigation is to evaluate the building code formula for estimating light-frame wood building's fundamental period for seismic analysis, through intensive field testing and numerical modelling. The project also aims to propose an alternative simplified rational approach to seismic analysis of these structures. The paper presents ambient vibration (AV) testing results of light-frame wood buildings in Canada. The dynamic characteristics of the measured buildings, such as natural frequency, mode shapes and equivalent structural damping were obtained from Frequency Domain (FD) analysis of ambient motion records. Using a simplified method of period estimation based on the Rayleigh approximation while using the building mass and replacing the stiffness of shear walls by their length showed reasonable fit when compared with the FE model results and AVT measured periods. A formula was developed based regression analysis of tested buildings. The expression is a function of building height, floor area and shear wall length and it was shown to provide a reasonably good fit with the measured results.

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Keywords: Timber structures; lateral drift; fundamental building period; stiffness; light frame wood buildings; seismic analysis

INTRODUCTION

The dynamic characteristics of a structure determine its behavior under loading such as wind or seismic action. Despite the transient nature of earthquakes and their dynamic effects on buildings, many building codes around the world, including the National Building Code of Canada (NBCC 2015), allow designers to analyze regular buildings (that are without significant stiffness or mass irregularities) using equivalent static load methods due to their simplicity in application. Also, the current Canadian building code provides procedures to determine wind loads on structures of different types, but no specific guidance is provided to estimate the fundamental frequency of a structure that should be considered when designing for wind load effects.

For structures having light-frame wood shear walls as their lateral force resisting system, the NBCC empirical formula for approximating the fundamental building period, T_a , for seismic load calculation, is a function of only the building height, h_n :

$$T_a = 0.05h_n^{3/4} \quad (1)$$

The NBCC also provides formulas to estimate the fundamental period of other structural systems such as steel moment-resisting frames, reinforced concrete moment-resisting frames, other moment-resisting frames, and steel braced frames, in relation to seismic design.

There are several issues associated with using the current NBCC (2015) period formula for light-

frame wood structures, including its suitability for light frame wood shearwalls and the scatter associated with the measured data even when compared with buildings on which the formula is based. The coefficients and exponents in these formulas (e.g. Equation (1)) were derived based on regression analysis using the fundamental periods measured for several buildings in California during the 1971 San Fernando earthquake (ATC 1978). The measured data used for the derivation of Equation (1) were taken on reinforced concrete shear wall buildings and large scatter was observed in the collected data when the measured period was related to only the height of the structures (Saatcioglu and Humar 2003). It is therefore questionable whether this equation can reflect the fundamental periods of buildings of other materials (such as in light-frame sheathed wood structures) and at different locations outside California where construction practices may also significantly differ.

The current study provides a comprehensive database of light frame wood buildings and develops an estimate for the building period that is more representative of this construction type. An expression for an alternative building period is proposed in this paper which provides significantly less spread in the data and incorporates key parameters that represent the building mass and stiffness, rather than simply relying on the height of buildings, which is the case in the current period formula (e.g. NBCC 2015).

The following sections provide a review of the available literature and highlight the needs for the current study. The experimental and numerical investigations are described and comparison between them established. Also, the alternative period formula is proposed and discussed.

Previous attempts to improve the code period formula

According to the NBCC, Equation (1) has general applicability to shear walls of various materials, such as concrete, steel, and wood. Over the past few decades, the adequacy of this equation has been questioned by researchers and efforts were made to suggest improvements. Lagomarsino (1993) proposed a period formula as a function of building height based on measurements made on 52 reinforced concrete buildings (including both concrete frame buildings without shear walls and reinforced concrete shear wall buildings) in Italy. Farsi and Bard (2004) performed ambient vibration tests on 35 reinforced concrete shear wall buildings located in Grenoble, France, to identify the fundamental period of these buildings. The study also provided an improved period equation as a function of building height. Gilles and McClure (2012) established a database of modal properties of 39 concrete buildings located in Montreal using ambient vibration testing. The study compared the measured fundamental periods with those calculated from the NBCC formula (Equation (1)) and suggested an improved expression which was linear with the height with coefficient of 0.020.

Other studies proposed equations for fundamental period estimate that incorporated building plan dimensions as variables in addition to building height. A study by Goel and Chopra (1998) evaluated the code period equation specified in US building codes (NEHRP-94; SEAOC-96; and UBC-1997) based on field measurements of nine concrete shear wall buildings during strong ground motions in California. There was large scatter in the measured data when compared to periods obtained by code equations. An improved formula was then suggested, which incorporated a parameter related to shear wall dimensions. Lee et al. (2000) focused on the evaluation of the period formula in the Korean building code using a data base of 50 reinforced concrete shear wall buildings whose periods were measured using ambient vibration testing. The authors proposed an alternative equation that incorporated the height of the building as well as

the wall width per unit plan area. Saatcioglu and Morales (2011) also evaluated the code period formula for concrete shear walls and frame buildings and proposed an equation that included the moment of inertia (second moment of area of the building plan) and building height. The proposed equation was derived from the data base of building periods obtained from measurements during strong earthquake motions. The equation was linear with height but non-linear with the moment of inertia, and it showed good correlation with the measured periods. A recent study by Nikoo et al. (2017) developed the Self-Organization Feature Map (SOFM) algorithm to estimate the fundamental periods of reinforced concrete shearwall buildings. The building period estimates were compared to the periods obtained using FEMA-450, ATC3-06, and the Costa Rican code equations. A database of measured periods of 78 reinforced concrete shearwall buildings was assembled based on published data and used as a reference for the comparison between the developed algorithm and the code. The study concluded that the SOFM model was better capable of predicting the fundamental period values when compared with estimates obtained from the code.

Fewer studies have been conducted on the dynamic characteristics of light-frame wood buildings. Kharrazi (2001; 2006) investigated vibration characteristics of low-rise timber buildings using ambient vibration, forced vibration, and full-scale shake-table testing. His research focused on single- and two-storey houses with different sheathing configurations using laboratory shake table testing for validation. The study established a correlation between measured periods during ambient and forced vibration tests, and resulted in empirical formulas to increase the period measured by AVT to predict the period expected during strong shaking. Kharrazi suggested two relationships for the fundamental frequencies of wood frame houses, exposed to low (Equation (2)) and high (Equation (3)) levels of seismic excitation.

$$F_V = \alpha 0.65(F_{AV})^{1.1} \quad (2)$$

$$F_V = 0.32(F_{AV})^{1.4} \quad (3)$$

106

107 Where F_V and F_{AV} are fundamental frequencies extracted from forced vibration and ambient
 108 vibration test records, respectively. The factor α in Equation (2) takes on the values of 0.76,
 109 1.03, and 1.42 for non-engineered, engineered and stucco houses, respectively.

110 The vibration behavior of wood-based buildings was also studied by Camelo (2003) who
 111 developed a database of dynamic properties through analyses of available earthquake records on
 112 five shearwall wood frame buildings instrumented by the California Strong Motion
 113 Instrumentation Program, and by performing forced vibration testing on five other buildings
 114 located in California. Eight sets of earthquake records were utilized to obtain the fundamental
 115 frequency and damping ratio of one-, two- and three-storey wood frame buildings. Two full-scale
 116 shake-table tests were also performed at University of California San Diego and University of
 117 California Berkeley to validate the proposed period formula. The compiled database, excluding
 118 the results of ambient vibration field testing, was analyzed to obtain the best fit curve for the
 119 median fundamental period, T as a function of building height, h_n (in feet). The median period
 120 formula, in Equation (4), was proposed to represent the seismic behavior of wood buildings
 121 more accurately than the formula provided in the Uniform Building Code (UBC-97).

$$T = 0.032h_n^{0.55} \quad (3)$$

Hafeez et al. (2014) investigated the period formula in the National Building Code of Canada through modeling of a light wood frame building using a finite element model. The study estimated the building stiffness using the deflection equation for wood shear walls from the current Canadian timber design standard (CSA 2014). The calculated fundamental periods using a multiple-degree-of-freedom shear-building model were compared with the AVT results of a light frame wood building. The study showed that the calculated periods based on the deflection equation with no cumulative effect and those based on the shear wall tests stiffness were consistent with the measured period values. The calculated period based on the deflection formula including the cumulative effect yielded a value that was larger than the upper limit stipulated by the NBCC (2015).

The above review of the state of knowledge clearly emphasizes the need for more investigation of the dynamic characteristics of light-frame wood buildings for seismic analysis and earthquake-resistant design. It is the aim of the current study to evaluate the adequacy of contemporary code formulas for estimating the fundamental period of timber buildings, specifically those consisting of light-frame wood shear walls, and develop a reliable expression for building fundamental period based on field testing and numerical modelling. The essential elements that define the research methodology are ambient vibration field testing, analysis of recorded AV measurements, and stiffness estimation of lateral load resisting systems.

Ideally, building period measurements would be taken during a moderate to strong earthquake to be more representative than ambient motion results for seismic design. Although these measurements would still be uniquely associated with the particular seismic load characteristics, the amplitude of building motions would be more representative of that experienced in actual design-level earthquakes. However, measurements of dynamic motion of light-frame wood

buildings during real earthquakes are rarely available, and field ambient vibration tests on a large number of representative light-frame wood buildings are conducted, as an alternative, to create a database. Numerical models of the tested buildings with their initial stiffness are developed, and once validated using the AVT measurements the models are re-analyzed with higher force values (reduced stiffness) and the fundamental periods are compared with expressions proposed in the literature. Finally, regression analysis of the periods measured by AVT is also conducted to develop a more reliable period formula to replace Equation (1) used in the NBCC for buildings with light frame wood shearwalls as lateral load resisting systems.

MEASURED BUILDINGS

In total, 47 light-frame wood buildings of both regular and irregular layouts, located in moderate to high seismic zones in Canada, were tested and analyzed. The locations, heights and plan geometries of all tested buildings are provided in Table 1.

Table 1 Buildings tested using AVT method

Building ID	Location	No. of Storeys	Height (m)	Length (m)	Width (m)	Geometry
B1	Kamloops, BC	5W+1C+B*	17.4	70	40
B2	Kamloops, BC	5W+1C+B	17.4	89	70
B3	Kamloops, BC	5W+1C+B	18.2	184	20
B4	Quebec city, QC	6W	18	51.8	19.9
B5	Kamloops, BC	4W+1C	18.1	60	24

B6	Orleans, ON	4W+B	14.2	44	16.5
B7	Ottawa, ON	4W+B	14.9	65	33
B8	Vancouver, BC	4W+B	14.9	25	17
B9	Fredericton, NB	4W+B	13.6	51.4	41
B10	Okanagan, BC	5W	16	30	12
B11	Okanagan, BC	4W+1C	16	35	31
B12	Vancouver, BC	3W+2C	20.2	55	26
B13	Kamloops, BC	2W+3C	16	28	24
B14	Kingston, ON	4W +B	14.3	80	18
B15	Kingston, ON	4W +B	14.3	57	18
B16	Kingston, ON	4W +B	14.3	18	45
B17	Kingston, ON	4W +B	14.3	50	34
B18	Kingston, ON	4W +B	14.3	52	24

B19	Kingston, ON	4W +B	16.79	52	32
B20	Kingston, ON	3W +B	15.8	51	30
B21	Kingston, ON	3W +B	15.8	26	15
B22	Ottawa, ON	3W +B	12	20	9
B23	Ottawa, ON	3W +B	11.83	18.7	9.4
B24	Ottawa, ON	3W +B	11.83	18.7	9.4
B25	Ottawa, ON	3W +B	10.11	15	12
B26	Orleans, ON	3W +B	12.3	36	18
B27	Orleans, ON	3W +B	12.3	18	15
B28	Ottawa, ON	3W +B	12.3	18	15
B29	Ottawa, ON	3W +B	12.3	18	15
B30	Ottawa, ON	2W +B	6.4	20	7.6
B31	Orleans, ON	4W	14.28	50	24
B32	Orleans, ON	4W	14.28	41	24
B33	Orleans, ON	3W +B	13.1	18	15
B34	Orleans, ON	3W +B	13.1	18	15

B35	Boucherville, QC	2W	6	9.7	9.4
B36	Longueuil, QC	2W	6	9.1	6.7
B37	Brossard, QC	2W	6	8.5	7.3
B38	Brossard, QC	2W	6	9.7	9.4
B39	Brossard, QC	2W	6	9.1	7.3
B40	Brossard, QC	1W	3	10.3	8.5
B41	Kamloops, BC	4W	3	70	50
B42	Okanagan, BC	5W	16	61	35
B43	Kingston, ON	4W +B	16.79	80	56
B44	Kingston, ON	3W +B	15.8	56	51
B45	Orleans, ON	3W +B	12.3	54	15
B46	Orleans, ON	4W	14.28	91	24
B47	Kingston, ON	4W +B	14.3	96	80

* W, C and B represent wood, reinforced concrete and basement, respectively.

AVT INSTRUMENTATION AND DATA COLLECTION

Six Tromino sensors were used to measure the ambient motion (acceleration and velocity) of the building floors in three orthogonal directions (see example in Figure 1). Each sensor is equipped with three high gain orthogonal electrodynamic velocimeters, three low gain orthogonal electrodynamic velocimeters, and three orthogonal digital accelerometers.

Figure 1 Tromino® (sensor) with radio antenna during data collection of building B18

As shown in Figure 1, the sensors have compact portable size and weight, and are kept in position without any anchorage to the floor. Their acquisition frequency range is 0.1-256 Hz and the signal is transmitted using radio waves. Multiple sensors were used for each test set-up, and several set-ups were used for each building. A master (reference) sensor was placed at a fixed location with good signal (that is with relatively large motion, typically on top floor or roof if accessible) and multiple roving sensors were moved to the pre-identified locations determined by studying building plans and access considerations. To obtain an accurate modal response, synchronization of the recorded signals is essential; the sensors form a wireless chain and their synchronization is achieved using radio antennas. Figure 2 shows a schematic radio network among the master sensor (S1) and the roving sensors (S2, S3, S4, S5 and S6) used on a typical floor of building B9. The building response was recorded for eight consecutive minutes for each

measurement setup at the sampling frequency of 128 Hz. The basic principle of AVT is that excitation forces are undefined and the recorded response is assumed to be a broadband, stationary Gaussian white noise, with approximately equal energy content throughout the frequency range of interest (Bendat and Piersol 2000).

Figure 2 Radio networking among sensors in Building B9

The recorded signals of each sensor are the floor velocity and acceleration components measured in two orthogonal horizontal (in the case of building B9, North-South and East-West) and Up-Down directions. Vertical acceleration and velocity records were removed from the raw measurements as the modal extraction makes use of only horizontal velocities in this project. Figure 3 shows an example of a recorded horizontal velocity component in Building B9.

Figure 3 Example of velocity record for building B9

EFDD ANALYSIS OF RECORDED MEASUREMENTS

ARTEMIS Extractor (Structural Vibration Solutions A/S, 2010) software was used in this research to process synchronized records and allow extraction of modal building parameters such as natural frequencies, mode shapes and approximate equivalent internal damping ratios using a technique called enhanced frequency domain decomposition (EFDD). This method proceeds with the estimation of the spectral density function (SDF) of each time record, which describes the energy distribution of the motion with respect to frequency. Singular value (eigenvalue) decomposition (SVD) of each PSD matrix is performed giving n sets of singular vectors (eigenvectors) corresponding to n measured configurations, which represent an approximation of the building natural frequencies and mode shapes. The singular values obtained from each of the n measured configurations are averaged by normalizing the area under the preceding singular value curve. These averaged singular values are plotted against frequency to offer potential peaks for identification of the building's natural frequencies excited by ambient sources. A single-degree-of-freedom (SDOF) bell-shaped function is produced for each measured configuration, by considering all frequencies in the vicinity of the resonant frequency (potential peak) and the modal parameters are identified in the neighborhood of the peak point of this SDOF spectral bell-shaped function, as illustrated in Figure 4. Typically, peaks defined by the bell curve are in

fact narrow frequency bands that may contain more than a single natural mode, as illustrated below for Building B9.

Figure 4 Singular value plot showing SDOF bell-shaped function, building B9

An estimate of the damping ratio for each identified natural sway frequency mode is obtained by converting the SDOF bell-shaped function to time domain using Inverse Fast Fourier Transform (IFFT) and calculating its logarithmic decrement. It is noteworthy to mention that the equivalent damping ratio is only a rough approximation, considering the many signal manipulations leading to its determination and the fact that structural damping is not typically viscous in nature (logarithmic decrement). An example of the two fundamental mode shapes for Building B9 extracted from the recorded measurements can be seen in Figure 5, as displayed by the ARTeMIS software. The AVT-extracted properties of the buildings are listed in Table 2. It should be noted that the dash in Table 2 indicates that no value could be extracted from the records due to high noise.

220 **Table 2 Measured natural periods and estimated modal damping ratios of wood-frame**
 221 **buildings**

Building ID	Natural Period (s)		Estimated Modal Damping Ratio (%)	
	Transverse	Longitudinal	Transverse	Longitudinal
B1	0.20	0.29	-	6.0
B2	0.29	0.31	2.3	2.9
B3	0.46	0.35	3.8	2.7
B4	0.37	0.29	2.5	1.8
B5	0.28	0.35	4.5	5.5
B6	0.20	0.19	4.3	2.4
B7	0.25	0.23	4.6	3.1
B8	0.40	0.25	2.3	2.2
B9	0.20	0.19	3.7	-
B10	0.26	0.23	-	1.5
B11	0.22	0.27	2.4	2.2
B12	0.29	0.32	2.1	2.3
B13	0.32	0.19	4.7	4.1
B14	0.33	0.23	1.4	3.2
B15	0.31	0.29	2.2	4.6
B16	0.29	0.27	2.3	3.3
B17	0.27	0.22	2.1	4.0
B18	0.24	0.19	3.4	-
B19	0.23	0.18	1.2	1.4
B20	0.23	0.26	1.0	-
B21	0.27	-	1.4	-
B22	0.18	0.14	3.1	4.6
B23	0.21	0.23	1.1	3.7
B24	0.35	0.29	1.9	1.1
B25	0.20	0.17	1.8	1.5
B26	-	0.24	-	2.6
B27	0.29	0.23	1.4	3.2
B28	0.26	0.29	-	1.1
B29	0.26	0.32	3.3	2.2
B30	0.20	0.12	1.9	1.3
B31	0.20	0.16	2.2	1
B32	0.21	0.16	1.7	3.8

B33	0.22	0.28	5.7	-
B34	0.24	0.28	3.6	-
B35	0.13	-	-	-
B36	0.15	-	7.3	-
B37	0.14	-	-	-
B38	0.12	-	4	-
B39	0.17	-	3.4	-
B40	0.11	-	-	-
B41	-	0.26	-	2.3
B42	0.26	0.21	1.4	-
B43	0.21	0.19	1.1	1.2
B44	0.26	0.24	1.9	2.8
B45	0.23	0.28	1.4	1.3
B46	0.19	0.16	-	1
B47	0.21	0.23	3.3	0.23

Figure 5 Test results of Building B9

NUMERICAL MODELLING

Several numerical modeling techniques have been used by various researchers to simulate the behaviour of light-frame wood structures with shear walls (e.g. Folz and Filiatrault 2004; Collins et al. 2005a; 2005b; Doudak et al. 2005; and Casagrande et al. 2015). In the present study,

selected buildings from the database were modeled using the commercial software SAP2000® (Computers and Structures 2016). The selection of the buildings was based on the availability of detailed building information obtained from the structural drawings including the building area, shearwall location and length, as well as details about the shearwall construction details such as nail size and spacing, sheathing thickness and hold-down type. The goal of this part of the study is to develop a simplified analytical model that would mimic the dynamic behavior of light-frame buildings at a level of loading consistent with that experienced by the structure when AVT is conducted.

Simplified 3D linear elastic models of the building structures were created in order to determine their fundamental sway period in each principal direction. The structural model was assumed to be pinned at its base and the floor diaphragms are assumed to be perfectly rigid. The building masses were lumped at the centroid of each floor and the rotational moment of inertia of the floor diaphragm was accounted for in the model. The in-plane lateral stiffness of the wood shear walls was estimated using the deflection equation provided in the Canadian timber design standard, CSA O86 *Engineering Design in Wood* (CSA 2014). The developed building models were validated using ambient vibration tests results of natural frequencies and mode shapes. For this purpose, the initial stiffness of shear walls, denoted as $K_{(0-10)\%}$, was determined and assumed to represent the level of loading at ambient conditions. A simplified 3D linear elastic model of Building B9 is shown in Figure 6.

Figure 6 3D model of B9 with shear wall stiffness $K_{(0-10)\%}$

Once the FE model results were validated with field results at stiffness $K_{(0-10)\%}$, the models were re-analysed with reduced stiffness (assumed to be $K_{(10-40)\%}$, in accordance with ASTM E2126), in order to predict the fundamental period of wood buildings exposed to seismic action. The FE models were also modified to capture the vertical effects of hold-down wall anchors whose motion is activated during an earthquake, by introducing a vertical spring representative of the hold-downs vertical stiffness.

RESULTS AND DISCUSSION

Comparison between Computed and Measured Building Periods

Table 3 compares the computed and measured periods at ambient level obtained for the buildings selected for analysis. Significant differences are observed in a few particular cases, such as the

longitudinal period of buildings B9, B43 and B45. These differences are mainly attributed to construction configuration or irregular plan shape. On average, the FE models overestimate the periods in the transverse and longitudinal directions by 13% and 23%, respectively. It is expected that the model would yield higher periods compared to the field measurements as the model is based on linear assumptions while the initial stiffness in real wood buildings is nonlinear and affected by connection behavior. Also, the model does not account for any contribution of windows, doors and other architectural finishes and components which may contribute to the lateral stiffness of the building, while their mass is taken into consideration.

Table 3 Natural periods of tested buildings vs. FE model results

Buildings	T (s)				Comparison	
	Field Results		Model Results		Model/test	Model/test
	Trans	Long	Trans	Long	Trans	Long
B5	0.28	0.35	0.30	0.40	1.07	1.14
B6	0.20	0.19	0.22	0.23	1.10	1.21
B9	0.20	0.19	0.22	0.28	1.10	1.47
B22	0.18	0.14	0.19	0.17	1.06	1.21
B25	0.20	0.17	0.22	0.20	1.10	1.18
B43	0.21	0.19	0.25	0.26	1.19	1.37
B44	0.26	0.24	0.27	0.28	1.04	1.17
B45	0.23	0.28	0.27	0.38	1.17	1.36
B46	0.19	0.16	0.21	0.19	1.11	1.19

Comparison with NBCC and literature

The fundamental period results from the current study were compared with the expressions developed based on data points obtained during low level earthquake shaking and forced vibration tests conducted by Camelo (2003) and the expression in the NBCC (2015) edition. In order to compare the AVT results with earthquake-level shaking data, a best fit line (with coefficient of correlation $R^2 = 0.58$) was drawn through the AVT data from the current study (see Figure 7 and Figure 8). The same best fit line is modified using Kharrazi's (2006) models to

predict periods at low and high level ground shaking (Equations (2) and (3)) are included in Figure 8.

Figure 7 Comparison of fundamental periods obtained from Current study (AVM test data) and Equation (4)

Figure 8 Comparison of fundamental periods obtained from NBCC equation, Equation (2), Equation (3), Equation (4), and current study AVM test data

Figure 7 shows that Equation (4) (Camelo, 2003) provides slightly higher period values than those extracted from the AVM tests. However, in Figure 8 (omitting the data points of Figure 7) Camelo's expression provides smaller periods than those obtained with Equations (2) and (3). As expected, the periods calculated using the NBCC formula for seismic analysis are larger than those extracted from AVM tests and calculated using Equation (4) and Equation (2) (for low ground shaking). It is also seen that the NBCC curve lies between the best fit lines of the two Kharrazi's models for predicting seismic periods representing low and strong ground shaking (Equation (2) and Equation (3)). This observation is consistent with the assumption that the NBCC formula is developed based on moderate-level ground shaking, but the main issue remains its large uncertainty since the only variable considered is the building height. The main

difficulty in comparing any level of vibration testing results with the NBCC expression stems from the fact that the NBCC equation is not defined for a specific level of seismic loading or vibration amplitude. From the comparison presented in Figure 8, Equation (2) appears reasonable and conservative for modifying the building period measured at ambient conditions.

Comparison with period equation for wind response

Whereas a modification of the measured periods at AVT level is needed for seismic design, AVT periods can be directly compared with the code expression for the wind response period. In Figure 9, the natural frequencies of several tested structures (from the current study and from literature (Feldmann 2015; Camelo 2003; Ellis and Bougard 2001; and Kharrazi 2001)) are compared with the curve in Eurocode EC 1 Part 1-4 (2010) (Equation (5) where h is in m).

$$n_1 = \frac{46}{h} \quad (4)$$

The plot in Figure 9 shows that Equation (5) provides a reasonable estimate of the fundamental frequencies when compared with the tested structures.

A least-square curve fit with coefficient of correlation $R^2 = 0.74$ was used to approximate the measured points from the current study, resulting in Equation (6), which provides estimates only slightly lower than the Eurocode formula.

$$n_1 = 44/h \quad (5)$$

Given the variability in field measurements and the uncertainty associated with such crude estimates of the fundamental building period, it can be concluded that the experimental results obtained in the current study provide further validation of the Eurocode equation for the purpose

313 of wind design.

314 **Figure 9 Frequency over height relationship**

315

316 **Deriving a Seismic Period Formula for Light-Frame Wood Buildings**

317 The current study shows that Equation (1) provides higher period estimates than the measured
 318 (AV) for wood shear wall buildings, and that considering the building height as the only variable
 319 in the period equation is not sufficient to describe the dynamic characteristics of light-frame
 320 wood structures. NBCC (2015) allows the use of so-called rational analysis methods other than
 321 the empirical equation (Equation (1)) with the limit that the calculated period shall not exceed
 322 twice the value obtained from Equation (1). In this part of the study, a simplified period
 323 estimation method is suggested for light-frame wood buildings, based on the Rayleigh
 324 approximation expressed in Equation (7).

$$T = 2\pi \sqrt{\frac{\sum_{x=1}^n (W_x \delta_x^2)}{g \sum_{x=1}^n (F_x \delta_x)}}$$

(6)

Where W_x , F_x are the seismic weight, and the force at level x , respectively, and δ_x is the elastic lateral displacement due to force F_x , n is total number of floors, and g is acceleration due to gravity. δ_x is taken as $1/k$, where k is the lateral stiffness provided by shear walls at level x .

Nine buildings with known geometric and structural information, were studied to assess whether the stiffness of shear walls can be expressed as a function of their length (see Figure 10). The selection was made based on the availability of construction details and structural drawings, including the dimensions of the building, the location and length of the shearwalls.

Figure 10 Lateral stiffness of the measured buildings vs. shear wall length

334 This simple method of estimating the period for wood shear wall buildings was validated by
 335 comparing the periods calculated by Rayleigh approximation (T_{Rayleigh}) with those obtained using
 336 the AVT measurements (T_{measured}). The correlation between T_{Rayleigh} and T_{measured} is displayed in
 337 Figure 11, where the dashed reference line represents the ideal fit.

338 **Figure 11 Correlation between AVT measured and Rayleigh periods**

339 T_{Rayleigh} is also compared with the fundamental periods obtained by finite element modeling
 340 (T_{model}) in Table 4.

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Table 4 Periods from AVT measurements, finite element modeling and Rayleigh approximation

Buildings ID	Periods (s)					
	Measured		Rayleigh		Model	
	Trans	Long	Trans	Long	Trans	Long
B5	0.28	0.35	0.28	0.42	0.30	0.40
B6	0.20	0.19	0.19	0.21	0.22	0.23
B9	0.20	0.19	0.20	0.26	0.22	0.28
B22	0.18	0.14	0.19	0.15	0.19	0.17
B25	0.20	0.17	0.20	0.19	0.22	0.2
B43	0.21	0.19	0.22	0.23	0.25	0.26
B44	0.26	0.24	0.25	0.25	0.27	0.28
B45	0.23	0.28	0.24	0.35	0.27	0.38
B46	0.19	0.16	0.16	0.22	0.21	0.19

Figure 12 shows a good correlation ($R^2 = 0.91$) between the periods obtained by FE modeling and by Rayleigh approximation using shear wall length to represent lateral stiffness.

Figure 12 Correlation between FE model and Rayleigh periods

Considering the above comparisons and the inherent variability of measurements and modeling

approximations, it can be concluded that the simple method of Rayleigh approximation considering lateral stiffness as a function of the length of wood shear walls, provides a reasonable fundamental period estimate for light-frame wood buildings. It does overestimate the measured results, but is nonetheless a good estimation when measurements are not available.

Proposed Period Formula for Light-Frame Wood Buildings

As discussed previously, a large spread was observed in the measured data when plotted against the building code period formula, which is solely a function of the building height. Figure 13 reiterates this observation, showing a poor coefficient of correlation of $R^2 = 0.27$. This has motivated the derivation of a better expression of the fundamental period for wood-frame buildings, as discussed next.

Figure 13 T_{measured} of selected buildings versus building height

Basic structural dynamics principles indicate that a building fundamental period is a function of the building mass and lateral stiffness. The building mass can be estimated with reasonable accuracy but since the majority of the mass is concentrated in the floors the floor area can be used to estimate the mass. Similarly, the shear wall stiffness can be approximated by the wall length aligned in the direction of interest. These practical simplifications allow the designer to evaluate a preliminary seismic design of a building before specific details are decided. To evaluate the fundamental building period dependency on these variables (height, stiffness and mass), the measured periods are plotted against $\left(\frac{h}{l} * A\right)$ for the two orthogonal directions in Figure 14, where h is the building height, l is the total shear wall length in a given main orientation, and A is the total floor area.

Figure 14 T_{measured} of selected buildings versus building geometric parameter $\left(\frac{h}{l} * A\right)$

It can clearly be seen from Figure 14 that the scatter in the measured data is less (R^2 value of 0.81) than that presented in Figure 13 where only the building height was considered (R^2 value of 0.27). For completeness, Table 5 gives the values of $\left(\frac{h}{l} * A\right)$ for each considered building.

Table 5 Natural periods of tested buildings vs. $\left(\frac{h}{l} * A\right)$ parameter

Building ID	$\frac{h}{l} * A \text{ (m}^2\text{)}$		Measured Periods (s)	
	Trans	Long	Trans	Long
B5	434	814	0.28	0.35
B6	173	197	0.20	0.19
B9	170	184	0.20	0.19
B22	127	61	0.18	0.14
B25	172	172	0.20	0.17
B43	296	237	0.21	0.19
B44	479	468	0.26	0.24
B45	203	398	0.23	0.28
B46	145	206	0.19	0.16

The regression in Figure 14 presents a significant improvement over that found in the building code for wood shear walls; however, since the best fit equation is based on measurements at ambient vibration level, it cannot be used directly for estimating building period for the purpose of calculating seismic base shear. In order to shift the proposed period expression to a level appropriate for seismic design, the expression developed by Kharrazi (Equation (2)) was used. The final form of the proposed period formula is presented in Equation (8), where values for $\alpha=0.045$ and $\beta=0.36$ are obtained by regression analysis that minimizes the error between the data points and the fit expression.

$$T = 0.045 \left(\frac{h}{l} * A \right)^{0.36} \quad (7)$$

A comparison of the building periods obtained by ambient vibration measurements, NBCC

formula (Equation (1)), Camelo’s period formula (Equation (4)) and that estimated using Equation (8) is presented in Figure 15 and Figure 16 for the transverse and longitudinal building directions, respectively.

Figure 15 Comparison of measured periods, code periods and periods by Equation (7)
(Transverse direction)

Figure 16 Comparison of measured periods, code periods and periods by Equation (7)
(Longitudinal direction)

With a few exceptions, it can be observed from these figures that the proposed expression (Equation (8)) yields period values that are lower than those obtained using the NBCC formula (Equation (1)) and higher than those using Camelo's period formula (Equation (4)). It should be noted that Camelo's formula depends only on building height and therefore is not capable of predicting different results in different principal directions. It should be emphasized that the above analysis and proposed new period formula are predicated on the fact that Kharrazi's equation (Equation (2)) is appropriate to modify AVT periods to earthquake level values. Furthermore, the methodology used here extrapolates Kharrazi's equation and assumes it is valid for taller wood buildings (up to six storey) than those used in its development (1-3 storeys).

The proposed expression is based on results from nine buildings only and it therefore should be considered as a rough, conservative estimate. When more buildings with known construction details are added to the database, the trends can be affirmed with less uncertainty and a more reliable expression can be developed.

CONCLUSION

The main objective of this study was to evaluate the adequacy of the NBCC formula for estimating the fundamental period of wood buildings, and develop a more reliable method of predicting the fundamental period of light-frame wood buildings. A comprehensive data base of measured dynamic properties of light-frame wood building has been compiled based on ambient vibration tests. Using a simplified method of period estimation based on the Rayleigh approximation while using the building mass and replacing the stiffness of shear walls by their length showed reasonable fit when compared with the FE model results and AVT measured periods.

Regression analysis on the measured data showed that a conservative better estimate of the building period (compared to the NBCC formula) can be obtained when the building height, floor area and shear wall length are included in the analysis. A formula was developed based on these geometric parameters and it was shown to provide a reasonably good fit with the measured results with a coefficient of correlation of R^2 in the order of 0.80. In general, the proposed formula yields period values that are lower than those obtained using the NBCC formula. AVT on more buildings with known construction details are needed to further refine the proposed expression.

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