

# EFFECT OF NON-STRUCTURAL COMPONENTS ON THE NATURAL PERIOD OF WOOD LIGHT-FRAME BUILDINGS

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**ABSTRACT:** This research deals with field testing of wood-framed and hybrid building structures with a view to establish a database of their essential dynamic characteristics, such as natural frequencies, mode shapes and structural damping. The main goal of this study is to investigate, through testing and modelling, the applicability of the simplified seismic demand calculation procedures of the National Building Code of Canada (NBCC) to wood frame structures, and suggest improvements where applicable. The research methodology initially involved a series of ambient vibration tests (AVT) conducted on wood-based buildings located in several regions of Canada. Modal properties were then extracted from the AVT records using frequency domain decomposition techniques, with the intent to improve the NBCC period formula for wood shear wall buildings. This paper presents ambient vibration field testing of a four-storey wood-framed building located in Ottawa, Canada; testing was done at two construction phases. The differences obtained between the measured natural periods at ambient conditions and the building code period are discussed.

**KEYWORDS:** Mid-rise wood buildings, Ambient vibration, Natural period, Lateral Stiffness

## 1 INTRODUCTION

Light frame wood buildings represent a typical construction type in North America for low-rise residential and commercial occupancies. The main types of structural components of wood-based buildings that resist lateral loads are horizontal diaphragms, vertical shear walls and roof (horizontal or inclined).

Historically light frame wood structures have shown good performance when considering life safety. However substantial structural and non-structural damages have been observed on wood buildings in strong earthquakes such as the 1994 Northridge with property loss of US \$20 billion [1] and the 1971 San Fernando earthquake, where single family home damages were reported to be \$58 to \$114 million [2]. Six of the seven collapsed buildings (four-storey) in San Francisco's Marina District were corner apartment wood buildings with first-story parking [3].

The maximum structural response values of velocity, displacement, and acceleration resulting from earthquake

motion can be determined from earthquake response spectra using the fundamental natural frequency and damping ratio as input values. Therefore, it is critical that the natural period and damping ratio of a building be estimated with reasonable accuracy, so that realistic values of base shear exerted on the building be used in design.

An inspection of the seismic behaviour of wood-frame constructions in a number of recent earthquakes has shown that the life-safety objective of building codes has largely been satisfied for peak ground accelerations (PGA) of 0.6 g (three- to four-storey buildings except for buildings where the soft-storey phenomenon was present) and sometimes higher at 0.8 g (for one- and two- storey buildings). Observations from earthquake damage surveys have shown that the satisfactory performance of multi-storey wood-frame buildings is achievable by enforcing proper design guidelines for engineered buildings in Codes and regulations [4].

In situ ambient vibration tests (AVT) are a valid alternative to computational methods to determine the dynamic characteristics of existing buildings. On one hand, sophisticated analytical techniques are less accurate than they appear even when detailed structural models are constructed. Simplified models also suffer from additional limitations: materially non-linear behaviour, complexity of structure, irregularity of building plan, unknown stiffness

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of diaphragms and significant contribution of non-structural elements that cannot be assessed precisely [5].

The lack of comprehensive information on the dynamic and seismic behaviour of wood buildings in Canada has motivated this investigation. The main goal of this research project is to create a data base for the fundamental sway frequencies of wood-framed and hybrid buildings in order to advance the understanding of their structural seismic response. For this purpose a non-destructive ambient vibration testing (AVT) method is used to collect in-situ data that is then processed; this experimental approach is known as Operational Modal Analysis (OMA). The dominant mode shapes, natural frequencies and modal damping ratios are identified from the measured response only (output only, input assumed random).

The dynamic investigation of wood buildings based on AVT requires sensitive instrumentation that can measure low amplitude micro vibrations caused by the surrounding environment on the structure (wind forces, base vibrations due to outside traffic, and various operational loads). By testing actual wood buildings in real conditions, the effects of non-structural components and foundations/soil type are taken into account.

## 2 FUNDAMENTAL PERIOD ( $T_a$ )

Recent research has shown that the current NBCC period formulae underestimate the building periods for concrete and moment resisting frame buildings [6] [7]. The same view point is under consideration by researchers for the case of shear walls in wood frame buildings.

The fundamental period of vibration  $T_a$  of a structure is an important parameter for calculating the design seismic base shear when using the equivalent static force procedure (ESFP), where horizontal internal forces  $V$  vary linearly along the building height if equal lumped masses are assumed at each floor. Note that ESFP is applicable only to buildings with no significant vertical or horizontal irregularity. Equation (1) shows the formula to calculate the equivalent lateral base shear force, where the period is required for determining the design spectral acceleration  $S(T_a)$  prescribed by the uniform hazard spectrum (UHS) provided by Code at the building site.

$$V = \frac{I_E W M_v S(T_a)}{R_d R_o} \quad (1)$$

where  $T_a$  is the fundamental period,  $I_E$  is the earthquake importance factor for the structure (set to 1 for normal importance residential buildings),  $W$  is the total building weight,  $M_v$  is the higher mode factor (set to 1 for low- and mid-rise wood buildings), and  $R_d$  and  $R_o$  are the ductility

and over-strength force modification factors, respectively. The National Building Code of Canada (NBCC) provides an empirical expression for computing the fundamental sway period of wood shear walls in seismic design using ESFP:

$$T_a = 0.05 * h_n^{3/4} \quad (2)$$

where  $h_n$  is the building height in m above the base. It is seen that this expression is entirely reliant on the building height and therefore does not account for parameters such as structural lateral stiffness and mass of structural and non-structural components, which in reality are the factors that determine the fundamental period of a building structure. This empirical formula is based on data from several measured buildings (mostly concrete) in California, subjected to ground motion during seismic events such as the San Fernando and Northridge earthquakes. The general accuracy of this formula is questionable considering that it has not been calibrated to wood-framed buildings. Since more and taller (mid-rise) wood-based and hybrid buildings are being constructed in Canada, there is a need to investigate the Code formula and the engineering seismic design.

Several field tests were conducted to obtain complementary data records of wood buildings. For some buildings, the ambient vibration measurements were recorded for different stages of the construction. Currently, more AVT measurements are planned to enrich the data base, anticipating that regression analysis will be performed on the extracted natural periods to obtain an improved formula for wood-frame shear walls. This paper discusses a case study of a four-storey residential condominium building where the fundamental frequencies, mode shapes and damping ratios were investigated.

## 3 FEATURES OF TEST STRUCTURE

The selected test structure is located in downtown Ottawa (Canada) with the building front facing an arterial road. Wind and noise from surroundings (mostly traffic) induce micro tremors in the building, which can be recorded by AVT instruments deployed on building floors/roof. The structure is resting on a concrete foundation having an irregular shape with plan symmetry along y-axis (see Figure 1a). The building has a footprint of 65.5 m X 33 m and total height of 12.7 m above ground level. Clear floor height is 2.77 m throughout. Each floor is composed of 19 residential units and a corridor, with two staircases (one at each end of the building) and two elevator shafts. The building plan and elevation are shown in Figure 1a and Figure 1b, respectively.

The floor system consists of 16" (406) mm deep pre-engineered wood I-joists spaced at 406 mm on centre and supported at exterior and intermediate load bearing walls. The floor corridors are made of 2" x 8" (38x184) mm joists. Sub-floor sheathing consists of oriented strand board

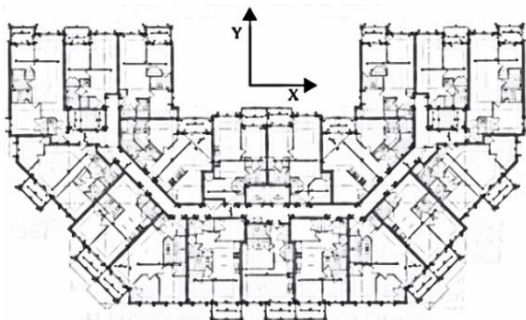
(OSB) panels connected with 15.5 mm 2½” Nails @ 100 mm spacing.

The wall system is made of Spruce-Pine-Fir (SPF) #1 grade. The wall construction details are provided in Table 1.

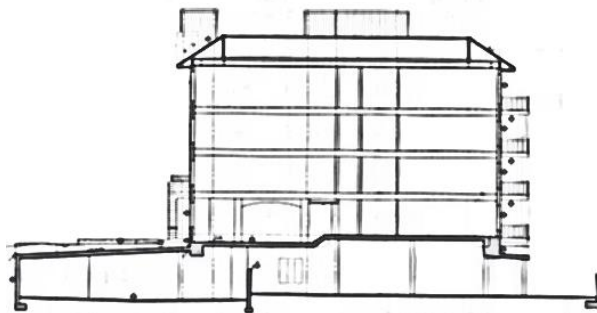
**Table 1: Wall schedule**

Vertical and Lateral Wall Schedule		
Wall Type	Wall Stud	Structural Sheathing
Party wall	2”x6”@8” (200) c/c staggered on 2”x8” Plate	1/2” OSB both sides 3” Nails @100 c/c along panel edges. (Block at joints) 3”Nails @12”c/c along intermediate studs
Corridors, stairs, elevator shaft and exterior walls	2”x6”@16”(400) c/c	¾” OSB one side 3” Nails @100 c/c along panel edges. (Block at joints) 3” Nails @12”c/c along intermediate studs

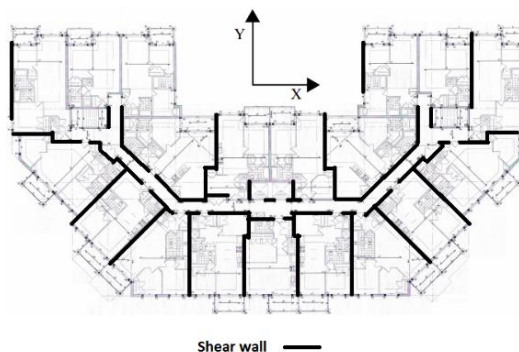
Most of the shear walls are oriented in the transverse short direction (y-axis) making the structure stiffer along this direction. Four shear walls are oriented diagonally that would also participate in the first mode response (see Figure 1c).



**a: Building plan**



**b: Building elevation**



**c: Distribution of shear walls**

**Figure 1: In plane configuration of the building**

## 4 BUILDING STIFFNESS

The strength and stiffness of the structure are mainly dependent on the characteristics of the structural components and their connections, such as shear walls nail-slip characteristics between the sheathing panels and framing elements, anchorage deformation, bending deformation of the studs and in-plane deformation of the sheathing panels. The stiffening effect of the “non-structural” components is significantly higher in light frame wood structures than in structures made of other materials such as masonry, concrete, and steel due to the relative low weight and stiffness of the wood shear walls compared to that of non-structural finish materials. Dynamic loading effects of wood framed shear walls has been investigated before and it was found that the effects of finish materials such as gypsum wall board and stucco on the cyclic response were much more significant than that of dynamic loading [8].

An experimental study performed on a full-scale two-story wood frame house to demonstrated the effect of non-structural wall finishes on the seismic performance of the test structure; the roof displacement decreased dramatically and no residual drift occurred after the wall finishes were incorporated in the test structure [9].

## 5 AMBIENT VIBRATION TESTING PROCEDURE

The experimental phase of the study relies on capturing low-amplitude ambient building vibrations and processing of the recorded measurements to extract the lower natural frequencies and mode shapes of the building under normal operating conditions. AVT has been widely used in the last 10-15 years in a variety of structural engineering applications and is now recognized as a practical and reliable experimental method to determine the modal properties of structures. One of the main advantages of AVT is that the preliminary results are available shortly after each test run and the testing can be performed on different sizes and types of structures.

### 5.1 AVT INSTRUMENTATION

The instrumentation used for AVT is comprised of acceleration/velocity sensors (shown in Figure 4). The sensors are wireless and their acquisition frequency range is (0.1 - 256 Hz) which clearly suffices to capture all significant natural frequencies of the tested building [10]. Each sensor is equipped with three orthogonal high resolution electrodynamic velocimeters and three orthogonal digital accelerometers. The sensors are synchronized using built-in radio communication, which is the most convenient way to achieve synchronization from short to intermediate distances. Radio synchronization of sensors with amplifiers depends on indoor and outdoor (open to air) locations. Generally, the synchronization among each couple of sensors can be possible at a distance not greater than 400 m [10].



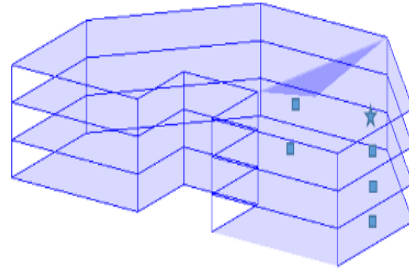
**Figure 4:** Tromino™ sensor with amplifier

### 5.2 TESTING PROCEDURE

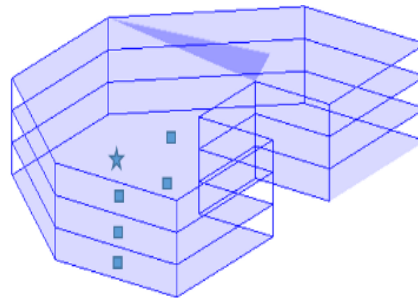
The four storey wood structure was first tested during construction, where all shear walls were constructed but no finishing materials were yet installed. The tests of the second phase were performed when the building was completely finished (all interior and exterior finishing materials in place).

The test setups were determined in advance by studying the building plans and available space; however, the final

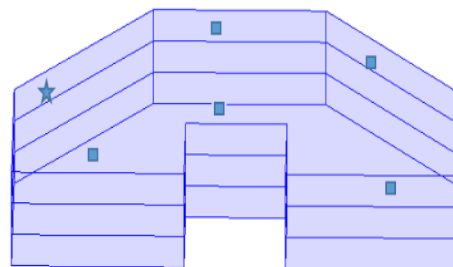
configurations of the sensors had to be adjusted on site based on reliable signal communication among the sensors. For instance, the roof was not accessible and therefore no sensors could be placed at that level. Multiple setup configurations were measured, with one reference sensor and five roving sensors. The instrumentation schematic layouts are shown in Figures 5a to 5c. In each setup configuration the reference sensor was positioned on the fourth floor to benefit from a stronger response signal, while the roving sensors were moved to the various pre-determined floor points of the structure.



**a: Setup 1**



**b: Setup 2**



**c: Setup 3**

**Figure 5:** Setup and sensor configurations

Legend:

- ★ Reference Sensor
- Roving Sensors

The velocities at each floor level were recorded in three orthogonal directions and each measurement record lasted

8 minutes at a sampling frequency of 128 Hz, which allows for sufficient down-scaling of the stored signals. Similar configurations and record duration were used for Phase 1 and Phase 2 testing. A sample of recorded signals of the master sensor in the horizontal and vertical directions are shown in Figure 6.

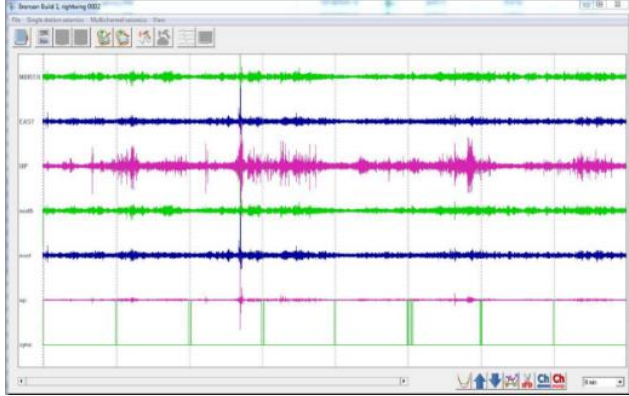


Figure 6: Raw signals data

## 6 EXTRACTION OF MODAL PROPERTIES

### 6.1 DATA PROCESSING

Ambient vibration signals are composed of a modal structural response and a random external excitation, often contaminated with noises. The pre-processing of the raw measurements was required to remove potential errors and to reduce the vertical measurements which are not required for current study.

The output of this pre-processing is a data file carrying the synchronized measurement setups to be used in modal analysis. Examples of pre-processed velocity data as time history and spectral density for configuration 5a are provided in Figure 7 and Figure 8, respectively.

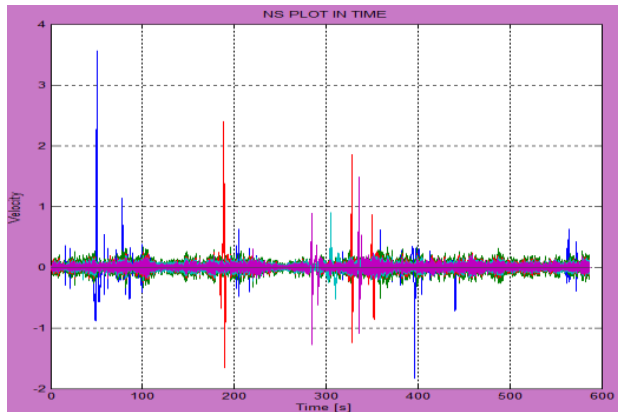


Figure 7: Velocity - time history

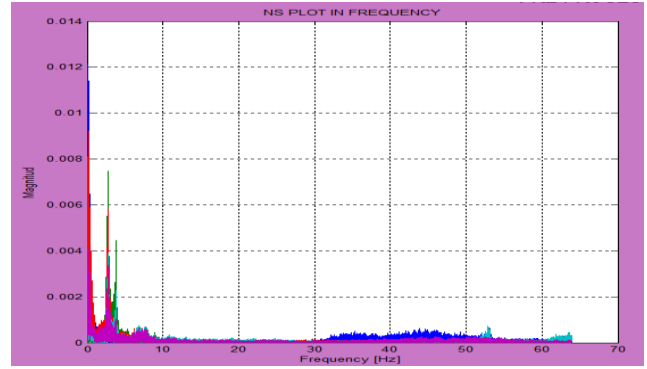


Figure 8: Velocity – spectral density

### 6.2 MODAL ANALYSIS

The pre-processed records were treated with autocorrelation functions, which were then transformed to frequency domain using FFT (Fast Fourier Transform) to generate power spectral density (PSD) matrices. The spectral density of a time signal describes the distribution of energy with frequency. Therefore, the decomposition of spectral density matrices into singular values function lines reflects a typical frequency spectra with the peaks carrying maximum energy contents of the signals. The spectral density function (SDF) between two time history records defined as  $x(t)$  and  $y(t)$  and having corresponding Fourier transform  $X(\omega)$  and  $Y(\omega)$  is given as:

$$G_{XY} = E[X(\omega)Y(\omega)^*] \quad (3)$$

Where,\* denotes the conjugate function.

### 7 MODAL IDENTIFICATION BY PEAK-PICKING

From the recorded measurements, the modal identification of the structure was performed using Peak Picking techniques following Enhanced Frequency Domain Decomposition (EFDD) and Frequency Domain Decomposition (FDD) methods implemented in commercial software (Artemis Extractor Software [11]). The FDD method is based on singular value decomposition (SVD) of each PSD matrix providing singular vectors and singular values of each discrete frequency. At a particular frequency the singular vector is representative of the structure's mode shape while the corresponding singular value indicates the contribution of each mode in the total energy carried out by the response signal. Figure 9 shows a plot of singular values. The EFDD technique is an improvement of FDD where, a single degree of freedom (SDOF) bell shape function is created and modal parameters are estimated using a range of frequencies in the neighbourhood of the picked point (SDOF spectral bell). The SDOF bell depends on modal assurance criteria (MAC)

that define the measure of consistency (correlation) between two vectors  $\{\phi_1\}$  and  $\{\phi_2\}$  as given by Equation (4):

$$MAC(\{\phi_1\}, \{\phi_2\}) = \frac{|\{\phi_1\}^H, \{\phi_2\}|^2}{|\{\phi_1\}^H, \{\phi_2\}| \cdot |\{\phi_1\}^H, \{\phi_1\}|} \quad (4)$$

MAC values can vary from 0 to 1, representing inconsistency and complete orthogonality between two mode shapes respectively.

In EFDD a different SDOF bell is identified for each mode shape of every measured configuration which is brought back to the time domain using inverse fast Fourier transformation (IFFT). This time function is an approximation of the SDOF autocorrelation function (see Figure 10a) of the corresponding mode [12]. A linear regression is performed on zero crossings plotted against time and the obtained slope represents the number of zero crossings per seconds or twice the number of cycles per second since the function crosses zero twice per cycle [13], so half of the regression slope of zero crossings provides frequency value (see Figure 10b). The SDOF autocorrelation function decays exponentially in a manner similar to the free response of a linear SDOF oscillator with viscous damping (see Figure 10c). For this reason, the logarithmic decrement technique, which is commonly applied to estimate the viscous damping ratio of an SDOF oscillator from its free vibration response, can be extended to the SDOF autocorrelation function [12].

The records measured at different sensor locations are analysed together to estimate the natural frequencies and damping ratios. The frequency range of the analysis was limited to the seismic response range of interest (0-20Hz) to reduce the noise effect and to accelerate data processing. By picking the highest and closely-spaced peaks of EFDD functions, the natural frequencies and corresponding mode shapes of the tested building were obtained. The set of mode shapes and frequencies most representative of the seismic response of the structure were then selected.

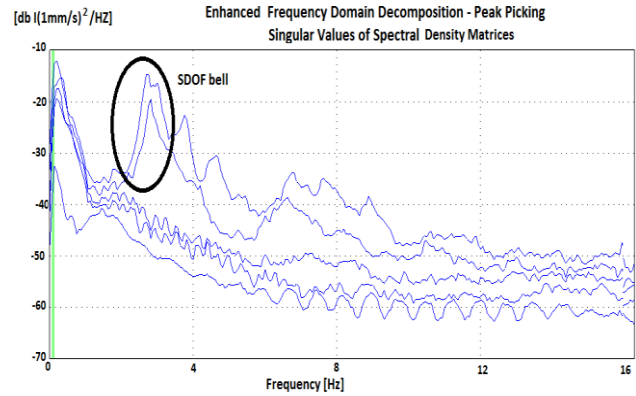


Figure 9: SDOF Bell in EFDD - Peak Picking

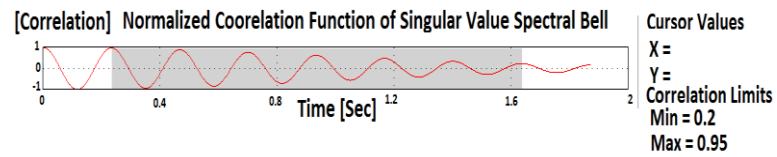


Figure10a: Example of SDOF autocorrelation function

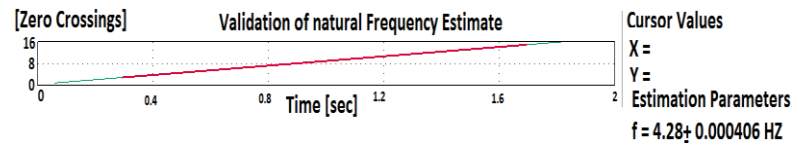


Figure10b: Improved estimate of frequency

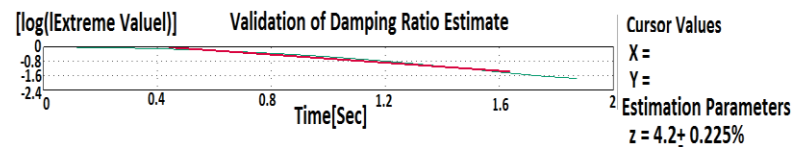
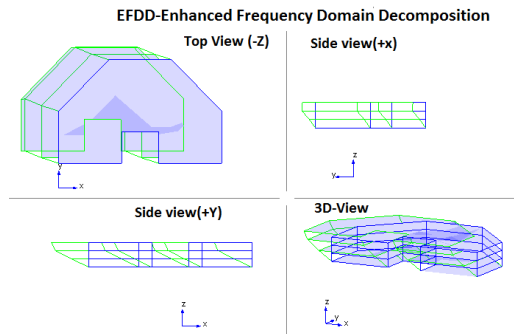


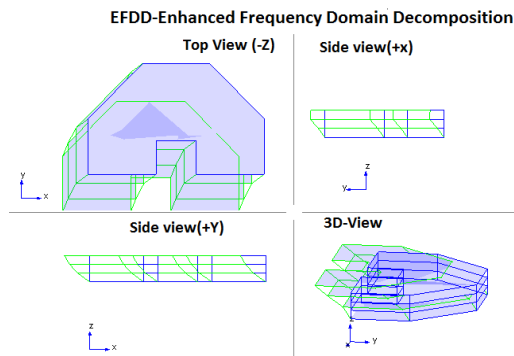
Figure10c: Estimate of damping ratio

## 8 RESULTS AND DISCUSSION

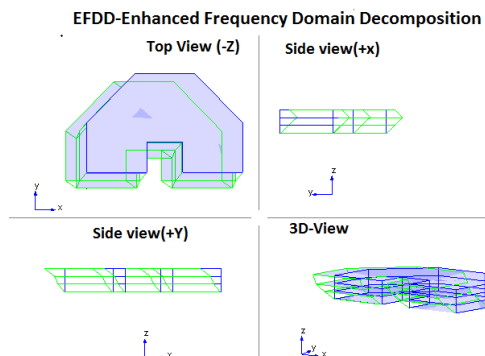
Results of fundamental sway modes in longitudinal (x-axis) and transverse (y-axis) directions are shown in Figures 11a, 11b, 11c and 11d for the two construction phases tested. Comparative results are also provided in Tables 2 and 3. The mode shapes were extracted at 80 % confidence level from setup 1 (Figure 5a), setup 2 (Figure 5b) and setup 3 (5c).



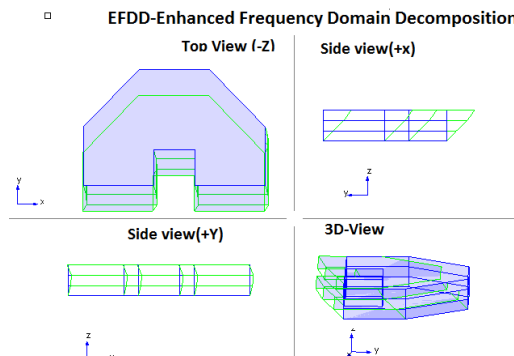
a: X-direction – Phase 1



b: Y-direction – Phase 1



c: X-direction – Phase 2



d: Y-direction – Phase 2

Figure 11: Sway response in Phase 1 and Phase 2

## 8.1 COMPARISON OF RESULTS FOR TWO CONSTRUCTION PHASES

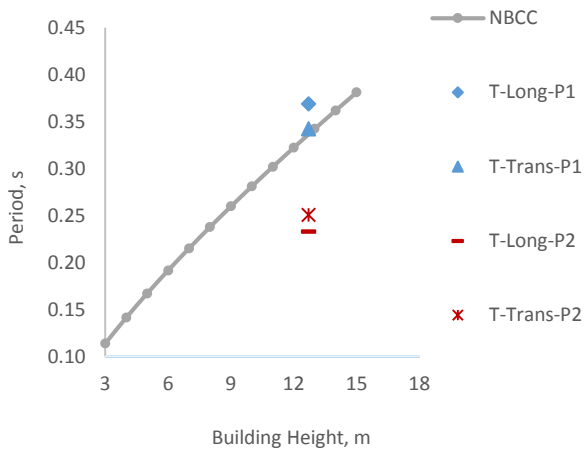
Table 2 shows the fundamental sway mode frequencies obtained in ambient condition for the test structure in the longitudinal (x-axis) and transverse (y-axis) directions. It can be seen that the frequencies in Phase 2 (complete building) are significantly higher in both directions than those obtained in Phase 1, indicating that the non-structural components have a significant effect in stiffening the structure. Generally it is a common practice for structural engineers to neglect the effect of non-structural components such as partition walls, external walls, parapet walls, stair walls, elevator shaft, etc. in mathematical modelling of a building for design purposes. This practice can have a significant implication on estimating the seismic base shear forces on the building, as longer periods will result in lower base shear values, as spectral accelerations of the UHS typically decrease for longer periods.

Table 2: Frequency estimation by EFDD

Construction Stage	Frequency (Hz)		Damping Ratio (%)	
	X-axis	Y-axis	X-axis	Y-axis
Phase 1	2.7	2.9	2.2	4.6
Phase 2	4.3	4.0	3.7	3.1

## 8.2 COMPARISON OF MEASURED PERIODS WITH CODE FORMULA

Figure 12 shows a comparison between the measured natural period of the building and those obtained from 2010 National Building Code of Canada (NBCC) period formula. Results in Figure 12 indicate that the code formula slightly underestimates the natural periods when non-structural elements are heavily damaged and no longer contribute to the stiffness and lateral load resistance of the building (represented in Phase 1). Furthermore, due to the highly nonlinear nature of wood shearwalls and characteristics of the loading, the natural period is expected to be significantly longer during a seismic event.



**Figure 12:** Comparison between measured periods and NBCC formula

Legend:

—●—	NBCC	National Building Code of Canada
◆	T-Long-P1	Period in longitudinal direction (Phase 1)
▲	T-Trans-P1	Period in transverse direction (Phase 1)
—	T-Long-P2	Period in longitudinal direction (Phase 2)
×	T-Trans-P2	Period in transverse direction (Phase 2)

## 9 CONCLUSIONS

The paper presents research related to the effect of non-structural components on the stiffness and thereby natural frequency of structures with light-frame wood shearwalls. The research methodology involves ambient vibration tests (AVT), where modal properties were extracted from using frequency domain decomposition techniques. This paper presents ambient vibration field testing of a four-storey wood-framed building, where testing was conducted at two construction phases.

The results showed that the frequencies in Phase 2 (complete building) were significantly higher in both directions than those obtained in Phase 1. This indicates that the non-structural components have a significant effect in stiffening the structure. A comparison between the measured natural period of the building and those obtained from the building code shows that the code formula underestimates the natural periods when non-structural elements are heavily damaged and no longer contribute to the stiffness and lateral load resistance of the building. This, together with the nonlinear behaviour of wood shearwalls, would lead to natural period being significantly longer during a seismic event, which in turn may have a

significant implication on estimating the seismic base shear forces.

## ACKNOWLEDGEMENTS

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