A novel three-dimensional seismic assessment method (3D-SAM) for buildings based on ambient vibration testing

by

Farshad Mirshafiei

August 2015

Department of Civil Engineering and Applied Mechanics
McGill University, Montreal

A thesis submitted to McGill University in partial fulfillment of the requirements of the degree of Doctor of Philosophy in Engineering

"The intuitive mind is a sacred gift and the rational mind is a faithful servant. We have created a society that honors the servant and has forgotten the gift."

Albert Einstein

Abstract

Most of current detailed seismic evaluation methods for buildings are based on numerical approaches. However, there is a need to use state-of-the-art interdisciplinary technologies and techniques to further facilitate such evaluations and improve their reliability, especially in many situations where detailed design documentation is not available. This study introduces a novel approach for seismic assessment of buildings, 3D-SAM, based on in-field ambient vibration measurements using acceleration/velocity sensors located on building platforms (floors and roofs).

In the experimental phase of this project, sixteen low and mid-rise irregular buildings designated as emergency shelters in Montreal, Canada were subjected to ambient vibration tests (AVT) and their lowest natural frequencies, corresponding mode shapes and estimates of modal damping ratios are reported. The rate of success of AVT in this study to capture at least the three lowest natural frequencies/modes is unlike previous studies where difficulty of performing AVT and modal extractions in low-rise buildings were reported. Furthermore, the measured natural periods of concrete structures and braced steel frames of the database are compared with those obtained from the Canadian building code period formulas and the results show agreement in the case of braced steel frames.

Due to the fact that in-situ experimental modal tests are low cost, and also owing to advances in sensing techniques and analysing procedures to derive the essential structural characteristic of buildings (operational modal analysis is well accepted in other engineering disciplines), the author developed a new three-dimensional seismic assessment method and software, called 3D-SAM in short form, that use this information to perform seismic assessment. The method incorporates torsional effects in predicting response, and therefore can deal with existing structural

irregularities, an important limitation of other existing simplified methods. It does not require the creation of any artificial numerical model and can easily be integrated into existing modal identification software. Applications of 3D-SAM to four buildings, low to high-rise, located in Montreal are presented in this study to illustrate and validate the proposed method; results are compared with those obtained using detailed and updated linear dynamic analysis of finite element models of the buildings. Next, a modified 3D-SAM is introduced that incorporates modification factors for the adjustment of modal properties to further extend the application of the method to stronger ground motions that may cause nonlinear response. Finally, the method is used for deriving the dynamic amplification portion of natural torsion on all floors of 16 low to mid-rise irregular buildings located in Montreal, Canada.

Sommaire

La plupart des méthodes actuelles utilisées pour l'évaluation sismique détaillée des bâtiments sont basées sur des approches numériques. Cependant, il est nécessaire d'utiliser des technologies et des techniques interdisciplinaires de fine pointe pour faciliter ces évaluations et améliorer leur fiabilité, en particulier dans de nombreuses situations où la documentation de conception détaillée n'est pas disponible. Cette étude présente une nouvelle approche pour l'évaluation sismique des bâtiments, 3D-SAM, basée sur des mesures de vibrations ambiantes à l'aide de capteurs d'accélération / vitesse situés sur les plates-formes de construction (planchers et toits) des bâtiments.

Dans la phase expérimentale de ce projet, seize bâtiments irréguliers de hauteur faible ou moyenne, désignés comme centre d'hébergement d'urgence par le Service de sécurité civile de Montréal (Canada), ont été soumis à des tests de mesures de vibrations ambiantes (AVT) dont on a pu extraire leurs fréquences naturelles en basse fréquence, les déformées modales et une approximation des taux d'amortissement modal. Le taux de succès des tests AVT dans cette étude pour capturer au moins les trois plus basses fréquences naturelles est à souligner compte tenu des études précédentes où la difficulté d'effectuer AVT pour les bâtiments de faible hauteur ont été signalée. En outre, les périodes naturelles de structures en béton armé et des cadres en acier contreventés contenus dans la base de données sont comparées avec celles obtenues des formules des codes du bâtiment et les résultats montrent un accord dans le cas des cadres en acier contreventés.

En raison du fait que l'analyse modale expérimentale peut se faire à faible coût, et aussi en raison de l'évolution des techniques de détection et des procédures d'analyse de signaux pour dériver les caractéristiques structurales essentielles des bâtiments, l'auteur a mis au point une nouvelle méthode d'analyse sismique tridimensionnelle et des logiciels, 3D-SAM, qui utilisent ces informations pour effectuer l'évaluation sismique. Cette méthode intègre les effets de torsion dans la prédiction de la réponse des bâtiments, donc, peut traiter les irrégularités structurelles existantes. La méthode proposée ne nécessite pas la création d'un modèle numérique d'analyse par éléments finis de la structure du bâtiment et peut être facilement intégrée dans le logiciel d'identification modale existant. La thèse présente l'application de la 3D-SAM à quatre bâtiments, de faible à grande hauteur, situés à Montréal à des fins d'illustration et de validation; les résultats sont comparés à ceux obtenus en utilisant une analyse dynamique linéaire détaillée et actualisée des modèles éléments finis des bâtiments. Ensuite, une modification est introduite à 3D-SAM qui consiste à ajuster les propriétés modales obtenues par AVT pour étendre encore l'application du procédé à des mouvements du sol plus forts. Et enfin, la méthode est utilisée pour dériver l'amplification dynamique de la torsion naturelle à tous les étages pour 16 immeubles de structures comportant des irrégularités et situés à Montréal (Canada).

Statement of original contributions

To the author's best knowledge, the original contributions of this research include:

- The first truly three-dimensional seismic assessment method (3D-SAM) directly based on ambient vibration tests (AVT);
- 3D-SAM routine written into the Matlab platform;
- The proposition of appropriate modification factors for derived AVT's modal properties based on previous earthquake experiences, to extend the application of 3D-SAM to the stronger based excitations;
- The database of AVT on 16 irregular low to mid-rise irregular buildings located in Montreal, Canada;
- The calculation of the dynamic amplification of natural torsion parameter for the whole data base by 3D-SAM to provide insight into the range of this parameter not researched to date.

Acknowledgments

This work was funded by the McGill Faculty of Engineering doctoral award program (MEDA) and the Natural Sciences and Engineering Research Council (NSERC) of Canada through the Strategic Research Grant Program (STPGP 396464) and the Strategic Networks Grant Program (NETGP 350698) through the Canadian Seismic Research Network.

There are many people who have helped me throughout the course of my graduate studies. I would like to thank my advisor Professor Ghyslaine McClure, for her guidance, encouragement and support throughout my graduate career. The opportunities for growth, presenting in many conferences and seminars, and the excitement of working in our group are deeply appreciated; I am honoured to have had the chance to be part of it.

Many thanks to my creative brother, Dr. Mehrdad Mirshafiei (Ph.D. in Electrical Eng.), and my friend, Dr. Iman Shamim (Ph.D. in Structural Eng.), who both helped me in development of the novel methodology, 3D-SAM.

My deepest appreciation goes to my family for their sustained encouragements and supports in all and every stage of my life; to my mother, Afsaneh, for her endless love; to my father, Aliakbar, one of the greatest Iranian architects who inspired me to pursue the structural engineering profession; and last but not least to my younger brother, Farzad, a true friend and a great architect.

Contribution of authors

Please note that this is a manuscript-based thesis consisting of two conference papers and three journal papers under review in peer-reviewed journals. These papers were written in collaboration with other authors. The title of the articles, name of the authors, and the related conference papers are listed below. It is worth mentioning that the author of this thesis is the sole student, among the co-authors, whom was responsible for conducting the research, in-situ experiments, analyzing the data, developing the methodology, preparing the manuscripts and presenting the research in the conferences. The author's supervisor, Prof. Ghyslaine McClure, provided invaluable guidance and editorial revisions throughout the entire process. Moreover, the author and Prof. McClure have filed the following patent:

Mirshafiei, F., McClure, G. (2015). A three-dimensional seismic assessment method (3D-SAM) and its applications. McGill University, US Provisional patent 62/136,875.

This thesis is consisted of the following publications:

Mirshafiei, F., McClure, G. (2012). Experimental modal analysis of emergency shelters in Montréal, Canada. Proc. 15th World Conference on Earthquake Engineering (15WCEE), Lisbon, Portugal, paper 4437, 10 p.

Mirshafiei, F., McClure, G. (2015). Ambient vibration tests on irregular low-rise buildings. Under review. A preliminary version of this paper was presented in: **Mirshafiei, F**., Asgarian, A., McClure, G. (2014). Operational Modal Analysis of Low Rise Buildings in Montréal, Canada. 4th International Structural Specialty Conference, CSCE, Halifax, Canada, paper CST-46, 9 p.

Mirshafiei, F., Mirshafiei. M., McClure. G. (2015). A new three-dimensional seismic assessment method (3D-SAM) for buildings based on experimental modal analysis. Under review. Postdoctoral Fellow Dr. Mehrdad Mirshafiei helped programming the 3D-SAM in Matlab. A preliminary version of this paper was presented in: **Mirshafiei, F.**, Asgarian, A., McClure, G. (2014). Seismic Vulnerability Assessment of Buildings using Ambient Vibration, Proceedings of the Twelfth International Conference on Computational Structures Technology, Civil-Comp Press, Stirlingshire, UK, Paper 226, doi:10.4203/ccp.106.226

Mirshafiei, F., McClure, G. (2015). Modified three-dimensional seismic assessment method for buildings (modified 3D-SAM) based on ambient vibration tests; extrapolation to higher shaking levels and measuring the dynamic amplification portion of natural torsion. Under review.

Mirshafiei, F., McClure, G. (2015). Application of a three-dimensional seismic assessment method (3D-SAM) based on ambient vibration tests to few buildings located in Montreal. Proc. 11^{th} Canadian Conference on Earthquake Engineering (11CCEE), Victoria, BC, Canada, 21-24 July, paper 95226, 10 p.

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1. Introduction

1.1 General

Economical loss from damages to structures because of earthquakes can lead to billions of dollars in populated urban areas. For example, total direct economic losses of $25.7 billion due to damaged buildings were paid by government and private insurance sources in the aftermath of the January 17, 1994 Northridge, California earthquake (moment magnitude of 6.7) as part of the recovery and reconstruction effort (Table 5-1 of Comerio et al. 1996). Moreover, the investment in non-structural components and building contents is far greater than the direct value of structural components and framing (Taghavi et al. 2003), and typically represents more than 80% of the total investment (Figure 1.1).

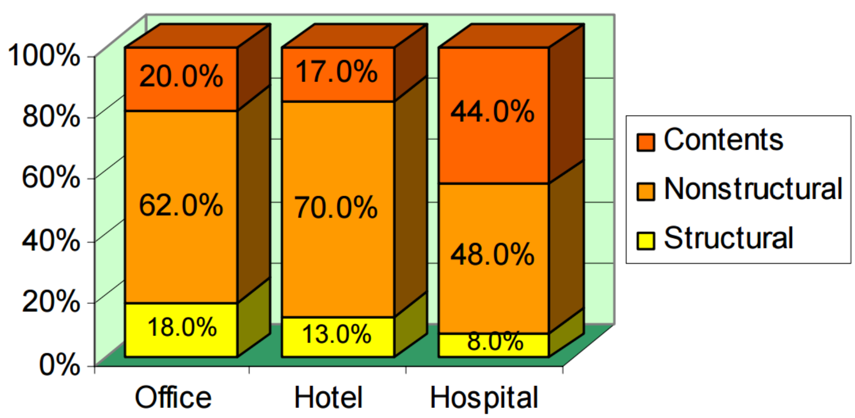

Figure 1.1 Typical investment in building construction (Taghavi and Miranda 2003)

Therefore, proper seismic assessment of structural and non-structural components of buildings is necessary for both weak and strong motions, in regions with moderate to high seismicity. Buildings designed before the inception of seismic provisions in codes of practice typically suffer damage after strong earthquakes. In order to check the safety margin of these potentially damaged buildings and evaluate performance of other existing structures, assessment of their seismic capacities need to be conducted. Accordingly, the subject of seismic evaluation of existing buildings has become more important in recent years and the awareness among governments and decision makers has increased. In fact, it is essential to have seismic evaluation portfolios for important post-disaster buildings such as schools and hospitals. This information will help to assess the building's behavior and performance for a possible future earthquake, identify whether the building is in need of retrofit, and provide a reference condition to recognize probable changes in the building's structural system after the occurrence of an earthquake. Current guidelines such as FEMA 154 and NRC 92 propose rapid visual screening methods for preliminary seismic assessment. Moreover, the existing linear and nonlinear static and/or dynamic analysis approaches based on modern standards and guidelines (ASCE 41, NIST 2010) enable the performance of detailed seismic assessment of buildings by engineers. To accurately assess a building with these numerical approaches requires data from detailed structural plans and some in-situ tests to identify material properties to construct the numerical models. However, there can be a significant variability in the predicted results obtained by various numerical models. According to a survey conducted in phase I of the ATC-55 project about the application of these assessment methods in structural engineering firms in the United States (FEMA 440), several respondents commented on issues about these analytical methods: their inaccuracy/variability, e.g. different analysis methods lead to significantly different results, the general complexity of these so-called simplified procedures, the sensitivity of the inelastic analysis approaches to assumptions regarding such parameters as initial stiffness, and the invariance of the loading patterns used in nonlinear static analysis procedures. There might also be interaction between non-structural components and structural elements that typically is not appropriately accounted for in numerical models. This interaction is especially important for weaker ground motions of moderate seismic regions and post-disaster buildings like hospitals and schools. Some researchers and specialized earthquake engineering firms use in-situ experimental modal tests to calibrate the numerical models and further improve the reliability of their seismic assessments. Currently, owing to advances in sensing techniques and analysing procedures, the most popular experimental modal test for large structures is ambient vibration testing (AVT). However, despite the unquestionable added value sensing techniques and operational modal analysis can provide, the calibrated numerical model approach is still not very popular among structural engineering firms. This can be partly related to the following facts: the model calibration process is somewhat complex; it still requires a very detailed finite element model (FE model) and at the end some discrepancies remain between the experimental and FE modal parameters; i.e. it is not feasible to calibrate a FE model to 100% of the test results.

To address the aforementioned problems, the author developed a simplified three-dimensional seismic assessment method and software (3D-SAM) based on modal characteristics obtained from AVT. To our best knowledge, 3D-SAM is the first three-dimensional seismic assessment methodology directly based on the observation of real modal properties of a structure obtained from ambient vibration tests. This approach can be used as a simplified alternative tool to the existing practice of linear calibrated numerical models based on in-situ derived modal properties. 3D-SAM is especially useful to assess buildings in moderate seismic regions due to the lack of data on recent earthquakes and the scarcity of damage information; the existing assessment methods are based on damage observations in high seismicity areas. This study is consisted of a presentation of a data base of AVT on 16 irregular low-rise buildings located in Montreal, Canada, the detailed presentation of 3D-SAM, its verification and finally its application to the seismic assessment of the buildings of the whole data base.

1.2 Objectives

The main goal of this research is to propose a new seismic assessment approach for existing buildings. This approach is based on experimental modal analysis and the resulted modal properties. To verify the new methodology, different response indicators calculated from both linear calibrated finite element models based on AVT and the new approach are compared together for four case studies. After proving the reliability of new methodology and its corresponding Matlab code, the tool is applied to a pool of 16 low and mid-rise irregular buildings designated as emergency shelters in Montreal, Canada that were tested by ambient vibration tests. Finally the dynamic amplification portion of natural torsion parameter is reported for all the irregular buildings in the data base.

The more specific research objectives are summarized as follow:

1) To carry out ambient vibration tests on 16 irregular low to mid-rise irregular buildings located in Montreal, Canada. These tests provide a data base of AVT on irregular concrete moment and braced steel frames, propose different layouts of sensor positioning for successful capturing of torsional and rigid/flexible roof mode shapes of irregular buildings, suitable sampling frequency and recording duration so that at least the three lowest natural frequencies, mode shapes and damping ratios can successfully be estimated by AVT.
2) To propose a novel three-dimensional seismic assessment methodology directly based on AVT in buildings.
3) To code the appropriate routine in Matlab to perform the proposed methodology.

4) To verify the methodology and the routine with four calibrated linear finite element models based on AVT.
5) To propose appropriate modification factors for the AVT's modal properties to further extend the application of the proposed method to stronger base excitations.
6) To apply the new methodology to all the buildings in the data base.
7) To provide insight into the dynamic amplification portion of natural torsion based on all of the assessed buildings.

1.3 3D-SAM methodology

3D-SAM is a direct top to bottom approach that makes use of in-situ derived modal properties and therefore bypasses the need for detailed engineering plans and FE analysis models. By extracting the dynamic properties of buildings from AVT, it is possible to calculate the building seismic response by convolution integral in the linear range according to classical structural dynamics theory. The 3D-SAM method predicts global seismic demands and response histories of buildings to a future earthquake. The whole process of the method, its inputs and outputs are illustrated in Figure 1.2. It should be mentioned that depending on the seismic demand parameter and intensity of the considered earthquakes, appropriate modification factors should be applied to the modal properties derived from AVT. Detailed explanations of the modification factors, application and limitations of 3D-SAM are discussed in chapter 6.

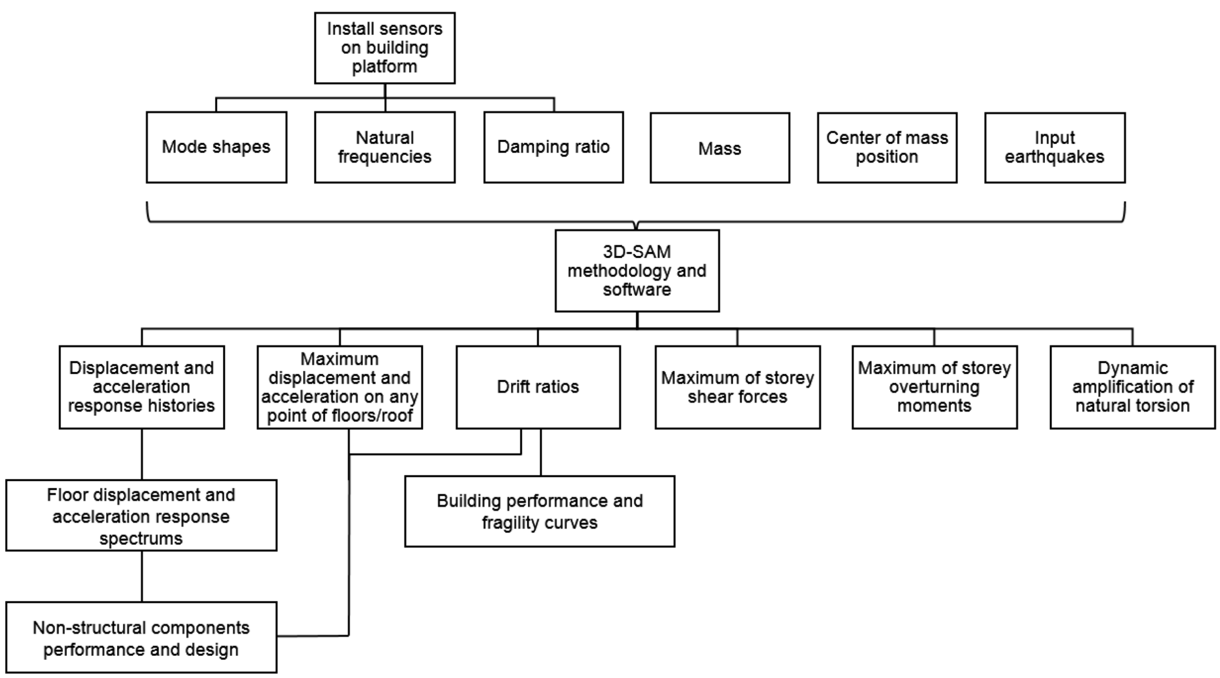

Figure 1.2 3D-SAM methodology, corresponding inputs and outputs

1.4 Thesis organization

This thesis is manuscript-based and divided into the following chapters:

Chapter 1 includes a general introduction to the research, its objectives, methodology and organization of the dissertation.

Chapter 2 presents a summary of the background and literature review. A literature review is presented on the following topics: ambient vibration tests and analysis of the recorded vibrations, rapid visual screening, detailed seismic assessment methodologies, seismic assessment based on AVT, change in modal properties from weak to strong base excitations and torsional effects in the National Building Code of Canada (NBCC).

Chapter 3 explains the AVT and the signal analysis procedure used in this study. It shows different sensor positioning and the resulted natural frequencies, damping ratios and mode shapes of eight low and mid-rise irregular buildings designated as emergency shelters in Montréal, Canada.

Chapter 4 presents the AVT results of the remaining low and mid-rise irregular buildings not covered in Chapter 3. Furthermore, based on the whole AVT experience developed in this research, different layouts of sensor positioning for successful capturing of torsional and rigid/flexible roof mode shapes of irregular buildings, suitable sampling frequency and recording duration are proposed as examples of good practice. Furthermore, natural periods of concrete structures and braced steel frames of the 16 tested buildings are compared with the values calculated using the NBCC period formulas.

Chapter 5 introduces the proposed three-dimensional seismic assessment method (3D-SAM) directly based on AVT-extracted data. This new methodology is applied to four buildings ranging from low to high-rise buildings. Then to assess reliability of the new methodology and its corresponding Matlab code developed in this research, different seismic demands calculated from 3D-SAM and calibrated linear finite element model of these buildings are compared together.

Chapter 6 introduces a modified 3D-SAM that provides modification factors for the modal properties to further extend the application of the method to stronger ground motions. Moreover, the method is used for deriving the dynamic amplification portion of natural torsion on all floors of the 16 low to mid-rise irregular buildings of the data base. Conclusions and recommendations about the range of the torsional effects are discussed.

Chapter 7 illustrates the application of 3D-SAM to four buildings to calculate global seismic demands such as: maximum relative floor displacements, story drift ratios, floor absolute accelerations, story shear forces, and overturning moments.

Although specific discussion and conclusions are presented at the end of each chapter, the general conclusions of this study and suggestions for future work are presented in **Chapter 8**.

1.5 References

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2. Literature review

2.1 Ambient vibration tests

There are two main forms of dynamic tests on civil structures, namely forced vibration tests (FVT) and ambient vibration tests (AVT). For FVT it should be possible to force or excite artificially a structure while at the same time measuring the applied load due to an electro-dynamic, hydraulic or mechanical shaker. Traditionally for buildings, rotating eccentric mass exciters (Figure 2.1) have been used (Littler 1988).

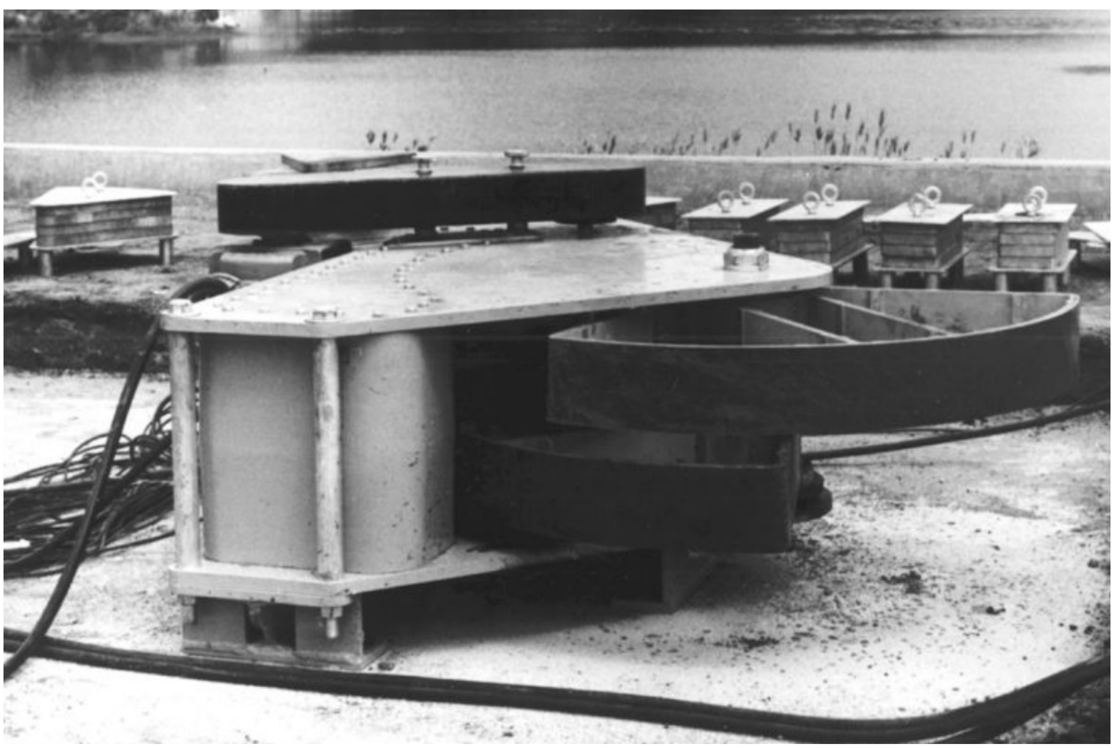


Figure 2.1 Counter-rotating eccentric weight vibration generator (Chopra 2007)

In FVT, when the system is excited by a known force and response is measured (output) then the output can be related to the input by the system properties. In fact, when transferring both input and output from time domain into the frequency domain by Fast Fourier Transform (FFT), then the division (quotient) of output over input results in the frequency response function (FRF, Figure 2.2). The FRF reveals the inherent dynamic properties of a linear system, independent of the excitation force and type.

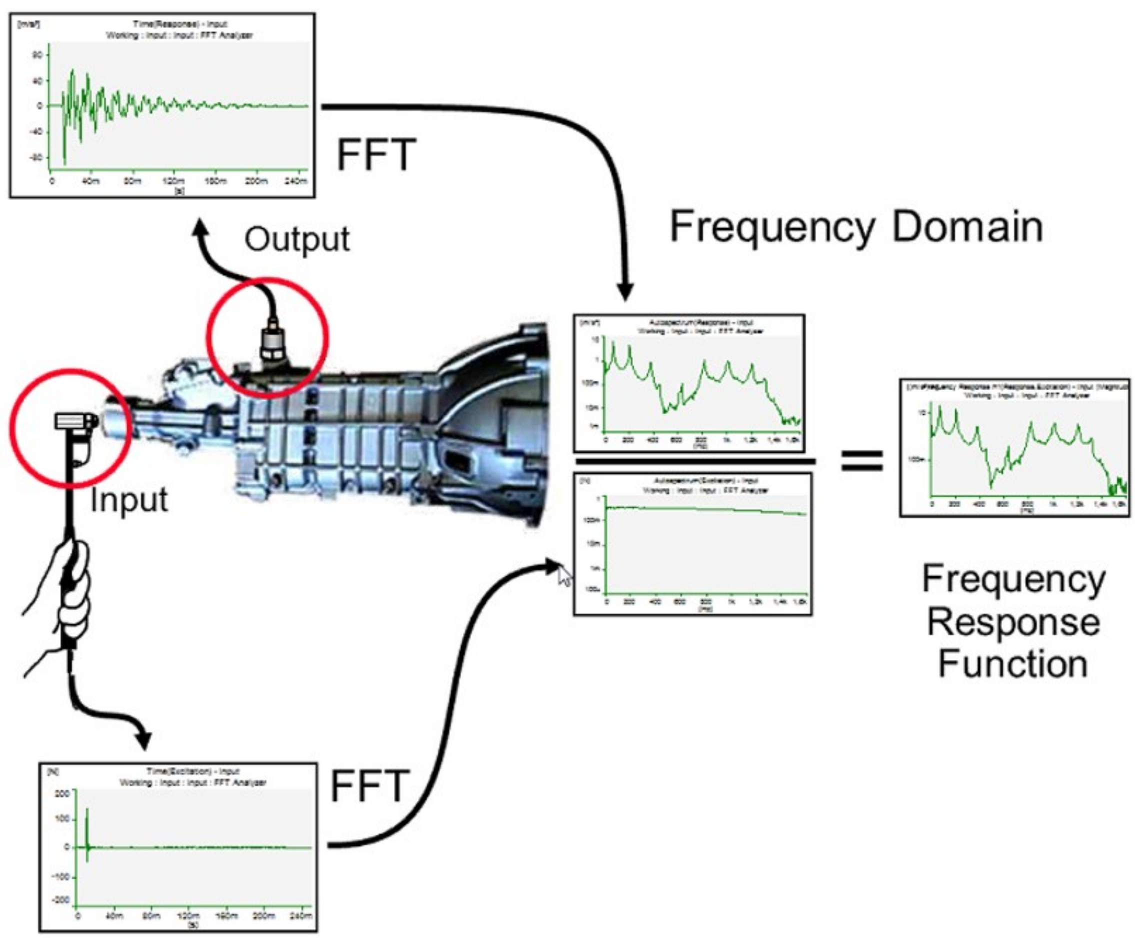


Figure 2.2 Forced vibration tests (From OMA short course 27 January 2012 McGill University)

Owing to technological advances in sensing techniques, ambient vibration testing has received more attention since the 1990s and has become the most popular method for testing real structures in recent years. AVT is of easy application in large structures, unlike large machinery associated with FVT, is low cost and its results are reliable. High resolution sensors are available at relatively low cost and can measure very small ambient accelerations/velocities in the buildings (Figure 2.3).

Figure 2.3 Wireless Tromino sensor and radio amplifier used for AVT

The purpose of AVT is to obtain the in-situ dynamic characteristics of a structure; its natural frequencies, mode shapes and modal damping estimates. Unlike forced vibration testing, the forces applied to the structure in ambient vibration testing are not controlled: The structure is assumed to be excited by wind, traffic, microtremors, and human activities. The measurements (velocities and/or accelerations) are taken for several minutes in the normal operational conditions of the structure, to ensure that all the modes of interest are sufficiently excited. In the modal identification of output-only systems the input loads are unknown and, thus the modal identification has to be carried out based on the responses only.

Although the displacements detected in both ambient and vibration generator tests are very small, the vibrator-induced motions may be several orders of magnitude greater than the ambient vibrations. However, it has been shown (Trifunac 1972; Lamarche et al. 2008) that forced and ambient tests will lead to consistent agreement of modal parameters of the building structure. Hans et al. (2005) also showed with series of tests that the FVT and AVT techniques yielded almost to the same results. Their study also confirmed that building natural frequencies tend to decrease while the vibration amplitude increases, but this reduction was observed to be very small, about only 2 to 5% while the excitation amplitude had been increased by 10^3. The same trend of consistency between results of AVT and FVT has also been observed by other researchers. In conclusion, the reliability of AVT in building structures has been proved. Although both methods of testing are based on relatively small levels of excitation compared to strong earthquake ground motions, the derived structural properties are invaluable since they offer results based on actual conditions of a structure.

In this research, TROMINO® sensors (portable ultra-light seismic noise acquisition system, 1.1 kg per unit) are used to measure ambient vibrations in buildings (http://www.tromino.eu/). Each sensor is equipped with three high gain orthogonal electrodynamic velocimeters (seismic microtremor acquisition), three low gain orthogonal electrodynamic velocimeters (strong vibration acquisition, e.g. traffic on bridges and similar), and three orthogonal digital accelerometers (scale ±5g). In AVT, normally results of the three high gain velocimeters are used, however, it is recommended to keep all the channels active during tests. The sensors are also equipped with internal/external GPS antennas to allow synchronization among different units outdoor and with a radio transmitter for indoor. The sensors can be connected to a personal computer using a USB cable and the recorded data can be downloaded using the Grilla software provided with the sensors.

The first step of a successful AVT campaign is to determine the layout of the sensors on buildings platforms and throughout the structure. The main principle in selecting the measuring points is to distribute the sensors in such a way that all the desired mode shapes be derived from the test. Therefore, sensors need to be located in positions that can capture the deformed shapes of a particular mode (not a modal node) with the needed resolution. Usually, the number of sensors are less than the number of required measurement nodes (e.g. 6 TROMINO® units were available for this study), therefore, several test setups are typically needed to perform AVT in a building. One or two sensors are required to be used as reference sensors; they remain in the same location and are active during all test setups. The other sensors are called roving sensors and are moved from one setup to the next to cover all the desired measurement nodes. The reference sensor needs to be placed in a point where all the desired modes have contribution to the response of that location; i.e. usually on top floors and at corner joints. In fact, the reference sensor acts as the connection point between the different test setups so that at the end of the test all the recorded data can be assembled and be representative of the appropriate mode shapes.

Duration of data records can have significant effect on the ability and quality of AVT to derive modal properties; longer acquisition time lead to better results (typically 10 minutes long data records were taken in this study). Moreover, the sampling frequency needs to be selected based on the Nyquist sampling theorem (Oppenheim 1989) which implies that aliasing (error) caused by discretization of a continuous signal can be avoided if the sampling frequency is greater than twice the maximum component frequency. Hence, in this case the sampling frequency should be at least twice the highest fundamental frequency of interest. For buildings we are typically interested in frequencies below 25 Hz (Gilles 2011).

One way of communication between different sensors is through radio communication (Used in TROMINO® units and this study as seen in Figure 2.3). It means that the sensors can form a wireless chain and communicate with each other using radio antennas. Therefore, all the recordings start at the same time and the recorded data will be synchronized. Between all devices on the chain, one sensor is set as the master sensor and the others are slave ones. The master can send commands to other slave sensors (Figure 2.4). Therefore, starting measurement on the master sensor automatically starts the recording on the other slave units concurrently.

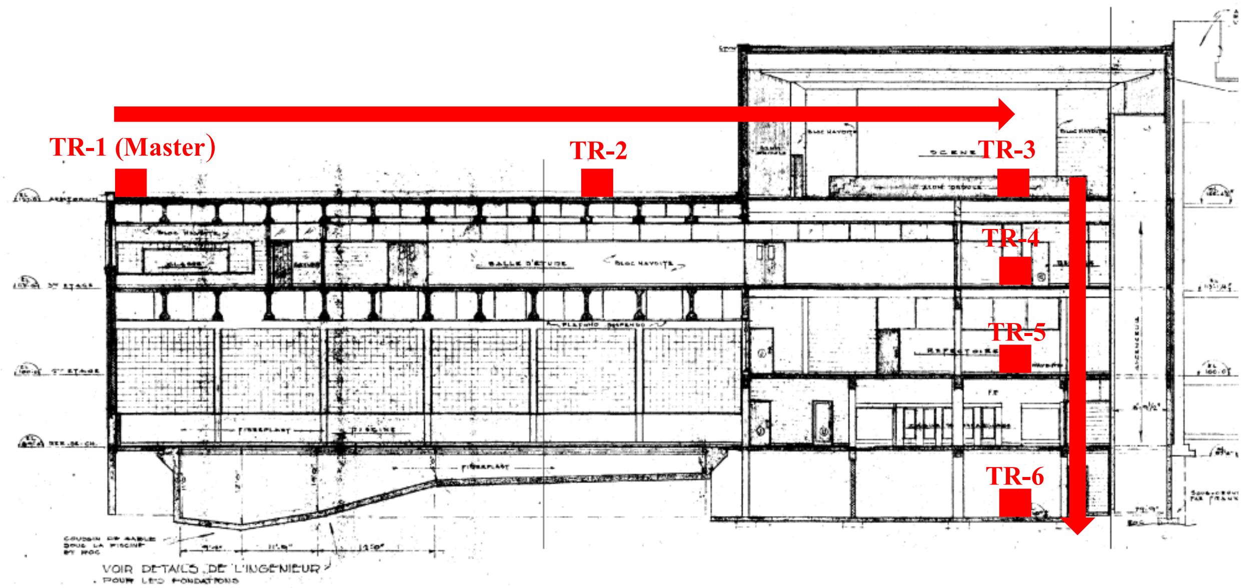


Figure 2.4 Order of the TROMINO® sensors to have a proper radio networking. In order for the radio synchronization to work properly, the master sensor (TR-1) should be close to the slave 2 (TR- 2). The slave 2 should be close to the slave 3; the slave 3 close to slave 4 etc.

Two AVT record analysis methods that are user friendly and widely used are Frequency Domain Decomposition (FDD) and Enhanced Frequency Domain Decomposition (EFDD); these methods extract the lower frequency modal parameters of the buildings (Brincker et al. 2001b, Jacobsen et al. 2006).

An essential step in any frequency domain system identification method is to calculate power spectral densities (PSD) of recorded data. Spectral density is a direct measure of a signal's energy content per unit frequency. The spectral density between two time history records x(t) and y(t), having corresponding Fourier transforms X(ω) and Y(ω), is defined as $E[X(\omega)Y(\omega)^*]$ (Norton 2003) where * denotes the complex conjugate and E[.] indicates the expected value operation. An initial estimate of the spectral density can be obtained by performing a Fast Fourier Transform

(FFT) for each raw time signal to obtain X(ω) and Y(ω) and simply omitting the expected value operation. According to the same equation the spectral density of one signal, x(t), is the square of the magnitude of the Fourier transform of the signal. Therefore, the unit of spectral density is the square of the unit of the original signal, x(t), per unit frequency. For instance, in this study signals are velocity response histories so that the spectral densities have units of [$(m/s)^2$/Hz]. However, it is common to quote spectral density in decibels (dB). The decibel is a logarithmic unit that indicates the ratio of a physical quantity (usually power or intensity) relative to a specified or implied reference level. For instance, taking the reference quantity equal to $(1m/s)^2$/Hz, the spectral density is calculated in dB unit as:

$$G\ [dB] = 10\ log_{10}[\frac{G[\left(\frac{m}{s}\right)^2/Hz]}{1\left(\frac{m}{s}\right)^2/Hz}] \tag{2.1}$$

It is useful to identify frequencies that contribute the most energy to a particular signal: if the input signal is a white noise, i.e. the input force is not a function of frequency but has a spectrum with constant (stationary) mean value at each frequency, then the output PSD matrix is directly related to the system properties (FRF matrix).

The PSD matrix is defined as the expected value of the product of Fourier transforms of all pairs of recorded data. However, as indicated in Equation (2.2), the PSD is estimated by dividing each signal into n sub-records of shorter duration and, omitting the expected value operation, averaging the multiplication of corresponding pairs of discrete Fourier transforms.

$$G_{xy}(\omega) \approx \frac{1}{n}\sum_{a=1}^{n} X^a(\omega) Y^a(\omega)^* \tag{2.2}$$

It is possible to estimate resonant natural frequencies by the classical frequency domain method called peak-picking, which involves plotting each spectral density function by considering one element of the PSD matrix over the frequency range of interest, and identifying the peaks as natural frequencies; the corresponding mode shapes are inferred by studying the relative magnitudes (and phases) of the spectral densities of the different signals, stored in the PSD matrix, at each identified natural frequency (Brownjohn 2003). Modal damping ratios can be roughly estimated by the half-power bandwidth method on any of the spectral density plots (Clough and Penzien 2003). However, this classical method has difficulties to identify closely-spaced modes. Therefore, two other more sophisticated methods (FDD and EFDD) have been used in the study to overcome this problem. The two methods are briefly presented next.

Frequency Domain Decomposition (FDD)

The first step of Frequency Domain Decomposition (FDD) is to estimate the spectral densities between all the recorded data channels to assemble the output PSD matrix, G (ω). The FDD proceeds to decompose the PSD matrix into a set of three matrices by singular value decomposition as follows:

$$G\ (\omega) = [U(\omega)][S(\omega)][U(\omega)]^H \tag{2.3}$$

Where H means Hermitian transformation (conjugate transpose of a matrix), [U] is unitary matrix (containing the singular vectors), [S] is the diagonal matrix of singular values and G is the power spectral density matrix. This decomposition is performed separately at each frequency. It should also be mentioned that singular values are listed in descending order along the main diagonal of [S] and are always real, non-negative quantities and on the other hand, the singular vectors are generally consisted of complex values. The singular vectors represent the system mode shapes and the corresponding singular values provide an estimate of the contribution of each mode to overall energy at each frequency. In fact, the singular value decomposition of the output PSD matrix is an approximation to its modal decomposition (Brincker et al. 2001b). Now plotting the singular values versus frequency, the natural frequencies of the structure are recognized as peaks. The first singular vector corresponding to each selected peak provides an estimate of the associated mode shape. Usually plotting only the first few singular values is sufficient. For well-separated modes, all mode shapes of interest can be picked on the first singular value alone (Figure 2.5).

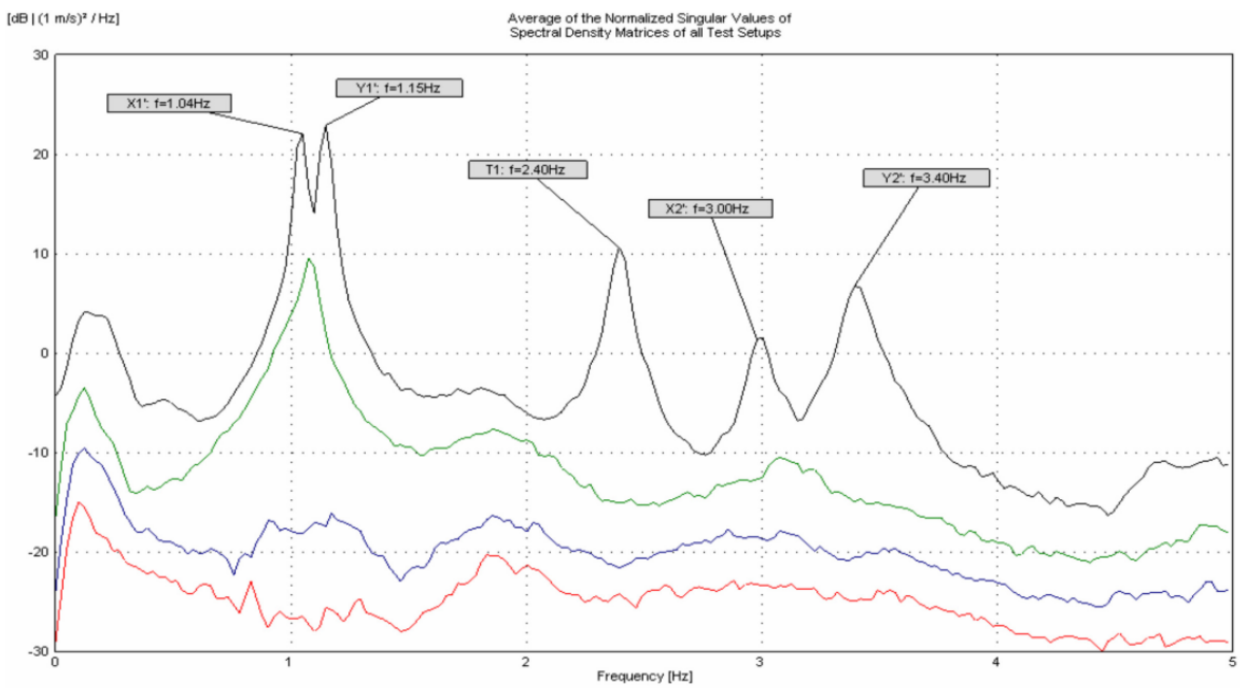


Figure 2.5 Singular value plot with well-separated modes (Gilles 2011)

However, in case of repeated modes, both modes are likely to have substantial energy at the same frequency. Therefore, both first and second singular values would be large at that particular frequency. Thus, the second singular value needs to be considered in addition to the first one (Figure 2.6).

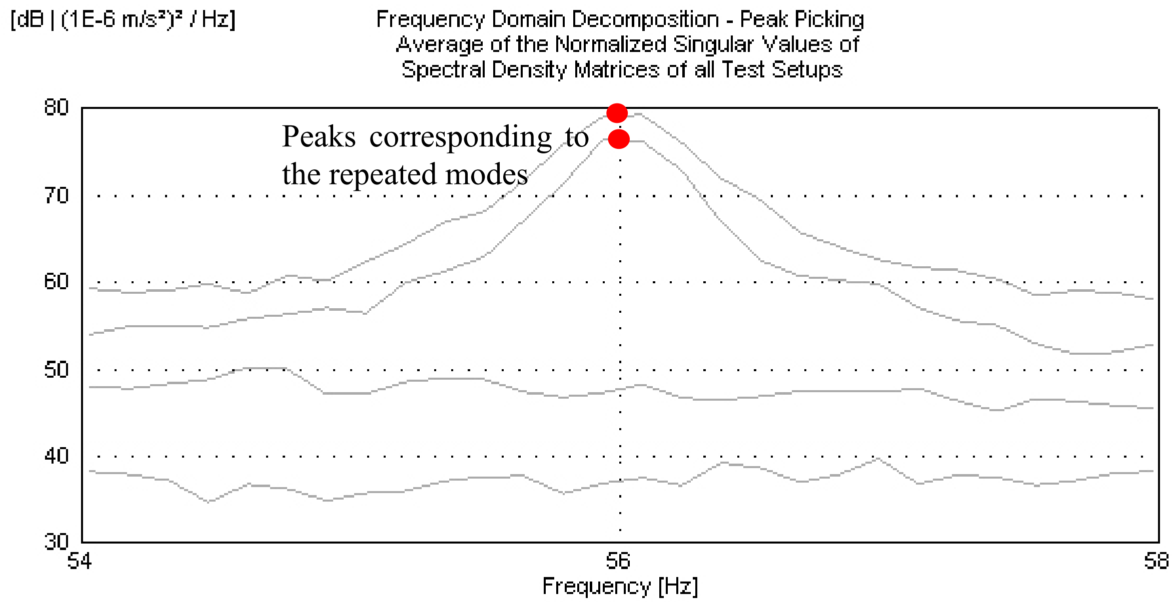


Figure 2.6 Singular value plot with repeated modes (adapted from Solution, S.V. 2010)

In most cases, few setups with one or two reference sensors need to be used to cover all the required measurement points. Therefore, at each frequency the singular values from all setups are averaged to result in the averaged singular curve. Then, the potential modal frequencies are estimated from the peaks of these averaged singular curves. Moreover, the components of the mode shapes at each frequency are obtained by considering the ratio of components of the singular vectors for the roving degrees of freedom to those for the corresponding reference degrees of freedom. This should be done for the singular vectors of each setup to obtain a global mode shape estimate (Brincker and Andersen 1999).

Enhanced Frequency Domain Decomposition (EFDD)

In FDD methodology the accuracy of modal estimation depends on how accurately the peaks are selected from singular value graphs. Poor peak-picking can lead to inaccurate estimates. To improve this prediction and also to get an estimate of the modal damping ratios, the Enhanced Frequency Domain Decomposition (EFDD) can be used.

A parameter that has an important role in EFDD is called modal assurance criteria (MAC), which is a tool to compare two mode shapes together and it provides a measure of consistency and correlation between two mode shapes. It is defined as follows:

$$MAC(\{\varphi_1\},\{\varphi_2\}) = \frac{\left|\{\varphi_1\}^H.\{\varphi_2\}\right|^2}{\left|\{\varphi_1\}^H.\{\varphi_1\}\right|.\left|\{\varphi_2\}^H.\{\varphi_2\}\right|} \tag{2.4}$$

The MAC value can vary between zero and one. A value near zero implies that mode shapes are not correlated and a value close to one indicates that the two mode shapes are almost similar.

In EFDD procedure, the first step is similar to FDD approach and the average normalized singular value plots for all test setups should be calculated and then the peaks are identified as potential modal frequencies. Next, in each setup, the singular vector (reference vector) corresponding to the chosen natural frequency is compared to other singular vectors at neighboring frequencies using the MAC (on each side of the FDD-identified frequency). A single degree of freedom (SDOF) bell function is constructed by considering all the frequencies around the peak for which the singular vectors correlate well with the reference vector at the peak. The range of the singular vectors to be included in the creation of the SDOF bell is based on the MAC criterion. If the MAC value of these vectors exceeds a user-specified MAC rejection level (set to 0.8 in the study) then the corresponding singular values are included in the description of the SDOF Spectral Bell. This SDOF bell is only defined at the frequencies within the user-specified MAC rejection level and is padded with zeros for the remaining frequencies (Brincker et al. 2001a). It should be mentioned that the identification of the SDOF bell has to be done for each mode and setup (Figure 2.7).

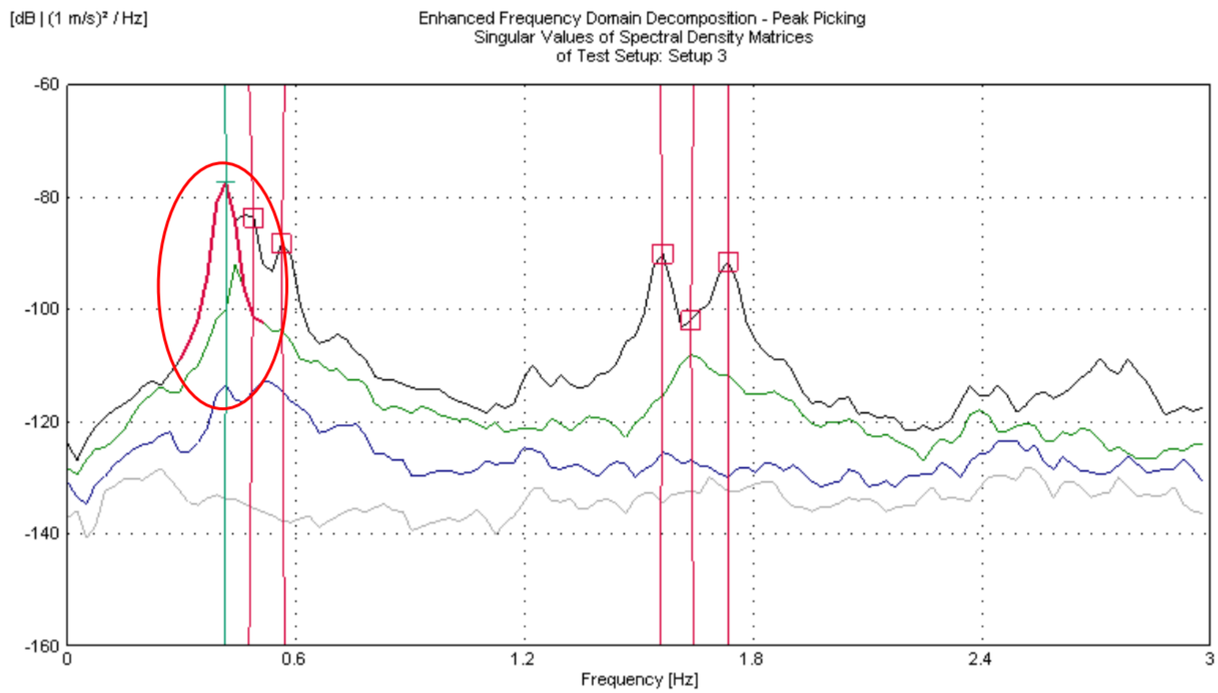


Figure 2.7 An example of a SDOF bell -in red color (Gilles 2011)

To calculate the natural frequency, the corresponding identified SDOF bell is brought back to the time domain using Inverse Fast Fourier Transform (IFFT). This conversion produces an approximation to the SDOF autocorrelation function (Figure 2.8), which is an exponentially decaying function that oscillates at the damped natural frequency of the corresponding mode shape (Bendat and Piersol 2000).

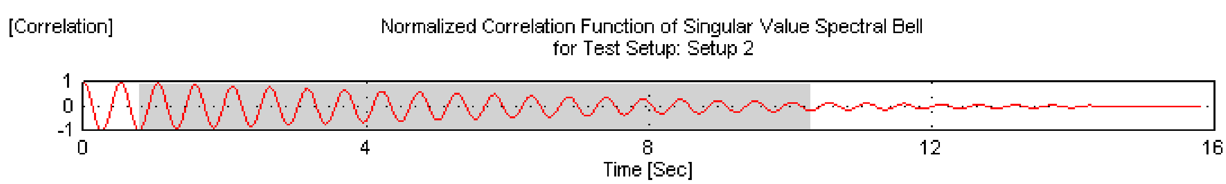


Figure 2.8 Example of SDOF autocorrelation function

An enhanced estimate of the natural frequency can then be found by counting the zero crossings of the SDOF autocorrelation function (Brincker et al. 2001a). Linear regression is performed on zero crossing plot and the slope resulting from the regression represents the number of zero crossings per second (twice the number of cycles per second). Therefore, the frequency is equal to half of this slop (Figure 2.9). It should be mentioned the identified SDOF bell doesn't always have a perfect bell shape, therefore, the user has to specify the correlation limits to do the calculations on the portion of the IFFT of the SDOF bell shape (autocorrelation function) which shows the exponential decay (shown by grey color in Figure 2.8).

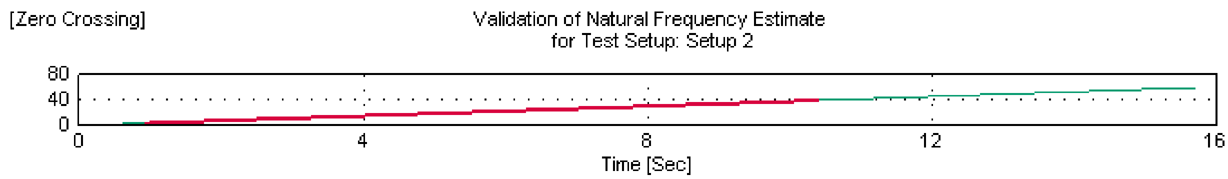


Figure 2.9 Improved estimate of natural frequency by using zero crossings

An improved estimate of the mode shape can also be obtained by weighted average of the singular vectors included in the SDOF bell by their corresponding singular values, therefore, giving more weight to the singular vectors near the peak. It should be mentioned that the SDOF bell process should be done for each setup and each mode. Hence, the improved mode shape from each setup only includes the measurement points in that particular setup. Therefore, to get the global mode shape all the mode shape components from all the setups should be assembled together.

The SDOF autocorrelation function (Figure 2.8) decays exponentially in a manner similar to the free response of a linear SDOF oscillator with viscous damping. Therefore, the logarithmic decrement technique that is used to estimate the viscous damping ratio of an SDOF oscillator for its free vibration response, can also be used in case of the SDOF autocorrelation function. Detailed explanation of this process can be found in (Clough and Penzien 2003 and Gilles 2011).

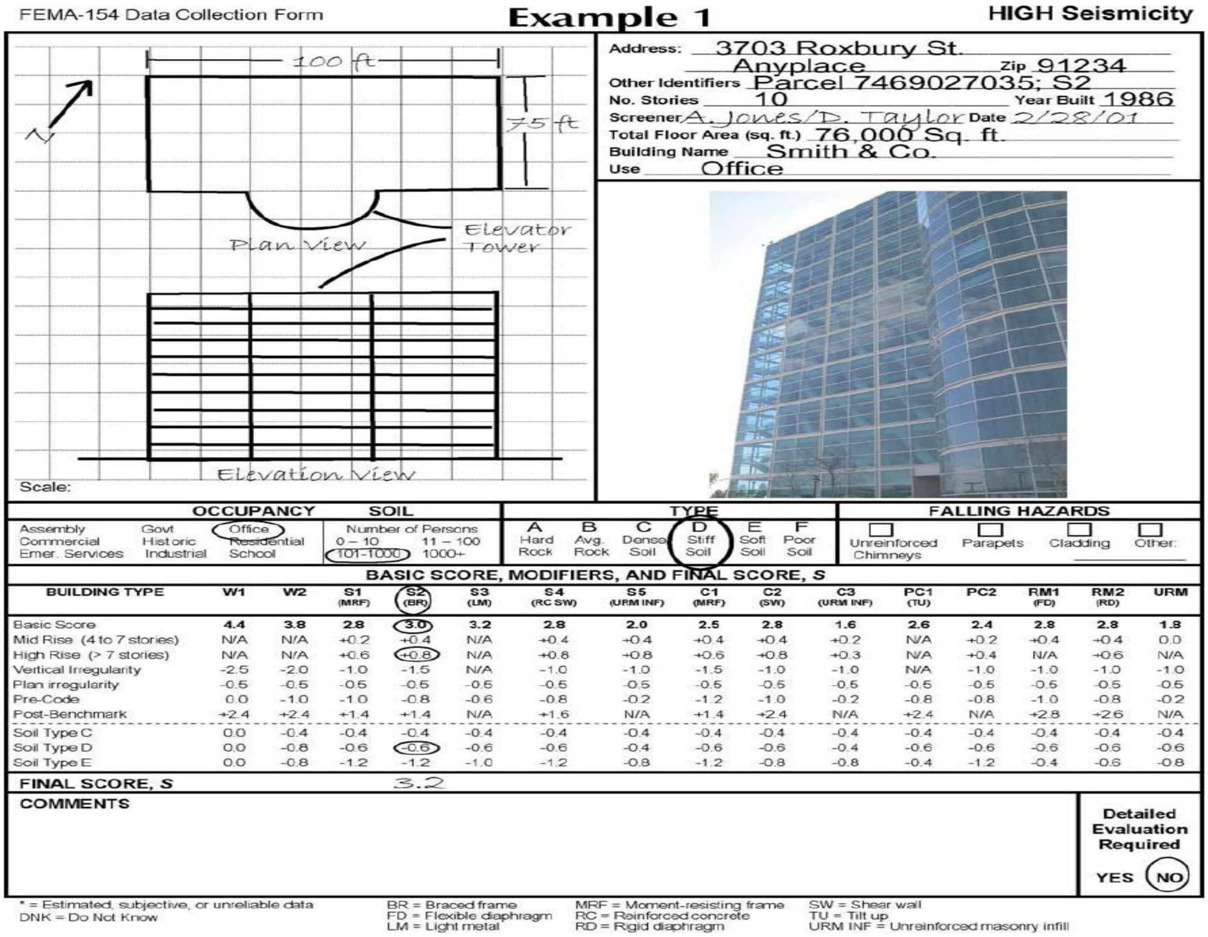
As mentioned before, in EFDD, for each mode in each setup a separate SDOF bell should be identified. Thus, estimates of natural frequency and damping ratio of a particular mode of vibration are found for each setup. Then, the results derived from all setups are compiled together to provide a data set and basic statistics (mean and standard deviation values) can be calculated.

2.2 Seismic vulnerability assessment of existing buildings

Two main procedures are typically employed for seismic vulnerability assessment of building structures; one is the vulnerability procedure based on field observations of building performance (and damage) during past earthquakes and the other one is the predicted vulnerability method (Karbassi 2010, Sandi 1982). The former is based on statistics on damages observed by structural engineers on similar building types during post-earthquake reconnaissance visits. It can be combined with the opinion of experts and used to derive damage probability matrices (DPM), which describe the probability that a building type is in a specific damage state for a given level of seismic hazard. This method can be quite reliable if the damage database is representative of the buildings to be assessed, typically in active seismic urban areas of the world (like the state of California, for example). However, in the absence of sufficient observed data, only the latter procedure that is based on calculations, expert opinions, design specification and detailed modelling can be employed. Subsequently, two popular methods are used and they may be seen as complementary: rapid visual screening (RVS) (FEMA 154, NRC 92) and subsequent linear or nonlinear detailed modelling when RVS indicates a significant seismic vulnerability (FEMA 356, FEMA 440, NIST 2010, ASCE 41).

Rapid visual screening (RVS)

Rapid visual screening of buildings for potential seismic hazards originated in 1988 with the publication of the FEMA 154 Report titled “Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook”. This document was written for a broad audience ranging from engineers and building officials to appropriately trained non-professionals; the Handbook provided a “sidewalk survey” method that allowed users to classify surveyed buildings into two groups: those acceptable with respect to life safety risk or those that may be seismically hazardous and must be evaluated in more detail by a design professional experienced in seismic design. The RVS method has been formulated to identify and rank building lateral force resisting systems (LFRS) that are potentially seismically vulnerable in an inventory of buildings. If a building receives a high score, it is considered to have adequate seismic resistance (and lower seismic vulnerability and risk); otherwise, it is flagged for a more detailed evaluation. A score of 2 is suggested in FEMA 154 as a “cut-off” based on current seismic design criteria. Using this cut-off level, buildings having an S score of 2 or less should be investigated by a design professional experienced in seismic design (FEMA 154, Figure 2.10).



FEMA-154 Data Collection Form **Example 1** **HIGH Seismicity**

Address: 3703 Roxbury St.
Anyplace Zip 91234
Other Identifiers Parcel 7469027035; S2
No. Stories 10 Year Built 1986
Screener A. Jones/D. Taylor Date 2/28/01
Total Floor Area (sq. ft.) 76,000 Sq. ft.
Building Name Smith & Co.
Use Office

Scale:

OCCUPANCY			SOIL		TYPE						FALLING HAZARDS			
Assembly	Govt	Office	Number of Persons		A Hard Rock	B Avg. Rock	C Dense Soil	D Stiff Soil	E Soft Soil	F Poor Soil	Unreinforced Chimneys	Parapets	Cladding	Other:
Commercial	Historic	Residential	0 – 10	11 – 100										
Emer. Services	Industrial	School	101-1000	1000+										

BASIC SCORE, MODIFIERS, AND FINAL SCORE, S

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	4.4	3.8	2.8	3.0	3.2	2.8	2.0	2.5	2.8	1.6	2.6	2.4	2.8	2.8	1.8
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8
FINAL SCORE, S				3.2											

COMMENTS

Detailed Evaluation Required
YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 2.10 An example of a data collection form for RVS (FEMA 154)

In recent years some researchers have tried to enhance the FEMA 154 rapid visual screening procedure or adapt it to other regions of the world; for instance, Tischer (2012) developed a rapid seismic screening method adapted to school buildings of the province of Québec from a database of sixteen schools (comprising 101 individual buildings) designated as post-critical shelters on the island of Montréal.

Linear or nonlinear detailed modelling

To select the appropriate detailed evaluation method, the first decision is whether to adopt the inelastic analysis procedure over the more conventional linear elastic analysis. In general, linear analysis is applicable when the structure is expected to remain nearly elastic for the level of ground motion of interest or when the design is such that the nonlinear response will be rather uniformly distributed throughout the building such that the mode shapes of the damaged building stay similar to those of the building in its usual operational conditions (FEMA 440). However, nonlinear analysis has the potential to provide a better understanding of the performance of buildings at moderate and severe damage levels if the simulation models are calibrated with actual building performance characteristics. There is also a wide selection of methods to conduct a nonlinear analysis such as: detailed time-accurate nonlinear dynamic analysis, simplified nonlinear dynamic analysis with either equivalent (condensed) multi-degree-of-freedom (MDOF) models or single-degree-of-freedom (SDOF) models and nonlinear static procedures (NSPs).

Nonlinear static procedures are popular in engineering practice and two variants are predominantly used. The first type is equivalent linearization techniques that are based on the assumption that the maximum total displacement (elastic plus inelastic) of a SDOF oscillator can be estimated by the elastic response of another SDOF oscillator with larger damping and natural period than the original. One of the most well-known and widely used forms of the equivalent linearization is the capacity spectrum method proposed by the Advanced Technology Council (ATC 40). The second type is the coefficient method; i.e. a displacement modification procedure (FEMA 356) that estimates the total maximum displacement of the oscillator by multiplying its elastic displacement response, assuming initial linear properties and damping, with one or more coefficients larger than unity. The coefficients are usually derived from series of nonlinear response history analysis of oscillators with varying natural periods and strengths. State-of-the-art nonlinear static procedures including the limitations of these methods for seismic evaluation of steel and reinforced concrete structures are summarized in (NIST 2010). The above procedures have recently been improved to take into account the higher mode effects and asymmetric-plan buildings: more advanced methods such as modal pushover analysis and the N2 method have also been developed (Fajfar 2000, Fajfar et al. 2002, Fajfar et al. 2005, Kreslin and Fajfar 2011, Chopra and Goel 2002 and 2004).

However, all the above methods have uncertainties in regard to numerical models and approaches; moreover, due to the lack of good quality drawings and unrecognized true behavior of connections and elements of the building, the creation of an accurate finite element model remains difficult for assessing existing buildings. Even though the effects of modeling uncertainties can be quantified with rigorous probabilistic analysis, the variability of their predicted results remains an issue in practical applications. The proposed methodology in this research can perform seismic assessment of buildings with poor quality drawings without making the complex finite element models and is based on ambient vibration test results.

2.3 Seismic vulnerability assessment using ambient vibration data

In the last decade few studies have been dedicated to find a link between the seismic performance assessment of a structure and ambient vibration test results. Boutin et al. and Hans et al. (2005, 2008), respectively, have shown that Timoshenko cantilever beam modelling was suited for describing the sway response of regular symmetric concrete moment frame and shear buildings and the results of this model were consistent with experimental modal characteristics obtained from AVT. Then, for a given LFRS based on this cantilever beam model, a so-called seismic integrity threshold was calculated which indicates the onset of structural damage: the building LFRS is predicted to remain elastic below this threshold, and by using linear dynamic analysis based on first-mode response, the story-drift ratios for elastic response were calculated. However, this simple model is applicable only to symmetric structures and shear-dominant LFRS buildings.

Michel et al. (2008) have discussed the evaluation of the lateral building stiffness from the AVT modal parameters and story-drift ratios. 2D lumped-mass shear beam models have been considered since they have low computational cost and apply to a large set of buildings, and elastic building motion under moderate earthquakes was computed by modal superposition analysis. However, constant mass was assumed for each floor, and the building torsional behaviour and any mode coupling effects were neglected. Moreover, building lateral motion was decomposed into the two main horizontal directions (longitudinal and transversal, assumed principal) and then the response was calculated. In a later study, Michel et al. (2009 and 2012) have used linear modal analysis to calculate fragility curves for the slight damage grade from modal parameters extracted from ambient vibration tests for 60 buildings in the city of Grenoble (France). Damage level was defined in terms of story-drift ratio (see HAZUS: NIBS 2003) corresponding to different grades of damage and for different LFRS. Fragility curves were developed, expressing the conditional probability $P[D=j \mid i]$ that a building will exceed a given damage state j for a prescribed level of ground shaking i. However, this method still has the same shortcomings as mentioned above; i.e. it ignores the possible variation of the mass at each floor, the coupling of modes in non-symmetric buildings and torsional effects. Also, the limits of applicability of the method are not clearly established. A recent study by Saeed (2013) based on AVT measurements of Montreal buildings (Gilles 2011, Gilles and McClure 2012) has attempted to propose a methodology that considers torsional effects to derive drift ratios and fragility curves for buildings subjected to ground motions. However, this method has the following shortcomings: (1) accurate estimation of mass at each floor is not truly considered; (2) significant importance of center of mass location in dynamic formulation consisting of torsional motion is ignored; (3) to consider three degrees of freedom (two translational and one rotational) per floor based on floor in-plane rigidity assumption, mode shape coordinates have to be calculated at center of mass on each floor. Therefore, proper optimization algorithm needs to be used to relate measured mode shape coordinates at sensor locations to center of mass ones which has been neglected in that study; (4) the method is not comprehensive, only calculates drift ratios, and doesn't provide other global seismic demands; (5) the method is limited to slight damage grade and performance; lacks extension of seismic analysis for strong earthquakes. To achieve this objective there is a need to apply appropriate modification factors to relate ambient modal properties to strong motion ones; and finally (6) the method has not been verified with other detailed seismic assessment approaches such as calibrated finite element models for real existing buildings.

Therefore, there is a need for introducing 3D-SAM as a more general three-dimensional method which will address the shortcomings of the current methods.

2.4 Modification of dynamic building properties obtained from weak-motion to strong-motion base excitations

The buildings' dynamic properties extracted from strong-motion records (with peak ground acceleration PGA > 0.1g) are expected to be different from those obtained using weak-motion such as low amplitude ambient vibration (PGA < 10^{-5}g). This difference is generally attributed to several factors that come into effect after the base motion exceeds the ambient levels: (1) the non-linear behaviour of the structural material (such as micro-cracking of the concrete at the foundation or superstructure); (2) connection slippage (in bolted steel structures and timber structures); (3) interaction between non-structural and structural elements; and (4) soil-structure interaction effects (Dunand et al. 2006).

Changes in modal characteristics and wandering of natural frequencies were also observed in undamaged structures (with slight or not visible damage) subjected to strong motion (Celebi 2007).

The normal tendency is for natural frequencies to decrease and damping ratios to increase with seismic intensity, while mode shapes are not altered much as long as no localized damage occurs. Such information can be found from data collected in buildings equipped with permanent strong-motion instrumentation where the building has not suffered visible structural damage during the strong base motion.

In this study a careful review of such buildings in the literature has been done; consisting of 18 buildings listed in (Dunand et al. 2004 and 2006, Celebi 1993, 2007 and 2009, Carreno et al. 2011, Soyoz et al. 2013, Singh et al. 2001) and another 21 buildings subjected to 1994 Northridge earthquake and its aftershocks (Todorovska et al. 2006 and 2007). And the following observations are made: (1) the strong-motion modal frequencies are decreased by a maximum of 30% and 40% of the corresponding values extracted from ambient vibration records for steel and concrete buildings, respectively; (2) the mode shapes are not changed from ambient to strong vibration levels (before the occurrence of damage); (3) the internal damping ratio for strong-motion response can be as much as 2 to 4 times larger than found using ambient measurements. Chapter 6 explains how application of the proposed method is extended to stronger ground motions based on these observations.

2.5 Torsional effects in National Building Code of Canada

Eccentricities between the centres of mass and rigidity at various floor levels in a building cause torsional motion during an earthquake. Seismic torsion leads to increased displacements at the extremities of the buildings. Structures with non-coincident centres of mass and rigidity are referred to as asymmetric structures and the torsional motion induced by asymmetry is referred to as natural torsion. Asymmetry may in fact exist even in a nominally symmetric structure because of uncertainty in evaluation of the centres of mass and rigidity, inaccuracy in the dimensions of structural elements, or lack of precise data on material properties. Moreover, torsional vibration may even result from rotational motion of the ground about the vertical axis. Torsions coming up from undetermined asymmetry and ground rotational motion are together referred to as accidental torsion (Humar et al. 2003).

In the simplified quasi-static procedure of the National Building Code of Canada (NBC 2010 section 4.1.8.11) torsional seismic effects are considered by applying torsional moments about a vertical axis at each floor level, derived separately for each of the following load cases considered:

$$T_x = F_x(e_x \pm 0.1D_{nx}) \tag{2.5}$$

Where F_x is the lateral force at each level and D_{nx} is the plan dimension of the building at each level x perpendicular to the direction of seismic loading being considered. Also, e_x is the natural eccentricity, i.e. that due to the centres of rigidity and mass being at different positions. De La Llera and Chopra (1994) show that the portion $0.05D_{nx}$ represents accidental torsion; the remainder takes into account natural torsion, including its dynamic amplification.

Moreover, NBC requires 3D dynamic analysis for torsionally sensitive structures, i.e. for which the sensitivity parameter B>1.7 (B is determined as the maximum value of B_x for each level; where $B_x = \frac{\delta_{max}}{\delta_{ave}}$, δ_{max} and δ_{ave} are maximum and average displacements of the building at extreme points of level x, respectively). However, even in the case of dynamic analysis, the effects of $0.1D_{nx}F_x$, i.e. accidental torsion effects that include dynamic amplification of the static effect of accidental eccentricities, should be calculated and then combined with the effects determined from a dynamic analysis that includes the actual eccentricities.

The code provisions for design against torsion are based on studies of elastic response of torsionally unbalanced buildings to earthquake motion, to a large extent, based on elastic response of a simple idealized asymmetric single-story building (Humar et al. 2003, De Stefano and Pintucchi 2007). Therefore, this topic still needs further investigations to consider other effects on torsion such as those due to vertical irregularities and eccentricities in multi-story buildings.

In chapter 6, the proposed method is applied to a pool of irregular buildings and the dynamic amplification portion of natural torsion on all floors in the buildings are calculated. Therefore, reported results are based on real building characteristics having different types of irregularities and number of stories.

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3. Experimental modal analysis of emergency shelters in Montreal, Canada

3.1 Summary

Two popular non-destructive methods to assess the dynamic properties of building structures are ambient vibration and forced vibration experiments. Ambient vibration testing offers an important advantage over forced vibration techniques as it does not require any special excitation of the structure. Massive structures may indeed require strong forced excitation levels that are not always possible in operational buildings, and floor system alterations are typically required to restrain the shaker, which is also constraining.

This paper presents the operational modal analysis results of a series of ambient vibration tests performed on low and mid-rise buildings designated as emergency shelters in Montréal, Canada. Fundamental mode shapes, modal frequencies and corresponding modal damping ratios were determined from the records, based on advanced frequency domain decomposition techniques available in commercial software. The modal identification is an important step in the validation of finite element analysis models, assessment of current structures and health monitoring purposes.

3.2 Introduction

When a building is subjected to dynamic loads, its structural response depends on the frequency content and magnitude of the forces exciting the structural system, the dynamic properties of the building (natural frequencies, mode shapes and damping ratios), the variation of these parameters in time if the building behaves nonlinearly during strong motions, as well as the soil type and foundation underneath of the superstructure. Therefore, the first step to predict the dynamic response of a structure is to estimate its natural or operational dynamic characteristics. However, it is important to acknowledge the difference between the dynamic response measured *in situ* and the response predicted by an idealized computational model under selected loading scenarios. Therefore, to help calibrate computational building models, and get more realistic results and improved understanding of their dynamic properties, three different categories of *in situ* tests have been developed through the last century (see a review in Hans et al. 2005) :

1) Ambient vibration test (AVT): Owing to technological advances in sensing techniques, this method has received more attention from the 1990s and has been the most popular method for testing real structures in recent years. AVT is of easy application in large structures, low cost and its results are reliable. High resolution sensors are available at relatively low cost (for engineering studies), which can measure ambient horizontal accelerations of the order of 10^{-5} g at the base level to 10^{-4} g at the top of the buildings.
2) Harmonic forcing (shaker): a harmonic shaker with controlled forcing frequency is used to identify the resonant natural frequencies of the structure. A typical device usually induces a horizontal acceleration of the order of 10^{-4} g at the building base and 10^{-3} g at the top, which is about 10 times greater ambient levels.
3) Shocks: Shock testing on buildings is performed by impacting the upper part of the structure separately along the two principal axes by means of a heavy mechanical shovel (impactor). This shock loading induces transient accelerations that are about a thousand times greater than the ambient level.

In all these tests, the accelerations are small enough to keep the structure within its elastic range of response. Several authors (Trifunac 1972, Lamarche et al. 2008) have shown that forced and ambient tests will lead to consistent agreement of modal parameters of the building structure. Hans et al. (2005) also showed with three building test campaigns that all three techniques yielded almost to the same results. Their study also confirmed that building natural frequencies tend to decrease while the vibration amplitude increases (from ambient to shock load), but this reduction was observed to be very small, about only 2 to 5% while the excitation amplitude had been increased by 10^3.The same trend has also been observed by other researchers: the reliability of AVT in real structures has been proved and such techniques have been used worldwide for updating finite element models (Venture et al. 2001, Yu et al. 2007a, Yu et al. 2007b, Tremblay et al. 2008, Lamarche et al. 2009), detecting changes in dynamic behavior of structures after retrofitting or damage, structural identification and predicting the seismic behavior of buildings (Gilles 2011, Gentile and Gallino 2008, Michel et al. 2008).

This paper presents results extracted from ambient vibration tests performed on three emergency shelters in Montréal. In total, nine buildings were tested and details of measurements, building characteristics and mode shapes are presented next. These are low and mid-rise buildings with both plan and vertical irregularities, thus exhibiting coupled sway and torsional modes in the low frequency range.

3.3 Ambient vibration testing procedure

3.3.1 AVT set up and protocol

In the study, seven Tromino™ wireless sensors (tromographs – see www.tromino.eu) are used to record horizontal and vertical accelerations and velocities of building roof/floors. A typical set up of the instrument is shown in Figure 3.1: it comprises the sensor itself (small red box) and a radio antenna amplifier which allows the network of sensors to communicate. The system is completely wireless and the data is recorded directly in the sensor for eventual download to a computer using a standard USB connection.

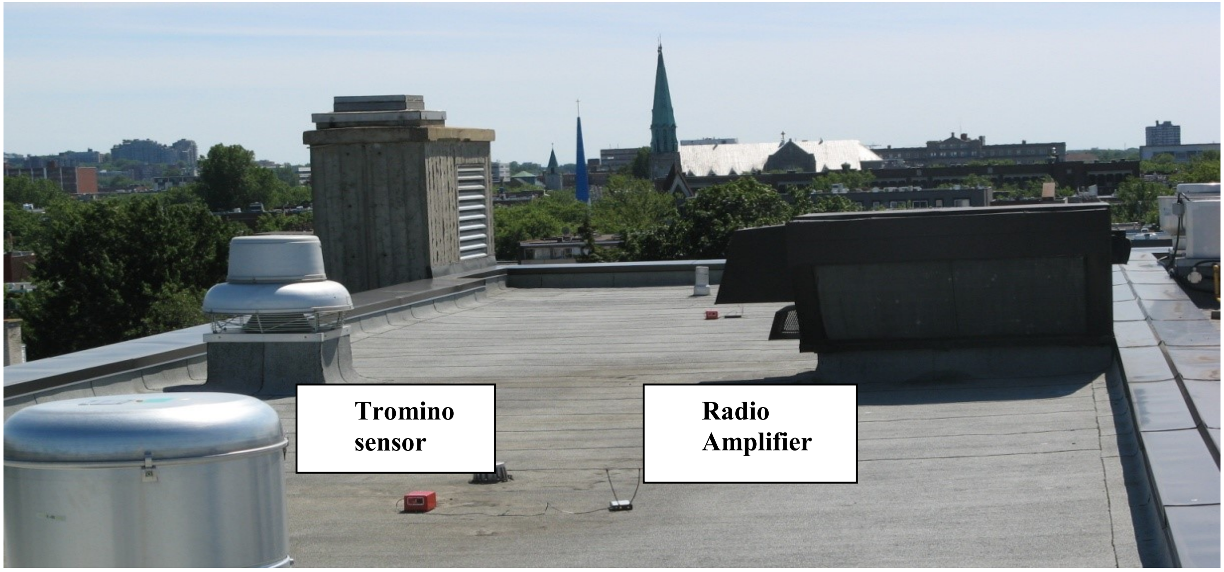


Figure 3.1 One Tromino™ sensor and its radio antenna amplifier

For each building, velocities resulting from ambient excitations (such as wind, traffic outside the building, normal building operations and human activities) are measured at least in three different locations on all floors, typically one near the center and the other two far away from the center, along a principal axis of rigidity or a main geometric axis. Each sensor records velocities and accelerations in three orthogonal directions: two in the horizontal plane and one in the vertical direction. Moreover, as the study deals with low rise buildings, some of them could have flexible roofs and in such cases, more sensors are deployed on the roof to capture this effect. Two sensors (instead of only one) are designated as reference and are deployed on the top floors and far from the center of rigidity. In general, ten-minute data records were taken at a sampling frequency of 128 Hz. Since only building frequencies below 20 Hz are of interest, the recorded data were decimated in order of three to reduce noise.

3.3.2 Estimation of modal parameters

In this study, ARTeMIS™ software (Solution, S.V. 2010) Handy Extractor version is used to treat the recorded data. Two methods are available, Frequency Domain Decomposition (FDD) and Enhanced Frequency Domain Decomposition (EFDD), to extract the lower frequency modal parameters of the buildings. The results obtained from both methods are compared, and the final modal characteristic estimates reported in this paper are based on the authors' view.

An important step in any frequency domain system identification method is to calculate power spectral densities (PSD) of recorded data. Spectral density is a direct measure of a signal's energy content per unit frequency. Therefore, it is a useful mathematical tool to identify frequencies that contribute the most energy to a particular signal: if the input signal is a white noise, then the peaks of the output PSD function correspond to the natural frequencies of the system. The first step of Frequency Domain Decomposition (FDD) method is to estimate the spectral densities between all the recorded data channels to assemble the PSD matrix, $G_{xy}(\omega)$. The PSD is defined as the expected value of the product of Fourier transforms of all pairs of recorded data. However, as indicated in Equation (3.1), the PSD is estimated by dividing each signal into n sub-records of shorter duration and, omitting the expected value operation, and averaging the multiplication of corresponding pairs of discrete Fourier transforms.

$$G_{xy}(\omega) \approx \frac{1}{n}\sum_{a=1}^{n} X^{a}(\omega)Y^{a}(\omega)^{*} \tag{3.1}$$

where * denotes the complex conjugate, $X(\omega)$ and $Y(\omega)$ are the discrete Fourier transforms of corresponding time history records and n is the number of sub records.

It is possible to estimate resonant natural frequencies by the classical frequency domain method called peak-picking, which involves plotting each spectral density function by considering one element of the PSD matrix over the frequency range of interest, and identifying the peaks as natural frequencies. However, this classical method has difficulties to identify closely-spaced modes. Therefore, two other more sophisticated methods (FDD and EFDD) have been used in the study to overcome this problem. The two methods are briefly presented next.

Frequency Domain Decomposition (FDD)

FDD is consisted of decomposing the PSD matrix into its eigenproblem form by singular value decomposition as follows:

$$G\ (\omega) = [U(\omega)][S(\omega)][U(\omega)]^H \tag{3.2}$$

Where H means Hermitian transformation, [U] is unitary matrix (containing the singular vectors), [S] is the matrix of singular values and G is the power spectral density matrix. The singular vectors represent the system mode shapes and the corresponding singular values provide an estimate of the contribution of each mode to overall energy at each frequency. In fact, the singular value decomposition of the output PSD matrix is an approximation to its modal decomposition (Brincker et al. 2001). Resonant frequencies are identified from the peaks on the first singular value plot, and at each resonant frequency, the first singular vector provides an estimate of the associated mode shape.

Enhanced Frequency Domain Decomposition (EFDD)

EFDD adds a modal estimation layer to the FDD peak-picking. It proceeds in two steps: the first step is to perform FDD peak picking as described above, and the second step is to use the FDD-identified mode shape to construct a single-degree-of-freedom (SDOF) spectral bell function which is used to estimate the natural frequency and damping ratio for the mode. The construction of the SDOF spectral bell is performed using the FDD identified mode shape as reference vector and proceeds with a correlation analysis based on a modal assurance criterion (MAC) (see Equation 3.3). MAC values are calculated between the reference vector and the other singular vectors (on each side of the FDD-identified frequency). If the largest MAC value of these vectors exceeds a user-specified MAC Rejection Level (set to 0.8 in the study) then the corresponding singular values are included in the description of the SDOF Spectral Bell.

$$MAC(\{\varphi_1\},\{\varphi_2\}) = \frac{\left|\{\varphi_1\}^H.\{\varphi_2\}\right|^2}{\left|\{\varphi_1\}^H.\{\varphi_1\}\right|.\left|\{\varphi_2\}^H.\{\varphi_2\}\right|} \tag{3.3}$$

The natural frequency and damping ratio are computed by transferring the SDOF spectral bell to time domain. This time function is similar to the auto correlation function of the velocity of a linear SDOF oscillator subjected to white noise excitation, and it is straightforward to determine the function frequency and equivalent viscous damping ratio by simple linear regression (Brincker et al. 2001).

3.4 Mode shapes of the tested shelters

3.4.1 Complex A: *Patro Le Prévost*

This complex constructed in 1975 and comprising five joint-separated buildings is illustrated in Figure 3.2. The north direction is an assumed reference that will be used for consistency in the orientation of all sensors.

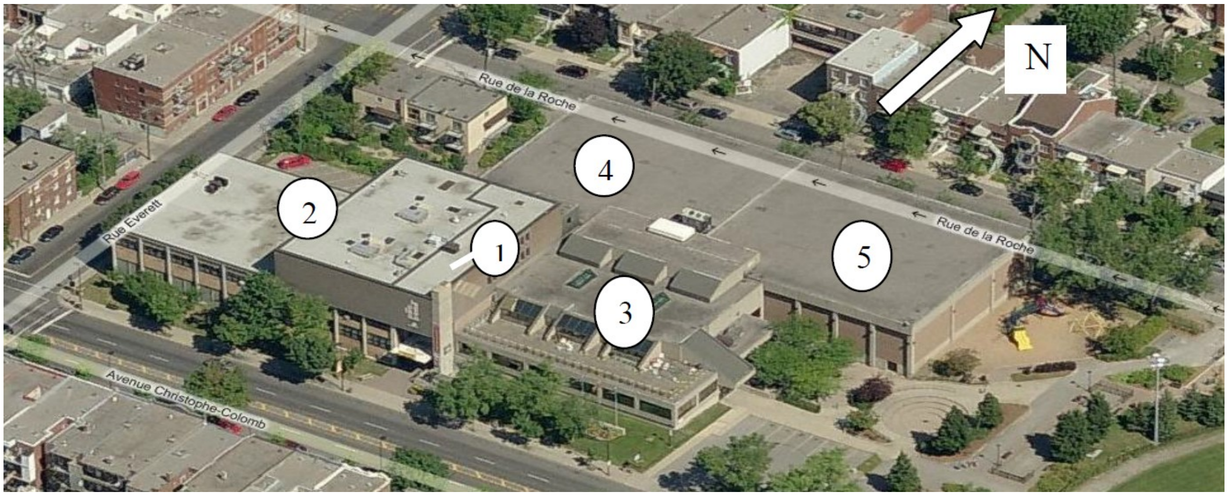


Figure 3.2 Complex A - *Patro Le Prévost*'s Bird's eye view

Building 1

This building is a six-story reinforced concrete moment frame, which comprises two basements, with total height of 21 m (above the foundation level). The first three floors have a rectangular shape approximately 6.4 m by 32 m, and the upper three stories have an L-shape plan (Figure 3.2). The position of the sensors used for all test setups combined together is shown in Figure 3.3: this layout is created in the ARTeMIS software and serves an approximate representation of the building shape. The blue arrows in the figure are the two reference sensors (also marked with R) and the rest (shown in green color) are roving sensors. AVT results with these sensors have allowed the extraction of the first three modes shapes of the building, illustrated in Figure 3.4.

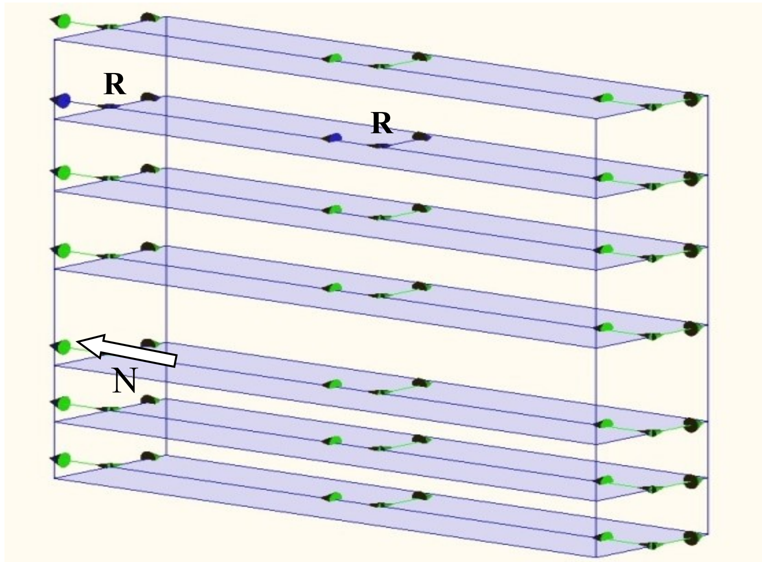


Figure 3.3 Sensor positions of all test set-ups - Complex A Building 1

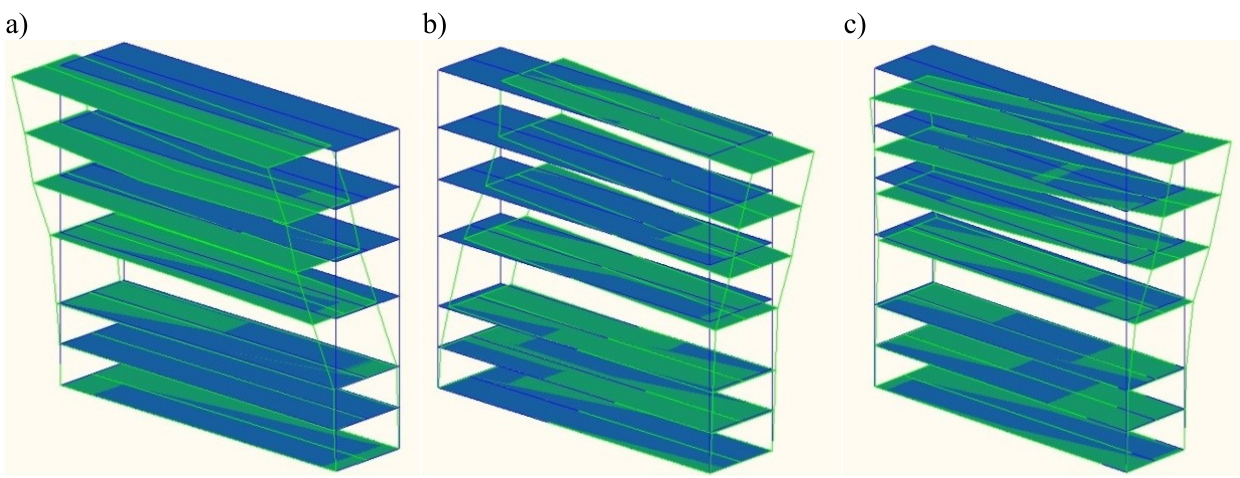


Figure 3.4 Mode shapes of Complex A Building 1: a) 1st flexural-torsional mode (3.33Hz); b) 2nd flexural-torsional mode (4.52 Hz); c) 1st torsional mode (5.47 Hz)

Building 2

This building is a five stories irregular reinforced concrete moment frame with the total height of 21 m. The bottom floors are rectangular, approximately 32 m by 46 m, respectively; however, the two upper floors have smaller dimensions of approximately 19 m by 32 m, respectively. The sensor positions for all test setups combined are shown in Figure 3.5, and once again the three lowest frequency mode shapes were extracted from the records and are illustrated in Figure 3.6.

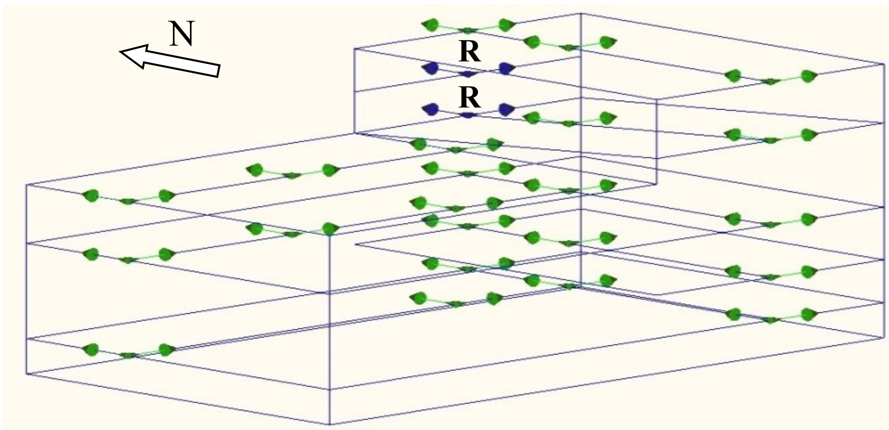


Figure 3.5 Sensor positions of all test set-ups - Complex A Building 2

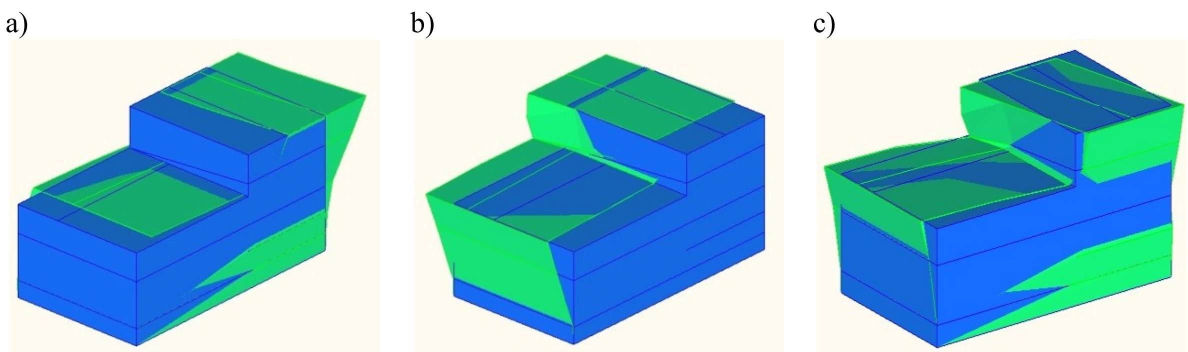


Figure 3.6 Mode shapes of Complex A Building 2: a) 1st flexural-torsional mode (3.38 Hz); b) 2nd flexural-torsional mode (4.56 Hz); c) 1st torsional mode (5.47 Hz)

Due to space limitations, not all tested buildings could be described in details here. The remainders three are *Patro Le Prévost*–Building 3, a concrete moment frame with approximate dimensions of 32m by 32 m and 14 m height, and Buildings 4 and 5, two similar two stories braced steel frame buildings of approximate dimensions of 31 m by 37 m and total height of 13 m, including the basement floor.

3.4.2 Complex B: *Centre Pierre-Charbonneau*

This complex is shown in Figure 3.7 and comprises three joint-separated buildings constructed in 1957. All three buildings are made of reinforced concrete moment frames. Building 1 has a rectangular plan with approximate dimensions of 20 m by 58 m for the ground floor. It has three stories (the first story is a basement) with a total height of 15 m. Building 2 has a rectangular plan with approximate dimensions of 11 m by 52 m for the ground floor: it has three stories with a total height of 11 m. Building 3 is a large single-storey gymnasium with a rectangular plan of 52 m by 58 m.

The sensor positions and the three lowest frequency mode shapes extracted from AVT are shown in Figure 3.8 and Figure 3.9, respectively.

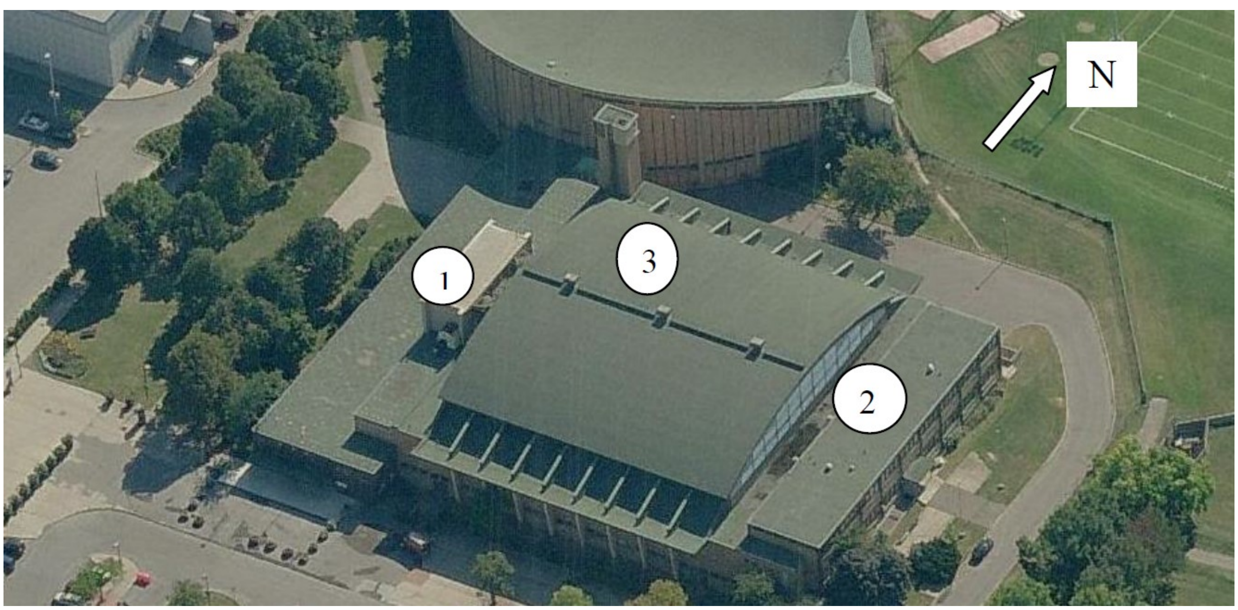


Figure 3.7 Complex B - *Centre Pierre-Charbonneau's* Bird's eye view

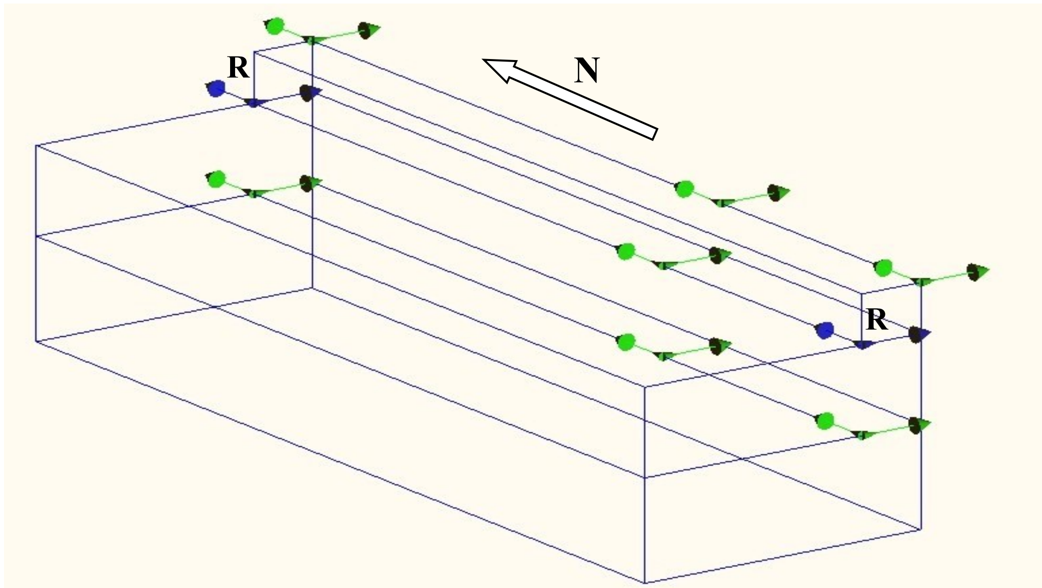


Figure 3.8 Sensor positions of all test set-ups – Complex B – Building 1

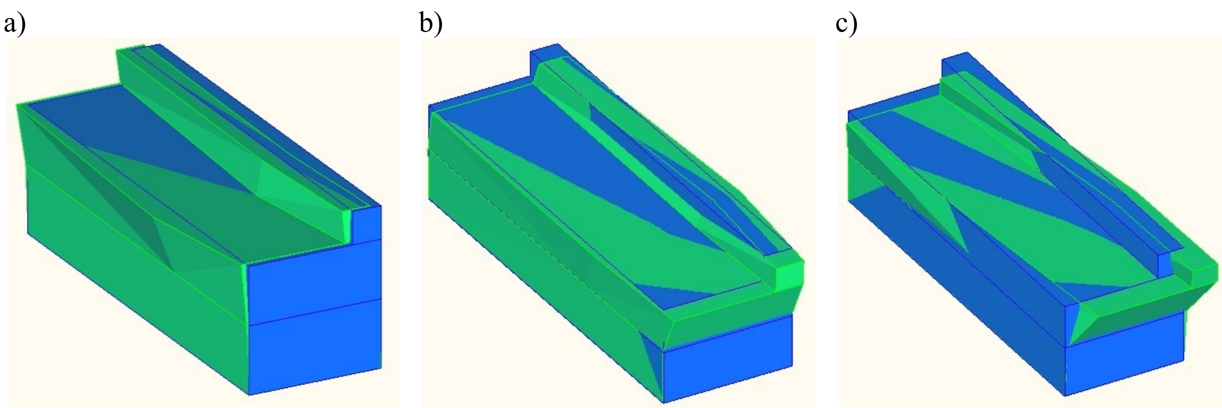


Figure 3.9 Mode shapes of Complex B - Building 1 1: a) 1st flexural-torsional mode (6.75 Hz); b) 2nd flexural-torsional mode (8.61 Hz); c) 1st torsional mode (9.95 Hz)

3.4.3 Complex C *: Centre Communautaire de Loisirs de la Côte-des-Neiges*

This complex shown in Figure 3.10 is a single building constructed in 1993 with a braced steel frame structural system. It has a typical plan of 30 m by 42 m and a total height of 20 m including one basement floor. The sensor positions and the three lowest frequency mode shapes extracted from AVT are shown in Figure 3.11 and Figure 3.12, respectively.

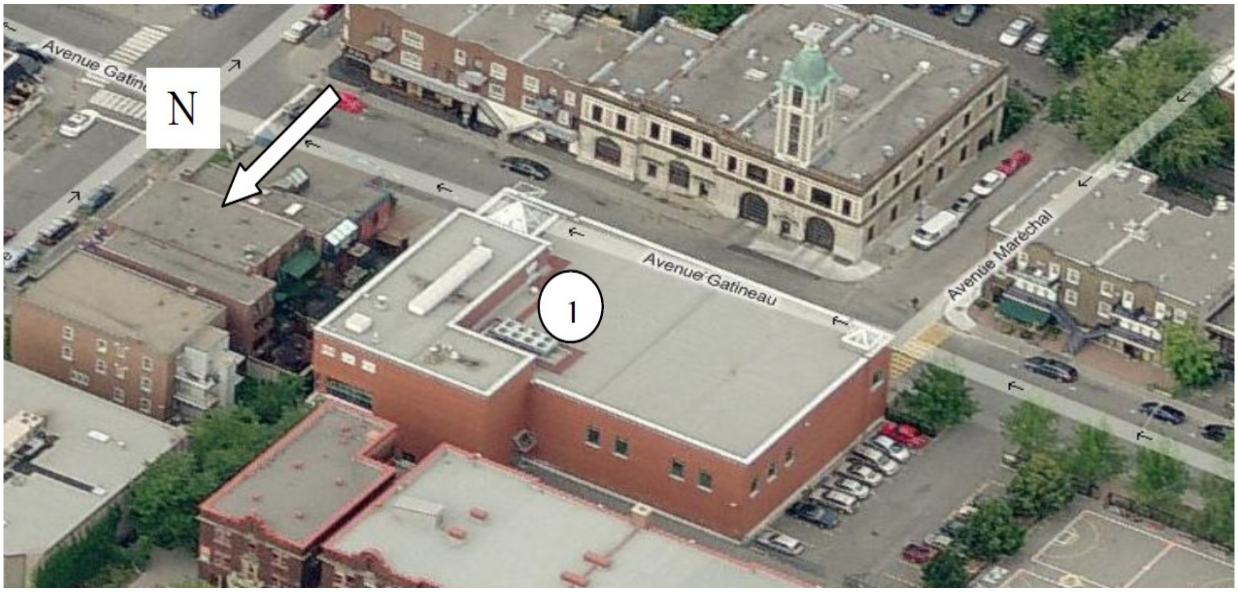


Figure 3.10 Complex C - *Centre Communautaire de Loisirs de la Côte-des-Neiges* Bird's eye view

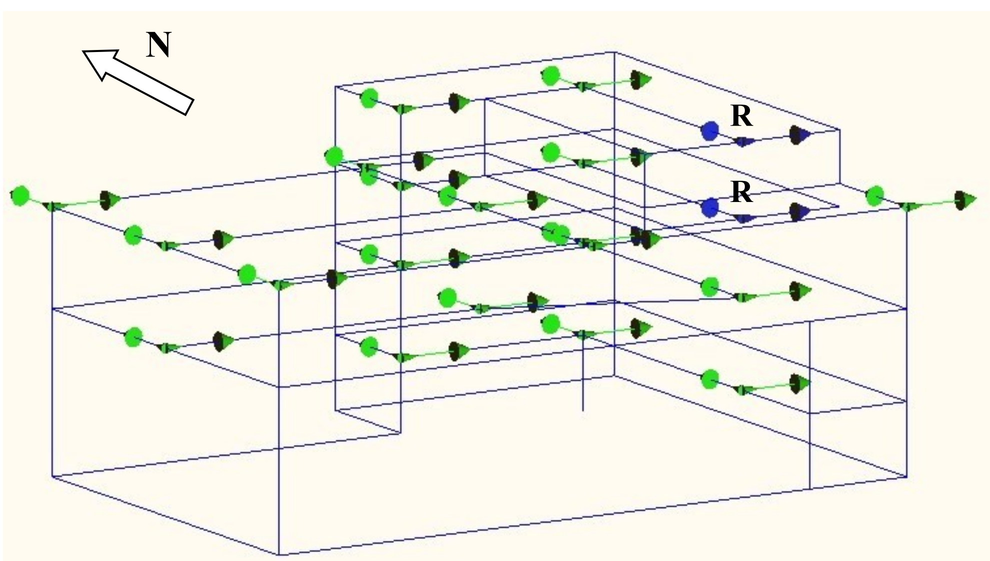
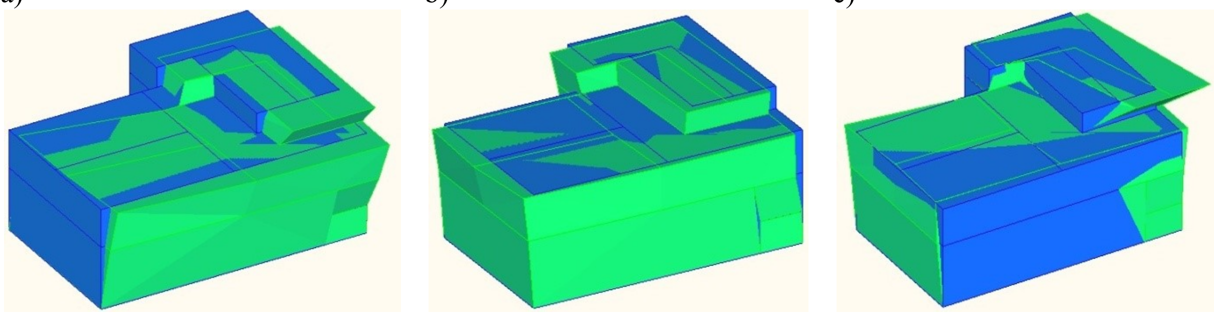


Figure 3.11 Sensor positions of all test setups – Complex C



a) b) c)

Figure 3.12 Mode shapes of Complex C : *Centre Communautaire de Loisirs de la Côte-des-Neiges:* a) 1st flexural-torsional mode (4.13 Hz); b) 2nd flexural-torsional mode (4.19 Hz); c) 1st torsional mode (5.67 Hz)

3.5 Natural frequencies and modal damping ratios

The values of the three lowest natural frequencies and corresponding modal damping ratios corresponding to emergency shelters illustrated in Section 3.4 are summarized in Table 3-1. The frequencies are chosen from the best result obtained from FDD and EFDD according to their mode shape configurations. The damping ratio estimates are only available from EFDD, using the logarithmic decrement of the autocorrelation function obtained from the translation of the Spectral Bell function in the time domain. For some cases, the damping ratio or frequency of the third mode could not be identified and the entry NA is shown in the table.

It is seen in the table that it was not possible to identify any of the fundamental frequencies of Building 3 of Complex B Centre Pierre-Charbonneau: this single story building is a large gymnasium with a curved roof and access to the roof to install sensors was not granted. Otherwise, the frequencies of at least the first two modes could be identified like the other eight buildings.

Table 3-1 AVT estimated natural frequencies and modal damping ratios for the first three modes

Building	**First mode**		**Second mode**		**Third mode**	
	Frequency (Hz)	Damping (%)	Frequency (Hz)	Damping (%)	Frequency (Hz)	Damping (%)
Complex A-Bldg 1	3.33	2.3	4.52	2.9	5.47	2.6
Complex A-Bldg 2	3.38	2	4.56	2.3	5.47	1.6
Complex A-Bldg 3	6.69	NA	7.78	NA	9.45	NA
Complex A-Bldg 4	3.73	2.5	5.44	2.3	NA	NA
Complex A-Bldg 5	3.31	3.2	5.23	1.5	NA	NA
Complex B-Bldg 1	6.75	2.9	8.61	1.4	9.95	2.4
Complex B-Bldg 2	5.42	1.5	5.69	1.3	9.99	2
Complex B-Bldg 3	NA	NA	NA	NA	NA	NA
Complex C	4.13	1.6	4.19	1.2	5.67	2.3

The coupled flexural-torsional mode shapes obtained for most buildings confirm that these buildings are characterized by structural irregularities. Such irregularities (vertical and horizontal) are usually obvious from the building topologies, but even buildings shape that look symmetric in geometric have eccentricities between their center of mass and center of rigidity at different floor levels.. However, the general trend in the results is that despite the coupling, the first two modes are mainly translational, while the third is torsional. Also, the approximate modal damping values extracted from the results are all below 3.2% viscous critical.

3.6 Conclusion

This paper has presented partial results of an on-going research project on field of assessment of low-rise irregular buildings. These results show the feasibility of AVT and structural identification for low and mid-rise irregular buildings. Coupled sway and torsional mode shapes expected to exist in these types of buildings are indeed identified. These test results are part of a larger database that will be used to validate a simplified procedure based on experimental structural parameters to assess the seismic vulnerability of irregular buildings.

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Link between chapter 3 and chapter 4

Chapter 3 presented ambient vibration test set-ups as well as the theoretical basis of the two methods available for analyzing the measured vibrations to extract the lower frequency modal parameters of the buildings, namely Frequency Domain Decomposition (FDD) and Enhanced Frequency Domain Decomposition (EFDD). Then nine irregular low and mid-rise buildings located in Montreal, Canada were tested by AVT and the corresponding lowest modal properties, i.e. natural frequencies, mode shapes and damping ratios, were reported.

In chapter 4, results of the remaining seven buildings in the database (16 tested buildings in total) are presented and several modal properties of the whole database that were not shown in the previous chapter are reported. Discussions about AVT protocols are more in depth in this chapter as special considerations for proper testing of irregular low and mid-rise buildings are necessary. Based on the experience of the previous tests, the sampling frequency in the tests of chapter 4 is changed from 128 Hz to 512 Hz to provide more data and help better capturing of the modes. It should be mentioned that the few cases for which the natural frequencies and damping ratios could not be derived in chapter 3, were reanalyzed here. An improved application of the EFDD method, removing spurious noise from the records, and proper decimation have led to successful capturing of the modal properties in the re-analyzing phase. At the end of chapter 4, a detailed discussion of the AVT results of the whole database is presented, with a comparison of the derived natural frequencies with the ones obtained from the empirical formulas of the National Building Code of Canada.

4. Ambient vibration tests on irregular low-rise buildings

4.1 Abstract

This paper presents the operational modal analysis results of 16 ambient vibration tests (AVT) performed on low and mid-rise irregular buildings designated as emergency shelters in Montreal, Canada. Fundamental mode shapes, modal frequencies and corresponding modal damping ratios are determined from the records using frequency domain decomposition methods. The rate of success of AVTs in this study to capture at least the three lowest natural frequencies/modes are unlike previous studies where difficulty of performing AVT for low-rise buildings were reported. Based on the experience of these tests different layouts of sensor positioning for successful capturing of torsional and rigid/flexible roof mode shapes of irregular buildings, suitable sampling frequency and recording duration are proposed. Furthermore, natural periods of concrete structures and braced steel frames are compared with the Canadian building code period formulas and results show agreement in the case of braced steel frames. This study can inform other researchers on practices to perform high quality ambient vibration testing on low rise irregular structures.

4.2 Introduction

An ambient vibration test (AVT) is a non-destructive output only experiment which is gaining popularity in building testing as compare to forced vibration test due to its low cost, similarities in accuracy of extracted modal properties and the fact that this vibration test is done in normal operational conditions with no need for an external source of excitation. When a building is subjected to dynamic loads, its structural response depends on the frequency content and magnitude of the forces exciting the structural system, the dynamic properties of the building (natural frequencies, mode shapes and damping ratios), the variation of these parameters in time during strong motions, as well as the soil type and foundation underneath the superstructure. Therefore, the first step to predict the dynamic response of a structure is to estimate its natural or operational dynamic characteristics. To this end, three different categories of in situ tests have been developed through the last century (see a review in Hans et al. 2005):

- Ambient vibration test (AVT): Owing to technological advances in sensing techniques, this method has received more attention during the 1990s and has been the most popular method for testing real structures in recent years. AVT is of easy application in large structures, low cost and its results are reliable. High resolution sensors are available at relatively low cost (for engineering studies) that can measure ambient accelerations, typically in the order of $10^{-5}g$ at the base level and $10^{-4}g$ at the top of buildings.
- Harmonic forcing (shaker): A harmonic shaker with controlled forcing frequency is used to identify the resonant natural frequencies of the structure. A typical device usually induces a horizontal acceleration of the order of $10^{-4}g$ at the building base and $10^{-3}g$ at the top, which is about 10 times the usual ambient levels.
- Shock testing: Shock testing on buildings is performed by impacting the upper part of the structure separately along the two principal axes by means of a heavy mechanical shovel (impactor). This shock loading induces transient accelerations that are about a thousand times greater than those experienced at ambient level.

In all these tests, the accelerations are small enough to keep the structure within its elastic range of response. Several authors (Trifunac 1972, Lamarche et al. 2008) have shown that forced and ambient tests will lead to consistent agreement of modal parameters of the building structure. Hans et al. (2005) also showed with three building test campaigns that all three techniques yielded almost to the same results. Their study also confirmed that building natural frequencies tend to decrease while the vibration amplitude increases (from ambient to shock load), but this reduction was observed to be very small, about only 2 to 5% while the excitation amplitude had been increased by factor of 10^3. The same trend has also been observed by other researchers, therefore, the reliability of AVT to capture modal properties of structures within their linear elastic limit with enough accuracy has been proved. AVT results have been used worldwide for updating finite element models (see for example Ventura et al. 2001, Yu et al. 2007a and b, Tremblay 2008), structural identification and examining natural period formulas in codes such as concrete shear wall (Gilles and McClure 2012, Farsi and Bard 2004) and light wood-frame buildings (Hafeez et al. 2014), as well as predicting the seismic behavior of buildings (Michel et al. 2008).

In the previous studies of ambient vibration tests for low-rise buildings difficulties in deriving good quality modal properties have been reported (Tobita et al., 2000 and Tischer et al. 2012). For instance, Tischer was able to identify the first three modal properties for only 28 of 101 tested low-rise buildings. Therefore, there is still need for research in modal identification of low-rise buildings. This paper presents and discusses details of measurements, building characteristics and modal properties extracted from ambient vibration tests performed on a set of 16 buildings designated as emergency shelters in Montreal, Canada. These are all low and mid-rise buildings with both plan and vertical irregularities thus exhibiting coupled sway and torsional modes in the low frequency range. Moreover, some of them have flexible roofs and/or foundation flexibility which needs special sensor positioning, record duration and sampling frequency. The rate of success of AVTs in this study to capture at least the three lowest natural frequencies/modes are unlike previous studies where difficulty of performing AVT for low-rise buildings were reported. Fundamental natural periods of the buildings are compared with the Canadian building code period formulas and results show agreement in the case of braced steel frames.

4.3 Ambient vibration testing procedure

During normal operation, a building structure is subjected to ambient vibrations of low amplitudes resulting from wind, tremors, occupants, surrounding traffic, etc. The fundamental assumption behind ambient vibration test is that the input causing motion has white noise properties in the frequency range of interest. This hypothesis implies that the input forces are not driving the structure at any particular frequency and therefore any identified frequency linked with significant strong response reflects a natural frequency of the vibrating structure. However, in reality some of the ambient noises may drive the structure at a particular frequency; for instance, an adjacent rotatory machine. In the latter case, the deformed shape of the structure at that frequency is called an operational mode. Therefore, there is a need to analyze AVT results with significant care to be able to differentiate natural structural modes from enforced operational modes (Ventura et al. 2003).

In this study, six Tromino™ wireless sensors (tromographs – see www.tromino.eu) are used to record horizontal and vertical accelerations and velocities of building roof/floors. Each device is comprised of the sensor itself and a radio antenna amplifier which allows the network of sensors to communicate (Figure 4.1); the sensors record velocities and accelerations in three orthogonal directions: two in the horizontal plane and one in the vertical direction. This system is completely wireless and the data are recorded directly in the sensor for eventual download to a computer using a standard USB connection. Performing a successful AVT requires careful preparation: visiting the site of each building in advance of the test date, investigating the lateral resisting and structural systems, available spaces and floor's covers where the sensors will be located, collecting all available emergency, architectural and structural plans, and finally preparing an appropriate strategy for the test layouts of sensor positions.

(a)

(b)

Figure 4.1 (a) Tromino sensor and radio amplifier; (b) Example of a radio network of Trominos. In order for the radio synchronization to work properly, the master sensor (TR-1) should be close to the slave sensor (TR-2). The slave sensor 2 should be close to the slave 3; the slave 3 close to slave 4 etc. (source: Tromino manual)

In the tests reported here, AVTs velocities are measured on proper points that reflect the global structural response; i.e. located far from center of rigidity, operating machinery or surfaces with carpet cover, usually at building corner locations to capture good quality signals as well as torsional behavior. Two sensors, one as back up, are designated as reference sensors and are deployed on the top floors where they can be excited by all modes. Having a backup reference sensor proves an asset for validating ambiguous results during the data extraction process. To capture good quality mode shapes in low rise buildings, there is no choice except putting sensors on the rooftop. In the presence of asphaltic roofing materials, it may be needed to use some sort of adjustable spikes to level the sensing devices (proper integrity should be kept between the sensor and spikes), which can lead to degradation in the quality of the records. Therefore, it is a good practice to use more sensors on the roofs than otherwise necessary on level and hard-surface interior floors. Furthermore, large roofs of low-rise buildings can be flexible therefore a layout of sensors at least on every one-third of the spans, such as used in the cases reported here, is needed to capture the in-plane mode shapes of the roof itself. Also, when the first reference sensor is located on the rooftop, an attempt should be made to put the second reference sensor on a hard-surface inside of a top floor level to increase quality of the recorded data. Ten-minute data recording is used in

AVTs in this study. Recording time is a critical value and should not be short especially for low-rise buildings to improve the quality of processing; experience of this research showed that ten-minute data recording is sufficient for low-rise building and improved extracting procedure of the structural mode shapes. The sampling frequency is set to 512 Hz which is higher than what is needed to find natural frequencies of buildings. However, experience of these 16 AVTs has shown that this higher sampling value is beneficial; provides the option to decimate data to any desirable order which increases signal to noise ratio, is an asset during the analysis phase, provides more data and can help better capturing of the modes.

In this research, ARTeMIS™ software (Solution, S.V. 2010) Handy Extractor version is used for record analysis. Two methods are available, Frequency Domain Decomposition (FDD) and Enhanced Frequency Domain Decomposition (EFDD), to extract the lower frequency modal parameters of the buildings. Both these methods display singular values of the spectral density matrices. The peaks represent either structural modes or operational modes: it is by comparing the results of both methods and investigating the shapes of the modes that the structural mode shapes can be identified. Details of the FDD and EFDD methods may be found in Brincker et al. 2001 and summarized in Mirshafiei and McClure 2012.

4.4 Modal properties of the tested buildings

Using the AVT procedure described above, 16 buildings located in Montreal are tested and their dynamic properties are derived. Testing details and results are discussed in this section.

4.4.1 Complex A: *Patro Le Prévost*

This complex constructed in 1975 and comprising five joint-separated buildings is illustrated in Figure 4.2 (separation lines are shown in red color). The north direction is an assumed reference that will be used for consistency in the orientation of all sensors.

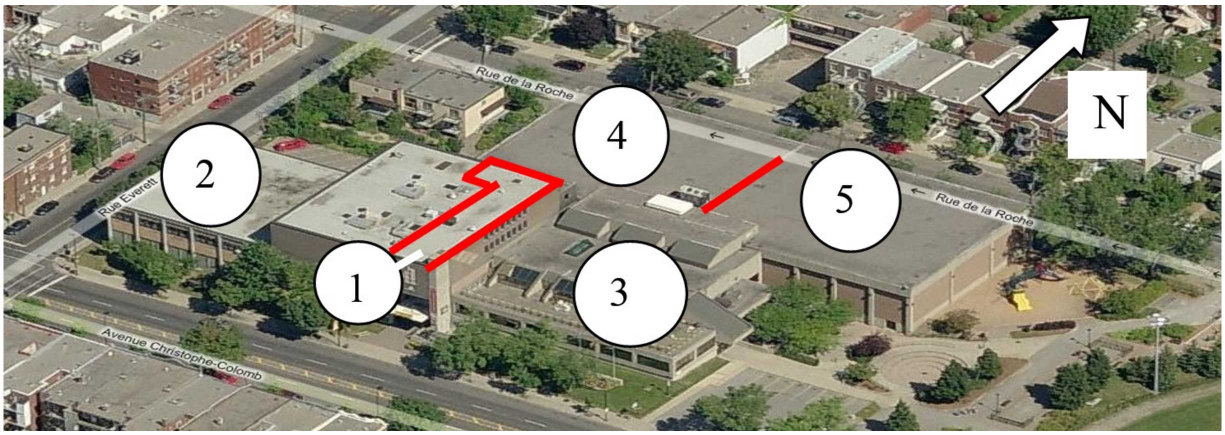

Figure 4.2 Complex A - Patro Le Prévost's Bird's eye view

Building 3

This building is a reinforced concrete moment frame with a height of about 13.5 m including one basement floor; 10.8 m above the ground level. Its floor plans are shown in Figure 4.3 where sensor positions are depicted by large black dots (reference sensors have a "R" notation). The building has concrete slabs with sufficiently large thickness to act as rigid diaphragms, therefore putting two sensors on each floor is sufficient, and three sensors (one back up) are deployed on each floor and in the corridors so as to cause the least disturbance for the residents; this sensor layout had led to successful capturing of the lowest three modal properties.

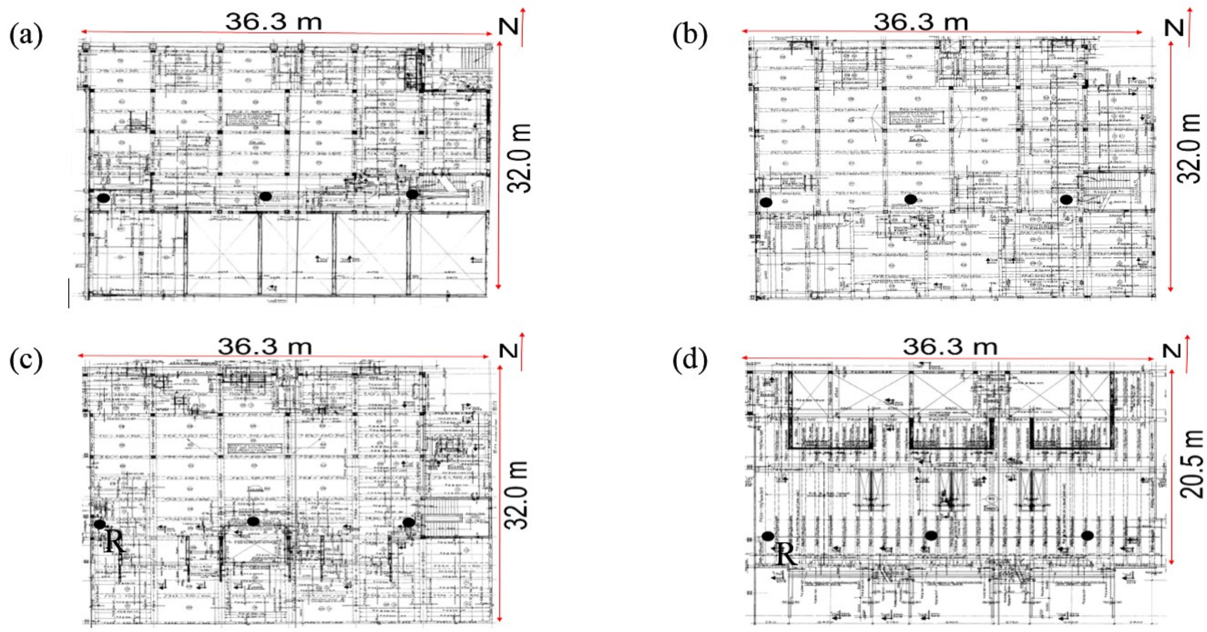

Figure 4.3 (a) Ground floor; (b) 1st floor-height above ground 3.3 m; (c) 2nd floor-height above ground 7.4 m; (d) Roof-height above ground 10.8 m.

AVT records were analyzed by ARTeMIS™ (Solution, S.V. 2010) and mode shapes, and corresponding natural periods and damping ratios are shown in Figure 4.4; the undeformed mode shape is represented in blue and the deformed shape in green color. In some parts green and blue colors coexist and green boundary lines are used to identify the mode shapes. No movement is identified at the ground level by inspection of the experimental mode shapes.

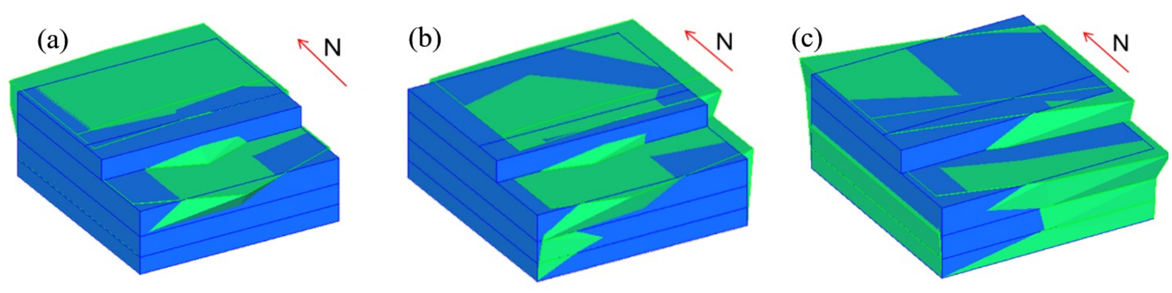

Figure 4.4 Mode shapes a) 1st flexural mode N-S dir. (0.15 s, damping ratio=0.021); b) 1st flexural mode E-W dir. (0.13 s, damping ratio=0.018); c) 1st torsional mode (0.11 s, damping ratio=0.021)

Buildings 4 and 5

These buildings are almost identical with the same structural system, architectural layout and material. They have a braced steel frame lateral structural system and height of about 13 m including one basement floors; 10 m above the ground level. Floor plans, sensor layout and modal properties for building 4 are shown in Figure 4.5 (almost identical modal properties were derived for building 5 so they are not being presented here). The mode shapes of these buildings exhibit only small flexibility in the roof so they can be assumed rigid for structural analysis. Details of modal properties of buildings 1 and 2 can be found in Mirshafiei and McClure 2012.

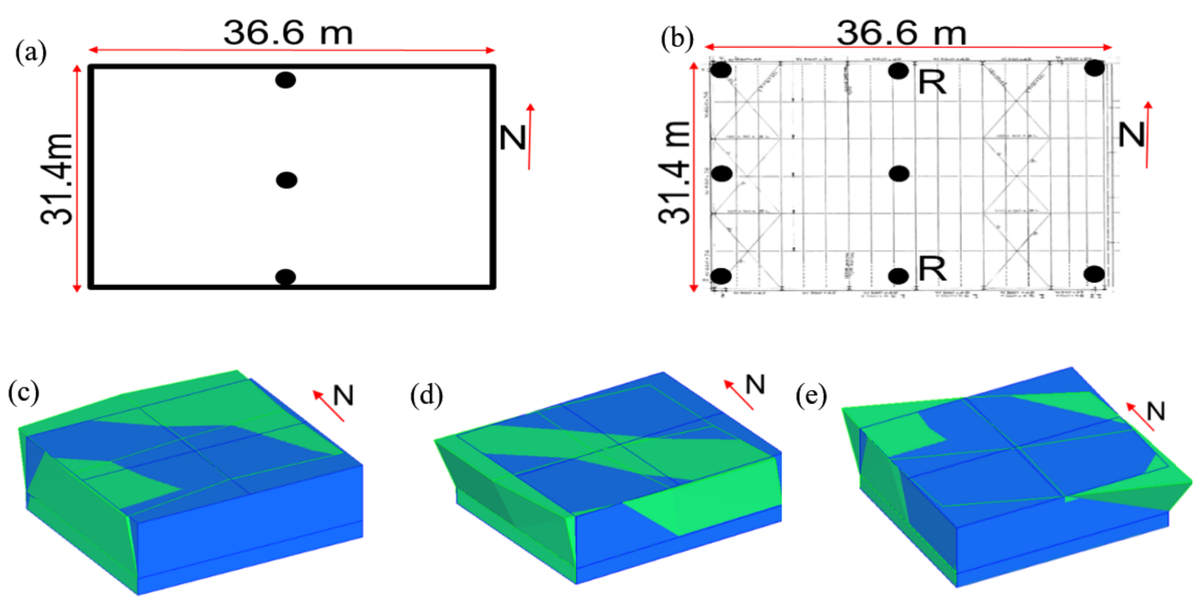

Figure 4.5 (a) Ground floor; (b) Roof-height above ground 10 m; (c) 1st flexural mode N-S dir. (0.27 s, damping ratio=0.025); (d) 1st flexural mode E-W dir. (0.18 s, damping ratio=0.023); (e) 1st torsional mode (0.1 s, damping ratio=0.012)

4.4.2 Complex B: *Centre des loisirs de Saint-Laurent*

This Centre is a single building constructed in 1993 with a reinforced concrete moment frame lateral structural system and height of about 11.7 m including one basement floor; 8.4 m above the ground level. A bird's view of the building and its floor plans are shown in Figure 4.6. Because this building has a large roof, the selected sensor layout on the roof is such as to capture possible flexible behavior. The first reference sensor is located off from center and corner, at the South-East section of the roof, to capture larger ambient motion and the second bending mode shape of the flexible roof. Due to the low building height, the second reference sensor is also placed on the roof at the North-West corner to allow capturing of the general torsional behavior. Modal properties of this building are shown in Figure 4.7. From observation of the modes shapes it is verified that floors and roof behave rigidly (all are concrete slabs).

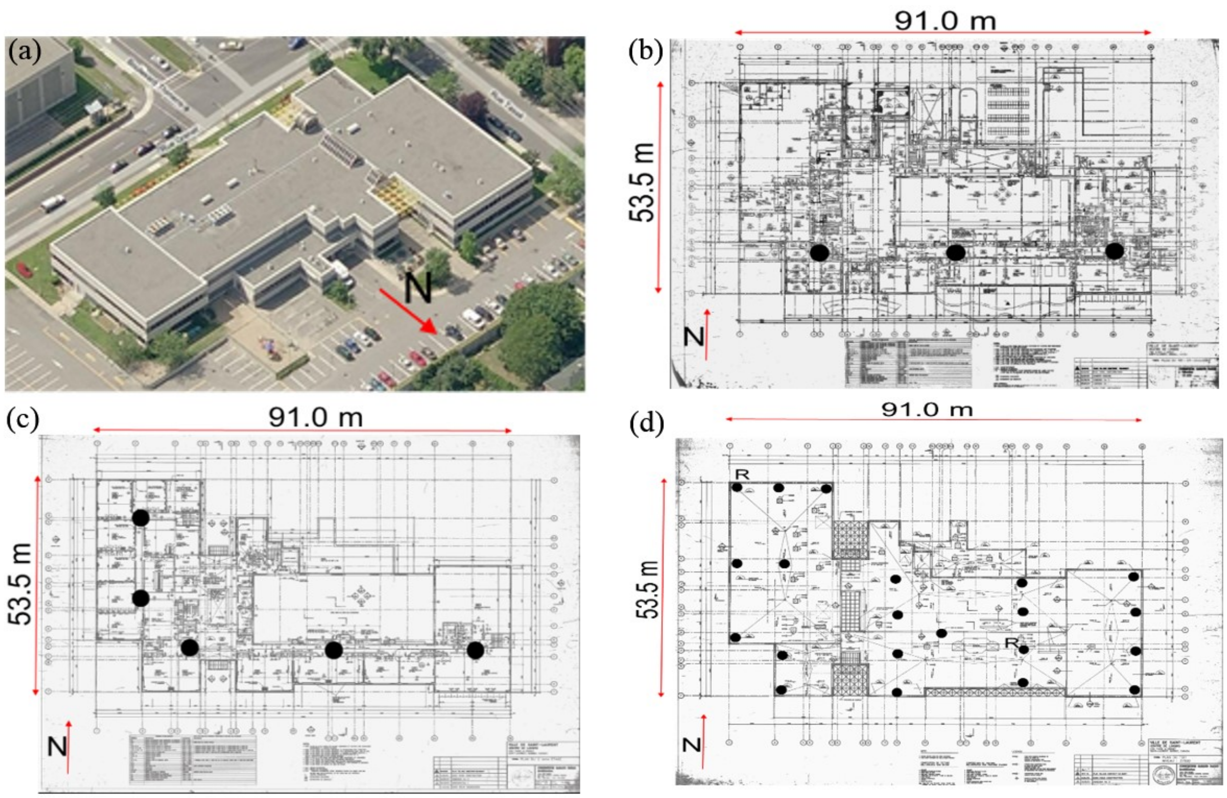

Figure 4.6 (a) Bird's eye view; (b) Ground floor; (c) 1st floor-height above ground 3.9 m; (d) Roof-height above ground 8.4 m

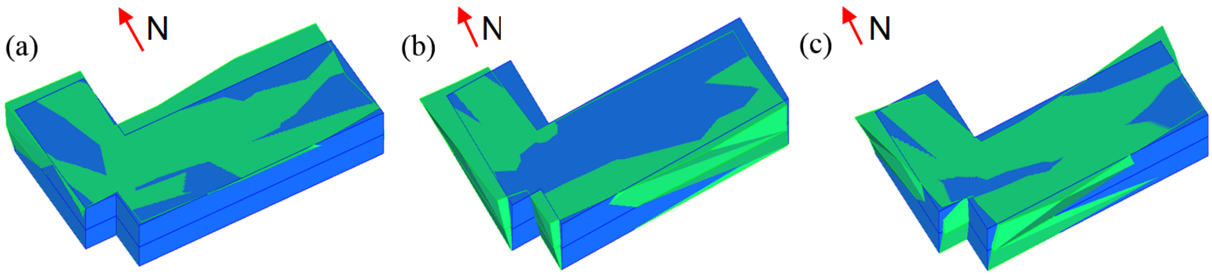

Figure 4.7 Mode shapes a) 1st flexural-torsional mode N-S dir. (0.19 s, damping ratio=0.020); b) 1st flexural-torsional mode E-W dir. (0.18 s, damping ratio=0.018); c) 1st torsional mode (0.13 s, damping ratio=0.021)

4.4.3 Complex C: *Centre du Plateau*

This building was constructed in 1961 with a reinforced concrete moment frame structural system and height of about 13.1 m including one basement floor; 8.4 m above the ground level. A bird's view of the building and its floor plans are shown in Figure 4.8 and its modal characteristics are shown in Figure 4.9. Sensors are located on the basement floor but no effect of foundation flexibility is observed. To verify rigidity of the roof a second test was done only on the roof with a different sensor layout (Figure 4.8f); one sensor is located off from center and corner to capture the second roof bending mode and other sensors are deployed in the middle of spans to capture the fundamental roof bending mode. However, the modal properties were found similar in the two tests and no roof flexibility was detected, which is consistent with the existence of a heavy roof concrete slab.

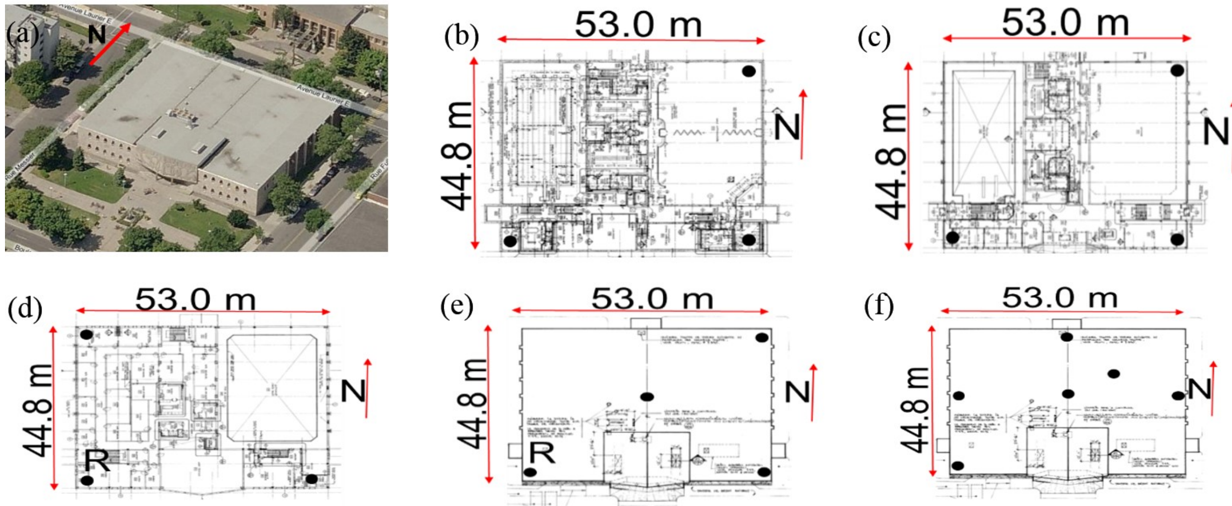

Figure 4.8 (a) Bird's eye view; (b) Basement-4.7 m below ground level; (c) Ground floor; (d) 1st floor-height above ground 4.2 m; (e) Roof-height above ground 8.4 m; (f) Second AVT done only on the roof to investigate possibility of flexible behavior

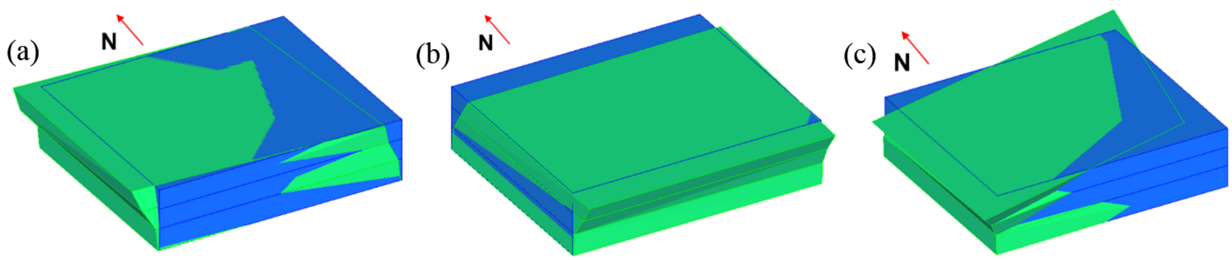

Figure 4.9 Mode shapes a) 1st flexural mode N-S dir. (0.23 s, damping ratio=0.017); b) 1st flexural-torsional mode E-W dir. (0.21 s, damping ratio=0.017); c) 1st torsional mode (0.16 s, damping ratio=0.033)

4.4.4 Complex D: *Centre sportif de LA Côte-des-Neiges*

This building was constructed in 1996 with a steel braced frame lateral structural system and height of about 11.6 m including one basement floor; 8. 4 m above ground level. A bird's view of the building and its floor plans are shown in Figure 4.10. The sensor layout on the roof is refined so as to capture possible flexible behavior. The reference sensors are chosen based on the procedure explained for Complex B. AVT records were analyzed and mode shapes, natural periods and damping ratios are shown in Figure 4.11. Sensors where placed on the basement floor but no movement was identified at the ground or basement levels by observing the experimental mode shapes. Furthermore, the mode shapes indicate that this roof is flexible and its first bending modes in both directions are derived which confirm the capability of such AVT procedure to identify flexible roofs.

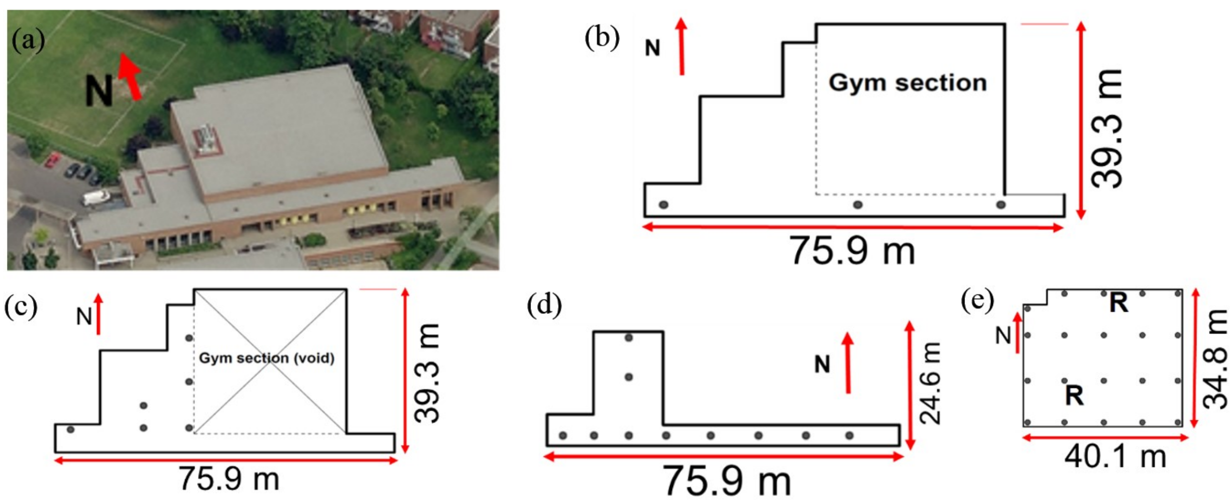

Figure 4.10 (a) Bird's eye view; (b) Basement 3.1 m below ground level; (c) Ground level; d) Bottom roof-height above ground 6 m; (e) Upper roof-height above ground level 8.4 m

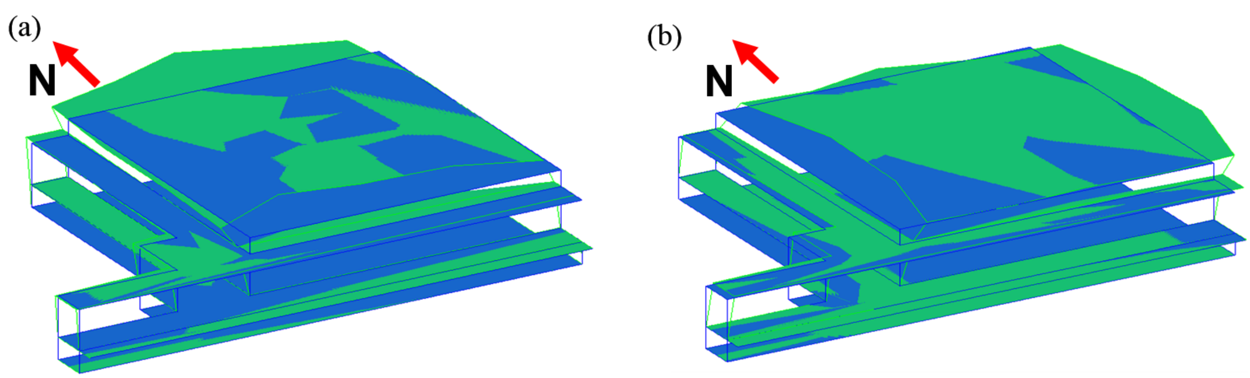

Figure 4.11 Mode shapes a) 1st mode-flexible roof N-S dir. (0.24 s, damping ratio=0.040); b) 1st mode-flexible roof E-W dir. (0.17 s, damping ratio=0.021)

4.4.5 Complex E: *Centre Roger-Rousseau*

This Centre is a single building constructed in 1976 with braced steel frame structural system and height of about 7.9 m above the ground level. A bird's view of the building and its floor plans are shown in Figure 4.12. Three tests were done on this building. The sensor layout for the first test is shown in Figure 4.12 and is a set up for an assumed rigid floor/roof building; the corresponding mode shapes, natural periods and damping ratios are shown in Figure 4.13. A second test was performed on the roof to identify any significant flexible behavior. The sensor layout is shown in Figure 4.14a, which is a typical one to be used for a flexible roof, with sensors placed on all one-third points of the spans. At least one reference sensor is placed at one-third of spans in both directions as shown in the Figure 4.14a away from one corner. The modal properties are reported in Figure 4.15. Results show that the roof has flexible in-plane behavior which was not identified from the 1st AVT test. This illustrates that significant attention is needed for large roofs and low-rise buildings, otherwise, some possible flexible roof modes can be missed. Finally, due to the presence of different roofing materials (see Figure 4.14b), a third test was performed on the roof to observe the effects of different surface layers on the quality of the acquired data. Sensors 1, 3 and 4 where placed on the different materials (typical membrane layer used on roofs, a gravel and bumpy elastic surfaces, respectively). The frequency contents of the recorded velocities show almost similar trends and peaks, which justify the sensor layout and the reliability of previous tests regardless of the difference in roofing material. It should be mentioned that the highest quality signal was recorded by sensor one; i.e. peaks corresponding to natural frequencies had higher energy (less noise was observed).

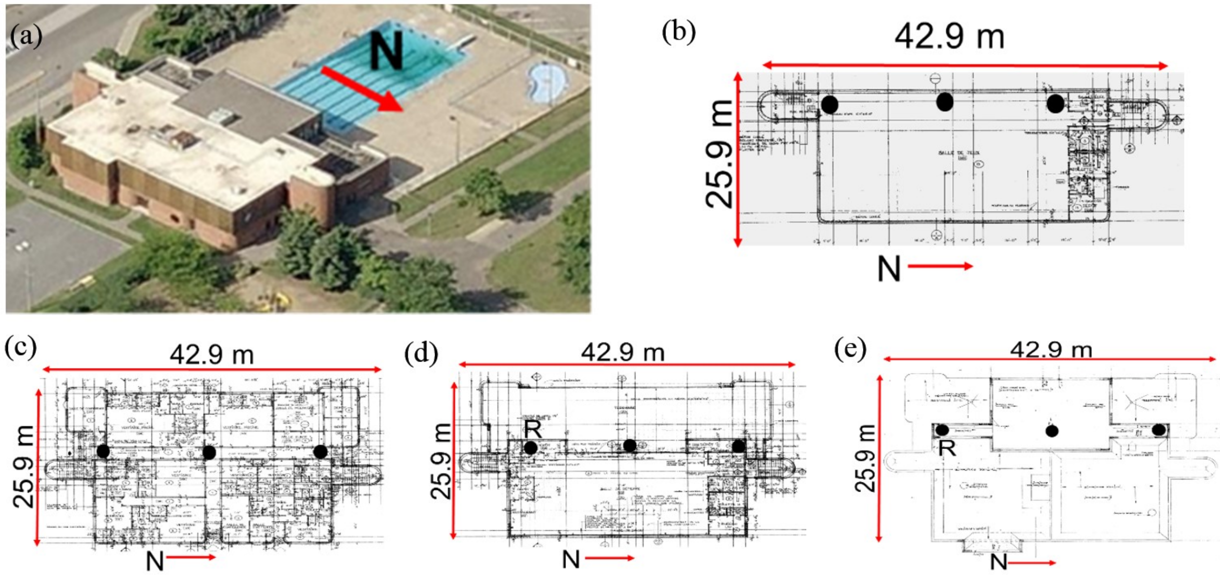

Figure 4.12 (a) Bird's eye view; (b) Basement 3.7 m below ground level; (c) Ground level; (d) 1st floor-height above ground level 3.7 m; (e) Roof-height above ground level 7.9 m

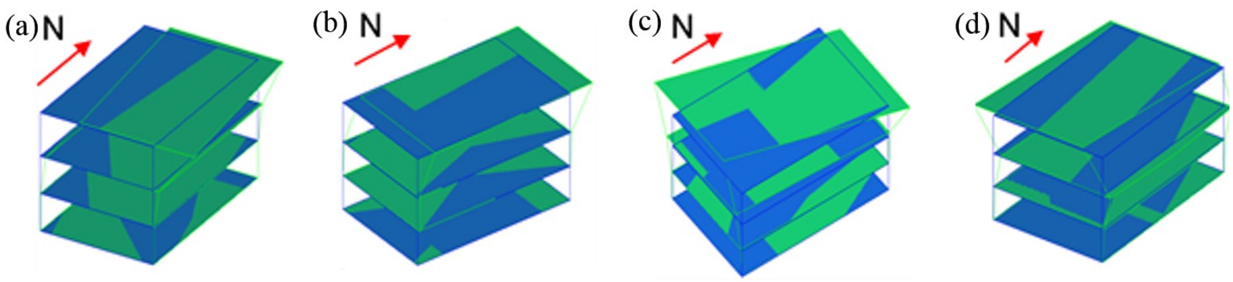

Figure 4.13 Mode shapes (a) 1st flexural-torsional mode E-W dir. (0.18 s, damping ratio=0.06); (b) 1st flexural-torsional mode N-S dir. (0.13 s, damping ratio=0.02); (c) 1st torsional mode (0.09 s, damping ratio=0.016); (d) 2nd flexural-torsional mode E-W dir. (0.08, damping ratio=0.01)

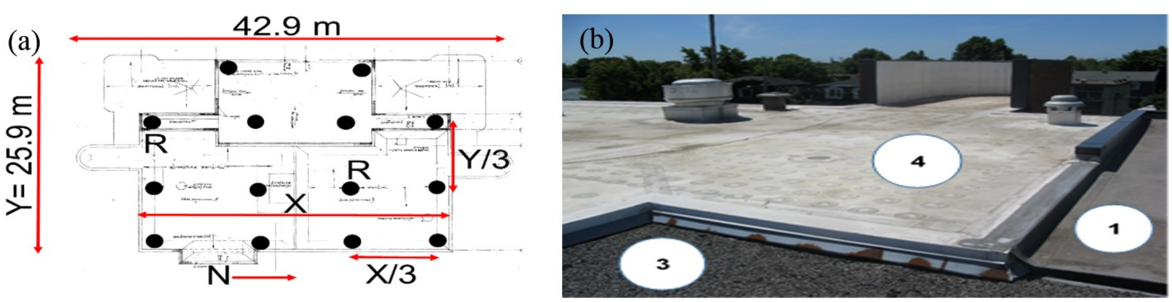

Figure 4.14 (a) Sensor layout for roof to capture flexible behavior; (b) Investigation of the effects of underneath layers on signal acquisition

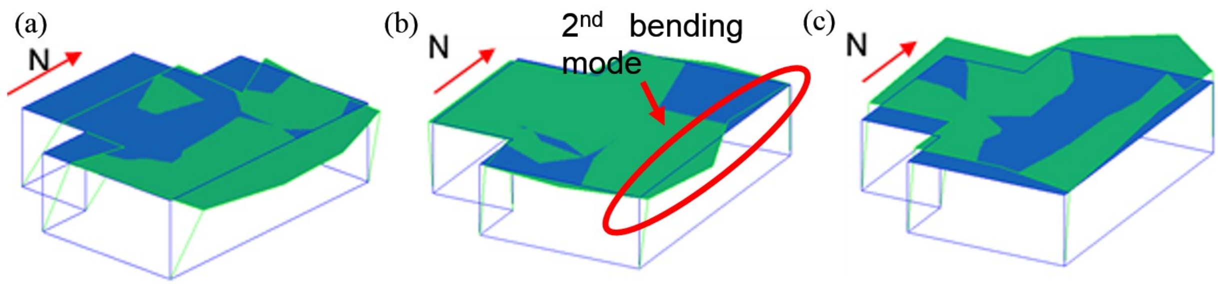

Figure 4.15 Mode shapes (a) 1st bending mode of roof E-W dir. (0.18 s, damping ratio=0.060); (b) 2nd bending mode of roof E-W dir. (0.14 s, damping ratio=0.042); (c) 1st bending mode of roof N-S dir. (0.13 s, damping ratio=0.020)

4.4.6 Complex F: *Centre Roussin*

This complex is comprised of three separate buildings. Building 1 was constructed in 1964 with reinforced concrete moment frame and height of 17.1 m including one basement floor; 13 m above the ground level. A bird's view of the building, a typical floor plan and the derived 6 modes from AVT are shown in Figure 4.16. Reference sensors were located in the south corner of the two upper floors. By looking at the mode shapes movement is identified at the ground and basement levels, which shows the importance of putting sensors on the basement and ground floor for cases that are sensitive to foundation flexibility. Building 2 was constructed in 1914 with a steel frame structural system with unreinforced masonry walls and height of 18.6 m above ground level; it has no basement. A typical floor plan and the derived first three modes from AVT are shown in Figure 4.17. Moreover, effects of foundation flexibility are again detected in the mode shapes. Building 3 was constructed in 1964 with a reinforced concrete moment frame with heavy infill walls and height of 17.1 m above ground level. Sensor locations (roof is a concrete slab so is assumed to be rigid) and modal properties are shown in Figure 4.18. This case is an illustration that special attention should be paid to repeated modes when analyzing AVT in frequency domain (using either FDD or EFDD methods), For instance, in the second building modes 2 & 3 have repeated frequencies, and in the third building modes 1 & 2 have also repeated frequencies.

Therefore, the two largest singular value curves of spectral density matrices have their high energy peaks at this repeated frequency. Hence, it is important to pick the first mode on the first singular value curve and the second mode on the second singular value curve.

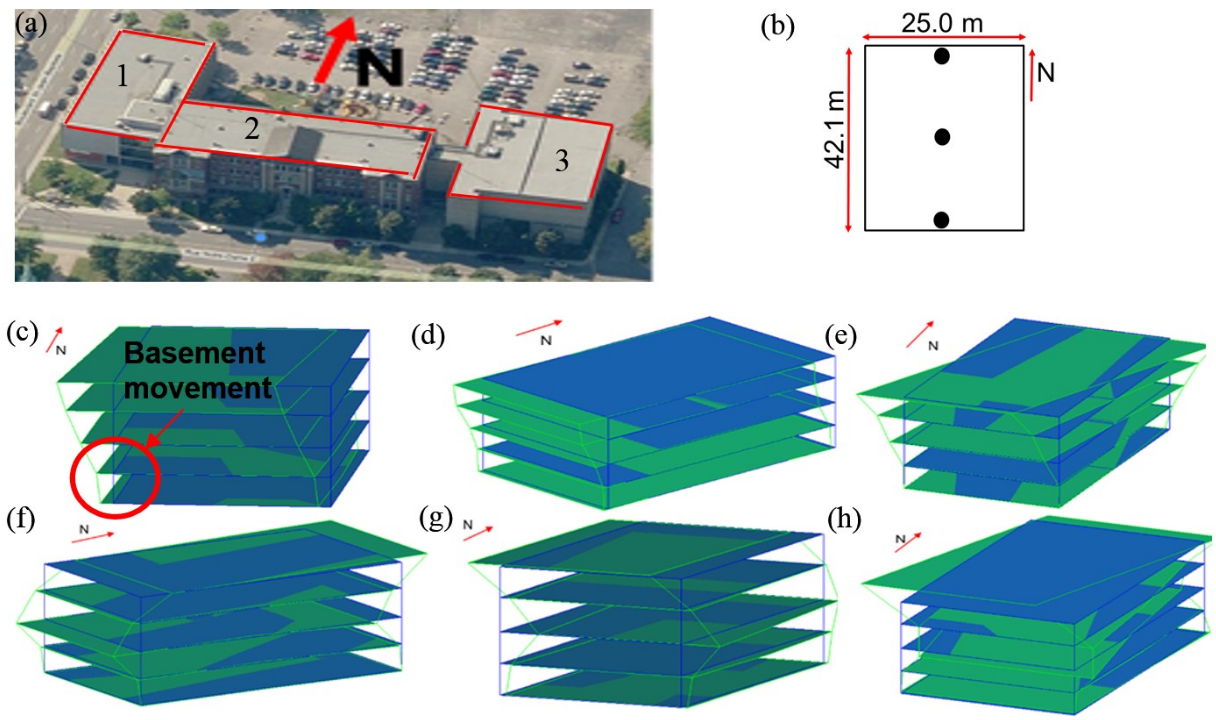

Figure 4.16 (a) Bird's eye view; (b) A typical floor plan; (c) 1st flexural-torsional mode E-W dir. (0.38 s, damping ratio=0.041); (d) 1st flexural mode N-S dir. (0.38 s, damping ratio=0.040); (e) 1st torsional mode (0.23 s, damping ratio=0.030); (f) 2nd flexural mode N-S dr. (0.13 s, damping ratio=0.020); (g) 2nd flexural mode E-W dir. (0.12 s, damping ratio=0.023); (h) 2nd torsional mode (0.1 s, damping ratio=0.010)

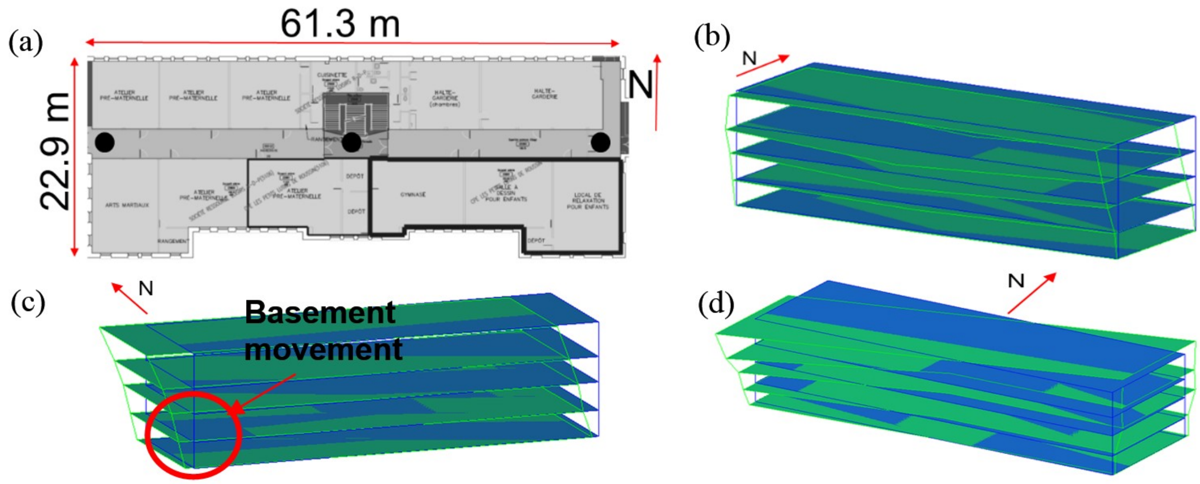

Figure 4.17 (a) A typical floor plan; (b) 1st flexural mode N-S dir. (0.5 s, damping ratio=0.05); (c) 1st flexural mode E-W dir. (0.38 s, damping ratio=0.05); (d) 1st torsional mode (0.38 s, damping ratio=0.037)

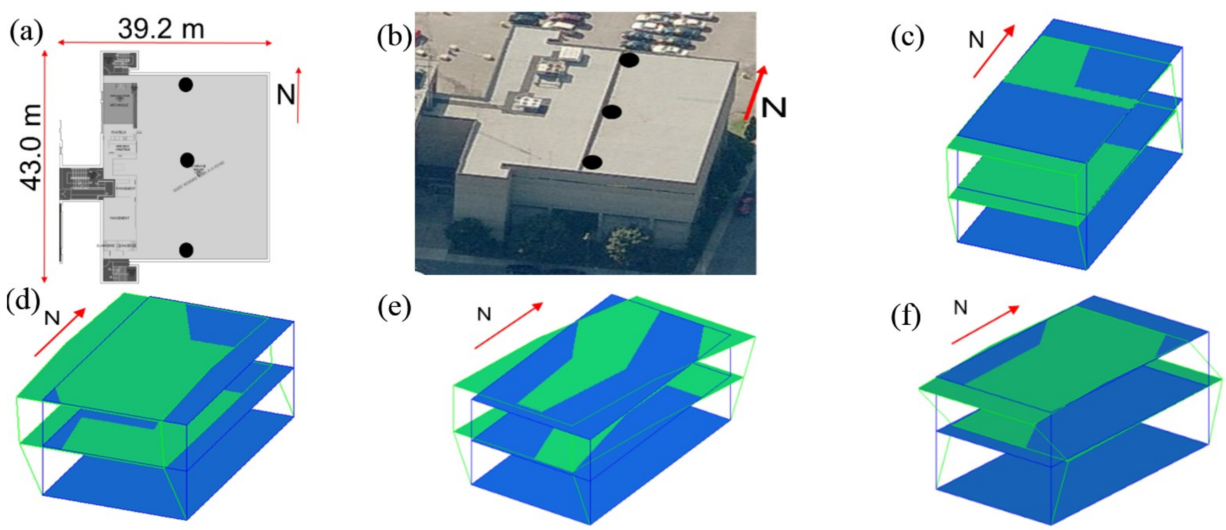

Figure 4.18 (a) 1st floor-height above ground level 8.8 m; (b) Roof-height above ground level 17.1 m (c) 1st flexural mode N-S dir. (0.38 s, damping ratio=0.036);(d) 1st flexural mode E-W dir. (0.38 s, damping ratio=0.039); (e) 1st torsional mode (0.15 s, damping ratio=0.014); (f) 2nd flexural mode N-S dir. (0.12 s, damping ratio=0.028)

4.5 Summary of results, natural periods and damping

The main characteristics of all 16 AVT tested buildings have been listed in Table 4-1. The lateral force resisting system (LFRS) types are categorized according to the FEMA 154 guideline where C1, C2, S2 and S5 stand for the following types, respectively: Concrete moment resisting frames, Concrete shear wall buildings, Braced steel frame buildings and Steel frame buildings with unreinforced masonry infill walls. Moreover, longest plan dimensions and the height above zero level (h_n), which is used to calculate the fundamental periods according to NBCC 2010, are shown in this table. Details of all buildings were explained in the previous section except for complex G (*Centre Pierre-Charbonneau*) and H (*Centre Communautaire de Loisirs de la Côte-des-Neiges*) which were omitted due to space limitations; their detailed information can be found in Mirshafiei and McClure 2012. For 15 out of the 16 tested low/medium rise irregular buildings at least the lowest three modal properties were derived by AVT procedure and the final results are listed in Table 4-2 (Complex D has a flexible roof and the first two mode shapes were identified).

The fundamental period of these buildings is also calculated according to the period formulas of the National Building Code of Canada (NRC/IRC. 2010) and results are shown in Table 4-2. The period formula $0.075*(h_n)^{3/4}$ is used for C1 structures. The same expression has been adopted by Uniform Building Code (UBC, 1997) and other modern building codes for concrete moment frames which is based on regression of vibration data measured on a set of buildings during the 1971 San Fernando earthquake and the afterward data set studied in Goel and Chopra 1997; the data used by Goel and Chopra (1997) are coming from structures shaken strongly but not deformed into the inelastic range. On the other hand, Table 4-2 shows that fundamental periods calculated from NBCC code are longer than AVT results. This is expected considering the low-amplitude vibrations measured in AVT, non-cracked sections and the effects of the presence of non-structural components. If the contribution of these non-structural components towards the building lateral stiffness is changed during higher amplitude vibrations, damage may occur and period elongation is expected (soil-structure interaction or foundation flexibility can also affect modal properties in stronger shakings). As a conclusion from the results shown, the code predicted periods for concrete moment frames are 1.95±0.45 (mean value ± standard deviation) times greater than AVT results.

In the case of braced steel frames the NBCC period formula is "$0.025*h_n$" proposed by Tremblay 2005 is in very good agreement with AVT results (0.98±0.15 times of AVT results). It is noteworthy that Tremblay proposed this period formula for braced steel frames through theoretical and parametric studies of 220 braced frame buildings, only two data points from a two-story building were measured periods while all others were periods from analytical models. Kwon & Kim 2010 and Günaydın & Topkaya 2013 also reported that this formula follows the building periods better than previously available equations (for instance NBCC 1995 and Eurocode 8). Therefore, the natural periods obtained from ambient vibration testing of braced steel frames can be used for higher vibration levels as they are in the range of predicted code period; the code formulas are representatives of fundamental sway mode periods when the building shakes strongly but only suffers no or little damage. Moreover, even though limited flexible roof behavior has been captured for community centers D and E (steel braced frames), the AVT results are very close to code predictions; however, one can expect to obtain longer periods in AVT and in stronger excitations as compared to the code formula for large single-story steel buildings with significant roof flexibility (Tremblay et al. 2008). It should be mentioned that Complex F-Bldg. 2 is a medium-rise steel frame building with unreinforced masonry infill walls and the code period formula "$0.05*(h_n)^{3/4}$" provides a value close to the AVT result.

According to the data collected, the mean values of equivalent viscous damping ratios for reinforced concrete and braced steel structures are 2.5±0.9 and 3.5±1.7 percentage of the critical damping (Complexes E&D with flexible roofs have increased damping levels for the steel type), respectively. These values are in the range of and consistent with what is expected under working stress conditions, no more than about half yield point (Chopra 2007). It should also be mentioned that damping ratios are expected to get larger under stronger vibrations.

Table 4-1 Tested buildings' characteristics

Name	Year	LFRS	Height (m) above ground	Dimensions (m*m)
Complex B	1993	C1	8.4	91*53.5
Complex C	1961	C1	8.4	53*44.8
Complex G-Bldg. 1	1957	C1	5.5	58*20
Complex G-Bldg. 2	1957	C1	7.7	52*11
Complex F-Bldg. 1	1964	C1	13	42*25
Complex F-Bldg. 3	1964	C1	17.1	43*39.2
Complex A-Bldg. 1	1975	C1	15.9	32*6.4
Complex A-Bldg. 2	1975	C1	18.6	46*32
Complex A-Bldg. 3	1975	C1	10.8	36.3*32
Complex G-Bldg. 3	1957	C1&C2	18	52*58
Complex E	1976	S2	7.9	42.9*25.9
Complex H	1993	S2	11.4	41.2*32.3
Complex D	1996	S2	8.4	75.9*39.3
Complex A–Bldg. 4	1975	S2	10	36.6*31.4
Complex A-Bldg. 5	1975	S2	10	36.6*31.4
Complex F-Bldg. 2	1914	S5	18.6	61.3*22.9

Table 4-2 Modal properties of the tested buildings extracted from ambient vibration measurements

Building	First mode			Second mode (AVT)		Third mode (AVT)	
	Period (s)		Damping (%)	Period (s)	Damping (%)	Period (s)	Damping (%)
	AVT	NBCC					
Complex B	0.19	0.37	2.0	0.18	1.8	0.13	2.1
Complex C	0.23	0.37	1.7	0.21	1.7	0.16	3.3
Complex G-Bldg. 1	0.15	0.27	2.9	0.12	1.4	0.10	2.4
Complex G-Bldg. 2	0.18	0.35	1.5	0.17	1.3	0.10	2.0
Complex F-Bldg. 1	0.38	0.51	4.1	0.38	4.0	0.23	3.0
Complex F-Bldg. 3	0.38	0.63	3.6	0.38	3.9	0.15	1.4
Complex A-Bldg. 1	0.30	0.60	2.3	0.22	2.9	0.18	2.6
Complex A-Bldg. 2	0.29	0.67	2.0	0.22	2.3	0.18	1.6
Complex A-Bldg. 3	0.15	0.45	2.1	0.13	1.8	0.11	2.1
Complex G-Bldg. 3	0.23	0.44	2.6	0.12	1.0	0.10	1.1
Complex E	0.18	0.2	6.0	0.14	4.2	0.13	2.0
Complex H	0.24	0.28	1.6	0.24	1.2	0.18	2.3
Complex D	0.24	0.21	4.0	0.17	2.1	N/A	N/A
Complex A–Bldg. 4	0.27	0.25	2.5	0.18	2.3	0.10	1.2
Complex A-Bldg. 5	0.30	0.25	3.2	0.19	2.0	0.10	1.5
Complex F-Bldg. 2	0.5	0.45	5.0	0.38	5.0	0.38	3.7

4.6 Conclusions

This paper has reported a series of examples of results that show how ambient vibration tests can be used in low and medium-rise irregular buildings to identify coupled sway and torsional mode shapes, flexible behavior of roofs and foundation flexibility effects. The results include the extracted modal properties of at least the lowest three frequency mode shapes (periods, mode shapes and modal damping estimates). The rate of success of AVTs in this study to capture at least the three lowest natural frequencies/modes are unlike previous studies where the inherent difficulty of performing modal analysis based on ambient vibration records for low-rise buildings were reported. Moreover, in case of concrete moment frames results show fundamental periods derived from AVT are much shorter as compare to the building code periods, however, for braced steel frames the AVT results are in a very good agreement with the NBCC 2010 code formula.

The proposed procedure to select proper AVT sensor layouts, increase the number of sensors, use two reference sensors, increase sampling frequency and recorded time duration in addition to considerations of operational, repeated modes and proper decimation of records provide useful guidance for researchers or engineers interested in building testing. The ability of AVT to identify the modal characteristics of low-rise irregular buildings, makes this test a promising tool for a wide range of structural engineering applications including in situ direct seismic vulnerability assessment of buildings and post-event damage assessment.

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Link between chapter 4 and chapter 5

Chapter 4 presented a detailed discussion of the results of ambient vibration tests on the 16 low and mid-rise irregular buildings of the data base. These tests proved the capability of AVT to derive at least the lowest three modes of vibration of low-rise buildings; modes can more easily be derived with AVT in high-rise buildings with proper sensor location as the ambient signal is less noisy and of higher amplitude than in lower buildings. Practical considerations on how to achieve successful ambient vibration tests and several propositions such as increasing the number of sensors, sampling frequency and recording duration were addressed.

The 16 buildings used for AVT in this research were designated as emergency shelters by *Centre de sécurité civile* of the City of Montreal, Canada. Therefore, knowing how these buildings would behave in the event of a future earthquake is essential. However, considering that good quality detailed engineering plans are usually not available for older buildings and that creating accurate detailed finite element models is time consuming and complex, the author has developed a new methodology and software, referred to as 3D-SAM, which can perform seismic assessment directly based on ambient vibration tests. The exceptional features of the 3D-SAM methodology are that it does not require detailed engineering plans and creation of a finite element model, as an equivalent model of the building is created directly from the dynamic characteristics extracted from AVT. Chapter 5 describes this methodology in details and compare its results with those obtained with four detailed finite element models.

5. A new three-dimensional seismic assessment method (3D-SAM) for buildings based on experimental modal analysis

5.1 Abstract

Most of current detailed seismic evaluation methods for buildings are based on numerical approaches. However, there is a need to use state-of-the-art interdisciplinary technologies and techniques to further facilitate such evaluations and improve their reliability, especially in many situations where detailed design documentation is not available. This paper introduces a novel approach for seismic assessment of buildings, 3D-SAM, based on in-field ambient vibration measurements using acceleration/velocity sensors located on building platforms (floors and roofs). The proposed method is practical: it can be used to assess the dynamic characteristics on almost any building type as it incorporates torsional effects in predicting response unlike existing simplified theoretical methods currently in use. As such, the method can deal with any structural irregularity (in vertical and or horizontal planes) that has a significant effect on its seismic response. It does not require the creation of any artificial numerical model and can easily be integrated into existing modal identification software. The details and application procedure of this new method are explained here. Applications to four buildings located in Montreal are presented that illustrate and validate the proposed method; results are compared with those obtained using detailed and updated linear dynamic analysis of finite element models of the buildings. The results confirm 3D-SAM as a reliable seismic assessment tool for most buildings in moderate seismic regions: its application to severe seismic regions is limited to buildings that must remain functional after an earthquake and would sustain only moderate damage.

5.2 Introduction

Evaluation of the seismic performance of buildings needs to be conducted as an essential first step of the risk mitigation process. The issue of seismic evaluation of existing buildings has become increasingly important in recent decades, especially in the context of performance-based design. The purpose of this evaluation is to assess/predict the building's behaviour during an earthquake, identify whether the structure is in need of preventive retrofit, and provide a reference condition to recognize damage in the building after the occurrence of a design-level earthquake.

Current seismic evaluation methods for buildings are based on linear and nonlinear static and/or dynamic analysis approaches (ASCE 41, FEMA 356 and NIST 2010). However, there is both uncertainty and variability in the predicted results obtained from the numerical models that are developed using different approaches. According to a survey conducted in phase I of the ATC-55 project about the application of these assessment methods in structural engineering firms in the United States (FEAM 440), several respondents commented on issues about these analytical methods: their inaccuracy/variability, e.g. different analysis methods lead to significantly different results, the general complexity of these so-called simplified procedures, the sensitivity of the inelastic analysis approaches to assumptions regarding such parameters as initial stiffness, and the invariance of the loading patterns used in nonlinear static analysis procedures. Therefore, there is still need for developing alternative simplified seismic evaluation methods, with recognized limitations and range of applicability. This need is particular in moderate seismic regions due to the lack of recent earthquakes and the scarcity of information, as the existing assessment methods are based on damage observations in high seismicity areas. The proposed solution is to use low cost in-situ experimental modal tests, owing to advances in sensing techniques and analysing procedures, to derive the essential structural characteristic of the buildings and then use this information for seismic response assessment and economical losses for slight to moderate damage levels.

This paper extends the work and scope of Mirshafiei et al. 2014 to introduce a novel three-dimensional experimental seismic assessment method (3D-SAM) based on ambient vibration measurements for low to moderate seismic regions. The proposed method is illustrated in details and verified for four buildings located in Montreal, Canada. Its limitations and applications are also discussed.

5.3 Background

5.3.1 Ambient vibration tests

The proposed method relies on ambient vibration testing of the building. A detailed explanation of the experimental modal analysis method and its application towards this research is presented in Mirshafiei and McClure 2012. The following main points are summarized here:

- Ambient vibration testing (AVT) is a reliable low cost tool to derive modal characteristics of buildings, i.e. mode shapes, natural frequencies and damping ratio estimates, using frequency domain decomposition of recorded motions (Mirshafiei et al., Brincker et al. 2001, Hans et al. 2005, Trifunac 1972, Gilles 2011); results would be very close to those obtained from more elaborate forced vibration testing and would therefore be appropriate to represent buildings subjected to weak ground motions.
- AVT can be used to identify coupled sway and torsional modes that typically exist in low and mid-rise irregular buildings (Mirshafiei and McClure 2012).
- AVT is used worldwide for updating finite element models (Ventura et al. 2001, Yu et al. 2007a, Yu et al. 2007b, Tremblay et al. 2008, Lamarche et al. 2009), structural

identification and predicting seismic behavior of buildings for moderate seismic regions (Gilles 2011, Gentile and Gallino 2008, Michel et al. 2008).

5.3.2 Seismic vulnerability assessment of existing buildings

Two popular methods are generally used for seismic assessment of buildings: rapid visual screening (RVS) (FEMA 154, NRC 1992) and linear or nonlinear detailed structural analysis when RVS indicates a significant seismic vulnerability (ASCE 41, FEMA 356 and NIST 2010, FEMA 440). Linear elastic analysis and/or nonlinear analysis can be selected to perform detailed evaluations. The nonlinear analysis has the potential to provide a better understanding of the performance of buildings at significant damage levels if the simulation models are calibrated with actual building performance characteristics. However, due to the frequent lack of good quality drawings (especially for older buildings designed in the pre-code decades) and unrecognized true behavior of connections and components/elements, the creation of an accurate finite element model remains a major difficulty for assessing existing buildings. Even though the effects of modelling uncertainties can be quantified with rigorous probabilistic analysis, the large variability of the predicted results remains an issue in practical applications. For buildings subjected to small/moderate earthquakes structural interactions between non-structural (architectural) elements and the structural lateral load resisting system are significant and these effects are not captured in traditional assessment methods developed for buildings in high seismic zones. As a result, reliable seismic vulnerability assessment is much harder to achieve by a finite element analysis model; fortunately analysis of AVT records can provide reasonable estimates of the actual building properties for these low to moderate seismic motions.

In recent years, few researchers have tried simple methods to use ambient vibration data for seismic assessment of buildings. Essentially, these methods have been based on 2D lumped-mass stick models strictly applicable to regular and symmetric structures amenable to linear dynamic modal superposition analysis (Michel et al. 2008, Boutin et al. 2005, Hans et al. 2008). In a later study, Michel and Guéguen 2009 and Michel et al. 2012 have used the same approaches to calculate building seismic fragility curves for the “slight damage” grade from modal parameters extracted from ambient vibration records for 60 buildings in the city of Grenoble (France). The damage level was defined in terms of story-drift ratio (NIBS 2003) corresponding to different grades of damage and for different lateral force resisting systems (LFRS). However, these methods have several limitations and shortcomings: identical mass assumption at each storey level and neglecting any building torsional behavior and coupling effects of lateral-torsional mode shapes resulting from structural irregularities due to the two-dimensional nature of these methods. These shortcomings have prevented the existence of a comprehensive seismic assessment tool and methodology based on AVT till today. Therefore, there is a need for introducing a comprehensive three-dimensional method which will address these shortcomings and can be used as a practical tool for engineers to assess the seismic performance of building structures.

5.4 Seismic assessment of buildings based on the 3D-SAM

This section discusses the proposed three-dimensional simplified assessment method (3D-SAM) for the seismic evaluation of existing buildings based on extracted modal parameters from ambient vibration records. The lower natural frequencies, which are typically excited by earthquakes (0.3 to 10 Hz), and their corresponding mode shapes and equivalent internal viscous damping ratios are derived from experimental modal analysis. These experimental modal properties are combined with building data collected from on-site inspection, possible available architectural and structural plans to provide input to the 3D-SAM method. Each building model can then be subjected to an ensemble of representative ground motion records and its global seismic demand parameters are computed based on classical linear modal superposition analysis theory.

Having extracted the dynamic building characteristics from AVT records, it is possible to calculate the building seismic response by time domain convolution (Duhamel integral) in the linear range. Unlike previous studies of seismic building assessment based on AVT, the equation of motion of the building model is considered in three dimensions, i.e. 3N degrees of freedom (N is the number of stories) are considered for each rigid floor diaphragm including two horizontal displacements and one in-plane rotational degree of freedom, as shown on Figure 5.1. In this way, coupling effects in sway modes and torsional modes are taken into account, which is known to be considerably important for low-rise buildings of complex geometry (for example schools and community centers with swimming pools and gymnasia) that usually do not possess symmetric plans; even building shapes that look symmetric in geometry have eccentricities between their center of mass and center of rigidity at different floor levels.

By application of the advanced dynamic analysis (Chopra 2007) the equation of motion of a simplified multi-degree-of-freedom building model (see Figure 5.1) subjected to a horizontal seismic force applied with the angle of β with respect to the x axis are written as follows:

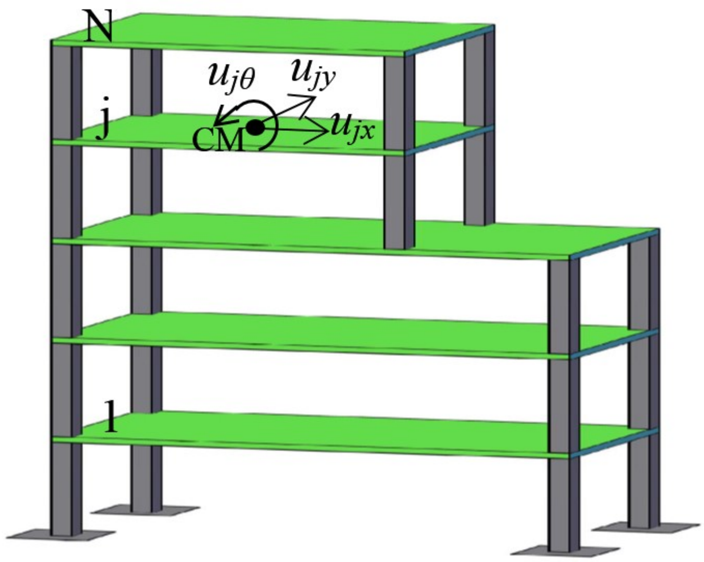

Figure 5.1 Schematic view of an irregular building with 3N degrees of freedom

$$[\boldsymbol{M}]\{\ddot{\boldsymbol{u}}\} + [\boldsymbol{C}]\{\dot{\boldsymbol{u}}\} + [\boldsymbol{K}]\{\boldsymbol{u}\} = -[\boldsymbol{M}]\boldsymbol{\iota}\ddot{u}_{gx}(t) \tag{5.1}$$

Where

$$[\boldsymbol{M}] = \begin{bmatrix}
m_1 & 0 & & & & \cdots & \cdots & & & & 0 \\
0 & m_2 & & & & & & & & & \\
 & & \ddots & & & & & & & & \\
 & & & m_N & & & & & \ddots & & \\
 & & & & m_1 & & & & & \ddots & \\
\vdots & & & & & m_2 & & & & & \vdots \\
\vdots & & & & & & \ddots & & & & \vdots \\
 & & \ddots & & & & & m_N & & & \\
 & & & \ddots & & & & & I_{o1} & & \\
 & & & & & & & & & I_{o2} & \\
 & & & & & & & & & \ddots & 0 \\
0 & & & & & \cdots & \cdots & & & 0 & I_{oN}
\end{bmatrix}$$

$$\{\boldsymbol{u}\} = \begin{Bmatrix} u_{1x} \\ u_{2x} \\ \vdots \\ u_{Nx} \\ u_{1y} \\ u_{2y} \\ \vdots \\ u_{Ny} \\ u_{1\theta} \\ u_{2\theta} \\ \vdots \\ u_{N\theta} \end{Bmatrix} \; and \; \{\boldsymbol{\iota}\} = \begin{Bmatrix} cos\beta \\ cos\beta \\ \vdots \\ cos\beta \\ sin\beta \\ sin\beta \\ \vdots \\ sin\beta \\ 0 \\ 0 \\ \vdots \\ 0 \end{Bmatrix}$$

$\boldsymbol{M}$ is a diagonal matrix with the diagonal values of the lumped mass and the moment of inertia of the floors about the vertical axis through the center of mass of each floor and $\boldsymbol{\iota}$ is the influence vector of 3N rows.

To solve equation of motion without knowing the stiffness matrix K we use the convolution integral (*CONV*). Moreover, the relative horizontal displacement vector $\{\boldsymbol{u}\}$ of all the floors/roof of the building model forced into vibration by ground motion can be written in modal coordinates $\{\boldsymbol{q}\}$ that uncouple the equations of motion by use of the expansion theorem:

$$\{\boldsymbol{u}\,(\boldsymbol{t})\} = [\emptyset]\{\boldsymbol{q}\,(\boldsymbol{t})\} \tag{5.2}$$

$$\boldsymbol{q}\,(\boldsymbol{t}) = \{q_1 \;\; q_2 \;\; \dots \;\; q_n\}^T \tag{5.3}$$

$$\forall\, i \in [1, n] \qquad q_i(t) = \frac{-P_i}{\omega_i^{'}} * CONV(\ddot{u}_g, e^{(-\zeta_i * \omega_i * (t))} * \sin(\omega_i^{'} * (t))) \tag{5.4}$$

$$\text{Where} \qquad P_i = (\{\phi_i\}^T[\boldsymbol{M}]\boldsymbol{\iota})/(\{\phi_i\}^T[\boldsymbol{M}]\{\phi_i\})$$

Equation (5.4) gives the response in the generalized (modal) coordinates using convolution integral with $\omega_i^{'} = \omega_i\sqrt{1-\zeta_i^2}$ the damped angular natural frequency, "P_i" is the participation factor of mode i, “n” is number of experimental modes derived from AVT (N=number of stories) and $\ddot{u}_g$ is the input earthquake record. It should be mentioned that ϕ, ζ and ω are the three-dimensional mode shape matrix of 3N×n, their corresponding natural frequencies and equivalent viscous damping ratios. However, as these parameters are extracted from the *in-situ* AVM tests further process needs to be performed on these complex modal properties before putting them as input for the algorithm; these additional steps are explained in the following paragraph.

3D-SAM uses the aforementioned procedures to generate the seismic assessment outputs from AVT records by using a Matlab routine to perform the classical dynamic analysis of the building model subjected to selected earthquake records. The structural input parameters are actual characteristics of the building: Floor/roof mass, modal matrix, natural frequencies, damping ratios, floor heights and dimensions, and position of corner joints (or any other joint on each floor). The mass matrix is estimated from available architectural and structural plans and other relevant information about the building. The modal matrix is estimated from frequency domain decomposition analysis of AVT records (in this study ARTeMIS™ software (Solution SV 2010) is used to extract modal properties). Since experimental damping is always non proportional, the mode shapes derived from AVT are complex modes. However, for ordinary buildings, i.e. where structures do not consist of two or more parts with significantly different levels of damping, special energy-dissipating devices or a base isolation, the assumption of classical damping is appropriate and the degree of complexity in the modes are small. Therefore it is possible to estimate normal modes from the real part of experimental complex modes; this assumption can be checked for each mode by looking at complex numbers corresponding to nodes of that particular mode and checking the difference between phase angles of different nodes to remain almost equal to either zero or 180^0. Moreover, the input experimental modal matrix for 3D-SAM should have the mode shape coordinates at the center of mass (C.M) on each floor. Since the AVT measured nodes of the mode shapes are not located on the floor C.M, there is a need to determine the mode shape values at C.M from those of the measured nodes: This is done by calculating the position of C.M and assuming in-plane rigidity of the floors.

With the application of all the previous steps, the 3D-SAM method provides relative displacement vectors at center of mass on each floor. By assuming rigid in-plane movement of each floor, relative displacement vectors can be obtained at any floor location including building corners. Absolute accelerations are simply estimated by taking the second time derivative of relative displacement vectors and adding the ground acceleration. After multiplying the horizontal components of absolute acceleration of C.M by its mass, the inertia force at each floor is computed which leads to shear and overturning moment calculations. Having absolute accelerations of each floor, it becomes easy to determine individual floor acceleration response spectra, which can then serve to assess the seismic vulnerability of the non-structural components of the building that are acceleration-sensitive.

The whole process of the simplified 3D-SAM method (which is based only on in-situ data without making a calibrated finite element model) and its software implementation can provide several response indicators of buildings within reasonable amount of time. In the following section, four examples of buildings located in Montreal, Canada are used for verification of the proposed method: the displacements and accelerations of C.M and corner joints of the AVT tested buildings are calculated by both 3D-SAM and updated linear finite element model (using Sap2000 (Computer and Structures 2009)) and the comparison between the results is discussed. The other global seismic drift/force demands can easily be derived from the displacements and accelerations.

5.5 Validation and application of the 3D-SAM method

5.5.1 Validation of the algorithm using a virtual building

As a first step to validate the integrity/correctness of the 3D-SAM Matlab routine, a 3-story virtual irregular building was assessed by 3D-SAM and results were compared with a detailed linear dynamic analysis model using Sap2000. The building is asymmetrical and its LFRS comprises a reinforced concrete moment frame and six concrete shear walls; Plan view and a three-dimensional perspective of the building are shown in Figure 5.2. Three dynamic degrees of freedom are defined at the center of mass on each floor and a lumped mass/inertia is assigned to each DOF.

The model is subjected to a synthetic horizontal ground motion along its asymmetric direction Y. This ground motion has been adopted from a study done by Assi (2006). This record corresponds to a Magnitude 6 event, epicentre distance of 30 km, duration of 8.89 s, return period of 2500 years and scaled appropriately to be compatible with the NBC Uniform Hazard Spectra (UHS) for Montreal according to NBCC 2005 (the hazard has been slightly reduced for Montreal in the 2010 edition of NBC). The scaled response spectrum of this ground motion compared with UHS is shown in Figure 5.3.

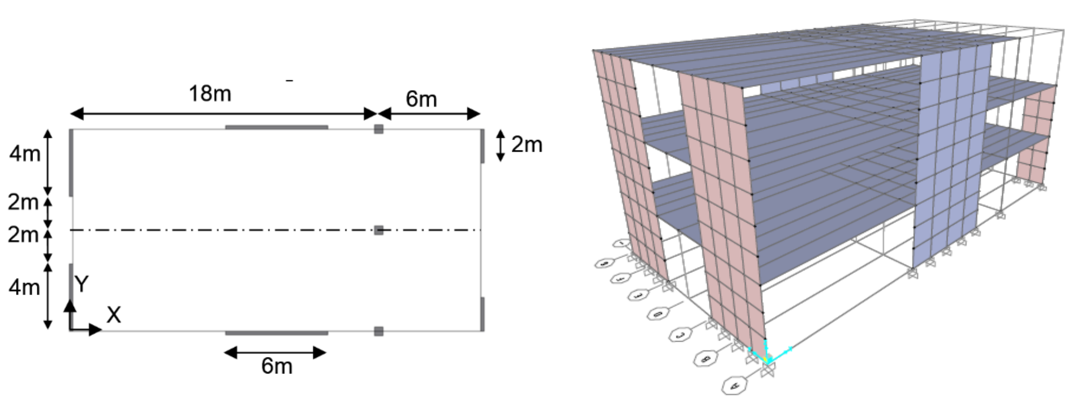

Figure 5.2 a) Plan view; b) 3D view

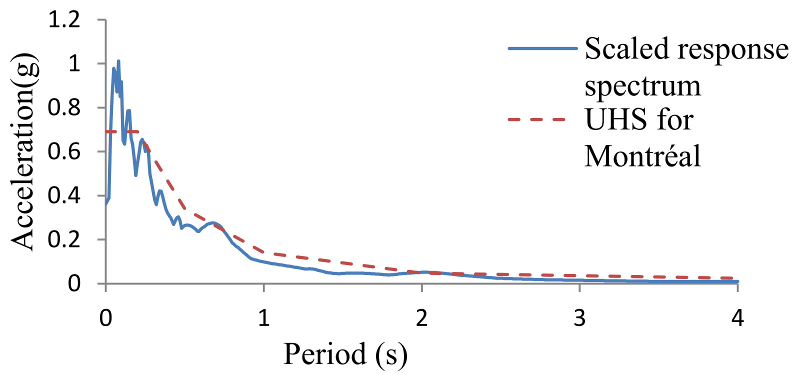

Figure 5.3 Comparison between scaled response spectrum of input ground motion and UHS for Montreal

All modal properties were derived from modal analysis in FE model (instead of AVT) and were also chosen as the inputs for the 3D-SAM. Of course in the other field investigated cases in this paper the input modal data are extracted from the ambient vibration test records. Moreover, as in most low-rise buildings, only the first few lower frequency modes are derived from proper AVT (at least first three lower frequencies (Mirshafiei et al. 2014)) truncated modal analysis was done considering only the first mode, then the first two modes, and finally the first three modes (the first two modes combined sway and torsional modes and the third was dominantly torsional). The complete modal superposition was also performed as reference. In summary, the following conclusions were made from this simple example:

1) Accuracy of the Matlab routine was verified; as expected seismic demands from 3D-SAM based on modal properties derived from FE model were equal to the results coming directly from FE linear analysis.
2) Relative displacements are dominated by the fundamental mode (sway coupled with torsion) and are predicted relatively well at the center of mass and corner joints along or perpendicular to the direction of earthquake based on the first three modes including torsional mode shape.

3) Absolute accelerations in the earthquake load direction can be estimated based on the first few modes.

Therefore, having the first few lowest frequency modes from AVT for low rise buildings is sufficient to capture their linear seismic response (for mid and high-rise buildings it is easier to capture higher frequency modes by AVT). Conclusions of this example and validation of the 3D-SAM algorithm were also checked for the following buildings.

5.5.2 Application to existing buildings

3D-SAM method is now illustrated on tested buildings located in Montreal. As mentioned previously, researchers have used finite element model updating based on AVT results in building applications since the last decade. The goal of such a process is to calibrate the finite element model (FE model) in a way that its natural frequencies and mode shapes be as close as possible to that extracted from AVT records; of course, getting an exact match for all modes and frequencies is not feasible. In short, the reference condition that researchers have been trying to achieve from their numerical approach has been the actual/experimental modal properties. This is the main advantage of the proposed 3D-SAM approach, which is based on this reference condition and uses directly the modal properties obtained from experimental results, thus avoiding the need for a FE model. In the following three case studies, the seismic response of the buildings is computed from the both approaches, 3D-SAM and the calibrated linear dynamic FE model in Sap2000, to illustrate the differences between the two procedures. It should be emphasized that the two methods should yield the same results in the ideal case where the calibrated FE model would match exactly the experimental modal data, which is generally not feasible.

In the analysis phase, the three tested buildings have been subjected to an ensemble of synthetic earthquakes compatible with Montreal's moderate seismicity, covering the appropriate frequency range of interest, peak ground acceleration, magnitude, epicentral distance and duration; the generated records were also scaled up or down to match the UHS of Montreal as closely as possible in different range of periods. The conclusions obtained from the results generated by the various earthquake inputs remain the same so only one set of results corresponding to the same earthquake that was shown in Figure 5.3 will be discussed next for each building example.

Centre Communautaire de Loisirs de la Côte-des-Neiges

This low-rise irregular building was constructed in 1993 with a steel braced frame structural system; its height is about 20 m including one basement floor. A bird's view of the building and its floor plans are shown in Figure 5.4. Figure 5.4c, d and e are the outlines of the mechanical room, bottom roof and upper roof (roof of mechanical room), respectively. The AVT sensor positions are also depicted in this figure by black dots (the reference sensors are denoted by the letter R on the sketches). In the study, six vibration sensors (tromographs – see www.tromino.eu) are used to record ambient horizontal and vertical accelerations and velocities of building roof/floors. This sensing system is completely wireless and the data are recorded directly in the individual sensors for eventual download to a computer using a standard USB connection. Each sensor records velocities and accelerations in three orthogonal directions (6 channels): two in the horizontal plane and one in the vertical direction; note that only the horizontal components of the measured records are used in this research. Moreover, as this building is a low rise, it could have a flexible roof therefore more sensors are deployed on the roof to capture this possible effect. Two sensors (to have a reference backup and capture higher quality mode shapes) are designated as references and are deployed on the top floors and far from the center of rigidity to record any torsional effects. Five set-ups each for a ten-minute data record were taken at a sampling frequency of 128 Hz. ARTeMIS™ software (Solution, S.V. 2010) Handy Extractor version is used to process the recorded data. Two methods were used to extract the lower frequency modal parameters of the buildings: Frequency Domain Decomposition (FDD) and Enhanced Frequency Domain Decomposition (EFDD). Mode shapes, natural periods and damping ratios are shown in Figure 5.5. The undeformed building shape is shown in blue and the deformed mode shape is in green color; in some parts the green and blue colors coexist and the green boundary lines should be used to identify the deformed mode shapes. According to the National Building Code of Canada formula (NBCC 2010), the fundamental period of this steel braced building to be considered for other simplified seismic analysis methods is 0.28 seconds (considering height up to the bottom roof), which shows consistency with AVT result (0.24s). It should also be mentioned that three sensors were also put on the ground floor but no movements were seen in the mode shapes on the ground level. Furthermore, from the experimental mode shapes it is seen that the assumption of rigid floors and roofs is applicable for this building.

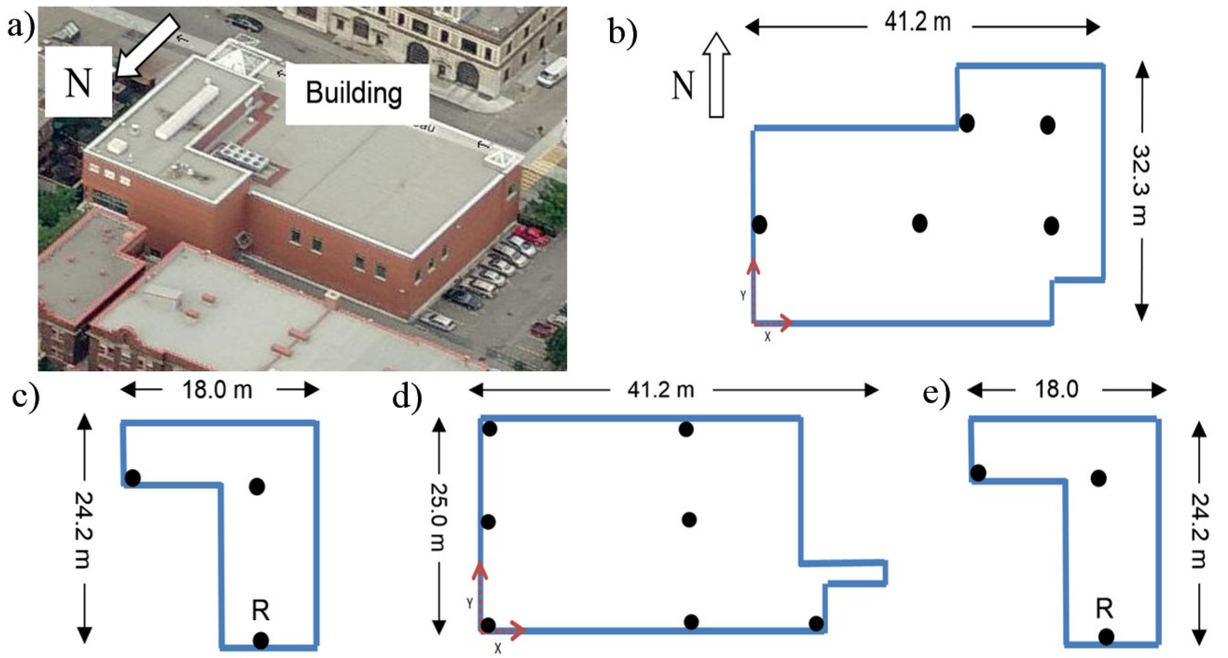

Figure 5.4 a) Bird's eye view of the tested building; b) 1st floor-height above ground 5.4 m; c) 2nd floor-height above ground 10 m (mechanical room) ; d) 3rd floor-height above ground 11.36 m (bottom roof) e) 4th floor-height above ground 14.5 m (upper roof)

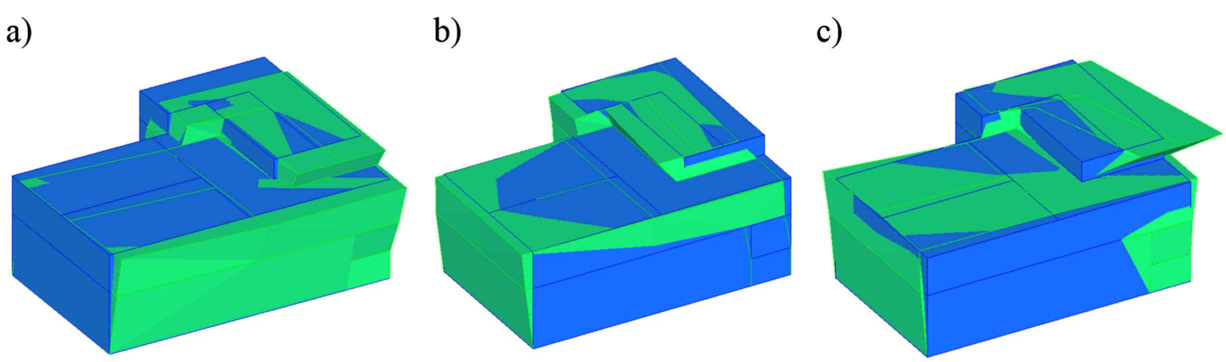

Figure 5.5 Mode shapes a) 1st flexural-torsional mode (0.24 s, damping ratio=0.016); b) 2nd flexural-torsional mode (0.24 s, damping ratio=0.012); c) 1st torsional mode (0.18 s, damping ratio=0.023)

In the next step, this steel braced frame building is modelled in details (walls and ceiling's supporting truss system are included) in SAP2000 and calibrated to AVT modal properties. The calibrated model is then subjected to the synthetic horizontal ground motion along its Y direction (short) and its response is calculated. A 3D view of the Sap model is shown in Figure 5.6. In addition, a comparison between the natural frequencies and mode shapes (by modal assurance criteria) of AVT and FE results is presented in Table 5-1a and Table 5-1b, respectively, and shows the FE model is reasonably well calibrated with the AVT modal properties.

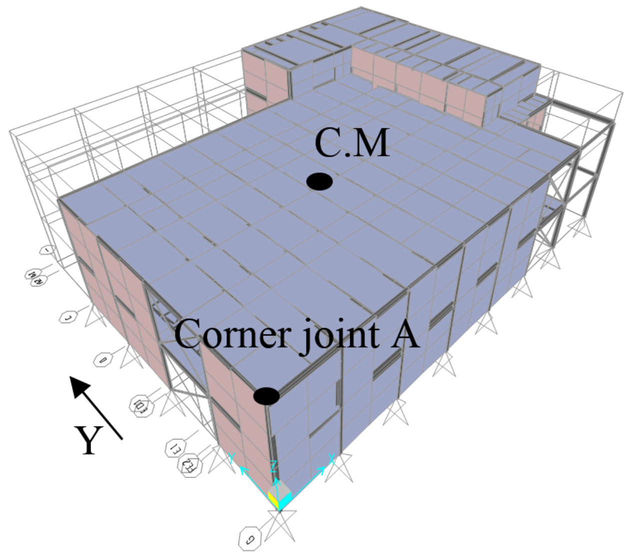

Figure 5.6 3D view of the FE model

Table 5-1 a) Comparison of natural frequencies between AVT and FE model; b) Modal assurance criteria (MAC) values between FE model and AVT mode shapes

a)

Comparison	Natural frequencies (Hz)		
	1st mode	2nd mode	3rd mode
FE model	4.1	5.0	7.0
AVT	4.1	4.2	5.7
Difference (%)	0	19	23

b)

MAC values	FE model			
		Mode1	Mode2	Mode3
AVT	Mode1	**0.97**	0.00	0.04
	Mode2	0.02	**0.92**	0.00
	Mode3	0.01	0.01	**0.80**

Afterwards, the seismic response of the structure is calculated directly from AVT as per 3D-SAM. The displacement and acceleration time histories of C.M and a corner joint A (location shown in Figure 5.6) are compared for three cases and four representative graphs are shown in

Figure 5.7: Sap2000, 3D-SAM and “frequencies AVT-modes Sap2000”. The latter case, for discussion purposes, is a hybrid one in which the mode shapes are determined from FE model and the natural frequencies are extracted from AVT records. It should be mentioned that displacement and acceleration response history analyses of C.M and the corner joint aligned and perpendicular to the direction of the ground motion were derived from FE model by considering a truncated modal superposition of the first three modes and then a complete superposition of all modes, which resulted in practically the same results. This confirmed that using the first three modes from AVT is enough for this low rise building.

The relative displacement and absolute acceleration of C.M, and the relative displacement of the corner joint in direction of the applied earthquake are almost the same from the three aforementioned methods. For the building response in the direction perpendicular to the applied earthquake (and also the absolute acceleration in direction of the applied earthquake) the calibrated FE model is overestimating the response, while 3D-SAM and “frequencies AVT-modes Sap2000” yield almost the same results. This shows that for this example the accuracy of the predicted response is more influenced by the accuracy of the natural frequencies than that of the mode shapes. Good results are obtained if mode shape estimates are reasonable (when diagonal values of the MAC matrix between numerical model and AVT are close to unity – see Table 5-1b). In fact, if a calibrated FE model is updated to have the same natural frequencies and mode shapes as the ones extracted from AVT records (the reference condition which is assumed to reflect the true structural character of a building), it will yield the same results as those coming directly from 3D-SAM. Recognizing that constructing a 100% calibrated FE is time consuming and not often feasible, the 3D-SAM can be used as a good alternative to calculate the desired global seismic demands. Finally, another comparison was made between displacement and acceleration response histories of corner joint and C.M from 3D-SAM method: this comparison shows the acceleration and displacement are almost the same for the corner joint and C.M in the direction of applied earthquake, however, as expected, the response values are much higher for corner joints in the direction perpendicular to the applied earthquake.

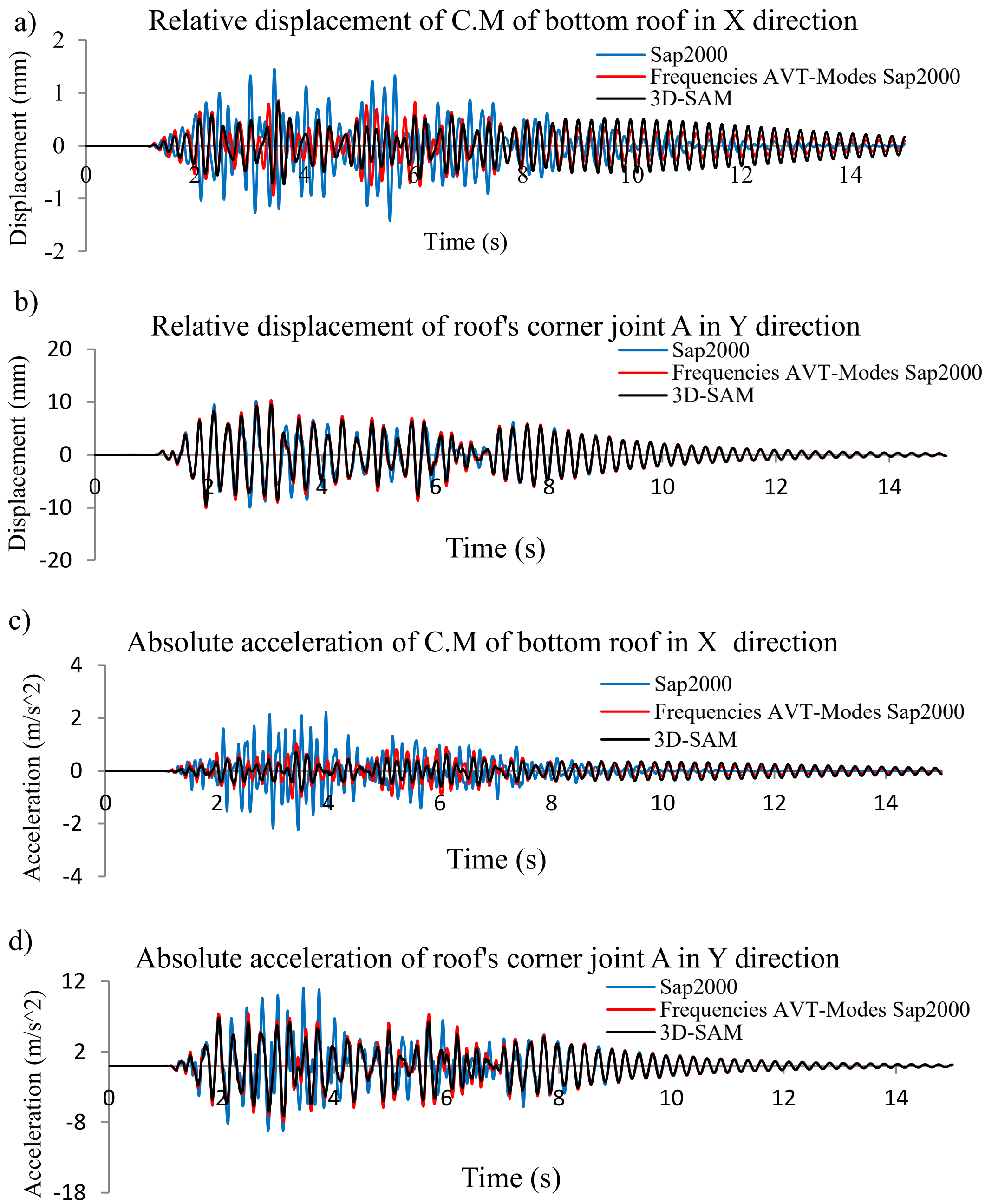

Figure 5.7 Comparison between response history analyses of C.M and corner joint of the bottom roof by three different methods - earthquake is applied in Y direction

CHU Sainte-Justine Hospital

This hospital building was constructed in 1957 with a reinforced concrete moment frame with infill walls; it is an 11 storey building with the height of about 39.6 m above the ground level. A bird's eye view, the typical plan of the building and its finite element model are shown in Figure 5.8. It is also seen in the plan view that three sensors are deployed on each floor and by using the same approach as discussed in the previous section the modal properties of this building are derived and depicted in Figure 5.9.

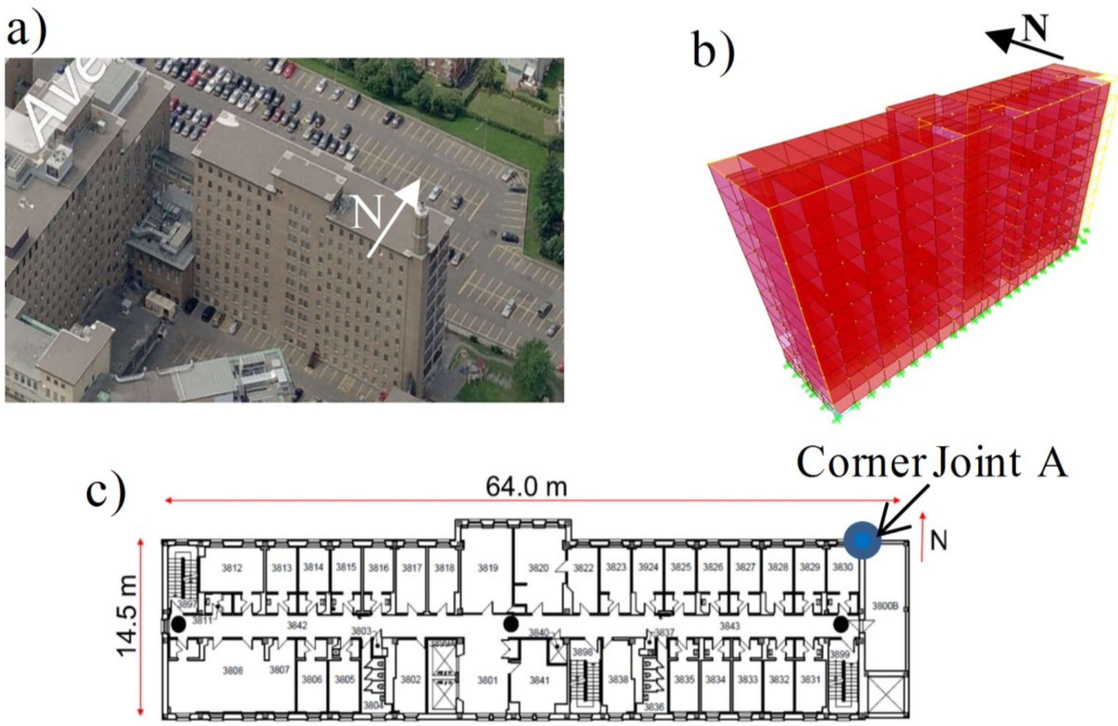

Figure 5.8 a) Bird's eye view; b) 3D view of the FE model (Asgarian 2012); c) a typical floor plan and sensor positions (black dots)

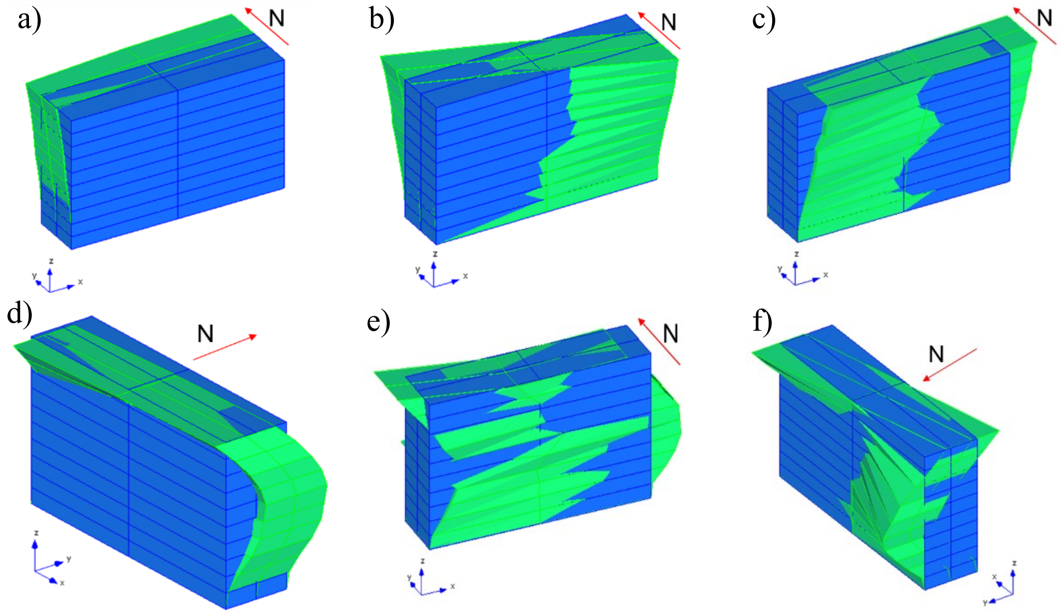

Figure 5.9 Mode shapes (Asgarian 2012) a) 1st flexural-torsional mode N-S dir. (0.53 s, damping ratio=0.017); b) 1st torsional mode (0.4 s, damping ratio=0.012); c) 1st flexural mode E-W dir. (0.37 s, damping ratio=0.011); d) 2nd flexural mode N-S dr. (0.19 s, damping ratio=0.008); e) 2nd flexural mode E-W dir. (0.14, damping ratio=0.009); f) 2nd torsional mode (0.13 s, damping ratio=0.017)

A detailed linear elastic finite element (FE) model of the building had been generated in FE model and calibrated using AVT results in a previous study by Asgarian 2012. This study had investigated the effect of infill walls on the seimic reponse and floor response spectra using the calibrated FE model. In terms of natural frequencies it is seen in Table 5-2 that the FE model is well calibrated with the test results, however, as indicated in Table 5-3 there is poor correlation between the torsional modes 2 and 5. This provides a good example that shows the 3D-SAM method would achieve better response prediction results than the detailed FE analysis. To compare the results of 3D-SAM and linear FE analysis, both models are subjected to the same earthquake record (Figure 5.3) compatible with the UHS of Montreal acting in the North-South short direction. The displacement and acceleration time histories of C.M and a corner joint A (location shown in Figure 5.8c) are compared for four cases and representative graphs are shown in Figure 5.10:

Sap2000, 3D-SAM, "frequencies AVT-modes Sap2000" and "frequencies Sap2000-modes AVT". For this building, as modes 2 and 5 of the Sap2000 model are not accurate, the predicted response histories perpendicular to the direction of the applied earthquake by the FE model are very small compared to the better predictions obtained by 3D-SAM. As seen in the graphs, the hybrid case where modes are from AVT and natural frequencies from FE leads to closer results to the 3D-SAM. Response histories in the direction of the applied ground motion are better predicted by FE and similar to that of 3D-SAM. It should be mentioned that displacement and acceleration response history analyses of C.M and the corner joint aligned and perpendicular to the direction of the ground motion were derived from FE by considering a truncated modal superposition of the six lowest frequency modes and then a complete modal superposition, the two procedures yielding practically the same results. This confirms that as a larger number of modes can be extracted form AVT records (in this case up to 6 modes) in high rise buildings, the 3D-SAM method will still provide accurate results.

Table 5-2 Comparison of natural frequencies between AVT and FE model

Comparison	Natural frequencies (Hz)					
	1st mode	2nd mode	3rd mode	4th mode	5th mode	6th mode
FE model	1.9	2.7	3.0	6.1	8.4	9.1
AVT	1.9	2.5	2.7	5.4	7.4	7.8
Difference (%)	0	8	11	13	13	17

Table 5-3 MAC values between FE model and AVT

MAC (AVT & FE model)	FE model						
		Mode1	Mode2	Mode3	Mode4	Mode5	Mode6
AVT	Mode1	**0.99**	0.13	0.00	0.01	0.00	0.00
	Mode2	0.68	**0.06**	0.27	0.02	0.00	0.01
	Mode3	0.00	0.71	**0.99**	0.00	0.00	0.00
	Mode4	0.09	0.05	0.15	**0.75**	0.00	0.07
	Mode5	0.62	0.04	0.00	0.48	**0.01**	0.00
	Mode6	0.02	0.04	0.07	0.02	0.79	**0.92**

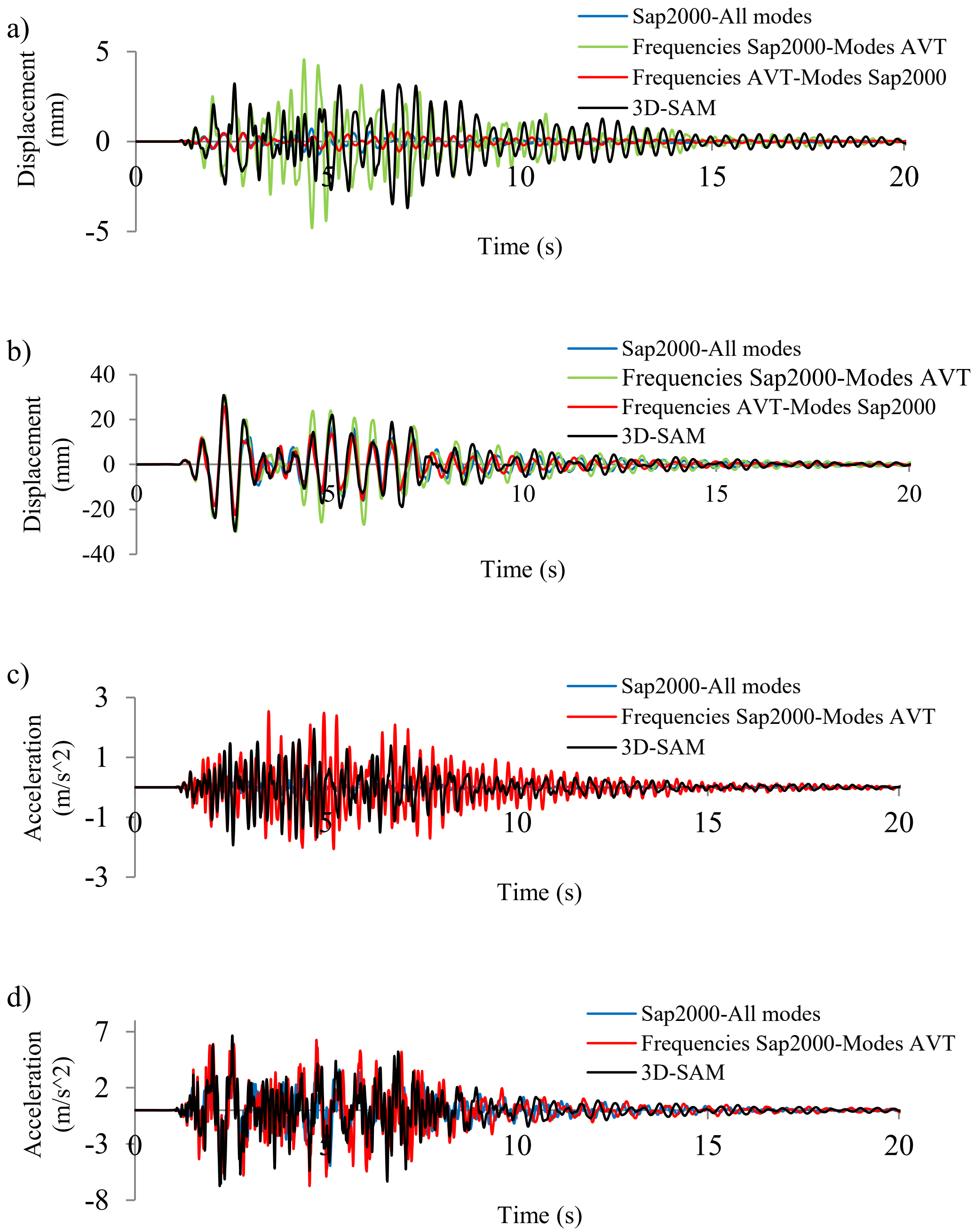

Figure 5.10 Comparison between response history analyses of the 9th floor's corner joint A by different methods-earthquake is applied in Y direction: a) Relative displacement in X direction; b) Relative displacement in Y direction; c) Absolute acceleration in X direction; d) Absolute acceleration in Y direction

Burnside building

This building, located on McGill University downtown campus, was constructed in 1969 with a reinforced concrete shear wall system; it is a 13 storey building with the height of about 47 m above the ground level; it has a basement beneath the ground level. A 3D view of the building, a typical plan and the sensor layout on the floors, all the sensor locations used in AVT and the FE model are shown in Figure 5.11. The modal properties of this building were derived via AVT (done by Gilles 2011) and are depicted in Figure 5.12. This earlier study had been done by using only two sensors therefore the modal coordinates were not available for all the floors, and the input modal matrix for 3D-SAM was calculated by interpolation and extrapolation of the available nodes. Also, the FE model is an equivalent model calibrated with AVT results and is made up of 4 equivalent columns representing shear walls around the building and two equivalent central columns in place of the interior columns and the elevator concrete shaft. Each floor is rigid in plane and the lumped floor masses are assigned to the floor centroids.

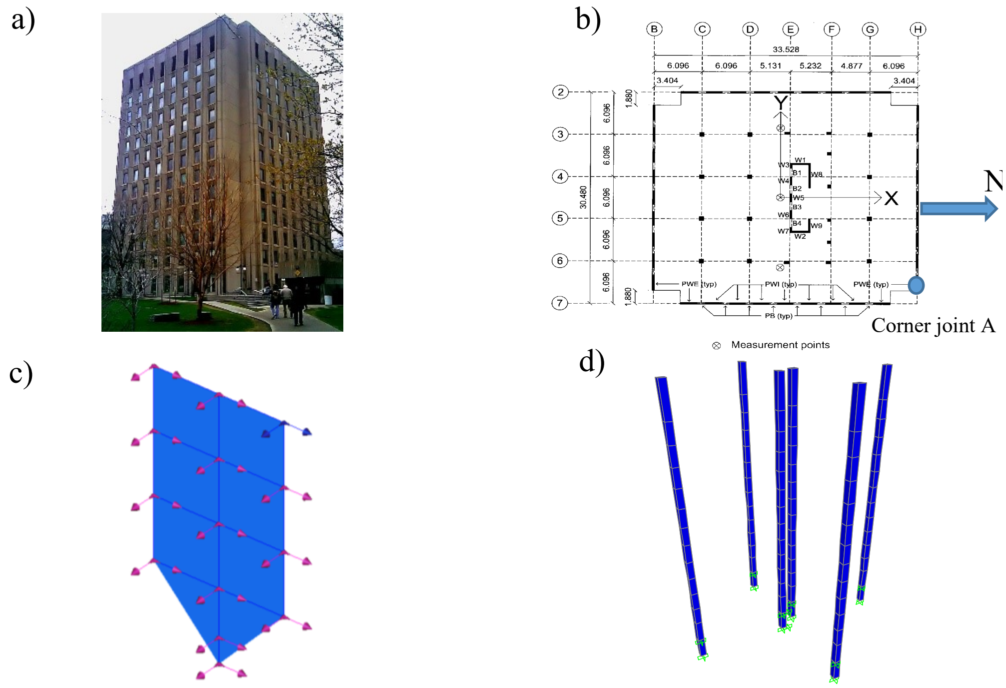

Figure 5.11 a) 3D view of the building; b) a typical floor plan and sensor positions; c) sensor layout; d) 3D view of the equivalent FE model

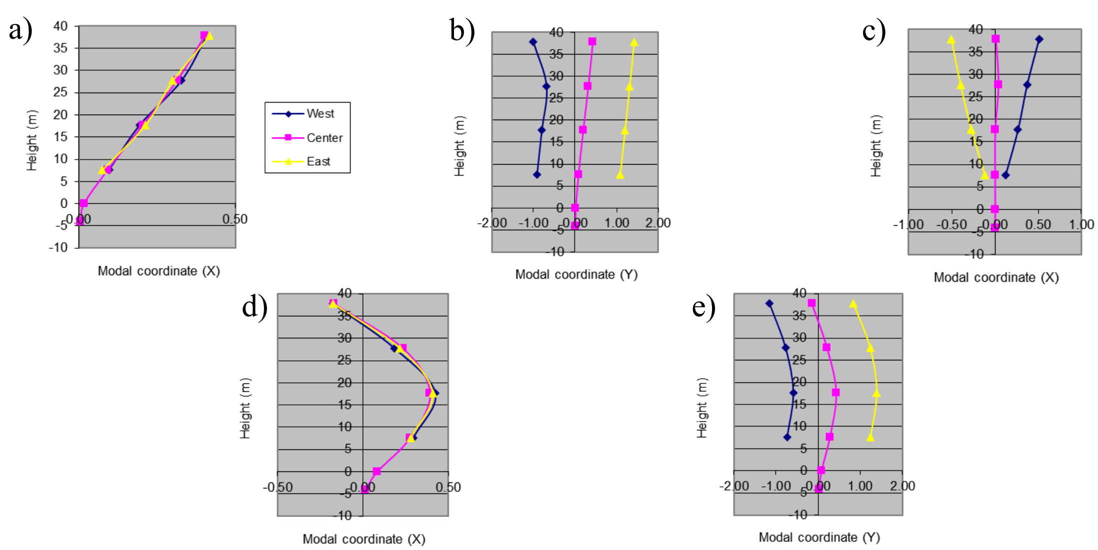

Figure 5.12 Mode properties by Gilles 2011; a) 1st flexural mode N-S dir. (0.7 s, damping ratio=0.018); b) 1st flexural mode E-W dir. (0.68 s, damping ratio=0.017); c) 1st torsional mode (0.41 s, damping ratio=0.02); d) 2nd flexural mode N-S dr. (0.22 s, damping ratio=0.023); e) 2nd flexural mode E-W dir. (0.21, damping ratio=0.023)

Table 5-4a compares the natural frequencies obtained with the calibrated FE model and AVT records; the first three modes of the FE model are well calibrated with the experimental results while modes 4 and 5 differ. In terms of modal assurance critera (Table 5-4b) a good correletaion exists between the FE and AVT modes. Again, the 3D-SAM and FE models are subjected to the earthquake record (Figure 5.3) compatible with UHS of Montréal in the North-South direction. The relative displacement and acceleration time histories of the roof corner joint A and C.M (location shown in Figure 5.11b) are compared for three cases and representative graphs are shown in Figure 5.13: Sap2000, 3D-SAM, "frequencies AVT-modes Sap2000". This building is quite regular therefore the responses perpendicular to the direction of the seismic input were almost zero using the three methods. Responses aligned with earthquake record are well predicted by both 3D-SAM and FE. In fact, the time histories graphs shown compare the displacement and acceleration response histories of imperfect calibrated FE models and hybrid cases (where updated FE models better represent the AVT results) with the 3D-SAM. These graphs clearly highlight the differences of these methods and show the simplified 3D-SAM as a good alternative for the linear updated FE models.

It should be mentioned that the accuracy of the 3D-SAM outputs depends on a good AVT obtained with a sufficient number of measuring nodes. Such good quality knowledge of the modal properties of the building is important to reduce the uncertainty of the model predictions of seismic response. It is acknowledged that sophisticated dynamic model predictions, based on numerical procedures, do not have a guaranteed accuracy in the absence of careful model calibration. Furthermore, model calibration is a crucial step that is simply not feasible at the building design stage and rather difficult (rarely done in fact) in routine engineering practice except for buildings of strategic importance.

Table 5-4 a) Comparison of natural frequencies between AVT and FE; b) MAC values between FE and AVT

a)

Comparison	Natural frequencies (Hz)				
	1st mode	2nd mode	3rd mode	4th mode	5th mode
FE model	1.4	1.5	2.4	6.3	6.6
AVT	1.4	1.5	2.5	4.5	4.7
Difference (%)	0	0	4	40	40

b)

MAC (AVT & FE)	FE model					
		Mode1	Mode2	Mode3	Mode4	Mode5
AVT	Mode1	**0.97**	0.00	0.00	0.00	0.00
	Mode2	0.00	**0.98**	0.03	0.00	0.01
	Mode3	0.00	0.00	**0.95**	0.00	0.00
	Mode4	0.02	0.00	0.00	**0.99**	0.00
	Mode5	0.00	0.02	0.00	0.00	**0.99**

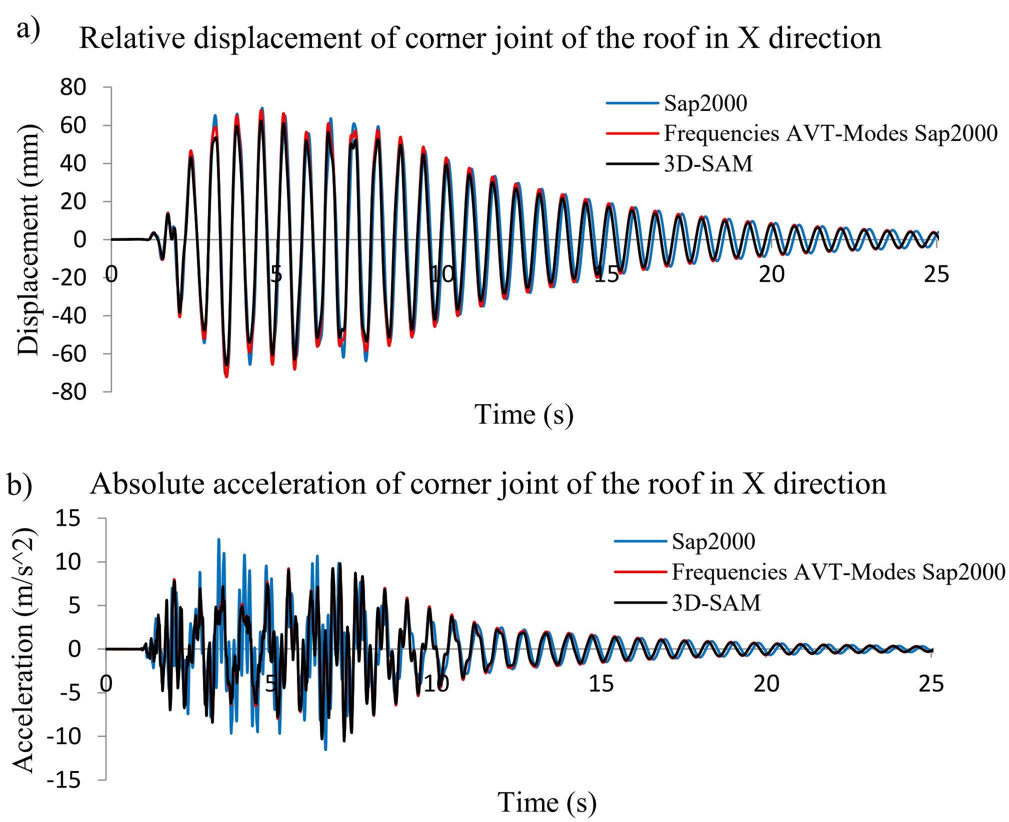

Figure 5.13 Comparison between response history analyses of the roof corner joint A by different methods-earthquake is applied in X direction

5.5.3 Discussion

The experimental basis of the 3D-SAM method brings a reduction in the uncertainty associated with the determination of structural building properties and eliminates the need to make detailed finite element models to predict global seismic demands for buildings located in low to moderate seismic regions. Moreover, consideration of three-dimensional (torsional) and higher mode effects, difficult to quantify in analytical methods, are also its main advantages. The global seismic demands of a building are calculated based on experimental ambient vibration tests without further numerical modelling; this is equivalent to using a linear finite element model (FEM) calibrated 100% with the derived modal properties from the AVT. The examples presented above have illustrated that the method enables the evaluation of different engineering seismic demand parameters such as floor displacements, story drift ratios, floor absolute accelerations at center of mass and any other point and direction on the floor, story shear forces, overturning moments and floor response spectrums.

As the procedure is based on linear dynamic analysis, the story shear forces and overturning moments are considered to be valid if the building behaves linearly during an earthquake, i.e. structural damage levels are kept low, and otherwise seismic forces are overestimated. For moderate structural damage, it is necessary to apply an appropriate nonlinear factor to relate this linear demand to its corresponding nonlinear value. The linear assumption should hold true in post-critical buildings, and in buildings located in regions with low and moderate seismic hazards. Displacements and drift ratio demands from linear analysis should provide better estimates of the maximum deflections and drift ratios based on equal displacement theory; the displacements calculated for a non-linear structure are suitably close to the displacements calculated for the same linear structure. Drift ratios can be compared with code/published limits for different building types and damage grades (slight and moderate) and can be used to produce fragility curves (Michel et al. 2012, Mitchell et al. 2010).

5.6 Conclusions

In this paper, a simplified three-dimensional seismic assessment method, 3D-SAM, is proposed for buildings with rigid floor/roof diaphragms. The method is based on in situ data collected from buildings, i.e. building inspection, engineering drawings and ambient vibration measurements. Three-dimensional mode shapes and natural frequencies of the building in the low frequency range can be extracted from the AVT records, therefore allowing consideration of flexural and/or torsional modes in any building configuration. The 3D-SAM method is encoded in a Matlab routine calculating time histories of relative displacement and absolute acceleration of any point on a floor/roof in a desired direction; having these response histories, other global seismic demands like drift ratios, storey shear forces and overturning moments can be calculated. It is emphasized that the method does not require creation of any theoretical FE model and therefore is ideal to be used to assess old buildings with low quality structural drawings or to perform city-scale seismic assessment of buildings in moderate seismic regions.

The 3D-SAM method was compared with updated FE results for four buildings located in Montreal and shown to be a more efficient and accurate tool for seismic response prediction when compared to the current practice of "updated linear finite element methods based on experimental modal analysis". The method calculates the demands directly from the AVT experimental results which is faster, more precise, and robust. In fact, this simplified method represents the equivalent of an ideal 100% calibrated finite element model for linear dynamic analysis. The approach can be used for any building type, low/high rise and regular/irregular. It is useful to calculate building fragility curves for slight and moderate damage grades and to determine displacement as well as acceleration response spectra for different floors, which helps evaluating the seismic response of non-structural components.

The only essential restriction of the method is that it is based on the assumption of linear dynamic response of the building. Its application is therefore restricted to the seismic assessment of buildings expected to sustain limited (only low to moderate) damage during earthquakes.

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Link between chapter 5 and chapter 6

Chapter 5 has compared the results of four case studies for which the 3D-SAM methodology and the more traditional finite element modelling approach were applied. It was shown that 3D-SAM calculates the global seismic demand parameters equivalent to the 100% calibrated finite element models based on modal properties derived from AVT.

The proposed new methodology is based on linear dynamic analysis of an equivalent building model constructed with the modal properties derived from AVT, thus representative of the building structure when subjected to a very weak ground motion. However, experience of previous earthquakes has shown that building modal properties vary with ground-motion levels, therefore, there is a need to suggest appropriate modification factors to the modal properties derived from AVT so that the applicability of 3D-SAM can be extended to the stronger base motions. Chapter 6 proposes such modification factors based on the data collected from permanently instrumented buildings which have been subjected to earthquakes. Moreover, all of the 16 low to mid-rise irregular buildings that were tested in the research at ambient vibration levels are analyzed with the modified 3D-SAM and the dynamic amplification portion of natural torsion at all building floors in the dataset are reported and discussed.

6. Modified three-dimensional seismic assessment method for buildings based on ambient vibration tests; extrapolation to higher shaking levels and measuring the dynamic amplification portion of natural torsion

6.1 Summary

This paper presents applications of the modified 3D-SAM approach, a three-dimensional seismic assessment methodology for buildings directly based on in-situ experimental modal tests to calculate global seismic demands and the dynamic amplification portion of natural torsion. Considering that the building modal properties change from weak to strong motion levels, appropriate modification factors are proposed to extend the application of the method to stronger earthquakes. The proposed approach is consistent with the performance-based seismic assessment approach which entails the prediction of seismic displacements and drift ratios that are related to the damage condition and therefore the functionality of the building. The modified 3D-SAM is especially practical for structures that are expected to experience slight to moderate damage levels and in particular for post-disaster buildings that are expected to remain functional after an earthquake. In the last section of this paper, 16 low to mid-rise irregular buildings located in Montreal, Canada and that have been tested under ambient vibrations, are analyzed with the method and the dynamic amplification portion of natural torsion of the dataset is reported and discussed. The proposed methodology is appropriate for large scale assessments of existing buildings, and is applicable to any seismic region of the world.

KEY WORDS: modified 3D-SAM; seismic assessment; ambient vibration tests; experimental modal analysis; irregular buildings; torsional effects

6.2 Introduction

Potential economical loss from damages to structures due to earthquakes can reach billions of dollars in densely populated areas. For instance, a total direct economic losses of $25.7 billion due to damaged buildings was paid by government and private insurance sources for recovery and reconstruction in California following the January 17, 1994 Northridge earthquake of moment magnitude of 6.7, (Table 5-1 of Comerio et al. 1996). Furthermore, the investment in non-structural components and building contents is far greater than the value of structural components and framing (Taghavi and Miranda 2003), and typically represents more than 80% of the total investment. To avoid such economic strain, proper seismic risk assessment of structural and non-structural components of buildings is necessary for both weak and strong motions, in regions with moderate to high seismicity. The usual practice in structural engineering firms is to use linear and nonlinear numerical building models for quantitative seismic assessments (ASCE 41, FEMA 356, and NIST 2010). With this approach, detailed structural plans and some in-situ tests to identify material properties are necessary to construct the numerical model. Also, there can be variability of the predicted results by different numerical models (FEMA 440). There might also be interaction between non-structural components and structural elements that typically is not appropriately accounted for in numerical models; such interaction is especially important for weaker ground motions of moderate seismic regions and post-disaster buildings like hospitals and schools. Consequently, some researchers and specialized earthquake engineering firms make use of in-situ experimental modal tests to calibrate numerical models and further enhance the reliability of seismic assessments. Currently, owing to advances in sensing techniques and analysing procedures, the most popular experimental modal test for large structures is ambient vibration testing (AVT). The reliability of AVT to derive modal properties such as natural frequencies, mode shapes and estimates of internal equivalent modal damping ratios has been demonstrated in several studies (Brincker et al. 2001, Trifunac 1972). However, despite the unquestionable added value sensing techniques and operational modal analysis can provide, the calibrated numerical modelling approach is still not very popular among structural engineering firms. This can be partly related to the fact that the model calibration process is somewhat complex: it still requires a very detailed finite element model (FE model) and at the end some discrepancies remain between the experimental and FE modal parameters. It is not feasible to calibrate a FE model to 100% of the test results.

To address the aforementioned problems, the authors have introduced and verified a new three-dimensional seismic assessment method and software (3D-SAM) (Mirshafiei et al. 2015, Mirshafiei and McClure 2015). A brief summary of the method and its verification with an example of low-rise irregular building are presented in the first part of this paper. To our best knowledge, 3D-SAM is the first three-dimensional seismic assessment methodology directly based on the observation of real modal properties of a structure obtained from ambient vibration tests. This approach can be used as a simplified alternative tool to the existing practice of linear calibrated numerical models based on in-situ derived modal properties. The method is especially useful to assess buildings in moderate seismic regions due to the lack of data on recent earthquakes and the scarcity of damage information; the existing assessment methods are based on damage observations in high seismicity areas. In fact, this novel methodology completely bypasses the need for detailed engineering plans and FE models, which is particularly appealing for seismic assessment of older buildings that lack proper technical documentation. The results obtained by 3D-SAM are equivalent to those that can be produced with a fully calibrated linear numerical model in terms of natural frequencies and mode shapes. As such, the method significantly reduces the analysis time (attractive for urban scale assessments) and is more reliable than the linear updated numerical models.

To address the fact that even an undamaged structure shows wandering of its natural frequencies with the amplitude of shaking (Celebi 2007), a modified 3D-SAM is introduced that provides modification factors for the modal properties to further extend the application of the method to stronger ground motions. The modified 3D-SAM is illustrated with a detailed case study of an irregular building. Finally the method is used for deriving the dynamic amplification portion of natural torsion on all floors of 16 low to mid-rise irregular buildings located in Montreal, Canada and tested by AVT. Most of the previous studies about torsion are based on a simple numerical single-storey asymmetric building model (De Stefano and Pintucchi 2007), therefore, derived measures of torsional effects provide insight to this complex parameter.

6.3 3D-SAM

6.3.1 3D-SAM overview

The 3D-SAM method predicts global seismic demands, response of buildings and their performance to a future earthquake based on their in-situ derived modal properties (typically obtained by ambient vibration tests and sensing techniques). The whole process of the method, its inputs and outputs are illustrated in Figure 6.1.

By knowing the dynamic properties of buildings from AVT, it is possible to calculate the building seismic response by convolution integral in the linear range according to classical structural dynamics theory. Unlike in previous studies of seismic building evaluation based on AVT (Michel and Guéguen 2009, Michel et al. 2012) the equation of motion is considered in three dimensions and earthquake records can be applied in any direction at the building base, i.e. three degrees of freedom are assumed for each rigid floor diaphragm including two orthogonal horizontal displacements and one rotational degree of freedom.

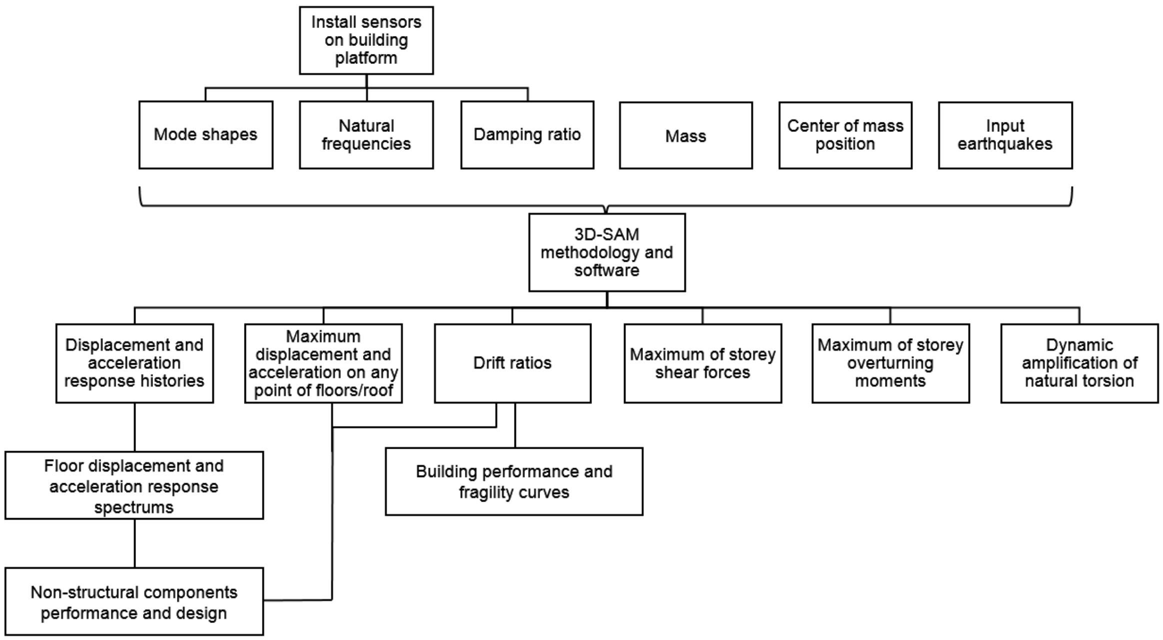

Figure 6.1 3D-SAM methodology

It is seen in Figure 6.1 that 3D-SAM is a direct top to bottom approach that makes use of the in-situ derived modal properties and therefore bypasses the need for detailed engineering plans and FE analysis models. The method is capable of providing the displacement and acceleration response histories at any point and direction on a rigid floor even if a sensor has not been placed exactly at that point during AVT. This makes the method particularly appealing for irregular and torsional sensitive buildings. More details about 3D-SAM and its verification by comparison to several calibrated FE models are shown in Mirshafiei et al. (2015) and Mirshafiei and McClure (2015). A summary of the verification process for one case study also presented in Mirshafiei et al. (2015) is shown next.

6.3.2 3D-SAM versus an updated FE model for an irregular building

Centre Communautaire de Loisirs de la Côte-des-Neiges is a low-rise irregular building located in Montreal which was constructed in 1993 with a steel braced frame structural system; its height is about 20 m including one basement floor. A bird's view of the building and its floor plans are shown in Figure 6.2. Black dots represent the sensors in the AVT (the reference sensors are indicated by the letter R). Figure 6.3 displays the extracted mode shapes and the corresponding natural periods and damping ratios; the red and black boundary lines are used to identify the undeformed and deformed mode shapes, respectively.

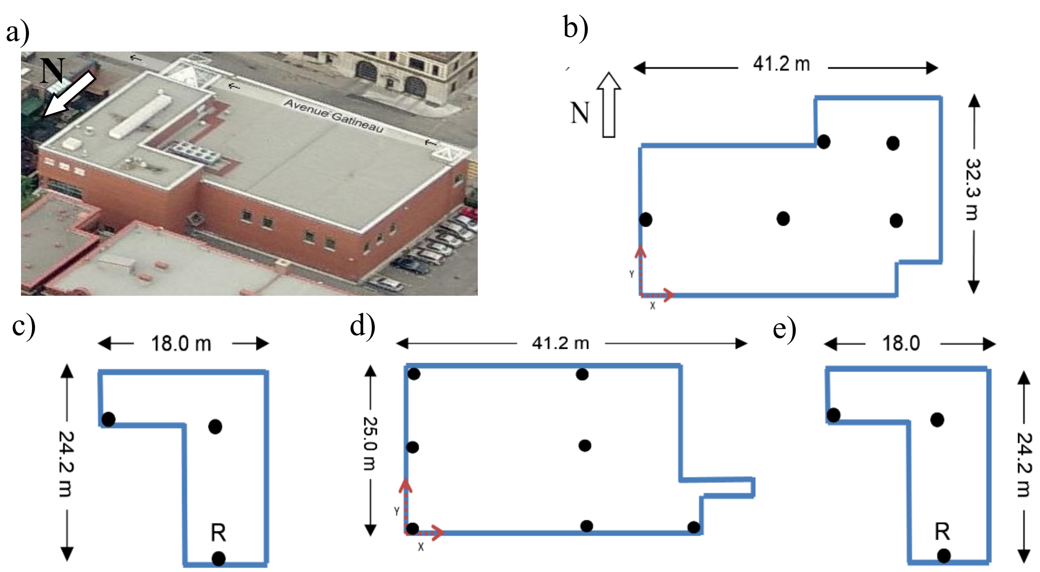

Figure 6.2 a) Bird's eye view of the tested building; b) 1st floor - height above ground 5.4 m; c) 2nd floor - height above ground 10 m (mechanical room) ; d) 3rd floor - height above ground 11.36 m (lower roof) e) 4th floor - height above ground 14.5 m (upper roof) (Mirshafiei et al. 2015)

a) b) c)

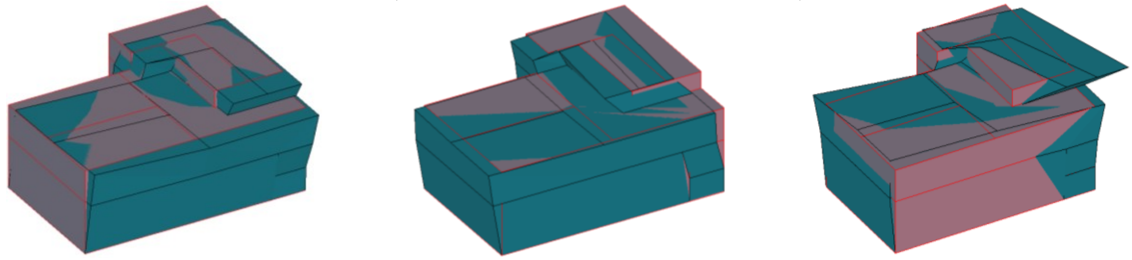

Figure 6.3 Mode shapes a) 1st flexural-torsional mode (period 0.24 s, damping ratio 0.016); b) 2nd flexural-torsional mode (period 0.24 s, damping ratio 0.012); c) 1st torsional mode (period 0.18 s, damping ratio 0.023) (Mirshafiei et al. 2015)

This steel braced frame building was modelled in details (walls and ceiling's supporting truss system are included) in SAP2000 (Computers and Structures 2009) and calibrated to AVT modal properties (Figure 6.4). Comparisons between the natural frequencies and mode shapes (by modal assurance criteria) of AVT and FE model results are presented in Table 6-1a and b, respectively. It is seen that discrepancies between the FE model and in-situ test results remain despite the detailed character of the structural model. This illustrates that investing more time and effort to build an updated FE model cannot provide a 100% calibrated model. Next, ten synthetic horizontal ground motions compatible with Montreal's Uniform Hazard Spectra (UHS) were selected as input to the building base and both the FE model and 3D-SAM model were used to calculate different response indictors. Due to space limitations, the only comparison shown in Figure 6.5 is for the displacement and acceleration response histories of corner joint A (Figure 6.4a) for one earthquake record (Figure 6.4b) applied in Y direction, however, the conclusions remain the same. More information can be found in Mirshafiei et al. (2015).

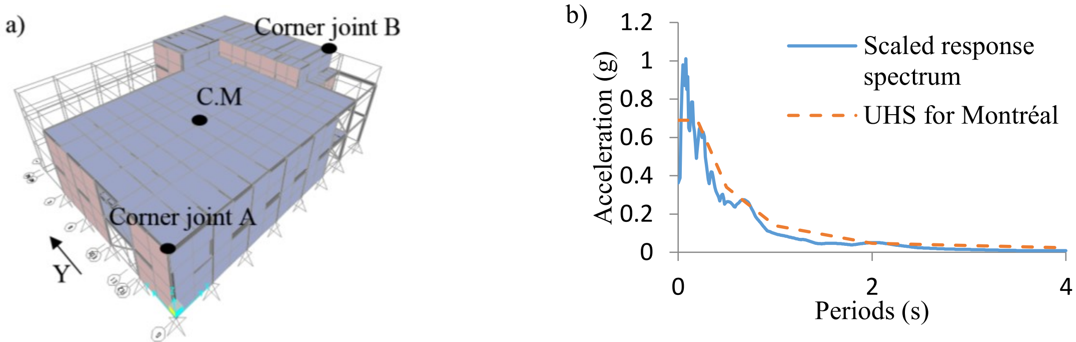

Figure 6.4 a) 3D view of the FE model; b) response spectrum of the input ground motion and UHS for Montreal (Mirshafiei et al. 2015)

Table 6-1 a) Comparison of natural frequencies between AVT and FE model; b) Modal assurance criteria (MAC) values between FE model and AVT mode shapes

a)

Comparison	Natural frequencies (Hz)		
	1st mode	2nd mode	3rd mode
FE model	4.1	5.0	7.0
AVT	4.1	4.2	5.7
Difference (%)	0	19	23

b)

MAC values	FE model			
		Mode1	Mode2	Mode3
AVT	Mode1	**0.97**	0.00	0.04
	Mode2	0.02	**0.92**	0.00
	Mode3	0.01	0.01	**0.80**

Figure 6.5 shows that differences between the two methods are larger for acceleration results than for displacements. In fact, if a calibrated FE model could be updated to have the same natural frequencies and mode shapes as the ones extracted from AVT records, then 3D-SAM and FE model would yield exactly the same results. Therefore, 3D-SAM is very attractive and efficient to predict response directly based on the properties derived from the AVT, without the need for the FE model updating phase.

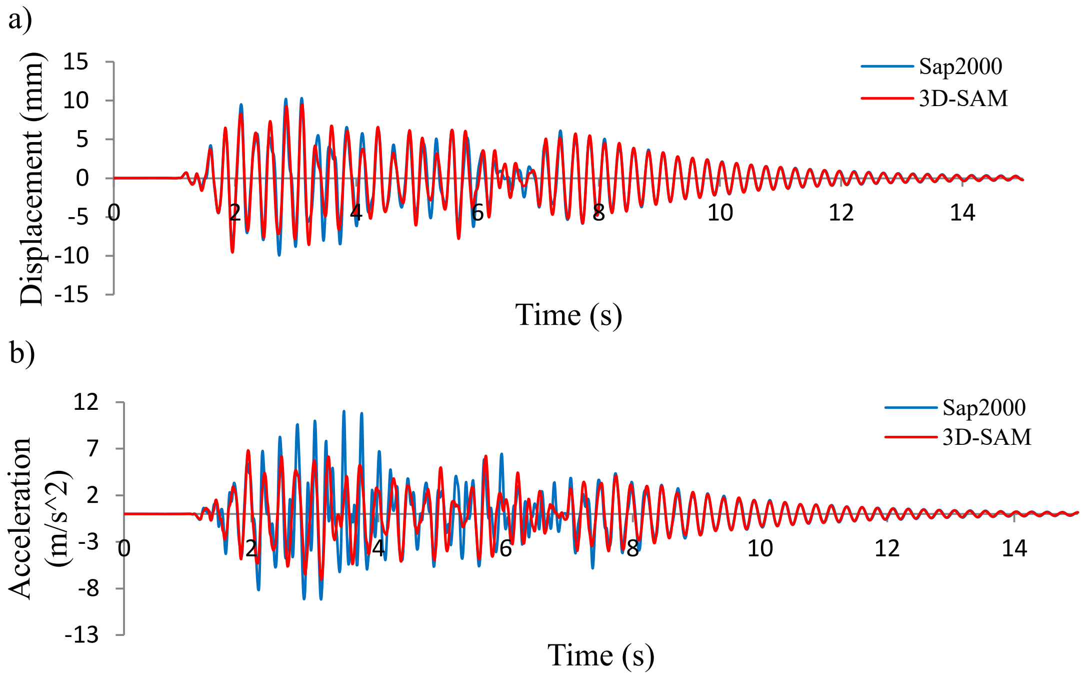

Figure 6.5 Comparison between relative displacement and absolute acceleration histories of corner joint A by FE model and 3D-SAM- earthquake is applied in Y direction

6.4 Modified 3D-SAM for buildings subjected to moderate to strong earthquakes

6.4.1 Modified 3D-SAM

The dynamic building properties extracted from strong-motion records (peak ground acceleration PGA > 0.1g) are expected to be different from those obtained using weak-motion such as low amplitude ambient vibration (PGA < 10^{-5}g). This difference is generally attributed to several factors: (1) the non-linear behaviour of the structural material (such as micro-cracking of the concrete at the foundation or superstructure); (2) connection slippage (in bolted steel structures and timber structures); (3) interaction between non-structural and structural elements; and (4) soil-structure interaction effects (Dunand et al. 2006). For instance, consider the simple example of a single storey, single bay with length of two times the storey height, concrete moment frame with fixed supports and rigid connections (beams are considered axially rigid with no shear deformation). The change in natural sway frequency when the cracked section properties (0.4 and 0.7 of the gross second moment of area of the beam and columns, respectively) are considered is due to the change in stiffness; for different ratios of α (second moment of area of beam / (4*second moment of area of column)), a maximum reduction of almost 20% in the natural frequency was observed as compared to the case with the gross sectional properties (lateral stiffness, k, is expressed as $k = \frac{24\, E\, I_{Column}}{storey\ height^3} * \frac{12\alpha+1}{12\alpha+4}$ (Chopra 2007)).

Changes in modal characteristics and wandering of natural frequencies were also observed in undamaged structures (with slight or not visible damage) subjected to strong motion (Celebi 2007). The normal tendency is for natural frequencies to decrease and damping ratios to increase with seismic intensity, while mode shapes are not altered much as long as no localized damage happens. Therefore, appropriate modification factors can be applied to the modal properties derived by AVT (minute amplitude motion) for an improved prediction of the linear response of the building, before the structure reaches a damage state due to strong excitations. Such modification factors can be derived from data collected in buildings equipped with permanent strong-motion instrumentation where the building has not suffered visible structural damage during the strong base motion. After careful review of such buildings in the literature, consisting of 18 buildings listed in (Celebi 2007, Dunand et al. 2004 and 2006, Çelebi 2009, Çelebi et al. 1993, Carreno and Boroschek 2011, Soyoz 2013, Singh et al. 2014) and another 21 buildings subjected to 1994 Northridge earthquake and its aftershocks (Todorovska et al. 2006 and 2007), the following observations are made: (1) the strong-motion modal frequencies are decreased by a maximum of 30% and 40% of the corresponding values extracted from ambient vibration records for steel and concrete buildings, respectively. These results were obtained on diverse building types and heights, and their detailed descriptions are found in the above references. As for the seismic assessment and earthquake performance based-design, the floor displacements, drift ratios and subsequent damage are the key elements, applying these maximum modification factors to the AVT natural frequencies is considered conservative; (2) the mode shapes are not changed from ambient to strong vibration levels (before the occurrence of damage); (3) the internal damping ratio for strong-motion response can be as much as 2 to 4 times larger than found using ambient measurements. Consequently, to be conservative and according to the earthquake performance based-design concept mentioned above, the damping ratios derived from AVT can be multiplied by the factor 2. The above are approximate and conservative modifications. As the number of buildings being permanently instrumented is increasing and more data from earthquake events become available, further refined modification factors can be used in 3D-SAM.

Moreover, the prediction of non-linear seismic demands using linear analysis has been widely used for seismic design, codes and vulnerability assessment; The Equal displacement rule (EDR) has been widely accepted since the 1960s (Veletsos and Newmark 1960) and its validity has been confirmed by several numerical and experimental investigations (Lestuzzi and Badoux 2003, Michel et al. 2014). This rule has been found to be generally correct for buildings with natural frequencies under 2 Hz. In a more recent study Michel et al. (2014) showed that the linearized method performs well when the strength reduction factor (ductility factor) is less than 2. Therefore, EDR should be valid for buildings with low to moderate ductility (strength reduction factor less than 2), post-disaster structures (hospitals, community centres, schools, emergency shelters, etc.), and structures that are expected to suffer low to moderate damage during an earthquake.

With the proposed adjustment of AVT modal characteristics, the 3D-SAM methodology and software can be used for seismic assessment of the aforementioned types of structures for which EDR remains valid, and also for seismic assessment of buildings where the purpose is finding linear demands. Using a similar approach, 3D-SAM can be applied to buildings susceptible to more severe earthquake-induced damages upon availability of more refined modification factors to relate nonlinear response to the linear one.

6.4.2 Application of modified 3D-SAM

Another building case study is used to illustrate the application of 3D-SAM and the effects of modified modal properties on the seismic demands. This is a six-storey reinforced concrete moment frame building that is part of a Montreal community centre built in 1975 (the building is shown by red boundaries in Figure 6.6a). It comprises two basements and its height above ground level is 15.9 m. The first three floors have a rectangular shape of approximate dimensions 6.4 m by 32 m, and the upper three stories have an L-shape plan. The three lowest frequency mode shapes have been derived from AVT, i.e. 1st flexural-torsional mode (3.33Hz, damping ratio (ξ) of 2.3%), 2nd flexural-torsional mode (4.52 Hz, ξ = 2.9%), and 1st torsional mode (5.47 Hz, ξ = 2.6%), and details of the test can be found in Mirshafiei and McClure (2012).

The building is subjected to an ensemble of ten synthetic earthquakes (response spectra are shown in Figure 6.6b and based on a study by Assi (2006)) compatible with Montreal's moderate seismicity, and covering the appropriate frequency range of interest, peak ground acceleration, magnitude, epicentral distance and duration. Examples of global seismic demand results analyzed by 3D-SAM when the earthquakes are applied aligned X direction are shown in Figure 6.7.

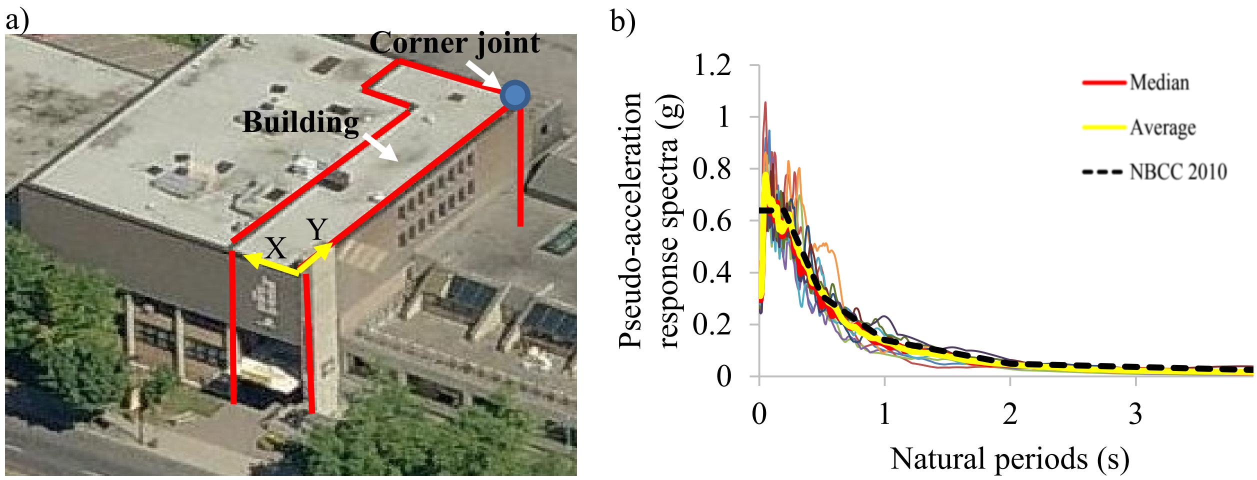

Figure 6.6 a) Bird's eye view of the tested building; b) Pseudo-acceleration response spectra of the synthetic earthquakes compatible with UHS of Montreal (NBCC 2010)

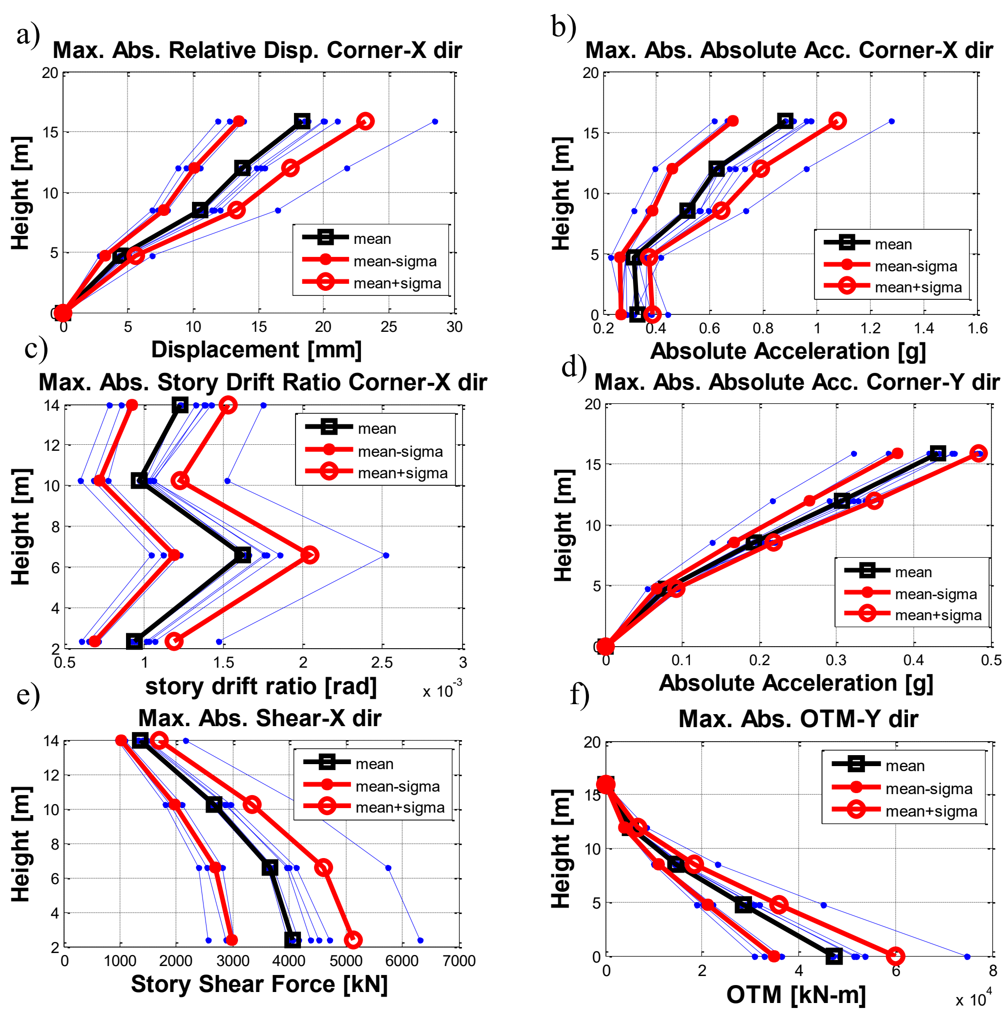

Figure 6.7 Global seismic demands derived from 3D-SAM with ten earthquakes

The red circles in Figure 6.7 on the mean ± sigma graphs show the floor locations along height, except for drift ratio and shear force, where they indicate half story heights. It is seen that 3D-SAM is capable of finding maximum responses aligned and perpendicular to the direction of the applied earthquakes. Next, the effects of the modification factors in natural frequencies and damping ratios are investigated. The natural frequencies have been decreased by factors of 10%, 20%, 30% and 40% and the change in the mean value of the different seismic demands are observed; for each reduction factor the building is reanalyzed with the ten earthquake records applied in X direction. As shown in Figure 6.8 the displacements and drift ratios of different floors are increased as natural frequency reductions get larger, as expected. Therefore, as discussed in section 6.4.1 in the context of EDR and to obtain conservative predictions of displacements and drift ratios, it is suggested to use the 40% reduction in natural frequencies for concrete structures to calculate maximum responses or to use mean + sigma of response lines in Figure 6.8a and b. The drift ratios can be compared with code/published limits for different building types and damage grades to produce fragility curves and predict building performance for a future earthquake. In case of forces as is seen in Figure 6.8c, decreasing the natural frequencies will result in reduced forces, which provide improved results as compared to the original overestimated outcomes of the linear AVT-based analysis. A word of caution: predicting reasonable seismic force values from linear analysis with strong shaking is not the best practice and further analysis of such structure with alternative tools and methodologies is suggested. Finally, Figure 6.8d shows that accelerations also decrease as the natural frequencies are decreased. Therefore, if the goal is to report conservative predictions of the maximum floor acceleration, 0% reduction in natural frequency or mean + sigma of response lines can be used.

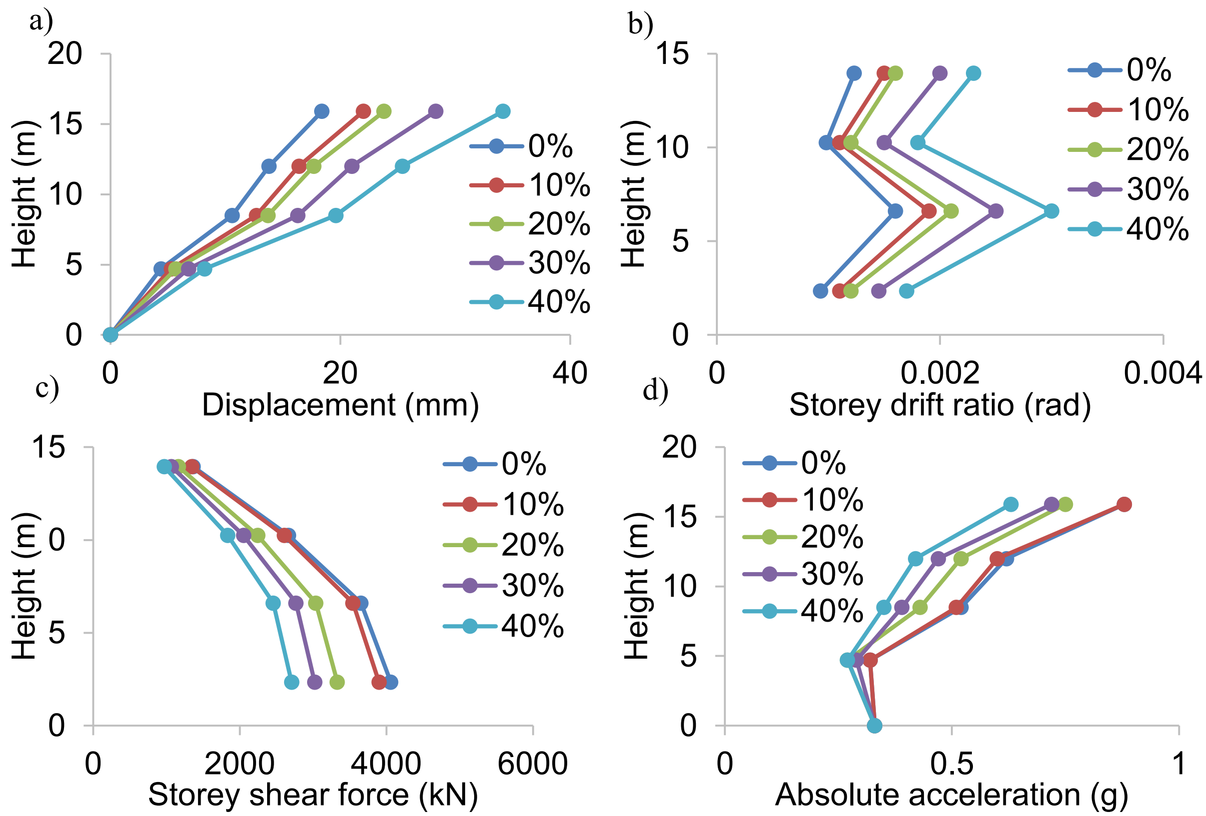

Figure 6.8 Change in the mean value of seismic demands for different reduction factors in natural frequencies

Figure 6.9 demonstrates the effect of varying damping in the mean value of different seismic demands. As it was mentioned in section 6.4.1 damping during strong-motion excitation can be 2 to 4 times of the values extracted from AVT records. Therefore, for each modified damping value the building is reanalyzed with the ten earthquake records applied in X direction, and the mean values of the seismic demands from each analysis are obtained. As expected all seismic demands get smaller when the damping ratios increase. With a view to find reasonable estimates of the maximum displacement and drift ratio demands, it is suggested to double the damping ratios obtained from AVT before inputting them into the 3D-SAM model. It should be mentioned that the suggested modification factors can be refined upon availability of more data from permanently instrumented buildings to further enhance the accuracy of predictions.

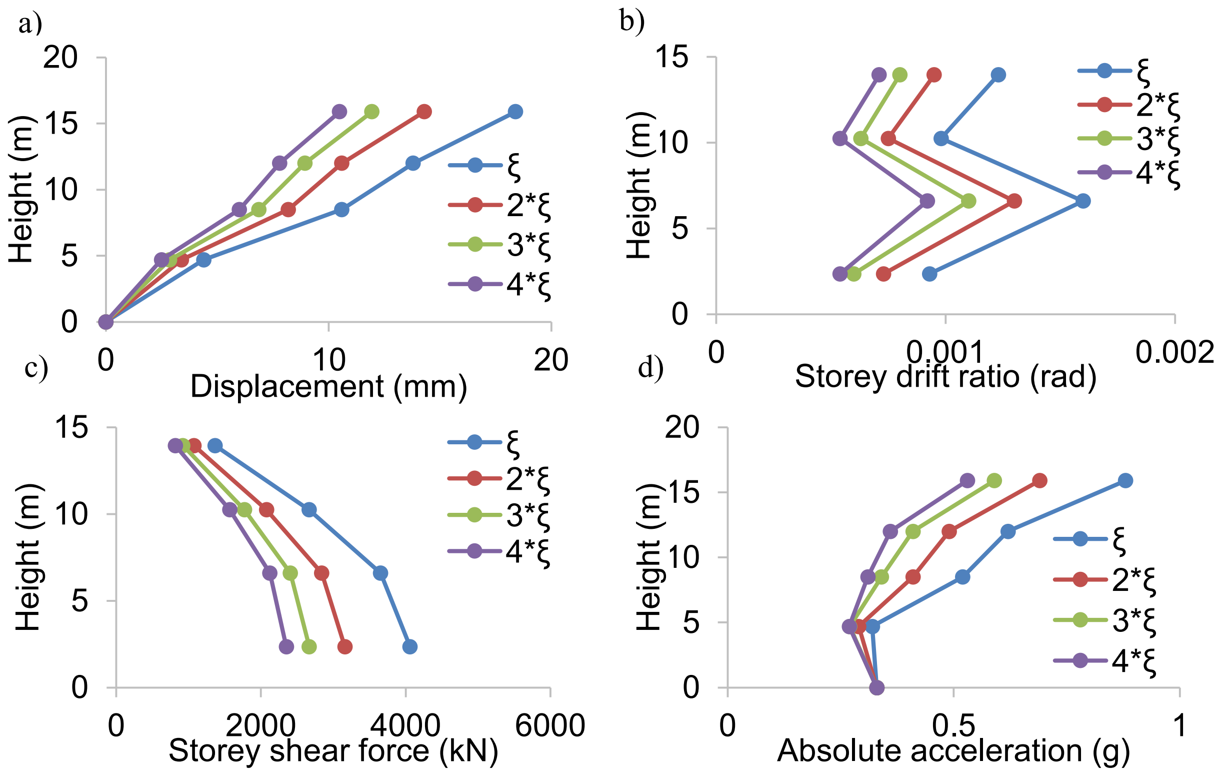

Figure 6.9 Change in the mean value of each seismic demand for different damping ratios

6.5 Dynamic amplification of natural torsion

In irregular buildings, eccentricities between the centres of mass and rigidity at each floor cause torsional motion during an earthquake. This torsion leads to increased displacements at the extremities of the buildings. Structures with non-coincident centres of mass and rigidity are referred to as asymmetric structures and the torsional motion induced by asymmetry is referred to as natural torsion. Asymmetry may in fact exist even in a nominally symmetric structure because of uncertainty in the evaluation of the centres of mass and rigidity, inaccuracy and variability in the dimensions of structural elements, or lack of precise data on material properties. Torsional vibration may also result from rotational motion of the ground about the vertical axis. Torsions coming up from undetermined asymmetry and ground rotational motion are together referred to as accidental torsion (Humar et al. 2003).

In the simplified quasi-static procedure of the National Building Code of Canada (NBCC 2010 section 4.1.8.11) torsional seismic effects are considered by applying torsional moments, about a vertical axis at each floor level:

$$T = F(e \pm 0.1D_n) \tag{6.1}$$

Where F is the seismic lateral force at each level (the force should be considered in both orthogonal directions), D_n is the plan dimension of the building at each level "n" perpendicular to the direction of seismic loading being considered, and "e" is the natural eccentricity due to the centres of rigidity and mass being at different positions. De la Llera and Chopra (1995) showed that accidental torsion is represented by $0.05D_n$ and the remainder (another $0.05D_n$) accounts for the dynamic amplification portion of natural torsion.

Moreover, NBCC requires 3D dynamic analysis for torsionally sensitive structures for which the sensitivity parameter B > 1.7. B is taken as the maximum of all values of B_n in both orthogonal directions. B_n is equal to $\frac{\delta_{max}}{\delta_{ave}}$; δ_{max} is the maximum storey displacement at the extreme points of the structure at level "n" in the direction of the earthquake induced by the equivalent static forces acting at a distance $\pm 0.1D_n$ from the centres of mass at each floor, δ_{ave} is the average of the displacements of the extreme points of the structure at level "n" produced by the same forces. B_n is calculated for all building floors and for two orthogonal directions.

The building code provisions and research for design against torsion are mostly based on studies of elastic response of torsionally unbalanced buildings to earthquake motion and to a large extent based on elastic response of a simple idealized asymmetric single-story building (De Stefano

Pintucchi 2007). Therefore, this topic still needs further research to consider other effects on torsion, vertical irregularities, and eccentricities in multistory buildings.

3D-SAM is capable of providing information about the torsion from the ambient vibration tests. The various outputs of the 3D-SAM method are determined from the calculated relative displacement vectors at center of mass on each floor. By assuming rigid in-plane movement of each floor, relative displacement vectors can be obtained at any floor location including building corners. Absolute accelerations are simply estimated by taking the second time derivative of relative displacement vectors and adding the ground acceleration. Multiplying the horizontal components of absolute acceleration of the floor center of mass by the floor mass, the inertia force at each floor is computed which leads to the determination of shear force and overturning moment. Finally, the second time derivative of angular displacement at the center of mass yields the angular acceleration "α". The moment of inertia of the each floor, I_o, about the vertical axis through the center of mass multiplied by "α" yields the resultant torque at center of mass. This torque divided by the inertia force leads to the additional eccentricity that represents the dynamic amplification portion of natural torsion.

In this section, we apply earthquakes along two orthogonal axes independently; i.e. E_x is a base motion along the x direction and E_y is a base motion along the y direction. Contrary to static analysis, horizontal inertia forces exist in both orthogonal directions even if the earthquake is applied along one direction. Therefore, considering the torque from the 3D-SAM analysis, two definitions can be used to determine the additional eccentricity that can replicate the dynamic amplification portion of natural torsion on the center of mass of each floor, see Figure 6.10a:

1) $\frac{Torque}{Total\ inertia\ force}$ is calculated on each floor and time step during an earthquake record and the mean value for the total analysis time is reported as "ecc". Moreover, the components of this eccentricity are reported as "ecc_x" and "ecc_y" along x and y axes with the same approach, Figure 6.10b.

2) $\frac{Torque}{Inertia\ force\ in\ direction\ of\ the\ applied\ earthquake}$ is calculated on each floor and time step during an earthquake record and the mean value for the total analysis time is reported as "ecc_dir_time". Furthermore, this fraction is calculated as the maximum torque divided by the maximum inertia force in the direction of the applied earthquake (the maximum value is the peak value from the whole analysis time) and is called "ecc_dir_max".

To be able to report all the eccentricity quantities in percentage, they are divided by the plan dimension perpendicular to the direction of the seismic loading being considered.

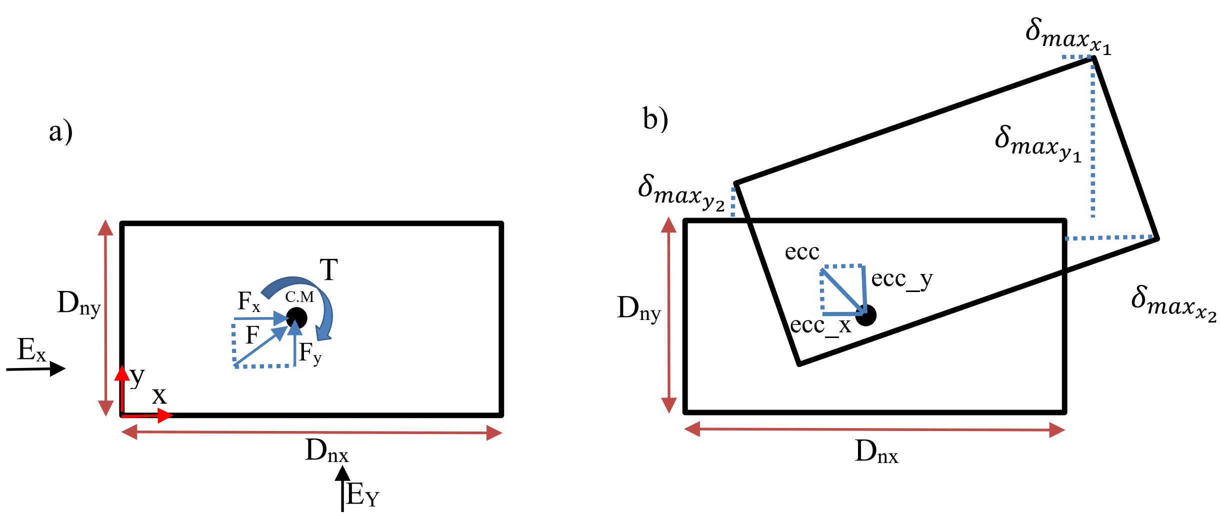

Figure 6.10 a) Total inertia force, its components and the torque generated at a floor centre of mass; b) Eccentricities that can represent the dynamic amplification portion of the natural torsion

To demonstrate the defined eccentricities, the case study of section 6.3.2 is subjected to the ten earthquakes being applied in y direction and results are shown in Figure 6.11. Furthermore, the mean values from the results of "ecc_dir_time" are plotted versus B_x (Figure 6.11f) where $B_x = \frac{\delta_{max_{y_1}}}{(\delta_{max_{y_1}}+\delta_{max_{y_2}})/2}$.

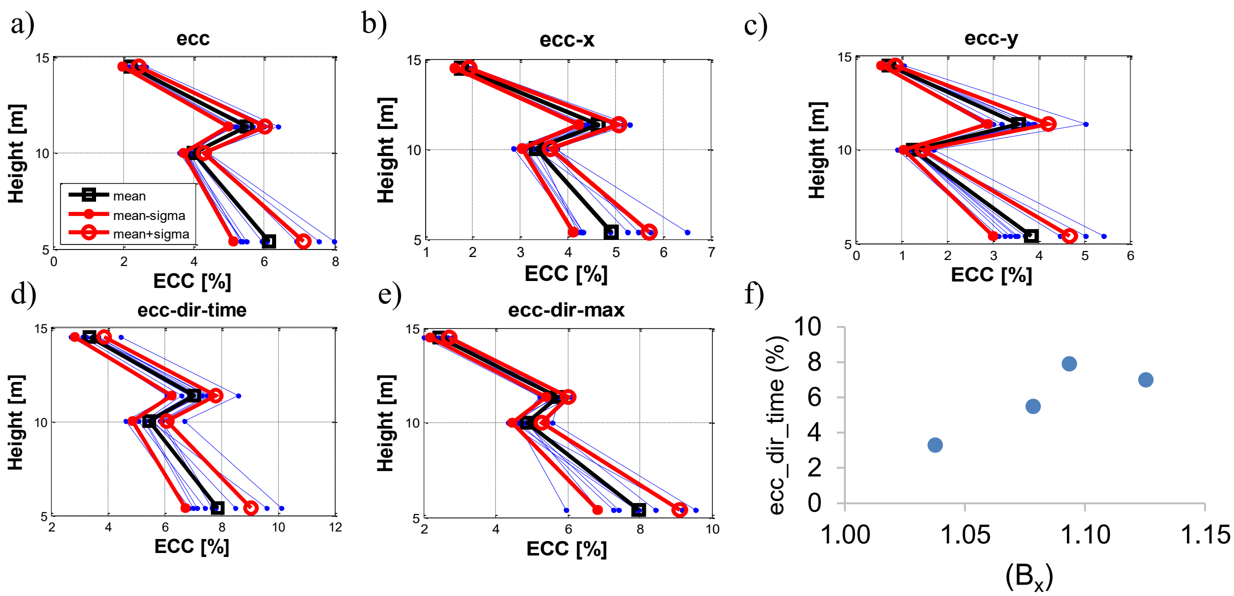

Figure 6.11 Floor eccentricities (%) due to the dynamic amplification portion of natural torsion for the case study of section 6.3.2- earthquake is applied along y direction

Next, to get better insight about the dynamic amplification portion of natural torsion and its equivalent additional eccentricity, the same 16 irregular buildings located in Montreal and discussed previously in Mirshafiei & McClure (2012) and (2015) were considered (see Table 6-2). These buildings were tested by AVT and analyzed by 3D-SAM. The lateral force resisting system (LFRS) types are categorized according to FEMA 154 where C1, S2 and S5 stand for the following types, respectively: Concrete moment resisting frames, Braced steel frame buildings and Steel frame buildings with unreinforced masonry infill walls. Longest floor plan dimensions and height above ground level are also given. Moreover, the properties of three lowest frequency modes were extracted from AVT records for all the buildings and the results are listed in Table 6-3 (details can be found in Mirshafiei & McClure (2012) and (2015)).

Table 6-2 Characteristics of tested buildings

Building ID	Year of construction	LFRS	Total height (m)	Number of floors for ecc. calculations	Largest plan dimensions (m*m)
Complex B	1993	C1	11.7	2	91*53
Complex C	1961	C1	13.1	2	53*45
Complex G-Bldg. 1	1957	C1	15.0	3	58*20
Complex G-Bldg. 2	1957	C1	11.0	3	52*11
Complex F-Bldg. 1	1964	C1	17.1	4	42*25
Complex F-Bldg. 3	1964	C1	17.1	2	43*39
Complex A-Bldg. 1	1975	C1	21.0	4	32*6
Complex A-Bldg. 2	1975	C1	21.0	4	46*32
Complex A-Bldg. 3	1975	C1	13.5	4	36*32
Sainte-Justine Hospital	1957	C1	39.6	10	64*14
Complex E	1976	S2	7.9	2	43*26
Complex H	1993	S2	20.0	4	41*32
Complex D	1996	S2	11.6	2	76*39
Complex A–Bldg. 4	1975	S2	13.0	1	37*31
Complex A-Bldg. 5	1975	S2	13.0	1	37*31
Complex F-Bldg. 2	1914	S5	18.6	4	61*23

Table 6-3. Modal building properties extracted from AVT records

Building No.	Mode 1		Mode 2		Mode 3	
	Period (s)	Damping (%)	Period (s)	Damping (%)	Period (s)	Damping (%)
Complex B	0.19	2.0	0.18	1.8	0.13	2.1
Complex C	0.23	1.7	0.21	1.7	0.16	3.3
Complex G-Bldg. 1	0.15	2.9	0.12	1.4	0.10	2.4
Complex G-Bldg. 2	0.18	1.5	0.17	1.3	0.10	2.0
Complex F-Bldg. 1	0.38	4.1	0.38	4.0	0.23	3.0
Complex F-Bldg. 3	0.38	3.6	0.38	3.9	0.15	1.4
Complex A-Bldg. 1	0.30	2.3	0.22	2.9	0.18	2.6
Complex A-Bldg. 2	0.29	2.0	0.22	2.3	0.18	1.6
Complex A-Bldg. 3	0.15	2.1	0.13	1.8	0.11	2.1
Sainte-Justine Hospital	0.53	1.7	0.40	1.2	0.37	1.1
Complex E	0.18	6.0	0.14	4.2	0.13	2.0
Complex H	0.24	1.6	0.24	1.2	0.18	2.3
Complex D	0.24	4.0	0.17	2.1	N/A	N/A
Complex A–Bldg. 4	0.27	2.5	0.18	2.3	0.10	1.2
Complex A-Bldg. 5	0.30	3.2	0.19	2.0	0.10	1.5
Complex F-Bldg. 2	0.5	5.0	0.38	5.0	0.38	3.7

Each building is analysed using its 3D-SAM model subjected to the same ten earthquakes in the two orthogonal axes independently (Figure 6.6b) and the different defined eccentricities are calculated for each earthquake and on each floor. The mean eccentricity values are calculated from the response to the ten base motion records (same procedure as seen in Figure 6.11) for each building. In total 104 eccentricity values are found for the whole database (all buildings and all floor and roof levels). $B_x = \frac{\delta_{max_{y_1}}}{(\delta_{max_{y_1}} + \delta_{max_{y_2}})/2}$ and $B_y = \frac{\delta_{max_{x_1}}}{(\delta_{max_{x_1}} + \delta_{max_{x_2}})/2}$ (Figure 6.10b) were obtained from the response time histories and all the eccentricities were plotted against different combinations of B_x and B_y to observe any relation between eccentricities and these B parameters. The following combinations were selected: B_x, B_y, average of (B_x, B_y), maximum of (B_x, B_y), square root of ($B_x^2 + B_y^2$), and finally whichever of (B_x, B_y) that is in the direction of the applied earthquake. Due to space limitation, only the eccentricities versus the average of (B_x, B_y) are shown in Figure 6.12, however the conclusions remain the same for other scenarios. It is seen that for the whole dataset the eccentricities are scattered in relation to the defined parameters B; there might be different range of the eccentricities for a particular B. This is expected as during the course of the dynamic response history analysis for a particular floor of one case study, there may be a large dynamic torque at C.M as compare to the inertia force that leads to a larger needed equivalent eccentricity to accommodate such a torque, however, this may not be the case for another case study with the same B. The B parameter depends on the relative displacements of the extreme points and therefore depends on the position of these points with respect to the centre of mass and the angular rotation at the centre of mass. Therefore, results of this study shows no clear relation between the defined eccentricities and B parameters for different layout of diverse buildings.

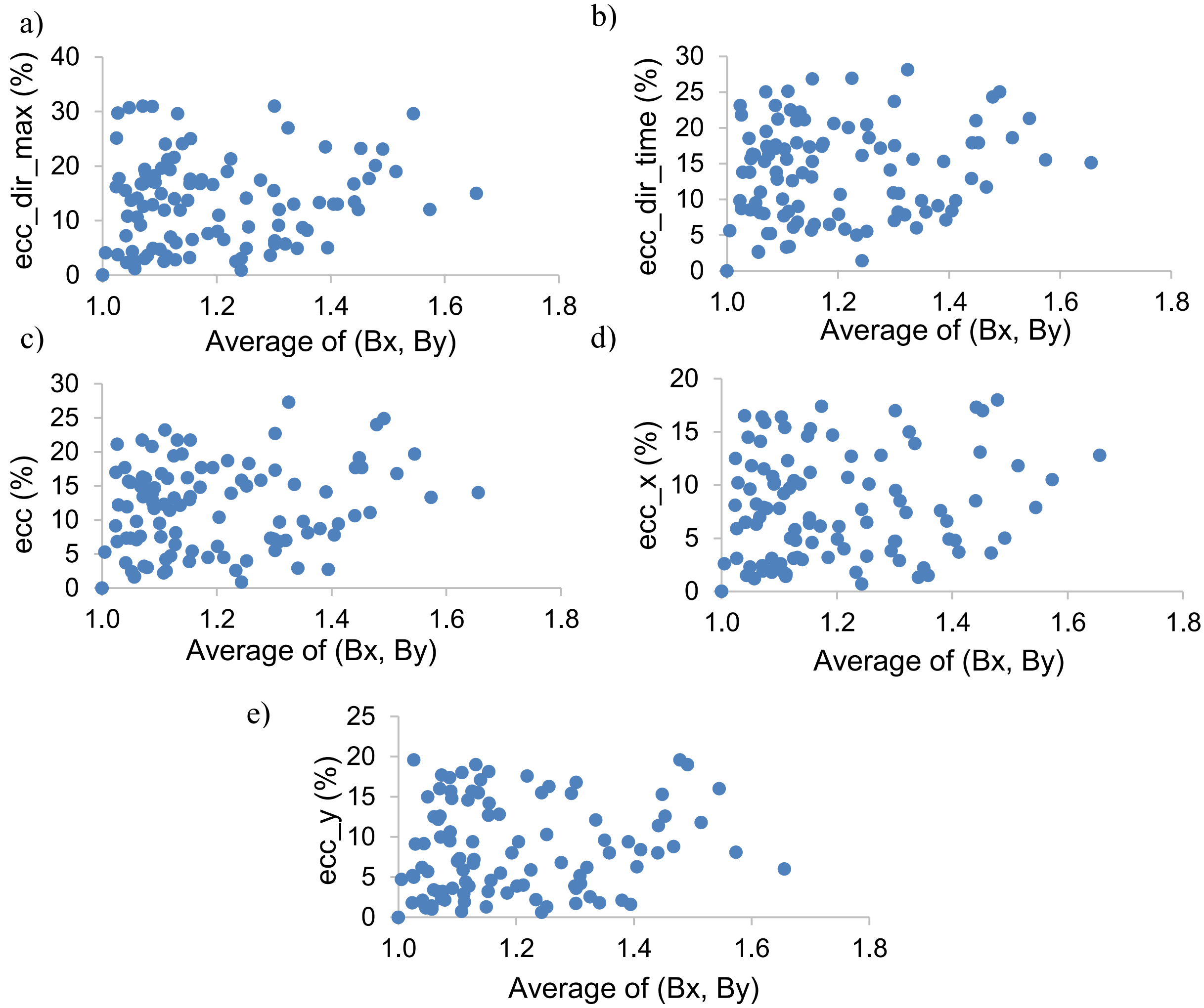

Figure 6.12 Different defined equivalent eccentricities for the dynamic amplification portion of natural torsion for 16 irregular buildings located in Montreal

It is important to clearly distinguish between the defined torsional sensitivity parameter of the NBCC code and that defined in this study; the code prescribes the limit for the torsional sensitivity parameter based on a study of a single-storey building and using equivalent static analysis considering natural and accidental torsion, while in this study the calculations are made from dynamic analysis, for different buildings, and the correlation between the defined B parameters and the dynamic amplification portion of natural torsion was investigated.

Table 6-4 shows the range of the calculated eccentricities from this study, as an indication of the expected range of the dynamic amplification portion of natural torsion. It should be mentioned that for any defined eccentricity type, the mean value and the mean + one standard deviation value had a probability of non-exceedance of almost 50% and 80%, respectively, for the whole dataset. These results are valid for the seismicity level corresponding to Montreal, Canada and cannot be extrapolated to other seismic regions without proper validation.

Table 6-4 Equivalent floor eccentricities of the dynamic amplification portion of natural torsion of 16 buildings in Montreal

	ecc_dir_max (%)	ecc_dir_time (%)	ecc (%)	ecc_x (%)	ecc_y (%)
Mean	13.1	13.6	11.7	7.7	8.2
Mean+sigma	21.3	20.3	18.1	12.7	13.9
Median	13.0	13.8	12.2	7.0	7.1

6.6 Conclusion

In this paper the 3D-SAM, a new simplified 3-dimensional seismic assessment methodology was verified for an irregular low-rise building located in Montreal. Results showed a good agreement between updated linear FE model and the 3D-SAM results. Therefore, 3D-SAM approach can be an attractive tool for researchers who would like to predict linear response histories and global seismic demands directly based on the properties derived from the AVT; they can bypass the FE model phase and the need for detailed engineering plans.

For stronger base excitations, the modal properties derived from ambient vibration levels should be modified to accommodate changes in system properties during strong motions. Therefore, it was shown that to be conservative and according to the earthquake performance based-design concept, to be able to have a reasonable prediction of drift ratios and displacements, the damping ratios derived from ambient vibration records can be multiplied by the factor two, mode shapes can remain unchanged and finally natural frequencies can be reduced to the maximum of 30% and 40% of the corresponding values extracted from ambient vibration records for steel and concrete buildings, respectively. Another way of considering changes in natural frequencies is to redo the response history analysis each time by different natural frequency reduction factors of 10%, 20%, 30% (and 40% in case of concrete structures) and then use the mean plus one standard deviation of all the results as the conservative outcome. Moreover, after having a reasonable prediction of the dynamic properties of the building before yielding point, in cases that equal displacement rule is valid, the method application can be extended to buildings subjected to strong earthquake excitations. This includes buildings with low to moderate ductility (ductility factor less than 2), post-disaster structures (hospitals, community centres, schools, shelters), structures that suffer moderate damage, and more generally for buildings with natural frequencies under 2Hz. Consequently, 3D-SAM methodology and software can be used for seismic assessment of these types of structures, and any building seismic analysis with the purpose of finding linear demands. 3D-SAM may be applied to buildings susceptible to more sever damages during an earthquake upon availability of more refined modification factors to relate nonlinear response to the linear one.

And finally in the last section of this paper, 16 irregular buildings that had been tested via AVT were analyzed by 3D-SAM. For each building ten earthquakes compatible with UHS of Montreal were applied independently along two orthogonal axes and additional eccentricities equivalent to dynamic amplification portion of natural torsion were reported on each floor. Mean value and mean plus one standard deviation of these equivalent eccentricities that can represent the existing dynamic torque on C.M, by shifting inertia force from C.M, were shown to be almost 13% and 20%, respectively, for the whole database. The authors suggest using the same procedure for other cities with different seismicity, to get an estimate range of the dynamic amplification portion of natural torsion from the real buildings.

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Link between chapter 6 and chapter 7

In chapter 6, modification factors were introduced in 3D-SAM to extend its range of applicability from ambient noise levels to stronger base shaking, with a view to perform seismic assessment of buildings subjected to stronger shaking. Moreover, the application of this methodology to calculate the dynamic amplification portion of natural torsion of irregular buildings was presented and the results were discussed.

In chapter 7, the full application of 3D-SAM is demonstrated with the calculation of the global seismic demands of four post-disaster buildings located in Montreal, Canada. The calculated response indicators are: response histories of relative displacements and absolute accelerations, maximum relative floor displacements, story drift ratios, floor absolute accelerations, story shear forces, overturning moments and dynamic amplification portion of natural torsions.

7. Application of a three-dimensional seismic assessment method (3D-SAM) based on ambient vibration tests to few buildings located in Montreal

7.1 Abstract

Ambient vibration testing applied to building structures is an in-situ modal experiment where low amplitude structural motions are recorded during the building's normal operations or everyday activities. Recorded motions at various points using local sensors are then processed using frequency domain decomposition techniques with a view to extract the essential dynamic properties of the building: mode shapes, natural frequencies and corresponding modal damping ratios. The derived experimental modal properties are used directly as input of the 3D-SAM (three-dimensional seismic assessment method and software), which is based on linear dynamic analysis and calculates the building's global seismic demand parameters such as relative floor displacements, story-drift ratios, floor absolute accelerations for any point on the floors, story shear forces, overturning moments and floor response spectra. All these seismic demands are calculated without the need to make any finite element model of the building, which makes the method very attractive to assess existing buildings. The method is inherently capable of considering torsional behavior in the response prediction and can be used as a seismic assessment tool for irregular buildings. The paper will present a few examples that illustrate the application of this new method for buildings designated as post-critical shelters in the city of Montreal, Canada. Moreover, modification factors for extension of the 3D-SAM method from low vibration excitations to higher levels are introduced.

7.2 Introduction

Earthquakes cause damages to buildings resulting in hundreds of billions of dollars of economic loss and pose a threat to human lives. These risks are compounded by a growing urban population and an ageing infrastructure that occur both in developed and emerging countries. As governments become aware of these risks, they are budgeting billions of dollars and implementing risk evaluation and mitigation plans before future seismic events occur.

Current detailed seismic evaluation methods for buildings are based on numerical approaches (ASCE 41, FEMA 356, and NIST 2010) that are time consuming and costly, and not necessarily accurate. For example, in order to evaluate a building, an engineering firm needs to collect the building's detailed structural plans, take some in-situ tests to identify material properties, and then build a numerical model of the building using all that information. The process is further complicated by the fact that many older buildings (pre-CAD systems) do not have as-built structural drawings that are up to date with sufficient detailing. Also, buildings may be already damaged and need to be further assessed, some of them are of irregular shape, and also there might be interaction between non-structural components and structural elements, which was ignored at the design stage. Therefore, we saw a need for developing alternative simplified rational seismic evaluation methods for existing buildings, with recognized limitations and range of applicability. This need is particularly important for moderate seismic regions due to the lack of data on recent earthquakes and the scarcity of information, as the existing assessment methods are based on damage observations in high seismicity areas.

The proposed solution is to use low cost in-situ experimental modal tests, owing to advances in sensing techniques and analysing procedures, to derive the essential structural characteristic of the buildings and then use this information for seismic response assessment and prediction of economic losses for slight to moderate damage levels. Nowadays, the most popular experimental modal test for large structures is ambient vibration testing (AVT). Modal parameters such as natural frequencies, damping ratios and mode shapes are derived from AVT by application of well-known frequency domain analysis techniques available in commercial software. AVT is of easy application, a low cost method for large structures and its results were shown to be as reliable and similar to that of forced vibration tests (Brincker et al. 2001, Trifunac 1972, Gilles 2012, Mirshafiei and McClure 2012). The AVT-derived modal properties are used by researchers to calibrate numerical models and then improve the reliability of their seismic response assessment of buildings. However, this calibrated model process is somewhat complex, still requiring a very detailed finite element model (FE model) and in the end some discrepancies remain between the experimental and FE modal parameters, as it is not feasible to calibrate a FE model to 100% of the test results. This added complexity and increase in analysis time partly explain why AVT, sensing techniques and operational modal analysis software are still not that popular among structural engineering firms despite the undisputable added value they provide. In recent years, few researches have proposed simple models to use ambient vibration data directly to assess seismic demands of buildings. These models, strictly applicable only to symmetric structures, have been based on 2D lumped-mass assumption (Michel et al. 2009, and 2012). These models have many important shortcomings that jeopardize the reliability of their results: they assume constant mass for each floor and neglect building torsional behavior and coupling effect of lateral-torsional mode shapes. These shortcomings, inherent to the 2D approach, have prevented the introduction of a comprehensive seismic assessment tool and methodology based on AVT till today.

To address this need for a more realistic method that duly account for non-symmetry and the three-dimensional nature of buildings, a new three-dimensional seismic assessment method and software (3D-SAM) was introduced and verified in Mirshafiei et al. (2015). In this paper, a summary of this novel methodology is presented and illustrated with a building example. Then, the modification factors for extension of the 3D-SAM from small intensity earthquakes to higher excitation levels are introduced. Finally, the application of the 3D-SAM is shown for four building case studies located in Montreal.

7.3 3D-SAM

7.3.1 3D-SAM summarized description

To our best knowledge, 3D-SAM is the first comprehensive three-dimensional methodology and software based on the observation of real modal properties of a structure obtained from ambient vibration tests and other sensing techniques. The experimental modal properties are combined with building data collected from on-site inspection and the available architectural and structural plans, to provide input to the 3D-SAM method. Each building model can then be subjected to an ensemble of representative ground motion records and its global seismic demand parameters are computed.

By knowing the dynamic properties of buildings from AVT, 3D-SAM uses time domain convolution (Duhamel integral) to calculate the building seismic response in the linear range. Contrary to previous studies of seismic building evaluation based on AVT, the equation of motion is considered in three dimensions, i.e. three degrees of freedom are assumed for each rigid floor diaphragm including two horizontal displacements and one in-plane rotational degree of freedom. In this way, the coupling effects in sway modes and torsional modes are taken into account. Moreover, as the modal parameters are extracted from the *in-situ* AVM tests, further processing needs to be performed on these complex modal properties before putting them as input for the 3D-

SAM; these additional steps are explained in details in Mirshafiei et al. (2015). With all these procedures, the 3D-SAM method produces:

1) displacements and accelerations (relative and/or absolute) at any location and direction on floors and roofs;
2) global seismic demands such as storey shear forces, overturning moments, maximum displacements and accelerations at any floor and location; drift ratios which may lead to development of fragility curve and prediction of building performance for different damage grades;
3) drift ratios and absolute acceleration on each floor that will determine the non-structural performance;
4) displacements and acceleration response spectra for any location on floors which determine the performance of non-structural components;
5) and finally, an estimate of the dynamic amplification portion of natural torsion for each floor.

7.3.2 Verification of 3D-SAM with one building example

The building used in the verification is located on McGill University downtown campus (Burnside Hall) and was constructed in 1969 with a reinforced concrete shear wall system. It has 13 storeys with a height of about 47 m above ground level; it also has a basement beneath the ground level. A 3D view of the building, a typical floor plan and sensor layout on the floors, FE model and the applied ground motion are shown in Figure 7.1. The modal properties of this building were derived via AVT (five lowest frequency modes, work done by Gilles (2012)). The FE model is an equivalent model calibrated with AVT results and is made up of 4 equivalent columns representing shear walls around the building and two equivalent central columns in place of the interior columns and the elevator concrete shaft. Each floor is rigid in plane and the lumped floor masses are assigned to the floor centroids. The model is then subjected to a synthetic horizontal ground motion along its X direction. This record corresponds to a magnitude 6 event compatible with the National Building Code of Canada (NBCC) Uniform Hazard Spectra (UHS) for Montreal according to NBCC 2005. The relative displacement and acceleration time histories of the roof corner joint A and centre of mass C.M. (location shown in Figure 7.1b) are compared between the FE model (created in Sap 2000) and the 3D-SAM; the representative graphs are shown in Figure 7.2. The graphs show good agreement between the two methods. In fact, if the FE model is fully calibrated to the modal properties derived from AVT, the two approaches will yield the same results. Therefore, the simplified 3D-SAM is a good alternative to linear updated FE models and leads to reliable results without the need for making a numerical model.

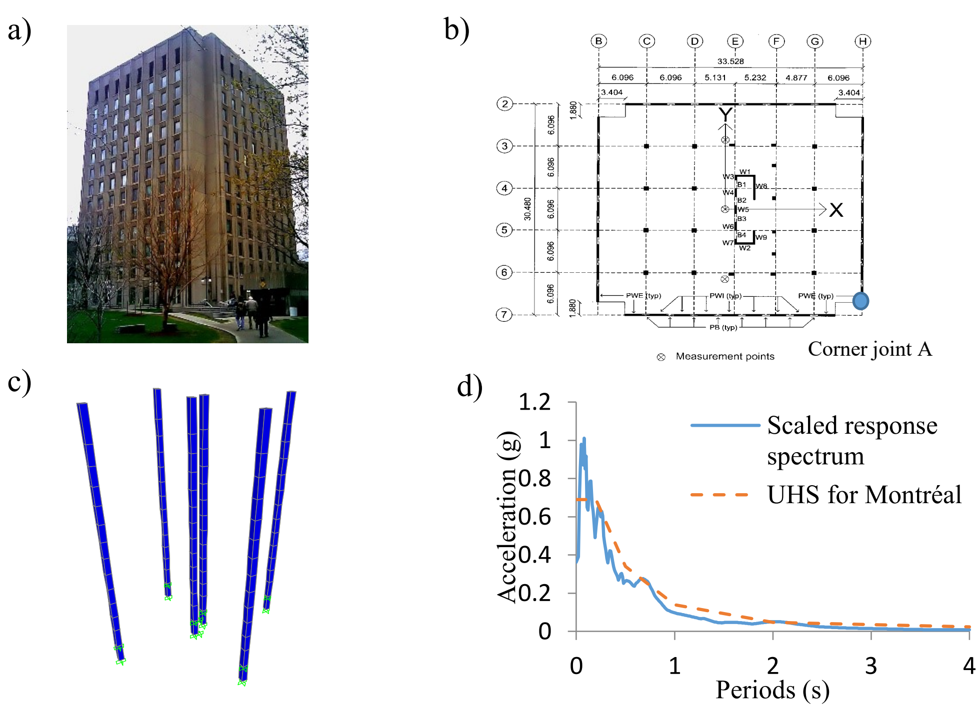

Figure 7.1 a) 3D view of the building; b) a typical floor plan and sensor positions; c) 3D view of the equivalent FE model; d) response spectrum of the input ground motion and UHS for Montreal

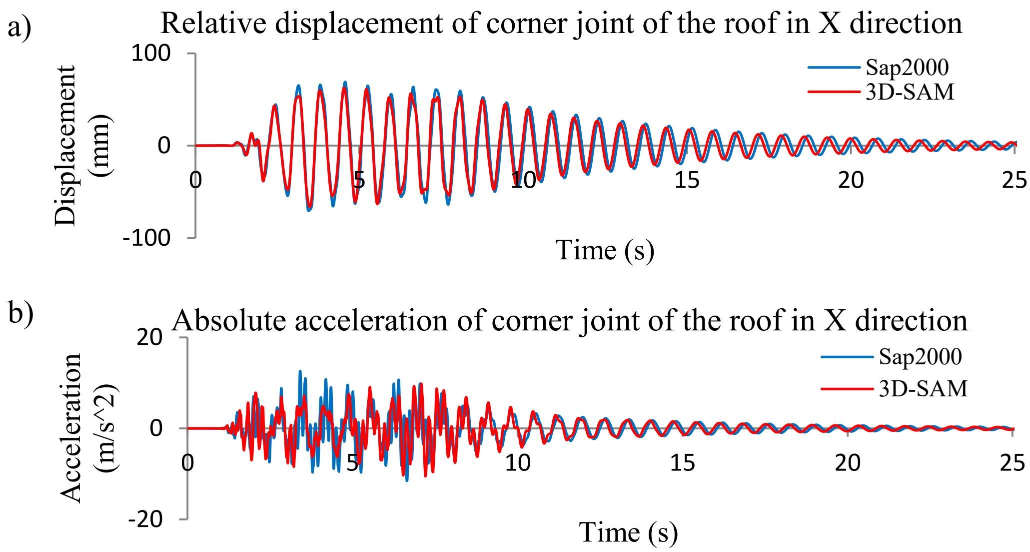

Figure 7.2 Comparison between response history analyses of the roof corner joint A by different methods-earthquake is applied in X direction

7.4 Modified 3D-SAM for higher amplitude motion

Because of the low amplitude range of ambient vibrations (PGA<10^{-5}g), some of the dynamic properties obtained from weak-motion are generally expected to be different from those obtained using strong-motion (PGA>0.1g). This difference has been observed between the ambient vibrations and seismic ground motions (Dunand et al. 2006) and is mostly linked to variations in natural frequencies and damping levels. The change in modal characteristics and wandering of natural frequencies were even observed in undamaged structures (not visible damage) subjected to strong motion (Celebi 2007). The trend is for natural frequencies to decrease and damping ratios to increase with seismic intensity, whereas mode shapes are not significantly affected. To have a correct prediction of the linear response of a structure subjected to a strong earthquake there is a need to have its modal properties for higher vibration levels corresponding to the state of the structure prior to yielding happens. Appropriate modification factors can be derived to relate low vibration modal properties to the properties at higher vibration levels. Such modification factors can be derived based on observations of buildings equipped with permanent strong-motion instrumentation where the building has not suffered from visible structural damage during a strong excitation. After careful review of data for 18 such buildings available in the literature (Dunand et al. 2006, Celebi 2007 and 2009, Carreno and Boroschek 2011), the following observations were made:

1) the strong-motion modal frequencies are decreased by a maximum of 30% and 40% of the corresponding values extracted from ambient vibration records for steel and concrete buildings, respectively;
2) the mode shapes have not changed from ambient to the strong vibration levels (before the occurrence of damage);
3) the overall internal damping observe in for strong-motion response can be 2 to 4 times larger than in ambient measurements.

Based on these general observations, one can apply the appropriate modification factors to the AVT modal properties before inputting them in the 3D-SAM procedure. According to the performance-based design concept and to remain conservative in the assessment of building displacements and drifts, it is suggested that the natural frequencies be decreased by 30% and 40% for steel and concrete buildings, respectively, and that internal damping ratios be multiplied by two. Moreover, the prediction of non-linear seismic demands using linear analysis has been widely used for seismic design as prescribed in codes and for vulnerability assessment. To obtain the best representative linear system at higher shaking levels, the modified modal properties based on the increased natural period and damping ratio suggested above should be used. Therefore, the 3D-SAM application range can further be expanded by use of the equal displacement rule (EDR).

7.5 Application of 3D-SAM to four case studies

7.5.1 General

In each case study, ten synthetic records (Figure 7.3) are applied to cover the entire frequency range of interest, PGA, magnitude, epicentral distance and duration. The generated records were also scaled up or down to match the UHS of Montreal as closely as possible in different ranges of periods.

The global seismic demand parameters are calculated for all selected ground motions as well as their mean values and standard deviations.

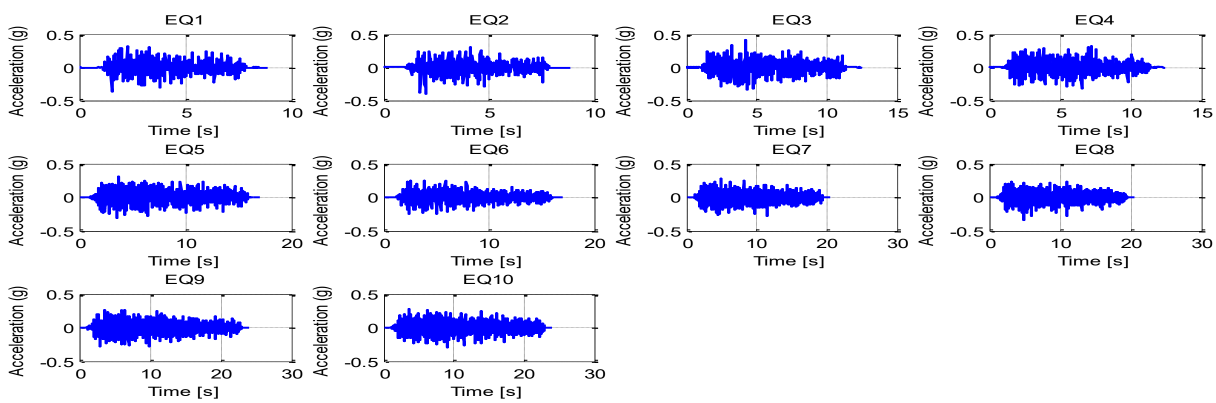

Figure 7.3 Ten earthquake records compatible with UHS for Montreal, Canada

7.5.2 Burnside building

This is the same building used in Section 7.3.2 for verification of the 3D-SAM. Figure 7.4 shows some of the important seismic demands obtained from the modified 3D-SAM method when earthquakes are applied in Y direction. Red circles on mean ± sigma graphs show the location of the floors along building height except for shear forces, which are given at half-story heights.

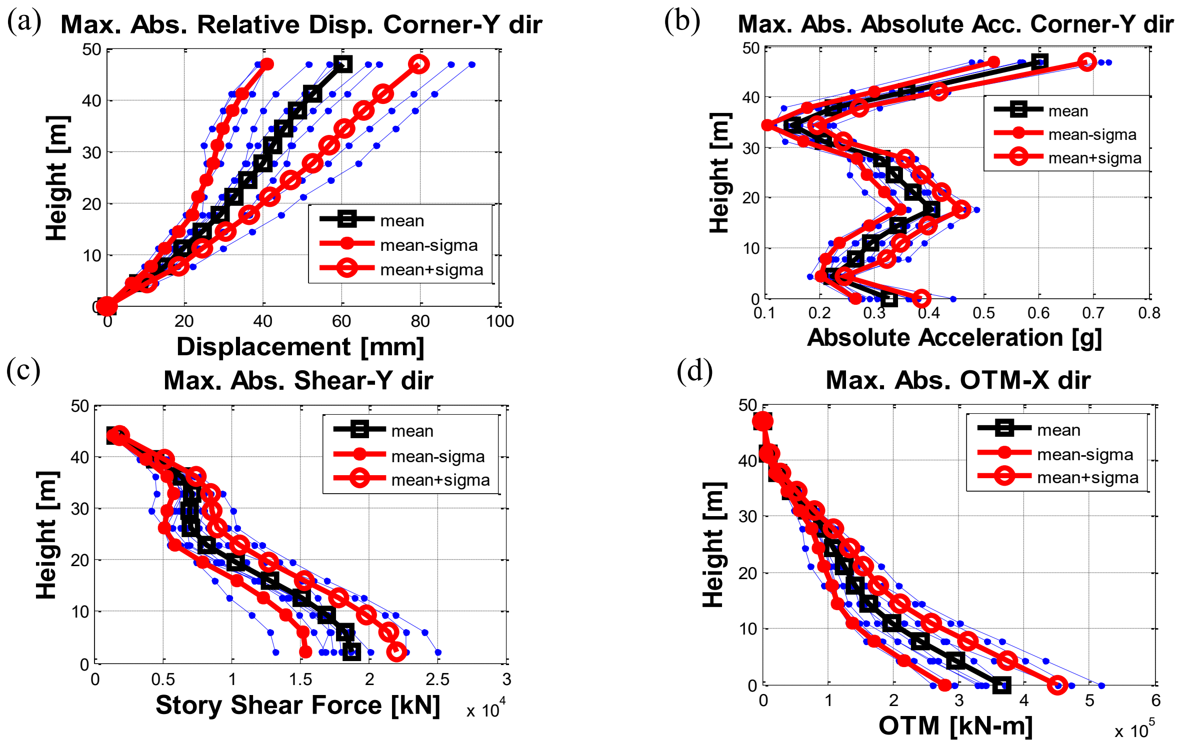

Figure 7.4 Maximum seismic demands from 3D-SAM method for Burnside building

7.5.3 Centre du Plateau

This building was constructed in 1961 with a reinforced concrete moment frame structural system and height of about 13.1 m including one basement floor, 8.4 m above the ground level. A bird's eye view of the building, its floor plans and sensor positions (black dots) are shown in Figure 7.5. The building's modal characteristics derived from AVT and analyzed by ARTeMIS™ software are shown in Figure 7.6. Then the building is subjected to the ten earthquakes and analyzed by means of the modified 3D-SAM method (Y is aligned N.S. dir., X is aligned E.W. dir. and earthquakes are applied in north-south direction). Figure 7.7 shows some of the important seismic demands. The accelerations are calculated in both X and Y directions. By providing the acceleration, response spectra at each floor can be derived and non-structural components can be assessed. Also, with the knowledge of drift ratios between adjacent floors, the building performance and damage state can be predicted. Moreover, in the case of an irregular building 3D-

SAM can calculate the dynamic amplification portion of natural torsion on each floor. From Figure 7.7d it is seen that the dynamic amplification of natural torsion is equivalent to an eccentricity of 10% for this building.

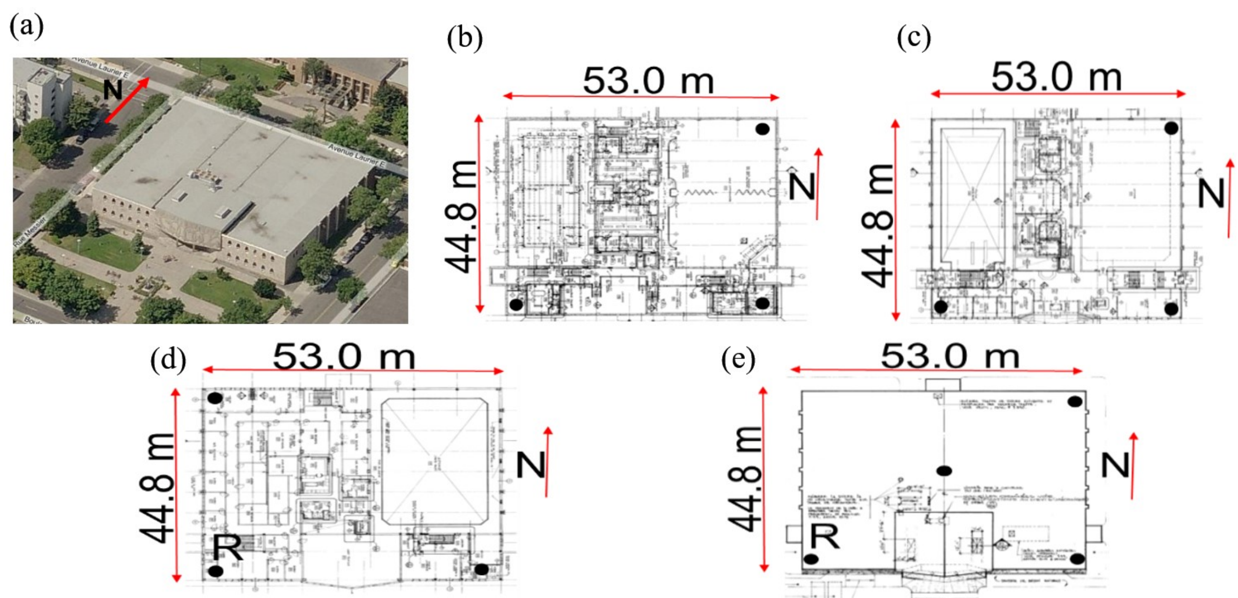

Figure 7.5 (a) Bird's eye view; (b) Basement - 4.7 m below ground level; (c) Ground floor; (d) 1st floor-height above ground 4.2 m; (e) Roof - height above ground 8.4 m

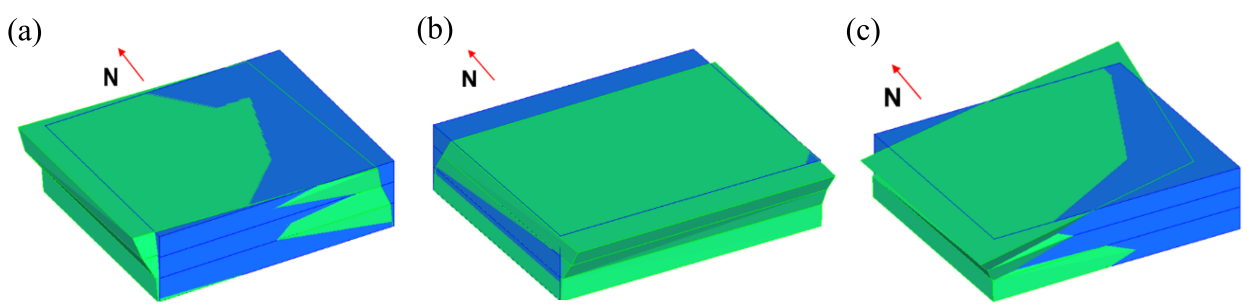

Figure 7.6 Mode shapes a) 1st flexural mode N-S dir. (0.23 s, damping ratio=0.017); b) 1st flexural-torsional mode E-W dir. (0.21 s, damping ratio=0.017); c) 1st torsional mode (0.16 s, damping ratio=0.033)

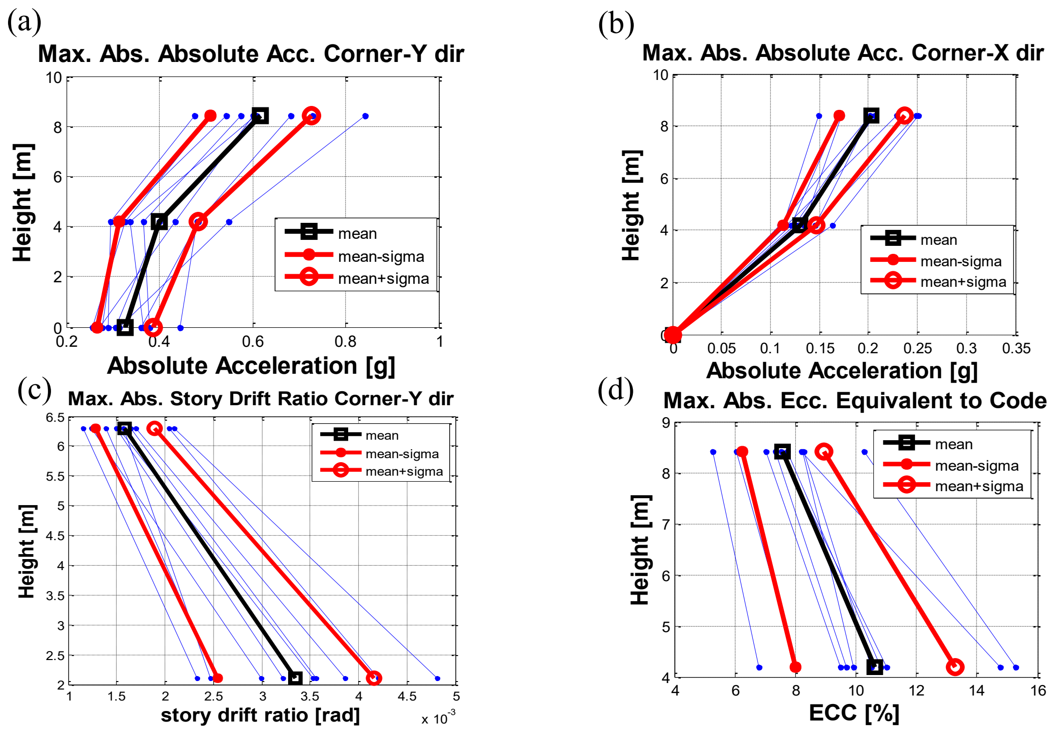

Figure 7.7 Maximum seismic demands from 3D-SAM method for Centre du Plateau

7.5.4 Centre Roger Rousseau

This community centre is a single building constructed in 1976 with braced steel frame structural system and height of about 7.9 m above the ground level. A bird’s eye view of the building, a typical floor plan and sensor locations, and the four derived modal properties from AVT are shown in Figure 7.8.

The building is subjected to the ten earthquakes in NS direction and analyzed with the modified 3D-SAM method; some of the derived seismic demands are shown in Figure 7.9.

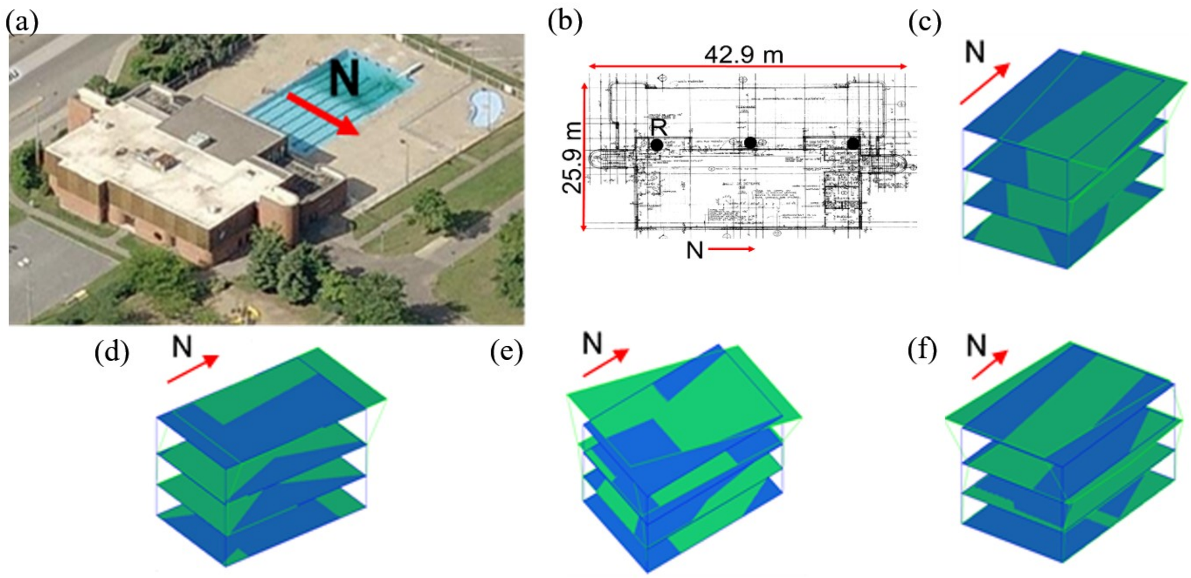

Figure 7.8 (a) Bird's eye view; (b) Typical floor plan and sensor positioning; (c) 1st flexural-torsional mode E-W dir. (0.18 s, damping ratio=0.06); (d) 1st flexural-torsional mode N-S dir. (0.13 s, damping ratio=0.02); (e) 1st torsional mode (0.09 s, damping ratio=0.016); (f) 2nd flexural-torsional mode E-W dir. (0.08, damping ratio=0.01)

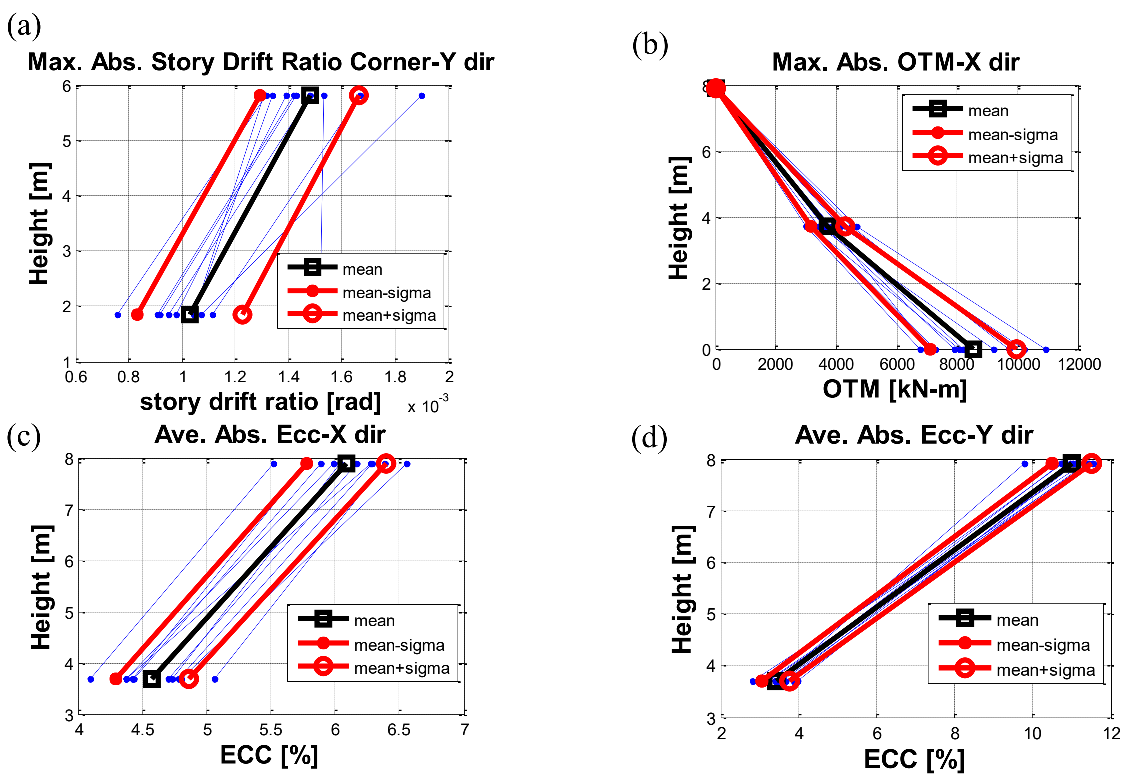

Figure 7.9 Maximum seismic demands from 3D-SAM method for Centre Roger Rousseau

7.5.5 Centre Roussin

This community center was constructed in 1964 with reinforced concrete moment frame and height of 17.1 m including one basement floor, and 13 m above the ground level. A bird's eye view of the building, a typical floor plan and sensor locations, and the derived 6 modes from AVT are shown in Figure 7.10. Some of the important seismic demands of the building derived by modified 3D-SAM are shown in Figure 7.11.

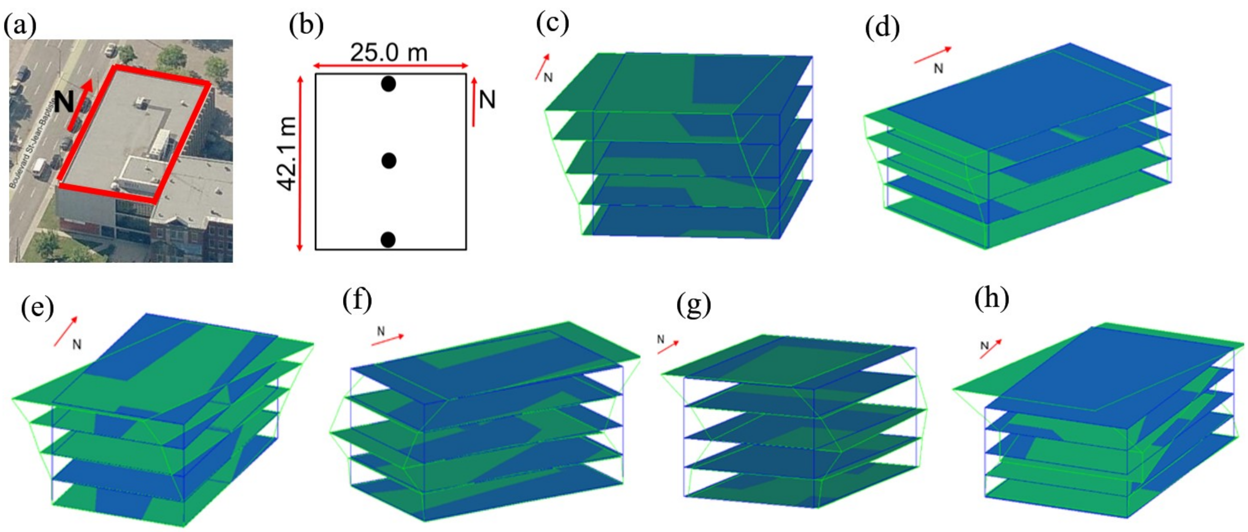

Figure 7.10 (a) Bird's eye view; (b) A typical floor plan; (c) 1st flexural-torsional mode E-W dir. (0.38 s, damping ratio=0.041); (d) 1st flexural mode N-S dir. (0.38 s, damping ratio=0.040); (e) 1st torsional mode (0.23 s, damping ratio=0.030); (f) 2nd flexural mode N-S dr. (0.13 s, damping ratio=0.020); (g) 2nd flexural mode E-W dir. (0.12 s, damping ratio=0.023); (h) 2nd torsional mode (0.1 s, damping ratio=0.010)

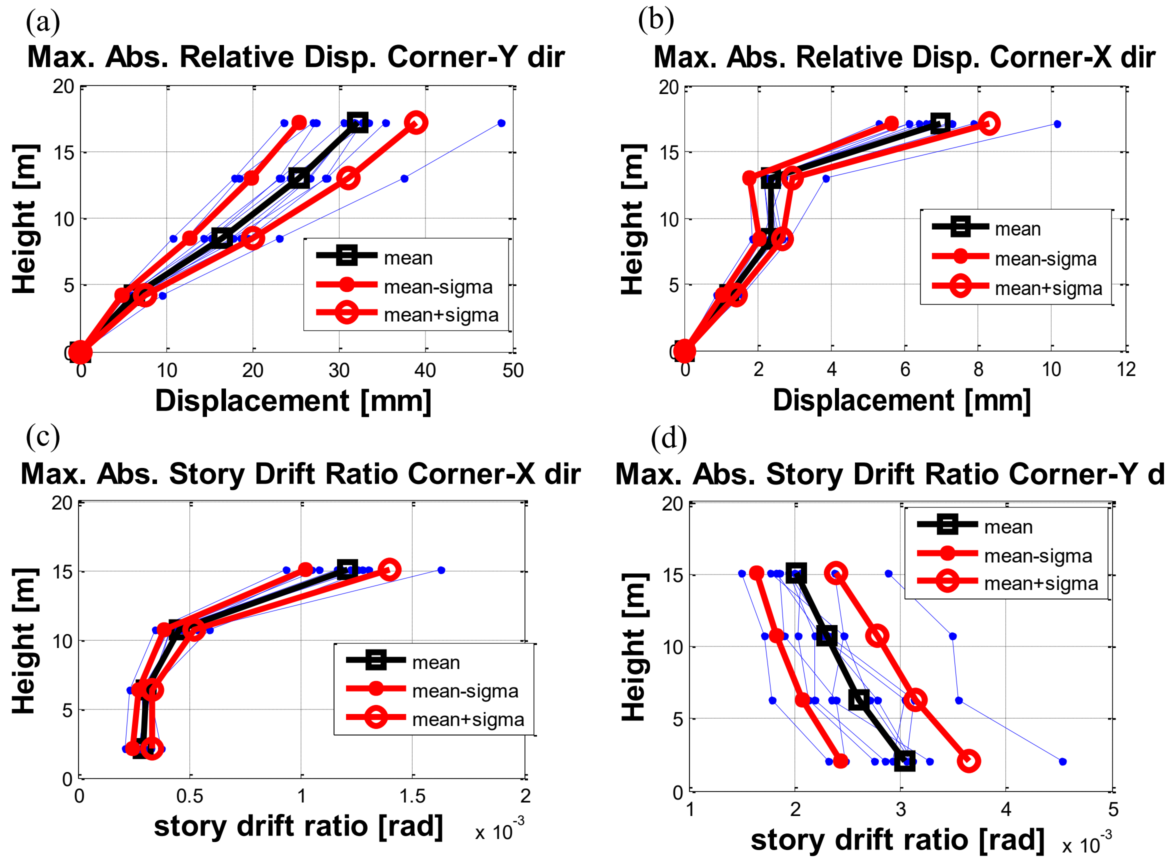

Figure 7.11 Maximum seismic demands from 3D-SAM method at a corner joint for Centre Roussin

7.6 Conclusion

In this paper a simplified 3-dimensional seismic assessment method directly based on ambient vibration testing, 3D-SAM, is briefly presented and verified for a high rise building located in Montreal. The 3D-SAM methodology and software calculate response histories of relative displacements and absolute accelerations at any location and direction on the building floor as well as the following seismic demands: maximum relative floor displacements, story drift ratios, floor absolute accelerations, story shear forces, overturning moments and dynamic amplification portion of natural torsions. All these demand parameters are calculated for any selected number of ground motions (can be applied in any direction) along with their mean and standard deviations. The calculated absolute accelerations on each floor can be used to find response spectra that lead to prediction of non-structural component seismic performance. Moreover, drift ratios and displacements of the corner joints can be a good indicator of the building performance for an eventual design-level earthquake and the subsequent damage states. Application of the 3D-SAM was demonstrated with four post-disaster buildings located in Montreal. Moreover, appropriate modification factors for further expansion of the 3D-SAM application from weak to the stronger ground motions were proposed. It is emphasized that the method does not require the creation of any detailed FE model and is solely based on modal properties of the current condition of buildings derived from ambient vibration tests. The 3D-SAM method is a more efficient and accurate tool for building seismic response prediction if compared to the current use of linear calibrated finite element methods based on experimental modal analysis. The method calculates the seismic demands directly from experimental modal characteristics of the building.

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8. Conclusions and future work

8.1 Conclusions

This research has introduced a novel methodology for the three-dimensional seismic assessment of existing buildings directly based on the dynamic building characteristics extracted from ambient vibration tests. A set of sixteen low and mid-rise irregular and two high-rise buildings located in Montreal, Canada were subjected to ambient vibration tests (AVT) and this data base was used to validate and demonstrate the reliability of the methodology. The following conclusions can be drawn from this study:

1) Ambient vibration test showed its capability to derive at least the lowest three modal properties of irregular low-rise buildings. The practical experience gained in these tests showed how increasing the sampling frequency, number of sensors, record duration of ambient vibrations as well as using two reference sensors (instead of a single one) in test set-ups and decimation during the analysis phase, can significantly improve modal identification. Moreover, in the case of reinforced concrete moment frames, results showed that the fundamental periods derived from AVT are much shorter than the NBCC 2010 building code formula periods, while for braced steel frames the AVT results are in agreement with the NBCC 2010 code formula results. According to the data collected, the mean values of equivalent viscous damping ratios for reinforced concrete and braced steel structures were 2.5±0.9 and 3.5±1.7 percentage of the critical damping, respectively.

2) The proposed three-dimensional seismic assessment method, 3D-SAM, is applicable to buildings with rigid floor/roof diaphragms. When the method was applied to four of the tested buildings, it was shown to be a more efficient and accurate tool for seismic response prediction than the current practice of "updated linear finite element methods based on experimental modal analysis". Because the method calculates the demands directly from the AVT results, it is faster, more precise, and robust. In fact, the proposed method represents the equivalent of an ideal 100% calibrated finite element model for linear dynamic analysis. It is emphasized that this simplified method does not require creation of any theoretical FE model and incorporate torsional effects in predicting response therefore is ideal to be used to assess existing buildings, and in particular older buildings with low quality structural drawings, or to perform city-scale seismic assessment of buildings.

3) After careful literature review of data collected on buildings with permanently instrumented sensors that had been subjected to moderate to strong earthquakes but suffered no significant damage, the following conclusions were made: (1) the strong-motion modal frequencies are decreased by a maximum of 30% and 40% of the corresponding values extracted from ambient vibration records for steel and concrete buildings, respectively; (2) the mode shapes are not changed from ambient to strong vibration levels (before the occurrence of damage); (3) the internal damping ratio for strong-motion response can be as much as 2 to 4 times larger than those found using ambient measurements. Two approaches were proposed to consider these effects in the method: (1) to get a conservative estimate of drift ratios and displacement demands, the damping ratios derived from ambient vibration records can be multiplied by the factor two, mode shapes can remain unchanged and finally natural frequencies can be reduced to the maximum of 30% and 40% of the corresponding values

extracted from ambient vibration records for steel and concrete buildings, respectively. (2) the damping ratios derived from ambient vibration records can be multiplied by the factor two, mode shapes can remain unchanged, however, the response history analysis is re-run each time by different natural frequency reduction factors of 10%, 20%, 30% (and 40% in case of concrete structures) and then use the mean plus one standard deviation of all the results as the conservative outcome for drift ratios and displacement demands. After having a reasonable prediction of the dynamic properties of the building before yielding point, the 3D-SAM provides reasonable estimate of displacement and drift ratios to strong earthquake excitations. More precisely, this includes buildings with low to moderate ductility (ductility factor less than 2) and buildings with natural frequencies under 2Hz.

4) The 16 irregular buildings tested by AVT were analyzed by 3D-SAM and additional eccentricities equivalent to dynamic amplification portion of natural torsion were reported on each floor. Mean value and mean plus one standard deviation of these equivalent eccentricities that can represent the existing dynamic torque on C.M, by shifting inertia force from C.M, were shown to be almost 13% and 20%, respectively, for the whole dataset. Therefore, the proposed method can provide insight into the complex parameter of torsion and also these results show that to perform seismic assessment of irregular buildings it is necessary to use simplified methods that consider torsional effects.

8.2 Suggestions for future work

Considering the limitations of the method developed in this research and the practical results presented, the following areas could be investigated in more depth:

1) With the availability of more data from permanently instrumented buildings subjected to earthquakes, the modification factors to relate ambient vibration dynamic properties to stronger excitation properties can be refined and then used in 3D-SAM.
2) More research can be done on the relation between linear and nonlinear demands so that results obtained with the 3D-SAM method can be extended to buildings that may suffer severe damage during an earthquake.
3) 3D-SAM predictions can be compared with the response of buildings subjected to earthquakes, (assuming some AVT data is available for these buildings before the earthquake occurs), to better understand and quantify the range of applicability of the method and the adequacy of the modification factors implemented to extend this range.
4) As the proposed method can provide acceleration and drift ratios for any location and direction on building platforms, it becomes an attractive tool to generate floor response spectra and assess the seismic response of non-structural components and equipment inside a building. Such work is currently under way by a colleague PhD student at McGill.
5) Similar work can be carried out on buildings located in other cities and areas of different seismicity to get estimates of the dynamic amplification portion of natural torsion. This type of research can provide insight through this complex parameter that has not been extensively studied yet.