

Article

Governing Lateral Load on Tall Buildings in Canadian Regions

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Abstract: Urbanization has led to a significant increase in the construction of tall buildings in Canada. The design of tall buildings must ensure structural integrity in withstanding lateral loads, such as wind and earthquakes. The tendency for a specific lateral load to govern building design varies based on the building characteristics, building height, and location of the building. There is a need to identify the governing lateral load (i.e., wind or earthquake) for use in the preliminary design and city-scale assessment. This study examines the governing lateral loads for tall buildings across different Canadian regions through a parametric analysis of a typical high-rise building based on the Commonwealth Advisory Aeronautical Council (CAARC) building. This research evaluates varying building heights, structural systems, and geographic locations under the guidelines of the National Building Code of Canada (NBCC). The analysis identifies the dominant lateral load, providing insights into assessing the existing infrastructure and optimal design strategies for enhancing building sustainability and resilience. Our findings highlight the critical role of geographic location in determining lateral load impacts and the necessity of context-specific design to promote long-term structural performance and environmental sustainability.

Keywords: tall buildings; lateral load; sustainable design; urban infrastructure; wind load; earthquake load; finite element analysis



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1. Introduction

Migration to metropolitan locations, known as urbanization, has increased rapidly over the past century. Currently, over 80% of Canada's population resides in urban areas, a value only expected to increase [1]. There are many factors promoting urbanization, including how technological advancements have drastically changed the nature of jobs available—away from labor-intensive rural jobs such as agriculture and toward more urban, service-sector jobs [2]. Additionally, the desire to limit urban sprawl encourages the preservation of natural environments and wildlife while also mitigating the increased infrastructure costs required to develop in new areas. In Ontario, for example, the Provincial Growth Plan for the Greater Golden Horseshoe requires that a minimum of 50% of all residential development must occur within delineated built-up areas [3]. This represents one of the many decisions made with infrastructure sustainability and resilience in mind in developing sustainable urban practices [4,5].

To avoid urban sprawl while also meeting the needs of an increasing population, high-rise buildings are a popular solution in areas with restricted real estate availability, as they can greatly increase population density. This has caused tall buildings to become increasingly prevalent in Canadian cities: currently, tall buildings represent approximately 10.66% of all residential structures per capita, with a growth rate that surpassed all other structure types between 2016 and 2021 [6]. However, designing tall structures brings its own challenges. Canada's wide range of physiographic regions presents structures with unique combinations of wind and seismic hazards that manifest as lateral loads. The design of tall structures has become increasingly complex as wind and seismic loads have a profound effect on the lateral behavior of a building, especially since they are often

categorized as dynamically sensitive [7]. Additionally, the design and analysis of these structures are further complicated as the introduction of innovative materials and structural systems are being implemented for sustainable development [8].

All structures in Canada require specifically designed lateral force resisting systems (LFRS) to withstand wind and seismic loads [9]. Wind and seismic designs differ in nature, as they are predominantly reliant on pressure and inertial forces, respectively [7,10]. Additionally, the governing load case typically considers either wind or seismic as the governing load since the two events are considered mutually exclusive [11]. Furthermore, it is possible for one of the lateral hazards to govern the shear while the other hazard governs the bending moment design. As such, the exact effects and design requirements of lateral load on a building are unclear during the early stages of project design and in assessing existing structures. This classification is essential in the preliminary design stage, as considering appropriate structural systems and identifying additional criteria, such as the seismic ductility level, need to be chosen. Current methods require an iterative process of designing the structure, checking the ductility requirements, and then repeating the process until all requirements are met. Identifying lateral load hazards for existing structures is a challenging process, as there is no guarantee that prior design information and assumptions are available and that the structure was designed to up-to-date code provisions.

Additionally, the prevalence of tall structures in modern life has driven the evolution of engineering design methods and has led to the development of advanced design considerations that combine several ideologies, such as reliability-based, performance-based, and multihazard design [11–14]. Frameworks proposed for these methods can utilize dynamic analysis, fragility assessments, life-cycle cost assessments, and probabilistic failure criteria to meet specific building performances, such as peak inter-storey drift and horizontal acceleration, while also allowing for controlled structural inelasticity [14]. These advanced methods achieve more economical building designs while maintaining the safety and performance criteria of the currently prescribed capacity-based standards. The emerging area of multihazard design considers the sequential effects of several natural hazards and provides provisions to manage the risk and economic impacts of multihazard effects on buildings using a holistic approach [15,16]. However, the effectiveness of this approach is dependent on the inherent presence and magnitude of several hazards, namely wind and seismic, in this case. The need to identify the governing lateral load acting on tall buildings in this respect was first showcased in the conference paper under the same title [17]. The study previously analyzed only the shear load demand on tall structures and found that both wind and seismic loads could govern certain locations. This paper expands the analysis to include the governing bending moment and conduct a more thorough assessment. Knowing the governing lateral load prior to detailed design is useful during the early stages of a project, for example, since lateral load mitigation techniques often depend on the nature of the governing load.

This paper is organized into five sections. Section 1 (this section) introduces the need for preliminary predictions of the governing lateral load of a tall building. Section 2 provides an overview of the problem description used for determining the governing lateral load, and outlines the finite element analysis (FEA) modeling process utilized. Section 3 presents the results and discussion of the findings, and Section 4 concludes the study, providing directions for future research that are potentially worth investigating.

2. Materials and Methods

The parametric study features models of similarly designed buildings consisting of 5 cities, 4 building heights, 3 seismic site classes, and 2 LFRS, as summarized in Figure 1. These parameters were carefully selected to ensure numerical variance in the cases assessed. The combination of the parameters identified and shown in Figure 1 results in 120 model cases. In selecting the 5 locations to be assessed, preference was given to identifying cities with substantial variance in the following three categories: spatial location, characteristic wind load, and seismic spectra. The definition of a tall building can vary qualitatively, as a

single mid-rise in a suburban neighborhood can be portrayed as a tall building. Regarding the 4 building heights utilized for the parametric analysis, the heights considered range from 60 m to 180 m, all of which we consider to be tall buildings from an engineering perspective. Three site classes represent the extremes of potential soil conditions and their effect on seismic load, and two common structural systems have been considered: concentrically braced frames and moment-resisting frames. Significant variability in the study results is expected to be obtained by combining these criteria.



Figure 1. Model Cases.

To maintain consistency between cases, certain principles, such as symmetry and uniformity, were applied to the building design. These principles restricted the development of undesirable effects such as torsion and nonuniformity in building stiffness. For example, beam sections are kept constant across each storey to prevent unnecessary variance in stiffness. This principle was also applied to the symmetric selection of columns and braces, where appropriate. Both the base shear and base moment of each model were investigated to assess which lateral load governs the design.

To determine the governing lateral load, this paper introduces two Lateral Load Ratios (LLR), one defined as the ratio of wind-induced base shear to seismic-induced base shear, as shown in Equation (1). Additionally, the second LLR is defined as the ratio of the wind-induced base moment to the seismic-induced base moment, as shown in Equation (2):

$$LLR_V = \frac{Base\ Shear_{Wind}}{Base\ Shear_{Seismic}} \quad (1)$$

$$LLR_M = \frac{Base\ Moment_{Wind}}{Base\ Moment_{Seismic}} \quad (2)$$

The LLR is used to indicate the governing lateral load of a building; values greater than 1.0 indicate the design is governed by wind, whereas values less than 1.0 indicate the design is governed by seismicity. It is important to note that there exist several types of mitigation techniques used in tall building design, which influence the LLR [16,17]. Mitigation techniques consist of approaches such as load reduction and optimization. Namely, aerodynamic optimization, base isolation systems, and various types of dampers are all examples of techniques used to improve building performance. However, many techniques are naturally only effective for a certain lateral load type, and so this research purposefully does not make use of these techniques for modeling purposes to quantify a high-level and unmitigated LLR.

Each design case followed the framework outlined in Figure 2, based on NBCC procedures. The remainder of this section further outlines each component seen in the dominant lateral load determination framework. To quantify the lateral load demand, linear static analysis techniques are utilized, as per the NBCC. The lateral load demands are then compared, as discussed in the following subsection, regarding the definition and application of the LLR. In determining the model geometry, the conceptually designed models closely resemble the CAARC building. The models consist of repeating modules of 5 m × 5 m reinforced concrete slabs supported by steel beams. The modules are repeated in a 6 × 9 arrangement to form a total floor area of 30 m × 45 m. A conceptual diagram of the models developed can be seen in Figure 3. Determining the case parameters are as described surrounding the commentary in Figure 1, and the detailed study parameters are outlined in detail in Table 1 below.

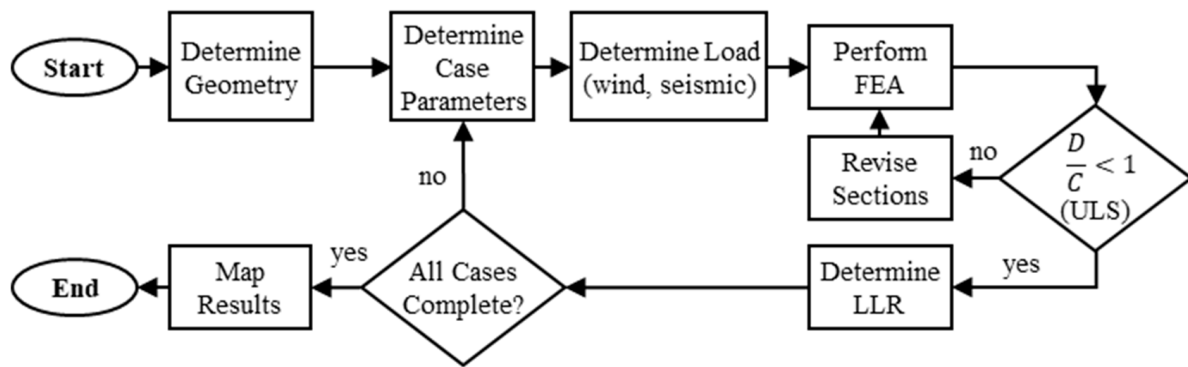


Figure 2. Dominant lateral load determination framework.

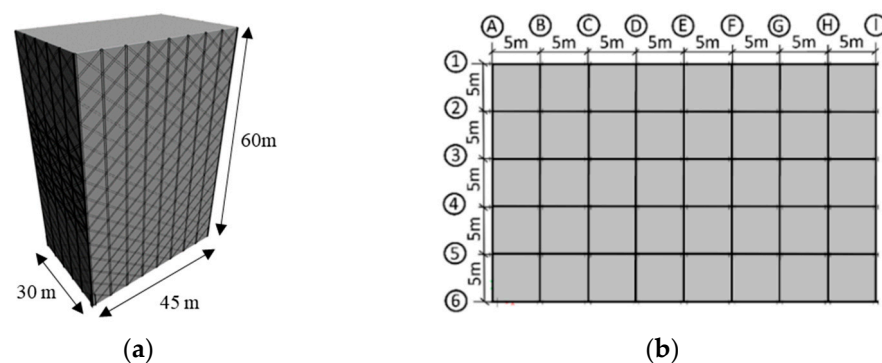


Figure 3. (a) Conceptual render of a 60 m CBF model (b) Model floor plan.

Table 1. Study parameters.

Location	Building Height	Site Class	LFRS
Moncton (M)	180 m	A	Concentrically Braced Frame (CBF)
St. John's (SJ)	140 m	C	
Toronto (T)	100 m	E	
Vancouver (V)	60 m		Moment-Resisting Frame (MF)
Whitehorse (W)			

The models follow a naming convention of the city, building height, LFRS, and seismic site class. For example, a particular model case would be a 180-meter-tall building located in Vancouver, BC, with a seismic site class ‘C’ and constructed with concentrically braced frames. The model’s name under the abbreviation scheme used is “V_180 m_CBF_C”. It is important to note that only 8 models consisted of unique design sections, which were then duplicated across multiple hazard cases to promote consistency in results. The unique designs were comprised of buildings of all 4 heights for both CBF and MRF LFRS. The geometric designs of both the CBF and MRF models are displayed in Figure 3.

Dead, live, wind, and earthquake loads were considered when assessing the models. Regarding the gravity load, the self-weight of the structure superimposed dead load and weighted average live load were used. A conservative allowance of 3.0 kPa was allocated as a superimposed dead load to account for partition walls and various finishings. Additionally, based on use and occupancy, a weighted average live load of 2.25 kPa was also applied to the structure. These loads were applied to the structure, considering the appropriate load combinations as per the NBCC.

Regarding the lateral load on the structure, linear static analysis methods were utilized for both wind and seismic to determine the forces applied to the building. The static procedure defined by the NBCC was used for wind load analysis, where the external pressure due to wind was determined using Equation (3) [18]. The seismic force was also determined as per the NBCC, using the equivalent static force procedure, as seen in Equation (4). Further explanation of these procedures can be found in Section 4 of the NBCC.

$$p = I_w q C_e C_t C_g C_p \quad (3)$$

where: p = specified external pressure, I_w = importance factor for wind load, q = reference velocity pressure, C_e = exposure factor, C_t = topographic factor, C_g = gust effect factor, and C_p = external pressure coefficient.

$$V = \frac{S(T_a) M_v I_E W}{R_d R_o} \quad (4)$$

where: $S(T)$ = design spectral response acceleration, T_a = fundamental lateral period of vibration of the building, M_v = factor to account for higher mode effect, I_E = earthquake importance factor, W = dead load, R_d = ductility-related force modification factor, and R_o = over-strength-related force modification factor.

The important factors for both wind (I_w) and seismic (I_E) are considered normal and, therefore, equal to one. For wind design, the models correspond to flat and rough terrain buildings. The authors found that considering a single exposure coefficient was sufficient when applying linear static wind load analysis because, under this analysis, results corresponding to other conditions can be quickly assessed by directly multiplying by the ratio of effective change in exposure coefficient (C_{e2}/C_{e1}). Wind pressure was applied directly to the diaphragm of each storey. The analysis included wind angles of attack at both 0 and 90 degrees. For seismic design, the higher mode factor was assumed to be 1.0 in all cases, while ductility and over-strength factors are respective to the LFRS outlined in Table 1, as appropriate on a case-by-case basis. The spectral acceleration values were found by inputting the seismic site class, longitude, and latitude of the city hall for each location into the 2020 National Building Code of Canada Seismic Hazard Tool [19].

Using the geometry and applied load determined, finite element analysis (FEA) was used to analyze the structure, which is expanded upon in Section 3. After successfully designing the models, the base shear and base moment experienced by the structure were determined for the governing cases of both wind and seismic load combinations. The LLR calculations for each model were then conducted, as per Equations (1) and (2), to provide a clear interpretation of the governing load across model cases.

Structural analysis of the models is executed utilizing ETABS FEA software (version 18.1.1). The models are created using the CSA S16-14 steel design code and the corresponding CISC steel section database. Corresponding to the model cases established in Section 2, the building's layout, encompassing beams, columns, slabs, and bracing elements, is populated into ETABS. The analysis utilizes standard material properties, with steel members defined by ASTM A992, including a yield strength of 345 MPa (50 psi) and concrete elements having a compressive strength of 27.58 MPa (4000 psi) and a density of 23.6 kN/m³. Rigid diaphragms are defined at the storey level, and applied wind loads are assigned directly to rigid diaphragms at the storey level. Fixed support nodes are utilized at the ground level, with no consideration given to the potential for below-ground storeys and their effect on boundary conditions. Under the MRF models, beam elements are assumed to have rigid connections with no further detailed design afforded. In CBF models, beam elements are modeled with the appropriate end releases and tension/compression limits, corresponding to the ductility and over-strength factors utilized from Table 4.1.8.9. from the NBCC [18]. The loading conditions included the structure's self-weight and the applied dead and live loads specified in Section 2. The lateral load analysis is considered linear static and is applied through load patterns that consider the model's location. Load

combinations were implemented in accordance with the NBCC. Additionally, wind and earthquake loads are applied in both the x and y directions.

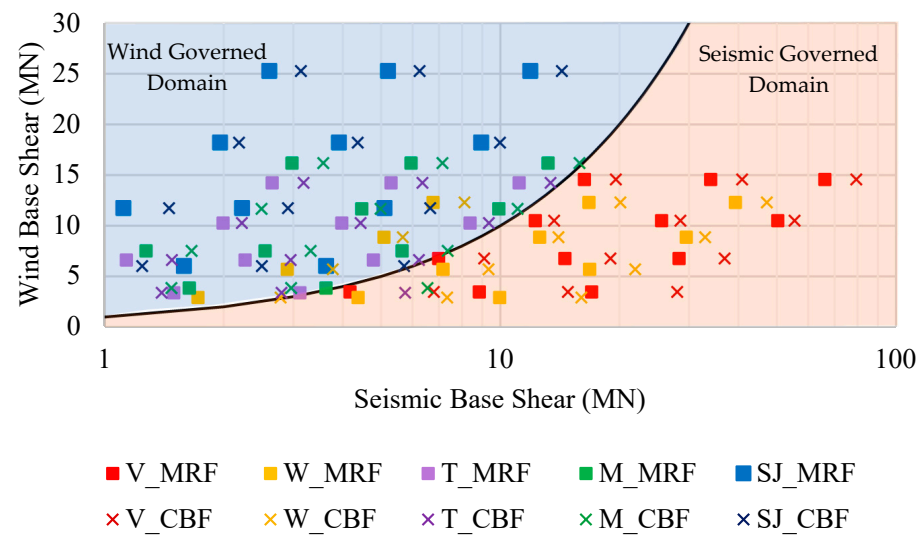
To maintain consistency between model cases, 8 unique models of unique design configurations represent the entire 120 models. The unique designs comprised buildings of all four total building heights, CBF and MRF LFRS. Models were each created with repeating storey geometry as identified in Figure 3, and section sizes were changed as appropriate. Modal analysis is conducted to determine the fundamental lateral period of the structure required during the calculation of lateral load as per the NBCC. After the analysis is completed, the demand-to-capacity ratio for all members is checked and is chosen to be below 0.95. Should members fail, design sections were increased to satisfy this requirement. Upon successful iterations, the resulting loads were recorded to determine the LLR. It should be noted that some of the models were not able to pass—particularly ones with a moment-resisting frame LFRS and heights of 140 m and 180 m. Generally, moment-resisting frames are not the sole LFRS in tall buildings. Due to the size of the members that would be required for the demand-to-capacity ratio to pass, it is not reasonably feasible. This is considered in the discussion toward the end of Section 4.

To form a basis for the validation of the models in this study, the building response of a 45-storey (180 m) version of the CAARC building was replicated and found to be in close agreement when compared against Chan et al. and Huang [14,20]. A validation model was created to replicate the structural system, load conditions, and analysis, finding overall agreement in the storey-response plots, including matching the maximum storey drift to within approximately 5% error. However, it is important to note that further comparison between the validation model and the model cases proposed in this study has limited benefit as there are substantial differences in analysis, load conditions, structural systems, and building geometry than what is proposed in this research, which deems direct comparison between the validation and study results difficult. However, equivalent design principles were applied to the models in this study and, therefore, were validated against the building response presented by Chan et al. and Huang.

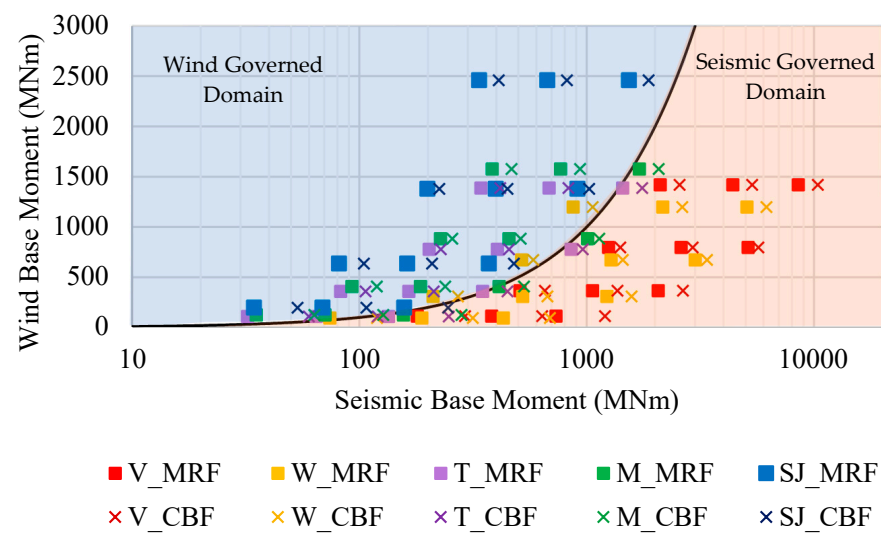
3. Results and Discussion

Analysis of the full dataset without categorization confirms that shear and moment designs are not equally impacted by each parameter, as seen when comparing Figure 4a,b. For shear, the LLR_V ranged from 0.124, indicating that seismicity governed, to 10.53, indicating that wind governed. Additionally, the wind load governed the shear load demand in 65% of all cases. In contrast, the LLR_M ranged from 0.095, indicating seismicity governed, to 7.83, indicating that wind governed. The wind load governed the bending moment load demand in only 51% of all cases. Furthermore, 9.2% of the LLR_V and 5.8% of the LLR_M were found to be between 0.90 and 1.1, indicating that the lateral load demand of both hazards was nearly equal. All but one of the cases with an LLR_V within this equal intensity range were classified as wind-governed, while all but one of the cases with an LLR_M within this range were classified as seismic governed. Curiously, none of the cases had both the LLR_V and LLR_M within the equal intensity range.

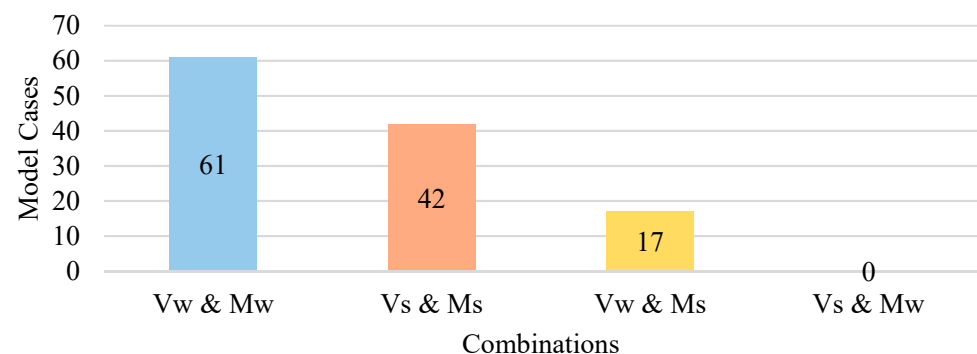
An interesting contrast between the behavior of the LLR due to shear compared to bending moment, as shown in Figure 4, is the noticeable inversion of predominant governing lateral load across all cases: 65% of cases are governed by wind loading regarding shear, while only 51% of cases are governed by wind loading regarding bending moment. This indicates that some cases are governed by different lateral loads for the shear and moment. The cases included are as follows: wind governs both shear and moment, seismic governs both shear and moment, wind governs shear, and seismic governs moment, and seismic governs shear while wind governs moment. To directly compare the relationships in which these occur, Figure 5 depicts the distribution of cases governed by the combinations.



(a) Wind vs Seismic Base Shear



(b) Wind vs Seismic Base Moment

Figure 4. Wind versus seismic for (a) Base Shear and (b) Base Moment.**Figure 5.** Shear and Moment Combinations of Governing Load Cases.

It can then be seen that 51% of the cases are governed entirely by wind load, 35% are governed entirely by seismic load, and 14% are governed by shear due to wind load and bending moment due to seismicity. Zero cases occurred where the seismic load governed

the structure's base shear while the wind load governed the bending moment. These results provide an interesting context about the behavior of the LLR in different cases. The results suggest that designs featuring nonuniform mass distribution and its effects on the building period can produce a distribution of inertial force, which diverges from the simplified single-degree-of-freedom seismic model. The cases governed purely by wind or purely by seismic allow for more direct design approaches catered to the governing load, whereas the cases in which both wind and seismic loads must simultaneously be accounted for may be indicative of more careful consideration in design. However, further analysis of the cases in which wind governs shear and seismicity governs the bending moment requires investigating the effects of the individual parameters of the study.

3.1. Impacts of Geographic Location

Geographic location was found to have the most significant effect on the lateral load demand due to the inherent variability in the presence and magnitude of a hazard in any given location. The five cities selected in the study each have a unique wind pressure associated with them, as per the NBCC climatic data. As such, considering the results by location also directly considers the effects of wind pressure on the building design. Figure 6 presents the governing loads of the model cases organized by location. The cities are organized from left to right in terms of low to high climatic wind pressure, corresponding to a range of 0.38 to 0.78 kPa.

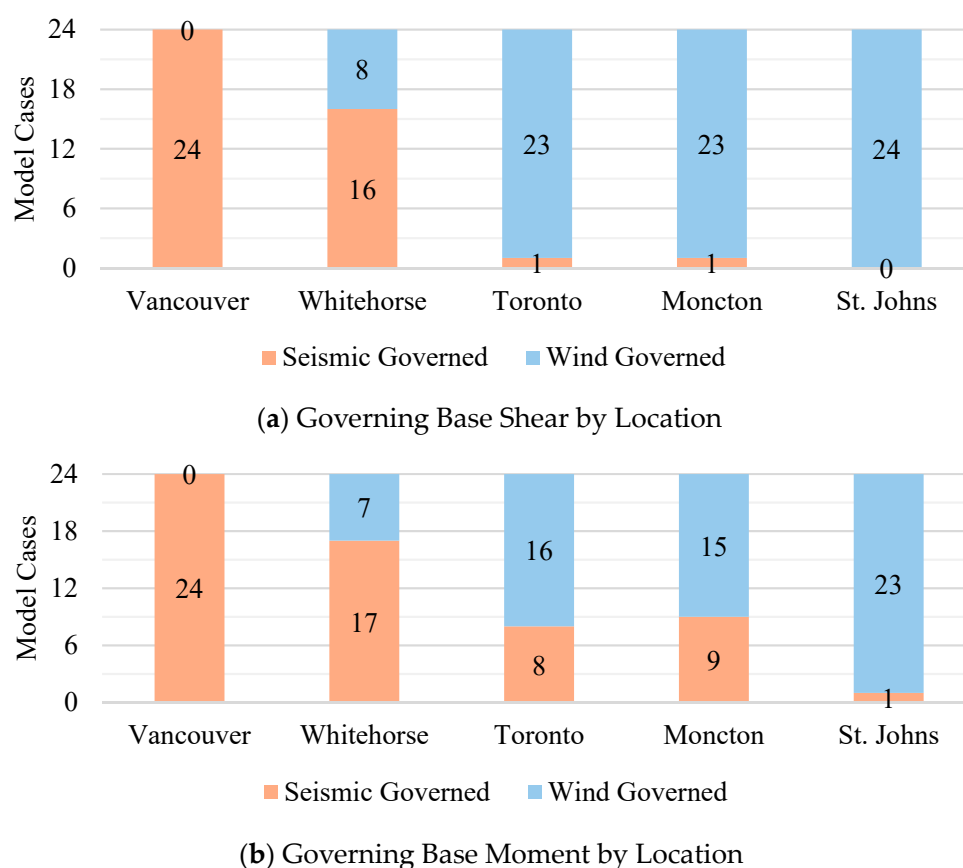


Figure 6. Governing force by location for (a) base shear and (b) base moment.

Comparing Figure 6a,b, it can be seen that the majority of the 14 cases with mixed governing loads (V_w & M_s) were found in Toronto and Moncton, with Whitehorse and St. Johns, each containing only one case. Additionally, of the cases with an LLR within the equal intensity range, Toronto and Moncton encompassed 73% of the cases for shear and 100% of the cases for the moment. Vancouver, Whitehorse, and St. Johns each encompassed

one of the remaining cases with an LLR_V within the equal intensity range. All cases within Vancouver were seismically governed, and the city represented the lower range of both LLRs, while almost all of St. Johns cases were wind-governed, and the city represented the higher range of both LLRs. Overall, variation in only the geographic location was found to magnify the LLR by up to 11.3 times (for both shear and moment) in the most extreme case.

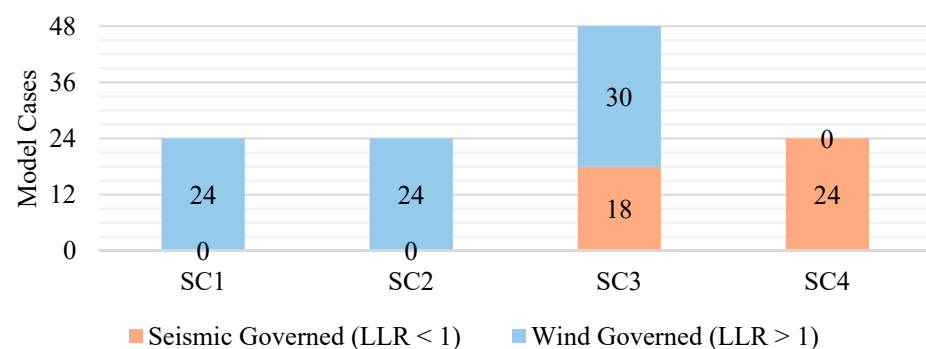
3.2. Impacts of Seismic Site Classification

Prior to conducting FEA, the seismic category (SC) of each of the models was determined. This is defined under NBCC clause 4.1.8.5 (2) and considers the importance factor of the building as well as the spectral acceleration, which is based on the location and seismic site classification. The seismic category determines the various limitations of the building (such as the ductility requirements), where SC1 is the lowest and SC4 is the most severe. Table 2 summarizes the seismic categories for each location and seismic site classification.

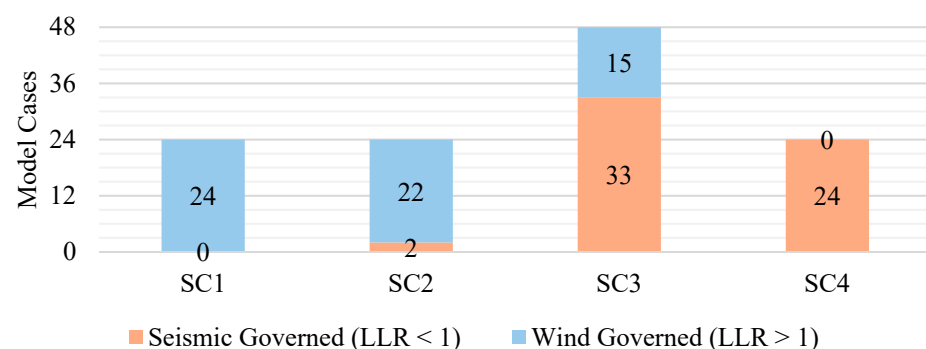
Table 2. Seismic category based on location and seismic site classification.

Location	A	C	E
Vancouver	SC3	SC4	SC4
Whitehorse	SC2	SC3	SC4
Toronto	SC2	SC3	SC3
Moncton	SC1	SC2	SC3
St. John's	SC1	SC1	SC3

As each location and site classification combination includes eight models, 48 of the models can be categorized under SC3, while 24 models are categorized under SC1, SC2, and SC4. Considering the results obtained from the FEM models, the model cases grouped by seismic category are displayed in Figure 7.



(a) Governing base shear by seismic category

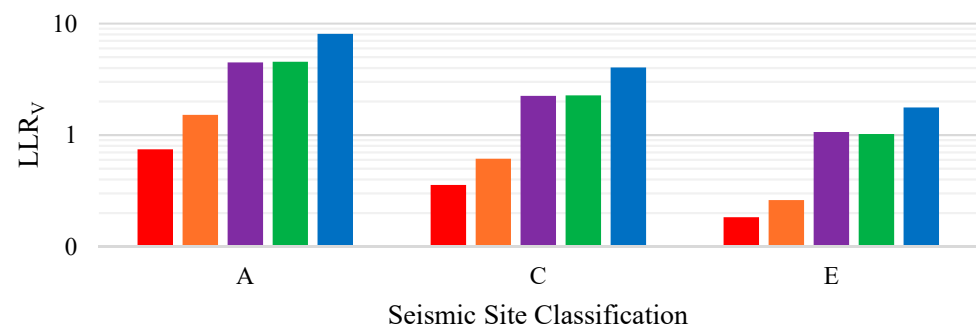


(b) Governing base moment by seismic category

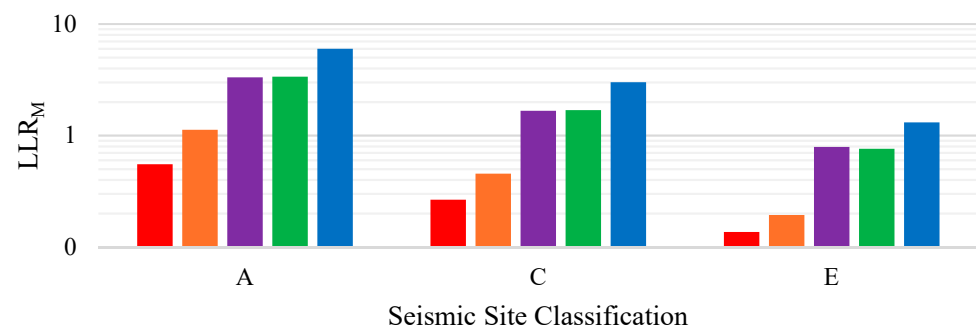
Figure 7. Governing force by seismic category for (a) base shear and (b) base moment.

From Figure 7, it can be observed that a higher seismic category corresponds to an increased likelihood of seismicity being the dominant lateral load. Of the 17 models with opposing dominant lateral loads for shear and moment design, two occur within SC2, and 15 occur within SC3. Additionally, of the models with LLRs within the equal intensity range, SC2 is found to encompass one case for shear and one case for moment, while SC3 encompasses the remaining ten cases for shear and six cases for moment. SC1 and SC4 are entirely wind- and seismic-dominant, respectively.

Figure 8 shows the LLR of equivalently designed 180-meter tall concentrically braced frames in the five cities selected in the stud and when grouped by seismic site classification. Site classification A is representative of hard rock, classification C is representative of dense soil, and classification E is representative of soft soils susceptible to seismic load. The seismic loads quantified are expected to have a relatively minor impact on class A scenarios and peak under class E scenarios. At first glance, it is clear that the LLR is largely dependent on the nature of the loads present in various cities. However, it can be observed that the seismic site class also plays a large role in scaling the magnitude of the LLR.



(a) Governing base shear by seismic site classification



(b) Governing base moment by seismic site classification

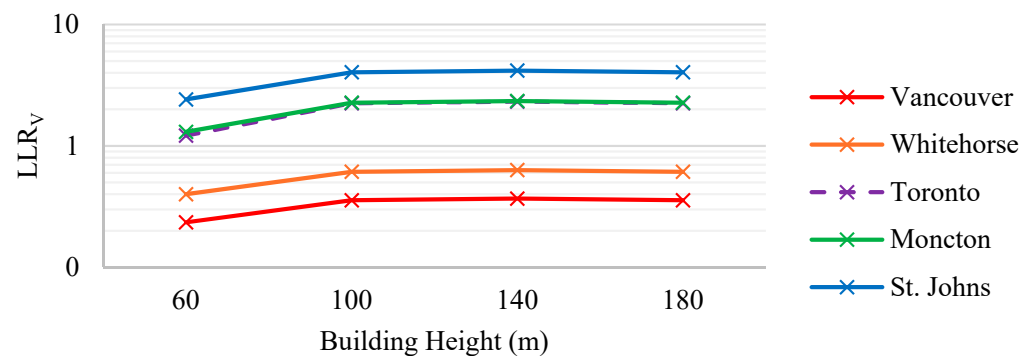
Figure 8. Comparison of (a) LLR_V and (b) LLR_M of different seismic site classifications (case: 180 m_CBF).

Since only models of one height and LFRS type are displayed in Figure 8, it should be noted that the trends observed are applicable under all cases. Considering seismic site classification is compared using equivalently designed buildings, any change in LLR would be due to the seismic load demand since the wind load demand between the cases remains constant. When changing the seismic site class from A to C and then C to E, the following observations are noticed for both shear and moment: The LLR at site class C was consistently between 37.9% and 50.1% of the LLR at site class A, while the LLR at site class

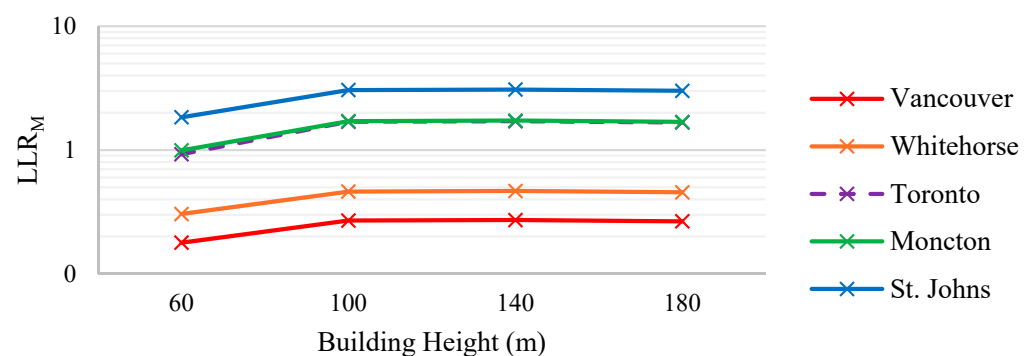
E was between 42.6% and 52.9% of the LLR at site class C. The range of these was primarily due to variations in location, with building height having a much smaller influence and LFRS having a negligible impact. Overall, changing the seismic site class is responsible for magnifying the LLR by up to 5.81 times when comparing sites A and E.

3.3. Impacts of Building Height and LFRS

Building height also has a significant effect on the LLR. The models in this study consist of different selections of structural members across buildings designed for each height. Due to this, the direct comparison of the LLR is slightly skewed as the self-weight; therefore, the seismic load is not exactly equivalent across building heights. The tendency for the LLR to increase as building height increases despite the increase in structure mass can be seen when comparing the 60 m and 100 m points in Figure 9, reaffirming the principle that the wind load effect increases as building height increases, more so than the increase in seismic demand. Conversely, when comparing the 100 m, 140 m, and 180 m tall models, the increase in LLR tends to flatten considerably. However, this is thought to be due to the modeling limitations stated later in this section. Overall, building height was found to increase the LLR by up to 1.85 and 1.83 times for shear and moment, respectively.



(a) Comparison of LLR_V by Building Height



(b) Comparison of LLR_M by Building Height

Figure 9. Comparison of (a) LLR_V and (b) LLR_M versus building height (case: seismic site class C).

Lastly, the effect of the LFRS on the LLR of a building is displayed in Figure 10. This comparison reveals that the LLR is directly proportional to the structural system used in the design (centrically braced frame or moment-resisting frame) and is largely influenced by the ductility and over-strength factors assumed as a part of the seismic analysis procedure. When comparing otherwise similar models, the MRF cases were found to have an LLR between 1.1 and 1.87 times the magnitude of the CBF cases.

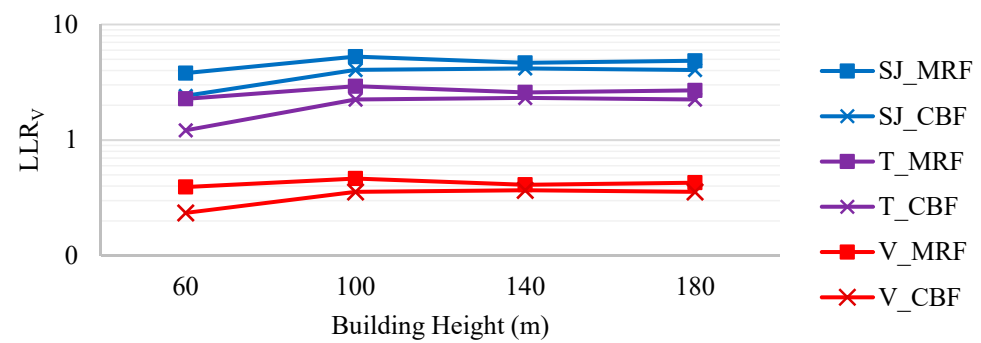
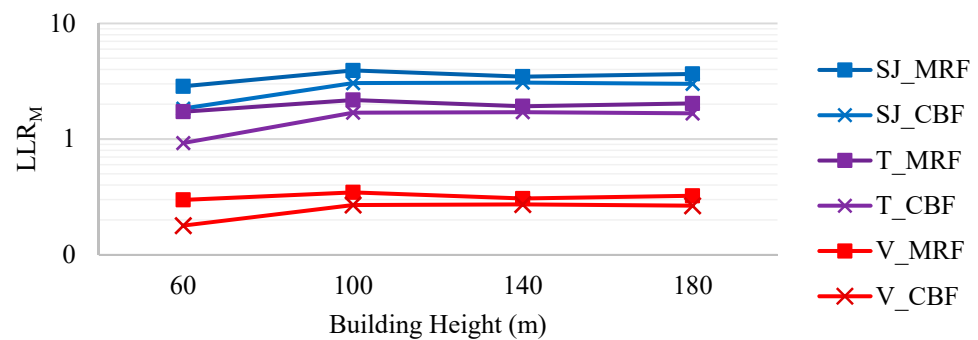
(a) Comparison of LLR_V by building height and LFRS(b) Comparison of LLR_M by building height and LFRS

Figure 10. Comparison of (a) LLR_V and (b) LLR_M versus building height and LFRS (case: seismic site class C).

3.4. Limitations and Future Research

As the analysis conducted in this study is of high order and only intended for high-level design considerations, significant limitations apply. The models outlined in this study vary between heights ranging from 60 m to 180 m and have a footprint of 30 m × 45 m. As such, these buildings are considered dynamically sensitive and require dynamic analysis to be designed to code [18]. However, as this study is primarily interested in quantifying the lateral load demand of conceptually designed buildings and/or assessing existing structures, utilizing static analysis procedures remains appropriate with the understanding that some inherent errors are not considered due to vibrational effects.

Additionally, model cases consisted of fundamental lateral periods ranging from 0.8 to 3.7 s. For calculation purposes, the fundamental lateral period was limited to 2.0 s, as per the NBCC. Models with a lateral period greater than the limit were evaluated with a higher spectral acceleration than the corresponding modal period. This effectively increases the seismic base shear and decreases the calculated LLR. These effects can be seen in Figures 9 and 10, manifesting as an uncharacteristic drop in LLR as building height increases.

To exacerbate the condition, satisfactory design could not be achieved for 140 m and 180 m height MRF buildings for all cases using standard wide flange sections. This is an expected issue, as it is unlikely to feasibly design such tall structures using moment-resisting frames for both gravity and lateral force resisting systems. Thus, the building designs for these cases are expected to deform plastically; however, the overall impact in terms of load quantification is expected to be minimal. Nonetheless, these findings outline the high-level parameters that may be considered in characterizing existing structures to identify the likely governing lateral load in future assessments.

4. Conclusions

This research aims to assess the governing lateral load on tall buildings in Canadian regions, which was accomplished by conducting a parametric study of 120 similarly designed tall structures across five cities to determine the LLR. From this study, the following conclusions can be drawn regarding the study parameters:

- LLR_V ranged from 0.124 to 10.5, and LLR_M ranged from 0.095 to 7.83.
- Many models consisted of an LLR between 0.9 and 1.1, indicating that the lateral load demand of these structures was nearly equal.
- 17 cases had different governing loads for the analyzed forces: the base shear was governed by wind, whereas the base bending moment was governed by seismicity.
- Geographic location had the largest impact on the LLR, accounting for magnifying the LLR by a multiple of up to 11.3.
- Seismic site classification was found to magnify the LLR by a multiple of up to 5.8.
- Building height and LFRS were found to have the smallest impact on the LLR, accounting for an increase in multiplier of up to 1.85 (LLR_V) and 1.87 (LLR_M).
- Some cities were dominated by a specific governing lateral load: in most cases, Vancouver, BC, is expected to be governed by a seismic load, and St. John's, NL, is expected to be governed by wind load.

These conclusions create an understanding of the impacts that wind and seismic loads have on buildings in various regions across Canada and provide valuable insight into the likelihood of either load to govern design. This information can be used at the early stages of design to estimate the performance of structural systems more accurately, identify the load type likely to cause potential damage, and improve the overall understanding of the relationship between building design and governing load. Streamlining initial building designs helps to promote sustainability by optimizing resource use, lowering costs, and enhancing the safety and resilience of buildings. These benefits collectively support the development of more sustainable urban environments. Additionally, these principles can be applied to existing structures in built-up areas to determine the probable governing lateral loads and in city-scale assessments to inform decisions regarding the sustainability of urban infrastructure.

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