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Research Article

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Effect Wind Loads of Hight Rise Building with Deferent Hight Concrete Moment Resisting Frame

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Abstract To improve the performance of existing buildings, they must adhere to the current codes' new level of ductile design. It is crucial to accurately evaluate the lateral load resistance. To achieve this, a project was undertaken to model the wind load responses of typical 20 and 40-story reinforced concrete (RC) building frames. The study focused on Laval city and used response spectrum analysis for a multi-degree of freedom system subjected to wind loads. ETABS software was used to conduct the analysis, with wind load records (GMR) chosen to represent the wind loads' nature in Laval City. Ultimately, the time history analysis was used to conclude the average displacement and base shear of the structures.

Keywords Wind loads, Response spectrum analysis, Time history analysis

Introduction

Effect of wind loads on high-rise buildings with different heights. High-rise buildings are vulnerable to wind loads, which can lead to structural failure, posing a significant threat to the safety of occupants. Therefore, understanding the impact of wind loads on high-rise buildings is crucial to ensure their safety and stability; structural engineers have started since a long time studying the behavior of wind loads and measuring using a variety of methods, including, anemometers or wind speed sensors, these sensors are typically mounted on the building's facade or rooftop and measure wind direction, speed, and turbulence. The data from these sensors can be used to estimate the wind loads on the building and Wind tunnel testing involves constructing a scaled model of the building and subjecting it to simulated wind conditions in a controlled environment. The resulting data can be used to estimate the wind loads on the full-scale building Also can be used Computational fluid dynamics (CFD) analysis, CFD involves using computer simulations to model the flow of air around the building and estimate the resulting wind loads. This method can provide a more detailed analysis of wind loads compared to other methods, but it can also be more time-consuming and expensive. It is important to note that wind loads on tall buildings can vary significantly depending on factors such as building height, shape, and location. Therefore, it is crucial to carefully consider the specific characteristics of the building and its environment when measuring or estimating wind loads. The evaluation of the wind loads resistance of existing structures, and their deficiencies is essential before an appropriate repair or upgrade system can be designed. This study presents an approach for the assessment of wind loads behavior of existing RC frames by using ETABS software which will be an efficient and reliable analytical tool. It predicts the real behavior of such structures during wind loads.

The objectives of this project are to analysis multi-stories RC frame buildings for available and effect wind load time histories. Furthermore, the effect wind loads behavior of two different buildings will be compared in terms of various responses such as base shear, and displacement. Moreover, the relationship between the effect of wind load intensities and the responses will be finding. Finally, the comparison of the fundamental period of buildings from modal analysis and NBCC 2015 recommendations will be illustrated.

Literature Review

High-rise buildings are subject to significant wind loads that can impact their structural integrity and overall performance. This literature review aims to explore the effect of wind loads on high-rise buildings with different height concrete moment resisting frames (CMRFs). By examining the existing body of literature, this review provides an overview of the design considerations, analysis techniques, and mitigation strategies employed to address wind-induced effects on high-rise buildings. The review concludes with suggestions for future research directions in this field. In addition, high-rise buildings are exposed to varying wind speeds and directions due to their increased height and exposure to open spaces. Understanding the impact of wind loads on these structures is crucial for ensuring their safety and reliability. The selection of an appropriate structural system, such as a concrete moment resisting frame, plays a vital role in mitigating the effects of wind-induced forces. Wind loads on high-rise buildings can result in several structural responses, including lateral displacements, overturning moments, and shear forces. These effects can cause excessive building motions, stress concentrations, and potential damage to non-structural elements. Analyzing and predicting these wind-induced effects are essential for designing robust and resilient high-rise structures. Moreover, concrete Moment Resisting Frames (CMRFs) Concrete moment resisting frames are widely used in high-rise construction due to their inherent strength and stiffness characteristics. The design of CMRFs involves consideration of various parameters, such as column and beam dimensions, reinforcement detailing, and connection design. The literature review examines studies that investigate the performance of CMRFs under wind loads in high-rise buildings of different heights.

Numerical simulations and wind tunnel experiments are commonly employed to evaluate the wind-induced response of high-rise buildings. Computational Fluid Dynamics (CFD) models are used to study the flow characteristics around the building, while physical models in wind tunnels allow for accurate measurement of wind pressures on the building surfaces. The literature review discusses the methodologies employed for wind load analysis in high-rise buildings. Furthermore, to enhance the performance of high-rise buildings under wind loads, various mitigation strategies are employed. These include the use of tuned mass dampers, passive control devices, and innovative structural designs. The review highlights studies that investigate the effectiveness of these strategies in reducing wind-induced vibrations and optimizing the structural response of high-rise buildings. Several case studies of high-rise buildings with different heights and CMRF configurations are analyzed in the literature review. These case studies provide valuable insights into the performance of specific structural systems under wind loads and highlight potential areas for improvement. Thus, research Gaps and future directions despite significant advancements in wind engineering research, there are still several areas that require further investigation. Future studies could focus on the development of advanced numerical models, the incorporation of multi-hazard effects (e.g., wind and seismic loads), and the evaluation of sustainable and cost-effective design strategies. To sum up literature review demonstrates the importance of considering wind loads when designing high-rise buildings with concrete moment resisting frames. It highlights the various methodologies for wind load analysis, discusses the performance of CMRFs under wind loads, and explores mitigation strategies to enhance the structural response. The review also identifies research gaps and suggests future directions for the field. Understanding and addressing wind-induced effects are essential for the safe and efficient design of high-rise buildings.

Methodology

The ETABS software is employed to construct 3D models of concrete buildings and perform all analyses. The software is capable of forecasting the geometric nonlinear behavior of space frames when exposed to static or dynamic loads, factoring in both geometric nonlinearity and material inelasticity. To gauge the wind load performance of any structure, it is necessary to estimate its dynamic characteristics and anticipate its reaction to potential wind loads throughout its service life.

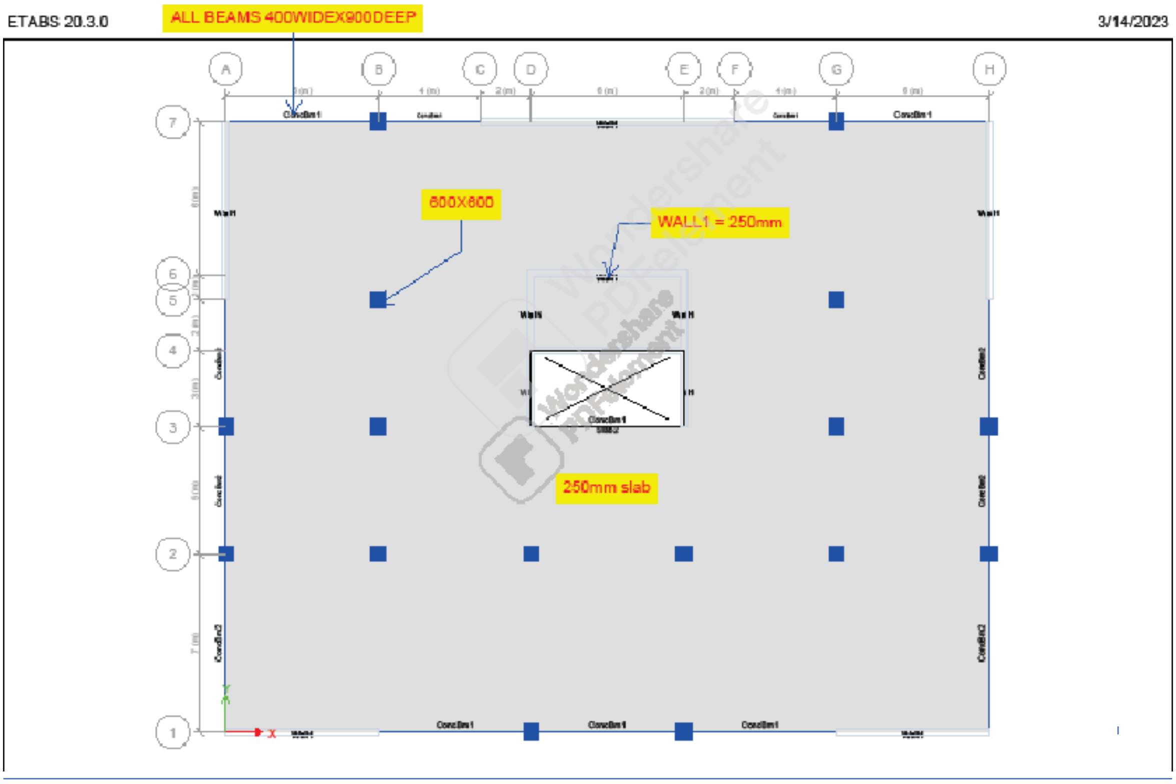

Figure 1: Plan and Elevation of the structure.

Description of Structure

There are two buildings situated in Laval, Canada, which have heights of 20 and 40 stories respectively. The geometric details of these buildings are shown in Figure 1. The building has eight bays that are six meters wide in the north-south direction and three bays in the east-west direction. This includes a central six-meter corridor bay and two nine-meter bays. The first story is 4.85 meters tall, while the remaining stories have a height of 3.65 meters. The assumed values for the yield strength of reinforcing steel (fy) and the 28-day compressive stress of concrete (fc') are 3500MPa and 30MPa, respectively.

Modeling of buildings

In this analysis, the buildings are represented by a series of transverse frames linked together by rigid connections. To simplify the analysis, both the exterior and interior frames are assumed to be ductile and kept the same. This allows for the use of a single frame and its corresponding floor mass in the two-dimensional analysis. Accidental torsion is not considered to maintain consistency with this approach while obtaining design forces. The 3D building is modeled using a space frame, and its centerline dimensions are shown in Figure 2.

Material and cross-sectional properties are defined based on the provided design information in the table below. The beams are assigned as 400x600mm concrete elements for all stories, while two types of columns are used: 450x450mm at external corners and 500x500mm at interior supports. Additionally, a 120 mm typical slab is assigned for each story. The dead load is 2.0 KN/m^2, super dead load is 2.5 KN/m2, and live load is 2.0 KN/m^2. The section and reinforcement details for the beams are also specified. In addition, to construct the finite element model, ETAbs software is used, Figure 1 a depict the plan and perspective 3D views of the 20- 40 story model in ETAbs.

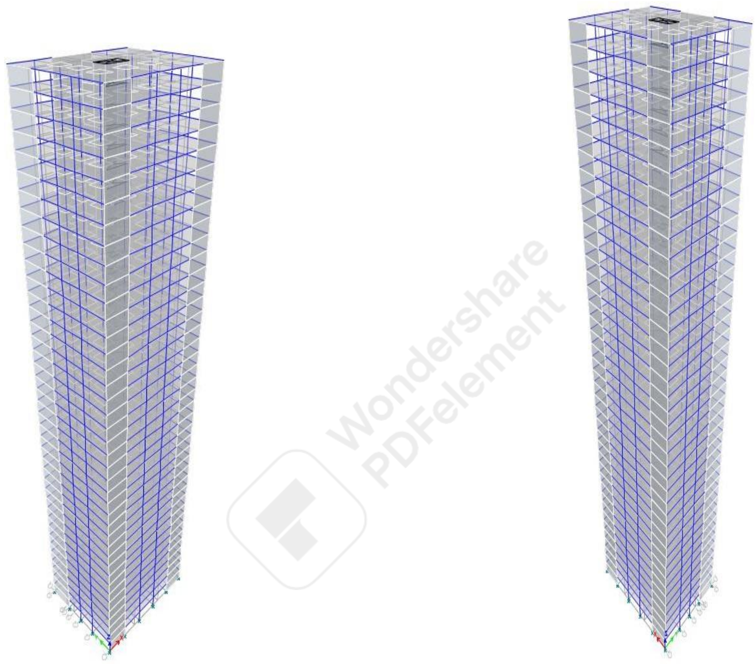

Figure 2: The 3D views of the 20-40-storey model in ETABS.

Linear Static Analysis

Before to performing a wind load analysis of the building, a linear static analysis was carried out on both models. The outcomes were compared to response spectrum and time history analysis techniques. During this analysis, only the weight of the building was taken into account. It is evident that for static loads, the displacements and base shear will be quite low, due to the structure's stiffness. The results, in the form of graphs, will be presented in the appendix, and conclusions will compare and evaluate them.

Linear Dynamic Analysis (Response spectrum analysis)

The response spectrum is obtained from time history and provides the peak response of a single-degree- of-freedom (SDOF) system with a given damping and time period subjected to specific wind load. This analysis is crucial for making design decisions in structural engineering. The maximum response of a structure is represented by response spectra graphs, which can be displacement response spectrum (S_d), velocity response spectrum (S_v), or response spectrum (S_a). In each type of response spectrum, the response is plotted against the period to determine the maximum response, such as displacement, velocity, or acceleration, for structures with different natural frequencies or periods. For undamped system:

$$S_v = \omega S_d$$

$$S_a = \omega S_v = \omega^2 S_d$$

The selection of wind load data is typically influenced by several factors, including the wind load history of the location, the building's height, shape, size, function, and the local wind climate. In this analysis, the design spectral acceleration according to NBCC-2015, as shown in Figure 4, has been defined in ETABS and applied to the model as a response spectrum load in the X and Y directions. The site type was set to C, and a damping ratio of 5% was considered.

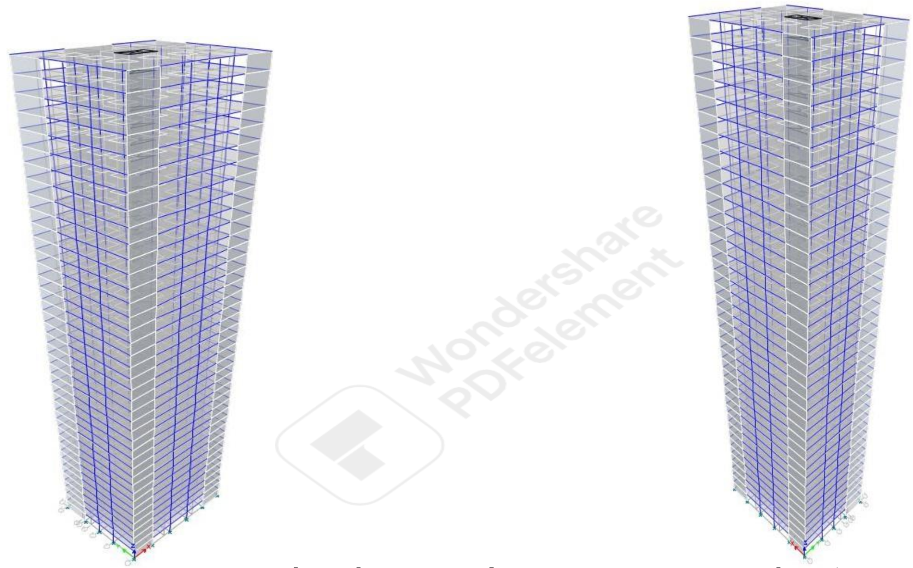

Figure 3: Design spectral acceleration used in response spectrum analysis (NBCC-2015)

The acceleration response spectrum of the original GMRs was determined and then scaled based on the design acceleration response spectrum of Laval city in NBCC 2015. GMRs from 15 locations were analyzed to obtain a final spectral acceleration that closely matches the design spectral acceleration of Laval city, using the ordinate scaling method.

Ordinate scaling method

In this method, the original GMRs are modified using a scaling factor which can be calculated according to the following equation:

$$scailing\ factor = \frac{Sa_{T1},ds}{Sa_{T1},GMR}$$

Where: Sa_{T1},ds Sa_{T1},GMR is the value of spectral ordinate in design spectral acceleration corresponding to T = 1s. is the value of spectral ordinate in the spectral acceleration of GMR corresponding to T= 1s.

Each GMR was assigned a scaling factor, which was used to multiply the data in the record, as shown in Table1. It is important to note that the scaling factors for GMR 4, GMR 5 GMR6 were significantly higher than the target values for this project and should be disregarded for a more accurate representation of wind loads in Laval city. These values indicate the maximum wind loads that the building may encounter during a wind event and can be utilized to design the structure to withstand these loads.

Table 1: Scaling factors

Accelerogram	**Sa,T1 GMR**	**Scale Factor Ordinate Scaling**
1	0.51684	0.628821299
2	0.15875	2.047244094
3	0.18101	1.795480913
4	0.05196	5.254811393
5	0.04112	6.903696498
6	0.10313	4.151362358
7	0.24868	1.306900434
8	0.2041	1.592356688
9	0.3315	0.980392157
10	0.15666	2.074556364
11	0.10198	3.186899392
12	0.09682	3.356744474

13	0.18605	1.746842247
14	0.13687	2.374515964
15	0.08913	3.646359251

The comparison between the design spectral acceleration and the average acceleration response spectrum is depicted in Fig. 4. Although the peak values of wind load acceleration differ between the two curves, they exhibit a similar trend for periods exceeding 0.5 seconds.

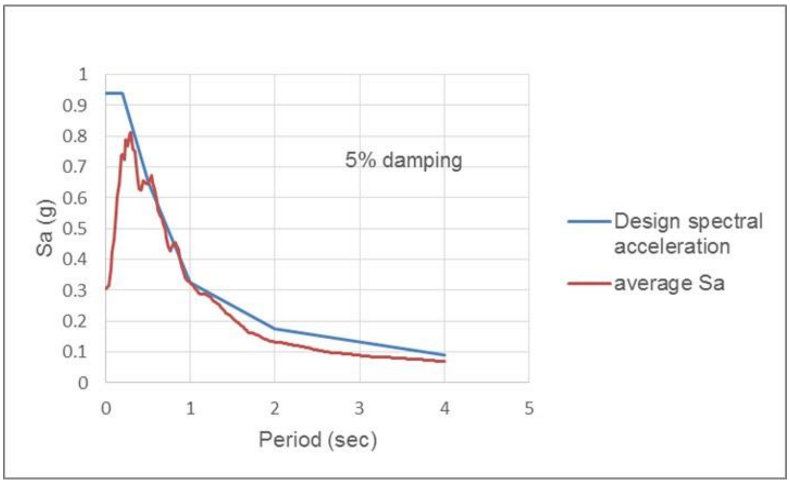

Figure 4: Design spectral acceleration and the average acceleration response spectrum.

Response spectrum analysis for modal-1 (20-story building)

Figure 5 displays the displacement values for each story of the 20-story building model in modal-1. The two graphs are colored in blue and red to represent the displacement in the X and Y directions, respectively. By utilizing the SRSS method to combine displacements from all modes, the absolute maximum displacement has been determined to be 180 mm in the X direction at story 20 and 135 mm in the Y direction. It is notable that the response spectrum was loaded in both X and Y directions, and the displacement in the Y direction is less than that in the X direction.

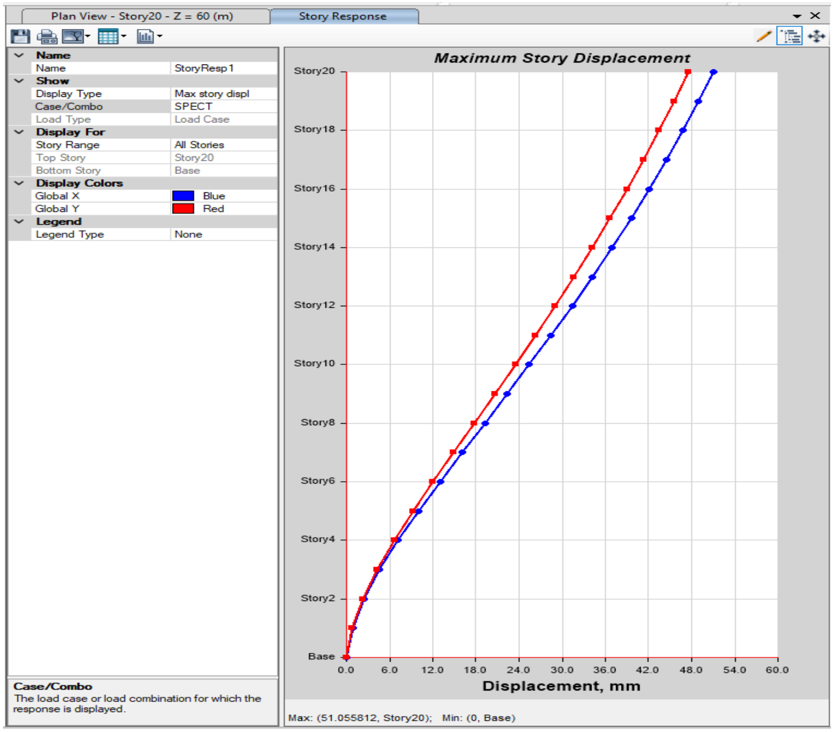

Figure 5: Maximum story displacement

In Fig. 6, the plot of maximum story drift indicates that the highest drift occurred in both X and Y directions in story 1. This result is consistent with the maximum displacement curve in Fig. 10, which shows the minimum slope of the displacement plot between stories 1 and 2, corresponding to the ultimate drift of the building.

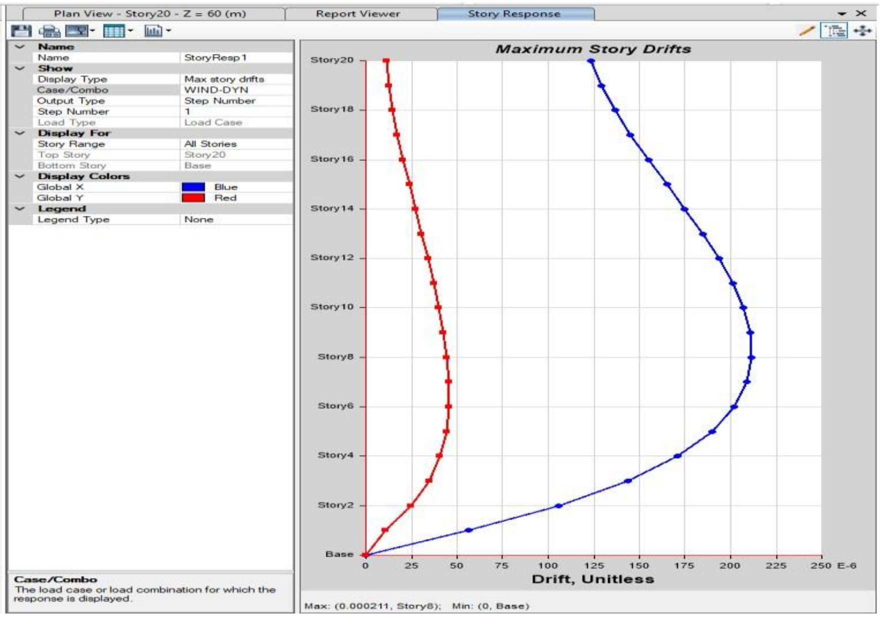

Figure 6: Maximum story drift

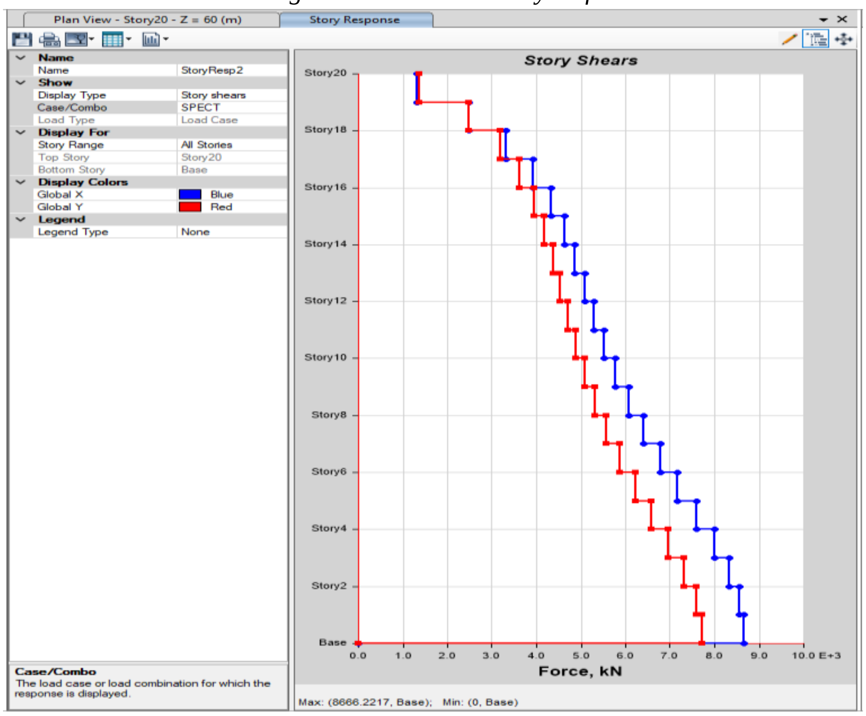

Figure 7: Story shears

To maintain stability during wind loads, a structure generates a shearing force at its base called base shear (Vb), which arises from the inertia of the upper mass resisting the building's movement. Figure 7 displays the shear force for each story. The greatest force occurred at the bottom of the building, reaching 18,000KN and 19000KN in the X and Y directions, respectively. The shear force decreases in magnitude as the height of the stories increases.

Response spectrum analysis for modal-2 (40-story building)

Figure 8 displays the maximum displacement for each story in modal-2 (40-story), where the blue and red curves depict the displacement in the X and Y directions, respectively. The highest absolute displacement values are observed at story 40, with 760 mm in the X direction and 640 mm in the Y direction.

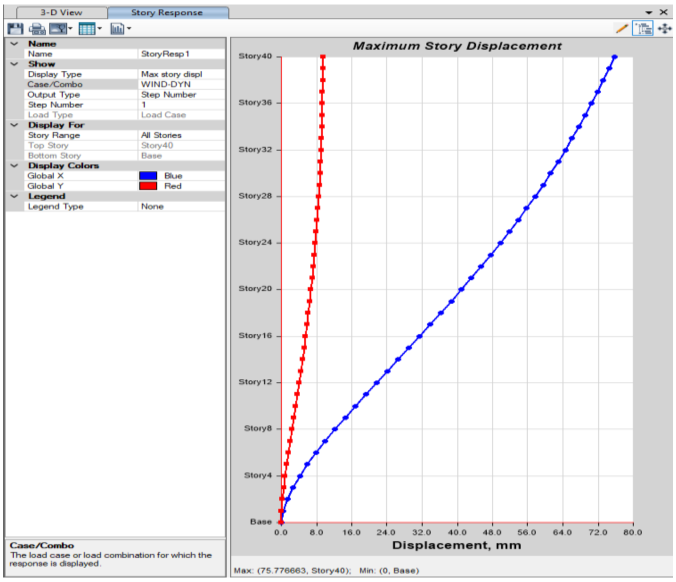

Figure 8: Maximum story displacement

The maximum story drift is plotted in Figure 9, according to the plot, the highest drift happened in story 2 in X direction was 0.000817mm. The maximum displacement curve in Fig.10 confirms the drift results, since the minimum slope of the displacement plot occurred in the portion between stories 1 and 2, which corresponds to the ultimate drift in the building.

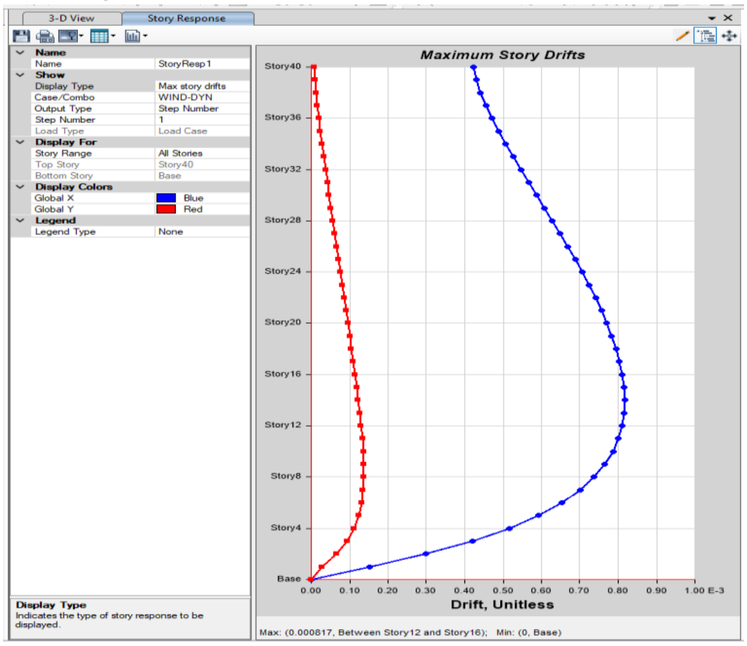

Figure 9: Maximum story drift

The shear force for each story is shown in Fig. 10. The highest force developed at the bottom of the building, and it were 24000 KN 28,000KN in X and Y respectively.

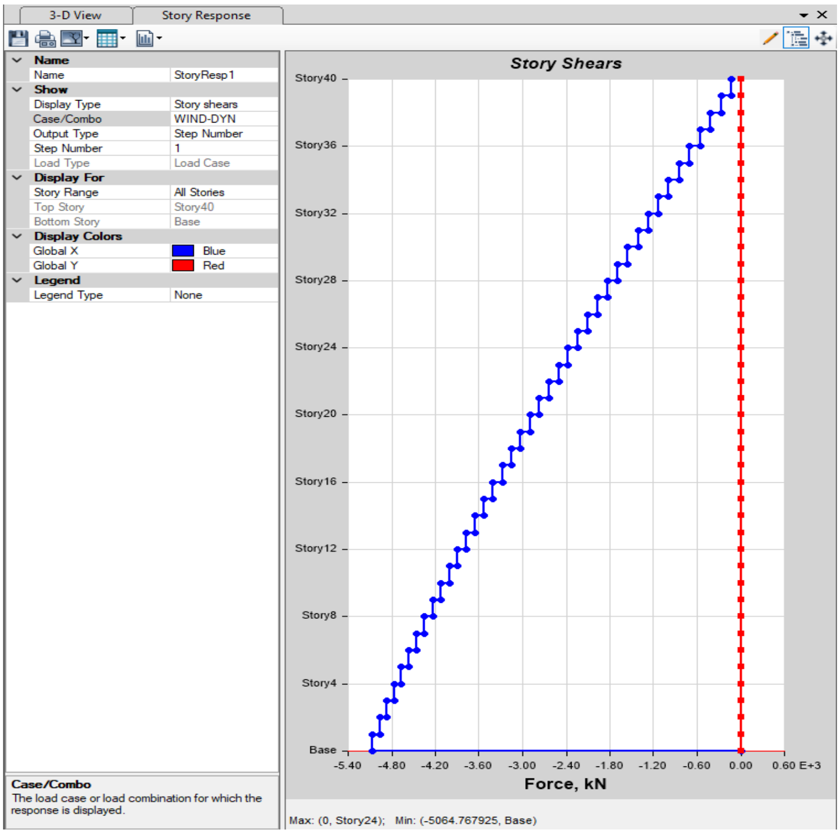

Figure 10: Maximum story shear

Time History Analysis

Inelastic time-history analysis is considered the most precise method to predict the force and deformation demands on different parts of a structure. However, it has limitations due to its sensitivity to the modeling and wind load characteristics, and it requires accurate modeling of the load-deformation characteristics. In this study, ETABS was used to perform nonlinear dynamic time-history analysis on 3D models of pre-existing buildings, using a sine function to define load cases in both X and Y directions, as illustrated in Figure 11.

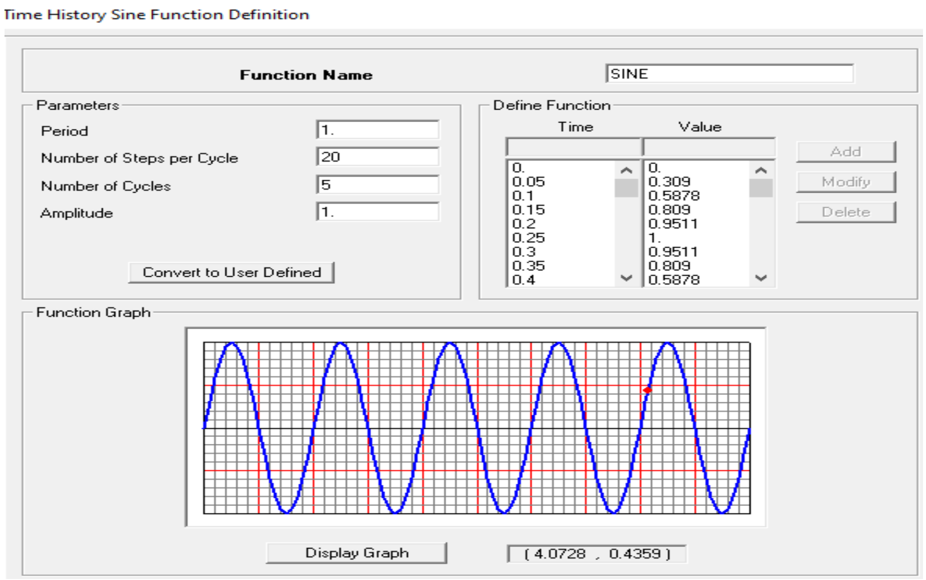

Figure 11: Defined time history load as sine function

Figure 12 shows the plot of spectral acceleration and time period for different damping ratios used in this project. It is clearly seen that increasing the damping ratio will reduce the spectral acceleration.

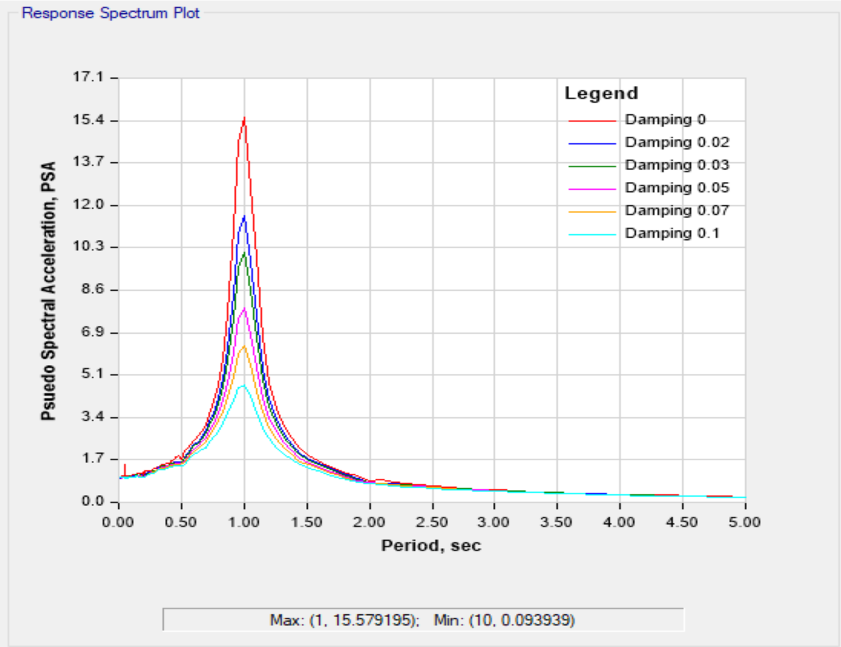

Figure 12: Pseudo spectral acceleration vs period plot

Time history analysis of model-1 (20-story)

Figure 14 illustrate the displacement of the 20-story building and calculated at each story location in both X and Y directions. The absolute maximum displacement is 220 mm and 230 mm respectively.

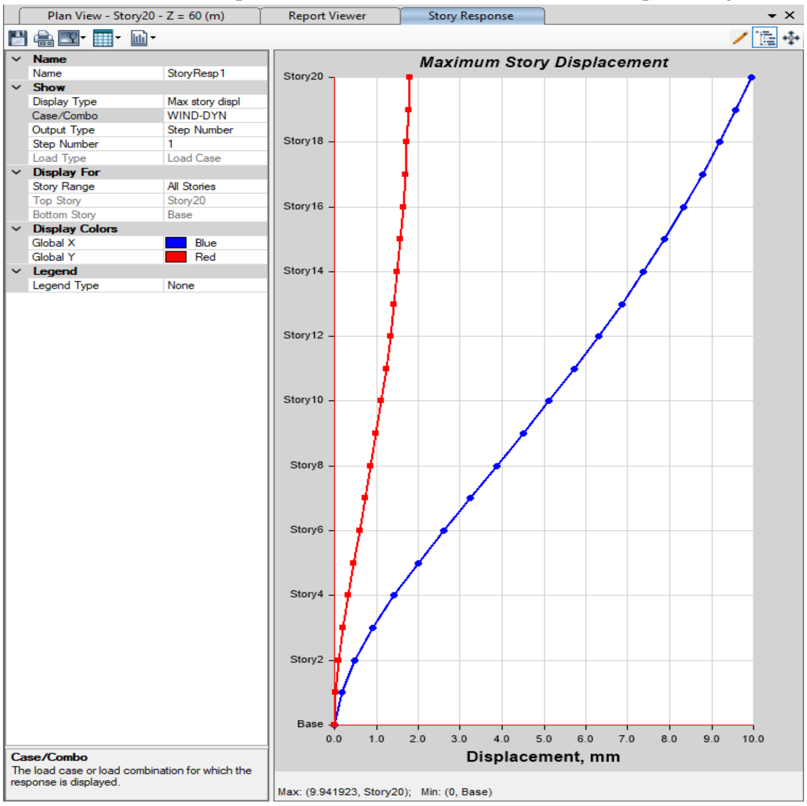

Figure 13: Maximum story displacement

The maximum story drift for modal-1 is plotted in Figure 14, according to the plot, the highest drift happened in story 1 in X direction was 0.000307 mm.

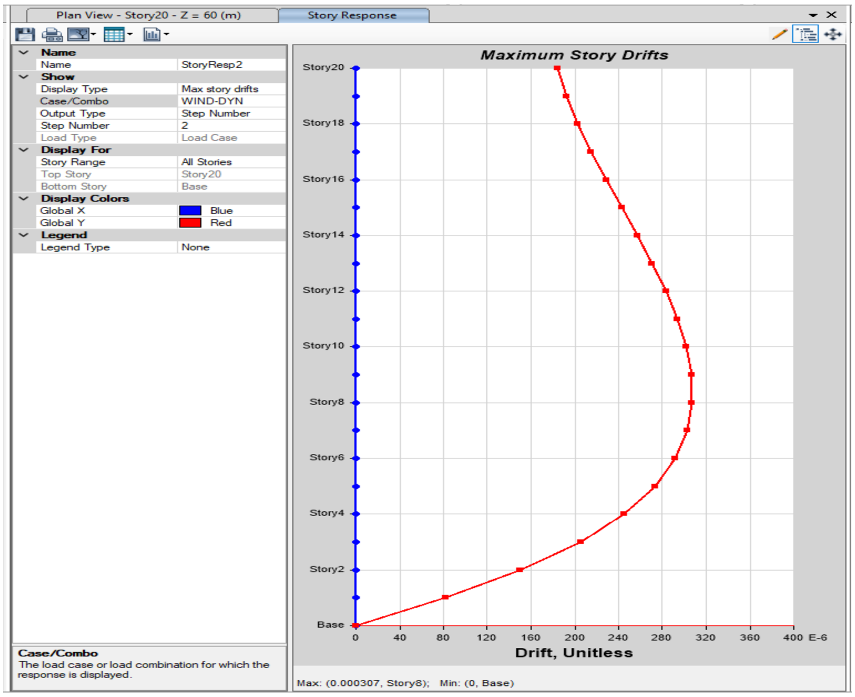

Figure 14: Maximum story drifts

Figure 15 displays the base shear for model-1 for each story. The graph shows that the maximum force was developed at the base of the building and it was around 21,000KN. The magnitude of the shear force decreased as we move up to upper stories. The most significant gap in the shear force occurred between story 3 and 4, indicating that story 3 experienced the highest force of approximately 17,000KN.

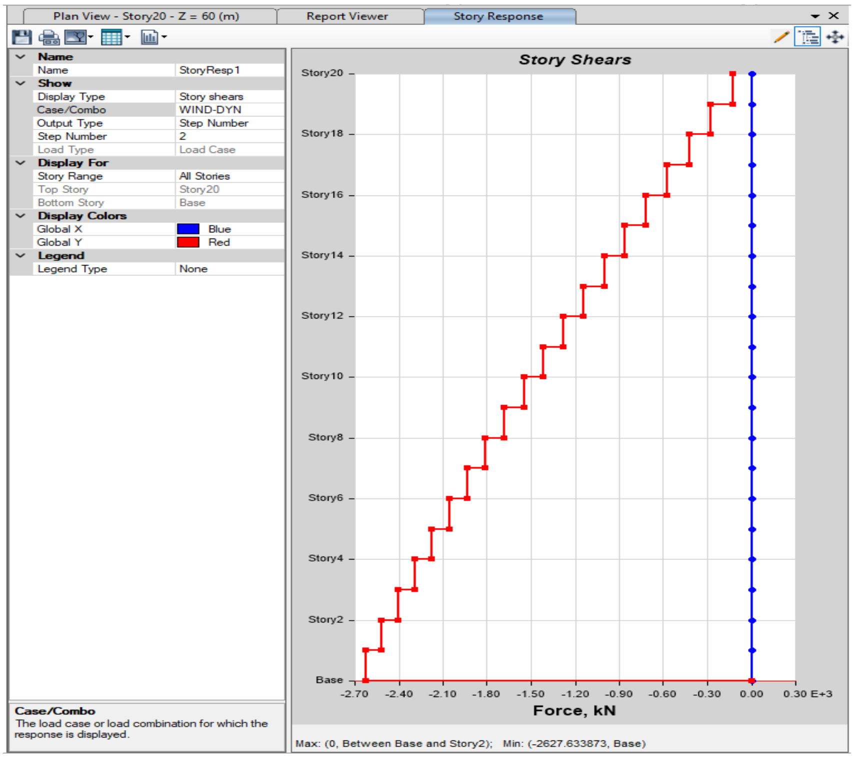

Figure 15: Maximum story shear

Time history analysis for model-2 (40-story)

The steps will be repeated for model 2 as the previous one, and displacements and maximum story shear of the 20-story building calculated at each story location, for both directions, the graphs will be in appendix, the result will be tabulated in the discussion part.

Maximum displacement comparison

Tables 2 and 3 illustrated the maximum displacements for each model and the result are shown as following.

Table 2: Maximum displacement comparison from static linear analysis

Structure	X-direction (mm)	Y-direction (mm)
20-story building	0.260	0.050
40-story building	2.150	0.100

Table 3: Maximum displacement comparison from response spectrum analysis

Structure	X-direction (mm)	Y-direction (mm)
20-story building	180	130
40-story building	760	640

Table 4: Maximum displacement comparison from time history analysis

Structure	X-direction (mm)	Y-direction (mm)
20-story building	230	200
40-story building	900	998

The displacement tables provided above show the displacement values in both X and Y directions. It is evident that the static analysis resulted in a very small amount of displacement due to the rigidity of the structure. On the other hand, the dynamic analysis resulted in a higher value of displacement compared to the spectrum analysis, as it considers all possible forces generated during the given time period.

Results & Discussion

In Table 5 obviously the base shear for static loads for static liner analysis is very low approximately zero due to rigidity of the structure. In addition, Table 6 and 7 show the base shear resulted by performing response spectrum and time history analysis corresponding to design spectrum in NBCC-2010 for both 6-story and 12-story buildings for average spectral acceleration. It is obvious that time history analysis has higher about of shear due its accuracy and fully detailed.

Table 5: Base shear comparison from static liner analysis

Structure	X-direction (KN)	Y-direction (KN)
20-story building	0	0
40-story building	1.2* 10-9	1.0* 10 –9

Table 6: Base shear comparison from response spectrum analysis

Structure	X-direction (KN)	Y-direction (KN)
20-story building	16000	17000
40-story building	22000	25000

Table 7: Base shear comparison from Time history analysis

Structure	X-direction (KN)	Y-direction (KN)
20-story building	12000	20000
40-story building	22000	32000

Comparison of Fundamental Periods of Buildings

The fundamental period of a building is an important characteristic that describes its behavior under wind loads. It can be used directly to determine the global demands on a structure due to a given wind load input. However, the period of a bare frame in its fundamental mode of vibration is generally higher than the value obtained using the expression recommended by NBCC-2015, as non-structural elements in a frame play a significant role in stiffening the structure and reducing its fundamental period of vibration. This study only considers bare frames and compares them to the 2015 NBCC recommendations. For concrete frames, the code recommends the use of an empirical expression to estimate the fundamental period.

$$T = 0.075(hn)3/4$$

Where,

T is the fundamental period

hn is the height of the building above its base.

In the event that the modal analysis of the frame results in a fundamental period (T) greater than the value obtained from the expression recommended by NBCC 2015, the lateral loads should be evaluated using the higher value of the period. The revised period should not exceed 1.5 times the fundamental period obtained from the modal analysis.

Fundamental periods of buildings

Table 8: Fundamental period (sec)

	20 story	**40 story**
Bare frame	1.865	2.202
NBCC-2015, Ta	0.790	1.303
1.5 Ta	1.180	1.950

Table 8 indicates the periods of bare frame models for the 20-story and 40-story buildings obtained through modal analysis. The period of the bare frame for the 20-story building is 1.447 seconds, which is higher than the value of 0.790 seconds obtained from the NBCC expression. Similarly, for the 40-story building, the period of the bare frame is 2.2 seconds, while the corresponding value from NBCC is 1.303 seconds. These results indicate that non-structural elements in a building can significantly reduce its fundamental period of vibration.

Conclusion

The wind load has been found to affect buildings in Laval, Canada. Therefore, in order to resist lateral loads, buildings in Laval need to be more flexible. In this study, two pre-existing concrete moment- resisting frame buildings, one with 20 stories and the other with 40 stories, were modeled in ETABS. The Linear Static, Response Spectrum, and Time History methods were utilized to generate static and wind load responses for each structure. These methods were then compared to determine the displacements and shear of the buildings. In addition, the displacement of buildings varies depending on the analysis method used. For example, the static analysis has very little displacement due to its rigidity, while the dynamic analysis results in a greater displacement. The time history method produces the highest values because it considers all possible forces generated. However, this method is resource-intensive, while the response spectrum method is more efficient and cost-effective. Finally, for critical structures, the time history method should be used instead of response spectrum analysis because it predicts the behavior of the structure more accurately.

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