Experimental Evaluation of ABL and Downburst Wind Loads on an Elevated Building

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A thesis submitted in partial fulfillment of the requirements for the Master of Engineering Science degree in Civil and Environmental Engineering
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Abstract

Elevated low-rise buildings are vulnerable to cladding damage underneath the structure due to extreme winds and the absence of proper wind loading codes and standards such as the National Building Code of Canada (NBCC). The space between the ground and the horizontal base surface of an elevated structure affects its aerodynamics differently compared to a ground-mounted structure. Despite the widespread use of elevated low-rise buildings, there is still limited understanding of the wind interaction across building surfaces for different stilt heights and wind types. This research aims to evaluate the impact of atmospheric boundary layer (ABL) and downburst wind loads on an elevated low-rise building with typical northern Canadian architecture, using experimental testing facilities at Western University. The Boundary Layer Wind Tunnel Laboratory was used to simulate ABL winds in open terrain and to measure the external pressure coefficients on the building model. Additionally, the WindEEE Dome was used to generate downburst-like winds and measure their resultant wind loads. The analysis of both datasets indicates that stilt height has a significant impact on surface pressures on the base surface of the building, resulting in increased peak suction near the corners and edges when the stilt height is increased. The wind loads from both test series were compared to the newly introduced ABL wind loading provisions for elevated structures in ASCE 7-22 to assess the adequacy of these design pressures on the study building. The enveloped negative external pressure coefficients due to both ABL and downburst winds were effectively covered by the ASCE 7-22 design loads for stilt heights below 2.5 m. However, 2.5 m and 3 m stilt heights produced external pressure coefficients which exceeded the design pressures of ASCE for tributary areas below 5 m². Therefore, further refinement of external design pressures and components and cladding zones may be necessary to ensure a more conservative design of elevated structures. The results of this study can be used to improve the NBCC by incorporating aerodynamic information for elevated buildings.

Keywords

Elevated structure, ABL wind, downburst, NBCC, ASCE, low-rise, wind tunnel, WindEEE, wind load, and components and cladding.
Summary for Lay Audience

Elevated structures, such as those used in arctic and coastal communities, provide climate-resilient infrastructure that reduces permafrost degradation and withstands flooding. These structures have their lowest floor elevated above ground level using compression-resistant structural members, such as piles or columns, allowing air/water flow between the ground and the base of the building. However, under extreme winds like downbursts and hurricanes, high-velocity winds can damage the cladding on the underside of the structure. Therefore, it is crucial to understand the aerodynamics and wind loading of elevated structures to establish appropriate engineering standards.

This study experimentally simulated both straight-line winds and downbursts to evaluate resulting wind loads on an elevated low-rise building. Straight-line winds were used to determine wind loading and provide a benchmark for comparison with downburst loads. Downbursts are severe wind events associated with notable structural damage, creating multi-directional high-velocity winds over a short duration. Pressure measurements were obtained on a building model to determine the external pressure coefficients across the building surfaces for both wind types.

The downburst-induced wind loads were compared to those of straight-line winds, revealing higher loads on component and cladding elements under downburst events. The datasets were then compared to current design pressures for elevated structures in ASCE 7-22 from the United States and Canadian wind loading provisions for seated buildings in NBCC 2020. The results indicated that the standards may underestimate the negative pressures on buildings with stilt heights above 2 m and generally underestimated positive pressures on cladding exceeding an area of 5 m². This study provides insight into wind loads on elevated structures and produces new aerodynamic data for codification in NBCC.
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Nomenclature

\[ A \quad \text{Tributary area} \]

\[ C_g C_p \quad \text{NBCC design external pressure coefficient for components and cladding} \]

\[ C_p \quad \text{Pressure coefficient} \]

\[ C_{p,\text{avg}} \quad \text{Area-averaged external pressure coefficient} \]

\[ C_{p,\text{H}_{\text{ABL}}} \quad \text{ABL pressure coefficient referenced to mean roof height} \]

\[ C_{p,\text{ref}_{\text{ABL}}} \quad \text{External pressure coefficient measured at the Boundary Layer Laboratory} \]

\[ \hat{C}_p^h \quad \text{Peak pressure coefficient on elevated building model} \]

\[ \hat{C}_p^{\text{stitted}} \quad \text{Peak pressure coefficient on ground-mounted building model} \]

\[ D \quad \text{Diameter of downburst downdraft} \]

\[ F_{WT} \quad \text{Wind tunnel factor} \]

\[ GC_p \quad \text{ASCE design external pressure coefficient for components and cladding} \]

\[ H \quad \text{Height of downdraft from the ground} \]

\[ K_d \quad \text{Directionality factor} \]

\[ p_0 \quad \text{Static pressure} \]

\[ P \quad \text{Pressure} \]

\[ R \quad \text{Cumulative parameter comparing peak pressures between stilted and ground-mounted model} \]

\[ T \quad \text{Moving average window} \]

\[ t \quad \text{Time} \]

\[ V_h \quad \text{Velocity at mean roof height} \]

\[ V_{\text{ref}} \quad \text{Reference velocity} \]
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<thead>
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<th>Symbol</th>
<th>Definition</th>
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<td>$V_{NBCC_{10}}$</td>
<td>50-year mean hourly wind speed at a height of 10 m in open terrain as reported by NBCC (2020)</td>
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<tr>
<td>$V_{WT_{10}}$</td>
<td>Wind tunnel mean hourly wind speed at a full-scale height of 10 m</td>
</tr>
<tr>
<td>$V_{3,s,,open}$</td>
<td>3 second gust velocity in open terrain</td>
</tr>
<tr>
<td>$V_{3600,s,,open}$</td>
<td>Mean hourly wind speed in open terrain</td>
</tr>
<tr>
<td>$Z$</td>
<td>Elevation ground level</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Building orientation</td>
</tr>
<tr>
<td>$\lambda_L$</td>
<td>Length scale</td>
</tr>
<tr>
<td>$\lambda_T$</td>
<td>Time scale</td>
</tr>
<tr>
<td>$\lambda_V$</td>
<td>Velocity scale</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density of air</td>
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Chapter 1

1 Introduction

1.1 Background

Over the past century, there have been indisputable changes to the Earth’s climate. Since the 1900s, the average surface temperature has risen approximately 1° Celsius with most of the warming occurring over the past 40 years (NOAA, 2021). Climate change is not limited to increasing global temperatures but extends to numerous environmental phenomena, including but not limited to melting ice sheets, decreasing snow cover, droughts, and increasing frequency and intensity of extreme weather events such as hurricanes and floods. The implications of climate change are extensive, especially to our built environment, where infrastructure is forced to withstand increasing climate stressors and their variability. Infrastructure must continually adapt to the impacts of climate change to reduce vulnerability and mitigate the negative effects of extreme weather events. Elevating buildings above ground level is a construction technique that has been adopted worldwide to combat two different environmental phenomena resulting from climate change: permafrost degradation and flood surging in arctic and coastal communities, respectively. In arctic climates, buildings are at an increasing risk of damage to their foundations due to increasing surface temperatures and the gradual degradation of permafrost. By elevating structures above the ground, the heat transfer from building floor to the frozen ground is minimized, thus reducing permafrost degradation, and ensuring foundation stability in high-latitude communities (Younis et al. 2023). Similarly, buildings within coastal communities are lifted from the ground to reduce the risk of damage under extreme flooding and storm surges. Elevated buildings have been widely adopted in many arctic and coastal communities; however, design wind loads for elevated structures are not currently included in the Canadian building code (NBCC 2020).

Elevated buildings in Arctic climates have been adopted to reduce permafrost degradation and minimize foundation instabilities. Permafrost was first introduced by Muller (1947) as an abbreviation of the term “permanently frozen ground” and has since been defined as a
ground which has remained below 0° Celsius for more than two years (Brown & Kupsch, 1974). Nearly 50% of the land in Canada is characterized by the presence of permafrost, and nearly the entire country north of the 60th parallel is enveloped by continuous layers of permafrost (Natural Resources Canada, 2022). Recent observed changes in Canadian permafrost include temperature increases associated with rising surface air temperatures, as well as the increased depth of the top layer of seasonally unfrozen soil, referred to as the active layer (Derksen et al., 2019). A study by Ramage et al. (2021) projected that by the year 2050, 42% of settlements within the Arctic Circumpolar Permafrost Region will be within permafrost-free areas, which are high-hazard zones. Permafrost degradation poses severe risks to the built environment as many structures within permafrost regions are built under the assumption that the ground can be treated as a frozen, stable state (Younis et al. 2023). However, increases in permafrost temperatures significantly decrease the bearing capacity of the ground, thus leading to structural damage and failure as observed through surface distortions and depressions on highways and damaged foundations and cracked walls on buildings (Hjort et al., 2022).

Buildings are elevated above the ground to reduce heat transfer from the building to the frozen ground to mitigate the risk of permafrost degradation. This technique uses various foundation systems such as concrete piles, shallow footings, and aluminum space framing to lift the buildings resulting in a ventilated crawl space below them. Oswell and Nixon (2015) conducted a geothermal assessment on the impact of rising temperatures on raised buildings in permafrost terrain. The authors conclude that raised buildings are generally immune to negative climate warming impacts and show that the degradation of permafrost under raised buildings is not likely to occur for many decades. Elevated structures are being constructed globally to combat permafrost degradation (Andreev, 2022; Shur & Goering, 2009) and have been incorporated into government housing guidelines and recommendations in northern Canada (Government of Nunavut, 2020; Government of Northwest Territories, 2021; Government of Yukon, 2021). These guidelines require that adfreeze piles and space frame foundations have an air space of at least 1 m, whereas shallow footings must have a minimum clearance of 0.6 m to improve ventilation and limit the potential for snow drifting. Comparatively, elevated structures built within coastal
communities are typically raised to 3 m above ground to mitigate the impacts of flooding and storm surges.

Coastal areas are prone to extreme natural disasters like hurricanes, coastal storms, and flooding, making infrastructure in these areas particularly vulnerable to structural damage. The expected annual losses due to hurricane winds and storm-related flooding to the United States residential sector is $34 billion, and 75% of those losses are attributed to coastal storms (Congressional Budget Office, 2019). Additionally, over 40 million people in the United States are estimated to live in a 100-year floodplain (Wing et al., 2018). These communities are at an extremely high risk of storm-related damage as seen following Hurricanes Laura, Florence, and Dorian. To achieve greater coastal resilience, the Federal Emergency Management Agency (FEMA) recommends elevating houses located in 100-year flood zones to at least the elevation corresponding to a 100-year flood (1% annual exceedance probability) (FEMA, 2005, February). By elevating the lowest horizontal structural members above the anticipated flood level, the forces of flooding on structures are greatly reduced.

Despite the widespread adaptation of elevated structures in both arctic and coastal regions, there are very few studies that have evaluated the associated risk of wind-related damage. The open crawl space beneath elevated buildings introduces wind loads, which have not been incorporated into building codes, such as NBCC 2015 and 2020, and are neglected during the design phase. The aerodynamic loading on elevated structures is anticipated to be significantly different compared to similar ground-mounted structures due to the exposure to winds on the structure’s base. Therefore, assessing and understanding the differences between elevated and ground-mounted buildings in terms of wind loads is important to provide effective design recommendations.

1.2 Wind Loads on Elevated Structures

Elevated structures are particularly susceptible to wind loading due to their increased roof height and their open underside. Despite their prevalence in northern communities, NBCC does not have provisions for such elevated buildings. Wind-induced damage to elevated structures is primarily associated with the roof and underside of the buildings and can be
categorized into two types of damage: structural damage and direct envelop damage. Structural damage due to wind involves the tops of frame walls shifting inward or outward, causing rotating about the base of the wall as observed following Hurricane Ivan and Hurricane Ike (Marshall, 2006; 2010). Additional structural damage reported from these hurricanes included the complete removal of wooden decks, and in rare instances, roof structures were blown off. Insufficient connections attributed to some structural damage, such as straight-nailed bottom plates being pulled out of the subfloor as well as a lack of adequate strapping and bracing leading to wall failure (Marshall, 2006). Direct envelope damage included the removal of roof shingles, damage to vinyl or hardboard siding and unprotected windows (Marshall, 2010). The cladding on the underside of pile-elevated structures was also damaged in Hurricane Ivan, as observed through the loss of gypsum board, plywood, and vinyl sheathing (FEMA, 2005, August). The observed sheathing loss was estimated to be due to rapid wind acceleration as it passed below the elevated building.

Limited experimental and numerical studies have been conducted to better understand the wind interaction with elevated buildings. A study by Holmes (1994) utilized experimental wind tunnel testing to evaluate the effect of elevation, roof pitch, and wind direction on a single-story gable-roof elevated house at 1:50 and 1:100 scale. The author reported higher magnitude roof and windward wall pressures on the elevated structure compared to its ground-mounted counterpart. The increased wind velocity at eaves height resulted in a 40-80% difference in pressures on the elevated building for the same windstorm event. Additionally, it was found that the roof pitch affected the roof pressures when the wind direction was normal to the roof ridge. Lower roof pitches resulted in stronger suction on the roof surfaces. The floor pressures of the building model’s floor pressures were not provided as this was outside the project’s scope.

Several wind tunnel studies on elevated buildings have been conducted at the NHERI Wall of Wind (WOW) Experimental Facility (EF) at Florida International University. Chowdhury et al. (2017) performed an assessment of the aerodynamics of elevated homes at the WOW EF for four elevations, including a slab-on-grade model. The authors demonstrated the importance of global loads on elevated structures, as there was a 78% increase in peak lateral force coefficients and a 107% increase in mean moment coefficients.
about the lateral axis for the highest stilt height, 3.66 m, relative to the slab-on-grade model.

Kim et al. (2019) used a combined field study and experimental testing approach to better understand the effects of building elevation on resulting wind loads. The authors assessed damage surveys published after Hurricane Michael (Sutley et al., 2018) and found that 50 of 69 elevated buildings surveyed had cladding damage on the floor underside. The cladding damage was similar to that following Hurricane Ivan and Ike in that it commonly consisted of a detachment of vinyl sheathing and fracture of hardboard, leaving structural members on the underside of the building exposed. The authors conducted large-scale wind tunnel tests at the WOW EF on a gable-roof building at four elevations from 0 m to 3.66 m full-scale. It was found that peak pressure coefficients on the roof surface do not differ significantly with an increase in building elevation. The two highest elevation cases resulted in peak floor pressure coefficients similar in magnitude to those on roof corners and ridges, commonly known as the critical locations for component and cladding design. In some cases, peak floor pressure coefficients were nearly 60% greater than those in the same region projected onto the roof. The findings were reported to follow similar qualitative trends in cladding damage from Hurricane Michael in terms of areas of peak pressures on the model building compared to areas of reported damage in the field.

Abdelfatah et al. (2020) conducted similar wind tunnel tests at the WOW EF and explored an area-averaging procedure to determine design zones on the floor underside of elevated structures. The authors investigated three stilt heights, 0.6 m, 2.15 m, and 3.65 m, as well as a slab-on-grade model and assessed pressure coefficients referenced to the mean roof height of each building. Similar to Kim et al. (2019), the authors found only minor changes in local peak pressure coefficients on the roof and walls as stilt height increased. The magnitude of negative pressure coefficients on the interior area of the floor underside increased from -1.2 to -1.9 for an increase of full-scale stilt height from 0.6 m to 3.65 m. This research was further developed to incorporate a stilt height of 5.2 m in full-scale as well as a two-story building model (Abdelfatah et al., 2022). The authors developed an empirical formula to estimate the zone width on the floor underside as a function of stilt height which could be used for both single-story and two-story buildings. This information, along with results and recommendations by Kim et al. (2020), later assisted with the development of component and cladding zones and design pressures in ASCE 7-22.
Despite clear evidence of cladding damage to elevated houses subjected to extreme winds, design guidelines have not yet been implemented to reduce the risk of wind-induced damage in Canada. Recently, design pressures on the underside of elevated structures have been included in ASCE 7-22 and are equivalent to those on the roof of a flat-roof building, suggesting that wind loads on the two surfaces are similar in magnitude. Currently, no parametric studies which evaluate the effect of roof slope and shorter stilt heights, within the range of 0.5 m to 1.5 m, have been conducted. While the introduction of such design pressures demonstrates significant progress towards the standardization of wind loads on elevated structures, there remains a lack of extensive study on such loads.

1.3 Downburst Wind Loads

Atmospheric boundary layer (ABL) wind profiles are currently used as the basis for estimating design wind loads on structures. The design pressures in both NBCC (2020) and ASCE 7-22 were estimated using ABL wind profiles in wind tunnel simulations to determine an acceptable level of conservation when designing for wind-induced pressures on a building. However, these synoptic winds do not represent the entirety of extreme wind events. Recently, building codes have begun to incorporate non-synoptic wind loads such as tornado wind loads (ASCE 7-22) to protect buildings in high-risk areas from tornadic winds up to approximately EF2 intensity. While this is an advancement in wind design requirements, current building codes still need improvements to cover all high-risk wind loads. Thunderstorm winds produce significantly different wind characteristics compared to those of ABL winds; therefore, it can be expected that thunderstorm wind loads on structures will differ from those of ABL winds. Similar to tornadoes, downbursts produce winds that are associated with damage to residential houses, transmission towers, wind turbines, and mobile homes. Reports of downburst-induced damage includes damage to concrete columns and platforms (Burlando et al., 2018) as well as the total collapse of roof structures and failure of components and cladding across a wide array of structures (Loredo-Souza et al., 2019). In a study by Lombardo (2012), the author concluded that thunderstorm wind speeds were the dominant extreme wind climate in the United States for mean recurrence intervals over 50 years. In an extensive project, titled “THUNDERR”, funded by the European Research Council (ERC), the detection, simulation, modeling, and
loading of thunderstorm outflows was investigated to better understand downburst wind loading and develop downburst-resilience infrastructure (Solari, 2020). The codification of downbursts has been discussed by several researchers (Kwon et al., 2020; Mason et al., 2010; Solari, 2020); however, downbursts are currently not incorporated into any building codes.

Earlier studies in downburst research include those by Fujita in 1976 by examining a series of aerial photographs taken following a severe thunderstorm that showed trees had been blown into a starburst pattern. This pattern was similar to the damage caused by a jet of descending air as it impacted the ground, causing a radial outburst of damage. This led to the definition of a downburst as strong downdrafts that induce an outburst of damaging winds on or near the ground (Fujita, 1978). Early research on downbursts began in the 1970s and 1980s and contributed to a vast collection of full-scale downburst measurements which provided the benchmark for downburst quantification. The two major field studies were the Northern Illinois Meteorological Research on Downburst (NIMROD) and the Joint Airport Weather Studies (JAWS) projects. These studies each recorded dozens of downburst events which are now considered the foundation for downburst research as they validated the existence of downbursts and provided a nationwide mapping of microbursts.

The structure and life cycle of downburst outflows were investigated using the data collected from these field studies and were later used to define downburst characteristics and better understand the meteorological formation of downbursts (Wilson et al., 1984; Hjemfelt 1987, 1988). Early research included the discovery of a rolling vortex that formed after the impact of the downdraft onto the ground (Hjemfelt, 1988).

The radially divergent outflow produces rapid changes in velocity and direction near the ground due to shear forces from the ground surface. The highly transient nature of downbursts is most important for wind engineering purposes as these winds can create strong horizontal velocities capable of damaging the surrounding infrastructure. Horizontal wind speeds from downburst outflows can reach up to 75 m/s (Letchord et al., 2002), exceeding design wind speeds used in many building codes. These velocities occur at lower elevations compared to maximum wind velocities from ABL winds. Maximum downburst-induced wind speeds occur at heights between 30 m and 100 m above ground (Hjemfelt,
1988; Holmes, 1999; Lin & Savory; 2006). Despite the high intensity and close proximity to the built environment, there is limited research focused on downburst applications to the built environment. However, downburst modelling methods have been developed to better understand the flow characteristics and resulting wind loads of downbursts. First developed in 1986 by Fujita, the impinging jet method has had wide applications among numerical and experimental testing techniques to simulate downbursts. The impinging jet method involves the rapid release of fluid onto a surface to produce the radial outburst characteristic of downbursts. This method has been employed numerically (Kim and Hangan, 2007; Mason et al., 2009a; Vermeire et al., 2011; Aboshosha et al., 2015) and experimentally (Letchford and Chay, 2002; Chay and Letchford, 2002; Mason et al., 2007; Mason et al., 2009b; McConville et al., 2009; Zhang et al., 2013; Junayed et al., 2019). Impinging jet tests are widely used among wind engineers due to their capacity to realistically produce downburst winds.

Despite its widespread occurrence in arctic regions and interiors of North America, downburst applications to wind loading on structures have had limited recognition. A study by Jubayer et al. (2019) simulated downburst winds at the WindEEE Dome and evaluated the resulting pressure distributions on a low-rise building model. The results were compared to those from ABL winds, and the authors reported pressure fluctuations on the roof surface that exhibited less symmetry and uniformity about the wind direction for downburst winds compared to ABL winds. Furthermore, the 3-s peak pressure coefficients on the roof were found to be over 20% higher for downburst winds compared to ABL winds for oblique wind directions. Zhang et al. (2013, 2014a, 2014b) used an experimental impinging jet model to simulate downburst-like winds and assess their resulting wind loads on small-scale low- and high-rise building models. Four R/D locations were used, and peak aerodynamic forces on the building model were reported at a location of R/D = 1. The building model was found to experience surface pressures nearly double those specified in ASCE 7-05. As the model was moved further away from the core center of the downdraft, the wind profile and surface pressures matched better with those of ABL winds. Additionally, high fluctuations of the surface pressures were observed and attributed to high turbulence levels in the outburst flow. These fluctuations were estimated to greatly increase the damage potential of low-rise buildings subject to downburst winds. Similarly,
Jesson et al. (2014) used an impinging jet method to create the transient features of downburst outflows and evaluate their ensuing aerodynamic forces on a cube and portal frame building model. The authors compare the surface pressure coefficients to those of Zhang et al. (2013) and report corner suction values over double those seen by Zhang et al. All other areas showed similar magnitudes of pressure coefficients.

Given the locations, such as the arctic regions, in which elevated structures are built, it is necessary to consider the effect of non-synoptic winds, specifically downburst winds. Both northern and coastal areas of North America are subjected to downburst events which produce drastically different wind loads due to higher ground shear and result in high turbulence intensities. While there are some studies that evaluate downburst wind loads on low-rise buildings, there have been no studies that consider the effect of downburst-induced winds on elevated structures nor the effect of stilt height on surface pressure distribution and magnitude.

1.4 Research Objectives

Given the rapidly increasing rate of extreme winds around the globe, it is evident that there is a growing need to address the impacts of wind loads on elevated structures. Elevated structures subjected to extreme winds are at a higher risk of damage compared to their ground-mounted counterparts. While efforts have been made to evaluate the aerodynamics of elevated structures, the impact of winds on the structural loading of such buildings requires further investigation. Additionally, downbursts produce winds that differ significantly from ABL winds; however, there remains insufficient data regarding the aerodynamics of downburst winds on various building types, including elevated structures.

The main research gaps addressed in this study include:

- Lack of sufficient ABL wind testing data and building code provisions for elevated buildings.
- Insufficient information regarding downburst winds on buildings considering the frequency and complexity of their flow structure.
The main objectives of the present study include:

- Evaluate the impact of stilt height and roof slope on local and area-averaged pressure coefficients under ABL winds on an elevated low-rise building model.

- Investigate aerodynamic loading on an elevated low-rise building model under an experimentally produced downburst-like impinging jet.

- Assess the adequacy of ASCE 7-22 components and cladding design pressures for elevated structures and provide recommendations for NBCC design provisions.

The current study utilizes the Boundary Layer Wind Tunnel Laboratory (BLWTL) and the Wind Engineering, Energy, and Environment (WindEEE) Dome at Western University to simulate ABL and downburst-like winds, respectively, and measure their resulting pressures on a high-frequency pressure integration (HFPI) model. The low-rise gable roof building model was tested at seven stilt heights, including a ground-mounted case used as the baseline test. The surface pressure measurements were used to calculate external pressure coefficients in accordance with the wind loading provisions in the National Building Code of Canada (NBCC) 2020 as well as ASCE 7-22. The local and area-averaged pressure coefficients were investigated and compared to current building code design pressures.

1.5 Thesis Overview

An overview of each chapter within this thesis is summarized as follows:

- Chapter 1: includes the introduction, research gaps, objectives, and thesis organization of the thesis.

- Chapter 2 includes the testing and data analysis methodology, results, and discussion of the ABL wind loads. Included is a detailed description of the wind tunnel testing at the BLWTL, including the building model, scaling, and experimental setup, and configuration. The analysis methodology and equations used to calculate external pressure coefficients are provided. Finally, the local and
area-averaged external pressure coefficients are presented and discussed, including comparisons to NBCC 2020 and ASCE 7-22 design pressures.

- Chapter 3 contains the testing methodology, results, and discussion of the downburst simulation at the WindEEE Dome. This chapter includes the test setup and building configuration, data analysis methodology, local and area-averaged external pressure coefficients, and a comparison with the ABL wind loads and ASCE 7-22 provisions.

- Chapter 4 concludes the study and summarizes key findings, limitations, and recommendations for future work.
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Chapter 2

2 Atmospheric Boundary Layer Wind Loads on Elevated Low-Rise Buildings

2.1 Introduction

Elevated structures have been adopted as a method of increasing infrastructure resilience to withstand adverse impacts of climate change such as thawing permafrost in arctic communities and storm surges in coastal regions. The elevation of structures using a stilt system introduces an air gap between the base of the building and the ground which changes the aerodynamics of elevated buildings compared to their ground-mounted counterparts. This ventilation gap creates negative pressures on the floor underside of the building which has been proven to result in structural and building envelope damage under extreme winds. Post-hurricane damage assessments in the United States have reported damage to the roofs, walls, and underside of elevated buildings (Marshall, 2006; Marshall, 2010; FEMA, 2005). Roof damage included the loss of shingles and, in extreme cases, the complete uplift of the roof from the base of the structure. Wall damage involved the rotation of walls inwards or outwards about the base of the buildings as a result of insufficient connections at the intersection of walls and floor. The failure of gypsum board, plywood, and vinyl sheathing on the floor underside of elevated structures was widely reported and attributed to extreme winds. Despite clear evidence of damage to elevated buildings under high wind speeds, there remains a lack of quantification of wind loads on elevated buildings. The current building code in Canada (NBCC 2020) does not provide design wind loads for elevated structures. However, recently, the building code in the United States (ASCE 7-22) has been updated to include design wind loads for both component and cladding as well as the main wind force resisting system (MWFRS) for elevated buildings. While the codification of wind loads on the base of elevated structures demonstrates progress towards the increased resiliency of elevated structures, the effect of stilt height on surface pressures remains widely uninvestigated.

Researchers have used experimental wind tunnel testing to evaluate the aerodynamic loading on elevated structures. Holmes (1994) used wind tunnel tests to conduct a
parametric study on elevated houses typical to Australian architecture. The author investigated the effect of building elevation, roof pitch, and wind direction on the pressure magnitude and distribution on single-story gable-roof elevated house at both 1:50 and 1:100 scale. The elevated model showed increases in pressure on the walls and roof of the building compared to the ground-mounted building and reported an estimated 40-80% increase in surface pressures when the stilt height increased from ground-level to 2.1 m. The floor pressures on the building model were, however, not evaluated as this was outside of the scope of the project.

More recently, The Wall of Wind (WoW) Experimental Facility (EF) at Florida International University has been used to evaluate wind loads on elevated buildings typical to coastal United States. Chowdhury et al. (2017) conducted an aerodynamic loading assessment at the WoW EF for a low-rise building model at 1:5 scale with four stilt heights. The authors reported a 78% increase in peak lateral force coefficients and a 107% increase in mean moment coefficients about the lateral axis for the highest stilt height, 3.66m, compared to the ground-mounted model. Kim et al. (2020) assessed post-hurricane damage surveys from Hurricane Michael (Sutley et al., 2019) and compared field observations to those of wind tunnel tests conducted at the WoW EF. The authors conclude that the fluctuation of roof pressure with stilt height was negligible; however, the two highest stilt heights resulted in high magnitude peak pressure coefficients on the floor underside of the building. These magnitudes were comparable to those on the roof and in some cases nearly 60% greater than those projected onto the same location of the roof. The findings were found to follow similar damage patterns to cladding damage from Hurricane Michael. (Abdelfatah et al., 2020) used WoW testing to explore an area-averaging procedure to determine component and cladding design zones on the floor underside of elevated structures. The authors reported an increase in suction from -1.2 to -1.9 on the interior area of the base for full-scale stilt heights of 0.6 m and 3.65 m, respectively. This research was continued in 2022 and incorporated an additional stilt height as well as the development of an empirical formula to estimate the zone width on the floor underside of elevated structures as a function of stilt height (Abdelfatah et al., 2022). The experimental results were supplemented with Computational Fluid Dynamics (CFD) simulations to assess airflow around the building. The authors reported that high suction along the edges of the
base were caused by winds acting perpendicular to the roof ridge and oblique wind directions were responsible for the high suction around the stilts. The authors recommended future work to incorporate different configurations of elevated buildings including a variation of stilt arrangements to validate and contribute to the quantification of external pressure coefficients on the floor surface.

The impact of stilt height on elevated building surface pressures has received greater attention as climate change continues to increase the intensity and frequency of extreme winds. Some efforts have been made to quantify wind loads on elevated structures; however, these studies are limited to building geometry and stilt height, distribution, and shape. Due to the lack of quantification of surface pressures on elevated structures, the first objective of the current study is to evaluate the local external pressure coefficients on an elevated low-rise building model using a boundary layer wind tunnel to simulate ABL winds. The impact of roof pitch and stilt height is investigated using the peak enveloped pressure coefficients across the surfaces of the building model to determine a correlation between stilt height and local pressures. Additionally, in light of the recent introduction of wind load provisions for elevated structures in ASCE 7-22, an independent analysis of the area-averaged external pressure coefficients on an elevated building model would be beneficial to the wind engineering community to comment on the effectiveness of the new provisions. Thus, the second goal is to compare area-averaged pressure coefficients on a low-rise gable-roof building model to the component and cladding design curves in ASCE 7-22 to assess its current performance. The current study aims to contribute towards the quantification of wind loads on elevated structures as well as assess the effectiveness of current building codes.

2.2 Boundary Layer Wind Tunnel Test of Elevated Buildings

This section contains a detailed overview of the methodology employed to produce aerodynamic data used in the analysis of atmospheric boundary layer (ABL) wind loads on an elevated low-rise building model. In Section 2.2.1, the selection and geometry of the case study building are discussed. Section 2.2.2 provides a description of the test cases and parameters for the current study. Next, Section 2.2.3 discusses the exposure conditions and
flow field characteristics used in the study. Finally, Section 2.2.4 outlines the methods used to analyze the surface pressures on the building model.

2.2.1 Case Study Building

The building selected for the current study is representative of an elevated public housing unit similar to that proposed by Debicka & Friedman (2009). The proposed public housing unit was designed with a level of flexibility to address the existing shortage of quality housing in the Canadian Arctic. The number of row-housing in northern Canada is gradually rising to accommodate the increase in population with limited funding. For example, from 2011 to 2021, the number of row houses in Nunavut increased by over 35% (Statistics Canada, 2021, 2011 Census of Population). Therefore, this structure type was selected for the current study as it represents a fast-growing building sector. The building was based on a two-story and four-unit row house, with plan dimensions of approximately 23.8 m and 15.8 m in full-scale. Similar architecture and floorplans are observed in coastal communities throughout the United States. Two roof pitches are analyzed in the current study: 3:12 (14.0°) and 6:12 (26.6°). The structure is elevated from the ground and a simple stilt system is employed where one stilt is found on each corner of the base of the building and the effect of height on wind loads is studied.

A high-frequency pressure integration (HFPI) gable roof model with two roofs was 3D printed at Western University’s Machine Services facility. The model consisted of one base and two exchangeable roofs which could be alternated and fastened using screws to secure the roofs. The building model was printed using acrylic powder for the structure and aluminum tubes for the stilts which were secured below the baseplate. With a length scale of $\lambda_L$ of 1:100, the building model had plan dimensions of 23.8 cm by 15.8 cm, an eave height of 7.3 cm, and roof pitches of 3:12 (14.0°) and 6:12 (26.6°). The model was fitted with 342 and 350 pressure taps for the 3:12 roof pitch and 6:12 roof pitch respectively. The pressure taps consisted of PVC tubing that was threaded through the four hollow stilts on the model and connected to the pressure system below the wind tunnel to ensure no interference with the wind flow around the model. Each of the pressure connections was comprised of 30 cm long PVC tubes with 1.35 mm inner diameter, connected to two
resistors and 33 cm long PVC tubes with 1.35 mm inner diameter which was connected to solid-state high-speed pressure scanners. Each of the pressure taps was connected to one of 22 pressure scanners with 16 pressure ports each. The pressure system captured measurements at a frequency of 500 Hz and the raw pressure time series were low-pass filtered at a cut-off frequency of 200 Hz before being used in the analysis. Additional details, such as tubing response, of the experimental system can be found in Ho et al. (2005). The building model and wind direction details are illustrated in Figure 2-1 while the tap layouts are shown in Figure 2-2 a) and b). The experimental model set up in the wind tunnel is shown in Figure 2-3 and the pressure tubes on the building model extending through the aluminum stilts are shown in Figure 2-4. Finally, photos of each roof pitch and stilt height are shown in Table 2-1.

Figure 2-1: Geometric detail schematic of building model and wind direction key.
Figure 2-2: Building dimensions and tap layout for 3:12 roof slope (a) and 6:12 roof slope (b) building models at model-scale.
Figure 2-3: Building model with 3:12 roof slope in Western University’s Boundary Layer Wind Tunnel Laboratory Tunnel II with open terrain conditions.

Figure 2-4: Building model with pressure tubes extending through the stilts.
Table 2-1: Model setups at different stilt heights in Western University's BLWTL.

<table>
<thead>
<tr>
<th>$h_s$ (cm)</th>
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<th>6:12</th>
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2.2.2 ABL Test Cases

The effect of stilt height on building surface pressures was investigated by varying the stilt height from 0 m, referred to as the “ground-mounted” case, to 3 m at 0.5 m in full-scale. These heights cover the range of stilt heights observed in northern Canada as well as coastal United States. At each of the seven stilt heights, pressure measurements were taken at twenty-one wind angles from 0° to 180° at 10° increments with an additional two measurements taken at 45° and 135°. The turntable was rotated in a clockwise direction such that the high-density roof taps were rotated into the direction of the upstream wind. The wind direction of wind relative to the building model is depicted in Figure 2-1. Each test case was performed for a total of 120 seconds. The total number of test configurations was 294; a summary of the test configurations is shown in Table 2-2.

<table>
<thead>
<tr>
<th>Table 2-2: Test parameters for the atmospheric boundary layer wind tunnel tests.</th>
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<td><strong>Model Scale</strong></td>
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<td><strong>Sampling frequency</strong></td>
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<td><strong>Sampling period</strong></td>
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<td><strong>Wind angles</strong></td>
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<td><strong>Heights</strong></td>
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<td><strong>Exposure conditions</strong></td>
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<td><strong>Average reference velocity</strong></td>
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<td><strong>Roof height turbulence intensity</strong></td>
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2.2.3 Exposure Conditions

Pressure coefficients on a low-rise gable roof building model with two roof slopes were measured using the BLWTL Tunnel II at Western University. The tunnel measures 39 m in length, 3.4 m in width, and 2.5 m in height and contains roughness elements dispersed throughout the length of the tunnel to simulate the desired terrain conditions and wind characteristics. The current study utilizes open terrain conditions to best simulate northern
Canadian topography where structures are surrounded by very few obstacles such as buildings or vegetation. The open terrain condition corresponds to a full-scale roughness length of 0.012 m. A sawtooth trip, five-foot red spires, and a 15” barrier are placed at the entrance of the tunnel after the contraction before the long stretch of roughness blocks to match the characteristics of the ESDU profiles. The roughness blocks have a nominally uniform height of 7.6 cm for 17 m (Banks 1 to 5) furthest upstream and 2.5 cm for 12.2 m (Banks 6 to 10) immediately upstream of the building model. It should be noted that the height of the roughness blocks relative to the building model is much larger in model-scale compared to full-scale and, as such, the roughness blocks are discontinued at 2 m from the front face of the building model to avoid the aerodynamic signature of individual blocks reaching the model.

The simulation of the open exposure at the building site was based on the wind characteristics described in ESDU 82026 and 85020 for mean wind speed profile and turbulence intensity respectively. The ESDU mean wind speed and turbulence intensity profiles were compared with sixteen pre-calibrated profiles of the BLWTL for open exposure, and the best match for the terrain condition was selected for the current study. The profiles used for the current study are plotted with the ESDU profiles as shown in Figure 2-5, where the y-axis is normalized by the mean mid-roof height of the building model, h. Since the pressure coefficients are referenced to the mid-roof height of the building, matching the turbulence intensities and mean wind speeds within this region was critical. Additionally, the reduced spectral density for the longitudinal wind angle is plotted in Figure 2-6 along with the corresponding ESDU spectrum. The profile measurements were taken in the wind tunnel without the building model present and the wind tunnel spectrum is shown to match that of ESDU reasonably well.
Figure 2-5: Wind speed and turbulence intensity profiles for open exposures at 1:100 scale.
2.2.4 Aerodynamic Data Analysis

2.2.4.1 Pressure Coefficient Calculation

The pressure data obtained at the BLWTL was non-dimensionalized into external pressure coefficients, $C_{p,ref}$, in accordance with Equation (2-1):

$$C_{p,ref} = \frac{p(t) - p_0}{\frac{1}{2} \rho V_{ref}^2}$$  \hspace{1cm} (2-1)

where, $p$, is the measured pressure on the face of the building as a function of time, $p_0$ is the static pressure, and $V_{ref}$ is the reference velocity at pitot height in the wind tunnel. The raw data obtained from the wind tunnel tests used a reference velocity taken as the mean wind speed at an upper level in the wind tunnel where the flow is uniform and has low turbulence. However, pressure coefficients used in building codes are referenced to the...
mid-roof height; therefore, the aerodynamic data were re-referenced to eave height by applying a conversion factor shown in Equation (2-2).

\[
C_{p,H_{ABL}} = C_{p,ref_{ABL}} \left( \frac{V_{ref}}{V_{h}} \right)^2
\]  

(2-2)

where, the subscript \( h \) denotes the mid-roof height of the building.

Both the local and area-averaged pressure analyses on the building model were conducted using 3-second external pressure coefficients (GCp). The results obtained from the wind tunnel were referenced to the mean hourly wind speed in open terrain (\( V_{3600} \)), which was referenced to a 3-second gust by applying a wind tunnel factor as described in St. Pierre et al. (2005). The wind tunnel factor, \( F_{WT} \), was made using Equation (2-3) below:

\[
F_{WT} = \left( \frac{V_{3600 \ s,\ open}}{V_{3 \ s,\ open}} \right)^2
\]  

(2-3)

The factor applied in this case was equal to 0.43, which is based on a Durst factor of 1.53 (ASCE 7-22), and no additional adjustments to the data were required.

The peak pressure coefficients were computed from Gumbel (Type I) distribution fitted from a number of observed extreme values rather than the single observed maximum and minimum values. The Lieblein (1974) best linear unbiased estimator (BLUE) method is employed to obtain the Gumbel parameters by fitting a ranked set of observed minima and maxima. The recorded time series is divided into ten equal epochs to collect the observed minima and maxima for the fitting. The probability of non-exceedance is taken as the 78\(^{th}\) percentile, a value commonly employed for determining statistical peak pressures on low-rise buildings (Kopp & Morrison, 2018; Gavanski et al., 2013; Cook & Mayne, 1980). Once the Gumbel distribution is obtained through the BLUE method, the peak corresponding to the duration of the epoch, which is one-tenth of an hour (i.e., 6 minutes), is calculated as the 78\(^{th}\) percentile value of the distribution. Then, the peak estimate for the full one hour is calculated using the method proposed by Cook & Mayne (1980).
The velocity scale, \( \lambda_v \), was calculated as the ratio between the mean hourly wind speed at 10 cm in the wind tunnel (corresponding to 10 m at full-scale), \( V_{WT10} \), and the 50-year mean hourly wind speed at the study location, as reported in the NBCC (2020), \( V_{NBCC10} \), as shown in Equation (2-4):

\[
\lambda_v = \frac{V_{WT10}}{V_{NBCC10}} \tag{2-4}
\]

The reported 50-year mean hourly wind speeds reported in NBCC (2020) were taken at 10 m height in open exposure terrain. The location used for the current study was Iqaluit, Nunavut, and the 50-year mean hourly wind speed is reported as 31.7 m/s (NBCC, 2020). The velocity scale calculated for the current study is 1:2.7. The corresponding time scale is calculated in accordance with Equation (2-5):

\[
\lambda_T = \frac{\lambda_L}{\lambda_v} \tag{2-5}
\]

where, \( \lambda_L \) is the length scale and \( \lambda_T \) is the time scale. Thus, the time scale for the current model is 1:36.5. Therefore, approximately 100 seconds in the model-scale correlates to an hour in full-scale. The 120 second long pressure measurements taken in the wind tunnel were thus truncated to 100 seconds to represent mean hourly pressure coefficients.

### 2.2.4.2 Area-Averaging Methodology

The area-averaged pressure coefficients are calculated from weighted average pressure time history within a pre-specified averaging area. The equation used to calculate the area-averaged pressure coefficient time histories is given in the equation below:

\[
C_{p,\text{avg}i}(t) = \frac{\sum(C_{p,i}(t) A_i)}{\sum A_i} \tag{2-6}
\]

where, \( C_{p,i}(t) \) is the pressure coefficient time history and \( A_i \) is the tributary area for the \( i^{th} \) tap. Here, the pressure coefficient time histories have been referenced to mid-roof height in accordance with NBCC (2020) and ASCE 7-22, and the wind tunnel factor was applied such that the data represents the 3-second gust pressure coefficients (GCp). However, the
design pressures in the NBCC (2020) and ASCE 7-22 cannot be compared directly as the former utilizes a mean hourly pressure coefficient for the design pressures (CgCp), whereas the latter uses a 3-second gust pressure coefficient (GCp) by applying the wind tunnel factor described in Equation (2-3) to ensure an equivalent comparison of the wind loads in the current study to both NBCC and ASCE design curves. The NBCC curves have been converted to 3-second gust pressure coefficients (GCp) by applying the wind tunnel factor described in Equation (2-3). Additionally, to compare the CgCp values from NBCC with GCp values from ASCE, the directionality factor embedded in the CgCp values was removed as this factor is not included in the ASCE components and cladding curves, but separately applies as the so-called factor Kd. The effects of directionality have been negated by applying a factor of $\left( \frac{1}{0.85} \right)$ to the NBCC design curves, a method adopted by Alrawashdeh & Stathopolous (2015). Therefore, the data presented represent GCp values calculated using the following expression:

$$GCp = C_{p_{3600s}} \times \left( \frac{V_{3600s,open}}{V_{3s,open}} \right)^2 \times \left( \frac{1}{K_d} \right)$$

(2-7)

The equivalent design pressure coefficients are shown in Figure 2-7.
The enveloped pressure coefficients in the current study were obtained by taking the worst peak external pressure coefficients over all twenty-one wind directions. The areas upon which the pressure coefficients were averaged were based on the effective wind areas used in the design pressure curves for NBCC 2020 and ASCE 7-22. The smallest averaging area, and thus the area most susceptible to higher averaged wind loads, is taken as the minimum tributary area of all faces. The maximum area averaging cell size was taken as the largest area of interest from the NBCC and ASCE envelope curves for each of the building zones. Due to disparities between the size of the averaging areas and the size of the building surfaces, there were gaps on each surface that were not captured by a whole averaging area panel. Thus, the base point for the area averaging grid generation was set to the windward building face corner to ensure the maximum peak pressures were represented. It should be noted that due to the tap density across the building model, only one tap was used in some cases to compute the peak pressure coefficients whereas it is typically recommended to use at least four pressure taps for each area-averaging zone (Kopp and Morrison, 2018).
results of the enveloped peak minimum and maximum pressure coefficients using the area-averaging procedure are presented for individual surfaces in Sections 2.3.2.

2.3 Results and Discussion

This section presents the results of the ABL-induced pressure measurements on both the ground-mounted and elevated building models for both roof slopes. An analysis of the statistical peak minimum and maximum local pressure coefficients is presented for the seven tested building heights. The effect of roof slope and building height on local peak pressures is also examined. Additionally, the pressure measurements are area-averaged to provide a component and cladding analysis, and comparison with the current NBCC components and cladding provisions for the roof and wall surfaces are made. Finally, the area-averaged base loads are calculated and compared to design pressures provided in the ASCE 7-22. All results in this section, except for comparisons to previous studies, which compares wind tunnel results without any conversion of gust durations, are presented in 3-s gust duration form with the application of Equation (2-7).

2.3.1 Building Geometry Effects on Local Pressures

2.3.1.1 Comparison to Previous Studies

The current model was validated using data reported by the National Institute of Standards and Technology (NIST) on the aerodynamic database which contains wind tunnel measurements on various building models from the BLWTL (Ho et al., 2005). Similar test setups were employed for both studies, although there were some geometric differences between the two building models. The normalized velocity and turbulence intensity profiles from NIST and the current study were plotted and presented in Figure 2-8. A comparison between the external pressure coefficients on the roof surfaces from the current study and the NIST database is provided to demonstrate the precision and adequacy of the current model. The wind characteristics at the maximum mean roof height and building dimensions for both studies are presented in Table 2-3. The NIST building model that presented the closest match in eave height, roof slope, and plan dimensions to the current model was selected for comparison.
Figure 2-8: Normalized velocity and turbulence intensity profiles from NIST and the current studies.
Table 2-3: Wind tunnel testing characteristics and building model for NIST and the current study.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>NIST</th>
<th>Current</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan Dimensions (m)</td>
<td>24.4 x 38.1</td>
<td>15.8 x 23.8</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>1:1.6</td>
<td>1:1.5</td>
</tr>
<tr>
<td>Eave Height (m)</td>
<td>12.2</td>
<td>7.3</td>
</tr>
<tr>
<td>Height : Width Ratio (h/w)</td>
<td>1:2</td>
<td>1:2.2</td>
</tr>
<tr>
<td>Roof Slope</td>
<td>3:12; 6:12</td>
<td>3:12; 6:12</td>
</tr>
<tr>
<td>Mean-Roof Height Turbulence Intensity (%)</td>
<td>16.8</td>
<td>15.9</td>
</tr>
<tr>
<td>Mean-Roof Height Velocity (m/s)</td>
<td>9.1</td>
<td>11.7</td>
</tr>
</tbody>
</table>

The mean external pressure coefficient contours on the roof surfaces for two roof slopes are shown in Figure 2-9 while the roof midline variation for both models are shown in Figure 2-10. The pressure coefficients from both studies were referenced to the mean roof height of the respective building models. These contours clearly demonstrate very similar magnitudes and distributions in pressure coefficient on the respective building models. The lower roof slope resulted in higher suction along the edge and ridge of the model compared to the steep slope. The lower roof slope allows for the separation of airflow at the edge and ridge of the building and is found to attach to the leeward face of the roof on the current model. The flow on the higher roof slope separates at the ridge and does not attach on the leeward side of the roof for either model. The midline plots show minimal differences between the magnitudes of mean and peak pressure coefficients across the roof between the two building models. The minor discrepancies between the two tests can be attributed to differences between the building geometry and terrain modelling, as the NIST model was tested in open exposure conditions corresponding to a roughness length of 0.01 m, whereas the current study corresponded to 0.012 m. The resemblance in pressure
distribution and magnitude between both models validates that the methodology of the current study is sound.

![Image of pressure coefficient contours](image)

**Figure 2-9:** Mean external pressure coefficient contours on NIST and current building model for 3:12 and 6:12 roof slopes at 90° wind direction.
2.3.1.2 Effect of Roof Slope on Roof Pressures

As Gavanski et al. (2013) highlight, the effects of the gable roof slope on wind-induced pressure coefficient distributions and their associated aerodynamics and vortex formation have been studied by many researchers (Holmes and Best, 1979; Stathopolous, 1979, Kanda and Maruta, 1993, as cited in Gavanski et al., 2013). These authors note that the separation and reattachment zones for mild to steep slope roofs have been found to vary significantly. When mild slope gable roofs are faced perpendicular to the wind direction, the flow separates at the leading edge, reattaches on the windward roof, and separates again at the ridge of the roof, whereas steep slope roofs experience positive pressures on the entire windward roof face, and uniform suctions on the leeward roof face indicating that the flow separates at the ridge (Reardon and Holmes, 1981; Holmes, 1983; Stathopolous, 1984, Gavanski et al., 2013). The current study examines these features using a mild and steep slope gable-roof building.

The effect of the roof slope is assessed using a building model with two different roof slopes, 3:12 and 6:12 (14° and 27° respectively). To evaluate the effect of roof slope alone, the exposure conditions and building height remain constant. Figure 2-11 shows the mean...
and standard deviation of the pressure coefficients on the roof of the two building models in open terrain exposure for the ground-mounted test case. Here, five angles are examined (0°, 30°, 45°, 60°, and 90°). It is clear that when the wind direction is perpendicular to the ridge of the building (i.e., θ = 90°), the flow features described by Gavanski et al. (2013) are observed. The lower slope of the roof creates separation at the leading edge of the building, as shown by the suction directly above the edge of the building. The flow reattaches near the middle of the roof and again separates at the ridge. In contrast, the steeper slope displays positive pressures on the windward face of the roof and minor suction on the leeward face. For both θ = 45° and 90°, the lower slope roof experiences suction on the entire windward face, whereas the steep roof shows almost entirely positive pressures. The largest difference in pressure coefficients between roof slopes is shown in the θ = 45° case. Here, the lower slope roof experiences suction across both faces of the roof with high magnitudes of localized suction on the leeward side of the ridge in the order of -0.8. The increase in roof slope results in a reduction in the suction of over 50%.

The standard deviation plots shown in Figure 2-11 (b) represent the wind pressure fluctuations on the roof of the building. The lower slope roof displays higher standard deviations compared to the steep slope roof, particularly for the oblique wind directions. The highest standard deviation of all presented cases is 0.4 and is found on the windward corner of the 3:12 roof slope with a wind direction of 30°. The corresponding point on the steep roof building has a standard deviation of 0.3. Both roof slopes under the 30° wind direction show similar trends in their standard deviation distribution, with the highest values at the windward corner and dissipating towards the ridge of the roof. The high standard deviation is indicative of higher turbulence and is likely associated with corner vortices which are more prominent on the low-slope roof.
Figure 2-11: Mean pressure coefficient (a) and standard deviation (b) on the roof of a ground-mounted building at different wind directions.

Figure 2-12 shows the peak minimum and peak maximum pressure coefficients on both roof slopes for a 0° wind direction. The highest suction is found on the corner of the 3:12 pitch roof and is equal to -2.2. In contrast, the highest negative pressure coefficient for the 6:12 pitch roof is -1.0. These higher suction values are likely due to the increased flow velocity across the windward edge of the lower slope roof, as the pitch does not significantly impede the flow. The obstruction of flow on the windward face of the steep roof can be seen in the peak maximum pressure plots. The highest peak maximum pressure...
coefficients on the mild and steep slope roofs are 0.2 and 0.5, respectively. The higher positive pressure coefficients on the steep roof are due to the higher impact of flow on the surface as the roof is angled into the flow more directly. The lower slope does not obstruct the flow to the same extent, therefore, resulting in lower positive pressures.

![Figure 2-12: Peak minimum and peak maximum 3-s gust pressure coefficient contour plots on 3:12 and 6:12 roof slopes at a 90° wind direction.](image)

The variation of peak minimum and maximum and pressure coefficients across the midline of the roof face for a 90° wind direction are plotted in Figure 2-13 and Figure 2-14, respectively. All three parameters show that the lower roof slope results in higher negative pressures on the windward face of the roof. The mean and peak minimum pressure coefficients on the leeward face of the roof decreased with distance from the ridge for the steep slope roof, whereas the mild slope roof experienced lower negative pressure coefficients away from the ridge. The most prominent difference between the two roof slopes is shown in the peak minimum pressure coefficients. The lower roof slope model produced peak minimum pressure coefficients 57% greater than those on the higher roof.
slope on average for this particular test configuration. The highest peak negative pressure on the 3:12 roof slope was approximately -1.8, whereas the corresponding tap on the 6:12 roof model resulted in a peak minimum pressure coefficient of approximately -0.4. The difference between peak maximum pressure coefficients on both roof slopes was not as significant as that from the peak minimum pressure coefficients. The higher roof slope produced higher peak maximum Cp values on the windward face compared to the lower roof slope, and vice versa on the leeward face of the roof surface. The mean pressure coefficient variation showed similar qualitative trends to the peak minimum Cp values for both roof slopes.

![Figure 2-13: Peak minimum and peak maximum pressure coefficient variation across the roof midline for 90-degree wind direction.](image-url)
Figure 2-14: Mean pressure coefficient variation across the roof midline for 90-degree wind direction.

The enveloped peak minimum pressure coefficients provide an evaluation of the highest suction regardless of wind direction. This is useful in determining a worst-case scenario, as all wind directions are taken into consideration in the wind load analysis. The enveloped peak minimum pressure coefficients are calculated using twenty-one wind directions from 0° to 180°. The enveloped peak minimum pressure coefficients at a given tap refer to the highest absolute magnitude value of all peaks from the 21 wind directions. The enveloped peak pressure contour plots for both roofs are shown in Figure 2-15. From these contour plots, it is clear that the lower roof slope also resulted in higher suctions on the corners of the roof for all wind directions. The lowest pressure coefficient is -3.6 and observed on the 3:12 roof pitch and occurs at a wind direction of 20°. The higher roof slope resulted in higher suction across the leeward face of the roof. This is likely due to the flow separation from the ridge of the roof, which does not reattach on the back face. Contrarily, the lower slope experiences flow separation on the edge of the windward face, as shown by the higher negative pressures in this region. The large discrepancies between pressure magnitudes and distributions between the two reported roof slopes indicate that the design of wind pressures should be dependent on the roof slope.
2.3.1.3 Effect of Stilt Height on Surface Pressure Distribution

The stilt height of elevated low-rise buildings significantly impacts the airflow below and around the structure, thus affecting wind loading on such structures. Abdelfatah et al. (2020) used wind tunnel tests at FIU’s WOW to compute local peak pressure coefficients on an elevated house at three different heights and one ground-mounted model. The authors found that increasing the building elevation resulted in higher magnitudes of peak pressures and greater areas of suction on the floor underside. The variation of negative peak pressures on the floor as a function of height was more significant compared to positive peak pressures. Additionally, high suctions on the corners of the floor were reported and attributed to the vortex formation around the stilts. Similar observations were made by Kim et al. (2020), where wind tunnel testing was used to evaluate peak wind loads on an elevated building model. The authors reported high negative 3-s gust peak pressure coefficients of up to approximately -7 near the edges of the building for the two highest elevations (2.13 m and 3.66 m full-scale). Amini and Memari (2021) studied wind pressures and flow patterns around raised buildings, but their studies are limited to mean values, whereas peak values are required for wind design. These studies have provided a benchmark for the evaluation of wind loads on elevated buildings; however, further research is required to better understand the relationship between building elevation and resulting wind loads. The
effect of the building height on the aerodynamic wind loads of the building model is investigated in this section from both a semi-qualitative and quantitative approach.

**Semi-Qualitative Aerodynamic Trends**

Seven building heights were tested from 0 m to 3 m, full-scale, at 0.5 m increments. The local enveloped peak minimum and maximum pressure coefficients on all surfaces of the building with 3:12 and 6:12 roof slopes were computed for each of the seven heights and are presented in Figure 2-16 and Figure 2-17, respectively. The contour plots are calculated by interpolating pressure coefficients using the Nearest-Neighbour interpolation scheme implemented through the “scatteredInterpolant” function in MATLAB, with the triangulation theory outlined in Amidror (2002). It should be noted that the shape of the stilts in the current study is circular; however, the contour plots show square areas where the stilts are located. This is due to limitations in the tap density within these regions, as the interpolation method cannot accurately predict pressure coefficients surrounding a circular area.
Figure 2-16: Local peak enveloped minimum pressure coefficients for building heights 0 m (a), 0.5 m (b), 1 m (c), 1.5 m (d), 2 m (e), 2.5 m (f), 3 m (g)
Figure 2-17: Local peak enveloped maximum pressure coefficients for building heights 0 m (a), 0.5 m (b), 1 m (c), 1.5 m (d), 2 m (e), 2.5 m (f), 3 m (g).
For the wall surfaces of the ground-mounted case, maximum and minimum peak pressure coefficients were approximately 1.2 and -1.8, respectively. These pressures were observed on the east and south walls. It should be remembered that the building was rotated from 0 to 180 degrees, with the east wall being rotated into the flow. Thus, the west wall did not experience direct wind flow, and therefore, the east wall is expected to produce the highest positive pressure values. The positive wall pressures did not show considerable variation with height compared to the ground-mounted building. The highest negative wall pressure was -2.5 and was found on the east face of the building with a stilt height of 1.5 m full-scale. The negative peak pressure coefficients were found to be higher on the lower edge of the walls of the elevated models compared to the ground-mounted model. This observation was also made by Abdelfatah et al. (2020) and was attributed to the presence of the air gap.

From the contour plots shown in Figure 2-16 and Figure 2-17, the variation of roof pressures with building height remains insignificant for both peak minimum and maximum pressures. This agrees with the findings of Kim et al., (2020) and Abdelfatah et al., (2020), who reported no significant changes in roof pressure coefficients as the building height increased. The dominant negative pressures on the building model are found on the corners of the roofs and are within -3.0 to -3.2 for all building heights. The variation of enveloped peak minimum pressure coefficients across the roof midline of the 3:12 roof slope building model is shown in Figure 2-18. The x-axis represents the distance from the pressure tap to the edge of the roof, Δ, normalized by the width of the roof, W. All stilt-heights showed the same qualitative trends between the distance of the tap to the edge of the roof and the peak pressure coefficient. The pressure coefficients on the windward face become weaker towards the ridge and show a sharp increase in suction immediately behind the ridge. Here, flow separation occurs, which results in high suction in the order of -1.4 to -2.0. These pressures then decrease in magnitude with distance from the ridge, an indication that the flow has reattached onto the leeward face of the roof. The pressure taps near the ridge of the roof generally resulted in the lowest peak minimum pressure coefficients for all building heights. The highest suction values were exhibited in the 3 m stilt height near the ridge and north edge of the building. Near the center of both faces of the roof, the ground-mounted model exhibited the lowest magnitude of peak pressure coefficients. From these
plots, there is no clear relationship between the stilt height and peak pressure coefficient as a function of the distance from the edge of the roof. However, all stilt heights followed the same qualitative pattern across the roof midline.

![Peak Minimum Cp Variation on Roof Midline, AOA = 90°](image)

**Figure 2-18: Enveloped peak pressure coefficient variation across roof midline for all stilt heights.**

The enveloped peak negative pressure coefficients on the base of the building significantly vary, as shown in the base contour plots of Figure 2-19. As the building elevation increases, the area and magnitude of suction on the floor underside increase as well. The center of the floor underside experiences less suction compared to the edges and areas surrounding the stilts. The higher suction near the edges and stilts of the floor are consistent with observed cladding damage from field studies, as outlined by Kim et al. (2020). The observed damage to these areas of the floor includes fastener failure and panel loss which is likely attributed to the flow separation at the edge of the building. As the ventilation space beneath the building increases, the reattachment of flow on the base of the building occurs closer to the center, resulting in greater areas and magnitudes of suction on the floor surface. The maximum peak negative value on the base was recorded as -3.2 and occurred at the north edge of the surface immediately adjacent to the stilt. This value was obtained at a building
height of 2.5 m, which is also the height at which the maximum peak positive pressure coefficients are recorded. The enveloped peak positive pressure coefficients reach maximum values in the range of 0.72 and occur around the interior side of the stilts. As the building height increases, the enveloped peak maximum pressure coefficients around the center of the building also generally increase. These results are consistent with the findings by Abdelfatah et al. (2022), as the authors reported 3-second peak pressure coefficients on the base surface in the order of -3.8 and -4.0 for stilt heights of 2.15 m and 3.75 m full-scale.

![Figure 2-19: Enveloped peak minimum pressure coefficient distribution on base surface at all stilt heights.](image)

The enveloped peak minimum and maximum pressure coefficients across the base midlines provide an indication as to how the pressures vary with distance from the edge of the floor as well as to compare this variation across multiple building elevations. The widthwise and lengthwise midline peak pressure coefficient plots are shown in Figure 2-20. Here, the x-axis represents the distance, Δ, of the tap from the edge of the base normalized by the corresponding width, W, or length, L. It should be noted that the “midline” used in the lengthwise plots is not in the center of the base; rather it is offset by a factor of 0.1 due to the pressure tap distribution. Both midline plots show that the peak minimum pressure coefficients are highest near the edges of the base and are at their lowest values towards the center of the surface. In addition, the suction on the base generally becomes stronger
as the height of the building increases, as shown by the maximum peak pressure coefficients being recorded for the building heights between 2-3 m full-scale. Interestingly, this trend is reversed for the pressure taps closest to the long edge of the base. In this location, the building model at a full-scale height of 0.5 m recorded the highest suction, and as the building was raised, these values lowered. This is not common to the lengthwise midline plot, thus indicating that the flow near the edge of the base of elevated buildings is dependent on the length or geometry of the floor. The peak positive midline plots show that the highest positive pressures are found at the highest building elevation. The 3 m building elevation had the highest peak positive pressure coefficients, which were found near the center of the surface. The variation of peak positive pressure coefficients with distance from the building edge shows a much less pronounced relationship compared to its peak negative counterpart. The increase in building height from 0.5 m to 3 m resulted in increases of 77% and 83% for peak positive and negative pressure coefficients, respectively, at select locations.

Figure 2-20: Midline plots of peak negative pressure coefficients along the building base surface.
Quantitative Pressure Trends on Each Face

A quantitative approach was used to evaluate the impact of stilt height on the magnitude of pressures on each face of the building. A cumulative parameter, $R$, that compares the peak pressure for each tap relative to the corresponding tap on the ground-mounted model was calculated in accordance with Equation (2-8). This parameter quantifies the difference between each stilt height relative to the range of peak values from the seated model case. The average of all taps on each face was calculated and plotted as shown in Figure 2-21. The $R$-values for each wall surface were averaged as they each represent the same quantitative trends.

$$ R = \text{Avg.}\{C_{p}^{\text{stilted}} - C_{p}^{h}\} $$  \hspace{1cm} (2-8)

![Figure 2-21: Average R-value for surfaces of building model at each height.](image)

This plot demonstrates the relationship between stilt height and peak pressure coefficients on the base surface of the building model. As the stilt height increases, the magnitude of negative pressure also increases. The correlation coefficient between the stilt height and the average peak pressure coefficient on the base is 0.7, indicating that this is a significant correlation. This relationship can be attributed to both the flow underneath the building, which causes higher velocities and lower pressures, creating a Venturi effect which is defined as the reduction in fluid pressure when a fluid flows through a narrow opening. As
the ventilation gap beneath the building increases, the velocity decreases and pressure increases, creating a stronger downforce towards the ground. This trend is not observed on either the roof or the walls of the building model. The walls showed slightly higher R-values compared to those of the roof; however, neither showed any correlation between the R-value and stilt height. It can be concluded that the impact of stilt height on the wall and roof pressure coefficients is negligible. Contrarily, the base pressures are significantly impacted when the building model is elevated above the ground and increases as stilt height increases.

2.3.2 Component and Cladding Wind Loads

Local pressure coefficients are important for estimating the maximum point loads on a building model. However, for the purpose of wind load design applications, area-averaged pressure coefficients provide a better estimation of the potential for component and cladding damage. Additionally, observed wind damage to elevated structures is largely comprised of component and cladding failures (Marshall, 2016, 2010; Sutley et al., 2018). Therefore, the objective of this section is to evaluate the peak-area averaged wind pressure coefficients for the estimation of wind loads on the components and cladding zones in accordance with both the NBCC (2020) and ASCE 7-22 wind loading provisions. The analysis process is detailed in Section 2.2.4.2, and the results of the analysis are discussed in Sections 2.3.2.1 – 2.3.2.3. Finally, zoning recommendations for the external pressure coefficients on the floor of elevated structures are provided in Section 2.3.2.5.

2.3.2.1 Ground-Mounted Model

The first section of this analysis examines the area-averaged external pressure coefficients on the ground-mounted model for various effective wind areas, calculated in accordance with NBCC 2020. Therefore, these values represent external pressure coefficients referenced to the mean hourly wind speed at mid-roof height of the building, referred to as CgCp values. A comparison between the scattered CgCp values and the NBCC 2020 design pressure coefficient envelope curves are presented in Figure 2-22 and Figure 2-23. The black data points represent the highest negative pressures, and the blue represents the highest positive pressures.
For the corner zone, Zone C, the results show that the CgCp from the current study are generally within the code provisions, except for a few extreme cases with smaller averaging areas which exceeded the NBCC provisions. The edge zone, Zone S, resulted in a higher frequency of CgCp values which exceeded those of the NBCC design curve. The average negative CgCp value within this zone for averaging areas less than 2 m$^2$ was approximately -3.1, whereas the code value is -3.6. For the interior roof zone, Zone R, the results showed CgCp values higher than the provision. These trends are observed in a study by Chavez et al. (2022), wherein CgCp values on a gable-roof building from both a field study and experimental wind tunnel study are compared to current NBCC component and cladding provisions. The authors report higher CgCp values in the interior roof zone compared to the provisions; however, a consensus on the underestimation of pressures within this zone was not reached.

![Figure 2-22: External pressure coefficients, CgCp, versus NBCC (2020) design provisions for the ground-mounted 3:12 roof slope model.](image-url)
Figure 2-23: External pressure coefficients, CgCp, versus NBCC (2020) design provisions for the ground-mounted 6:12 roof slope model.

The increased wind loads on buildings are acknowledged in ASCE as shown by the continual amendment of component and cladding loads, such as those in ASCE 7-22. Therefore, the results from the current study are compared to both ASCE 7-22 and NBCC 2020 design pressures using the equivalent GCp values. A comparison between the scattered peak area-averaged external pressure coefficients calculated in accordance with ASCE 7-22 (GCp) on the ground-mounted models for various effective wind areas and the NBCC 2020 and ASCE 7-22 design pressure coefficient envelope curves is presented in Figure 2-24 and Figure 2-25. These envelope curves correspond to the case of a perfectly sealed building which is assumed to be the case for the current study. The results show that ASCE 7-22 is more conservative, encompassing a greater number of external pressure coefficients in the current study compared to the NBCC design curve. It should be noted that the ASCE 7-22 provisions were updated in 2016, while those of NBCC have not been updated. The ASCE design pressures for both Zone C and Zone S fit the data from the current study well, as they are higher than all but three data points. These data points are considered outliers, thus justifying their exclusion in the discussion of code adequacy. The interior roof zone, R, presented a greater number of data points which exceeded NBCC and
ASCE design pressures compared to the ridge and corner roof zones. It should be noted that ASCE 7-22 provides different components and cladding design pressures for gable-roof slopes between 7°-20° and 20°-27° while the NBCC provisions cover slopes between 7° and 27°. For the current study, the lower roof slope design pressures from ASCE 7-22 would be more satisfactory for the higher roof slope data as 6.2% and 2.6% of the data points from the steep and mild interior roof zones, respectively, exceeded the ASCE design pressures. The interior roof zone design pressures were particularly insufficient for tributary areas greater than 3 m². Given the typical sizing of plywood or oriented strand board (OSB) is approximately 3 m², the underestimation of design loads for these tributary areas may contribute to cladding damage under extreme winds. Both the NBCC and ASCE positive design pressures were adequate for the mild roof slope; however, there was an exceedance of both codes for the steep roof slope. 22.3% of the positive GCp values on the interior roof zone exceeded the NBCC design pressures, whereas 2.7% exceeded those of ASCE. The higher positive pressures on the interior roof zone of the steep roof slope building are due to the increased horizontal velocity component onto the roof surface under winds perpendicular to the roof surface. Steeper roof slopes are more common in northern Canada to prevent snow accumulation on the roof. Therefore, both the negative and positive design pressures for higher roof slopes may require further refinement.

The NBCC and ASCE design pressures for the wall surfaces of both mild and steep roof slopes were lower than the current results. The 3:12 roof slope model had significant exceedance of both provisions for the edge and interior wall zones, E and W, respectively. The percentage of data which exceeded the NBCC and ASCE envelope curves was calculated by dividing the number of data points that were outside of the bounds of the provisions by the total number of data points, multiplied by 100. For the mild slope building, 82.8% and 49.7% of the total data points exceeded the NBCC and ASCE design pressures, respectively, for their corresponding averaging area. Comparatively, 59.5% of the positive GCp values on the edge wall zones exceeded NBCC design pressures on the steep slope, whereas there was a 13.5% exceedance from the ASCE provisions. The interior wall zones also had significant exceedance of GCp values from those presented in both NBCC and ASCE 7-22. A complete summary of all positive and negative GCp exceedances for the current study is presented in Table 2-4.
Figure 2-24: External pressure coefficients versus NBCC (2020) and ACSE 7-22 design provisions for the ground-mounted 3:12 roof slope model.

Figure 2-25: External pressure coefficients versus NBCC (2020) and ACSE 7-22 design provisions for the ground-mounted 6:12 roof slope model.
Table 2-4: Summary of the total percentage of GCp data points which exceed NBCC and ASCE design pressure for each zone.

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<th>Zone</th>
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<th>Negative GCp</th>
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</thead>
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<td>6:12</td>
</tr>
<tr>
<td></td>
<td>NBCC</td>
<td>ASCE</td>
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<tr>
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</tr>
<tr>
<td>S</td>
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</tr>
</tbody>
</table>

2.3.2.2 External Pressure Coefficients for All Silt Heights

The highest non-outlier value was calculated and plotted against the NBCC and ASCE design curves, as shown in Figure 2-26 and Figure 2-27, to evaluate the effect of stilt height on the area-averaged pressures for each zone. An outlier was defined as a value which exceeds three standard deviations from the mean for each dataset. These plots clearly outline the level of effectiveness of the NBCC and ASCE design envelopes for each zone and roof slope for all stilt heights. There is no clear relationship between the stilt height and area-averaged pressure coefficients on the roof and wall zones. As such, the
observations reported in Section 2.3.2.1 for GCp within each component and cladding zone are also observed here. The envelope curves for both ASCE and NBCC performed adequately for the corner roof zone, C, on both 3:12 and 6:12 roof slopes. The design pressures for the edge and roof zone, S, were underestimated for the higher roof slope, and the interior roof zone GCp values were underestimated for both roof slopes. The wall design pressures were inadequate for both roof slopes and all stilt heights.

Figure 2-26: Highest non-outlier GCp values for all stilt heights on 3:12 roof slope model.
To further investigate the effect of stilt height on the area-averaged pressure coefficients on the roof and wall zones, the deviation of the highest non-outlier GCp values from the NBCC curve was calculated and plotted in Figure 2-28. This calculation was made by taking the average of the absolute difference between each external pressure coefficient and the corresponding value from the NBCC curve for every zone and every height. The deviation from the design curve is a measure of how closely the results from the current study match the provisions in NBCC 2020. This method also allows for the impact of stilt height on the external pressure coefficient relative to the building code to be quantified. The absence of a slope for the corner roof zone plots indicates that there is no correlation between the stilt height and area-averaged pressures within this region. There is a slight slope on the edge and ridge, as well as interior roof zones, indicating a mild correlation between stilt height and deviation from the NBCC curve values within these regions. The correlation coefficients between stilt height and deviation of the curve for the mild roof slope are 0.3 for both Zone S and Zone R. This suggests that as stilt height increases, the effectiveness of the NBCC envelope curves decreases slightly. The respective correlation coefficients between stilt height and deviation from the NBCC curve for Zone S and R on
the steep slope model are 0.5 and 0.6, indicating a stronger relationship between the two variables for the steeper roof slope. The difference in wall GCp values as a function of the tributary area is not as evident compared with the roof slope. Therefore, the roof slope does not impact the wall wind loading for elevated structures. The deviation of data from the NBCC curves only shows a moderate relationship for the interior wall zone of the steep roof slope, as a correlation coefficient of 0.6 was calculated. The results of this analysis indicated that the component and cladding design pressures for roof and wall zones are under-conservative and may require dependency on the stilt height for elevated structures.

Figure 2-28: Deviation of highest non-outlier data from NBCC 2020 envelope curves for all stilt heights.

2.3.2.3 Base Wind Loads

The design wind load provisions for the bottom horizontal surface of elevated buildings have been recently included in ASCE 7-22 and have been equated to the design pressures for a gable-roof building with a slope less than 7°. The design wind loading zones are modified such that the height parameter represents the height of the stilt above grade, \( h_s \). This section aims to explore the effects of stilt height on the area-averaged wind loads on the base of elevated structures and compare current external pressure coefficients to the component and cladding curves given in ASCE 7-22. The same area-averaging procedure
as outlined in Section 2.2.4 was applied for the base of the current study building. Similar to the roof and walls, the averaging panels were generated from the southwest corner of the base as these regions contain a higher tap density.

As previously discussed, the negative pressures on the underside of the building increase with stilt height. Therefore, the highest external pressure coefficients on the base are reported on the 3 m stilt height model. These pressures are shown for the 3:12 roof slope building model in Figure 2-29. These plots demonstrate that the ASCE 7-22 design pressures are overall conservative and encompass nearly all the data from the current study. Generally, the negative design pressures were more conservative than positive, as demonstrated through the minimal exceedance of pressures from the ASCE 7-22 negative curve. Contrarily, the exceedance of pressures from the current study was more commonly observed for positive pressures, particularly in Zone 2. Zone 2 had cases of exceedance for all tributary areas of approximately 2.2 m$^2$ and smaller. The highest positive GCp value in Zone 2 was approximately 0.5, and the corresponding ASCE 7-22 design pressure is 0.3, therefore indicating that the positive design pressures may under predict external pressure coefficients within this region for small tributary areas. It should be noted that these external pressure coefficients represent the worst-case scenario of all test cases between roof slopes and stilt heights. Therefore, the current ASCE 7-22 design pressures on the base surface of elevated buildings are adequately conservative for all negative pressures for the current study.
Figure 2-29: External pressure coefficients on base surface of 3:12 roof slope building model at a stilt height of 3 m.

To provide a more expansive evaluation of external pressure coefficients on the base surface of elevated buildings subjected to ABL winds, the highest non-outlier values for all stilt heights and two roof slopes were calculated and plotted in Figure 2-30. To determine the outliers from the dataset of each averaging area within each zone, a specified threshold was calculated and any datapoint which exceeded this threshold was considered an outlier for the respective dataset. For the current study, the threshold was taken as three standard deviations. From these plots, it is clear that the area-averaged pressure coefficients on the base surface of the 3:12 roof slope model were higher than those on the 6:12 roof slope model, an observation made previously for local pressures. Additionally, the highest negative pressures were observed on the models with stilt heights of 2.5 m and 3 m, representing the highest two stilt heights tested. In both Zone 1 and 2 there were cases where the negative external pressure coefficients exceeded the ASCE 7-22 design pressures by approximately 23% on the 2.5 m and 3 m stilt models for tributary areas less than 1 m².
Observed damage to elevated structures due to wind events has involved the failure of gypsum and hardboard cladding on the underside of the structure (Amini & Memari, 2020). These cladding elements are typically no smaller than 2.88 m² (2 m x 2.4 m); therefore, there is a low risk of damage to cladding elements from the current study. The positive pressures, however, present a higher deviation from ASCE 7-22 design values, as there are cases of approximately 67% exceedance. There is no clear correlation between the location and magnitude of positive pressures on the base of the model. In Zone 2, the positive pressures on all stilt heights exceeded ASCE design pressures for areas less than 2.2 m². Above this threshold, the highest values were approximately equal to ASCE design pressures. These observations indicate that reassessing the positive design pressures may be necessary for conservatively designing the horizontal base surface of elevated buildings to withstand positive pressures.

Figure 2-30: Highest non-outlier external pressure coefficients on the base surface for all stilt heights.
2.4 Conclusions

This chapter evaluated the local and area-averaged external pressure coefficients on an elevated low-rise gable-roof building from stilt heights of 0 m to 3 m at 0.5 m intervals. Correlations between the roof slope and stilt height with respect to the surface pressure magnitudes and distributions were investigated. The components and cladding wind loads on the roofs and walls were compared to NBCC 2020 and ASCE 7-22 provisions. Finally, the adequacy of the newly incorporated components and cladding design pressures for elevated structures in ASCE 7-22 were evaluated using equivalent external pressure coefficients, GCp. The conclusions from this study are summarized as follows:

- The local pressure coefficients demonstrated a strong relationship between the stilt height and the peak negative pressure coefficients on the base surface. As the stilt height increased, the suction on the base increased in both magnitude and area. The average 3-second peak pressure coefficient across the base surface increased from approximately -1 to -3 for an increase in stilt height from 0.5 m to 3 m.

- The highest peak suction values were found around the edges of the base surface and surrounding the stilts of the model. This was likely as a result of increased turbulence and velocity around the stilts.

- The current study confirmed the findings by Kim et al. (2020) and Abdelfatah et al. (2020) which were used in the development of ASCE 7-22 design pressures for elevated buildings. The authors suggest that (1) the base wind pressures are similar in magnitude to those on the roof, and (2) the stilt height of elevated structures has negligible effects on the roof pressures. These relationships were observed in the current study.

- The GCp values from the current study were found to exceed ASCE 7-22 design pressures for the highest stilt heights (2.5 m and 3 m) in the corner and edge base zones for both positive and negative pressures. The positive ASCE design curves were found to underestimate the wind loads on the horizontal base surface of elevated structures above stilt heights of 2 m.
References


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Chapter 3

3 Downburst Wind Loads on Elevated Structures

3.1 Introduction

The atmospheric boundary layer (ABL) wind profile has been used as the basis of wind loading codes in North America for the past several decades. The National Building Code of Canada (NBCC) and American Society of Civil Engineers (ASCE) 7 wind provisions are based on a series of comprehensive studies which evaluated and quantified wind loading on low-rise buildings using wind tunnel testing techniques (Davenport et al., 1977, 1978; Stathopolous, 1979; Surry & Stathopolous, 1978). These studies used the ABL profile and associated wind characteristics to estimate wind loads on various structures and develop appropriate design pressures for national building codes and provisions. However, thunderstorm wind events are reported as governing wind speeds in many areas of the world. In Canada, thunderstorm winds have resulted in widespread damage to low-rise buildings and transmission towers. Furthermore, in 2022, the Northern Tornadoes Project, a project founded at Western University in 2017 to detect non-synoptic wind events throughout Canada, identified 94 downburst events (The Northern Tornadoes Project, 2023). The estimated maximum wind speeds from these events were up to 190 km/h and resulted in damage to infrastructure, forests, and vehicles. In Australia, nearly half of annual extreme gust wind speeds are attributed to gust front outflows and downbursts, and thunderstorm events are the basis of design wind speeds in most major cities (Holmes, 1999). In the United States, thunderstorm winds are found to dominate extreme wind climatology in many sites, and current design wind loads underestimate the design wind speeds in these locations (Twisdale & Vickery, 1992). Additionally, the highest and most frequent severe winds in several South American countries have also been recorded from downburst events (Duranona, 2015; Vallis, 2019; Pita & Schwarzkopf, 2016). Despite the increasing frequency and intensity of non-synoptic wind events such as downbursts, hurricanes, and tornadoes, the standardization of design wind pressures for non-synoptic winds is rare. For the first time, ASCE 7-22 incorporated tornado design pressures for the main wind force-resisting system as well as components and cladding. While this marks
significant progress towards the codification of non-synoptic winds, thunderstorm winds are continuously overlooked in the design of structures despite their destructive nature.

The importance of thunderstorm outflows has been recognized across the globe due to their severe implications for the infrastructure, transportation, atmospheric science, and insurance industries. Thunderstorms produce short, localized winds which are highly divergent and can result in damage to buildings and other structures. The severe winds that are developed during thunderstorms are due to the dynamic effect caused by density differences in convective cloud systems. The buoyant force of warm air against cold results in a strong downdraft of air and precipitation, which falls to the ground and induces an outburst of damaging winds, a phenomenon known as a downburst (Fujita, 1978). The impingement of the downdraft on the ground is capable of producing wind speeds up to 75 m/s (Fujita, 1990). Furthermore, a study by Romanic (2021) reported wind gusts in a downburst outflow which exceeded 20 m/s at the height of only 3 m above ground level. These extreme wind speeds near ground level can result in significant damage to the built environment. Observed structural damage due to downburst winds includes structural failures such as the collapse of low- and high-rise buildings, roof structures, and transmission lines, as well as component and cladding failure (Loredo-Souza et al., 2019; Pita & de Schwarzkopf, 2016; McCarthy & Melsness, 1996). Furthermore, between 2010 and 2020, there were 358 fatalities and over $3.3 billion in total damages due to thunderstorm winds across the U.S. (NOAA, 2021). Despite the widespread structural damage induced by downbursts, the complexity of downbursts continues to limit the ability to develop a shared analytical model for thunderstorm outflows and their wind loading, as discussed by Solari (2020).

The downburst outflow is highly unstable and transient in nature. Upon contact with the ground, the downdraft creates a horizontal vortex which propagates outwards, accelerating and expanding with distance from the core. Downbursts thus generate rapid changes in wind direction and turbulence, which invokes a high risk to the built environment. Due to the formation and propagation of the downdraft, the velocity profile of a downburst differs significantly from that of ABL winds. While ABL wind velocities increase with height, following a log law profile, downbursts produce maximum velocities closer to the ground
and dissipate gradually above the height of maximum velocity. A comparison between the ABL log law profile and downburst horizontal wind profile described empirically by Wood et al. (2001) is shown in Figure 3-1. The height of maximum radial velocities is typically within the range of 30 – 100 m above the ground (Fujita & Wakimoto, 1981; Wilson et al., 1984; Hjelmfelt, 1988). Kim and Hangan (2017) estimated the elevation of maximum radial velocity to occur at an elevation lower than 5% of the downdraft diameter. Therefore, a typical downburst with a diameter of 1000 m would result in the highest velocities lower than 50 m. This contrast between the location of maximum horizontal velocities between ABL and downburst winds may lead to the underestimation of wind loads within these regions. Additionally, the strong vertical components of vorticity and convection associated with downbursts are likely to produce significantly different wind loads on structures compared to ABL winds. A better understanding of flow-structure interactions of buildings under downburst winds is required to generate design wind loads for such structures.
Researchers have used three main approaches for simulating downbursts and estimating their resultant wind loads on structures: data-driven techniques using field studies, computational fluid dynamics (CFD) simulations, and experimental simulations using various laboratory tests. Early downburst research in the 1970s and 1980s contributed to the quantification of naturally occurring downbursts through data collected in two major field studies: the Northern Illinois Meteorological Research on Downbursts (NIMROD) and the Joint Airport Weather Studies (JAWS). Dozens of downburst events were recorded in these projects, which provided the foundation for downburst research. In addition to
NIMROD and JAWS, the Wind Engineering Research Field Laboratory (WERFL) at Texas Tech University was constructed in 1989 to document the effects of wind loads on a full-scale instrumented building. In 2009, Lombardo presented pressure on the full-scale building and velocity measurements from a nearby anemometer under a localized downburst event. The downburst-induced wind loads were compared to those measured on the same building during traditional ABL events and the authors reported that the pressure coefficients at the time of the peak downburst winds were similar in magnitude to the highest ABL-induced pressures. The author suggested that the ABL-derived pressure coefficients may not adequately estimate downburst wind loads. This research was later expanded upon by Lombardo et al. (2018), who confirmed that the pressure coefficients induced by downbursts trended towards the extrema of ABL-induced pressure coefficients. These field studies are useful in providing data to illustrate the nature of downburst winds on full-scale structures; however, there are several limitations with Doppler Radar detection systems used in the data acquisition of full-scale measurements. Field studies can only provide limited quantitative information due to low scanning frequency and poor spatial resolution of the radar near ground level, as noted by Zhang et al. (2013). Experimental and numerical simulations of downburst winds have been adopted using various downburst models to overcome these challenges.

Downburst models have been categorized into three main groups: ring-vortex, a cooling source, and impinging jet (IJ) models. These models can be implemented using both experimental and numerical simulations with experimental facilities and CFD methods, respectively, as well as analytically. The ring-vortex model simulates a descending air column that forms a vortex ring before reaching the ground surface. This model has been adopted by several researchers (Zhu and Etkin, 1985; Ivan, 1986; Schultz, 1990; Vicory, 1992; Savory et al., 2001; Jesson and Sterling, 2018) to gain an understanding of the primary vortex’s flow field’s main features in a downburst. However, upon the impact of the downdraft on the ground, the radial outburst of winds becomes the dominant flow field and is not accurately captured using the ring-vortex method as outlined by Savory et al. (2001). The cooling source method, originally developed by Anderson et al. (1992), produces a downdraft using negatively buoyant thermodynamic cooling, which matches closely with the meteorological formation of naturally occurring downbursts. This method
is implemented both physically and numerically. A liquid drop release method involving
the release of heavier fluids into lighter fluids is implemented in laboratory settings to
produce downburst-like features (Lundgren et al., 1992; Alahyari and Longmire, 1994;
Yao and Lundgren, 1996). The cooling source method is also implemented using both
Reynolds-Averaged Navier Stokes (RANS) simulations and Large-Eddy Simulations
(LES) (Proctor, 1988, 1989; Orf et al., 1996; Orf & Anderson, 1999; Mason et al., 2009,
2010; Vermiere et al., 2011; Zhang et al., 2013; Oreskovic et al., 2018). The cooling source
method presents many challenges, including accurately scaling physical simulations due to
differences in densities between liquids, as well as the inability to emulate the hydrometer
drag, which governs the speed trajectory of the downdraft (Orf et al., 2014). Additionally,
the cooling source method requires a very large computational domain, resulting in a high
number of grid points to acquire the desired spatial resolution near the ground. While the
ring-vortex and cooling system methods reproduce the physics of a real downburst
relatively well, they are often computationally expensive and do not emphasize the details
of the near-ground winds that are of utmost importance to wind engineering.

The impinging jet model is the most widely used method of simulating downburst outflows
and their interaction with the built environment. The IJ method involves the release of fluid
onto a flat surface. As the jet impacts the surface, the flow spreads out radially, generating
a wind pattern similar to that observed from a downburst. Fujita first implemented this
method in 1986, using an inverted plastic cylinder suspended in the air, which directed
airflow vertically downwards onto a flat plate. The author demonstrated the efficacy of this
approach by visualizing the formation and progression of the ring vortex using smoke.
Simulations using IJ models have been found to closely resemble the major flow features
of downbursts (Hjelmfelt, 1987) and have shown good agreement with full-scale data as
reported by Vicroy (1992), Holmes & Oliver (2000), and Sengupta & Sarker (2008). Many
applications of the IJ method have been developed and are described by Mejia et al. (2022).
In summary, these applications include IJ techniques such as analytical IJ models
(Oseguera & Bowles, 1988; Selvam & Holmes, 1992; Kim & Hangin, 2007), numerical IJ
models (Mason et al., 2009; Zhang et al., 2013; Aboshosha et al., 2015; Haines & Taylor,
2018; Huang et al., 2018; Iida & Uetmatsu, 2019; Ibrahim et al., 2021), and experimental
IJ models (Letchford and Chay, 2002; Chay and Letchford, 2002; Mason et al., 2007;
McConville et al., 2009; Junayed et al., 2019). IJ models can be further divided into two categories: stationary and translating/non-stationary. Early IJ models were stationary and steady simulations (Letchord & Illidge, 1999; Wood et al., 2001; Hangan et al., 2008; Zhang et al., 2013; Jesson et al., 2015; Elawady et al., 2017). More recently, researchers have been examining unsteady-state, or time-dependent, simulations of downburst events (Mason et al., 2009, 2010; Zhao et al., 2009; Elawady et al., 2017; Asano et al., 2019; Burlando et al., 2019; Junayed et al., 2019; Romanic and Hangan, 2020; Romanic et al., 2020). The Wind Engineering, Energy and Environment (WindEEE) Dome testing facility at Western University has been used for downburst simulations due to its large scale and capability of producing non-synoptic winds with varying intensities. The WindEEE Dome produces the largest geometric scales of experimentally produced IJ downbursts in the world (Canepa et al., 2022). This facility has been used to evaluate the effect of downburst winds on transmission lines and low-rise buildings (Jubayer et al., 2019; Elawady et al., 2017). A summary of the downburst models and simulation techniques can be found in Table 3-1. Considering the many advantages and disadvantages of different methods to simulate downburst winds, the experimental IJ technique has been carried out at the WindEEE Dome.
Table 3-1: Summary of downburst models and simulation methods.

<table>
<thead>
<tr>
<th>Model type</th>
<th>Description</th>
<th>Tool</th>
<th>Features</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downdraft Ring</td>
<td>Descending air column that forms a vortex ring before impacting the ground.</td>
<td>Analytical</td>
<td>Yes  No</td>
<td>Inaccurate simulation of near-ground flow field after the ring vortex impacts the ground.</td>
</tr>
<tr>
<td>Cooling source</td>
<td>Liquid drop release method involving the release of heavier fluids into lighter fluids to produce downburst-like features.</td>
<td>Experimental</td>
<td>Yes  No</td>
<td>Scaling challenges due to differences in fluid densities and inability to simulate hydrometer drag which influences downdraft velocity.</td>
</tr>
<tr>
<td>Impinging jet</td>
<td>Release of fluid onto a flat surface to simulate downburst-like outflows.</td>
<td>Experimental</td>
<td>Yes  Yes</td>
<td>No standardized scaling methodologies and difficulty producing translation.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Numerical</td>
<td>Yes  Yes</td>
<td>Lack of standardized turbulence models and difficulty simulating topographical effects.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Analytical</td>
<td>Yes  No</td>
<td>Lack of turbulence models for downburst outflow.</td>
</tr>
</tbody>
</table>
**Downburst Wind Loads on Buildings**

Numerical, experimental, and field studies have been conducted to better understand the flow features and aerodynamic loading of downbursts. Nicholls et al. (1993) conducted a 2D Large Eddy Simulation (LES) to examine the flow around a 50 m, full-scale, cubic building subjected to downburst winds. Similarly, Sengupta et al. (2001) evaluated the pressure distribution on a cubic building using a 2D numerical simulation and determined that the model failed to accurately capture the flow physics of the downburst. Chay and Letchford (2002) and Letchford and Chay (2002) studied the effects of a stationary and translating downburst, respectively, on a cubic building using an experimental impinging jet method. The authors evaluated the effect of building location relative to the center of the downdraft and found that downburst-induced wind loads were most comparable to those of ABL winds in the highest-pressure region, which was approximately equivalent to the distance of one downdraft diameter ($R/D = 1$). The authors also found that for translation speeds greater than 20% of the downdraft speed, the positive and negative pressures on the model were greater than for stationary downbursts. Sengupta and Sarkar (2008) presented velocity and pressure measurements made on a cubic building using LES simulated microburst and compared the results to an equivalent laboratory simulated microburst.

Zhang et al. (2013, 2014) used an experimental impinging jet model to simulate downburst winds and evaluate resulting wind loads on various building models. The authors determined that the maximum mean and peak wind loads occur when the building models are located at approximately one diameter from the center of the jet. Additionally, the authors reported significant differences in wind loading patterns for gable-roof buildings with the same plan dimensions but different roof pitches under experimentally simulated downburst winds. The lower roof slope, 16° slope, was found to result in higher magnitudes of suction along the windward edges of the roof surfaces under all three tested wind directions ($0^\circ$, $45^\circ$, $90^\circ$) compared to the 35° roof pitch model. The higher roof slope model did not observe the same conical-shaped pressure distributions on the roof surface under oblique wind directions compared with the 16° roof pitch model. Furthermore, the authors reported surface pressures significantly higher than those predicted by ASCE provisions.
when the building models were mounted within the core region of the downburst. When the buildings were placed in the outflow region of the downburst, the resulting wind loads were found to correlate well with ASCE provisions.

Jesson et al. (2015) used an impinging jet method to simulate downburst winds on various generic building models and reported pressure coefficients on a gable-roof building under 0°, 45°, and 90° wind directions. This study showed similar qualitative patterns with the roof pressures compared to those under ABL winds, as the oblique direction resulted in corner vortices which were associated with high localized suction. These observations were later validated in a study by Haines and Taylor (2018), where CFD simulations were used based on the impinging jet simulator of Jesson et al. (2015). The conical vortices lead to a sharp gradient in pressure distribution across the roof surface as well as an increase in lift on the gable-roof building. The largest pressure coefficients were reported on the roof when located at the region of highest maximum radial velocities.

More recently, Asano et al. (2019) investigated downburst wind loads on a flat-roof building model using an experimental impinging jet model and compared the results to those obtained in a turbulent boundary layer. It was found that a pulsed jet downburst model produces larger suction values on the roof and larger positive pressures on the walls compared to the turbulent boundary layer. The pulsed jet model was also found to increase the strength of conical vortices which were attributed to the higher negative peak pressures on the roof. A study by Jubayer et al. (2019) used the impinging jet technique to experimentally simulate downburst winds at the WindEEE Dome and evaluate their resulting pressures on a typical low-rise building with a roof pitch of 1:12 under three wind directions: 0°, 57°, and 90°. The oblique wind direction was found to produce the highest and most localized suction on the windward corner of the roof, with conical vortex formations appearing in the pressure distribution on this surface. Comparisons were made with results obtained for a building model with the same dimensions under ABL winds, published on the National Institute of Standards and Technology (NIST) database (Ho et al., 2005). The mean pressure coefficient distributions were both qualitatively and quantitatively similar between ABL and downburst tests. However, the downburst model
resulted in higher 3-s peak pressure coefficients by 20.1% on the roof for the corner angle case compared to the ABL tests. The perpendicular wind directions resulted in flow separation at the leading edge of the building and reattached at approximately the midpoint of the roof, a phenomenon commonly observed with ABL winds. However, this separation was reported to be less uniform across the leading edge of the building compared to ABL winds. Finally, when compared to ASCE provisions (ASCE 7-10), the code was found to provide conservative design loads for the building orientations that were tested, except for one roof zone (ASCE 7-10, Zone F), which was found to be inadequate.

The limited studies which investigate downburst wind loads on elevated buildings generally showed that the non-synoptic winds produced many discrepancies compared to ABL wind loads. These discrepancies include stronger conical vortices on the roof which result in higher peak negative pressures, higher positive pressures on the wall surfaces, and higher pressure fluctuations due to downburst winds. There remains a clear lack of knowledge regarding downburst wind loading and the adequacy of current building codes for downburst applications. While the inclusion of downburst design provisions has been suggested by several authors, these wind loads are not implemented in current building codes. Furthermore, there are currently no studies which evaluate the downburst wind loads on elevated structures. Therefore, the objective of this section is to evaluate the local and area-average wind loads on a low-rise elevated building model with two roof pitches and determine relationships between roof slope and stilt height on surface pressure magnitude and distribution. Finally, a component and cladding analysis was conducted for the purpose of evaluating the effectiveness of current building code provisions.

3.2 Experimental Downburst Methodology

3.2.1 Case Study Building

The building model used in the current study has the same geometry as that used for the ABL tests described in full in Section 2.2.1. The building model is representative of an elevated two-story low-rise gable-roof building, typically found in northern Canadian communities. The geometric scale is 1:100, and full-scale measurements of 23.8 m and 15.8 m with an eave height of 7.3 m. The building model was 3D printed at Western
University’s Machine Services facility and consisted of one base and two detachable roofs with pitches of 3:12 and 6:12. The model was fitted with 342 and 350 pressure taps across the building surfaces for the 3:12 (14.0°) and 6:12 (26.6°) roof pitches respectively (Figure 2-2).

The downburst tests were conducted at the Wind Engineering, Energy, and Environment (WindEEE) facility at Western University. The high-frequency pressure integration (HFPI) model was connected to the pressure system below the test chamber of the WindEEE Dome. The pressure taps of the building model were connected to two 0.5 m long PVC tubes that were connected by a stainless-steel restrictor used to minimize the distortion effect of the pressure fluctuations due to the tubing as demonstrated by Surry and Stathopolous (1978) and Irwin et al. (1979) for the use in wind tunnel tests. The pressure tubes were connected below the testing chamber to Electronically Scanned Pressure (ESP) scanners and Digital Temperature Compensation (DTC) Initiouns, used for the data acquisition. The tubing and data acquisition systems used for the current study were the same as those described in Jubayer et al. (2019). The frequency of pressure measurements was 500 Hz, and a total of 60 s of measurements were taken, including the initial 30 seconds when the downburst was pressurizing in the upper chamber. Therefore, approximately 30 seconds of downburst measurements were recorded. The raw pressure measurements were reduced to 50 Hz using a low-pass filter implemented in MATLAB.

A mast of eight fast-response Cobra probes (Series 100) developed by Turbulent Flow Instrumentation was placed approximately 70 cm beside the building model to capture the velocity flow field of the downburst outflow simultaneously with the pressure measurements. The mast was placed away from the model to ensure there was no flow interference from the building model in the velocity measurements. The simultaneous measurement of velocity and pressure ensures an accurate reference velocity for the calculation of pressure coefficients which is further discussed in Section 3.3.2. The Cobra probes were placed at heights of 2 cm, 4.5 cm, 7 cm, 9.5 cm, 12 cm, 15 cm, 20 cm, and 40 cm (Figure 3-2) and recorded velocity measurements at a sampling frequency of 1250 Hz for the duration of the downburst event. The mast was rotated towards the center of the downdraft to ensure the outflow was directly towards the Cobra probes, thus maximizing
the radial component of the flow. An additional ten downburst velocity measurements were taken in an empty chamber where the leading edge of the building model was located at a radial distance, R, equal to the bellmouth diameter, D, from the center of the bellmouth (R/D = 1) for model validation purposes, discussed in Section 3.3.1.

![Figure 3-2: Cobra probes mounted on a mast directed into the center of the downburst.](image)

### 3.2.2 Downburst Test Cases

The ratio between the radial location of interest and the diameter of the downdraft is an important parameter when discussing the flow field and resulting wind loads on structures. Previous studies using impinging jet models have found that maximum velocities typically occur around R/D = 1 (Chay & Letchford, 2002; J. Kim & Hangan, 2007; Zhang et al., 2013). (Junayed et al., 2019) found that the location of maximum velocity for downbursts produced at the WindEEE Dome with H/D > 1 was between R/D = 0.9 – 1.1 and was dependent on the Reynolds number. This was later supported by findings from the publicly available dataset of downburst velocity measurements from the WindEEE Dome, produced by Canepa et al. (2022). These records showed that the maximum radial velocity for a downburst generated using 20% RPM of the fans in the upper plenum resulted in maximum
velocities at a radial location of $R/D = 1$. Therefore, the building model in the current study was placed 3.32 m from the center of the bellmouth (center-to-center) to achieve an offset of $R/D = 1$ from the downdraft. The offset of the building model from the center of the downdraft was achieved by installing a wooden platform with a baseplate offset of approximately 1.5 m (center-to-center) onto the turntable in the testing chamber. The bellmouth was translated 1.82 m in the opposite direction to ensure the leading edge of the building model was located at $R/D = 1$. Figure 3-3 shows the building model and Cobra probe set-up on the wooden platform in the test chamber.

![Figure 3-3: Building model and Cobra probe set-up in the test chamber of WindEEE Dome.](image)

The current study investigated seven stilt heights, including a ground-mounted case, as well as four building orientations ($0^\circ$, $30^\circ$, $60^\circ$, and $90^\circ$) for each stilt height. Each test case was repeated five times, giving a total of 280 downburst tests on the two building models. A schematic depicting the building orientation is shown in Figure 3-4. A summary of the test parameters is shown in Table 3-2.
Figure 3-4: Building model orientation for 0° (a), 30° (b), 60° (c), and 90° (d) relative to the downdraft center.
Table 3-2: Test parameters for the downburst tests at WindEEE.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model Scale</td>
<td>1:100</td>
</tr>
<tr>
<td>Sampling frequency</td>
<td>500 Hz</td>
</tr>
<tr>
<td>Sampling period</td>
<td>60 seconds</td>
</tr>
<tr>
<td>Building orientations</td>
<td>$0^\circ$, $30^\circ$, $60^\circ$, $90^\circ$</td>
</tr>
<tr>
<td>Heights</td>
<td>0 cm, 0.5 cm, 1.0 cm, 1.5 cm, 2.0 cm, 2.5 cm, 3.0 cm</td>
</tr>
<tr>
<td>Roughness blocks</td>
<td>None – flush with the ground</td>
</tr>
<tr>
<td>Repetitions per trial</td>
<td>5</td>
</tr>
<tr>
<td>Total test cases</td>
<td>280</td>
</tr>
</tbody>
</table>

3.2.3 Downburst-like Impinging Jet Method

The WindEEE Dome facility at Western University was used to measure external pressures on a gable roof building model under downburst-like winds. The 25 m diameter hexagonal chamber is comprised of two chambers, an upper and lower chamber, 100 peripheral fans in the lower chamber and six larger fans in the upper chamber. The testing chamber is separated from the upper chamber with a bellmouth located on the ceiling of the lower chamber with mechanical louvres. A schematic of the downburst-like impinging jet production at WindEEE is depicted in Canepa et al. (2022). This testing facility can produce large-scale downburst-like impinging jets with downdraft diameters ranging from 3.2 m – 4.5 m, which corresponds to length scales in the range of 1:100 to 1:250 based on a novel scaling methodology developed by Romanic et al. (2020), wherein model- and full-scale downburst velocity measurements were decomposed, and statistical quantities were matched to estimate the appropriate length scales. The downbursts are simulated by running the six fans in the upper plenum until the desired pressure is achieved. The desired pressure of the testing chamber was set to 3.4 hPa, greater than the pressure in the testing chamber (Romanic et al., 2019). The mechanical louvres at the mouth of the bellmouth
were then opened, releasing the air into the testing chamber, producing a downdraft which impinges on the floor of the chamber and bursts radially outwards from the center.

The height between the bellmouth and test chamber floor is 3.8 m, thus enabling the simulation of a downburst with a height-to-diameter ratio (H/D) of 0.8 – 1.2. The ratio between the height of the downdraft from the ground and the diameter of the downdraft should be greater than one to ensure the complete development of the ring vortex above the ground. This criterion was explored by Xu and Hangan (2008) using an experimental impinging jet method wherein the authors tested four H/D ratios and their effect on the mean and turbulence velocity fields as well as the maximum radial velocity. The authors conclude that the confinement effects on the flow field are negligible for H/D ratios greater than one. Junayed et al. (2019) also investigated the impact of H/D ratios greater and less than one and compared their radial velocity profiles to existing full-scale data. It was found that the profiles with H/D greater than unity had a more pronounced “nose” shape profile and showed better agreement with full-scale data. Therefore, the bellmouth diameter for the current study was set to 3.2 m to obtain H/D = 1.2.

3.3 Results and Discussion

3.3.1 Velocity Flow Field

The radial velocity profile of the downburst outflow is the most commonly used criterion for validating experimentally and numerically simulated downburst-like winds. The radial velocity profile of a downburst follows a characteristic “nose” shape, with maximum velocities ranging between elevations of 50 m to 100 m and slowly dissipating in height above this region. While downburst diameters and magnitude of velocities vary between events, all downbursts produce this velocity profile, thus making it the basis for numerical and experimental validation. The radial velocity profile used for validation purposes in the current study is based on the slowly-varying mean velocity measurements using an averaging window of $T = 0.1$ s, a method previously employed by Romanic et al. (2019) and Junayed et al., (2019).

The averaging time was based on a velocity decomposition approach used for full-scale events (Choi and Hidayat, 2002a, 2002b; Holmes et al., 2008; and Solari et al., 2015) and
applied to velocity measurements taken at the WindEEE Dome (Junayed et al., 2019). The moving average filter, \( T = 0.1 \text{ s} \), was applied to the ten velocity time histories recorded in the empty chamber for the current study. The raw velocity time histories at a radial location of \( R/D = 1 \) and a height of 12 cm for one downburst trial are in an empty chamber are shown in Figure 3-5 for the radial (\( U \)), lateral (\( V \)), and axial (\( W \)) components of velocity. This case represents the downburst test with the highest maximum radial velocity, equal to approximately 14.9 m/s. Although the sampling period was 60 seconds, this plot shows a 15 second segment to emphasize the initial gust front and peak downburst velocity. The radial velocity signal was decomposed into the slowly-varying mean, residual fluctuations, slowly-varying standard deviation, reduced turbulent fluctuations, and turbulence intensity using the method developed by Choi and Hidayat (2002a). The velocity decomposition is shown in Figure 3-6 and the following equations were used to calculate the various components of the decomposition:

\[
\begin{align*}
    u(t) &= \bar{u}(t) + u'(t) \\
    u'(t) &= \sigma_u(t) + \bar{u}'(t) \\
    I_u(t) &= \frac{\sigma_u(t)}{\bar{u}(t)}
\end{align*}
\]  

where, \( u(t) \) is the radial velocity as a function of time, \( \bar{u}(t) \) is the slowly-varying mean of \( u(t) \), \( \bar{u}'(t) \) is the high-frequency residual fluctuations, \( \sigma_u(t) \) is the slowly-varying standard deviation, \( \bar{u}'(t) \) is the reduced turbulent fluctuations, and \( I_u(t) \) is the turbulence intensity.

The extraction of the slowly-varying mean velocity from the radial velocity time history was carried out using a moving average filter or running-mean, a method previously used by Choi and Hidayat (2002a), Holmes et al. (2008), and Solari et al. (2015). The proper averaging period will ensure that the residual fluctuations do not contain elements of the large-scale wind structure and that the time-varying mean portion of the signal does not include small-scale turbulence fluctuations. The slowly-varying mean velocity portion for an averaging period of 0.1 seconds does not contain high-frequency fluctuations while
maintaining the large-scale structure of the downburst gust front. Additionally, the residual fluctuations possess a near zero mean value, indicating that an averaging period of 0.1 seconds is sufficient. Further criteria for selecting an appropriate averaging period are outlined in Solari et al. (2015) and involve an analysis of the joint Fourier transforms and statistical properties of the reduced turbulent fluctuations. These criteria were investigated by Junayed et al. (2019) and Romanic et al. (2018) for experimentally produced downbursts in the WindEEE Dome for the same test setup as the current study, and an averaging period of 0.1 seconds was obtained independently for both studies. Therefore, the current study used an averaging window of 0.1 seconds for the validation of the downburst wind field.

![Figure 3-5: Velocity time histories for radial, lateral, and axial components at a location of R/D = 1 and height of 12 cm.](image)
The radial velocity profile at the instance of maximum velocity is plotted in Figure 3-7, along with two other radial velocity profiles obtained using the same experimental set-up (Canepa et al., 2022; Junayed et al., 2019) as well as full-scale measurements taken for the JAWS project (Hjemfelt, 1988). The full-scale profile represents the average profile of twelve separate downburst events. The current study compares well to both experimental and full-scale measurements and displays the characteristic “nose” shaped profile of a downburst. The maximum of the slowly-varying mean velocity was recorded as 12.4 m/s
at the height of 9.5 cm above the floor of the testing chamber. The profile from the current study is most comparable to the full-scale measurements obtained from the JAWS project, indicating that the current model accurately produces downburst outflows.

![Figure 3-7: Normalized radial velocity profile for instantaneous slowly varying mean velocities at R/D = 1.](image)

The velocity scale for the current model is estimated as the ratio between the model and full-scale peak velocities. The full-scale velocity measurements reported by De Gaetano et al. (2014) were used to compute the velocity scale, similar to the process employed by Jubayer et al. (2019). The reported full-scale peak velocity was 15.9 m/s, and the model-scale peak velocity from the slowly-varying-mean time history using an averaging window of 0.1 s was 12.4 m/s. Therefore, the velocity scale for the current model is estimated to be approximately 1:1.2. This is the same velocity scale estimated by Jubayer et al. (2019) and considering the same test set-up as well as the close similarity between radial velocity...
profiles and peak velocities, the time scale for the current model was taken as that calculated by Jubayer et al. (2019), 1:84. This time scale is based on matching statistical quantities (skewness, kurtosis, turbulence intensity, and 1-s peak velocity) of the model-scale downburst velocities with those of full-scale measurements through a minimization function. This procedure is described in full by Solari et al. (2015). From the time and velocity scales, the length scale for the current study is calculated as 1:101, which is nearly equivalent to the geometric scale of the building model (1:100).

### 3.3.2 Building Geometry Effects on Local Pressures

The subsequent sections present the local pressure coefficient magnitudes and distributions across the building surfaces of both 3:12 and 6:12 roof pitch models under various wind angles and stilt heights. The external pressure coefficients are calculated as outlined in Section 2.2.4; however, it should be noted that the pressure coefficients presented herein are a result of non-synoptic winds rather than synoptic, as discussed in Chapter 2. The non-stationary nature of the downburst flow suggests that the previously employed extreme value analysis is not applicable. Therefore, the peak pressure coefficients presented in this chapter represent the average of the minimum and maximum observed pressure coefficients from all five downburst tests. The pressure coefficients were referenced to the maximum 3-s peak wind velocity at mid-roof height for each trial using a moving average filter equivalent to 0.036 seconds in model-scale, which corresponds to 3 seconds in full-scale with the assumed time scale of 1:84. The peak pressure coefficients are taken as the mean of the maximum 3-s peak records from all five trials.

The pressure coefficient, $C_p$, for this section is defined as:

$$C_p(t) = \frac{p(t) - p_0}{\frac{1}{2} \rho \bar{U}_m^2}$$

(3-4)

where, $p(t)$ is the measured surface pressure as a function of time, $t$, $p_0$ is the static pressure, $\rho$ is the air density in the testing chamber, and $\bar{U}_m$ is the 3-s peak velocity measured for the individual test trial at mean-roof height.
3.3.2.1 Effect of Roof Slope and Wind Angle on Roof Pressures

Given the current information available regarding the interaction of downburst winds on buildings, further research is needed to better understand the impact of the downburst outflow direction on the pressure distribution of low-rise buildings to identify any patterns or trends that may emerge. Therefore, the effect of wind angle on the magnitude and distribution of pressure coefficients on the current ground-mounted building model was evaluated in the current study by rotating the baseplate of the model relative to the center of the bellmouth. The ground-mounted model provides a baseline upon which the elevated models can be compared.

Figure 3-8 contains contours of the standard deviation (a) and peak minimum pressure coefficient (b) distributions on the roof surfaces of both 3:12 and 6:12 roof pitch building models at a stilt height of 0 m. From these contour plots, it is evident that the difference in roof slope has a notable impact on the pressure fluctuations, magnitude, and distribution under downburst winds. The lower roof slope resulted in higher magnitudes of suction under all wind directions except the 0° test case compared to the 6:12 roof slope. These areas of suction are located on the windward corner or edge of the roof, depending on the angle with which the downburst outflow impacts the building. The 3:12 roof slope resulted in a secondary flow separation at the ridge of the roof, a common phenomenon for low-slope low-rise buildings (Simiu, 2011; Ozmen et al., 2016). The maximum observed peak negative pressure was approximately -3.5 and reported on the 3:12 roof at 60° orientation. Contrarily, the pressure at the same location on the 6:12 roof is approximately 78% lower than that on the 3:12 roof. This occurs due to the reduced wind velocity on the higher roof slope as a result of flow obstruction. The flow impacts the higher roof slope and subsequently decreases in velocity, causing less uplift on the corners and edges of the roof. The lower roof slope observes flow separation both at the leading corner and the ridge corner of the roof, whereas separation only occurs at the leading corner of the roof on the 6:12 slope model, similar to the flow features due to ABL winds. These regions are also susceptible to much higher fluctuations in pressure on the 3:12 roof slope, as observed in the standard deviation contours. The highest standard deviation for the 3:12 roof slope at a 60° angle is approximately equal to 0.5, whereas this value is approximately equal to 0.1
on the 6:12 roof slope. The higher standard deviations on the lower roof slope are a result of the increased turbulent kinetic energy resulting from higher velocities over the roof slope. In contrast, the 0° wind angle presented higher suctions for the 6:12 roof pitch versus the 3:12 pitch. The pressure variation on the higher roof slope is caused by the increased ridge height and blockage ratio. The stagnation point on the 6:12 roof slope is located at a higher elevation, thus creating a larger flow separation from the windward edge of the roof compared to the 3:12 roof slope. The lower roof slope is more streamlined, resulting in a smaller separation bubble and lower magnitudes of suction.

### Peak Negative Cp

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<tbody>
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### Std. Cp

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<td>90°</td>
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</table>

**Figure 3-8:** 3-s peak minimum pressure coefficient (a) and standard deviation (b) distributions on the roof surface of the ground-mounted building model.

The peak minimum and maximum pressure coefficient variations across the midline of both roofs for a 0 m stilt height and 90° orientation are shown in Figure 3-9. This plot
shows the notable difference in negative pressure variation between the two roof slopes and confirms the separation of flow at the leading edge and ridge of the lower slope roof and reattachment on the leeward face. The steeper roof slope has a more suppressed separation on the windward face of the roof because the wind direction is more perpendicular to the surface; this reduces the magnitude of suction at the edge. Tominaga et al. (2015) explain the difference in flow separation and recirculation regions using streamlines for three different roof slopes. The authors show that the recirculation eddy downstream of the building moves upwards and away from the building as the roof pitch increases. This agrees with the results of the current study, as the higher roof slope results in lower suction and higher peak positive pressures on the leeward surface of the roof. The lower roof slope shows the fluctuations in peak negative pressure coefficients immediately adjacent to the windward edge and ridge of the roof. The reduced impingement for the lower roof slope results in increased suction. While there is greater variation in negative pressures across the lower roof slope, the variation of positive pressures across the midline is greater for the higher roof slope. This is anticipated due to the size and location of the recirculation eddy caused by the increase in roof slope. Tominaga et al. (2015) observe significant flow field changes between roof slopes of 16.7° and 26.6° and suggest that the critical roof angle occurs between these roof slopes. Comparatively, the current study examines roof slopes of 14° (3:12) and 26.6° (6:12), which agrees with the findings of Tominaga et al. (2015), as there are significant differences between the pressure magnitudes and distributions between these two roof slopes. These results indicate that design provisions for low-rise gable-roof buildings would benefit from different wind loads between mild and steep roofs. ASCE 7-22 makes this distinction as component and cladding design pressures are provided for both roof slopes between 7° to 20° and 20° to 27°. However, this distinction is not made in NBCC as design pressures are provided for gable roof slopes between 7° to 45°. The component and cladding loads are discussed in more detail in Section 3.3.3.
Figure 3-9: Pressure coefficient variation across roof midline for 3:12 and 6:12 roof slopes.

3.3.2.2 Effect of Building Height on Surface Pressure Distribution

This section investigates the effect of stilt height on the magnitude and distribution of pressure coefficients across the elevated building models. This discussion is broken into a semi-qualitative analysis of aerodynamic trends and a quantitative analysis of pressure trends on each building surface. The semi-qualitative analysis consists of enveloped peak negative pressure coefficient contours for both roof slopes at all seven stilt heights, including the ground-mounted case. Pressure coefficient plots along the midline of both roofs for each stilt height are also plotted. The quantitative approach involves the calculation of a factor which describes the difference in pressure coefficients relative to the ground-mounted model. This factor facilitates a quantitative evaluation of the impact of stilt height on pressure coefficients on each face of the building surface. The objective of this section is to determine both qualitative and quantitative trends between pressure distribution and the stilt height of an elevated building.
Semi-Qualitative Local Pressure Trends

The enveloped peak minimum and maximum pressure coefficient contours on the 3:12 roof slope model are shown in Figure 3-10 and Figure 3-11 and respectively. The areas with the highest negative pressure are found to be on the corner of both the roof and the base surface. The highest peak negative pressure coefficient was approximately -4.0 and located on the windward corner of the roof at the maximum stilt height of 3 m. This is the region where the critical negative pressures are found for all stilt heights and range from -3.3 to -4.0. These localized peak pressures were all a result of an oblique wind direction, either 30° or 60°. The variation in magnitude of these critical negative pressures is negligible, and there is no correlation between the stilt height and the magnitude of the pressure coefficient, a common finding for ABL-induced pressures on elevated buildings (Kim et al., 2020; Abdelfatah et al., 2020). The peak negative pressure coefficients on the four walls of the building model show very little variation with stilt height. The biggest change in pressure on the wall surfaces as stilt height increases is the area and location of suction on the non-gable end walls. As stilt height increases, both the peak minimum and maximum pressure coefficients on the walls increase slightly in magnitude and coverage.

Generally, the peak negative pressure coefficient distributions across both building models are very similar to those of the ABL-induced pressures, which were previously discussed in Section 2.3.1.3. The roof pressures followed similar trends in terms of distribution and flow feature; however, the magnitude of these pressures was significantly higher under downburst loads compared to ABL for the same stilt heights and wind directions. When comparing wind loads on the ground-mounted model, the highest peak negative pressures were found at a wind direction of 60° on the windward corner and immediately downstream of the ridge for the downburst and ABL winds, respectively. These pressures were approximately -3.5 and -1.9 for the downburst and ABL tests, respectively. When comparing the pressure fluctuations on the roof of both models as a function of the standard deviation, the difference between the downburst (Figure 3-8, b) and ABL winds (Figure 2-11, b) is relatively small. The highest standard deviation for the downburst winds on the ground-mounted model with the 3:12 roof slope is 0.8 and located on the windward corner of the roof at a 30° wind direction. Comparatively, the highest standard deviation for the
same test conditions with ABL winds is 0.7 and was found at the same location. Downbursts are expected to produce higher pressure fluctuations due to the rapid variations in wind direction versus the more stable flow of ABL winds. While the magnitude of standard deviation is lower for the ABL wind than the downburst, the area of high standard deviation is much larger, covering approximately 20% of the windward roof surface, whereas the peak standard deviation for downburst winds is much more localized around the windward corner.
Figure 3-10: Peak minimum pressure coefficient contours on 3:12 roof slope at stilt heights 0 m (a), 0.5 m (b), 1 m (c), 1.5 m (d), 2 m (e), 2.5 m (f), 3 m (g)
Figure 3-11: Peak maximum pressure coefficient contours on 3:12 roof slope at stilt heights 0 m (a), 0.5 m (b), 1 m (c), 1.5 m (d), 2 m (e), 2.5 m (f), 3 m (g)
The pressures on the base of elevated structures are of particular concern due to reported damage to cladding under extreme winds. The contour plots in Figure 3-10 show an increase in negative pressure as stilt height increases. Figure 3-12 shows additional peak negative pressure coefficient contours on the base surfaces of the 3:12 roof slope model at all the elevated stilt heights to better visualize pressure distribution trends. The highest peak negative pressures on the base surface are located around the stilts and edges and are equal to approximately -2.0 at a stilt height of 3 m. It is expected that these regions would produce high localized suction due to flow separation and higher turbulent kinetic energy. As the stilt height increases, the larger ventilation space below the building results in higher wind velocities and larger flow separation around the edges of the base. The negative peak pressures for the 0.5 m stilt height are insignificant; however, a one-meter increase in stilt height nearly doubles the maximum suction values. The variation of flow separation and reattachment with stilt height is also observed in the peak maximum pressure coefficient contours. As stilt height increases, regions of positive pressure form around the interior corners of the stilts. This is likely due to vortex shedding around the stilts and is associated with high-pressure fluctuations.

Figure 3-12: Peak minimum pressure coefficient contour on the base surface of the 3:12 roof slope.
Figure 3-13 shows the variation in peak minimum pressure coefficients across two midlines of the base surface on both building models and all stilt heights. The 6:12 roof slope model showed slightly higher negative pressures for all wind directions across the base midlines, particularly for the taps closest to the windward edge. The impact of roof slope on base loads of elevated structures has not previously been investigated; therefore, the reason for the difference in base loading remains unspecified. Streamline visualization using particle image velocimetry (PIV) or computational fluid dynamics (CFD) methods would greatly enhance the understanding of this phenomenon. It may be speculated that the increased blockage ratio due to the higher roof slope would result in a larger volume of airflow being redirected underneath the building. The higher blockage ratio and stagnation point result in higher velocities and, thus, stronger separation at the leading edge of the base of the building. The midline plots also show that the 0.5 m and 1 m stilt heights have the lowest negative pressures, respectively, indicating that the aerodynamics of elevated structures above stilt heights of 1 m changes significantly. The relationship between stilt height and base pressures above 1 m is not clearly identified in these plots. For both parallel wind directions (i.e., 0° and 90°), the flow separation is observed at the leading edge of the base, and reattachment occurs at approximately 20-30% of the length or width of the building. For the 0.5 m and 1.0 m stilt heights, the reattachment occurs much closer to the edge of the base, within 10-20% of the dominant dimension, which is presumed to occur due to the restriction of airflow beneath the building. The oblique wind direction differs from the parallel directions due to the increase in suction towards the leeward edge of the base. Within the windward 40% of the base, the oblique wind direction shows a direct correlation between the stilt height and negative peak pressure. As the stilt height increases, the peak pressures increase for each increment in stilt height. Past this threshold, all stilt heights result in similar negative pressures.
Figure 3-13: Midline peak pressure coefficient variation across the base surface at different stilt heights.

Quantitative Local Pressure Trends

A quantitative approach was used to evaluate the variation of wind loads with stilt height on the elevated building model. Similar to Section 2.3.1.3, a cumulative parameter, \( R \), was used to compare the peak pressure for each surface tap relative to the corresponding tap on the ground-mounted model. This parameter is defined as the difference between the 3-s peak pressure on the ground-mounted model and those of the elevated cases, as shown in Equation (3-5). The \( R \)-values for the base surface are simply the average of the peak minimum pressure coefficient for all pressure taps, as they cannot be compared to the ground-mounted model.

\[
R = \text{Avg.} \left( \bar{C}_p^{\text{stilted}} - \bar{C}_p^h \right)
\]  

\text{(3-5)}
The difference in R-value between each of the wall surfaces is negligible, and therefore all the wall surfaces can be considered together. The average R-values for all wall surfaces are plotted along with the base and roof R-values for both roof slopes in Figure 3-14. The lower roof slope resulted in higher peak negative pressures on the base and roof and impacted the base loads most significantly. These graphs show that there is a moderate correlation between stilt height and average pressures across each building surface. This trend is more notable on the base and roof surfaces of the building. The R-values on the walls show a negligible increase with stilt height. Additionally, the lower roof slope produced higher wall pressures compared to the ground model for stilt heights between 0.5 m and 1.5 m, whereas the higher roof slope produced larger values above these heights. Contrary to the ABL winds presented in Figure 2-21, there is a correlation between the stilt height and peak pressure coefficients on the roof of the building relative to the ground-mounted model as a result of downburst winds.

![Figure 3-14: Average R-value for wall, roof, and base surfaces of building model with both roof slopes and all stilt heights.](image)

### 3.3.3 Component and Cladding Wind Loads

This section involves the evaluation of area-averaged peak enveloped pressure coefficients from the WindEEE downburst tests as well as a comparison with the component and
cladding design curves provided by both NBCC (2020) and ASCE 7-22. In this section, the component and cladding envelope curves from both NBCC and ASCE 7-22 are used for the comparison of the roof and wall pressures, whereas only the ASCE 7-22 provisions are used for the wind loads on the base of the elevated building due to the absence of such loads in the Canadian code. The envelope curves from NBCC 2020 were converted from mean hourly external pressure coefficients (CgCp) to 3-s gust external pressure coefficients (GCp) using the same process described in Section 2.2.4.2. A wind tunnel factor, \( F_{WT} \), calculated using Equation (3-6) below in conjunction with the Durst Curve was applied to the NBCC design curve. Additionally, the directionality factor embedded in the CgCp values was removed as this factor is not included in the ASCE components and cladding curves, but separately applies as the so-called factor \( K_d \). The effects of directionality have been negated by applying a factor of \( \left( \frac{1}{0.85} \right) \) to the NBCC design curves.

\[
F_{WT} = \left( \frac{V_{3\text{s,open}}}{V_{3600\text{s,open}}} \right)^2
\] (3-6)

The pressure coefficient time histories were calculated using the same method described in Section 3.3.2 and represent 3-s gust external pressure coefficients. To produce the enveloped pressure coefficient data, the area-averaging procedure outlined in Section 2.2.4.2. was employed. The area-averaged pressure coefficients were enveloped across the four wind directions that were tested for both building models and all stilt heights. The results of the area-averaging procedure are presented and discussed in Sections 3.3.3.1, 3.3.3.2, and 3.3.3.3.

### 3.3.3.1 Ground-Mounted Model

The area-averaged peak minimum and maximum enveloped pressure coefficients for the ground-mounted with mild and steep roof slopes are shown in Figure 3-15 and Figure 3-16, respectively. The ground-mounted building model provides a benchmark upon which the elevated test cases can be compared to evaluate the effect of stilt height on component and cladding loads. In general, the lower roof slope resulted in higher area-averaged loads across all roof zones compared to the steeper roof pitch. The design provisions for the corner roof zone (Zone C) were the most conservative for both roof slopes compared to the
design values for the other roof zones. From Figure 3-15, there were a total of three negative GCp values from the current study which exceeded the ASCE 7-22 design curve. These cases of exceedance were as a result of the smallest averaging area, 0.25 m$^2$. This averaging area resulted in exceedance of the NBCC design curve for the 6:12 roof slope as shown in Figure 3-16. The steeper roof slope did not result in any GCp values that exceeded the ASCE 7-22 design provisions for the roof corner zone. The ASCE 7-22 design curve encompassed nearly all GCp values from the current study for the roof edge zone (Zone S) for both roof slopes. Contrarily, the NBCC provisions underpredicted GCp values within this range, particularly for the lower roof slope model as shown by the exceedance of the curve in Figure 3-15. The interior roof zone, Zone R, contained a significant number of data points which exceeded both the ASCE 7-22 and NBCC 2020 design curves for both roof slopes and all averaging areas. For all components and cladding zones, the higher roof slope resulted in lower negative GCp values, thus demonstrating that the components and cladding provisions for higher roof slopes need not be as conservative as those for lower roof slopes. This observation is accounted for in the ASCE 7-22 provisions as the design curves for buildings with a roof slope between 7° and 20° are higher than those for roof slopes between 20°-27°. However, for the current study, the ASCE 7-22 design curves for the lower roof slope would have been a better estimate of the negative GCp values for the higher roof slope model. Finally, the positive GCp roof values from the current study were encompassed by the ASCE 7-22 design provisions for all zones and averaging areas. The 6:12 roof slope model resulted in higher positive GCp values compared to the 3:12 roof slope. The higher roof slope model resulted in several positive GCp values which exceeded the NBCC design provisions for all roof zones and averaging areas.

The GCp values within the wall zones showed less variation between the two roof slopes compared to those on the roof. The higher roof slope resulted in higher negative GCp values on the wall edge zone (Zone E), whereas the lower roof slope resulted in higher values on the wall interior zone (Zone W). The ASCE 7-22 negative design curve for the edge wall zone proved to be more conservative for the current study compared to the interior wall zone. There were cases of exceedance for all averaging areas on the interior wall zone for both positive and negative design pressures.
Figure 3-15: Component and cladding loads on walls and roof of 3:12 roof pitch building model at 0 m stilt height.

Figure 3-16: Component and cladding loads on walls and roof of 6:12 roof pitch building model.
3.3.3.2 External Pressure Coefficients for All Stilt Heights

The external pressure coefficients on the elevated building models were also calculated and compared with the NBCC 2020 and ASCE 7-22 components and cladding curves. The ASCE 7-22 building code has recently incorporated design wind loads for elevated buildings; however, these design pressures only extend to the bottom horizontal surface of elevated buildings while the wall and roof zones are treated the same as a ground-mounted building. The purpose of this section is to evaluate the effectiveness of ground-mounted envelope curves for the wall and roof zones of elevated structures. Therefore, the highest non-outlier values for each tributary area and each stilt height were extracted and plotted against the building code envelope curves, as shown in Figure 3-17 and Figure 3-18. An outlier is defined as a value that exceeds three standard deviations from the mean of the dataset, therefore representing the top 99.7% percentile.

These scatterplots show similar trends compared to the ground-mounted model. The corner roof zone on the 3:12 roof slope model contained GCp values which exceeded the ASCE 7-22 design curve for all averaging areas and several stilt heights. The smallest averaging areas, 0.25 m², resulted in GCp values higher than the ASCE 7-22 design curve for six stilt heights on the 3:12 roof slope model. Figure 3-17 shows that the highest GCp values on the corner zone of the 3:12 roof slope model resulted from the two highest stilt heights, 2.5 m and 3.0 m. The GCp values on the roof edge zone were below the ASCE 7-22 design curve for all averaging areas except 10 m², where all stilt heights for the 3:12 roof slope model resulted in higher GCp values than the curve. The interior roof zone resulted in exceedance of the ASCE 7-22 design curve for both roof slopes and all averaging areas for stilt heights equal to 1 m or higher. The positive design GCp values from the ASCE 7-22 curve were generally satisfactory for the current setup and model geometry although there were limited cases of exceedance on the interior roof zone for averaging areas greater than 5 m² on the higher roof slope. The NBCC provisions underestimated the negative GCp values for all roof zones on the 3:12 roof slope model for all averaging areas, whereas the design curve encompassed a greater number of peak GCp values on the corner and edge roof zones on the 6:12 roof slope model.
The highest non-outlier GCp values for both wall zones from both roof slopes generally exceeded both ASCE 7-22 and NBCC 2020 design curves. The higher roof slope resulted in higher GCp values on the walls compared to the lower roof slope. All stilt heights, except the ground-mounted model, exceeded the negative and positive ASCE 7-22 design curve for the edge and interior wall zones on the 6:12 roof slope model. The 3:12 roof slope model resulted in GCp exceedance of the ASCE 7-22 design curve on both the edge and interior wall zone for all stilt heights except the 0.5 m case. The GCp values for the 0.5 m stilt height were within the negative ASCE design curves for both wall zones for 15 of the 17 averaging areas. The positive design curves from ASCE 7-22 were satisfactory for the lower roof slope for stilt heights above 0.5 m. These results indicate that the roof and wall GCp values are dependent on the stilt height for an elevated structure, thus the development of design curves as a function of stilt height may result in a more accurate estimate of GCp values for these structures.

Figure 3-17: Highest non-outlier GCp values due to downburst winds for all stilt heights on the 3:12 roof slope model.
The effect of roof pitch and stilt height on the wall and roof component and cladding loads were further investigated as a function of their deviation from the NBCC envelope curve. The deviation from the NBCC design curve was calculated by dividing the highest non-outlier value for each averaging area with the corresponding GCp values from the NBCC curve for each stilt height and roof slope as shown in Equation (3-7) below. This calculation was based on negative GCp values only.

\[
Deviation_{h} = \sum_{a} \frac{\bar{GCp}_{h,a}}{GCp_{NBCC,a}} \tag{3-7}
\]

where, \(\bar{GCp}\) refers to the highest non-outlier GCp value from the current study for a stilt height, h, and averaging area, a, and \(GCp_{NBCC}\) refers to the NBCC 2020 design value. The results were plotted and shown in Figure 3-19, where positive values indicate that the GCp values from the current study exceeded those from the NBCC curve and negative values indicate that the curve was conservative.
From these plots, it is clear that the lower roof slope model in the current study resulted in GCp values that were, on average, significantly higher than the NBCC design values for all components and cladding zones. The relationship between stilt height and deviation from the NBCC curve observed in the roof zones indicates that the design curves used for ground-mounted structures may not be appropriate for components and cladding design for elevated structures. As the stilt height increased, the deviation of peak GCp values from the current study relative to the NBCC curves increased. The correlation coefficients between stilt height and deviation from the NBCC for Zone C, S, and R were 0.71, 0.75, 0.87, respectively for the mild roof slope model. These values indicate that there is a strong linear relationship between stilt height and deviation from the NBCC curves for their respective components and cladding zones. The corner and edge zones for the roof of the 6:12 pitch model showed a larger variation in deviation from the NBCC curve for different stilt heights compared to the 3:12 pitch model. Zone C showed that the design values were conservative for all stilt heights except 1.5 m and 3 m. Zone S was slightly less conservative as four of seven stilt heights resulted in an average deviation greater than the NBCC curve. The wall zones showed a moderate linear dependency between stilt height and average deviation of negative GCp values from the NBCC curve. The positive deviation values from these zones indicates that the NBCC design values underestimated the GCp values for both roof slopes for the current model. Generally, the correlation between stilt height and deviation from the NBCC curve observed in this section suggests that stilt-height-dependent envelope curves may be necessary to adequately estimate components and cladding loads.
3.3.3.3 Base Wind Loads

Figure 3-20 presents the scattered peak area-averaged pressure coefficients on the elevated low-rise building model at a silt height of 3 m, full-scale, from the WindEEE downburst tests compared to the ASCE 7-22 design pressure coefficient envelope curves for the horizontal surface of an elevated structure. The ASCE 7-22 components and cladding zones are illustrated in Figure 3-21. The area-averaged base loads were calculated for ten different full-scale tributary areas ranging from approximately 0.25 – 22.7 m² for each height, building model, and downburst test. The 3 m stilt height represents the worst-case scenario, as it was previously determined that this test case resulted in the highest local pressures. The scatterplots for the external pressure coefficients at all stilt heights compared to the ASCE 7-22 design curves can be found in Appendix A.

Figure 3-19: Deviation of data from NBCC envelope curve.
The ASCE 7-22 GCp values for interior zone, Zone 1, adequately estimated the area-averaged wind loads on the underside of the current building model for both positive and negative pressures. All data points from the current study for both roof slope models were within the bounds of the ASCE 7-22 design curves, indicating the provisions are
conservative. The edge zone, Zone 1, had minimal cases of exceedance from the ASCE 7-22 negative design curve for the two smallest tributary areas (0.25 m$^2$ and 0.56 m$^2$). These peak GCp values were likely due to the vortices which formed around the stilts of the model. There were cases of exceedance for the positive GCp values within Zone 2 for both roof slopes and all averaging areas. For averaging areas less than 2.2 m$^2$, the mean positive GCp value for each averaging area was higher than the corresponding ASCE design value. This indicates that the positive ASCE 7-22 design provisions for Zone 2 underestimated the GCp values on the base edge for the current building geometry and test configurations. The positive design values for Zone 3 were satisfactory for the current setup for averaging areas greater than 0.56 m$^2$. Below this threshold, the current model resulted in positive GCp values that were higher than those of ASCE 7-22. The negative design values for Zone 3 were satisfactory for the current model as all data points were within the bounds of the ASCE 7-22 design curve.

The highest non-outlier values for each height and building model were calculated and plotted against the design curve for every zone, as shown in Figure 3-22, to provide a clearer investigation of the highest positive and negative external pressure coefficients relative to the ASCE design curves. The positive pressure coefficients for all heights and tributary areas are found to lie either on the ASCE design curve or exceed the design values for Zones 2 and 3. The positive pressures in Zone 2 are slightly higher than those in Zone 3, indicating that the positive pressures vary with proximity to the edge of the base. As such, it may be necessary to have independent positive design curves for each zone. The relationship between stilt height and area-averaging wind loads is apparent in Zones 2 and 3. It is clear that from stilt heights of 0.5 m – 2 m, the area-averaging negative pressures increase with height. The lowest GCp values for all zones and averaging areas resulted from the 0.5 m stilt height test cases. The difference between GCp values for stilt heights between 2 m and 3 m is not as significant as that for lower stilt heights. The results of the current study indicated that the ASCE 7-22 design curves for the base surface of elevated structures adequately predicted the negative and positive loads within the interior zone; however, the positive design pressures were inadequate for the edge and corner zones. The negative design values for averaging areas less than 1 m$^2$ in the edge zone may underpredict
the wind loads within this region, as there were several cases of exceedance for stilt heights between 1.5 m and 3 m.

Figure 3-22: Highest non-outlier external pressure coefficients on the base of the building for all stilt heights under downburst winds.

The efficacy of the ASCE envelope curves as a function of stilt height was further examined by evaluating the average difference between the peak minimum pressure coefficients and the ASCE 7-22 envelope curve for each height. The difference between the scattered data and the corresponding curve value was calculated for each averaging area and height, and the mean deviation for each stilt height was then calculated, as shown in Figure 3-23. Positive values indicate that the mean deviation from the current study exceeded the ASCE 7-22 design curve, whereas negative deviations represent data that is within the bounds of the provisions.
Figure 3-23: Deviation of base components and cladding loads from ASCE envelope curve.

These plots demonstrate the relationship between the stilt height of the model and the deviation of data points from the ASCE design curve. This relationship is strongest for the negative GCp deviation compared to the positive GCp values. As the stilt height increases, the deviation of GCp values from the current study becomes increasingly close to the ASCE curve. This relationship is nearly linear, with correlation coefficients ranging from 0.85 to 0.91. This is an indication that the magnitude of negative pressure coefficients is also dependent on stilt height. The mean deviation of negative GCp for each stilt height was within the bounds of the ASCE design curve. However, from these plots, it may be estimated that buildings with stilt heights over 4.5 m may result in area-averaged wind loads greater than those predicted by ASCE 7-22.

The positive GCp deviation within the interior zone (Zone 1) is not dependent on stilt height. Comparatively, higher stilt heights resulted in higher GCp deviations within Zones 2 and 3. This indicates that the positive pressure envelope curves would benefit from incorporating a factor relative to the stilt height of the building for the underside of elevated structures. The current provisions utilize the same positive pressure envelope curve for all zones, which satisfies the current dataset, as there is little variation in positive pressures between zones. However, the positive envelope curves would better satisfy the current dataset if there were a larger variation between tributary areas and stilt heights. Furthermore, these zones also resulted in a positive mean deviation, indicating that the average GCp value within these zones was greater than the corresponding design value. Therefore, the design values generally underestimated the wind loads within the edge and corner zones for the current model. Further refinement in the component and cladding
Loading zones to address pressure variation with both stilt height and the tributary area may be required to better represent the downburst wind loads.

### 3.3.4 Comparison of Downburst vs. ABL Wind Loads

This section aims to summarize the difference between ABL- and downburst-induced external pressure coefficients, GCp, on the base of an elevated low-rise building model. The data presented in Figure 2-30 and Figure 3-22 represents the highest non-outlier enveloped area-averaged external pressure coefficients for all stilt heights due to ABL and downburst winds, respectively. This information has been combined and plotted in the scatterplots in Figure 3-24. A shift of data points below the unity line is observed for all zones, which indicates that the downburst winds generally produced higher external pressure coefficients compared to ABL winds. This shift is most prominent in Zone 2, particularly for the higher stilt heights. The lowest stilt height, 0.5 m, is not significantly impacted by the different winds as all data points lie close to the unity line. Zone 1 shows that the majority of GCp values are greater than those of ABL, although the difference is not as significant as that for Zone 2. Zone 3 had a larger variation across both sides of the unity line, indicating that the downburst wind field did not contribute to significantly different wind loads on the base corners of the model.

![Figure 3-24: Scatter plots of ABL and downburst peak negative GCp on the base of the building for all stilt heights.](image)

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3.4 Conclusions

The external pressures were measured for a low-rise gable roof building model with two roof slopes (3:12 and 6:12) and seven stilt heights (0.5 m to 3 m) in the WindEEE Dome impinging jet downburst simulator at Western University. Five 30-second continuous downbursts were simulated for each building orientation and stilt height, and the resultant surface pressures were measured. The surface pressure measurements were used to compute external pressure coefficients, GCₚ, in accordance with current NBCC and ASCE wind loading codes and compared to atmospheric boundary layer wind pressures obtained from wind tunnel tests at the Boundary Layer Wind Tunnel Laboratory. Additionally, the downburst pressures were compared to the new component and cladding load provisions in the ASCE 7-22 standard for elevated structures.

The evaluation of external pressure coefficients on the elevated model under downburst winds led to the following key findings:

- An increase in stilt height results in increased suction on the horizontal base surface of elevated buildings as well as the roof surfaces. The correlation between stilt height and surface pressure is stronger for the base surface compared to roof surfaces.
- A lower roof slope results in higher downburst-induced pressures on both the roof and base surfaces of an elevated building.
- Downburst winds resulted in higher external pressure coefficients on the base surfaces of the building model for both roof slopes compared to ABL winds.
- The ASCE 7-22 design curves for positive components and cladding loads are not conservative in estimating wind loads on elevated structures.
- The corner zones from ASCE 7-22 for the base surface of elevated structures are conservative compared to the cases considered in this study.
- The base edge and interior component and cladding zones from ASCE 7-22 may require further refinement to ensure design pressures adequately estimate loads within these regions.
References


Chapter 4

4 Conclusions

The current study evaluated the surface pressures on an elevated low-rise, gable-roof building using a high frequency pressure integration (HFPI) model tested under atmospheric boundary layer (ABL) winds and downburst winds simulated at the Boundary Layer Wind Tunnel Laboratory (BLWTL) and WindEEE Dome respectively. Surface pressure measurements were obtained from 352 and 360 taps distributed across the surfaces of the building model with roof pitches of 3:12 and 6:12 respectively. The building model was selected to represent a multi-unit rowhouse with plan dimensions similar to those in northern Canada. The building had full-scale plan dimensions of 15.8 m and 23.8 m, and an eave height of 7.3 m, with four stilts at each corner of the model’s base. Seven stilt heights were tested for both building models, ranging from 0 m to 3.5 m at 0.5 m increments. The ABL tests included twenty-one wind directions from 0° to 180°, whereas the downburst tests included four building orientations (0°, 30°, 60°, 90°).

The local and area-averaged pressures were evaluated to determine correlations between roof pitch, building orientation, and stilt height on the magnitude and distribution of pressure across the building surfaces. The statistical peaks from each tap time history were used for both the local and area-averaged pressure coefficient analysis. A component and cladding analysis was conducted by calculating the area-averaged external pressure coefficients in accordance with NBCC (2020) and ASCE 7-22. Finally, the adequacy of the current provisions was investigated with respect to the current model.

4.1 Summary of Findings

The conclusions drawn from this study are summarized as follows:

- The enveloped external pressure coefficients, $G_{C_p}$, pertaining to the component and cladding of a perfectly sealed building from downburst-induced wind loads exceeded those from ABL winds on the base horizontal surface of the building. These differences highlighted the impact of the highly divergent and intense winds associated with downburst events.
• A higher roof slope resulted in higher suction on the base surface of the building model under downburst winds, whereas the effect of roof slope was negligible for base loads due to ABL winds.

• An increase in stilt height results in higher suction on the base horizontal surface of the building model under both ABL and downburst winds. The total surface area of suction on the base surface also increases with stilt height, indicating that component and cladding design zones are dependent on stilt height. This agrees with the newly incorporated design zones for elevated buildings in ASCE 7-22.

• The component and cladding design loads in ASCE 7-22 were found to be conservative when estimating the negative ABL-induced wind loads on the base horizontal surface of elevated buildings. However, the positive design pressures were lower than observed external pressure coefficients from the current model.

• The downburst-induced component and cladding design loads exceeded NBCC and ASCE 7-22 design provisions for both positive and negative pressures on the wall, roof edge, roof ridge, and interior base zones. NBCC design pressures were inadequate for nearly all tributary areas and zones for the current study, whereas ASCE wind loads performed better for both ABL winds and downbursts.

4.2 Recommendations for Future Work

The following recommendations for future research can be made to supplement and extend the work completed in the current study:

• The analysis can be extended to various building geometries, surrounding buildings, and downburst wind fields to generalize downburst-induced wind loads on structures. Experimental testing is associated with several limitations such as geometric scale and economic cost. As such, numerical methods such as computational fluid dynamics could aid in the effort of conducting larger parametric studies of downbursts.
• Naturally occurring downbursts are reported to translate and consist of varying downdraft velocities. Future research may aim to incorporate translation, pulsating impinging jets, and/or ABL winds in conjunction with the downburst winds to better simulate downburst wind characteristics.

• Further work is recommended to standardize downburst scaling methods, as well as the use of reference velocities for the derivation of downburst-induced pressure coefficients.
Appendix

A: C & C Loads due to Downburst Winds on Base Surface
Figure A-0-1: GCp values on base horizontal surface from downburst winds.
Curriculum Vitae

Name: Kate Current

Post-secondary Education and Degrees:
The University of Western Ontario
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2016-2021 BESc.

Honours and Awards:
Dean’s Honour List
Alan G. Davenport Memorial Scholarship
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Teaching Assistant
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