Performance Requirements for Climate Resilience of Residential Roofs

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Abstract

The roof is one of the most important elements of the building envelope. It offers protection for the homeowners against weather elements. According to the Institute for Catastrophic Loss Reduction (ICLR) houses are less engineered and are much more prone to damages during weather events. Recent climatic changes have increased the severity of weather events and roof failures. This demands the necessity of updating the design, evaluation, and construction methods for asphalt shingle roofs with a holistic approach.

A disconnect exists between the service life of a roof and the advertised warranties of asphalt shingles by the manufactures. This is misleading and prevents homeowners from making informed decisions about their roof selection. This thesis reviewed not only the roof's performance in the field but also the current practices followed for the design, evaluation, and construction methods. The review respectively concluded that the current design is based on only historical data, both components and systems are mostly evaluated under lab conditions, rather than being exposed to the weather conditions experienced during the service life, and lack of installation quality assurance.

The objective of this thesis is to develop the initial framework for a “Holistic Approach” that would increase the resilience of residential roofs. This has been accomplished in three folds:

1. Quantified the changes in the design wind loads, using both the current and projected changes in wind pressures due to global warming magnitude. Data provided by Environment and Climate Change Canada was used for this analysis and a new web design tool “Climate-RCI” for climate severity classification of cities across Canada was created.

2. Performed an extensive experimental evaluation of components and systems, following established methods and procedures, to quantify the effect of weathering on the resistance of asphalt shingles and roof mock-ups. To achieve this both components and systems were evaluated. For the component, the resistance after being aged in the field for a minimum of 13 years displayed a maximum decrease of 59%. Measured properties of the majority of the aged components no longer met the minimum requirements outlined in the respective standards referenced in the National Building Code of Canada. The system resistance was significantly reduced due to weather exposure.

3. Developed quality assurance guidelines based on best practices in consultation with the roofing industry for various climatic severities.

This thesis identified three major future research areas, namely, calibration of an appropriate resistance factor for residential roofs, correlating lab verse field weather exposure conditions, and probabilistic determination of the installation process.

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Glossary

Symbols

I_w	Importance factor
q	Reference wind pressure
C_t	Topography factor
C_e	Exposure factor
C_pC_g	External peak value
C_{ei}	Internal exposure factor
C_{pi}	Internal pressure coefficient
C_{gi}	Internal gust coefficient
P	Pressure
ρ	Average air density
V	Wind speed
L_{ni}	Load effects
$\mathrm{R_n}$	Nominal strength
γ_i	Load factors

Abbreviations

ICLR	Institute for Catastrophic Loss Reduction
NBCC	National Building Code of Canada
OBC	Ontario Building Code
FEMA	Federal Emergency Management Agency
CSA	Canadian Standards Association
ASTM	American Society for Testing Materials
TAS	Testing Application Standard
FM	Factory Mutual
DOD	Degree of Damage
RCP	Representative Concentration Pathway
FS	Factor of Safety
LRFD	Load Resistance Factor Design
ASD	Allowable Stress Design
MD	Machine Direction
XD	Cross-Machine Direction
RH	Relative Humidity
PT	Pressure Tap

Chapter 1: Introduction

1.1 Background

The Institute for Catastrophic Loss Reduction (ICLR) reports that residential houses are less engineered and more prone to damages during severe winds [1]. Therefore, this thesis is focused on the most exposed element of a residential house, which is the roof covering. Steep slope residential roofs composed of asphalt shingles were selected due to their large popularity as a roof covering [2]. Also, there is a lack of guidelines for increasing their resistance against extreme winds. Roofs are mainly classified as low slope roofs, such as those typically used for commercial buildings and schools (Figure 1.1 a), and steep slope roofs used in residential houses (Figure 1.1 b).

Figure 1. 1 (a) Low slope roof [3]and (b) steep slope roof [2]

The Government of Canada is currently undertaking an initiative to integrate climate resiliency into building design, guides, and codes to mitigate damages from climate change. Until now, the concentration has been on increasing the resiliency and performance of low slope membrane roofs. Apart from typically being less engineered, residential roofs are experiencing an even higher demand due to climate change. This is because of the increased frequency and severity of extreme weather events, which translates into higher and repetitive loading on the roofs. Therefore, a holistic-based approach is essential to design and construct robust and durable roofs that will be able to perform under variable and severe weather conditions.

One of the loads roofs must resist is the wind uplift. Over a period of 30 years, wind was identified by the Insurance Bureau of Canada to be responsible for over 60% of catastrophe events that occurred in that time frame [4]. Wind is the major contributor to damages to residential buildings located in Canada, with the roof being the top element severely affected by it [5]. Wind can cause tremendous damages if the structures are not properly designed, constructed, and maintained to withstand the wind loading associated with the recent climate change. Therefore, this thesis will be concentrating on developing a holistic approach for the wind performance of steep slope asphalt shingle roofs.

1.2 Research Motivation

Documented records of damages by natural catastrophes show a substantial increase in the average cost of insured loss from disasters, which has increased from an average of USD 5.1 billion at the end of 1989 to USD 27 billion at the end of 2010 [6]. Disasters with a meteorological and hydrological origin are the leading cause, with over 75% of the damages [6]. Out of these wind and floods are the most disastrous [7]. This increasing trend of losses from natural catastrophes at an international level can be seen in a graph produced in 2011 by the global insurance company Munich Re, in Figure 1.2 [6].

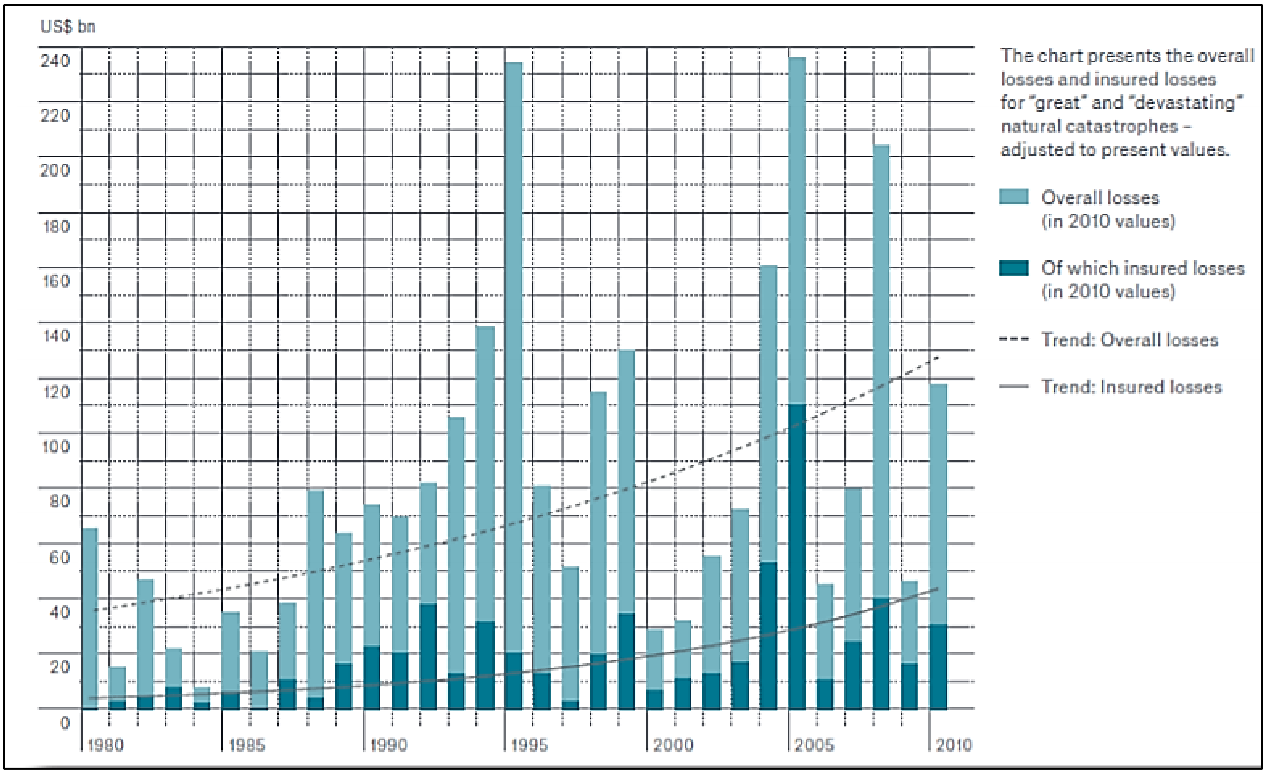


Figure 1. 2 Losses from natural catastrophes on a world level [6]

An increasing trend in the frequency of hurricanes and the resulting damages during the period of 1989 to 2017 can be seen in Figure 1.3 for the US. This trend continued in the US in 2019 with over sixteen thousand wind damage reports (Figure 1.4), registered predominantly within the period of April to August of each year during the so-called "Hurricane Season" [8].

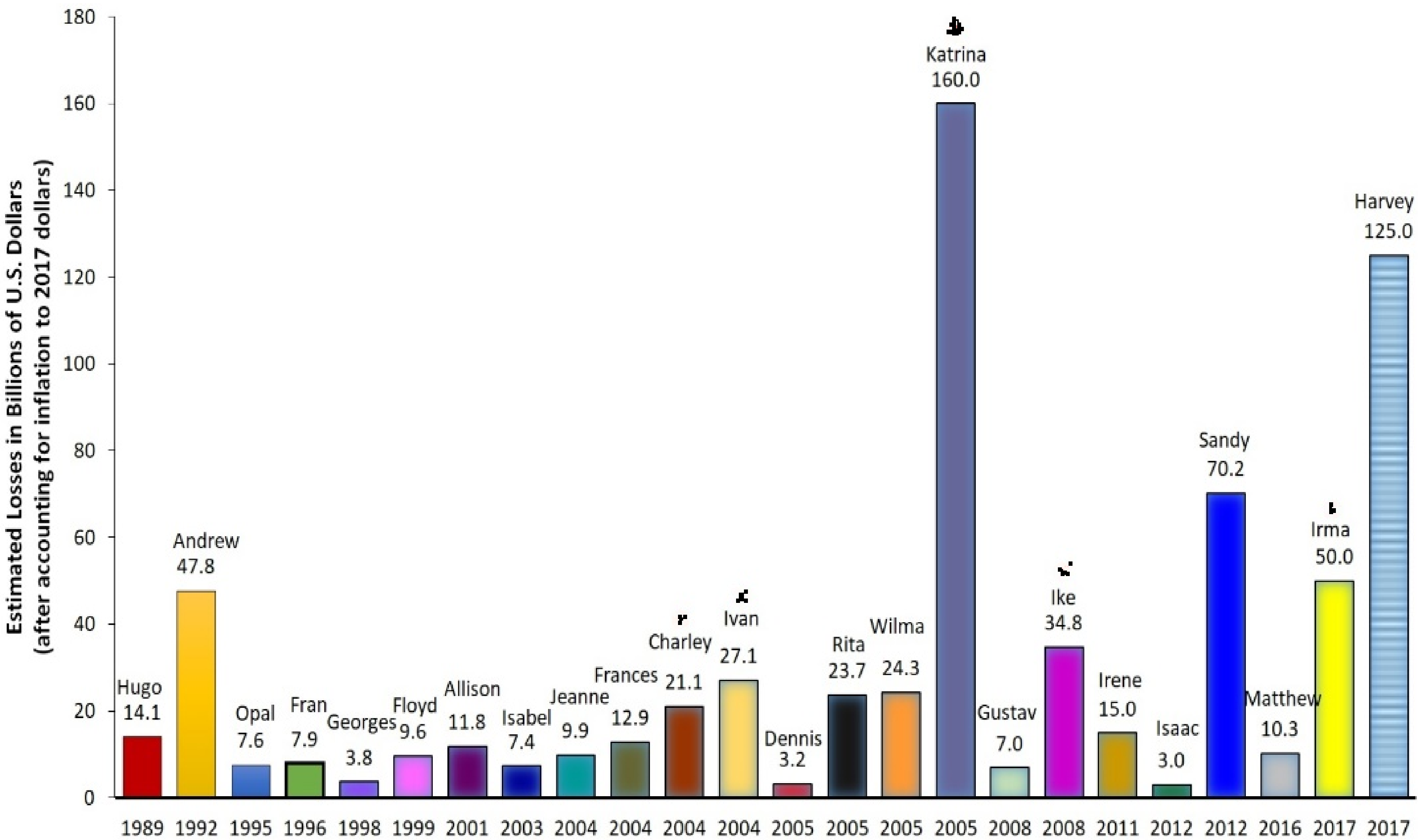


Figure 1. 3 Estimated losses due to hurricanes in the US from 1989 to 2017 [8]

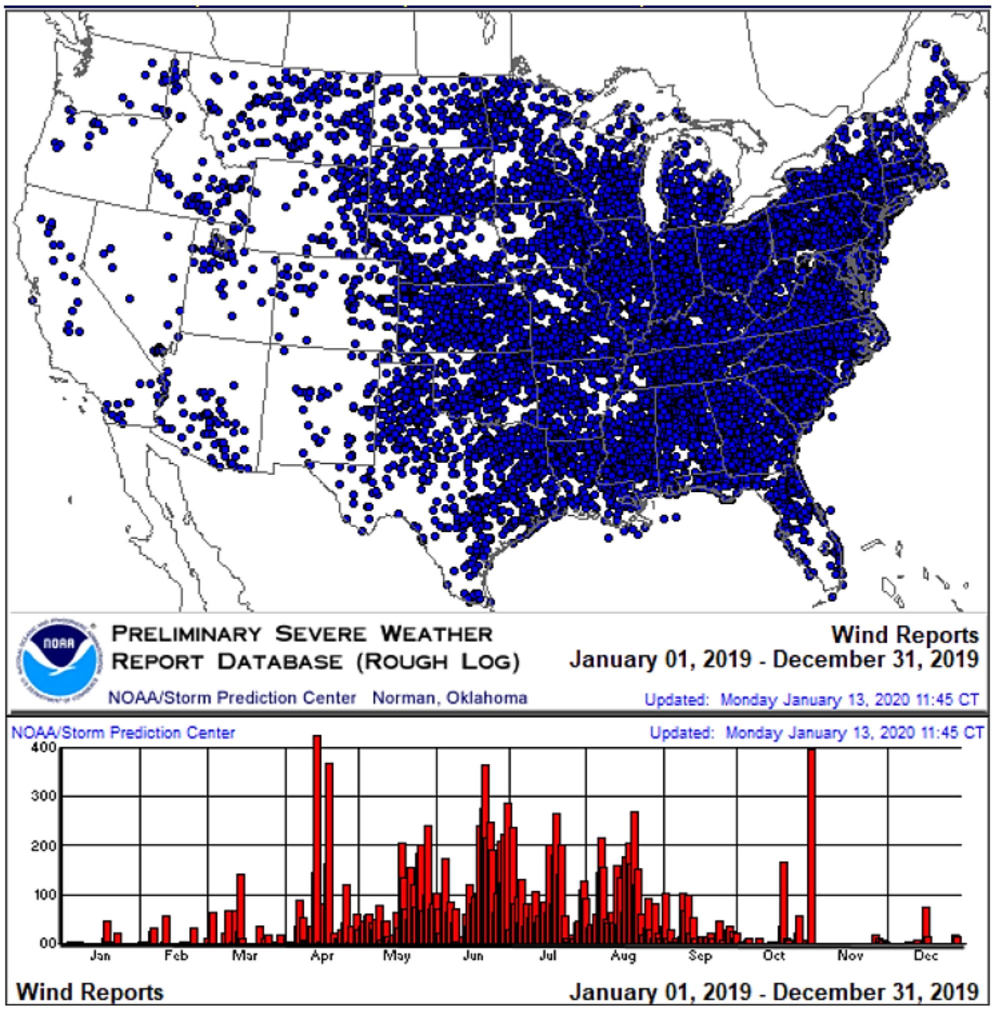


Figure 1. 4 Reports of wind damages across the USA for the year 2019 [8]

In Canada, the total number of catastrophic events caused throughout the years and the cost of damages were plotted from 1983 to 2018, using data available from the Insurance Bureau of Canada (Figure 1.5) [9]. This data was further broken down to identify the percentage of events caused by wind, rain or flooding, hail, or tornados/hurricanes. Starting in 2008 an interesting trend emerged and continued into later years with combined catastrophic events (wind, hail, flooding) being responsible for damages rather than a single event. Combined catastrophic events composed 48% of all events highlighting that the mitigation solution cannot be a single resolution but rather a comprehensive one. Apart from the upwards trend in the number of catastrophic events, the cost of the damages caused by these events is also increasing.

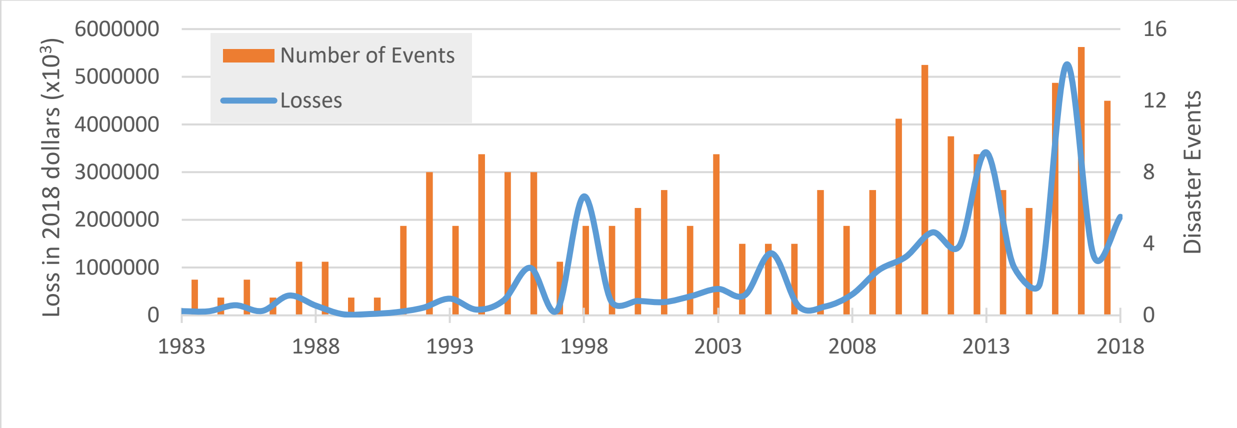


Figure 1. 5 Loss from catastrophic events in Canada [9]

These figures provide a brief overview of how costly and frequent damages caused by wind disasters are. Thus they highlight the need to mitigate the wind-induced damages of affected structures.

Climate change challenges conventional design methods, which can increase the failure risk when faced with frequent weather shifts. The current thesis motivation is witnessing the damages that severe weather such as storms, hurricanes, or tornadoes have caused in recent years, especially to residential houses. This thesis aims to determine a holistic approach to increase the resiliency of steep slope asphalt roof cladding to extreme wind loadings caused by the recent climate change.

1.3 Research Objectives

The main objective of the current research is to develop design recommendations to increase the resiliency performance of steep slope residential roof cladding. This will be achieved following a twofold approach. First, by quantifying the changes in the wind effect prompted by climate change, followed by determining the resistance check of components from the asphalt roof assembly. The load accounting for climate change projections and the resistance accounting for weathering effects are not specified as part of the design requirements in the National Building Code of Canada (NBCC) or other standards. This thesis' holistic approach aims to fill this void.

To achieve the main objective a review of the current design methods and construction practices for the cladding of steep slope residential roofs will be presented to identify the missing links in the determination of the load and resistance. Furthermore, the weathering effects on the resistance will be quantified at a component and system level.

The work completed in the current research is the first phase of the ongoing initiative to increase the resiliency of steep slope residential roofs. This work will contribute to the ongoing efforts of the National Research Council of Canada to integrate climate resiliency in the design codes. It will aid practitioners in using future climatic data in their designs.

1.4 Research Methodology

For estimating the effect of the climate change on wind loading to be applied in roof design, historical and projected changes in future climatic data were collected from the Environment and Climate Change Canada. To quantify the effects of the future climatic data a severity classification for roofing systems was introduced. Based on the current NBCC design recommendations for cladding and considering the future climatic data, a web tool Climatic-RCI was developed. Climate-RCI provides accessibility for the designers and practitioners to implement the new Holistic Approach.

To quantify the impact of weathering on the resistance of a steep slope residential roof, the resistance of the system was determined as well as the resistance of one of the components, asphalt shingles. On the component level, the quantification of weathering impact was achieved through experimental evaluation. These evaluations were performed at the National Research Council's Building Envelope's Interface Facility. On the system level, the quantification of weathering impact was determined by performing extreme wind testing using the Wind Induced Damage Simulator located at the University of Ottawa. Both the component and system testing was performed on two types of roofing samples: newly purchased/installed and naturally aged in the field through exposure to Ottawa's environmental conditions.

1.5 Thesis Structure

The research presented in this thesis is structured in chapters as described below:

- **Chapter 2 "Review of Current Practices and Performance"** contains a review of the current design and construction practices for the cladding design of steep slope residential roofs. The field performance these practices yield under severe weather as well as under typical conditions are briefly summarized. This is accomplished through the review of post-storm reports and a homeowners' survey.

- **Chapter 3 “Development of the Holistic Approach”** presents how global warming impacts the reference wind speed. It proposes how to address the missing links identified in Chapter 2 to improve the resiliency of steep slope residential roof coverings and describes the development of the Holistic Approach.
- **Chapter 4 “Experimental Setup and Procedures”** outlines the procedure and experimental setup used to quantify the impact of weathering on the component and system resistance.
- **Chapter 5 “Quantification of the Weather Impact on the Resistance- Results and Discussion”** presents and discusses the results obtained from the component and system testing.
- **Chapter 6 “Implementation of the Holistic Approach”** provides a step-by-step example of how the Holistic Approach can be implemented and outlines the quality assurance guidelines developed.
- **Chapter 7 “Conclusions and Recommendations for Future Work”** concludes and summarizes the research achievements of this thesis as well as outlines the suggested recommendations for future work on this exciting topic.

Chapter 2: Review of Current Practices and Performance of Roofing Systems

2.1 Introduction

The roof structure must be designed according to a set of required codes and standards. This ensures the roof will continue to perform as intended under the different loading conditions during its service life without failing and endangering the inhabitants' lives. The design of steep slope residential roof cladding follows the National Building Code of Canada's (NBCC) [10] requirements found in Part 4 or Part 9 depending on the occupied overall dimensions. The roofing systems' design procedure is reviewed and an overview is presented in the current chapter. The overview highlights the evolution of the reference wind pressure chronologically, along with its determination in the current version of the National Building Code of Canada.

Part 9 of the NBCC, apart from outlining design procedures, also provides construction requirements for steep slope residential roof cladding. These are summarized in this chapter. If constructed properly, then the roof can resist the load it is designed for. In this case, the resistance of the roof must be equal to or greater than the design load. This resistance can be determined for the entire steep slope residential roof cladding system or components composing the system. The evaluation protocols for determining these two types of resistances are reviewed and compared.

The field performances based on post-storm reports under severe weather are briefly summarized, whereas the performances under normal weather conditions are captured through a homeowners' survey.

This chapter concludes by identifying the missing links in the current design, construction, and evaluation approaches.

2.2 Design Approach for Steep Slope Residential Roof Cladding as per NBCC

In Canada, the construction industry relies on the National Building Code of Canada (NBCC) and provincial codes, such as the Ontario Building Code (OBC). Mostly, the OBC adopts the NBCC's requirements [11]. Based on a recent Memorandum of Understanding (MOU) between the federal and provincial governments signed on September 25th, 2020, future adoption of the NBCC by the provinces, will be "*timely and consistent*" [12]. This agreement will minimize the current time lag that exists in the adoption of the NBCC by the provincial codes. The NBCC was first published in 1941. It aims to update every five years to ensure new technologies, materials, methods, and other necessary regulatory requirements are incorporated. The NBCC code, however, only provides the minimum requirements for the design and construction of new buildings. The current version of the NBCC code issued in 2015 contains two volumes and a total of 9 parts. The parts discussed in this thesis, can be seen identified in Figure 2.1.

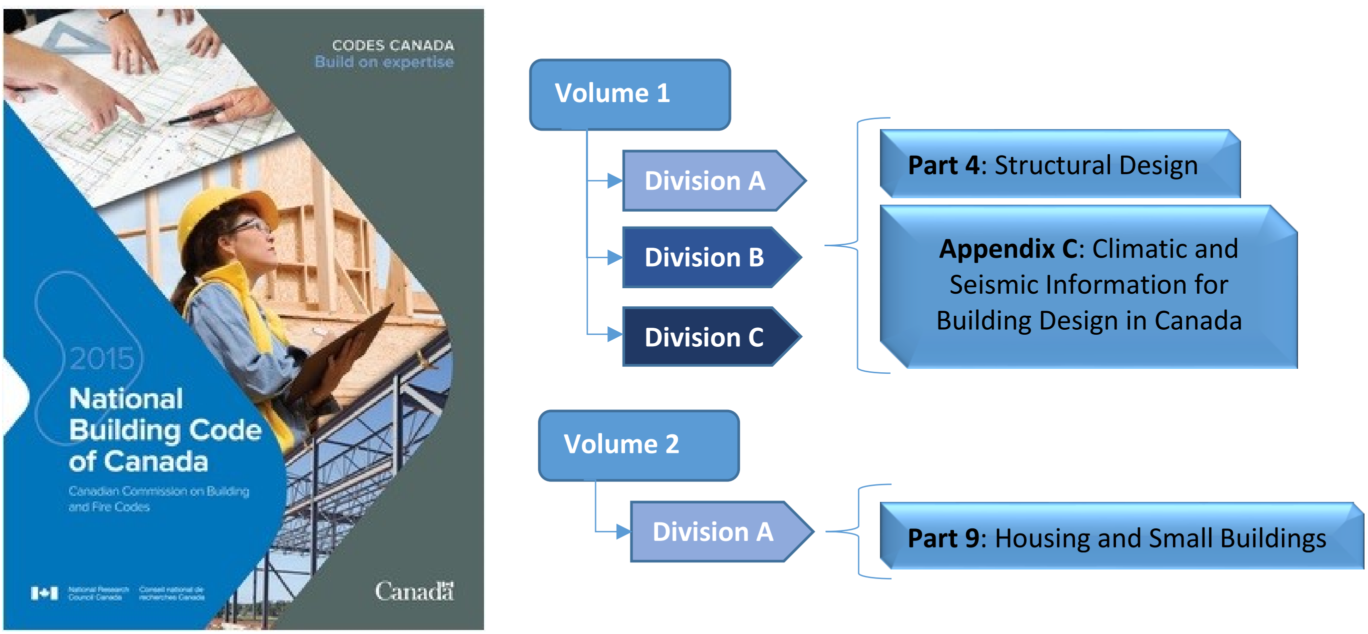


Figure 2. 1 Current engineering practice for cladding design

NBCC-Part 4, "Structural Design," concentrates on calculating the different loads that the building will need to withstand during its design life, such as dead, live, wind, snow or rain, and earthquake loads. Some of these loads are highly dependent on the location of the structure. Appendix C of the NBCC identifies the climatic loads for locations across Canada, while also providing a brief description of the data's origin. This appendix must be consulted during the design phase. NBCC-Part 4 is applicable for buildings with major occupancy, post-disaster buildings, and residential and small buildings with an area greater than 600 m^2 or a height greater than three storeys. NBCC-Part 9, "Housing and Small Buildings," provides prescriptive requirements for houses and small buildings that are either three stories high or less than 600 m^2. To design structures classified under the NBCC-Part 9, calculating the wind load acting on the roof is not a requisite.

In the 2015 version of the NBCC, the design procedures for wind load calculations are outlined in section 4.1.7. The wind uplift pressure on the roof is determined by calculating the net pressure between the external and internal pressures using the following formula [10]:

$$\boldsymbol{P = I_w q C_t (C_p C_g C_e - C_{pi} C_{gi} C_{ei})} \tag{2.1}$$

The importance factor (I_w) is specified in Table 4.1.7.3 of the NBCC as low, normal, high, or post-disaster and serves to provide additional strength for structures such as schools or hospitals [10].

The wind depends on the structure's location, and to account for this, different locations have unique reference wind pressures (q). This value can be found in the last column of Table C-2 in Appendix C for 678 different locations across Canada (Figure 2.2).

Table C-2
Climatic Design Data for Selected Locations in Canada

Province and Location	Elev., m	Design Temperature				Degree-Days Below 18°C	15 Min. Rain, mm	One Day Rain, 1/50, mm	Ann. Rain, mm	Moist. Index	Ann. Tot. Ppn., mm	Driving Rain Wind Pressures, Pa, 1/5	Snow Load, kPa, 1/50		Hourly Wind Pressures, kPa	
		January		July 2.5%												
		2.5% °C	1% °C	Dry °C	Wet °C								S_s	S_r	1/10	1/50
British Columbia																
100 Mile House	1040	-30	-32	29	17	5030	10	48	300	0.44	425	60	2.6	0.3	0.27	0.35
Abbotsford	70	-8	-10	29	20	2860	12	112	1525	1.59	1600	160	2.0	0.3	0.34	0.44
Agassiz	15	-9	-11	31	21	2750	8	128	1650	1.71	1700	160	2.4	0.7	0.36	0.47
Alberni	12	-5	-8	31	19	3100	10	144	1900	2.00	2000	220	2.6	0.4	0.25	0.32
Ashcroft	305	-24	-27	34	20	3700	10	37	250	0.25	300	80	1.7	0.1	0.29	0.38

Figure 2. 2 Climatic data in the NBCC Table C-2 [10]

The hourly wind pressures were first incorporated into the NBCC code in 1961. These were based on the hourly averaged wind speed measured through 45B contact anemometers, which recorded speed measurements during an entire hour [13]. In 1961 only a limited number of stations recorded data (Figure 2.3). However, that soon changed with U2A anemometers being installed at every airport and these measured the average one-minute wind speed at 10 m above the ground in open terrain with no obstructions, and then this switched to a two-minute average in January 1985 [13].

The range of wind speed data that is currently used in the NBCC, is collected from various weather stations across the country for the past 10 to 54 years. The different measurements from several sources can be seen in Figure 2.3, in contrast to 1961 where only one source of data was available from a limited number of stations [10]. When different wind speed sources are used, all the values are converted to the same units (km/h), and data is corrected to a 10 m height using a wind profile if it is measured at different heights [14]. Similar information for the rain and temperature design parameters can be found in Appendix C of this thesis.

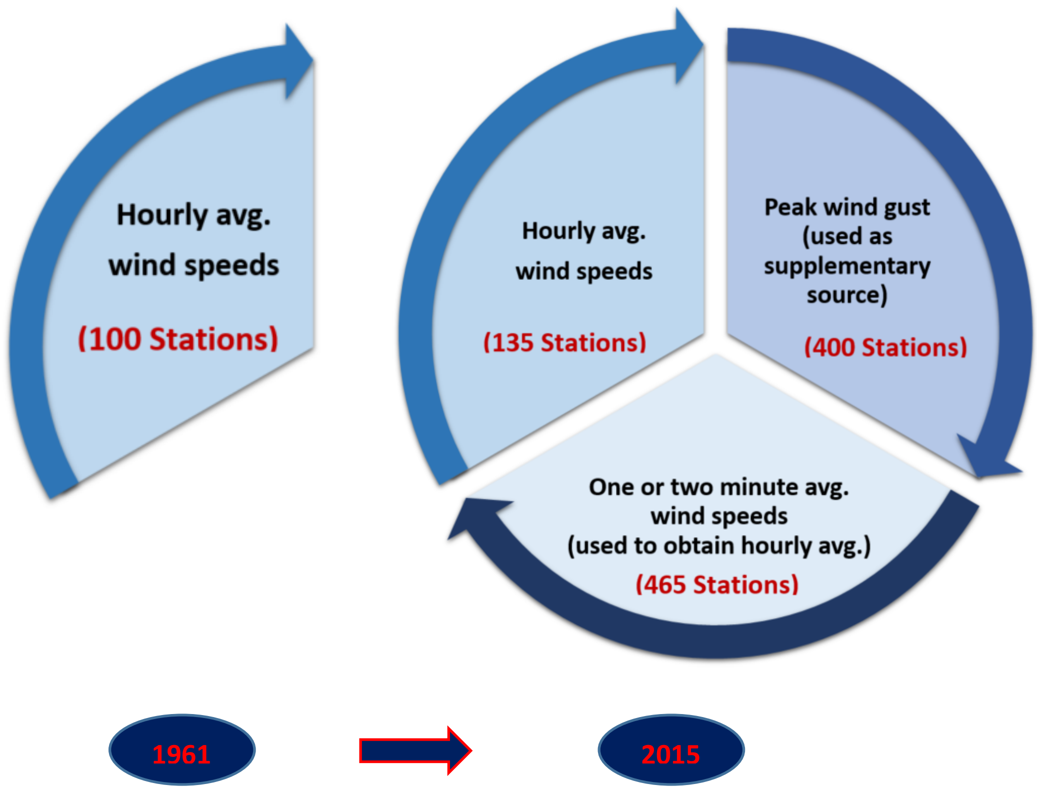


Figure 2. 3 Range of wind data available from weather stations

The values listed in Table C-2 in the 2015 version of the NBCC code are determined in three steps [10], [15]:

- **Step 1**: determining the maximum annual wind speed
- **Step 2**: fitting the maximum annual wind speed data to the Gumbel distribution using the method of moments
- **Step 3:** computing the reference wind pressure for 10 and 50 years return periods.

The conversion from wind speed to pressure is done using the following equation [10]:

$$q = \frac{1}{2}\rho V^2 \tag{2.2}$$

where ρ is the average air density (1.2929 kg/m^3) and V is the wind speed (m/s).

The Gumbel distribution formula is expressed below [16]:

$$F_{GU}(x;\theta) = exp(-exp\left(-\frac{(x-u)}{a}\right)) \tag{2.3}$$

where x is the value of the random variable analyzed and "u" and "a" are the location and scale parameters, respectively. The random variable for a specific return period "T" is given by the following equation:

$$x_T = u + a(-\ln\left(-\ln\left(1-\frac{1}{T}\right)\right)) \tag{2.4}$$

Using the method of moments the parameters are calculated as shown below, where γ is taken as 0.5772 and "s" is the standard deviation and "m" the sample mean :

$$\hat{a} = s\sqrt{6/\pi} \tag{2.5}$$

$$\hat{u} = m - \gamma s\sqrt{6/\pi} \tag{2.6}$$

There were three return periods of 10 years, 30 years, and 100 years in the 1995 version of the NBCC code. The 10 years and 100 years return periods were determined based on the 30 years return period and a ratio of its standard deviation to the annual maximum wind speed [15]. This approach was replaced by the use of a coefficient of 1.4 alongside the 50 years return period. This combination was calibrated based on a coefficient of variability that has a probability of failure of 1.35x10^{-3} [15]. The load factor of 1.4 is introduced to account for uncertainty in the elements used in the Gumbel distribution [16]. According to Hong et al. [16], using the Gumbel distribution is preferred over other methods when determining the wind speed return periods due to its straightforwardness. However, one of its drawbacks is the limited data used, which is restricted only to maximum annual wind speeds.

The effect of the topography on the structure is taken into account using the topography factor (C_t). This factor is taken to be equal to one unless the building is located on a hill or an escarpment with a specific slope as detailed in NBCC section 4.1.7.4 [10]. The presence of a hill or an escarpment can significantly affect the wind loads acting on a building. It can cause "… *sudden accelerations in wind velocity that can sometimes increase wind loads by up to 80%*" [17].

The effect of the density of surrounding terrain and buildings is taken into effect through the Exposure Factor (C_e), for which the NBCC identifies three possible exposures:

- Opens terrain- "scattered buildings, trees, or other obstructions, open water or shorelines" [10].

  $$C_e = (\frac{reference\ height}{10})^{0.2} \quad (2.7)$$

- Rough terrain- "suburban, urban, or wooded terrain for at least 1 km or 20 times the height of the building" [10].

  $$C_e = 0.7(\frac{reference\ height}{12})^{0.3} \quad (2.8)$$

- Intermediate- "when the rough terrain extends for less than 1 km and the building is less than 50 m tall, the value of C_e may be interpolated between the values for the open and rough terrains " [18]

  $$C_e = C_{er}\left(0.816 + 0.184\ log_{10}(\frac{10}{x_r - 0.0510}\right) \leq C_{eo} \quad (2.9)$$

The presence or the lack of structures and or obstructions affects the velocity and turbulence of the wind. In the case of rough terrain, the wind speed can be low, but it has a higher chance of displaying higher turbulence [17].

The product of the gust factor (C_g) along with the external pressure coefficient (C_p) for low-rise buildings is given in a number of figures in the NBCC, covering [10]:

- walls
- various roof slopes
- primary structural members
- cladding and secondary structural members.

The figure used for determining external C_pC_g for the cladding of single-span gabled and hipped roofs with a slope greater than 7° can be seen in Figure 2.4.

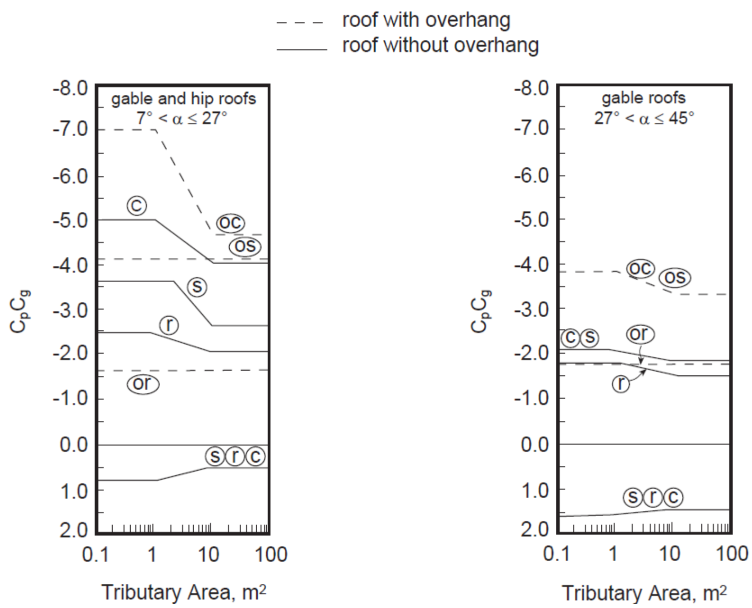


Figure 2. 4 External C_pC_g for single-span gabled and hipped roofs with a slope greater than 7◦ for structural members and cladding [10]

In the equation (2.1) discussed above, the reference wind pressure is the only factor affected by climate change. However as described above, the conventional design procedure uses a reference wind pressure based on historical data. The effect of climate change needs to be accounted for to allow for the design of more resilient roof cladding. How the reference wind pressure can be affected by climate change will be discussed in more detail in Chapter 3.

2.3 Construction Methods for Steep Slope Residential Roof Cladding as per NBCC

This section reviews the process of steep slope residential roof cladding installation outlined in Part 9 of the National Building Code of Canada. In the overall house construction schedule, the roof installation begins once the following are completed:

- preparation of building site
- pouring of the foundation
- completion of wall framing,

This can correspond to the third month if the entire house construction is planned for 6 months [19].

The first step in constructing a residential roof is constructing the frame which must be able to withstand the wind, snow, and rain loads. This is achieved mostly through the use of dimensional lumber which are joined together by sill plates and nails. Since 2012, the OBC has added an extra clause to reduce the spacing of roof sheathing fasteners on intermediate supports to 150 mm, from the original 300 mm [4]. However, this only applies if the sheathing supports are spaced at a distance greater than 406 mm on centre [11]. This change was introduced to mitigate the failures observed during post-storm investigations.

The installation of the deck on top of the frame can be seen in Figure 2.5. The roof deck which can be constructed out of OSB or plywood serves as the foundation of the roof upon which everything is placed and connected to.

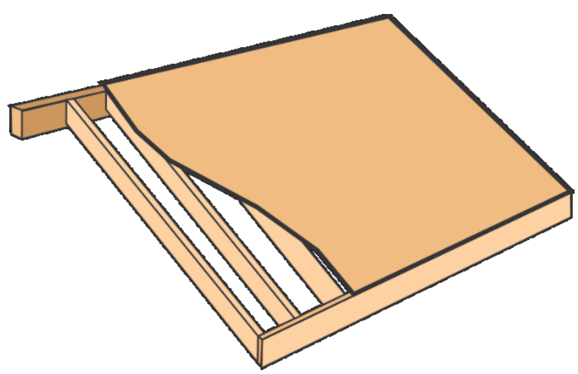


Figure 2. 5 Deck installation on top of the roof frame

For OSB, NBCC-Part 9 recommends the requirements outlined in CSA O437.0 "*OSB and Waferboard*" and CSA O325 "*Construction Sheathing*" to be followed. OSB panels are installed such that the face orientation forms right angles with the members [10], while plywood is required to be installed such that the grain forms right angles with the roof frame. NBCC-Part 9 also specifies one of the following standards to be followed for plywood:

- CSA O1231 "*Douglas Fir plywood*"
- CSA O151 "*Canadian softwood Plywood*"
- CSA O153 "*Poplar Plywood*"

A minimum spacing of 2 mm is required between the joints of plywood, OSB, and waferboards while the minimum thickness of roof sheathing is classified based on the maximum spacing of the supports and can be seen in Figure 2.6 [10].

Table 9.23.16.7.-A
Thickness of Roof Sheathing
Forming Part of Sentence 9.23.16.7.(2)

Maximum Spacing of Supports, mm	Minimum Thickness, mm				
	Plywood, and OSB, O-2 Grade		OSB, O-1 Grade, and Waferboard, R-1 Grade		Lumber
	Edges Supported	Edges Unsupported	Edges Supported	Edges Unsupported	
300	7.5	7.5	9.5	9.5	17.0
400	7.5	9.5	9.5	11.1	17.0
600	9.5	12.5	11.1	12.7	19.0

Figure 2. 6 Extract of NBCC 2015- Part 9 (Thickness of Roof Sheathing) [10]

The minimum length of fasteners required according to the NBCC can be seen in Figure 2.7 and it depends on the sheathing thickness and the type of fastener used [10].

Table 9.23.3.5.-A
Fasteners for Subflooring and for Sheathing where the 1-in-50 HWP < 0.8 kPa and $S_a(0.2) \leq 0.70$
Forming Part of Sentence 9.23.3.5.(1)

Element	Minimum Length of Fasteners, mm				Minimum Number or Maximum Spacing of Fasteners
	Common or Spiral Nails	Ring Thread Nails or Screws	Roofing Nails	Staples	
Board lumber 184 mm or less wide	51	45	n/a	51	2 per support
Board lumber more than 184 mm wide	51	45	n/a	51	3 per support
Fibreboard sheathing up to 13 mm thick	n/a	n/a	44	28	
Gypsum sheathing up to 13 mm thick	n/a	n/a	44	n/a	
Plywood, OSB or waferboard up to 10 mm thick	51	45	n/a	38	150 mm o.c. along edges and 300 mm o.c. along intermediate supports
Plywood, OSB or waferboard over 10 mm and up to 20 mm thick	51	45	n/a	51	
Plywood, OSB or waferboard over 20 mm and up to 25 mm thick	57	51	n/a	n/a	

Table 9.23.3.5.-B
Fasteners for Sheathing where 0.8 kPa ≤ 1-in-50 HWP < 1.2 kPa and $S_a(0.2) \leq 0.90$ or where $0.70 < S_a(0.2) \leq 0.90$
Forming Part of Sentence 9.23.3.5.(2)

Element	Minimum Length of Fasteners, mm			Minimum Number or Maximum Spacing of Fasteners
	Common, Spiral or Ring Thread Nails	Screws	14-gauge Staples	
Board lumber 184 mm or less wide	63	51	63	2 per support
Board lumber more than 184 mm wide	63	51	63	3 per support
Plywood, OSB or waferboard up to 20 mm thick(1)	63	51	63	150 mm o.c. along edges and 300 mm o.c. along intermediate supports; and for roof sheathing where HWP is equal to or greater than 0.8 kPa and less than 1.2 kPa, 50 mm o.c. within 1 m of the edges of the roof
Plywood, OSB or waferboard over 20 mm and up to 25 mm thick	63	57	n/a	

Figure 2. 7 Extract from NBCC 2015- Part 9 (Roof Sheathing Fastener requirements)

Research has shown that increasing the minimum sheathing fastener requirement from 51 mm to 63 mm for nails can reduce the loss of sheathing. This is further improved if it is combined with a reduction in fastener spacing [20]. The incorporation of hurricane straps can reduce the failure and can lower the risks of losing the sheathing even further [20]:

- one hurricane strap – 35 to 96% probability of failure
- two hurricane straps – 0 to 18% probability of failure.

Once the wood deck is installed, the eave protection is installed (Figure 2.8). This assists in keeping the roof free of leaks and ice dams (Figure 2.9) which can form when there is poor ventilation and loss of heat from the attic [2]. The melted snow on top of the roof freezes once it reaches the edge of the roof.

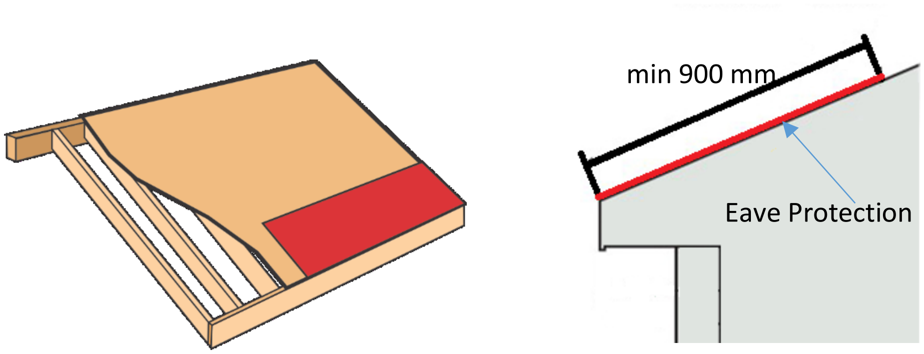


Figure 2. 8 Eave installation on a steep slope asphalt roof on top of the deck

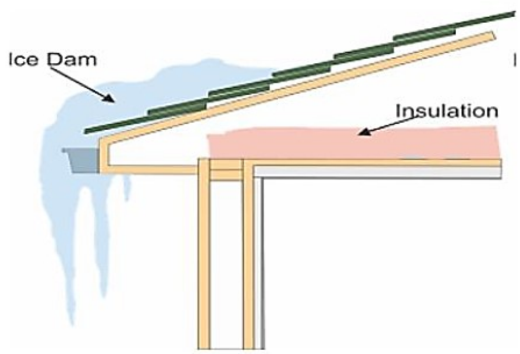


Figure 2. 9 Ice dam formation on the roof [2]

The NBCC references CSA A123.22 "*Self-Adhering Polymer Modified Bituminous Sheet Materials Used as Steep Roofing Underlayment for Ice Dam Protection*" for eave protection. Eave protection is required to cover a minimum distance of 900 mm from the edge of the roof (Figure 2.8) [10]. The materials that can be used as eave protection are listed in section 9.26.5.2 and can be summarized into the following [10]:

- asphalt-saturated felt
- roofing roll
- glass fibre or polyester fibre base sheets
- self-sealing modified bituminous membrane

The requirements of eave protection when is it and is not required are listed in section 9.26.5.1 which can be seen in Figure 2.10. The cases when eave protection is not required are either due to the design or the climate, where the risk of ice dam creation does not exist [10].

9.26.5. Eave Protection for Shingles and Shakes

9.26.5.1. Required Eave Protection

1) Except as provided in Sentence (2), eave protection shall be provided on shingle, shake or tile roofs, extending from the edge of the roof a minimum of 900 mm up the roof slope to a line not less than 300 mm inside the inner face of the exterior wall.

2) Eave protection is not required

a) over unheated garages, carports and porches,

b) where the roof overhang exceeds 900 mm measured along the roof slope from the edge of the roof to the inner face of the exterior wall,

c) on roofs of asphalt shingles installed in accordance with Subsection 9.26.8.,

d) on roofs with slopes of 1 in 1.5 or greater, or

e) in regions with 3 500 or fewer degree-days.

9.26.5.2. Materials

1) Eave protection shall be laid beneath the starter strip and shall consist of

a) No. 15 asphalt-saturated felt laid in two plies lapped 480 mm and cemented together with lap cement,

b) Type M or S roll roofing laid with not less than 100 mm head and end laps cemented together with lap cement,

c) glass fibre or polyester fibre coated base sheets, or

d) self-sealing composite membranes consisting of modified bituminous coated material.

Figure 2. 10 Extract from NBCC 2015- Part 9 (Roof Sheathing Fastener requirements) [10]

The underlayment is stated as optional in the NBCC but it can be installed on top of the wood deck (Figure 2.11). It aids in protecting the roof from water infiltrations especially in the case when the shingles might be blown away. NBCC references the following standards for underlayment:

- CSA A123.22 "*Self-Adhering Polymer Modified Bituminous Sheet Materials Used as Steep Roofing Underlayment for Ice Dam Protection*"
- CSA A123.3 "*Asphalt Saturated Organic Roofing Felt*".

Underlayment is considered as a secondary protection against water infiltration in case of shingle blow-off [21]. The Federal Emergency Management Agency (FEMA) suggests three different underlayment installation options (Figure 2.12). The underlayment can be self-adhered or fastened using capped-head nails, as shown in Option 1 and Option 2, respectively. As an added protection FEMA also has the sealing of the deck joints in Option 3 to be achieved before the underlayment installation. This would prevent water infiltration in case the underlayment, as well as the shingles, were blown off.

When both an underlayment and eave protection are installed on the roof, the underlayment will be installed after the eave protection. The NBCC requires the underlayment to be installed parallel to the eave protection with a minimum overlap of 100 mm. The underlayment can be asphalt-saturated sheathing paper with a minimum weight of 0.195 kg/m^2 or No. 15 plain or perforated asphalt-saturated felt which are outlined in more detail in section 9.26.6.1 of the NBCC [10].

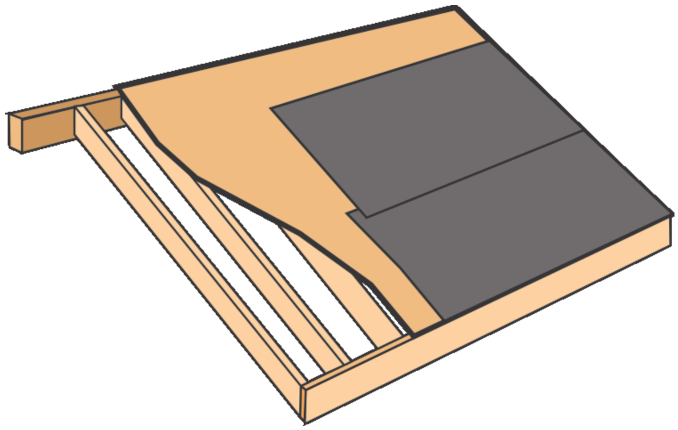


Figure 2. 11 Underlayment Installation on top of the roof deck

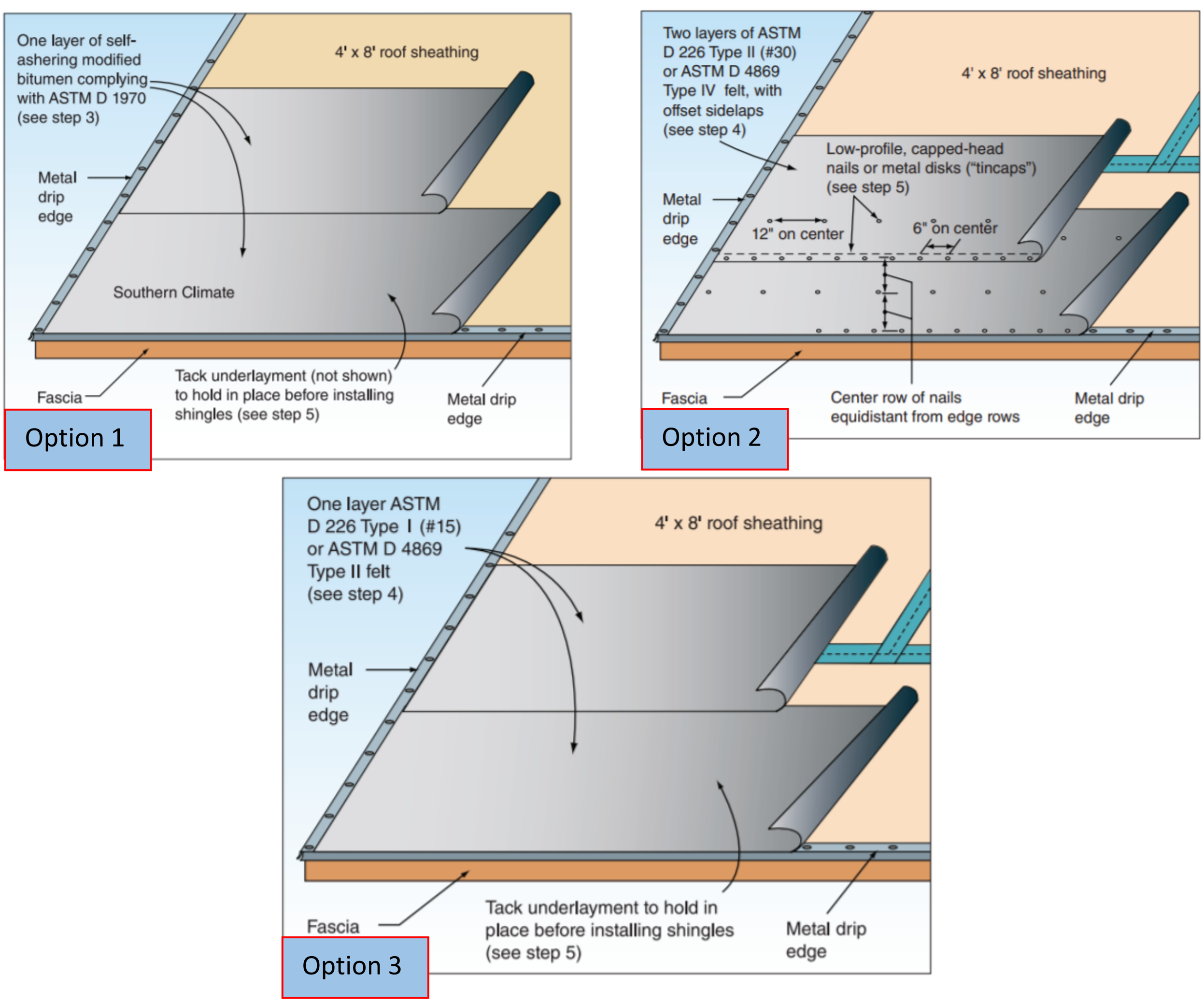


Figure 2. 12 The suggested underlayment options from FEMA [21]

The last component to be installed are the shingles. There are different types of shingles, wooden shingles, shakes, slates, tiles, and solar shingles. However, asphalt shingles are one of the most popular types of roofing cover used in residential steep slope roofing in the last 100 years [22]. This is mostly an attribute to their economic price and their variety of designs.

The origin of shingles goes back to 1840, during which coal tar was used in combination with felt and sand, however, coal tar became expensive and was replaced with asphalt [23]. The shingle product has been continuously modified and changed, to increase its wind uplift resistance, tear resistance, improve its life span, and these improvements among others continue to this day. Shingles are composite materials, whose composition can be summarized into the following: asphalt matrix, reinforced mat, and filler [24] (Figure 2.13)

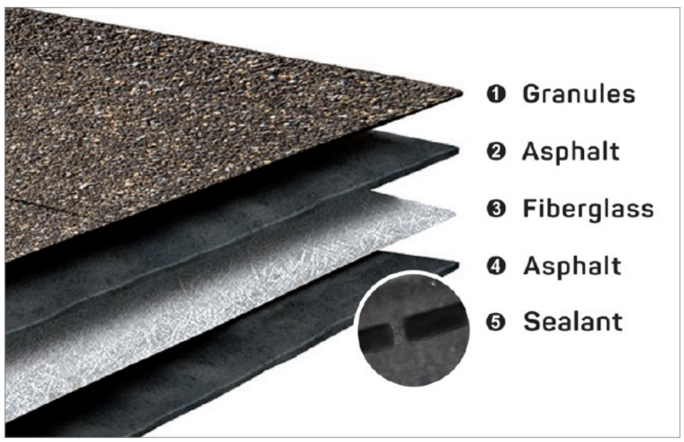


Figure 2. 13 The different layers composing the shingle [24]

Each of these layers serves a specific purpose in aiding the performance of the shingle and ensuring it provides protection. Noone et al. summarize the role of each layer [22]:

- The body of the shingle is composed of asphalt which enables the waterproofing quality of shingles and determines their resistance to weathering.
- The fiberglass mat is the reinforcement of the shingles and determines to a high degree the physical properties of the shingles.
- The granules protect the asphalt from the sun’s ultraviolet radiation which can expedite the thermal oxidation in shingles leading to faster degradation and loss of pliability.
- The sealant on the self-sealing strip located underneath the shingles is activated from heat and allows the shingles to be adhered to each other.

All the layers need to meet a certain performance quality for a successful shingle. The shingles are layered on top of each other as shown in Figure 2.14.

Figure 2. 14 Shingle Installation for mock-up construction at the University of Ottawa

Today the most common types of shingles are architectural shingles and 3-tab shingles [25] (Figure 2.15). Architectural shingles are composed of laminated layers, while 3-tab shingles are composed of a single layer.

Figure 2. 15 Types of Asphalt Shingles [25]

Shingle installation begins at the bottom of the edge of the roof and shingles are placed in an overlapping manner such that the seam or joint is overlapped by the upper layer of shingles. This placement allows the shedding of water, snow, and ice to be directed to the eaves and then gutters and ensures there is no leaking into the house. Shingles are installed on the roof deck using fasteners in accordance with manufacturers' installation guidelines along the fastener line. On the back of each shingle, there is a sealant strip that becomes activated due to the heat from the sun. This allows the bond between the shingle layers to develop and it transforms the individual shingles into a united system.

Asphalt shingle requirements in the NBCC (9.26.7 and 9.26.8) are classified into two groups depending on the slope of the roof and can be seen summarized in Table 2.1 [10]:

- slope greater than 1 in 3
- slope less than 1 in 3

Table 2. 1 Asphalt Shingle Requirement Summary

Slope	Equal or Greater than 1 in 3	Less than 1 in 3
Coverage	❖ Minimum 2 thickness of shingle over the entire roof	❖ Minimum of 3 thicknesses of shingle over entire roof except for the first two courses.
Starter strip	❖ Extend 12 mm beyond the eaves and rake. Minimum fastener spacing of 300 mm along the bottom edge. ❖ Can be: • minimum 300 mm wide type M mineral-surfaced roll • shingles of the same weight and quality as those used with tabs facing up the roof slope • pre-manufactures starter strip	❖ Minimum width of 200 mm. ❖ Applied on a continuous band of cement. ❖ Extend 12 mm beyond the eaves and rake. Minimum fastener spacing of 300mm along the bottom edge. ❖ Can be: • minimum 300 mm wide type M mineral-surfaced roll • shingles of the same weight and quality as those used with tabs facing up the roof slope • pre-manufactures starter strip
Securing of Tabs	❖ Minimum of 25 mm diameter of plastic cement under the center of each tab or ❖ Interlocking devices or ❖ Self-sealing strips	❖ Cold application cement (Minimum rate 0.5 L/m^2) ❖ Hot application asphalt (rate 1 kg/m^2)
Securing of Shingle Course		❖ First course secured on a continuous band of cement
Head Lap	❖ Minimum 50 mm	
Fasteners	❖ Minimum of 4 nails or staples for 1 m wide shingles (no exposed fasteners) ❖ Minimum of 6 staples if 11mm crown staples are used ❖ Located 20 mm to 40 mm from each end of the strip shingle ❖ Located a minimum of 12 mm above the tops of cut-outs	
Hips and Ridges	❖ Shingles extend a minimum of 100 mm on either side of the hip or ridge ❖ Shingles lapped a minimum of 150 mm	❖ Shingles minimum of 300 mm wide for triple coverage ❖ Cemented to roof shingles and each other with cement ❖ Fastened with nails or staples (40 mm above the butt of the overlying shingle and 50 mm from each edge)

NBCC references CSA A123.5 "*Asphalt shingles made from glass felt and surfaced with mineral granules*" which provides requirements the shingles must meet, such as the physical requirements, dimensions, and weight. The full requirements listed in CSA A123.5 can be seen in Figure 2.16. All the component evaluations and wind uplift system tests do not account for Canadian climate requirements. The evaluations listed below are only determined at laboratory conditions with the exception of cold curl resistance, pliability, behaviour upon heating, and fastener pull-through.

Table 1
Physical requirements for asphalt shingles
(See Clauses 5 and 8.1–8.10.)

Requirement	Max	Min
Behaviour upon heating		
Loss of volatile matter, %	1.5	—
Sliding of granular surfacing, mm (in)	1.6 (1/16)	—
Tear strength (cross-machine direction), g (lb)	—	1600 (3.5)
Tensile strength		
Machine direction, kN/m (lb/in)	—	10.5 (60)
Cross-machine direction, kN/m (lb/in)	—	7.0 (40)
Wind resistance		
ASTM D3161, Class A, min [97 km/h (60 mph)]	—	Pass
Pliability at 0 °C (32 °F)		
Machine direction	—	4 of 5 pass
Cross-machine direction	—	4 of 5 pass
Granule loss, g	1.0	—
Fastener pull-through resistance at 0 °C (32 °F), N (lbf)	—	124 (28)*
Coating weatherability, cycles	—	60
Coating stain tendency, index	20	—
Cold curl resistance at –18 °C (0 °F), mm (in)	7 (1/4)	—

* *For laminated shingles, when tested in the double-ply area, the minimum fastener pull-through resistance shall be* 186 N (42 lbf).

Note: *ASTM D3161 is used for the determination of wind resistance of asphalt shingles using a fan induced method.*

Figure 2. 16 Extract from CSA A123.5 [26]

Alongside the shingle installation, accessories such as vents and flashing (Figure 2.17) are also installed. The flashing prevents water infiltration and assists in shedding water from the roof. It can be found [24]:

- in areas where a valley is formed by the roof structure
- wherever there are penetrations in the roof
- all around the perimeter of the roof in the form of a drip edge.

Figure 2. 17 Accessories (vents and flashing) [24]

The materials that can be used for flashing are listed in the 2015 NBCC (9.26.4). Ventilation is very important in minimizing problems the roof can have due to attic overheating, moisture and condensation built-up as well as the formation of ice dams. Vents according to the NBCC are required to provide transfer of moisture and their minimum area is 1/300 of the insulated ceiling area [10].

2.4 Evaluation Protocols for Steep Slope Residential Roof Cladding

The resistance of the roof must be equal to or larger than the design load. This resistance of any steep slope residential roof can be determined either as an entire cladding system or as individual components composing the system. Evaluation protocols for these two approaches are reviewed and compared below.

2.4.1 Component Resistance Evaluation

By taking asphalt shingles as components, the standards compared in this section concentrate on the tear, tensile, and fastener pull-through strength. By quantifying these properties one can correlate them with the wind resistance of the shingles [27] . By reviewing the different standards available it can be noticed that there is a lack of information regarding the evaluation of the overlap sealing strength of the shingles and the determination of the properties at various temperatures and conditions. With the exception of four tests (pliability, cold curl resistance, behaviour upon heating, and fastener pull-through) all the other tests are performed at laboratory conditions. However, it is important for the shingles to be conditioned and tested under a range of different conditions to simulate the different climatic conditions they will encounter during their service life.

The overlap strength of the shingle is not currently required to be evaluated by the NBCC. This property needs to be evaluated because unsealing of the shingles is witnessed in relatively new roofing systems. Poor sealing lowers the integrity of the system, hence increasing the risk of failure. All of the roof properties listed in Table 2.2 do not require to be tested after being weathered or aged. The roof's weathering and aging lead to the degradation of the components and thus lowered performance [22]. How the shingle will behave once it has been aged needs to meet certain requirements. This knowledge would allow a better understanding of the performance of the shingles.

Another property that is missing from the evaluation criteria is testing the peel strength of the eave protection when it has adhered to the roof deck. Throughout its service life, the roof will be exposed to various moisture conditions and that effect on the adhesion properties needs to be evaluated. The fastener pull-through resistance of the underlayment needs to be evaluated to understand the protection it can offer in the case of when shingles fail.

Table 2. 2 Highlights from the review of the existing Component Evaluation Protocols

Property	Standard	Test Specimen	Test Condition	Remarks
Tear strength, Fastener pull-through	**ASTM D228** *Standard test methods for sampling, testing and analysis of asphalt roll roofing, cap sheets and shingles used in roofing and waterproofing*	**Tear**: 10 specimens 76 mm by 63 mm ± 3% **Fastener pull-through**: 10 specimens 98 mm by 98 mm ± 3 mm	Room temperature 23 ± 2 °C for 2 hours before testing.	**Tear**: Referenced in CSA A123.5. **Fastener pull-through:** Referenced in CSA A123.5 with the variation of 0°C.
Tensile Strength	**ASTM D146** *Standard Test Methods for Sampling and Testing Bitumen-Saturated Felts and Woven Fabrics for Roofing and Waterproofing*	10 specimens 25 mm by 150 mm	Room temperature 23 ± 2 °C for 2 hours before testing.	Referenced in CSA A123.5.
Uplift Resistance	**ASTM D6381** *Test method for measurement of asphalt shingle mechanical uplift resistance*	**Procedure A:** Bottom piece 178 mm x 95 mm ± 3 mm, Top piece: 95 x 114 mm ± 3 mm **Procedure B:** Bottom piece 102 mm x 152 mm ± 3 mm, Top piece: 38 mm x 95 mm ± 3 mm	Condition at selected temperature to replicate field conditions , ±1.5°C for at least 1 hr.	Conditioning specimens at selected temperatures to allow the sealing of the shingles.

2.4.2 Roofing System Resistance Evaluation

Test protocols available for wind resistance of residential roofs as a system are summarized in Table 2.3. TAS 100 subjects the test mock-up to eight different time intervals with different velocities ranging from 16 m/s up to 49 m/s as shown in Figure 2.18. Intervals with zero wind speed are maintained for 10 minutes while all the other intervals are maintained for 15 minutes with the exception of the 49 m/s wind speed interval. Failure is classified as any roof covering being blown off or permanently deformed upward without returning to its original position.

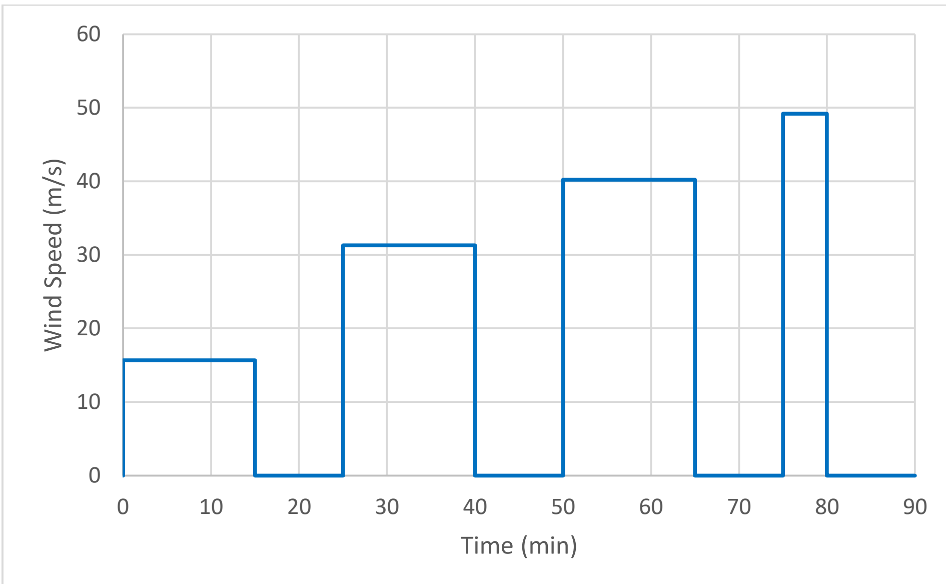


Figure 2. 18 TAS 100 cycle load (adapted from TAS 100 [28])

ASTM D3161 "*Standard test method for wind-resistance of asphalt shingles*" is a fan-induced method that classifies shingles into three classes [29]:

- Class A- Test velocity of 27 m/s (97 km/h),
- Class D- Test velocity of 40 m/s (145 km/h),
- Class F- Test velocity of 49 m/s (177 km/h)

With the above speeds, the test is run for two hours or less if a failure occurs and failure is defined as:

- Disengaging of interlocking feature or product tab
- Detachment of any product from the deck
- Failure of sealant.

ASTM D7158 "*Standard Test Method for Wind Resistance of Sealed Asphalt Shingles (Uplift Force/Uplift Resistance Method)*," classifies shingles into the following three classes [30]:

- Class D- Wind speed up to 52 m/s (187 km/h),
- Class G- Wind speed up to 69 m/s (249 km/h),
- Class H- Wind speed up to 87 m/s (312 km/h).

This test method identifies failure when the calculated uplift force exceeds the measured uplift resistance.

Table 2. 3 Highlights from the review of the existing system evaluation protocols

Property	Standard	Test Specimen	Conditioning	Remarks
Wind Uplift Resistance	**ASTM D3161** *Test method for wind resistance of steep slope roofing products (fan induced method)*	1.27 x 1.68 m	57-60°C for 16 hrs	Fan-induced procedure, delivering a stream of air across the exposed surface of the test specimens through the use of a fan. Referenced in CSA A123.5
Wind Uplift Resistance	**ASTM D7158** *Standard test method for wind uplift resistance of asphalt shingles*	Test deck (D3161): 0.9 x 1.27 m at a slope of 1.6	1)Uplift rigidity test: condition specimen at 23°C for 2 hours 2) Shingle uplift coefficient and mechanical uplift tests: condition specimen at 57-60°C for 16 hrs	Calculates uplift force at a specified velocity, compares it to the mechanical uplift resistance of the shingle. Higher wind speed testing. The use of a sensor allows to determine an accurate failure and not depend only on observations.
Wind and wind-driven rain resistance	**TAS 100 -95** *Test procedure for wind and wind-driven rain resistance of discontinuous roof systems*	Test wood deck that is 2.4 m x 3 m, comprised of APA 32/16 span rated sheathing of 12 mm thickness and	3 continuous days at 57-60°C for six hours each day	Wind generator(s) used. Various test wind speeds ranging from 16 to 49 m/s. Longer conditioning time. Larger test specimen. Incorporation of wind-driven rain.
Wind Uplift Resistance	**TAS 107-95** *Test procedure for wind resistance testing of non-rigid, discontinuous roof system assemblies*	Test wood deck that is not less than 1.27 m by 1.68 m comprised of APA 32/16 span rated sheathing of 12 mm thickness	57-60°C for 16 hrs	The test machine is capable of delivering a horizontal stream of air through a rectangular opening, 305 mm. One wind speed of49 m/s. Different wind direction angle (test minimum 3): - head-on - bottom of the target area parallel to the opening - test specimen rotated 30 and 60 degrees

A similar trend to that of component test protocols is also noticed for the system evaluation; the only conditioning that occurs is to ensure the shingles seal acts as one component. The current system test methods do not account for climatic variability which affects the shingles' behaviour and resistance. The roof system will age once it is exposed to field weathering and various weather conditions. This will affect the way the components of the system interact with one another as well as the overall resistance of the system.

Commercial roofs can be tested using both standardized static and dynamic tests, which are also accepted by the industry. Static tests consist of a pressure chamber, a full-scale specimen, and an applied pressure difference which is constantly increased until the roof fails. Factory Mutual Global 4470 is a static test accepted by the industry and used across North America [31]. An example of the applied static pressure can be seen in Figure 2.19 [31]. It starts at a pressure of 1.4 kPa (30 psf) and is increased by an increment of 0.7 kPa (15 psf). Each pressure step is maintained for 60 seconds.

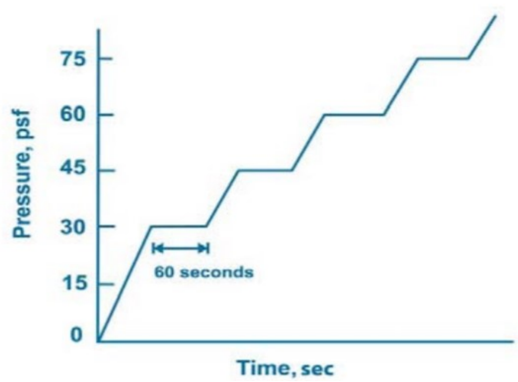


Figure 2. 19 Static load cycle [31]

Dynamic tests are thought to be a better replication of the extreme wind events, where the winds are variable and produce dynamic pressures on the roof. The failure modes from static pressure testing did not match failure modes recorded after extreme wind events [32]. When the failure modes of the static test were compared with those of the dynamic test, two different failure modes were observed [33]:

- fastener pullout from the deck was the main failure mode while using the static test
- for the dynamic test shear failure of the membrane at the fastener was the main failure mode.

An example of a dynamic load cycle using CSA A123.21 is shown in Figure 2.20 [31].

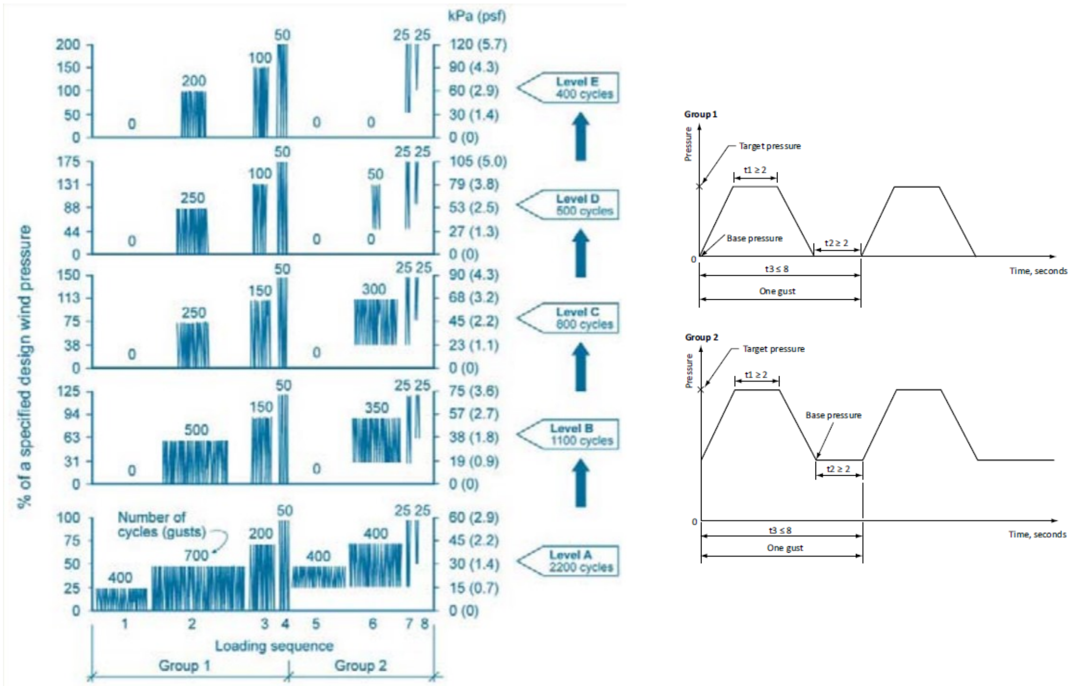


Figure 2. 20 Dynamic load cycle ([31], [34])

Commercial roofs can be tested using standardized dynamic tests which can replicate the dynamic pressures acting on the roof during extreme wind events. However for steep slope asphalt roofs currently a similar standardized test for dynamic testing that is accepted by the industry does not exist.

The University of Florida created a dynamic test protocol for wind uplift testing of roof sheathing, but the results were inconsistent especially when compared to the results from static tests. The inconsistency is believed to be related to the overestimation that occurs when the wood roofs are tested using static pressure [32].

These test methods do not replicate real conditions that the shingles will be exposed to and do not account for wind variability, climate effects, location, and height of the house. All the system tests available are static and they do not account for varying dynamic loads that the roof will experience in the field. The system tests are performed at laboratory conditions. This does not take into account the effect of weather exposure on the shingles and the roof as well as the extra thermal stress that causes the system. The system tests consist of installed shingles directly onto the deck; they do not account for the presence of other elements such as underlayment or eave protection and the possible effect they might have.

2.5 Roof Field Performance

In this section, the performance of steep slope residential roof cladding that was constructed using the methods and components outlined in Section 2.3 is discussed under two different conditions. First under severe weather from various post-storm reports, which highlight the types of damages, age of the roofs, and possible reasons why the failures occurred. Second, the service expectancy of the roof cladding under typical Canadian weather conditions is captured through a homeowners' survey which highlights the age, damages, and general issues homeowners experienced.

2.5.1 Performance under Weather Events

Unanimously insurance claim reports agree that the first element which fails under extreme wind forces is the residential roof. The failures that residential roofs can experience can be broadly classified as [4]:

- Roof structural failure
- Roof covering failure

In the case of a roof structural failure, the entire assembly is blown away, as shown in Figure 2.21. In this figure, the weakest link is the roof-wall junction. In Figure 2.22 the connection between the deck sheathing and the structural frame failed, and the deck was blown away along with the shingles and underlayment. This exposes the house's interior to external climatic conditions and hence increasing the severity of the damages [20].

Figure 2. 21 Example of structural failures of roofs where the roof assembly was blown away [20]

Figure 2. 22 Example of structural failure of the roof where the deck sheathing was blown away [35]

In the case of roof covering failure, the deck sheathing remains attached to the structural frame. However, the roof coverings are blown off from the deck. An example of this type of failure can be seen on a roof located in Ottawa, Ontario [36] where the shingles were blown off during a sudden wind event in the summer of 2020 (Figure 2.23). In this scenario, the interior is still protected from water ingress by the underlayment, if it is installed underneath the shingles. Shingle failure analyses highlights the attachment of the shingles as the weakest link.

Figure 2. 23 Example of shingles blow off after a wind event in Ottawa 2020, [36]

However, there are cases where the underlayment was also blown away along with the shingles as shown in Figure 2.24. The sheathing was exposed to weather elements, increasing the risk of water infiltration and damage and mold growth.

Figure 2. 24 Shingle blow off with peeled underlayment after a wind event in Ottawa 2020 [36] (a) and Angus [4] (b)

The Degree of Damage (DOD) of these failures is classified depending on the percent loss. For example, if the roof structural and covering failure described above occurred during a tornado and more than 20% of the roof covering material and the roof deck were lost, it would be classified under category 4 DOD. A category 4 DOD corresponds to tornadoes with wind speeds ranging from 130 to 185 km/h (81 to 115 mph) using a standardized method known as the Enhanced Fujita (EF) scale. This scale is used to record the damaged area, types of failures and estimate tornado wind speeds, thus, this estimation also accounts for construction variability by providing a wind speed range instead of a single wind speed value [20]. If a structural failure occurred during a tornado and more than 50% of the roof was blown off, this would be classified as category 6 DOD. Category 6 corresponds to wind speeds ranging from 165 to 230 km/h (103 to 143 mph). Similarly, the Saffir-Simpson scale is also used to record the severity of hurricanes and their damages, with roof damage ranging from minimal loss to loss of the entire roof [37].

Damages observed after tornadoes and hurricanes display similar failure patterns and damages [20]. Tremendous damages can be caused when structures are not adequately designed to withstand wind forces, and an increase of 25% in wind speed directly impacts the risk, causing a risk increase of 650% [4]. Over a period of 33 years, wind was identified by the Insurance Bureau of Canada to be responsible for 62% of catastrophic events that occurred in that time frame [4]. This upwards trend in disasters is also observed in recent years with the tornado occurrences in Goderich (Figure 2.25) and in Dunrobin (Figure 2.26) with $130 million and $295 million in damages, respectively [38].

Figure 2. 25 Tornado damages in Goderich, ON in August 2011 were determined to be $130 million [38]

Figure 2. 26 Tornado damages in Dunrobin on September 2018 were determined to be $295 million [39]

The range of damages that residential roofs endure after extreme weather events can vary greatly; the damages sustained are recorded through forensic surveys. The information provided from these post-storm surveys is necessary for establishing a detailed database of various damages resulting from wind events with different degrees of severity, such as the “Northern Tornadoes Project” [40] . Such databases can help identify weak links in the current construction methods and assist in creating a baseline for comparison for when the witnessed failures are attempted to be replicated both in the lab and numerically. The database allows a deeper understanding of the problem, leading to applicable solutions that help mitigate failures. The post-storm database is also useful in understanding how the nature of damages has changed over the years and serves as a feedback mechanism to see the effectiveness of implemented mitigation measures.

Wind-related damages in recent years have led the Institute of Catastrophic Loss Reduction (ICLR) to propose the need for immediate measures for the protection of roofs, walls, and upper and lower story connections, to mitigate damages from extreme weather [39]. This call for immediate measures is also based on ICLR's previous analysis of post severe wind events. It concluded in 2018, and identified roof structures and the roof-wall connection as those having a higher probability of being damaged [1]. Roof coverings, such as asphalt shingles, are also prone to damages during hurricanes or other severe wind events, like displacement or complete removal [41] or even fasteners being pulled out [42]. Damages due to strong hurricane winds in 1965 caused entire roofs to be lifted off the buildings, damaged corners, and ridges of roofs, as well as roof coverings [43].

As can be seen, despite the year when post severe wind events observations were made, the pattern of observed failures is consistent. This leads to the question: Has nothing improved in terms of design and construction for the last 56 years, have lessons not been learned from the history, or is it the climate change effect causing the structures to experience new higher loads compared to the loads they have been designed for?

2.5.2 Roof Performance under Typical Conditions (Homeowners' Experience)

To understand the issues homeowners were encountering with their roofs during their service life under typical Canadian weather conditions, in 2018 the National Research Council of Canada launched a survey to all of its 3,000 employees across Canada. In this section, only the questions directly related to this thesis are discussed. However, all of the survey questions and answers from over 1,200 employees can be found in Appendix A. The survey revealed that the majority of the participating homeowners (90%) had a steep slope roof (Figure 2.27) and about 92% of these roofs had asphalt shingles as a roof cover (Figure 2.28).

Figure 2. 27 Survey breakdown of the types of roof slopes for residential homes [2]

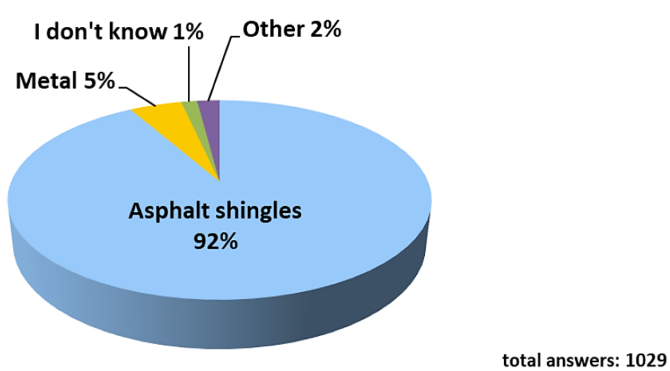


Figure 2. 28 Survey breakdown of the roof coverings most steep slope residential roofs [2]

Homeowners that had a roof that was less than 15 years in service was 79%, and this number decreased significantly for homeowners with a roof that was older than 15 years, which can be seen in Figure 2.29 [2].

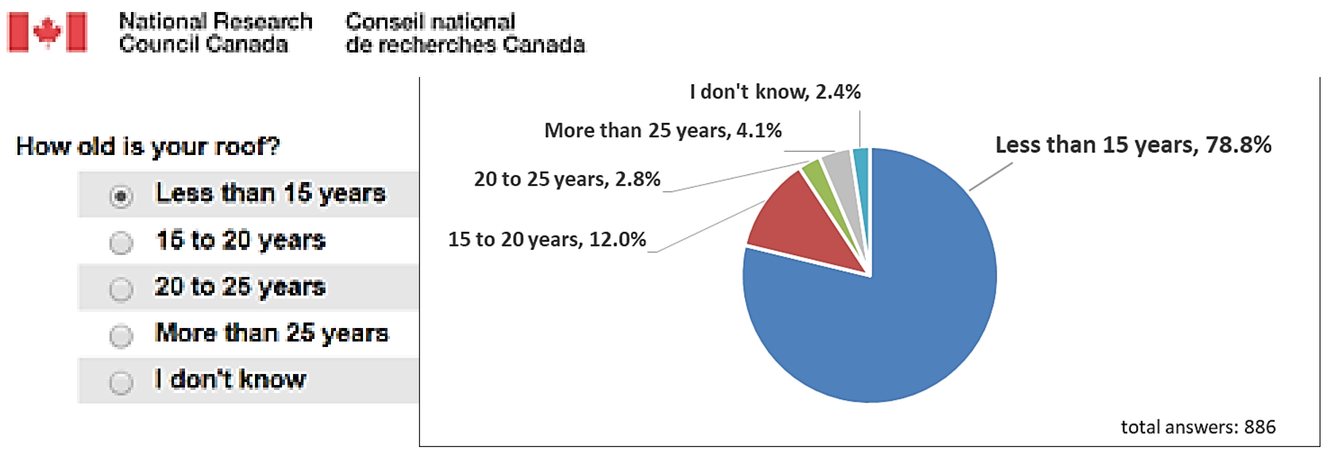


Figure 2. 29 Survey breakdown of the age of the steep slope residential roof ages [2]

According to the survey, the most common issues homeowners were facing, were curling, clawing, ice damming, water infiltration, granule loss and cracking, shingle blow-off, and shrinking (Figure 2.30). Damages such as curling, granule loss but also buckling, distortion, and colour change are some of the main reasons homeowners replace their roofs [44]. These damages can lead to covering failure during a severe wind event because they affect the integrity of the shingle.

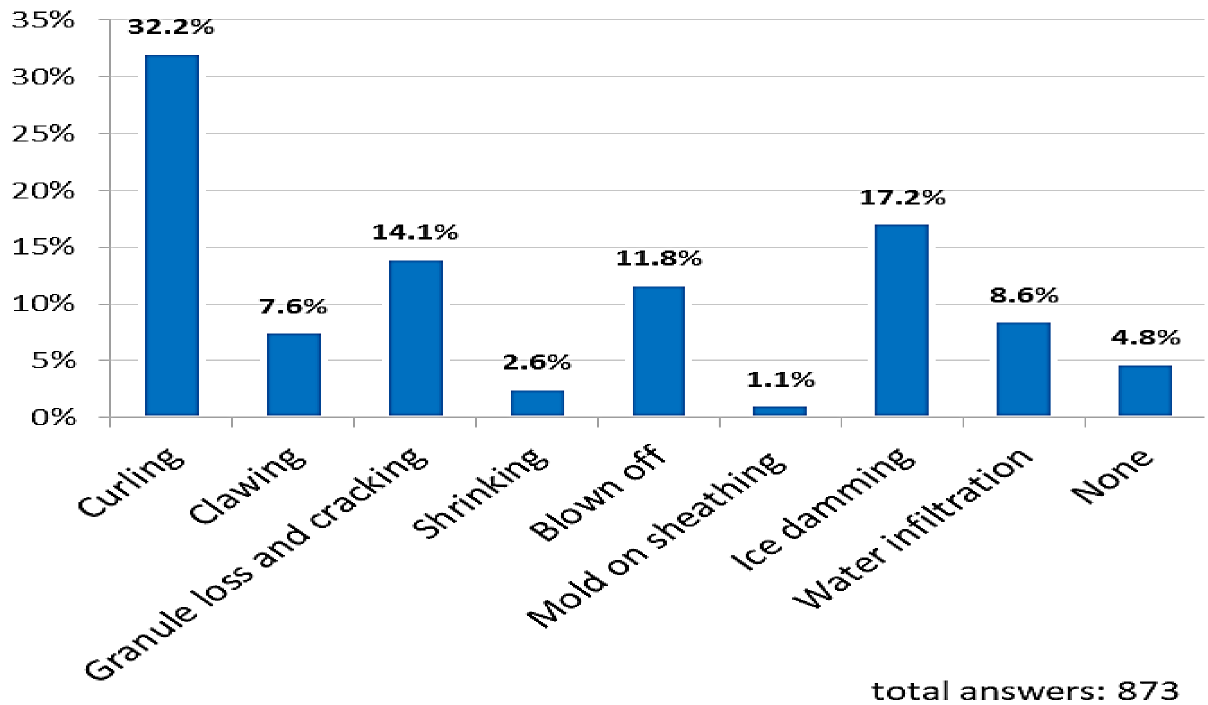


Figure 2. 30 Survey breakdown of the main issues affecting shingles [2]

These issues are occurring on roofs with shingles that are less than 15 years in service despite most manufacture advertising their products as having a 20-year, 30-year, 40-year, 50-year, or lifetime warranty. This missing link between the warranty of the shingle vs its actual lifespan is one of the major comments received from homeowners. Their discontent of needing to replace their roofs before reaching its advertised warranty needs further security and addressment [2].

Manufacturers provide a wide range of warranties for residential roofs, such as a 20-year, 30-year, 40-year, 50-year, or lifetime warranty when shingles are installed on homes. This often is misunderstood by the common Canadian homeowner as the lifespan of the installed shingle roof. In fact, the warranty offered by the manufacturers covers only manufacturing defects without considering the actual lifespan of the shingle or installation quality [44]. For a roof warranty to be valid, the roof needs to be installed in full adherence to the manufacturer's installation guidelines and properly maintained. Also, the warranty only covers the cost of the new materials and not the installation cost unless otherwise specified. It should be noted that the labour cost can be four times higher than the material cost. In some cases, the warranty of the shingle might increase from a 30-year warranty to a 50-year warranty with no actual change to the product being sold [44]. However, apart from the warranty provided by the manufacture, there is also a warranty provided by some roofing contractors or associations. This is called a roof guarantee, and it provides a warranty on the installation and materials, ensuring the coverage of any problems the roof may encounter [45] [46]. Irrespective of the manufacturer's warranty or association's guaranty, most roofs have a lifespan that does not exceed 12 to 18 years due to several issues originating either from defective materials, incorrect installation, or inadequate ventilation, or improper maintenance [44].

The large discrepancies present in the warranties offered by manufacturers and the service life of shingles can be very misleading and prevent homeowners from making informed decisions in their roof covering selection. Selecting a shingle purely based on its advertised warranty is not sufficient to ensure a quality resilient roof. This is because the performance of the roof is equally dependent on the installation quality and maintenance.

2.6 Concluding Remarks

This chapter established a baseline of the current practices followed in the design and construction of steep slope residential roof cladding. The majority of roofs of residential houses are classified under Part 9 “Housing and Small Buildings” and do not account for variations due to the location and climate effects. Thus most of the houses follow prescriptive designs. Therefore it is necessary to incorporate the effect of future climatic projections into the design of residential roofs to increase their resiliency against increasing global temperatures, precipitation, flooding, and extreme winds.

The common roof construction steps were outlined along with the requirements stated in the NBCC for each component. The resistance of a roof can be determined either as a system or at a component level. The main protocols used for component and system evaluations are outlined. The damages observed from post-storm surveys are consistent through the years and studies are under development to mitigate these damages. However, there are still a lot of missing pieces and a Canadian standard or test protocol does not exist for system testing of residential roofs accounting for Canadian weather conditions. The majority of both component and system testing do not take into account the effect of different climatic conditions and the evaluation is performed mainly on unaged specimens.

The current chapter concludes by exploring the field performance of the roof cladding under both severe and typical weather conditions. These highlight the range of damages that the roof cladding can experience leading to the conclusion that there are missing elements that are lowering the roof cladding performance. One might intuitively think that a new roof will perform better during severe weather. Older roofs are anticipated to encounter more damages due to:

- the roof having been exposed to several different climatic and load cycles;
- the materials have naturally aged and degraded with time, and their durability can be lower [47].

These observations were formulated after the hurricane seasons in 2004 and in 2008, which showed that roofs constructed six to seventeen years ago had sustained more damages. However, that is not always the case. During the 2014 Angus tornado, it was reported that many of the houses and roofs which were damaged, were constructed and had a service life of less than three years [4]. These failures could have been facilitated by a combination of the following causes:

- inaccurate construction and installation;
- incompatible materials and site conditions;
- higher applied loads on the roofs than the designed load, due to the effect of climate change not being taken into consideration;
- Inadequate sealing among the overlaps of shingles leading them to not behave as a system to assist in the load resistance;
- faulty materials.

Roof design is complex, and many different variables can influence its overall performance. An obvious missing element is quality assurance. Therefore, a comprehensive approach is needed to address these issues by looking at the problem from many angles as opposed to only one. This aspect is discussed in more detail in Chapter 3.

Chapter 3: Development of the Holistic Approach

3.1 Introduction

Expecting a roofing system to perform for its designed load is less realistic because of all the variables that need to be taken into consideration during its construction and service life. The design needs to be accurate and the selected materials need to provide sufficient resistance that should be higher than the design loads. The construction and installation must be performed with a required degree of quality assurance to truly ensure the roof will perform as intended. Construction techniques have changed over the years to adapt to new and emerging technologies favouring a reduction in construction time, increased quality, and the development of new materials. However, there are missing elements that leave newly constructed houses susceptible to damage. As part of the thesis contribution, this Chapter proposes an approach to address these missing links to improve the resiliency of steep slope residential roof coverings. The proposed approach has three folds:

(1) developing the use of future predicted climatic data (3.2);

(2) introducing the resistance chain for the determination of the weakest link (3.3);

(3) implementing a Holistic Approach (3.4).

This Chapter begins by discussing how the reference wind pressure can be affected by climate change. Then using the projected data provided by Environment and Climate Change Canada, a severity classification for the reference wind pressure is developed, via a web tool “Climate-RCI” for ease of use to designers. This is followed by an applied example of the design values for steep slope residential roof cladding based on the procedure outlined in Chapter 2, for three different cities across Canada. This example compares the wind pressure values obtained using the current and future reference wind pressures.

This Chapter continues by introducing the resistance chain for the steep slope residential asphalt roofs, identifying the missing links, and providing how they can be strengthened.

To conclude, the determination of the Holistic Approach is explained after an overview of the Allowable Stress Design (ASD) and Load Resistance Factor Design (LRFD). With the LRFD introduction, how this has been adapted for residential roofs is discussed by expanding recent commercial roofs application.

3.2 Load Accounting for Future Climate Projections

3.2.1 Introduction

The National Building Code of Canada is aiming to incorporate climate change mitigation measures and to increase resiliency due to increasing global temperatures, precipitation, flooding, and extreme winds ([48], [49], and [50]). This inclusion will allow new buildings that are designed and constructed to withstand future climatic loads during their design life and to ensure they would be durable and perform satisfactorily.

As can be seen, the climatic loads that populate Table C-2 of the NBCC, which is partially reproduced in Figure 2.2, are used in the current design practice and are all based on historical data. This assumes that the climate has not changed and is static and stationary. In fact, the climate has been changing for the past 11,700 years [51]. Due to anthropologic activity [52] this change has occurred at higher levels. According to Environment and Climate Change Canada (ECCC), Canada will continue to grow warmer under various greenhouse gas emission scenarios, which are predicted from climate models [53]. Both the highest (representative concentration pathway (RCP) 8.5) and lowest (RCP2.6) emission scenarios that Canada is expected to experience, can be noticed in Figure 3.1. The X-axis in Figure 3.1 shows the time scale from the year 1900 to 2100, as the climate models require historical data to create a historical baseline before simulating and predicting the future climate. The Y-axis shows the temperature change for the previous years and the temperature increase expected annually for Canada. The lowest emission scenario in the year 2100 corresponds to an increase of 2.5°C in the annual temperature, while the highest corresponds to an increase of 7.7°C.

This pattern, of a rapid increase in the temperature starting in the late 19th century, can be seen in Figure 3.1. Before the late 19th century a very gradual increase in temperature can be seen but after that, the increase is much steeper. It continues to increase in the future under both different climate change scenarios represented by the red and blue lines. The increase in temperature (air and sea) due to an increase in the atmospheric greenhouse gases has resulted in a decrease in snow and ice coverings and an increase in seawater levels. The fast rates at which these occurrences are happening cannot be explained by natural causes alone [53]. Despite using climatic data based on historical observations, the 2015 NBCC code does highlight that the climate is changing and the intensity and frequency of extreme weather will be a constant in the future [10].

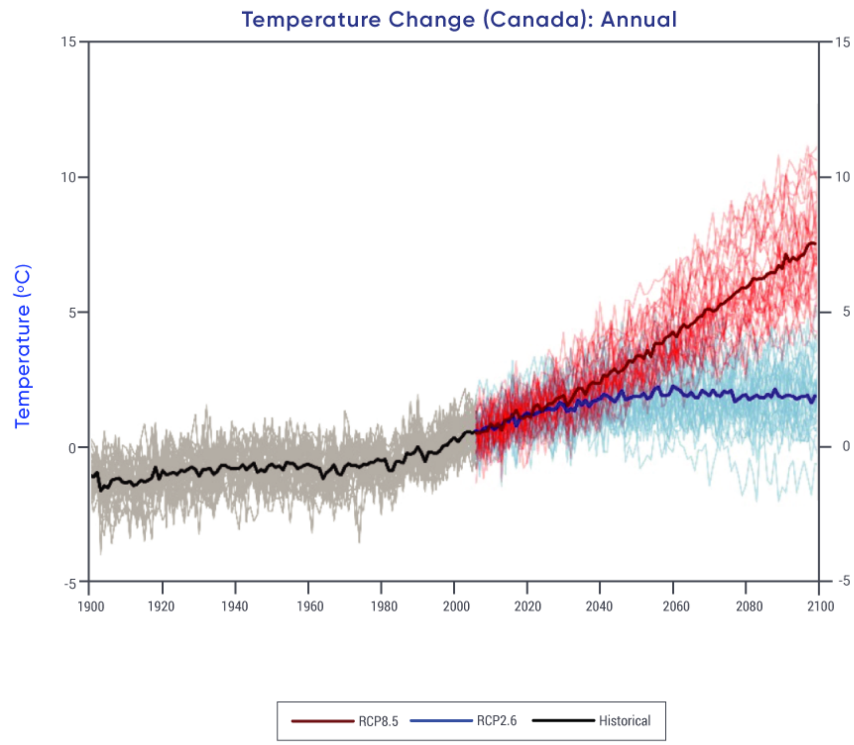


Figure 3.1 Effect of greenhouse gas emissions on annual temperature change in Canada [53]

3.2.2 Impact of Global Warming Scenarios on Wind Data

Environment and Climate Change Canada (ECCC) proposes several different global warming scenarios to occur over the next 70 years. These scenarios take into consideration various greenhouse gas emissions. The model currently being used by the ECCC has an increase in the global warming temperature varying from 0.5°C to 3.5°C. The specific time period of the global warming temperature increases can be seen in Table 3.1 [54]. Based on the climate model, each global warming magnitude is anticipated to last for 31 years. The midpoint of each of these time periods is expressed in parentheses.

Table 3. 1 Global warming magnitude and corresponding time-period [54]

Global warming magnitude	Time-period (center year of the 31-year time-period)
0.5°C	2001-2031 (2016)
1.0°C	2013-2043 (2028)
1.5°C	2024-2054 (2039)
2.0°C	2034-2064 (2049)
2.5°C	2044-2074 (2059)
3.0°C	2053-2083 (2068)
3.5°C	2062-2092 (2077)

On a Canada-wide scale, the percent change in the wind pressures throughout Canada can be seen in Figure 3.2 and Figure 3.3 for a global warming magnitude of 0.5°C and 3.5°C, respectively [54]. A significant trend or extreme changes in the wind load are not observed for Canada and have also not been witnessed in European countries such as Germany or Italy [55].

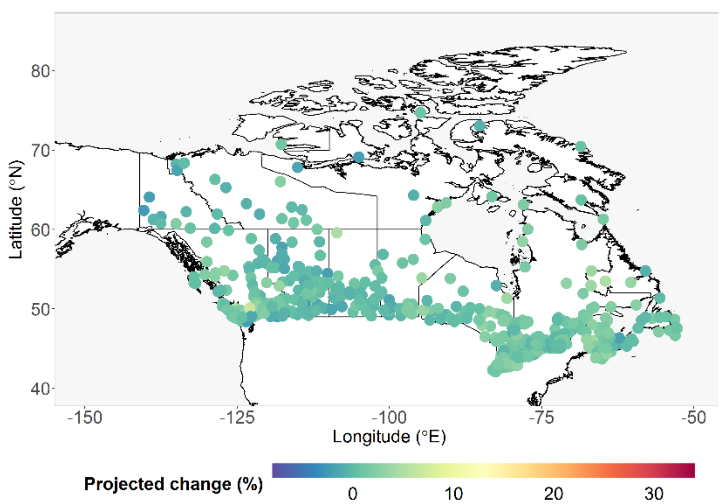


Figure 3.2 Percent change increase of the reference wind pressure under 0.5°C global warming magnitude [54]

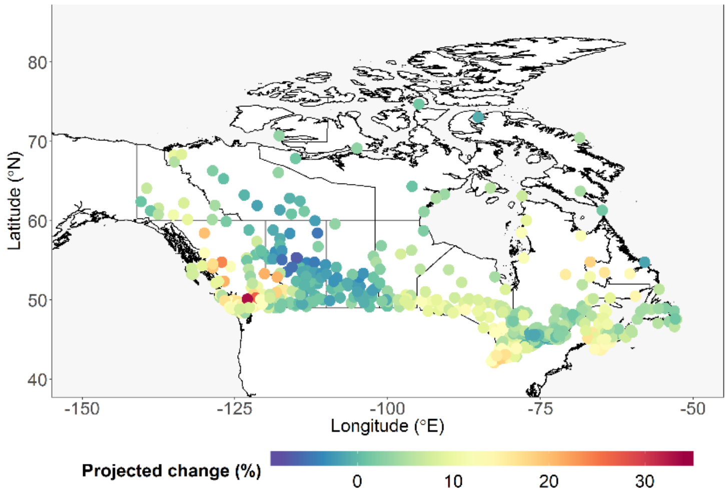


Figure 3.3 Percent change increase of the reference wind pressure under 3.5℃ global warming magnitude [54]

The wind data for the average of 1 in 50 hourly wind pressure across different global warming magnitudes for three different cities across Canada is shown in Figure 3.4.

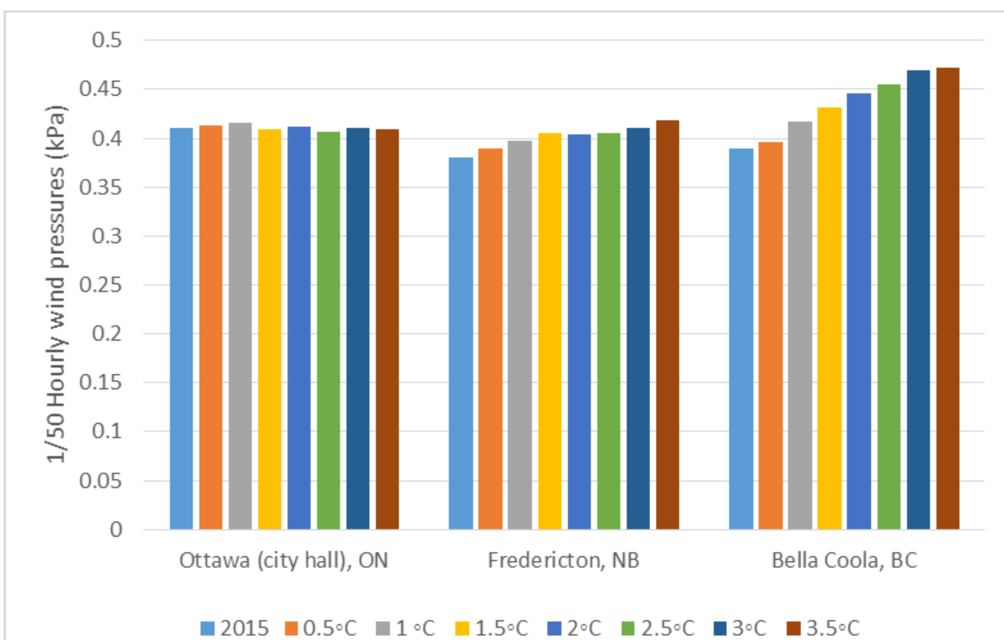


Figure 3.4 Hourly wind pressure comparison for various global warming magnitudes (adapted from Gaur et al. [54])

Since the climate is still changing, continuing to use stationary climatic data would lead to the inability to properly design roof structures. Therefore, it is necessary to incorporate the use of reliable climate models in the design process to determine current and future loads on roof structures [55]. Croce et al. [55] investigated the relationship between the reliability index of a structure and its probability of failure when projected climatic data was used compared to stationary data. It was concluded that when projected data was used the reliability of the structure was higher, while the probability of failure was lower. This leads to designing more resilient structures that will be able to withstand the intensity and frequency of extreme weather that is one of the consequences of climate change.

3.2.3 Reference Wind Pressure Change Severity Classification

Knowing the current reference wind pressure and the predicted values under different degrees of global warming magnitudes allows for the design of structures with higher reliability and resiliency. To provide guidance for the engineers when using future data and to allow for uniformity, from the global warming magnitudes provided in Table 3. 1, 2°C is selected to be utilized for roof design. This increase of temperature is expected to occur over the next 30 years. This period corresponds to the design life of roofing systems, which varies from 15 to 20 years. Furthermore, not all locations will be experiencing the same degree of changes, and to account for that a wind speed change severity classification was determined. The classification accounts for the increased percentage all the locations in Canada were experiencing, for different global warming temperatures, then the wind speed change severity was classified into three ranges. These ranges "Normal", "Severe", and "Extreme" can be seen in Table 3. 2 [54].

Table 3. 2 Severity Classification Ranges for Wind [54]

Range	Wind
Normal	Change < 5%
Severe	5% ≤ Change < 10%
Extreme	Change ≥ 10%

For 2°C global warming magnitude, the percent of locations in each classification is shown in Figure 3.5. It can be noticed that 69% of the investigated locations will be in a normal range of wind speed change and only 29% and 2% of the locations will be in the extreme and severe ranges, respectively. The percent increase of up to 20% change can be noticed only for the south of British Columbia in Figure 3.5 6, which presents the mapping of the percent changes for all of Canada, while milder changes are noticed in the central and west parts of Canada.

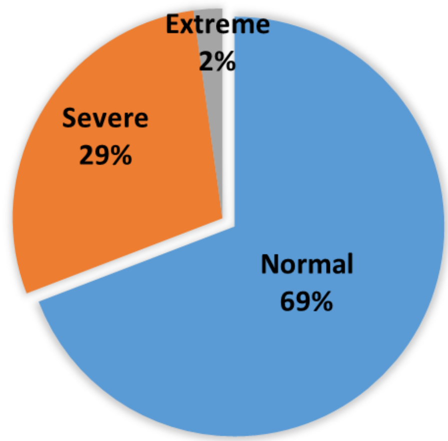


Figure 3.5 Percent of locations in each climate severity classification for 2°C global warming magnitude

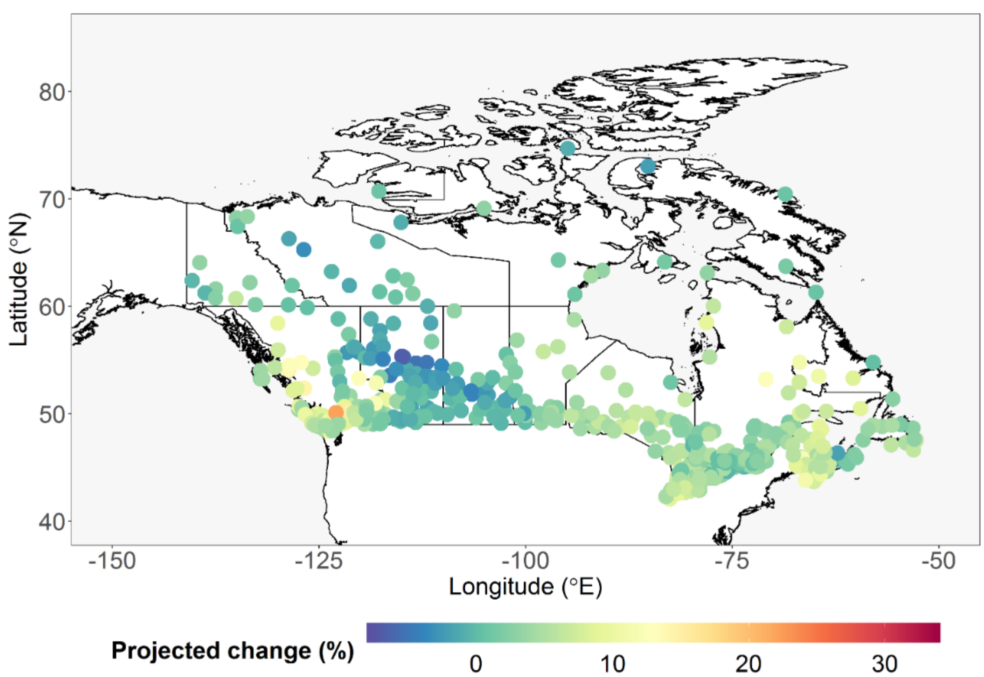


Figure 3.6 Percent wind increase under 2°C global warming magnitude [54]

Bella Coola, Ottawa, and Fredericton, representing three different severity categories across Canada were selected. To illustrate the design impact, wind uplift pressures on the cladding, using the current NBCC design procedure are calculated. The calculations are based on a residential house with a height of 14 m, width of 10 m, and length of 16 m. The house has a hipped roof with a slope of 18 degrees while the terrain around the building is rough. There are no hills or escarpments, and thus the topographic factor (C_t) is equal to 1. The importance level of the building was considered as normal, while the building openings are assumed to be "*Non-uniformly distributed openings of which none is significant or significant openings that are wind-resistant and closed during storms*" [10]. Detailed calculations can be seen in Appendix B while a summary of the calculated wind pressure for the three roof zones (Corner, Edge, and Field) are provided in Table 3.3.

Table 3. 3 Wind uplift pressures for a residential roof in different locations using the current NBCC climatic data

Roof Zone	Bella Coola (q = 0.39kPa)	Ottawa (q = 0.41 kPa)	Fredericton (q = 0.38 kPa)
Corner	-2.23 kPa	-2.35 kPa	-2.17 kPa
Edge	- 1.67 kPa	-1.76 kPa	-1.63 kPa
Field	-1.19 kPa	-1.25 kPa	-1.16 kPa

Ottawa, Bella Coola, and Fredericton were analyzed for a 2°C global warming. Based on the classification in Table 3. 2 fall in the respective categories of Normal, Extreme, and Severe. The design values obtained using the reference wind pressure corresponding to a 2°C global warming are presented in Table 3.4.

Table 3. 4 Wind uplift pressures for a residential roof in different locations using future climatic data

Roof Zone	Bella Coola (q = 0.45kPa)	Ottawa (q = 0.42 kPa)	Fredericton (q = 0.40 kPa)
Corner	-2.57 kPa	-2.40 kPa	-2.31 kPa
Edge	- 1.93 kPa	-1.80 kPa	-1.73 kPa
Field	-1.37 kPa	-1.28 kPa	-1.23 kPa

When comparing Table 3. 3 and Table 3.4 the effect of using future climatic data can be seen in the form of percent change in Table 3.5. The percent change for Bella Coola is much higher due to it falling under the "Extreme" wind severity classification, while both Ottawa and Fredericton were respectively classified as "Normal" and "Severe" and their percent change values are much lower. This approach allows the effect that a specific global warming magnitude will have on the wind load to be accounted for.

Table 3. 5 Percent change of wind uplift pressures for a residential roof in different locations using future climatic data

Roof Zone	Bella Coola (q = 0.45kPa)	Ottawa (q = 0.42 kPa)	Fredericton (q = 0.40 kPa)
Corner	15%	3%	6%
Edge	15%	3%	6%
Field	15%	3%	6%

As part of the present thesis contribution, an online web tool "Climate-RCI" was developed. Using "Climate-RCI", designers can obtain the reference wind pressures for various global warming temperatures. Throughout this thesis a 2°C global warming magnitude is used, however, "Climate-RCI" provides reference wind pressures for all the global warming magnitudes ranging from 0.5°C to 3.5°C. "Climate-RCI" can be accessed at (https://nrc.canada.ca/en/research-development/products-services/software-applications/climate-rci/).

3.3 Resistance Chain for Steep Slope Asphalt Roofing Systems

Based on the installation procedure outlined in Section 2.3, the resistance chain is shown in Figure 3.7 for a steep slope residential roof. The deck is the structural support on which the eave protection membrane is adhered to. The underlayment is fastened to the deck. Once these components are installed, the shingles are laid and are attached by the use of fasteners and self-sealing strips at the overlaps located towards the edge of the shingle. All the elements of the chain are connected to each other and provide resistance against wind loading. To ensure the resistance is achieved, each of the individual links is required to overcome the load imposed on them and to continue to maintain and the connection between the elements. In the case that one of the chain links is broken, the roof components no longer act together as part of the system. This scenario happens when the wind uplift load exceeds the resistance chain. Thus, it is important to understand how all these components behave individually, and equally also together as a system in order to make an informed decision on component selection and construction methods.

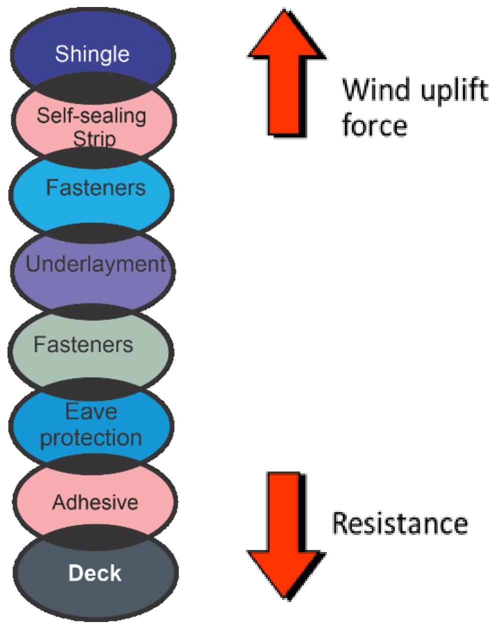


Figure 3.7 Resistance chain for steep slope asphalt roofing system

The shingles are the first layer of protection the residential roof has against wind, rain, snow debris, and exposure to sunlight. The shingles need to be able to overcome the stress and strain determined by the temperature fluctuations and the wind uplift. This is mainly achieved through the fiberglass mat that the shingles have in their body. The shingles are fastened in place and their self-sealing strip is activated through heat, thus ensuring the connection between the overlapping strips of the shingles' layers. The self-sealing strips of shingles allow the shingles' edge to adhere to the shingle below. This aids in preventing the wind uplift forces underneath the shingle from increasing. However, the activation of the self-sealing strips can fail either due to natural weathering of the adhesive or because of the load being applied onto it. Its failure can increase the failure risk of the shingles because the uplift pressure being applied will not have sufficient resistance. When the upper layers of shingles start detaching, this can cause the fastener to be pulled out of the deck or pulled through the shingle. Fasteners are high-stress areas and they transmit the force to the fiberglass mat and deck and assist the shingles in acting as a unified component. This is further achieved by the activation of the sealant strip located underneath the shingles.

The next components are the eave protection and the underlayment respectively, which aid more in the long term with the protection against the infiltration of the water and thus ensuring the roof is in good condition. The underlayment is fastened or adhered to the deck. The eave protection is typically adhered to the deck and is installed before the underlayment. The last component of the resistance chain is the roof deck, which apart from serving as the foundation of the roof upon which the rest of the elements are installed, it also serves in transmitting forces from the above deck level to the roof frame. Having a continuous resistance chain for the roof components is very important to have a continuous chain for the rest of the residential house to ensure that a complete continuous load transfer occurs.

As mentioned above the shingles are the first layer of protection for residential roofing systems. However, as discussed in Chapter 2, their failure exposes the remaining of the roof system to water infiltration and this is one of the most frequent damages to occur during a severe wind. This leads to identifying the connection of the shingle to the rest of the roof system as the weakest link in the resistance chain. This connection occurs through two modes simultaneously, fasteners and overlap seals. The modes of evaluation discussed in Chapter 2 cover the evaluation of fastener pull-through and overlap seal strength. However, the latter is not referenced in the NBCC or CSA A123.5 and thus not required to be evaluated. Therefore, this leads to the overlap seal strength to be identified as the missing link and its evaluation should be included in the NBCC. Another element that is not currently covered in the NBCC is the evaluation of the tear strength of the underlayment and the fastener pull-through. The underlayment provides secondary temporary protection for the roof, in case of when shingles are blown away. If the underlayment also fails, the roof is exposed to water infiltration which causes irreversible damage. The fourth weakest link is the strength between the eave protection and the deck. This interaction needs to be specified along with how this changes under a range of different temperatures and humidity conditions. The delamination of the eave protection from the deck can cause water infiltration.

3.4 Holistic Approach

3.4.1 Allowable Stress Design vs Load Resistance Factor Design

The Allowable Stress Design (ASD) method requires the necessary strength to be less than or equal to the allowable strength and it is expressed by the following equation [56]:

$$\sum_1^i L_{ni} \leq \frac{\mathrm{R_n}}{\mathrm{FS}} \tag{3.1}$$

The left side represents the required strength with the L_{ni} term representing the load effects. The right side represents the design strength or the allowable strength, where Rn is the nominal strength and the term FS is the factor of safety which is always greater than 1. The ASD approach uses a safety factor to account for uncertainties in the nominal strength and the load effects.

The Load Resistance Factor Design (LRFD) procedure is expressed using the inequality below and similarly to the ASD approach, the left side represents the required strength while the right side represents the design strength [56].

$$\sum_1^i \gamma_i L_{ni} \leq \emptyset \mathrm{R_n} \tag{3.2}$$

The term L_{ni} represents the load effects while γ_i represents the load factors for different load effects. It is always greater than one. The load factor increases based on the expected variability due to the probabilistic description of the loads. Rn represents the nominal strength while Ø is the resistance factor. The resistance factor is always less than one and accounts for uncertainty and variability for a specific material, failure, or limit state.

Assuming the load and resistance are random variables that follow a Gaussian distribution function, their distribution is shown in Figure 3.8 [57]. The Gaussian distribution is a normal probability distribution, with the following formula [56]:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-(x-\mu)^2/2\sigma^2} \tag{3.3}$$

In a Gaussian distribution, the values of the mean, median, and mode are the same as represented by the dashed line in the middle of the distribution curve (Figure 3.8).

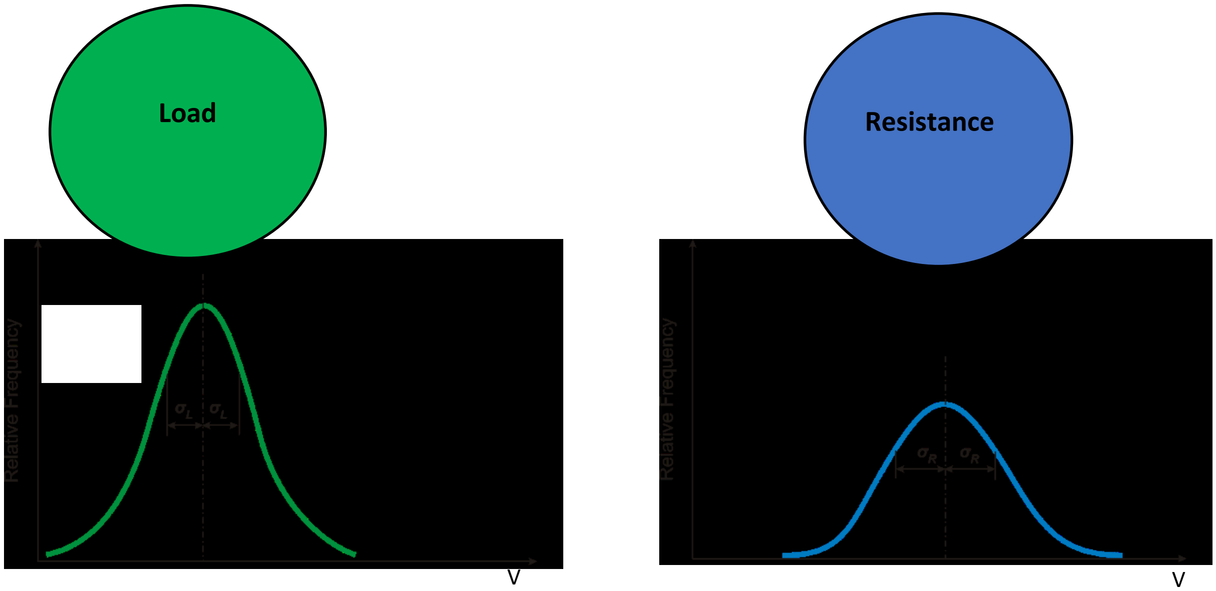


Figure 3.8 Gaussian distribution for load and resistance [57]

The area underneath each curve of the Gauss distributions represents the total probability of 100%. By assuming the distribution of the load and resistance as known, the distribution of the difference between the two values can be found as the failure area highlighted in red in Figure 3.9, indicating that the difference between the resistance and the load is negative for this part of the curve. The term β on the graph (Figure 3.9) represents the reliability index and a larger value will correspond to a higher safety zone for the considered design equation [56].

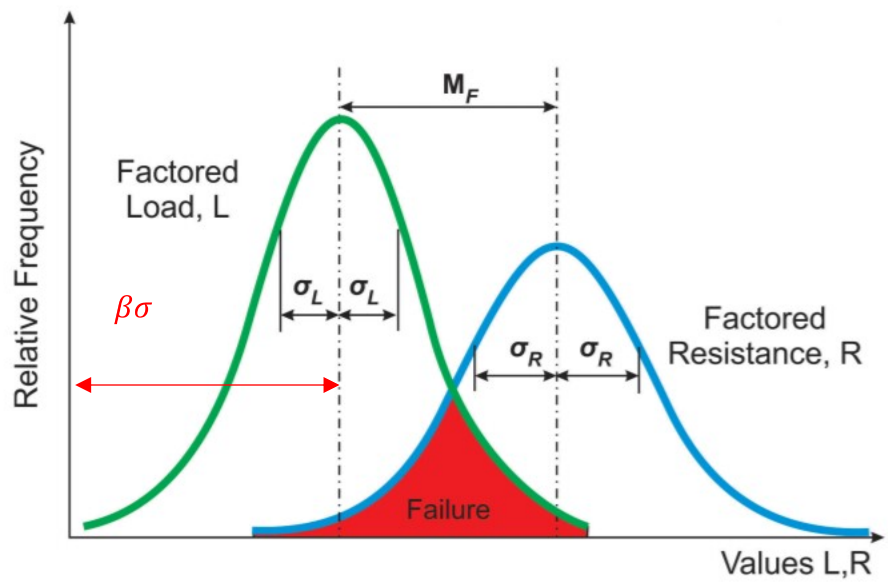


Figure 3.9 The LRFD approach [57]

Therefore, the factored load is the product of the load with a load factor to account for variabilities in the load, while the factored resistance is the product of the system's resistance and a resistance factor to account for variabilities in the material properties and uncertainties in the resistance determination.

The LRFD approach has been adopted for main wind force resistance systems; however, for the design of cladding, the ASD is still used. To overcome this difference there are ongoing discussions to bridge the gap [57]. This will be achieved by the adoption of the LRFD for cladding design. For some commercial roofs this has been achieved and how this can be achieved along with the steps required for steep slope residential roofs will be discussed in more detail in the next section.

3.4.2 Proposal of a Holistic Approach for Residential Roofs

The overall goal of the design process is to increase reliability and to decrease the risk of failure. However, the design process is closely related to the construction, and each phase contains its own risk. The risks associated with the design and construction are accounted to a certain degree by the LRFD approach. In the case of labour-intensive roof installation, construction uncertainties are higher. Incorporating the "Installation" metrics into the LRFD approach leads to the creation of the "Holistic Approach" for residential roofs.

The "Holistic Approach" takes into account the load, resistance, and installation of the roofing systems and aims to achieve the "Sweet Spot" (Figure 3.10). Having a larger “Sweet Spot” means the failure zone will be smaller. This will ensure to the owner the roof that is being designed and constructed is more resilient.

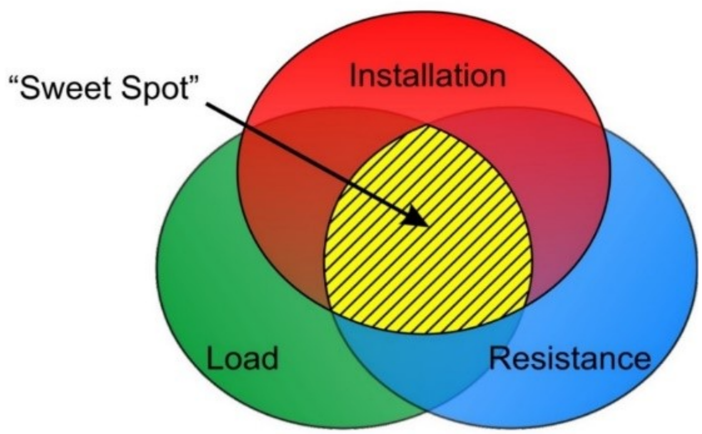


Figure 3.10 "Holistic Approach" to extend the service life of roof assemblies [57]

The "Installation" component is identified from the analysis of post-storm damages as the missing link. This explains why roofs designed for a specific load and constructed to achieve a certain system resistance did not perform as expected [57].

The first step of the "Holistic Approach" is determining the wind uplift load, based on the provisions outlined in the 2015 NBCC, along with future climatic data. This was discussed above and in Sections 2.2 and 3.2 in more detail. Using future climatic data obtained from Environment and Climate Change Canada (ECCC), the reference wind pressure is calculated and used in the final uplift pressure.

Determining the load that the roof needs to withstand is not enough to ensure that the constructed roof will actually withstand the design load. The roof system needs to be evaluated in a simulated lab environment to determine the system's sustained wind uplift pressure, which corresponds to the system's resistance. The resistance was discussed in more detail in Section 2.4. This resistance needs to be higher than the design requirement. If the resistance is higher than the design requirements, then this is the optimum system to be recommended for installation in the field. Therefore, the roof that is constructed in the field is not expected to yield a lower wind uplift value than that of the lab-tested roof system. However, if the tested resistance is lower than the design requirements, then this system cannot be installed in the field.

The third component, "Installation" is also assumed to also follow a Gaussian distribution as can be seen in Figure 3.11. By introducing "Installation" the risk associated with installation uncertainties can be minimized along with the failure area, which is an indicator of increased design reliability (Figure 3.12). Residential roofs are not fully engineered structures and do not undergo as much quality assurance from a party not associated with the project when compared with commercial roofs. This study only proposed the need to consider installation in the holistic approach. However, the probabilistic determination of the installation process should be carried in detail through future research.

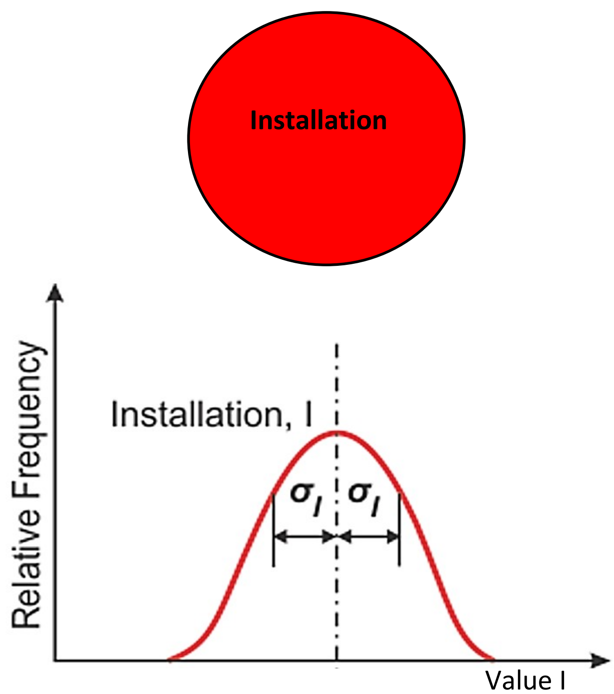


Figure 3.11 Gaussian distribution for Installation [57]

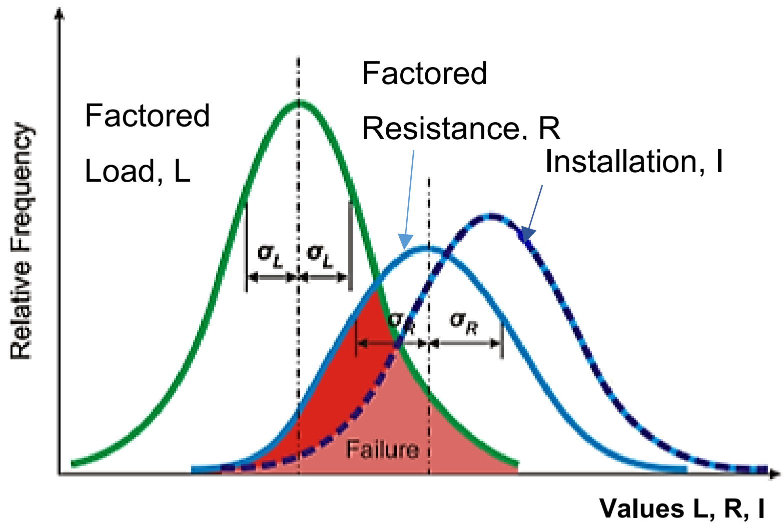


Figure 3.12 Increasing the reliability through Load, Resistance and Installation [57]

A typical statement used amongst roofers is, *"If the roof didn't leak then it was a good job!".* Considering the large variables all partaking in the roof construction to ensure it will be durable, the “Holistic Approach” needs to be adopted. This approach needs to take into effect climate change when determining the loads acting on the building. If only the factored load and factored resistance are taken into consideration in the overall design process, then this produces a large failure zone. This occurs because installation uncertainties are not considered. The roof system that was tested in the lab and whose resistance is higher than the design requirement is assumed to be installed in the field. However, this is not true because, during the construction process, there can be deficiencies. These can lead to the roof system to withstand a lower wind uplift pressure than that obtained for the tested resistance through laboratory evaluations. According to ICLR, construction errors are responsible for the majority of failures, such as the missing roof to wall roof sheathing fasteners or toe-nailed roof to wall connections being below code requirements [4]. This is the reason for which the third element of the "Holistic Approach", which includes installation, is required. By the introduction of the installation uncertainty to the LRFD approach, the failure zone is minimized, while the certainty of achieving the "Sweet Spot" is increased. The "Sweet Spot" allows that the wind uplift pressure withstood by the roof after installation is higher than the code requirement. By following this approach the damages and failures observed in Chapter 2 will be mitigated. By taking a case study, the steps involved in applying the Holistic Approach are provided in Chapter 6.

For steep slope asphalt roofs, the system resistance risk is similar to that of commercial roofs. For commercial roofs, the system resistance risk is composed of the minimum of the individual resistances of the different components. This is due to the minimum value representing the weakest link (Figure 3.13) [57]. In Figure 3.13 only some of the components composing the system resistance are shown for simplicity. For commercial roofs, the resistance factor for various systems was determined by Special Interest Group on Dynamic Evaluation of Roofing Systems (SIGDERS) using the following procedure [57]:

- Form probability distribution
- Compute the reliability parameters
- Compute the reference resistance
- Calculate the Resistance factor "Reference Resistance/mean tested resistance"

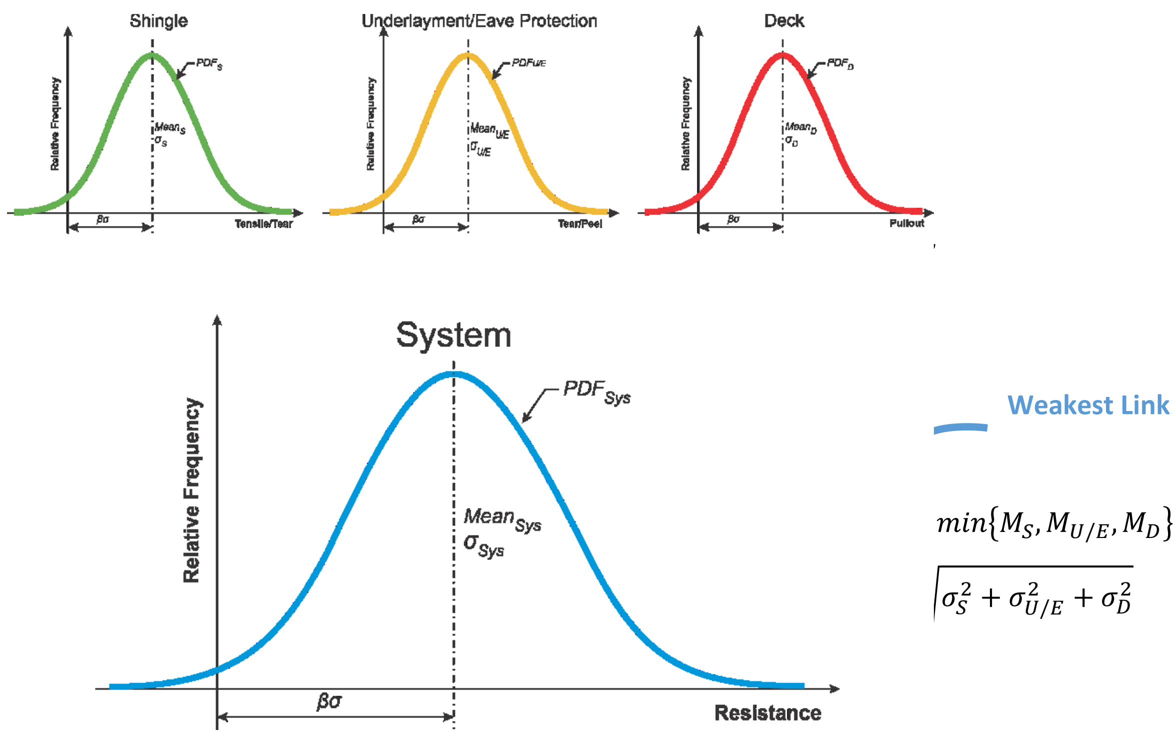


Figure 3.13 System Resistance for steep slope asphalt roofs (adapted from SIGDERS [57])

For low slope roofs, the resistance factor has been determined to be 0.65 [57]. This factor needs to be determined for residential roofs such that the factored resistance can be used for the LRFD. The resistance factor for steep slope residential roofs is a missing link and is required to be determined. Due to the vast amount of data required to develop the resistance factor, this research will not concentrate on developing the resistance factor, this is a limitation of this thesis. The current study will concentrate on providing the overall steps required to fulfill the Holistic Approach including the resistance factor.

3.5 Concluding Remarks

As presented, wind speed and therefore wind load applied to roofing systems can be unpredictable and variable. It can cause tremendous damages if structures are not adequately designed to withstand it.

This chapter addresses the missing links that need to be addressed to improve the resiliency of steep slope residential roof coverings. Initially the implementation of the future climatic data in the design load, which is the first step of the “Holistic Approach”, is performed. Thereafter, to provide guidance for using future climatic data, the global warming magnitude of 2°C was selected for roof design. This increase is expected to occur over the next 30 years, which corresponds to the design life of roofing systems.

Moreover, the resistance chain for the steep slope asphalt residential roofs was defined along with the weakest links. The resistance is the second step of the “Holistic Approach”. The resistance of a roof is achieved through the interaction of all the roof components together. The roof will be as strong as its weakest link. The weakest links identified need to be properly addressed to ensure that the roof’s overall performance is not affected.

A review of post-storm reports and industry consultations highlights a missing link, which is taking into consideration uncertainty and risk due to installation. By incorporating installation, the third step, in the LRFD approach, the "Holistic Approach" can be applied for residential roofs, ensuring more resilient roofing systems.

This chapter concludes with the development of the “Holistic Approach”, which addresses the missing links highlighted through the review of the design and construction approaches and the field performance of the installed roofing systems.

Chapter 4: Experimental Setup and Procedures

4.1 Introduction

This Chapter outlines the procedure and experimental setup used in the current study to quantify the impact of weathering on the component and system resistance. The quantification of the impact of weathering on the component resistance is achieved through characterizing asphalt shingles' mechanical properties. The standards followed for each of the component tests are described, along with the experimental setup. The quantification of the impact of weathering on the system resistance is achieved through the use of the Wind Induced Damage Simulator, by concentrating on the behaviour of the cladding. The experimental setup, mock-ups, and protocol followed are discussed, along with the classification of different failure modes.

4.2 Experimental Setup and Procedures – Component Resistance Evaluation

Test specimens were prepared in accordance with CSA A123.5 *"Asphalt shingles made from glass felt and surfaced with mineral granules"* [26]. CSA A123.5 provides the minimum and maximum property values that a shingle should meet. It also refers to a set of standards for the test methodology to evaluate these properties. The following component properties that can be correlated with the wind performance are evaluated as part of the present study:

- Tensile strength.
- Tear strength.
- Fastener pull-through,

This section outlines the experimental procedure followed for each of the component tests.

4.2.1 Tensile Strength Evaluation

The specimen preparation and determination of the tensile strength of the shingles was completed in accordance with ASTM D146 *"Standard Test Methods for Sampling and Testing Bitumen-Saturated Felts and Woven Fabrics for Roofing and Waterproofing"* [58]. For each shingle sample, ten shingle specimens with dimensions of 25 mm by 150 mm were prepared. During the manufacturing process, each shingle sheet has a machine direction (MD) and cross-machine direction (XD) (Figure 4.1). The machine direction is the longer length of the shingle, while the cross-machine direction is the shorter length. CSA A123.5 requires both the MD and XD directions to be evaluated and ten samples were extracted for testing from both directions.

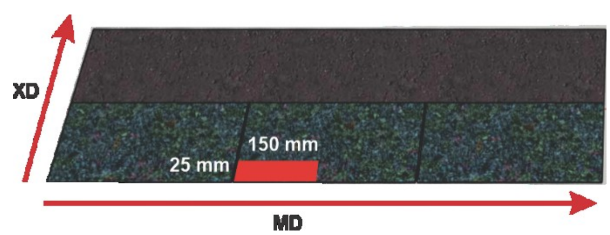
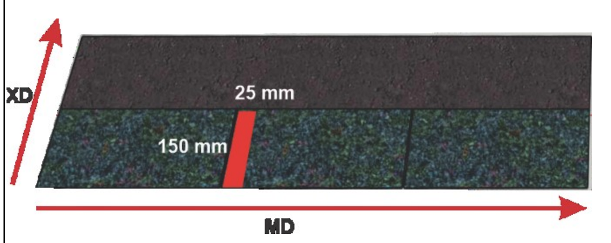
Machine Direction Specimen (MD)	Cross-Machine Direction Specimen (XD)
	

Figure 4. 1 Specimen preparation for tensile strength evaluation

The specimens were conditioned at lab conditions (23±2°C and 50±5% RH) for 2 hours before testing. When performing the tests, the specimens were clamped at the edges by the upper and lower grips, leaving a test area of 75 ± 3 mm. The grips are connected to a Universal Instron testing machine model 5566 (Figure 4.2), and a constant crosshead speed of 51 mm/min is selected as required by the standard.

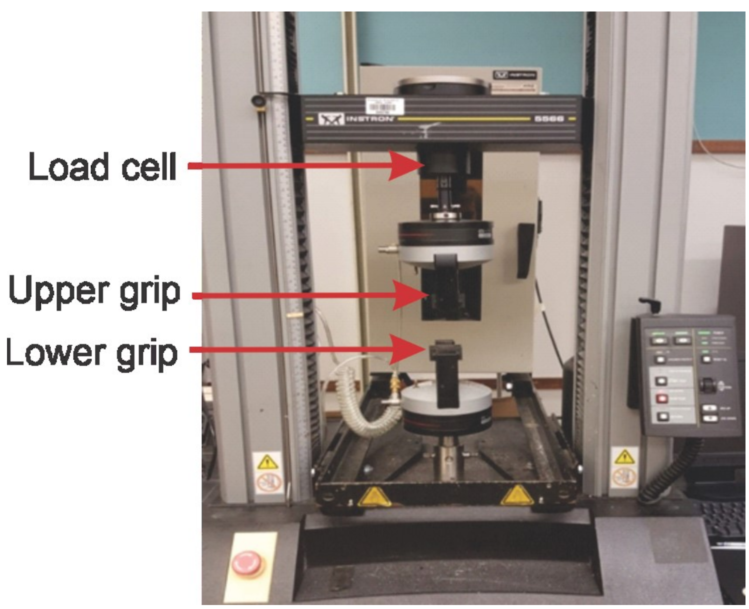


Figure 4.2 Instron setup for tensile strength evaluation

The crosshead applies an increasing force to the specimen clamped between the moving upper grip and the stationary lower grip and the specimen is pulled until it breaks. A typical load extension time history is shown in Figure 4.3. The force required to break the shingle is recorded in Newtons, while the extension is measured in millimetres. The tensile strength is then calculated as outlined in ASTM D146, which differs from the typical expression of maximum force per unit cross-sectional area, due to the shingle cross-sectional area being very small. The force in Newtons is converted to a force per unit length (KN/m) by multiplying the force by a coefficient of 0.04 as prescribed in ASTM D146. The load applied on the shingle specimen against the extension occurring within the specimen increases until point (a), where the transition from elastic region to plastic region occurs until point (b). Once the maximum load is reached at point (b) on the graph the specimen breaks and that is represented by the sudden drop in the graph.

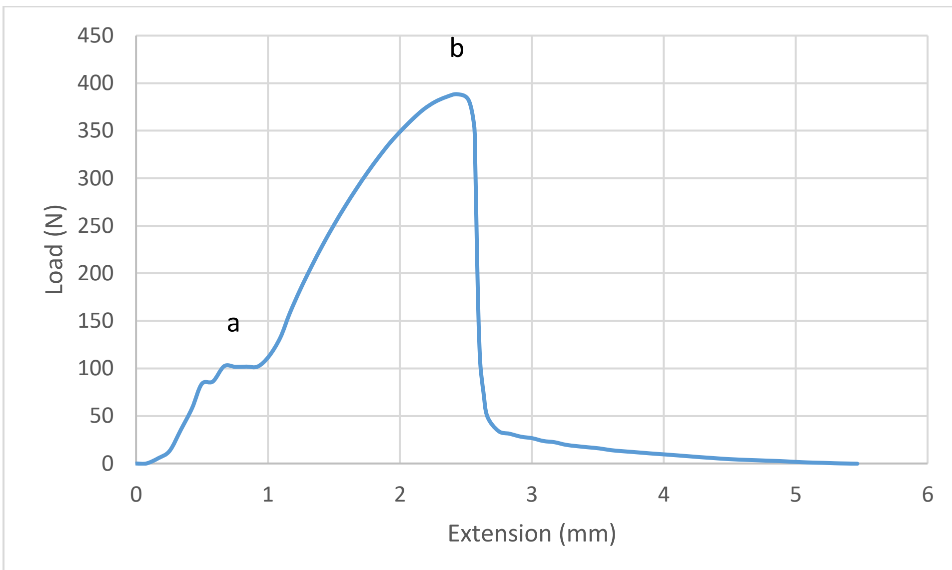


Figure 4.3 Typical load-extension time history

When the Instron test is conducted, if the breaking occurred within 25.4 mm of where the clamps were located, this is identified as a false break. This specimen is discarded, and additional specimens are added to the test program. An example of a breaking mechanism considered acceptable, which occurs in the middle of the test area, is illustrated in Figure 4.4.

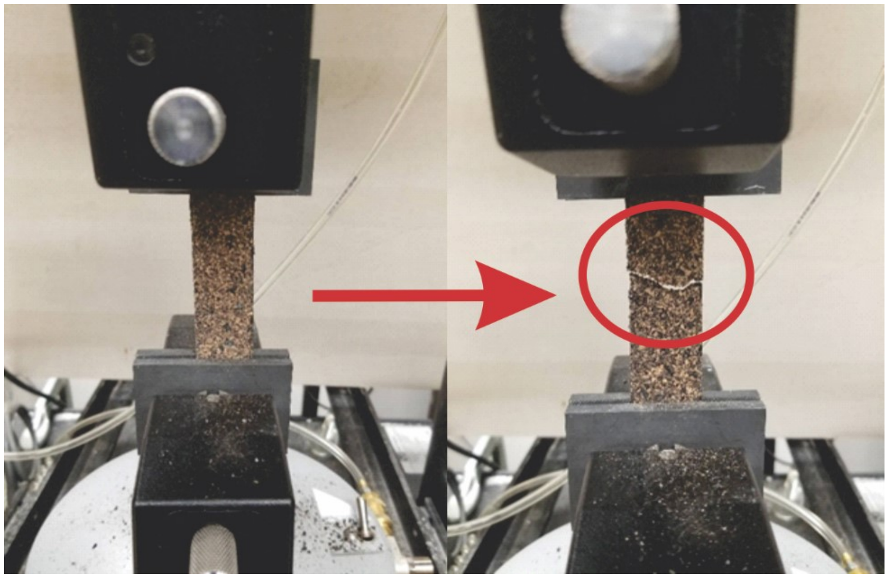


Figure 4.4 Acceptable specimen break for tensile strength evaluation

4.2.2 Tear Strength Evaluation

For the tear strength, ten specimens of dimensions 76 mm by 63 mm ± 3% were prepared and tested in accordance with section 13 of ASTM D228 *"Standard Test Method for Sampling, Testing, and Analysis of Asphalt Roll Roofing, Cap Sheets, and Shingles Used in Roofing and Waterproofing"* [3] (Figure 4.5). The shingles' tear strength was determined using an electronic Elmendorf ProTear testing machine (Figure 4.6).

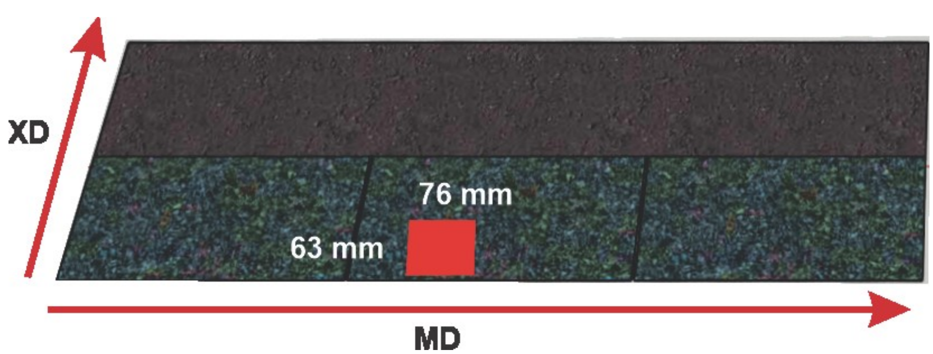


Figure 4.5 Specimen preparation for tear strength evaluation

The steps followed for the tear strength determination are summarized below:

- **Step 1**: The specimen is placed between the clamps with the granule side facing away from the pendulum.
- **Step 2**: The specimen is secured using the “Clamp” button.
- **Step 3**: An initial tear is performed as shown in Figure 4.7, by lowering the initial tear handle.
- **Step 4**: The handle is returned to the original horizontal position.
- **Step 5**: The “Test” button is selected and the pendulum is released.

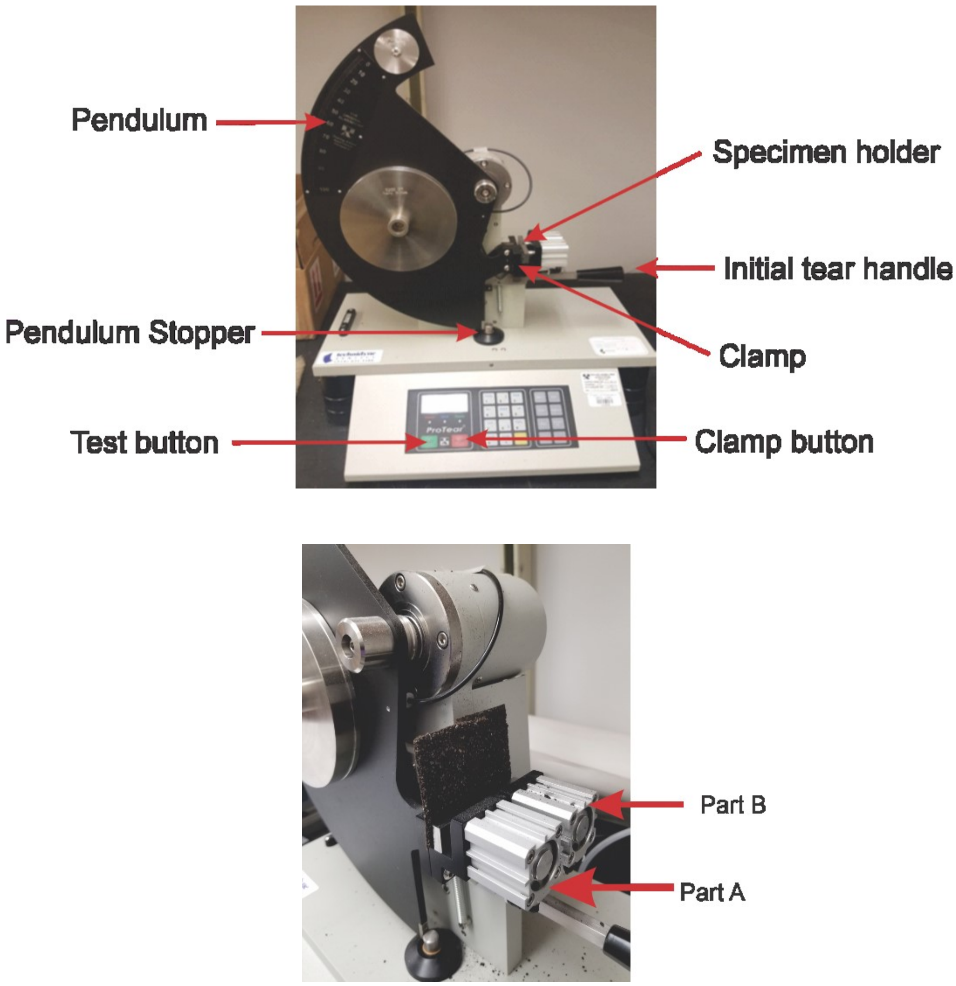


Figure 4.6 Elmendorf ProTear setup for tear strength evaluation

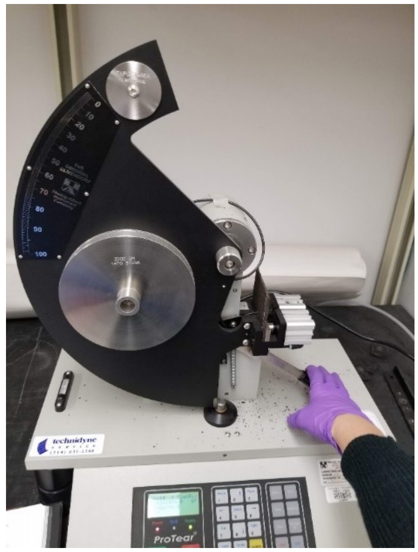


Figure 4.7 Initial tear before testing and specimen holder parts

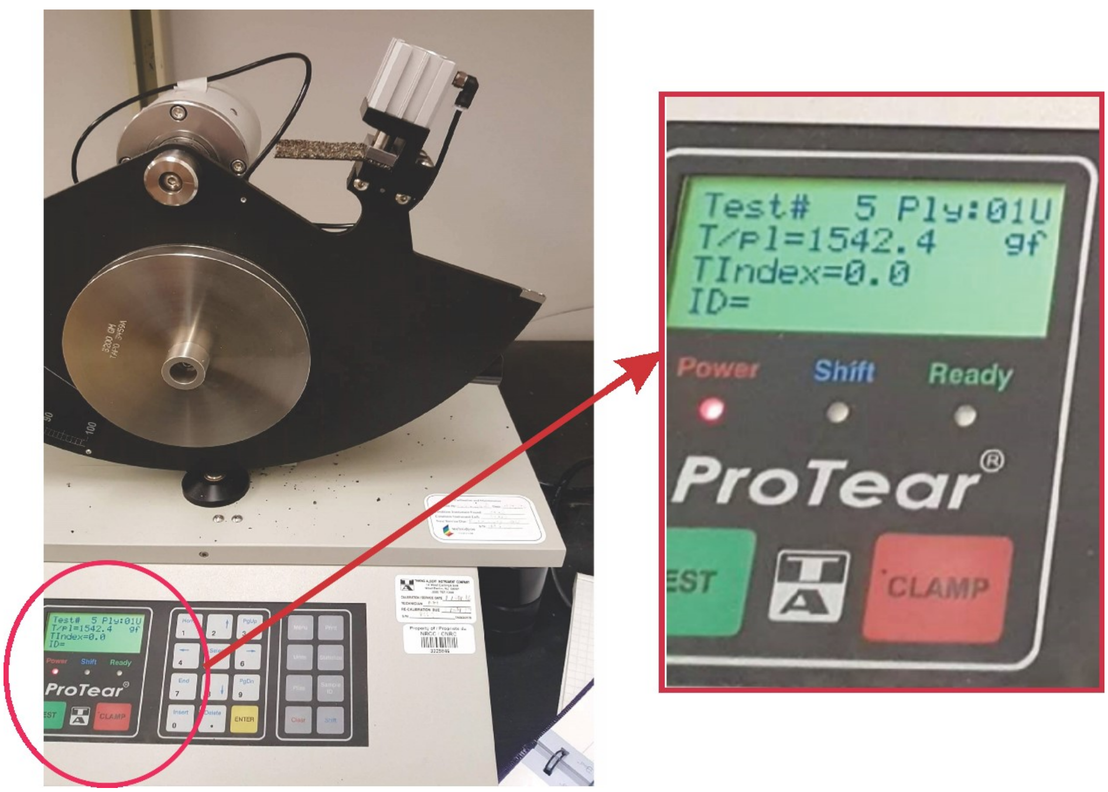


Figure 4. 8 Typical output from the Elmendorf ProTear

The specimen holder is composed of two independent parts with respective independent clamps (Figure 4.6). Part A is attached to the pendulum while Part B is stationary and attached to the base holder of the machine. Once the specimen is placed in the specimen holder between the clamps, it is fixed and an initial tear is performed to allow for the test to be performed. If the initial tear would not be performed the pendulum would fail to tear the specimen and it wouldn't move substantially from its original upright position. This constitutes the completion of steps one to four. When step five is executed and the "Test" button is pressed, this allows the pendulum stopper to be lowered allowing the pendulum to fall. The pendulum stopper was previously keeping the pendulum in place and preventing its premature release. The tear strength is measured through the absorption of energy during the tear of the shingle while the potential energy of the pendulum is transferred to kinetic energy. This tears the shingle in half, with one half being in Part A, and the other in Part B. The tear strength is reported as grams (g). A typical output obtained from the ProTear once the test is completed is shown in Figure 4.8.

4.2.3 Fastener Pull-Through Strength Evaluation

Specimens for the fastener pull-through resistance were prepared and tested in accordance with section 14 of ASTM D228 *"Standard Test Method for Sampling, Testing, and Analysis of Asphalt Roll Roofing, Cap Sheets, and Shingles Used in Roofing and Waterproofing"* [59]. Ten square specimens of 98 ± 3 mm were prepared from the nailing area where the fastener would normally be installed according to the manufacturer's specifications (Figure 4.9). In the center of each specimen, a 38 mm long galvanized roofing fastener is inserted (Figure 4.10). The roofing fastener has a 9.5 mm diameter head which rests on the granular surface of the shingle. The reference values established in 2005 determined the fastener pull-through resistance at laboratory conditions of 23℃ instead of 0℃ as outlined in CSA A123.5. The same approach was followed in the current study to ensure the reference values obtained in 2005 could be used as reference when quantifying the effect of weathering. For this reason, ASTM D3462 *"Standard Specification for Asphalt Shingles Made from Glass Felt and Surfaced with Mineral Granules"* [60] was used to determine the acceptable test values under laboratory conditions.

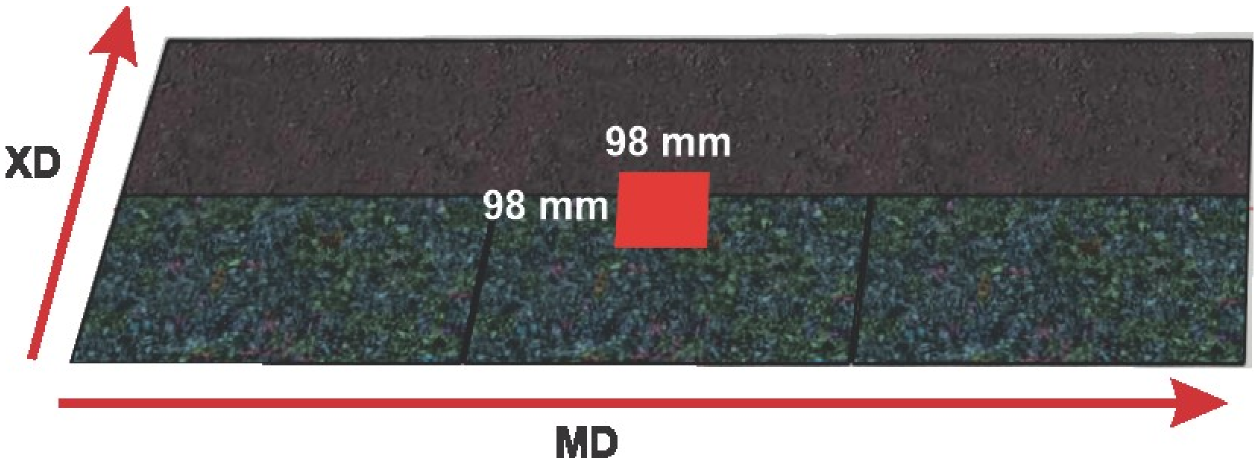


Figure 4.9 Specimen preparation for fastener pull-through evaluation

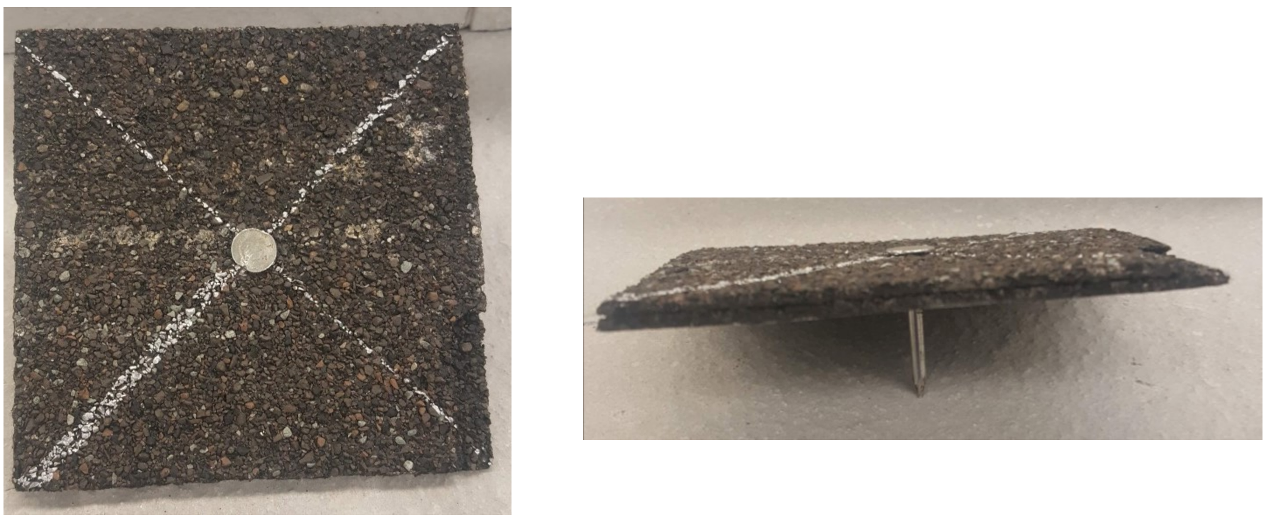


Figure 4.10 Fastener placement in the shingle fastener pull-through specimen

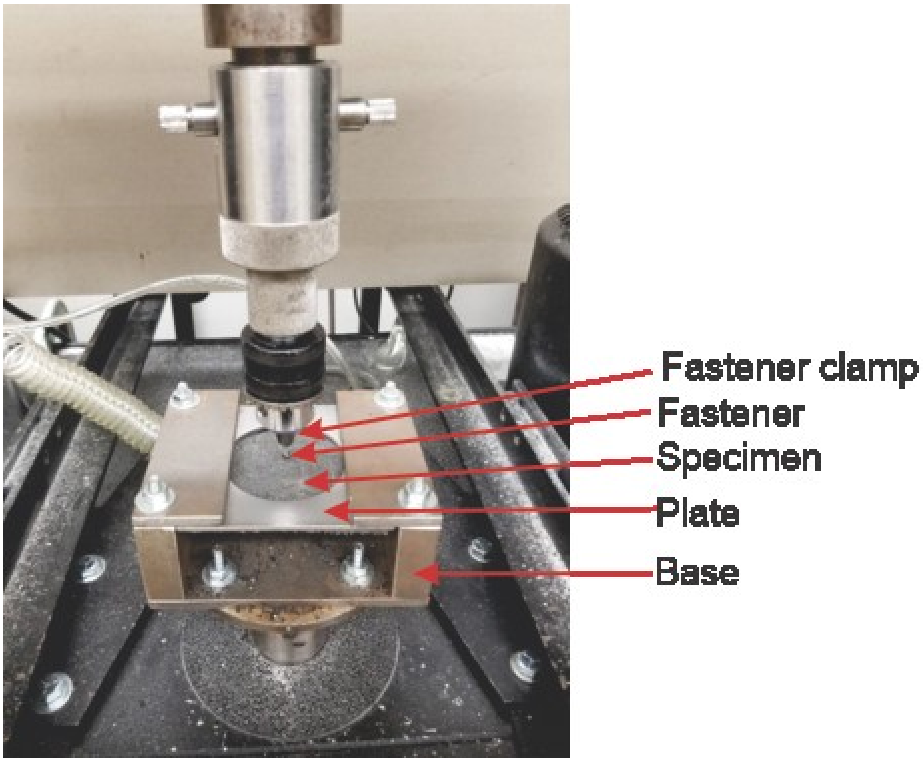


Figure 4.11 Instron setup for fastener pull-through evaluation

Testing is performed using a Universal Instron testing machine model 5566 with the specimen holder shown in Figure 4.11. The specimen holder is composed of a steel base and a plate, such that only the fastener will be pulled, while the shingle specimen is held in place by the plate. The base is secured to the frame of the Instron, while the plate is placed on top of the specimen and both are placed on the base. The specimen is arranged with the granules facing downwards and the underside of the shingle and the shank of the nail facing upwards. The fastener shank is clamped inside the fastener clamp and is pulled upwards at a constant speed of 100 mm/min. A typical load extension time history is shown in Figure 4.12. The load applied on the fastener increases until the maximum load is reached causing the fastener to be pulled through the specimen. For the fastener pull-through test the maximum force required to pull the fastener through the shingle is recorded and the average of ten specimens is determined along with the standard deviation. This test simulates the uplift force created underneath the shingles, causing the shingle to lift and the fastener head to be pulled through.

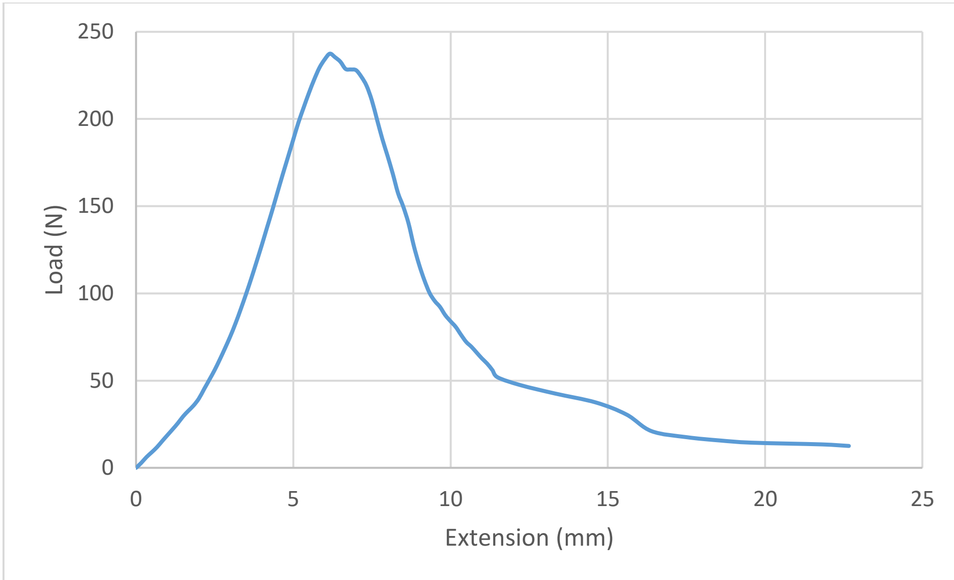


Figure 4.12 Fastener pull-through test output example

4.2.4 Sample Selection and Labelling

As part of an ongoing research and development effort, the National Research Council has been evaluating the performance of shingles from the early 2000s [63]. To demonstrate the effects of aging on the durability of shingles, the current study compared the data with the ongoing National Research Council's Canadian Climate Resiliency Requirements for Asphalt Shingle Roofing project. Two types of shingles were evaluated in this study: shingle Type 1, an architectural shingle where the shingle is composed of two laminated layers, and shingle Type 2, a three-tab shingle where the shingle is composed of one layer. Type 1 and Type 2 shingles are products from different manufacturers and a direct comparison between the two cannot be accomplished. The samples were labelled as follows:

- **New 2005**: Shingles that were initially acquired and tested in 2005, both Type 1 and Type 2.
- **Aged 13 Years**: Type 1 shingles that were acquired in 2005 and were installed on a mock-up at the NRC outdoor exposure site and were naturally aged for 13 years and tested again in 2018.
- **Aged 15 Years**: Type 2 shingles that were acquired in 2005 and were installed on a mock-up at the NRC outdoor exposure site and were naturally aged for 15 years and tested again in 2020 by the present study.

- **Lab 13 Years**: Type 1 and Type 2 shingles purchased in 2005 and stored at lab conditions for 13 years before testing in 2018 by the present study.
- **New 2020**: Type 1 shingles purchased in 2020 from a supply store in Ottawa and tested as part of the current research. Note that Type 2 was no longer being sold in Canada, however, the existing samples listed above were tested to provide more information on the effect of weathering.

For shingle Type 1 the different samples ("Aged 13 Years", "Lab 13 Years", and "New 2020") can be seen in Figure 4.13. The colour between "New 2020" and "Lab 13 Years" is similar; however, when compared to "Aged 13 Years" there is a drastic decolourization. The weathering due to exposure to environmental elements especially to UV radiation during the past 13 years have caused granule loss and colour change.

Figure 4.13 Shingle Type 1 "Aged 13 Years" vs "Lab 13 Years" vs "New 2020"

Due to the limited amount of shingles stored in the lab, only five specimens were available for testing of the "Lab 13 Years" samples, instead of the required ten. After being removed from the mock-up the Type 1 "Aged 13 Years" and Type 2 "Aged 15 Years" shingles were kept in laboratory conditions for 24 hours before the specimens were prepared. Table 5.1 summarizes the test matrix followed in this study ("New 2020", "Lab 13 Years", "Aged 13 Years", and "Aged 15 Years") as well as the baseline available for comparison ("New 2005"). In total, a number of 80 specimens were tested for tensile strength, 40 specimens for tear strength, and 35 specimens for fastener pull-through strength. The data for "New 2005" was obtained from the ongoing NRC study, to be used for assisting in the quantification of the weathering impact on the shingles.

Table 4. 1 Test matrix for the component investigation

Property	New 2005	Lab13 Years*	Aged 13 Years	Aged 15 Years	New 2020
Tensile Strength	Type 1 Type 2	Type 1 Type 2	Type 1	Type 2	Type 1
Tear Strength	Type 1 Type 2	Type 1 Type 2	Type 1	Type 2	Type 1
Fastener Pull-through Strength	Type 1	Type 1	Type 1	Type 2	Type 1

*Only five specimens were tested.

4.3 Experimental Setup and Procedure – System Resistance Evaluation

The shingle component testing performed provided information regarding the strength of the shingle for different failure mechanisms, as stipulated in the standards CSA A123.5 and ASTM D3462. These quantified the effect of weathering on the shingle through individual tests which typically provide an estimate of the wind resistance [27]. However, to determine the effect of weathering on the roofing system resistance, mock-ups are tested under simulated wind conditions. To quantify the effect of weathering both new and weather aged mock-up were tested. The Wind Damage Simulator (WDS) located at the University of Ottawa, which has capabilities of simulating high-intensities wind pressures, was used for the roofing system resistance evaluation. A general schematic of the system resistance experimental setup in the WDS with the roof mock-up, dummy structure, inlet, and outlet along with other elements can be seen in Figure 4.14. Each of these elements will be discussed in greater detail in this section.

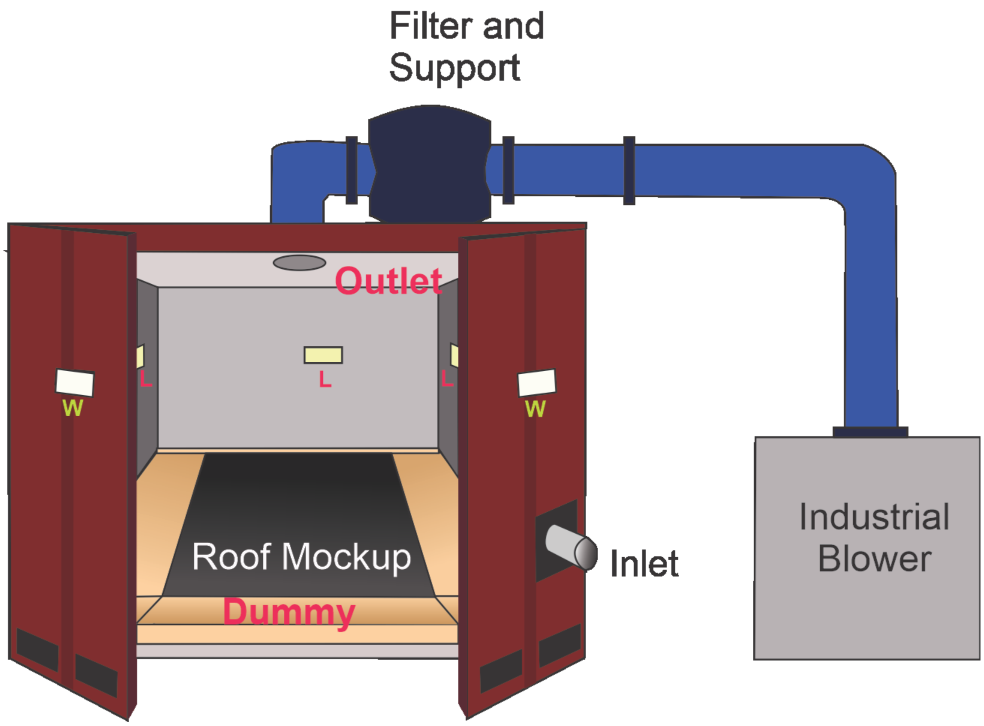


Figure 4.14 Sketch of the Wind Damage Simulator with the roof mock-up

The WDS is a steel box with the following dimensions 3.65 m (width) by 3.65 m (length) by 3 m (height). The WDS experimental facility allows the simulation of multi-directional wind flows, which can be experienced during severe weather. This would not be possible with a conventional wind tunnel, which recreates a unidirectional wind flow that would be found in the lower troposphere layer of the atmosphere [61]. The WDS facility has the capability to employ up to 20 inlet openings on the lateral walls and an industrial fan connected to the ceiling outlet, with a maximum speed of 3,560 RPM. However, due to safety reasons, the maximum fan speed used in the current investigation was kept below 3,000 RPM. This can be correlated to a wind speed of 237 km/h at the pressure taps located underneath the outlet. The industrial fan is depicted as "Industrial Blower" in Figure 4.14, while a few of the inlet openings are represented by black rectangles, as these were covered during the experiment. In Figure 4.14 the three lights WDS has are represented by the yellow rectangles (L) which are used to illuminate the testing chamber during testing, to allow observations to be made through the viewing windows (W), depicted by the white rectangles. The roof mock-up of 1.8 m x 1.8 m was placed in the center of the WDS directly below the outlet, thus with direct exposure to suction, and a dummy model was constructed to surround the roof mock-up, in order to have air sealed model continuity with the testing chamber.

Initially, the test was run with all the inlets closed; however, due to the permeable nature of the shingle, the necessary pressure differential could not be created between the regions underneath the shingle and the exposed top surface of the shingle. If the pressure differential between the upper and lower surfaces of the shingles is not achieved, the uplift of the tested mock-up will not occur. To overcome this, a test approach, during which extreme wind conditions were simulated, was conducted by opening only one inlet and directing the stream of high wind speed towards the shingles of interest. The inlet selected can be seen in Figure 4.16; the inlets of the lower row could not be used because these were underneath the roof mock-up and the wind could not reach the shingles' top surface.

The tests were ran at five intervals of 1,300 RPM, 1,700 RPM, 2,100 RPM, 2,400 RPM, and 2,800 RPM. These intervals correspond to a pressure of 0.7 kPa, 1.4 kPa, 2.1 kPa, 2.8 kPa, 3.5 kPa. Each of the intervals was maintained for 30 minutes and the protocol can be seen visualized in Figure 4.15. This test's pressures are similar to the one discussed under Section 2.4.2.

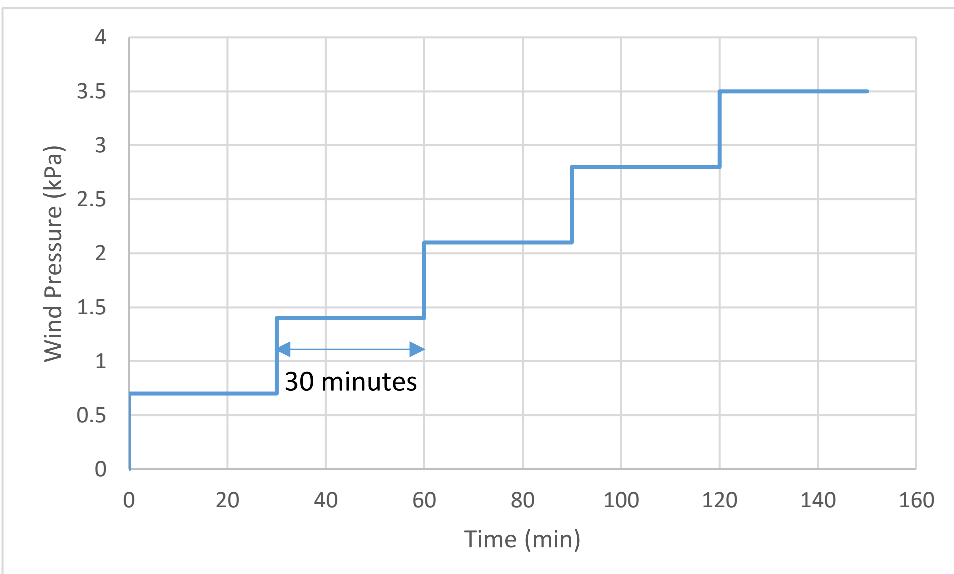


Figure 4.15 Test protocol followed for current research

The roofing system test protocol used is based on the FM Global 4470 [31], which recommends sustaining each wind-induced pressure for 60 seconds before moving to the next pressure level. The current test method was modified to sustain each pressure for 30 minutes because this duration is closer to Testing Application Standard (TAS) 100 [28], which alternates between 15 to 10 min intervals of high wind speed and zero wind speed. Therefore, each wind-induced pressure was maintained for 30 minutes before moving to the next pressure. During each pressure level observations were made which are summarized and discussed in Chapter 5. Also, to ensure that the shingles of interest are directly exposed to the incoming wind speed from the inlet, an extension was installed outside the inlet with a given angle of 60 degrees, as can be seen in Figure 4.16. The extension is kept in place by four bolts, however, due to the extension being made from steel, a few wood supports were placed underneath to provide support for the heavy weight. A camera was placed at the top window and video recording of the tests was also recorded.

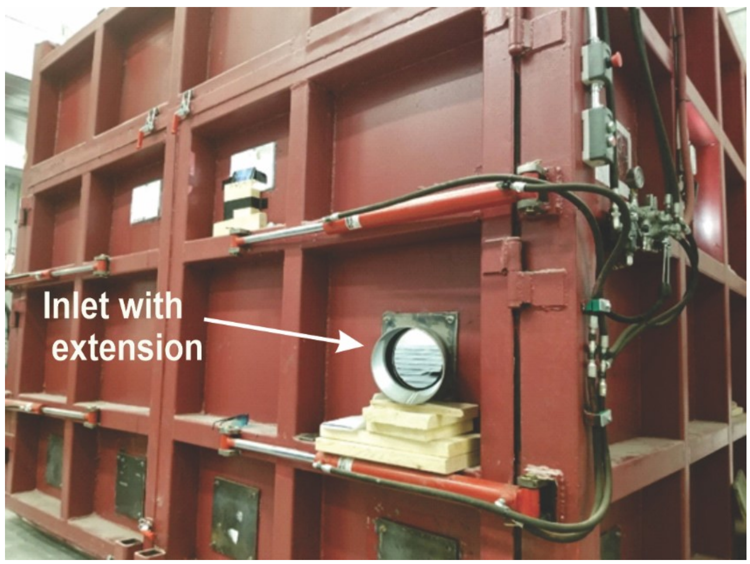


Figure 4.16 Inlet selected to stay open during the test

The roofing system experimental program consisted of testing four roof mock-ups, which are discussed in more detail below. The joints created between the roof mock-up and the dummy structure were sealed with a self-adhering membrane to further ensure continuity of the roof mock-up and the presence of no gaps (Figure 4.17). The shingles on the mock-up have been assigned a number to assist in the observation process during the test (Figure 4.18).

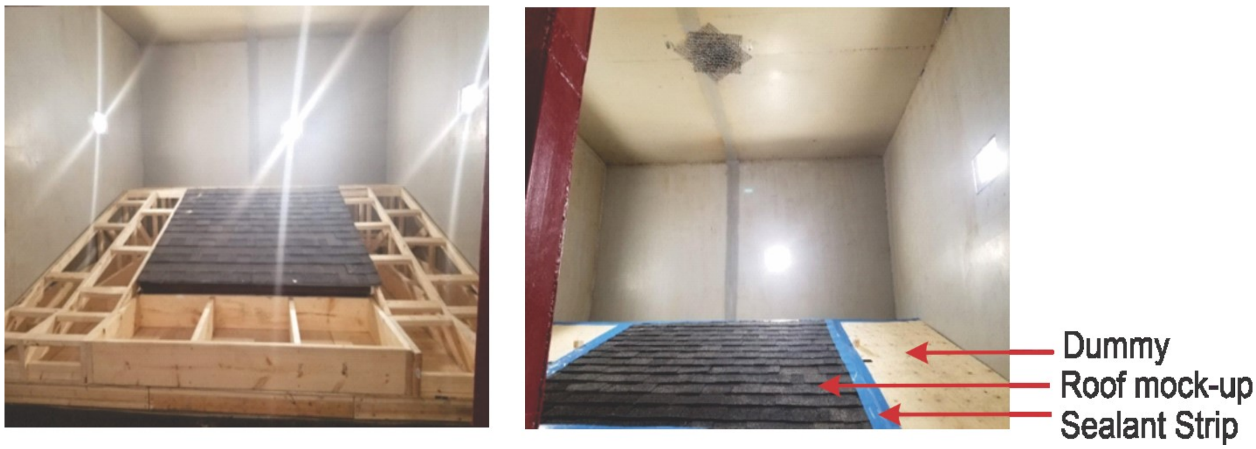


Figure 4.17 Mock-up setup in Wind damage simulator

Figure 4.18 Shingle row identification labelling

4.3.1 Instrumentation and Data Acquisition System

To measure the wind-induced pressures during the experiments, four pressure taps were installed. As the shingles were expected to fail during the test, two pressure taps were installed on the right side of the mock-up, where one was connected to the Scanivalve DSA 3217 pressure scanner and the other to the pressure calibrator Fluke 719 30G (Figure 4.19 a). Another pressure tap was installed at the same height but on the left side of the mock-up (Figure 4.199 b). These were installed at two-thirds of the roof length, which corresponded to the projected location of the fan located at top of the wind box. This placement favoured the recording of the maximum pressure applied on the roof mock-up without interfering with the integrity of the mock-up and the shingles. The fourth pressure tap was installed at the inlet; however, due to high wind speed at the time of the experiment, this detached, and data was not recorded for all the tested cases.

Figure 4.19 Installation of pressure taps near the mock-up

All pressure tap locations can be seen in Figure 4.20 along with the support structure constructed to keep the pressure taps near the mock-up in the required upright position. This position ensures the wind pressure is perpendicular to the pressure tap. The values were recorded using a Scanivalve DSA 3217 pressure scanner, shown in Figure 4.21, which is powered through a power supply and connected to a laptop via an Ethernet cable. A pressure calibrator was used to verify the values being recorded by the Scanivalve DSA 3217 module. Figure 4.21 shows how the measured pressure at the pressure tap is transmitted to the Scanivalve DSA 3217 module where it is processed and entered into the laptop. The laptop uses a Scanivalve application open to receive and store the data.

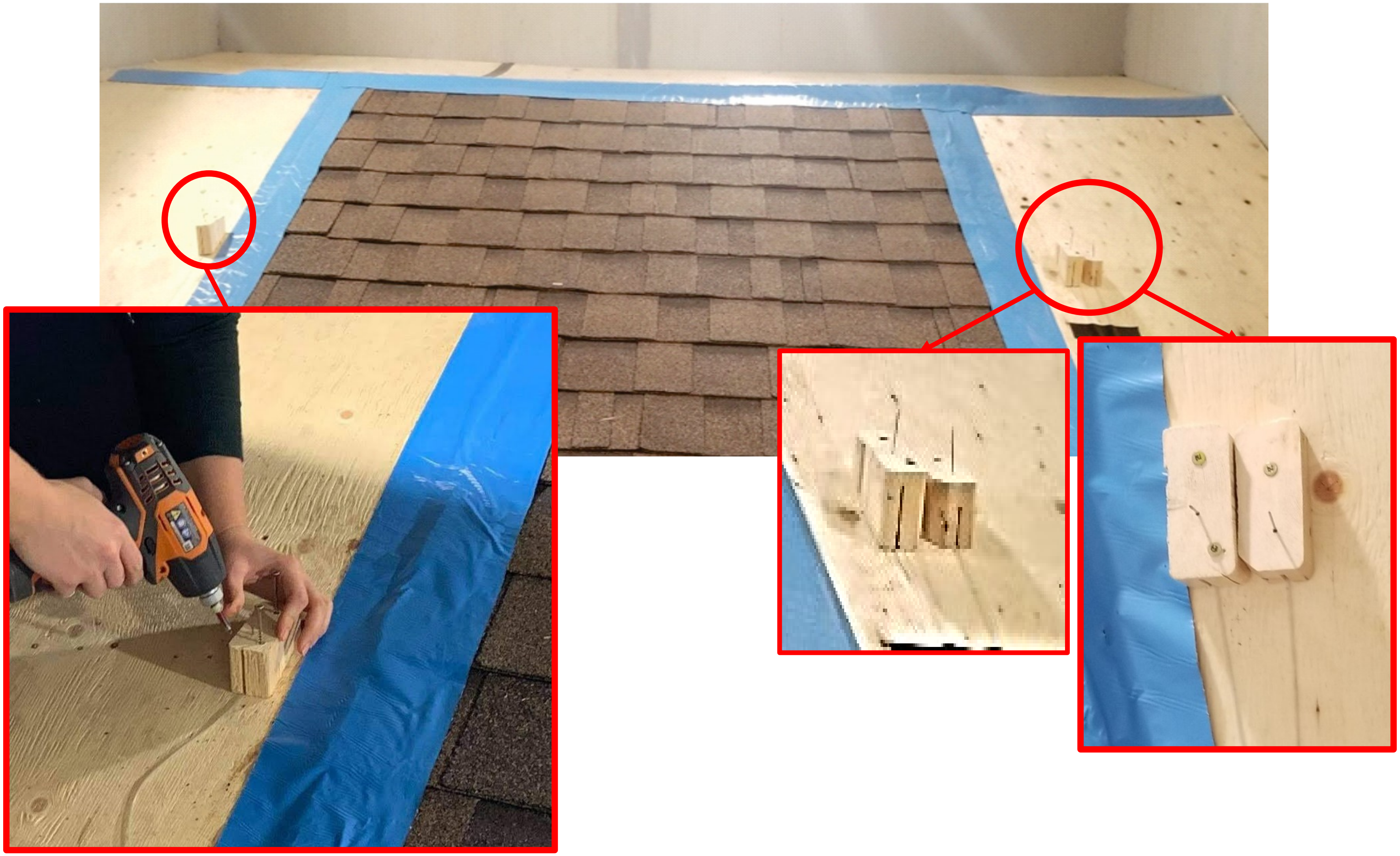


Figure 4.20 Pressure taps locations on the mock-up in the wooden support structure

Figure 4.21 Schematic layout of data recording (adapted from Moioli et al [62]*)*

4.3.2 Mock-up Labelling

According to the NBCC for wind load calculations, the roof has three distinguishable zones: corner, edge, and field [10]. These three zones for a gable and hip type steep slope residential roof can be seen in Figure 4.22. The wind pressure is highest in the corner and lowest in the field. During this study four mock-up were tested, two that represented the field zone of the roof and two that represented the edge zone. The mock-ups representing the edge zone had eave protection, as discussed in Chapter 2, which is required for a length of 900 mm along the edge.

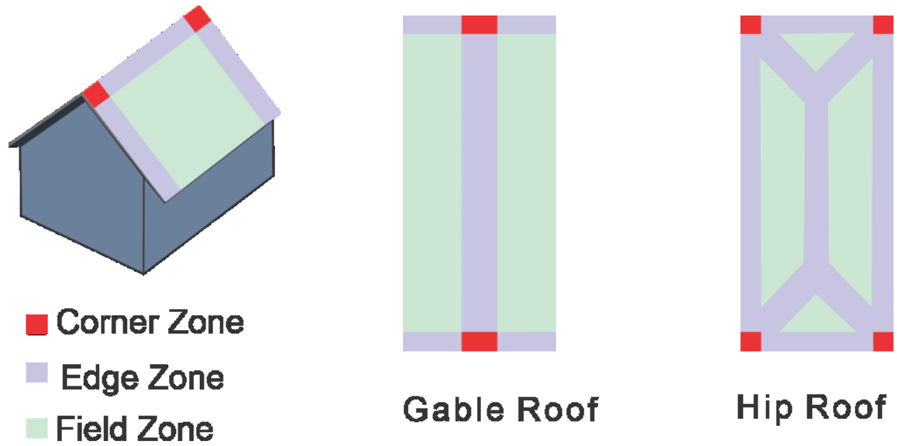


Figure 4.22 The three roof zones on a gable roof and hip roof [10]

Two of the mock-ups representing the field and edge of the roof were naturally aged at NRC, in outdoor environmental conditions for 15 years. Both were transported to the University of Ottawa laboratory for testing. These were labelled “Aged Field 2005” and “Aged Edge 2005”, respectively. The other two mock-ups representing the edge and field were newly constructed and labelled as “New Edge 2020” and “New Field 2020”, respectively. The newly constructed mock-ups were built following the current construction methods for steep slope roof cladding as outlined in NBCC and Section 2.3. Their construction was performed by maintaining the same roof trusses as the aged mock-ups and replacing the deck and all other above deck components. The construction of the new mock-ups was performed under laboratory conditions. Therefore, the new shingles were not exposed to the heat needed to allow the self-sealing sealant to be activated. The self-sealing sealant in roofs constructed in practice would typically be activated after exposure to the sun. However, this is not always the case, either due to roofing construction occurring in unfavorable weather conditions, or due to positioning of the roof in a north, or east direction which would further minimize the probability of the sealing occurring ([47], [22]). As it can be seen in some cases, this seal between the shingles does not activate due to insufficient exposure to sun and heat, or due to a wind event occurring right after the installation. Thus, the construction was performed under laboratory conditions without sealing. Therefore, four roof systems: “New Edge 2020”, “New Field 2020”, “Aged Edge 2020” and “Aged Field 2020” tested in the WDS can provide a direct correlation of new vs aged performance.

4.3.3 Steps Involved in Resistance Evaluation

To summarize, the following experimental procedure was used for the mock-up testing:

- Install roof mock-up and the dummy in WDS.
- Seal edges between the mock-up and dummy with a self-adhering membrane.
- Install pressure taps and connect them to the Scanivalve DSA module.
- Connect Scanivalve DSA module to the power supply and the laptop via Ethernet cable.
- Check the scanner, power supply, and laptop are all properly connected and the correct units and application files are open.
- Open the selected inlet and install the inlet extension.
- Set up the required equipment and test module.
- Close the doors of the WDS and secure them with the latches.
- Turn on the blower and set the required motor speed in RPM.
- Check that the pressure applied is correct through the readings, and keep running the rpm for 10 minutes before starting to record the data to allow the flow to stabilize.
- Start test, run for 30 minutes then increase the RPM to achieve the next required pressure level.
- RPMs of 1,300 RPM, 1,700 RPM, 2,100 RPM, 2,400 RPM, and 2,800 RPM were tested which correspond to pressures of 0.7 kPa, 1.4 kPa, 2.1 kPa, 2.8 kPa, 3.5 kPa, respectively.

4.3.4 Termination of the Test

The test was carried out until any of the types of damage identified below were noticed or if the fan reached a maximum of 2800 RPM. To determine when the sample tested is considered to have failed, a number of failure modes were identified:

- Failure mode 1: Shingle pulled through from the fastener attachment.
- Failure mode 2: Fastener pulls out from the deck.
- Failure mode 3: Disengagement of the self-sealing strip and permanent movement from the original position.
- Failure mode 4: Shingle breakages.
- Failure mode 5: Permanent curling of shingles.

When the wind flow from the inlet reaches the shingle, an uplift pressure is created on top of the shingle which allows one of the above failure modes to occur if the system is not strong enough to resist. For the roofing industry's practical purpose the wind pressure can also be expressed in terms of speed, by using equation (2.2) described in Chapter 2. Thus, the test pressures 0.7 kPa, 1.4 kPa, 2.1 kPa, 2.8 kPa, 3.5 kPa correspond to airflow speed of 33 m/s, 47 m/s, 57 m/s, 66 m/s, 74 m/s, respectively. However, due to variation in the current experimental procedure and applicability of equation 2.2, the calculated wind speed does not represent the actual wind speed that a roof might experience in the field. Also note that the natural wind has varying intensity, duration, and turbulence [29]. The test method used does not account for the type of exposure and density of the surrounding terrain, gusts, and building height, as described by NBCC. These elements can greatly affect the wind speed acting on the roofing systems.

4.4. Concluding Remarks

This Chapter outlined in detail the procedure and the selected experimental setup to quantify the impact of weathering on the component and system resistance. The importance of each of the experimental tests was demonstrated. The standards followed for each of the component tests were described along with the setup and sample requirements. The quantification of the impact of weathering on the system resistance was achieved by concentrating on the behaviour of the cladding. The experimental setup, mock-ups, and the test protocol followed were outlined along with the classification of different failure modes. During the review process, this study failed to identify any other existing characterizations that impacted the weathering on the system resistance.

Chapter 5: Quantification of Weathering Impact on Resistance – Results and Discussion

5.1 Introduction

The results on the quantification of weathering impact are presented and discussed in two folds, the resistance of the roofing components and the resistance of the roofing systems. The values obtained are presented, analyzed, and compared against standard values wherever applicable. By comparing the results obtained from the investigated new and field-aged shingles for both the components and systems the effect of weathering was determined.

5.2 Quantification of Weathering Impact on Component

5.2.1 Discussion on the Tensile Strength

The results from the tensile strength test for the machine direction (MD) and cross-machine direction (XD) for shingle Type 1 can be seen in Figure 5.1 and Figure 5.2, respectively. The example shown in Figure 4.3 (Chapter 4) can be seen highlighted as a black dot in Figure 5.1, which shows an average of all the ten values obtained for each of the four groups. All of the evaluated properties with standard deviation values can be found in Appendix E. A total of fifteen specimens were tested for each direction (MD and XD). However, the specimens which failed closer to the clamps (less than 25.4 mm) were discarded, as explained in Chapter 4. The values reported are the average of the ten specimens with an acceptable breaking mechanism. The red line in the graphs represents the minimum requirement as stated in CSA A123.5, which is 10.5 kN/m for the MD direction and 7 kN/m for the XD direction [26].

Type 1 shingle when tested new in both 2020 and 2005, passed the minimum criteria required. However, after being naturally aged for 13 years Type 1 shingle "Aged 13 Years" displayed a reduction of 52% in tensile strength in the MD direction (Figure 5.1) and 30% in the XD direction (Figure 5.2). This comparison is based on the value of the "New 2005" shingles when tested in 2005. The MD direction tensile strength values are higher than the XD direction strength values due the machine direction being typically stronger ([64], [65]). The shingle is a composite material (discussed in Chapter 2) therefore the tensile strength can be influenced by both the strength of the fiberglass mat and the asphalt coating. The "Lab 13 Years" displayed a smaller decrease in strength when the same comparison was completed with an 18% reduction in the MD direction and 4% in the XD direction.

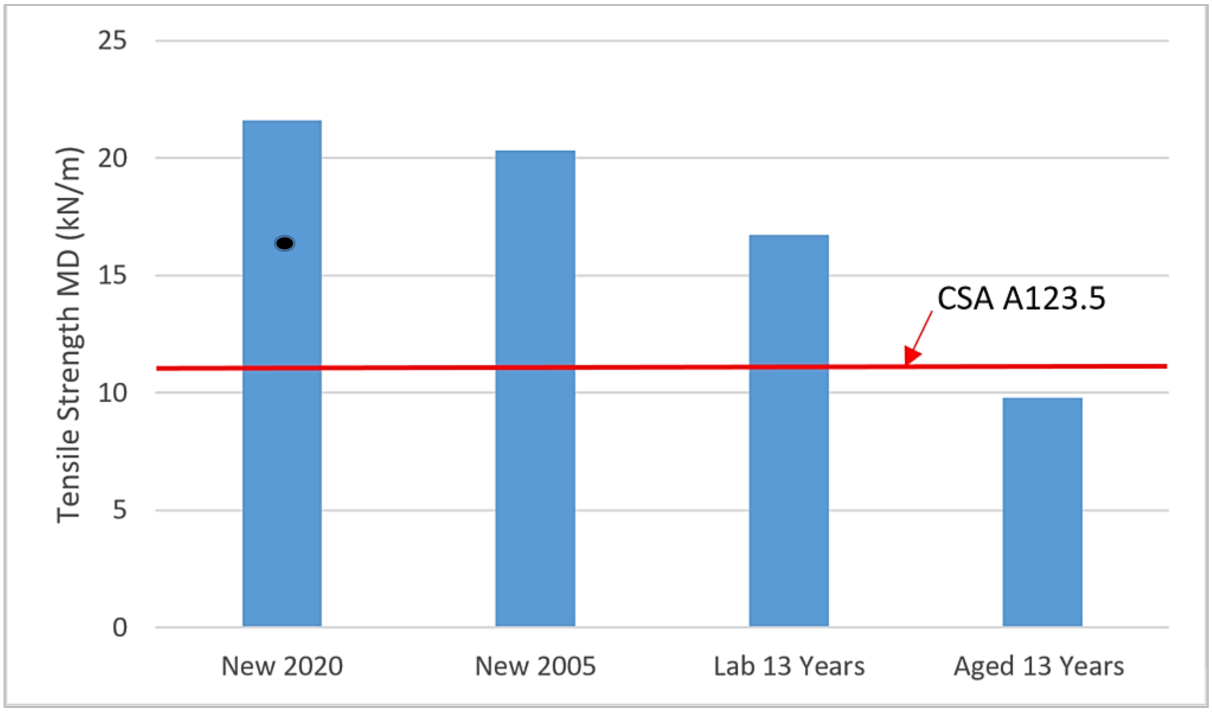


Figure 5.1 MD tensile strength evaluation results for Type 1 shingle

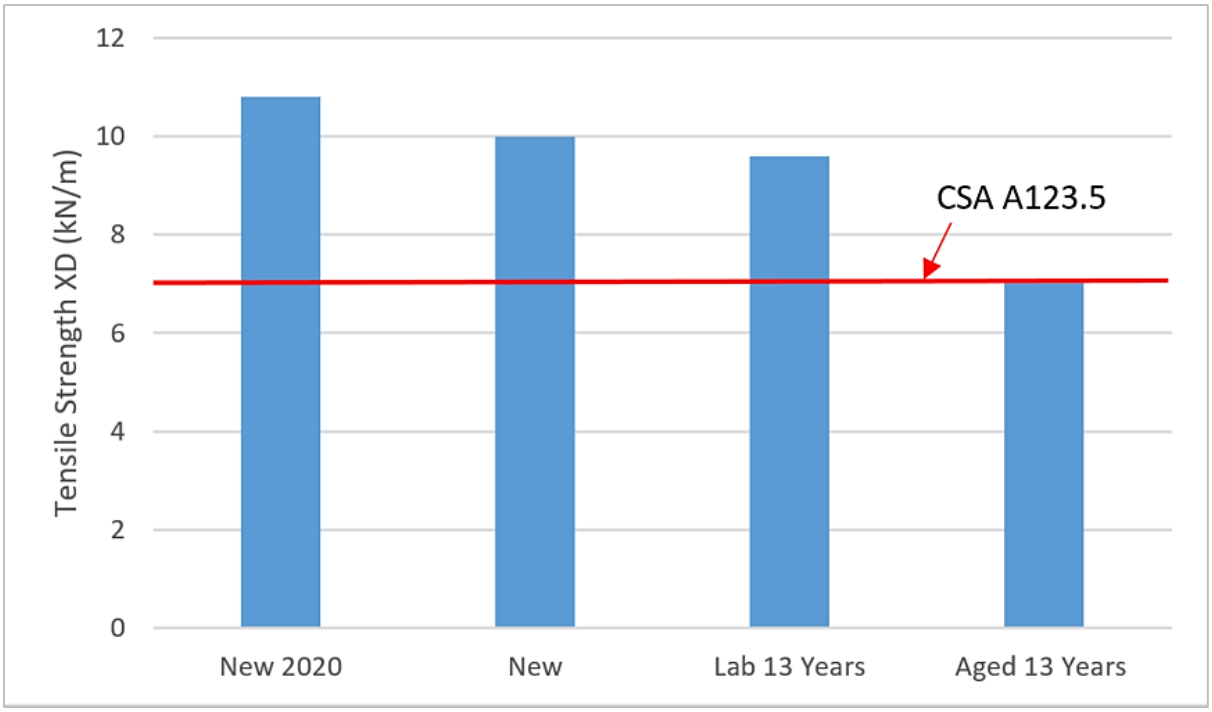


Figure 5.2 XD tensile strength evaluation results for Type 1 shingle

Figure 5.3 shows examples of the failed shingles after the tensile evaluation for "New 2020", "Lab 13 Years", and "Aged 13 Years" for shingle Type 1 in MD direction. The XD direction specimens also had a similar failure mode as that of the MD direction. For all of them, the breaking of the shingles has occurred roughly in the middle of the specimen, due to this being the only acceptable failure mechanism as explained in Chapter 4. For "Aged 13 Years" the breaking appears to be fairly linear which could indicate a more brittle failure due to the shingles having lost their elasticity as a result of weather exposure and thermal oxidation ([66], [67]). The discoloration and the presence of fewer granules were noticed on the older samples especially on the "Aged 13 Years" and "Aged 15 Years" specimens, from Type 1 and Type 2 shingles, respectively.

Figure 5.3. Type 1 tensile strength failure modes for MD direction

For Type 2 the results can be seen in Figure 5.4 and Figure 5.5, for MD and XD directions, respectively. Due to the fact that the Type 2 shingles were no longer being sold in Canada, the “New 2020” for Type 2 could not be evaluated for this study.

Type 2 shingles successfully passed the minimum criteria required by CSA A123.5 [26] when they were tested in 2005. However, after being naturally aged for 15 years Type 2 (“Aged 15 Years”) displayed a reduction of 49% in tensile strength in the MD direction (Figure 5.44) and 37% in the XD direction (Figure 5.55). These values were obtained by comparing “Aged 15 Years” against “New 2005”. Thus, “Aged 15 Years” shingles no longer meet the CSA A123.5 requirements for MD tensile strength, in contrast to the XD tensile specimens which still met the CSA A123.5 requirements. When the same comparison is performed for “Lab 13 Years” for Type 2 a reduction of 31% and 21% is witnessed, for the MD and XD directions, respectively.

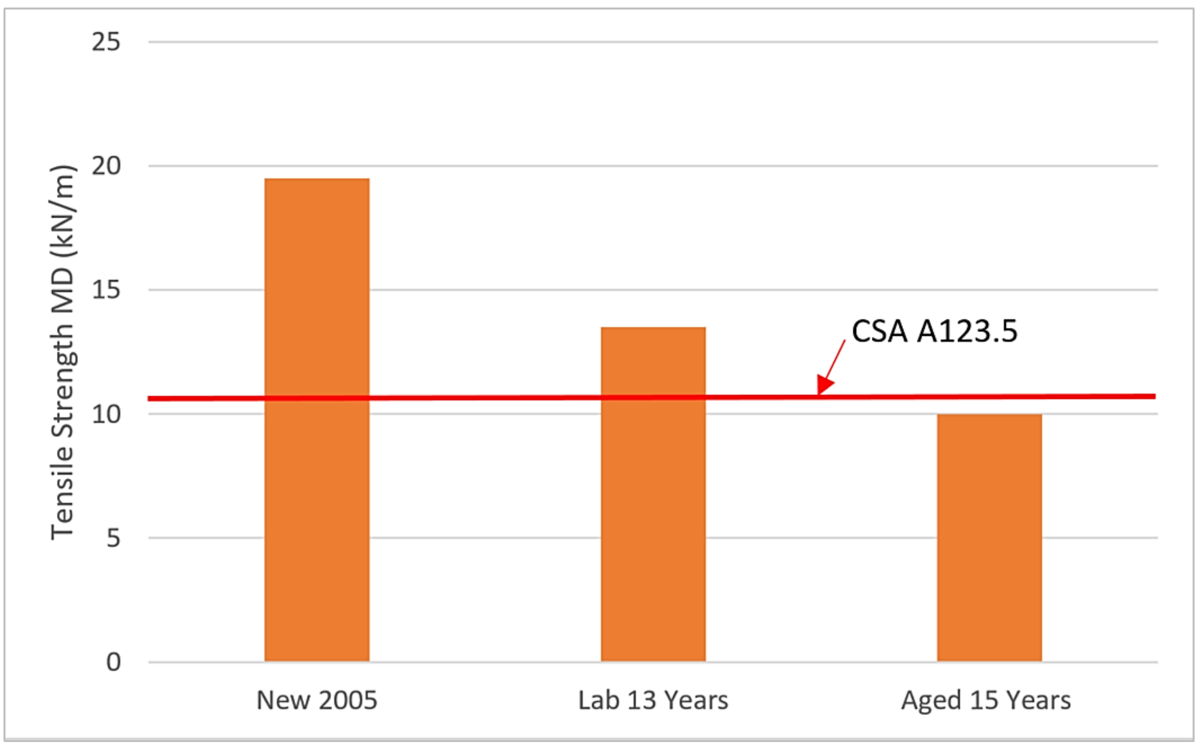


Figure 5.4 MD tensile strength evaluation results for Type 2 shingle

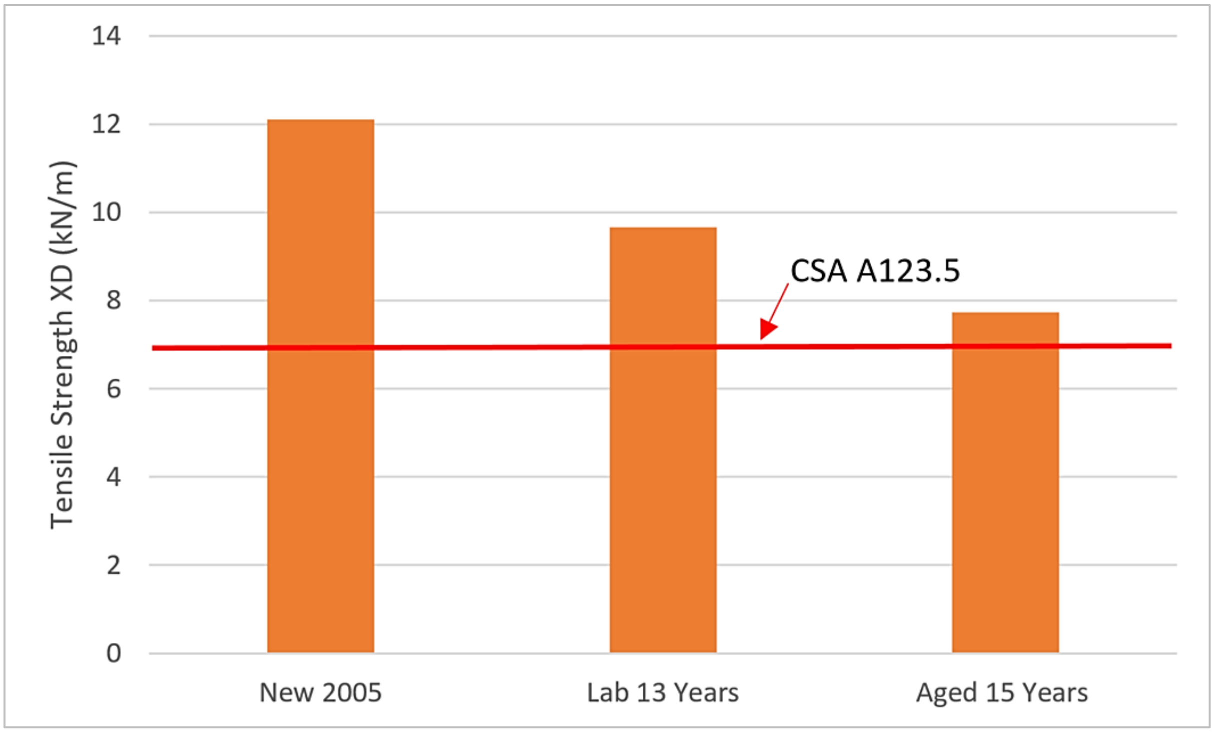


Figure 5.5 XD tensile strength evaluation results for "Type 2" shingle

Figure 5.6 shows an example of the failure for "Lab 13 Years" and "Aged 15 Years" for Type 2 shingles. The "Aged 15 Years" and "Lab 13 Years" displayed similar failures. The "Aged 15 Years" displayed a slightly sharper break than "Lab 13 Years" indicating the brittle nature of the aged shingle due to weather exposure. The break of the "Lab 13 Years" specimens were not as brittle, due to the specimens not experiencing the same levels of thermal oxidation caused by weathering. The similarities amongst the two could be attributed to "Aged 15 Years" still meeting the requirements for the XD direction and nearly meeting them for the MD direction. The discoloration and the presence of fewer granules was noticed on the "Aged 15 Years" samples when compared to the "Lab 13 Years" samples, which did not show significant loss of granules.

Figure 5.6 Type 2 tensile strength failure modes

To conclude, the tensile strength is dependent on the strength of the reinforcement mat used in the shingle. The exposure of the shingle to environmental conditions has contributed to lower tensile strength values. However, the tensile strength depends greatly on the strength of the glass fiber reinforcement mat and should be evaluated through the use of a three-point bending test to determine the flexural modulus. This method will provide more consistent results [68]; however, this test was not incorporated in the text matrix because this is not required by the CSA A123.5.

5.2.2 Discussion of the Tear Strength

The tear strength test is necessary for evaluating the resistance of the shingles for shear failure mechanisms as specified by CSA A123.5. The results for the tear strength can be seen in Figure 5.7 and Figure 5.8 for Type 1 and Type 2 shingles, respectively. The red line represents the minimum value stated in CSA A123.5, which is 1600 g.

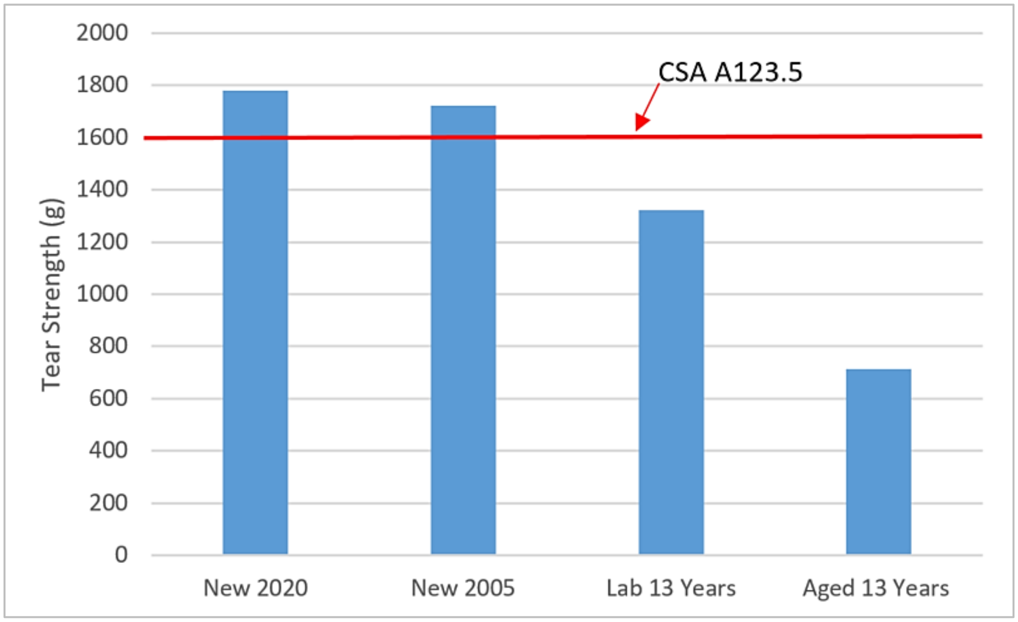


Figure 5.7 Type 1 tear strength evaluation results

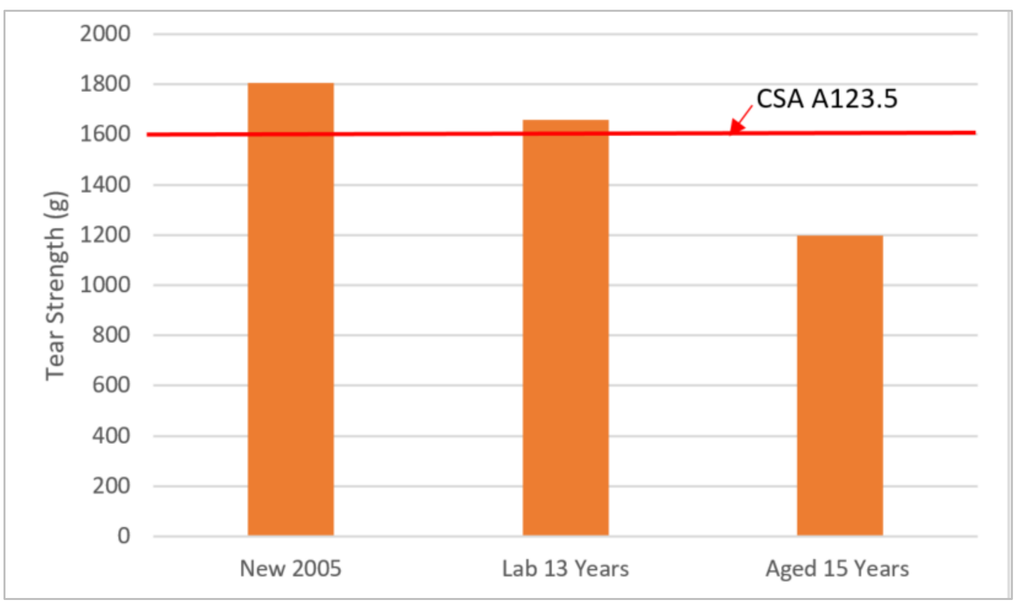


Figure 5.8 Type 2 tear strength evaluation results

Type 1 shingle samples “New 2020” and “New 2005” passed the minimum criteria required by CSA A123.5, as predicted. However, the “Aged 13 Years” Type 1 samples, which were naturally aged in Ottawa’s environmental conditions displayed a reduction of 59% in tear strength (Figure 5.7). Type 2 shingles experienced a tear strength reduction of 34% (Figure 5.8). Tear strength varies depending on the age and the degree of exposure [66] and this is clearly demonstrated by the current study. The tear strength has decreased due to the age of the shingle (“Lab 13 Years” and “Aged 13 Years” and “Aged 15 Years”) but the decrease is larger for the shingles that had longer exposure to environmental conditions. The “Aged 13 Years” and “Aged 15 Years” shingles were obtained from the field mock-ups facing south, therefore they have been exposed to more sunlight and this can lead to shingles having a lower tear strength [66]. The more the shingles are exposed to heat and UV radiation the faster the oxidation of the shingle occurs and it loses its flexibility [67].

Figure 5.9 is an example of how the tear of a sample would occur for Type 1 and Type 2. As it can be seen for “New 2020” the tear appears to be straight down the middle of the shingle showing the flexibility of the shingle due to the asphalt being relatively new. The other samples’ tear appears to deviate from the middle which could indicate the asphalt of the shingle has stiffened. This shows the shingles are no longer able to handle the force generated, thus placing a greater strain on the fiberglass reinforcement of the shingle, leading to the force to find a more pliable path in the shingle.

To conclude the age of the shingle has an effect on tear strength. This is shown by the decrease in tear strength exhibited by “Lab 13 Years” samples. However, the weather exposure of the shingle has the largest effect on the tear strength, expressed by the reduction in strength in the “Aged 13 Years” and “Aged 15 Years” samples. When the age and weathering effects are combined, the shingles’ performance can no longer meet the CSA’s minimum requirements for either Type 1 or Type 2 shingles.

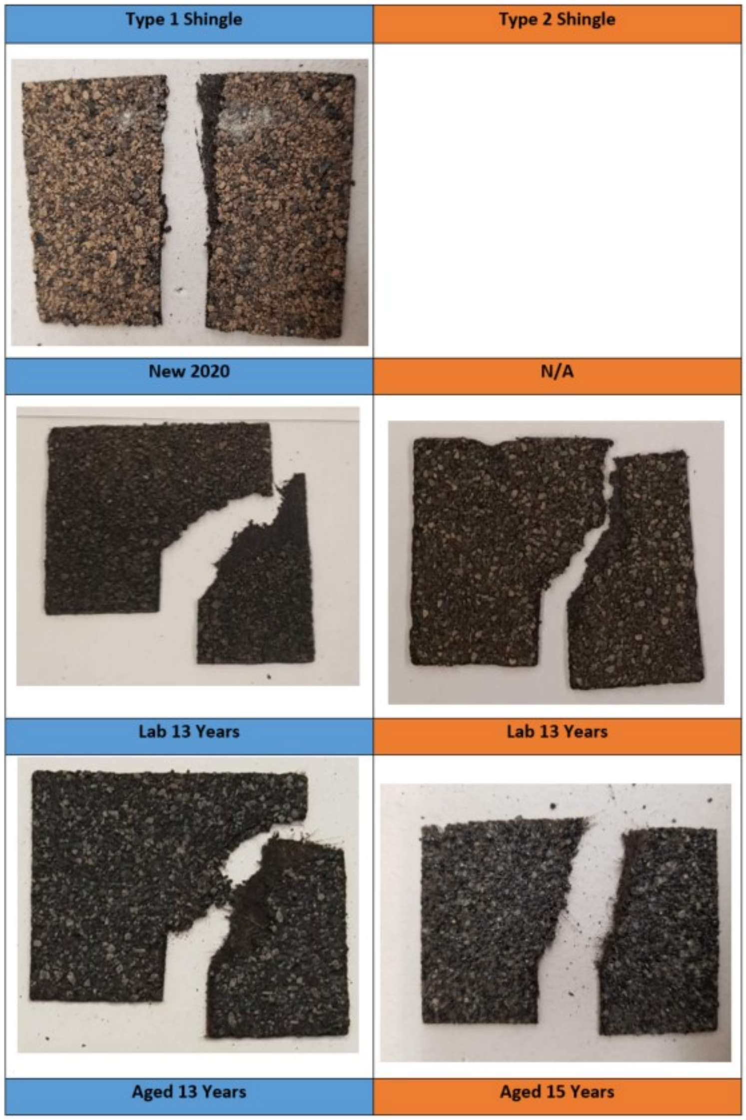


Figure 5.9 Type 1 and Type 2 tear strength failure modes

5.2.3 Discussion of the Fastener Pull-Through

The fastener pull-through experiments were carried out at room temperature and were compared with requirements in ASTM D3462 "*Standard Specification for Asphalt Shingles Made from Glass Felt and Surfaced with Mineral Granules*" [60]. CSA A123.5 requires specimens to be tested at 0°C. The minimum fastener pull-through force recorded for shingle and multi-layer products were respectively 90 N and 135 N. Type 1 shingle is a multi-layered product and the results can be seen in Figure 5.10.

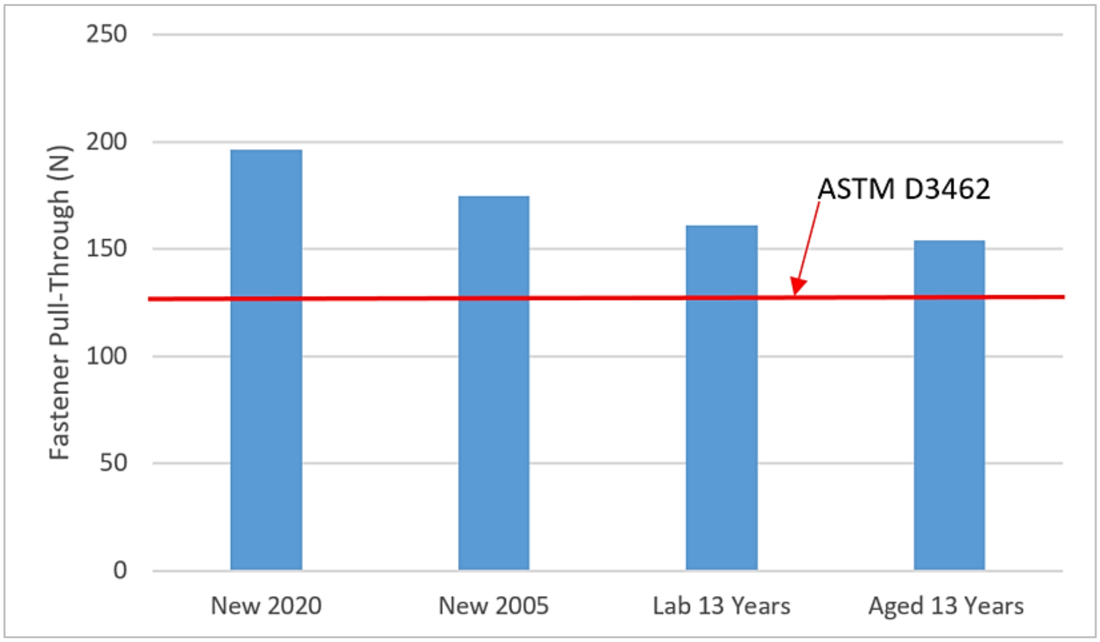


Figure 5.10 Type 1 fastener pull-through resistance evaluation results

All the tested Type 1 shingles continued to meet the requirements of ASTM D3462. However, there is a reduction of 12 % for the fastener pull-through resistance, due to aging displayed by Type 1 shingle ("Aged 13 years"). Figure 5.11 shows the failure modes of the fastener pull-through resistance which occurred for Type 1 "New 2020", and Type 2 "Aged 15 Years" shingles. The failure modes were all the same with no significant difference. Type 2 shingles have a fastener pull-through resistance of 83.2 N. There is no available data for shingle Type 2 when it was tested in 2005 and a comparison could not be reached; however, the aged shingle does not meet the requirements of ASTM D3462.

To conclude the effect of weathering was not as severe for fastener pull-through resistance for Type 1 shingles when compared with the tensile and tear properties. This can be attributed to the fastener pull-through specimens being prepared from the shingle area where the fastener is installed, and this area is covered by the shingle located above. The overlapping shingle placement ensures the fastener area is protected from weathering and water infiltration from the penetration created by the fastener. Therefore, the fastener pull-through specimens were less exposed to the Canadian climate than the tensile and tear specimens, which were prepared using the exposed weathered part of the shingle.

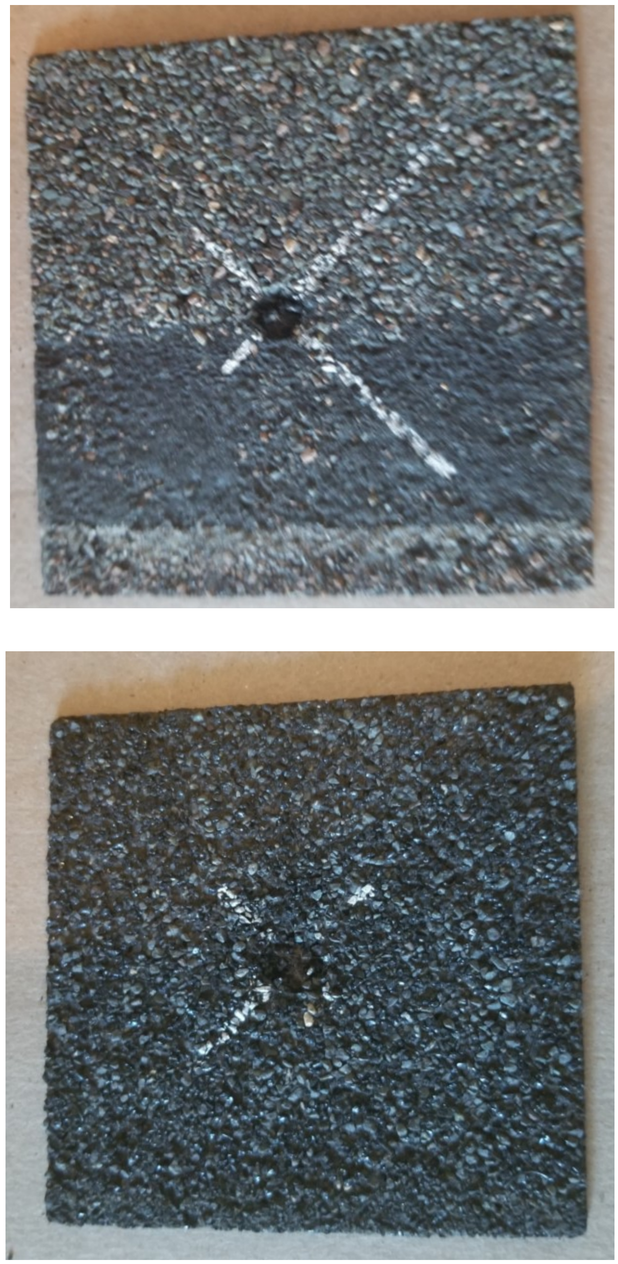


Figure 5.11 Type 1 "New 2020" (Top) and Type 2 "Aged 15 Years" (Bottom) fastener pull-through resistance failure modes

5.2.4 Concluding Remarks on the Component Investigation

When comparing Type 1 and Type 2 shingles, for different aging conditions, it was noticed that there was a significant drop in the strength of the specimens that have been exposed to weather conditions when compared to those stored in the lab. This can be seen in Table 5.1 which summarises the properties corresponding to the component resistance investigation. The data from “New 2005” samples were considered as reference and were used to determine the decrease or increase in strength. In Table 5.1 “New 2005” samples are represented as 100%.

Type 1 shingles “Lab 13 Years” samples exhibited limited decreases for tensile MD and XD and fastener pull-out strengths of 18%, 4%, and 8%, respectively, and a more significant decrease for tear strength of 23%. The tested strengths for Type 1 samples “Aged 13 years”, decreased even further to 52%, 30%, 59%, and 12% for tensile MD and XD, tear and fastener pull-through strengths, respectively. This proved that exposure to natural environmental conditions has a more detrimental effect than just storing the same samples in laboratory conditions. Simultaneously, this proved that the weathering has a significant effect on the resistance of the shingle components, and as expected, “New 2020” samples had higher strengths for all three tests: tensile, tear, and fastener pull-through.

For Type 2 shingles, “Lab 13 Year” samples experienced a small decrease in strength. For tensile MD and XD, and tear this was 31%, 21%, and 8%. For “Aged 15 years”, similar to the behaviour of the aged Type 1 “Aged 13 Years” shingles, the decrease was much higher. The “Aged 15 Years”, shingles showed a reduction of 49%, 37%, and 34% for tensile MD and XD directions and tear strengths, respectively. These reductions are lower than the strength reductions noticed for Type 1 shingles at “Aged 13 Years” conditions. The above comparison of strength reduction from “New 2005” to “Lab 13 Years” and “Aged 15 Years” could not be made for the fastener pull-through, because no evaluation was performed in 2005 for these samples, and thus no reference data existed. Also, the comparison of “New 2005” and “New 2020” for Type 2 shingle could also not be completed due to the shingle no longer being sold in Canada, as explained in section 4.2.4.

The top of the shingle that is exposed to weathering conditions is covered in granules that aim to protect the shingle from ultraviolet radiation. Due to their dark colour shingles absorb significant heat during sun exposure, also accompanied by UV exposure, which is higher during the summer, this is also accompanied by UV exposure. However, according to Terrenzio et al. [38], the effect of UV exposure is minimal compared to the effect of heat. Thus, the performance of the shingle greatly depends on the exposed environment.

Table 5. 2 Summary of component evaluation – strength decrease or increase in percentages

Property	Shingle Type	New 2005 (reference)	Lab 13 Years	Aged 13 Years	Aged 15 Years	New 2020
Tensile Strength	Type 1	100%	-18% MD, -4% XD	-52% MD, -30% XD	N/A	+6% MD, +8% XD
	Type 2	100%	-31% MD, -21%XD	N/A	-49% MD, -37% XD	N/A
Tear Strength	Type 1	100%	-23%	-59%	N/A	+3%
	Type 2	100%	-8%	N/A	-34%	N/A
Fastener Pull-through Strength	Type 1	100%	-8%	-12%	N/A	+12%
	Type 2	N/A	N/A	N/A	N/A	N/A

5.3 Quantification of Weathering Impact on System Resistance

In this section, the system resistance of the evaluated new and aged mock-ups is presented. By testing the resistance of the system as a whole, the interaction of the resistance chain links is examined via observations recorded through each stage of the test.

5.3.1 New Mock-up Investigation

The first system analyzed in this section is the "New Field 2020" roof. In Table 5.2 the average test pressure recorded, at each 30 min time-step, from the pressure taps located on either side of the mock-up are tabulated. It can be seen the pressure has gradually increased from 0 to 3.7 kPa. PT1 is the pressure tap located on the right of the mock-up while PT2 is located on the left. Very small pressure differences were noticed on the two lateral edges of the tested roof sample, as shown in the summary of the measured pressures and percent changes. The low percent change confirms that a constant pressure distribution is registered on either side of the system. Such validation was performed for each mock-up testing to confirm the validation of the reported pressure.

Table 5. 3 Summary of measured pressured at two sides of the Field mock-up.

Time (min)	PT1 (Right) (kPa)	PT2 (left) (kPa)	Percent Change (%)
30	0.739	0.738	0.05
60	1.497	1.471	1.74
90	2.250	2.225	1.09
120	2.798	2.796	0.06
150	3.716	3.712	0.12

A summary of the test steps, pressure, duration of each test step, and remarks can be seen in Table 5.3, while Figure 5.12 captures the performance via photos of each test step along with the description of the event that occurred. The photos are obtained at the level of the testing chamber inlet to capture the impact of the wind directed to the shingles where the uplift most likely began to occur.

At the beginning of the “New Field 2020” test (0.7kPa), there was no fluttering of the shingles observed. However, as time progressed, the fluttering appeared near the end of the time cycle of step one (Figure 5.12, a). During the light fluttering, the uplift force created underneath the shingles is resisted by the system through a combination of fastener attachment and sealed shingle strips. For step 2, at 1.4 kPa, light fluttering of the middle row of shingles occurred towards the middle of the time cycle, and further intensified towards the end of the cycle (Figure 5.12 b). At step 3, for 2.1 kPa, a significant fluttering was observed from the beginning for multiple shingles rows: 8, 10, 11, 12. This behaviour continued for the entire 30 min period of the time cycle tested. This fluttering is an indication that the uplift force created underneath the shingle is weakening the strength of the sealing strip connecting the overlapping rows of shingles. In Step 4, at 2.8 kPa, lifting and curling of the shingles was observed due to the disengagement of the sealant which occurred during the flutter encountered in Step 3. Also at step 4, (Figure 5.12 d) periodic lifting of row 6 of shingles was observed and constant lifting of rows 8 and 12 was noticed. The lifting of row 6 indicates that the uplift force generated was greater than the sealing strip strength.

Table 5. 4 New Field 2020 observation summary

Step	Pressure (kPa)	Duration (min)	Remarks
1	0.7	30	No movement at begin, fluttering near the end
2	1.4	30	Light fluttering right side which intensified
3	2.1	30	Shingle row 8, 10, 11, 12 began to heavily flutter which continued throughout testing
4	2.8	30	Period lifting of row 6. Shingles next to it initiated light fluttering, constant lifting of row 8 and 12.
5	3.5	30	Row 6 started lifting and shingle tore through the fastener; Row 13 started lifting; Shingle row 6 curled. Test considered to have failed.

a) 

Step 1 for "New Field 2020" mock-up testing. No movement was observed throughout this test phase until the end of the 30 minutes when very light fluttering among the shingles was observed.

b) 

Step 2 for "New Field 2020" mock-up. Light fluttering was observed.

c)		**Step 3** for "New Field 2020" mock-up. Lifting of the shingles. Disengagement of the self-sealing strip.
d)		**Step 4** for "New Field 2020" mock-up. Lifting and curling were observed due to the disengagement of the sealant which occurred in Step 3. At this step the curling and lifting would periodically return to the original position
e)		**Step 5** The first shingle row lifted in the picture is minimizing the uplift force experienced by the second lifted shingle row hence its lifting being slightly lower. Test failed due to disengagement of the sealant and a permanent movement from the original position of the shingles was observed.

Figure 5.12 The behaviour of the "New Field 2020" mock-up through the different stages of the test

During step 5 at 3.5 kPa, the test was considered to have failed due to the disengagement of the sealing strip and permanent movement of the shingles from the original position at the beginning of the time cycle (Figure 5.12 e). Therefore, the system’s sustained pressure is 2.8 kPa. However, to gather a better understanding on how the shingles would continue to behave under higher wind speeds the test was continued. The lifting and curling observed in step 4 became more severe and permanent as step 5 progressed. The shingles in row 6 started lifting and curling even more and eventually, causing the shingle near the fastener to tear (Figure 5.13) near the end of step 5, while row 13 shingles started lifting as well. At the end of step 5, the mock-up was observed up close to record the damage incurred in greater detail. Figure 5.13 shows the shingle had begun to be pulled through the fastener, and it can be classified under Failure Mode 1 as explained in Chapter 4. The fastener had begun to be pulled through the shingle (Figure 5.14 a). Under this failure mode, the uplift force created underneath the system was larger than the shingles’ resistance causing the fastener of the shingle to be pulled through from the shingle causing tearing around the fastener/shingle engagement area. This can also be seen in Figure 5.14 (b) where the fastener has been displaced from its original placement on the shingle (Failure mode 2) by a few millimetres.

The system had new shingles and their tear and fastener pull-through resistances, tested as part of the component evaluation (“New 2020”), were above the minimum requirements. Thus the shingles were still intact. However, due to the fact that self-sealing strip did not have enough time and heat exposure to properly adhere to the layer underneath, this caused the uplift of the shingle leading to permanent movement from their original position.

Figure 5.13 End of test observed failure modes (Failure Mode 1) - shingle pulled through the fastener

a) b)

Figure 5.14 End of test observed additional failure modes (Failure Mode 1)- shingle pulled through the fastener (a) and Failure Mode 2- fastener displaced and pulled out from the deck (b)

The second mock-up that was analyzed was "New Edge 2020". In contrast to the "New Field 2020" which represented the field zone of the roof, the current mock-up represents the edge zone of the roof. It was constructed similarly to the "New Field 2020", except for the inclusion of underlayment to represent eave protection found at the edge of the roof. The test conditions were maintained the same as before to have a direct comparison between "Edge" and "Field" systems.

The photographs combined with the observations are presented in a summarized version in Table 5.4 and in Figure 5.15. For step 1, where the test pressure was the lowest, as expected, no indication of shingles movement or fasteners displacement was noticed. For step 2 however, at 1.4 kPa light fluttering among the shingles of row 9 and 10 was recorded. This was followed by even lighter fluttering among shingles found on rows 6 and 7. This fluttering became heavier during test stage 3 when the pressure was increased to 2.1 kPa. Shingles on row 7 were no longer fluttering due to the heavy fluttering being experience by shingles on row 6. During steps 4 and 5 the fluttering and lifting of shingles on rows 6, 8, 9, and 10 continued. This resulted in the shingle from row 6 to curl during step 5.

Table 5. 5 "New Edge 2020" observations summary

Step	Pressure (kPa)	Duration (min)	Remarks
1	0.7	30	No movement throughout.
2	1.4	30	Light fluttering row number 9 and 10, row 6 and 7 very light fluttering.
3	2.1	30	Shingle row 6 8, 9, 10, 11 began to heavy flutter which continued throughout testing.
4	2.8	30	Period lifting of row 6 shingle next to it initiated light fluttering, lifting of row 8 and 9. Row 10 fluttering.
5	3.5	30	Row 6 started lifting along with 9, 10, and 8. Row 13 started lifting and the shingle in row 6 curled. Test failed due to disengagement of the sealant and permanent movement from original position of shingles.

a)		**Step 1** for "New Edge 2020" mock-up. No movement was observed on the mock-up due to the pressure created at this step.
b)		**Step 2** for "New Edge 2020" mock-up. This pressure caused light fluttering among the shingles.
c)		**Step 3** for "New Edge 2020" mock-up. Shingle fluttering.
d)		**Step 4** for "New Edge 2020" mock-up. The edge of the shingle is curling very slightly but returning to its original position.

e) 	**Step 5** for "New Edge 2020" mock-up. Curling of the shingle as well as lifting and its progression can be seen how the lifting and curling increases and causes the lifting of the shingle located above. The test was considered to have failed.

Figure 5.15 The behaviour of the New Edge 2020 mock-up through the different stages of the test

This mock-up failed at 3.5 kPa, due to the rows of shingles experiencing severe lifting (Failure Mode 3) and a permanent curl of the shingles (Failure Mode 4) presented in Figure 5.16. However, no shingle pull through or fastener pull out were observed and the only mode of failures observed were adhesive disengagement and permanent movement from the original position (Failure Mode 3) and permanent curl of the shingle (Failure Mode 4).

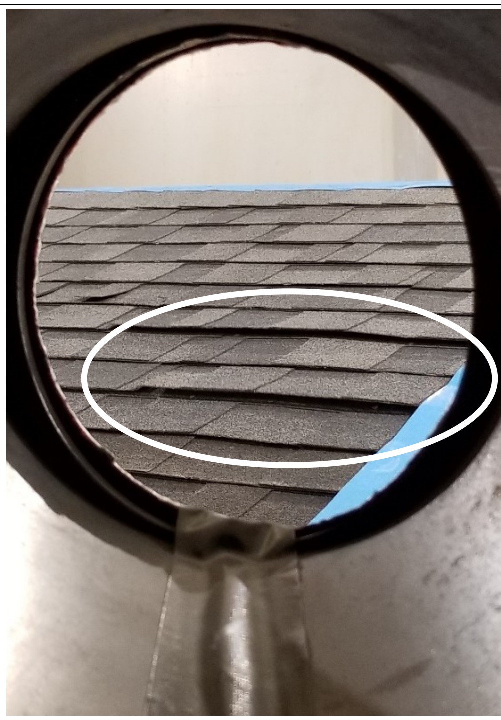
Figure 5.16 Failure mode for the tested "New Edge 2020" mock-up

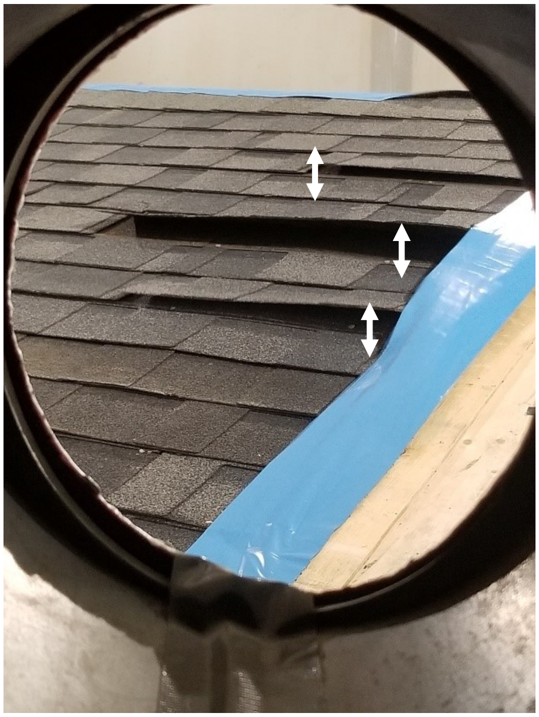
5.3.2 Aged Mock-up Evaluation

To quantify the effect of the roofing system aging under natural environmental conditions, two roof mock-ups were exposed to weather conditions in Ottawa for the past 15 years at the NRC Montreal Rd. Campus. The field and edge mock-ups that have been aged for 15 years were respectively labelled "Aged Field 2005" and "Aged Edge 2005" based on the year of installation. These were transported to UOttawa campus and were tested under the same experimental conditions used for the "New Edge 2020" and "New Field 2020" mock-ups. When the testing began there was no movement among the shingles, however, this changed during step 2. The pressure increase caused some fluttering among the shingles during step 3. The shingles located in rows 6 8, 9, 10, 11 began to react by alternating between heavy flutter and lifting, thus indicating that the sealant was no longer engaged in preventing the wind uplift of the shingles. In step 4 the behaviour witnessed in step 3 continued but became stronger due to the increase in pressure. This ultimately led to the breaking of the shingle located in row 8 (Failure Mode 3). However, the test was continued to gather more information and in step 5 the degree of failure of shingle in row 8 increased. Also, the shingle in row 6 broke. The test was continued and the shingles were blown off near the end of step 5, fasteners were partially pulled out and a few shingles broke (Figure 5.17 e). Following the same procedure of evaluating, the observations recorded throughout "Aged Field 2005" testing, the main findings were summarized in Table 5.5 and Figure 5.17.

Table 5. 6 "Aged Field 2005" observation summary

Step	Pressure (kPa)	Duration (min)	Remarks
1	0.7	30	No movement at all
2	1.4	30	Light fluttering
3	2.1	30	Shingle rows 6 8, 9, 10, 11 began to heavily flutter and lifting which continued throughout testing.
4	2.8	30	Test considered to be failed due to lifting row 6,7,8,9,10,11 as a result of the disengagement of the sealant. Shingles in rows 6, 8, and 11 experienced periods of heavier prolonged lifting. This led to shingle in row 8 to break with 6 min remaining.
5	3.5	30	Shingle rows 6 broke and shingle row 8's failure increased.

a) 	**Step 1** No movement of the shingles observed.
b) 	**Step 2** for “Aged Field 2005” mock-up. Light fluttering

c) 	**Step 3** "Aged Field 2005" mock-up showing lifting of the shingle.
d) 	**Step 4** "Aged Field 2005" mock-up. Broken shingle. Failure modes 3 and 4. Test failed.

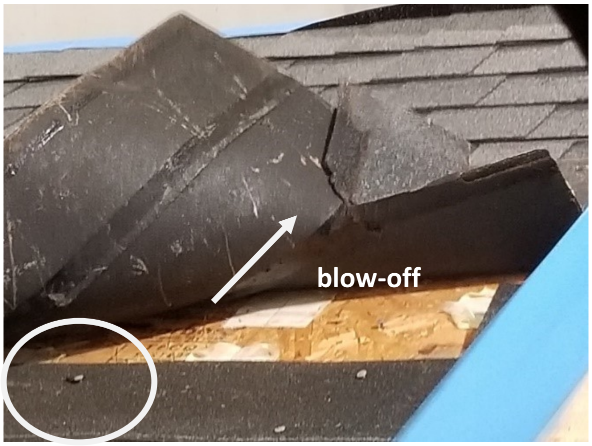
e) 	**Step 5**. Broken shingles rows 6 and 8. The fastener pulled out (Failure Mode 2). The shingles have pulled through the fastener and the sealing strip is no longer engaged therefore there are Failure Mode 1,3, 5.

Figure 5.17 The behaviour of the "Aged Field 2005" mock-up through the different stages of the test

Figure 5.18 Field zone failure mode observed at the end of the test (Failure Mode 1,3,4)

If the failure in Figure 5.17 (e) occurred in the field this would expose the deck to water infiltration. In Figure 5.18 (left) the permanent lifting of the shingle can be seen along with a broken shingle, this is identified as Failure Mode 4 and 3. For this to occur the sealant had to be disengaged and the shingle had to permanently moved from its original position and part of it is broken. In Figure 5.18 (right) the completely broken shingles (Failure Mode 4) can be seen along with missing fasteners, while in Figure 5.19 the fastener partially pulled out from the deck is observed (Failure Mode 2).

Figure 5.19 Fastener lifted from the shingle (Failure Mode 2) of the field zone mock-up

The “Aged Edge 2005” mock-up was considered to have failed at 2.1 kPa. This mock-up was used to establish the test protocol as discussed in Chapter 4 and was tested under both test protocols and for this reason, its results were represented with an asterisk in Figure 5.23. The mock-up failure registered at the end of the test can be seen in Figure 5.20 where the shingles have been pulled through the fasteners (Failure Mode 1) and the fastener itself has been partially pulled out from the deck (Failure Mode 2). The results are included in the concluding remarks. In Figure 5.20 the underlayment can be seen as it is being torn and in the process of being blown off after it was exposed as a result of the shingles being blown off. In Figure 5.21 the fastener can be seen pulled out from the deck (Failure Mode 2). This was caused because the uplift force generated under the shingle was so high that it caused the fastener to be displaced. A similar observation can be seen, in Figure 5.22; however, tearing of the shingle around the fastener can also be seen.

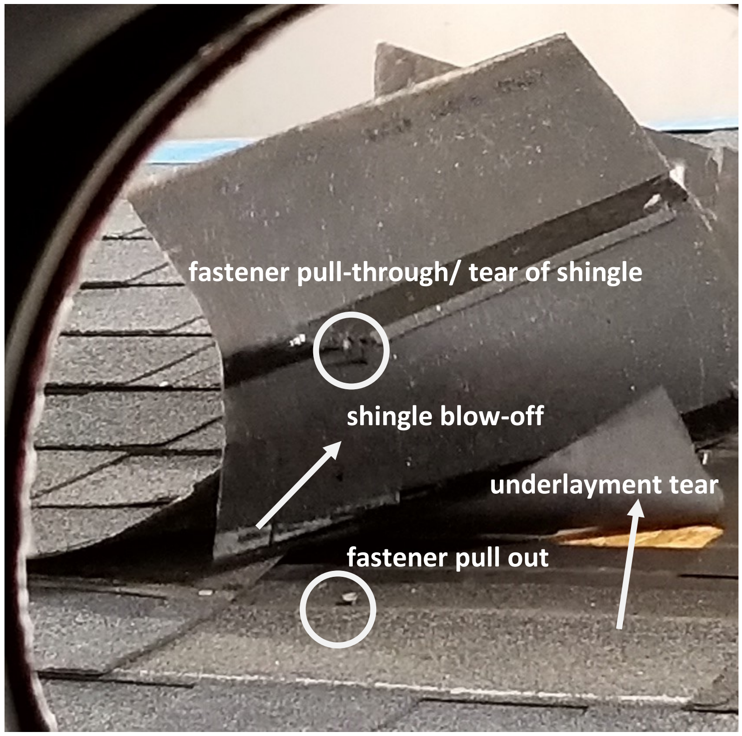


Figure 5.20 Shingle pulled through the fastener and the fastener lifted from the deck

(Failure Mode 1, 2, 3)

Figure 5.21 Fastener pulled out from deck and shingle tearing around fastener (Failure Mode2)

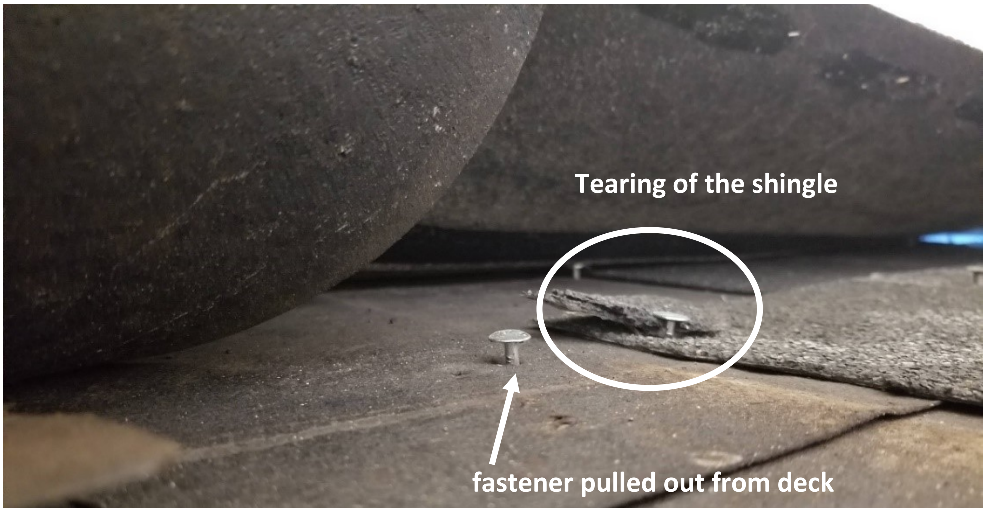
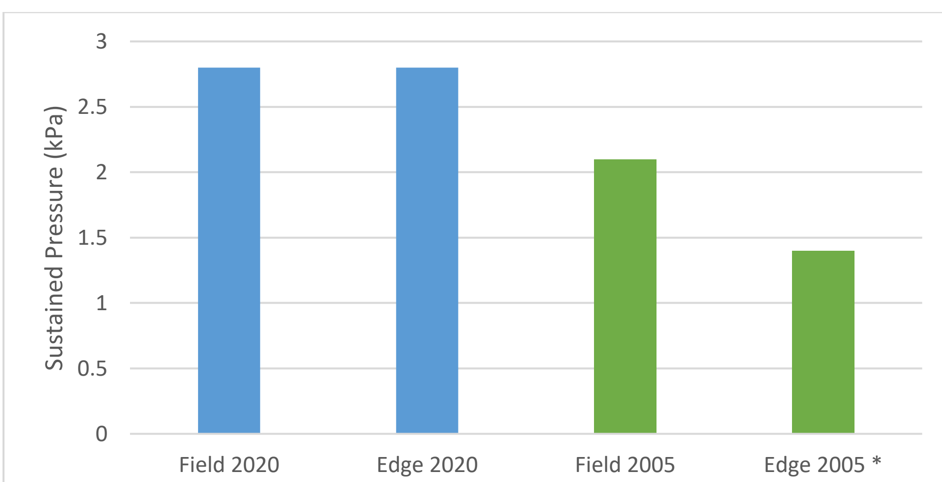


Figure 5.22 Fastener pulled out from deck and shingle tearing around fastener (Failure Mode 2, 4)

5.3.3. Concluding Remarks on the System Investigation

Two new roof mock-ups, “New Field 2020” and “New Edge 2020” were constructed and tested to determine the sustained wind pressure. Two aged mock-ups, “Aged Field 2005” and “Aged Edge 2005” were also tested. A comparison of the sustained pressures of all mock-ups are shown in Figure 5.23. As expected, the results obtained for the new mock-ups performed better than the aged mock-ups. To quantify the resistance reduction, the new mock-ups “New Field 2020” and “New Edge 2020” were used as a reference point. For the “Aged Field 2005” and “Aged Edge 2005”, a decrease in pressure the systems could resist of 33% and 50% was observed, respectively.



* *Edge 2005 mock-up was used to establish the test protocol as discussed in Chapter 4 and was tested under both test protocols.*

Figure 5.23. Comparison of the sustained pressure

Most of the failures were initiated due to disengagement of the adhesive, which propagated further system failures when the test was continued at higher wind-induced pressures. This highlights the importance of properly sealing overlapping shingle rows. For the new mock-ups “New Field 2020” and “New Edge 2020”, even though the adhesive failed, the fastener pull-through resistance and tear strength were higher than the aged mock-ups “Aged Field 2005” and “Aged Edge 2005”, which ensured the systems continued its resistance to wind uplift.

5.4 Concluding Remarks

Currently, there are no baseline performance criteria to be met for shingles after extended exposure to weather conditions. Also, there are no criteria stipulating the minimum strength of the overlap strength of shingles, which is critical in determining how well the shingles will perform against wind uplift as a system.

The current research identified the major gap in the existing roofing component evaluation process by accounting for the influence of climate change weathering on the durability performance of the shingles. Chapter 2 reviewed the current methods used for the component strength evaluation and identified that there is a gap in the determination of the minimum overlap strength of shingles, the peel strength of the eave protection, and the fastener pull-through of the underlayment. Therefore, the existing standards need to be revised to account for weathering impact of climate change on the performance and durability of the shingles. This will aid in the design of more durable residential roofs that will continue to perform under the new climate change weather conditions.

The determination of the tensile strength, tear strength, and fastener pull-through strength was achieved for two types of shingles, Type 1 and Type 2. Both types of shingles were initially tested in 2005, then after being naturally aged for 13 years (Type 1) and 15 years (Type 2) in outdoor conditions in Ottawa, as well as after being stored in the lab for 13 years. The samples were tested for tensile, tear, and fastener pull-through strengths. The objective of this study was to determine if the shingles still met the requirements after being aged for less than their advertised warranty.

Both new shingles which were manufactured in 2020 and 2005, met the minimum requirements outlined in the CSA A123.5 and ASTM D3462. Type 1 shingles were advertised as 30-year life span shingles. This gives false expectations to the Canadian homeowners that the shingles will continue to meet the performance requirements for 30 years of service life. However, after 13 and 15 years of natural aging, this was not the case. The component tests showed that the shingles no longer meet all the performance criteria. When comparing the “New 2020” to the “New 2005” for Type 1 shingle, it can be seen that the industry has enhanced the performance of the shingles. The same shingle type (Type 1) in 2020 yielded better performance properties than the new shingle stored in the lab since 2005. As it was stated in Chapter 2 the disconnection between the shingle warranty and its service life needs to be resolved. This can minimize the discontent that homeowners face when their shingles begin exhibiting problems before the advertised warranty is met. This disappointment of homeowners can be minimized by evaluating the shingle’s durability considering the climate change weathering influence and correlating to the life cycle performance.

Experimental evaluation of the system resistance of four roof mock-ups was also performed to establish the effect of weathering on a system level. The newly constructed mock-ups “New Field 2020” and “New Edge 2020” performed better than the aged mock-ups “Aged Field 2005” and “Aged Edge 2005”.

Existing resistance evaluation standards that are referred to in the current codes do not take into account the effect of different climates that shingles and asphalt roofing systems are exposed to during their design life. This is reinforced by the post-storm reports, which identified that roofing systems over six years in service showed significantly higher damages than those falling in the zero to four-year range [69]. These observations are validated by the present thesis when the results of the sustained pressures were compared between mock-ups of 2005 and 2020. Based on this extensive investigation, code change requests for the NBCC can be proposed for the inclusion of requirements for weather-exposed properties.

Chapter 6: Implementation of Holistic Approach for Steep Slope Residential Roof Cladding

6.1 Introduction

Chapter 3 presented the Holistic Approach (HA) for increasing the performance of roofing systems during their service life. HA specifies three requirements: load, resistance, and installation. The HA requirements were recently implemented in the commercial roofing sector. This was accomplished as part of the Government of Canada's climate adaptation initiative. National Research Council of Canada (NRC) recently developed a standard, *"Canadian Standard Association (CSA) A123.26 – Performance Requirements for Climate Resilience of Low Slope Membrane Roofing Systems."* CSA A123.26 is intended to be used by building designers, building owners, building code officials, product manufacturers, and installers when developing low slope membrane roofs (LSMR) for Canadian climate adaptation. Being part of the LSMR's initiative, the current research program is motivated to adopt a similar approach for steep slope residential roofs. This chapter will present and discuss the overall road map for adapting the HA for steep slope residential roof cladding, as well as the accomplishments achieved by the current study.

First, this chapter will provide the load requirements by presenting the newly developed web tool, *"Climate-RCI*". The web tool is developed as part of the CSA A123.26 by the author and it calculates future climate conditions such that it can be further used in the load calculation of the NBCC. Second, the resistance calculations are presented based on the LRFD procedure. Towards the development of a new standard for the steep slope application (similar to the CSA A123.26), installation guidelines are summarized as the third segment of this chapter. The ease of implementation of the present HA is highlighted by taking a case study.

For the case study, a building located in Ottawa is selected with the following overall dimensions: Height = 14 m, Width = 10 m, and Length = 16 m. This building has a steep slope with an 18-degree inclination angle and it is assumed that the roof needs to be replaced.

6.2 Determination of Design Load

The importance of incorporating future climatic data in the design was presented in Chapter 3, along with the framework of how this can be implemented with the web tool “Climate-RCI”. The “Climate-RCI” web tool was built with two integrated modules: the first module uses the data provided by Environment and Climate Change Canada. Using the projected changes made available by ECCC, the future projected values were determined for wind, rain, and temperature. For a given city, “Climate-RCI” estimates the severity of the climate change for three weather elements: wind, rain, and temperature. The second module provides the wind, rain, and temperature design parameters for roofs. This new web tool can assist engineers in the design process and it can be accessed at (https://nrc.canada.ca/en/research-development/products-services/software-applications/climate-rci/). Screenshots of the “Climate-RCI” steps are shown in Figure 6.1 and the descriptions of the steps are listed below:

- **Step a: Building Location**. The user is prompted to enter the city and the province where the building is located (Figure 6.1 a).
- **Step b: Global Temperature Increase**. The user is given the option to continue with the recommended 2℃ increase. The user can, apart from 2℃, select other temperature increases if interested (Figure 6.1 b). The 2℃ global warming temperature increase was considered by default because that degree of global warming, as explained in detail in Chapter 3, is expected to occur over the next 30 years. This time duration overlaps with the design life of roofs which ranges from 15 to 20 years.
- **Step c: Climate Severity and Design Parameters**. The user is presented with a summary of the location, the climate severity classification, and the reference wind pressure (hourly 1/50) (Figure 6.1 c).

The final result of the Climate-RCI web tool is the severity classification and the design parameters. For the current case study, the “Wind” weather element for Ottawa’s climate severity falls under the “Normal” classification. As explained in Chapter 3, the “Normal” severity implies that the change in the climatic wind load from the current value would be less than 5%.

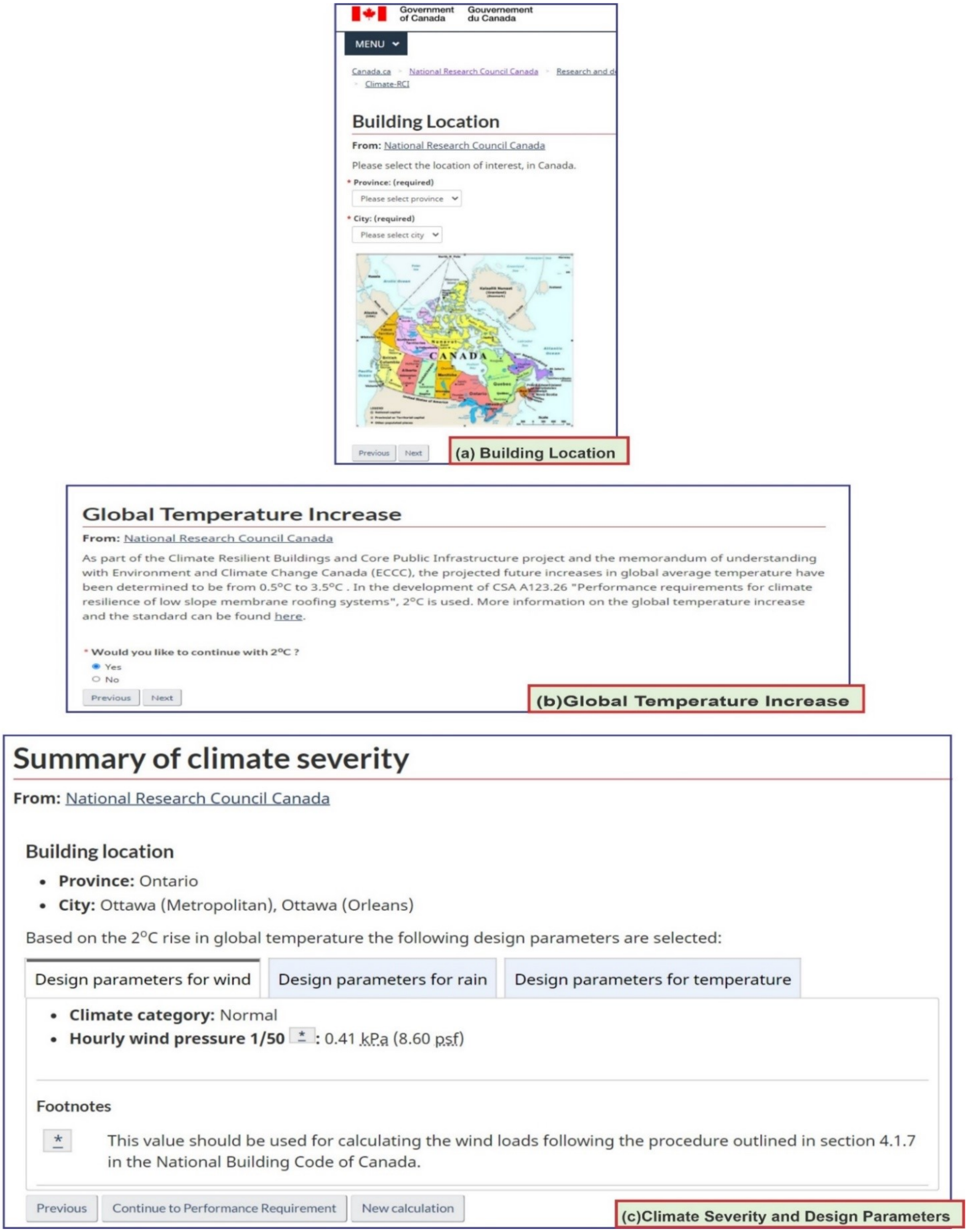


Figure 6. 1 Screenshots of "Climate-RCI"

Therefore the reference wind pressure (based on the projected climatic condition) obtained from the "Climate-RCI" web tool is used to calculate the wind load on the roof cladding, following the procedure outlined in the 2015 NBCC (section 4.1.7). For the case study building, the design wind pressures are: Edge Zone and Field Zones are respectively 1.80 and 1.28 kPa.

6.2 Evaluation of Resistance

The second step of the HA Approach is evaluating the resistance. To illustrate this, sustained pressures from the mock-ups tested and reported in Chapter 5 are used. The Field 2020 and Edge 2020 mock-ups are constructed based on components available in the market today that represent the field and edge zones respectively in the construction practice. Both mock-ups had a sustained pressure of 2.8 kPa. This value obtained from the experimental evaluation needs to be multiplied by a resistance factor to obtain the resistance of the system for the LRFD. As outlined in Chapter 3, the resistance factor is identified as a missing link for the asphalt shingle roofs. Determination of a suitable resistance factor requires extensive experimental evaluations, following a similar approach used for commercial roofs. For the present case study illustration, a resistance factor with a value of 0.65 that was determined for the commercial roofs was used.

When multiplying the sustained pressure of 2.8 kPa with a resistance factor of 0.65, the factored resistance of 1.82 kPa is achieved. The design validations are shown in Table 6.1. As it can be seen, the factored resistance is higher than the factored load derived both from the current NBCC reference wind pressure and the future projected reference wind pressure. This is valid for both the field and edge zones of the roof. In conclusion, the new tested systems installed with the respective components are suitable to be installed for the Ottawa case study and satisfy the LRFD approach.

Table 6. 1 Design validation of the tested system

Roof Zone	Current NBCC	Climate-RCI	Factored Resistance
Field	-1.25 kPa	-1.28 kPa	-1.82 kPa
Edge	-1.76 kPa	-1.80 kPa	-1.82 kPa

As presented in Figure 6.1, the case study city (Ottawa) falls under "Normal" wind conditions, which resulted in only a minimal increase in the overall uplift pressure. Table 6.2 examines to see if the same tested roof mock-up can be suitable to be installed in another city, with a severe or extreme wind climate severity classification. For this case, the same building is assumed to be located in Bella Coola, BC. The current reference wind pressure for Bella Coola is 0.39 kPa. "Climate-RCI" classifies Bella Coola as "Extreme" with a projected reference wind pressure of 0.45 kPa. The design validations are shown in Table 6.2. In this scenario, there are two observations. First, the factored resistance is higher than the current NBCC's factored loads. Second, the wind pressure calculated accounting for future climatic projections is higher than the factored resistance in the case of the edge zone. Thus the roof would not be able to withstand the expected wind loads during its service life of 15 to 20 years. This roof system would not be suitable to be installed in the city of Bella Coola. The resistance of the system would need to be increased such that the factored resistance would be equal to, or higher than the factored load.

Table 6. 2 Design validation of the tested system – extreme example

Roof Zone	Current NBCC	Climate-RCI	Factored Resistance
Field	-1.19 kPa	-1.37 kPa	-1.82 kPa
Edge	- 1.67 kPa	-1.93 kPa	-1.82 kPa

By using the same case study, but concentrating on the results obtained for the aged roof mock-up it can be seen if the aged systems would still be suitable to resist the wind loads. The factored resistance can be seen compared to the current and projected values in Table 6.3. The aged systems no longer meet the criteria of the factored resistance being higher than the factored load for the edge of the roof.

Table 6. 3 Design validation of the tested system – aged system (Ottawa)

Roof Zone	Current NBCC	Climate-RCI	Factored Resistance
Field	-1.25 kPa	-1.28 kPa	-1.37 kPa (Aged 2005)
Edge	-1.76 kPa	-1.80 kPa	-0.91 kPa (Aged 2005)

If the aged system was located in Bella Coola, the values can be seen summarized in Table 6.4. Again similar to the Ottawa example the system is not suitable for the edge, because the factored resistance is lower than the factored load using both the current and future projected reference wind pressure. Therefore if the use of the aged roof systems continued in either Ottawa or Bella Coola, this would increase the probability of failure due to the factored resistance of the edge of the roof no longer being higher than the factored load.

Table 6. 4 Design validation of the tested system – aged system (Bella Coola)

Roof Zone	Current NBCC	Climate-RCI	Factored Resistance
Field	-1.19 kPa	-1.37 kPa	-1.37 kPa (Aged 2005)
Edge	- 1.67 kPa	-1.93 kPa	-0.91 kPa (Aged 2005)

By achieving the first and second steps of the Holistic Approach, overlapping between the Load and Resistance bubbles was created, however, this does not take installation into account (Figure 6.2). The load and resistance can be determined correctly and the resistance can be higher than the load, however, if the installation is not implemented correctly the roof will have a higher probability of failure. This translates into no intersection between the third bubble, Installation, and the Load and Resistance bubbles.

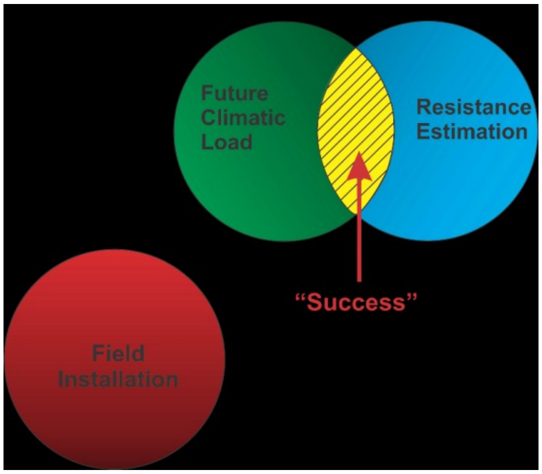


Figure 6. 2 Partial completion of the Holistic Approach

6.3 Guidelines for Quality Assurance Metrics

As stated in Chapter 2, installation is the missing link identified by industry through the post-storm reports [57]. This link is attributed to the majority of failures. To account for the installation requirements, a set of quality assurance metrics guidelines were created. These Guidelines for Quality Assurance Metrics (QAM) were compiled based on comprehensive consultations with industry partners, to identify the best practice currently used in the roofing industry. The QAM Guidelines provide recommendations for steep slope asphalt shingle roofs when identified as "Severe" or "Extreme" based on the climate severity. When a roof performance is assigned to the "Normal" classification that would lead to the minimum requirements outlined in the 2015 NBCC to be followed for the design and construction of steep slope residential roofs. This will be the scenario for the present case study building located in Ottawa.

When a roof performance is assigned the "Severe" classification that would lead to the "Severe" requirements outlined in the guidelines below to be followed for the design and construction. These requirements should be implemented in addition to those imposed by the 2015 NBCC. When a roof performance is assigned to the "Extreme" classification, then that would lead to the "Extreme" requirements to be implemented. The "Extreme" scenario can be viewed as "Gold" level where additional requirements are appended to the "Severe" requirements and the 2015 NBCC design procedures. This is shown schematically in Figure 6.3.

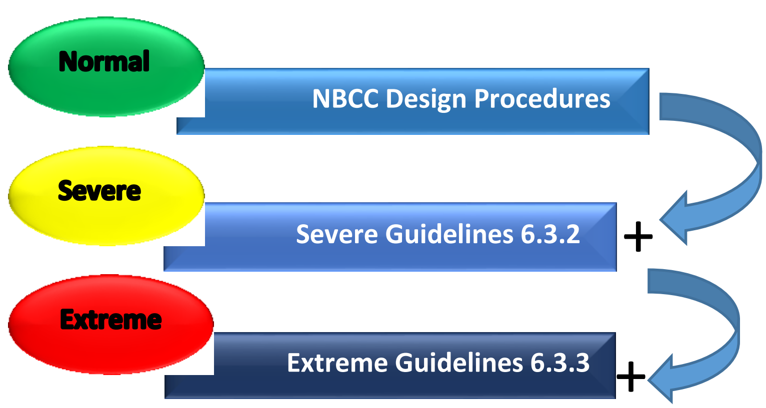


Figure 6. 3 Implementation of the requirements

The QAM guidelines are intended to be used by building designers, building code officials, product manufacturers, and installers. These guidelines do not address the climatic adaptation requirements for the structural support (deck) of the roofing systems.

6.3.1 Guidelines for Wind – Normal classification

- The wind loading requirements shall be determined in accordance with the *National Building Code of Canada,* Part 4, Part 9, and “Climate-RCI”. The reference pressure output from the “Climate-RCI”, which is based on the future climatic projections, shall be used in accordance with NBCC subsection 4.1.7.
- The requirements specified in NBCC shall be followed for design and construction.

6.3.2 Guidelines for Wind – Severe classification

- The wind loading requirements shall be determined in accordance with the *National Building Code of Canada,* Part 4, Part 9, and “Climate-RCI”. The reference pressure output from the “Climate-RCI”, which is based on the future climatic projections, shall be used in accordance with 2015 NBCC subsection 4.1.7.
- Shingles located at the eaves and edge of the roof must be installed using a minimum of six nails per shingle or in accordance with the manufacturer’s specifications for high wind regions.
- Shingles shall be installed above a minimum temperature of 10℃ or as indicated by the manufactures’ specifications.
- Asphalt roofing cement adhesive must be applied between the shingles on three equally spaced locations on each tab of the shingles located along the roof edges.
- Asphalt shingles must be fastened with corrosion-resistant fasteners with a minimum length of 19 mm.
- When installing shingles on top of the metal drip edge, two drops of asphalt roof cement, each 25 mm diameter drop must be applied on the underneath the shingles, metal drip edges, and on top of the shingles before the shingle is fastened.
- Shingles with 30% higher fastener pull-through resistance than the requirements (124 N) specified in the CSA A123.5 shall be installed.

6.3.3 Guidelines for Wind – Extreme classification

- Asphalt roofing cement adhesive must be applied between the shingles at three equally spaced locations on each tab of all the shingles. The adhesive must be applied in a quarter size amount to all specified locations.
- Asphalt shingles must be installed using six corrosion-resistant nails per shingle on all areas of the roof.
- Shingles with 50% higher fastener pull-through resistance than the requirements (124 N) specified in the CSA A123.5 shall be installed.
- During the roof application, an independent inspector shall be engaged to inspect the installation process once at the beginning, middle, and end of the installation to document that the work conforms to the presented guidelines, the applicable governing building code, and any quality assurance metrics established by the design specifications. The independent inspector shall document any non-conforming work and verify that the corrections made by the responsible trade conform to those parameters.

The guidelines provide guidance on how to perform the installation for different severity zones and also they provide some quality assurance measures required to be met. The implementation of these guidelines allows the third bubble, Installation to intersect with the existing bubbles of Load and Resistance (Figure 6.4). This allows the completion of the Holistic Approach and overlapping of all three Load, Resistance, and Installation.

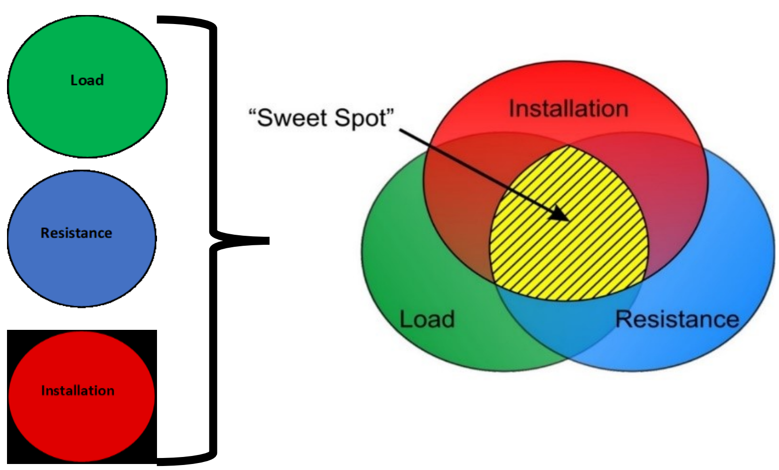


Figure 6. 4 Full completion of the Holistic Approach

6.4. Concluding Remarks

By taking a case study building located in Ottawa, this chapter highlighted the process involved in the implementation of the present holistic approach. The design load values for the roof were determined using current and future climate projections. As part of the LRFD validation process, the factored loads were compared with the factored resistances. To address the third step of the Holistic Approach, which is the installation, a set of quality assurance metrics were proposed in consultation with the best practices used in the roofing industry. The current research established how the future climatic loads can be incorporated in the design of roofing systems, while also demonstrating how the resistance for the steep slope residential roofs can be determined as well as providing quality assurance metrics. This represents the first phase of the ongoing initiative to increase the performance and resiliency of steep slope roofs in light of climate change. The elements that require further investigations are mentioned here and are discussed in more detail in Chapter 7.

Chapter 7 Conclusions and Recommendations for Future Work

7.1 Conclusions

The current research identified and quantified the impact of weather exposure on asphalt shingle roofs, through component and system evaluation. A total of 155 samples of aged and new roof shingles were tested for the component resistance evaluation. One of the main findings from the testing of specimens was that exposure to environmental conditions has a more detrimental effect on the strength than the samples stored in laboratory conditions. In some cases, the minimum requirements outlined in CSA A123.5 were no longer met after 13 or 15 years of weather exposure. This finding highlighted the need for the effect of weathering to be included as mandatory requirements for a comprehensive design evaluation. Determining the properties and the design values by incorporating weather-induced aging would allow for a more realistic estimation of roof component and system behavior at various stages of the roof's service life.

Systems evaluation was completed with all roofing components integrated and exposed to extreme winds. The main findings could be concluded as:

- the newly constructed systems performed better than the aged systems,
- disengagement of the sealing strip was the first to initiate failure modes that occurred across all mock-ups,
- the shingles' tear and fastener pull-through resistance were much lower on the aged mock-ups which led to failure,
- more testing is required to determine the resiliency factor for steep slope residential roofs.

The prediction of the future wind climatic data and the proposed performance evaluation are two of the steps for a new Holistic Approach (HA). This thesis presented a step-by-step approach for the HA.

The first step of the HA is evaluating the wind load, which needs to account for future climatic projections. This was achieved by creating a severity classification for roofs located in various cities across Canada and by developing an online tool, "Climate-RCI". The "Climate-RCI" web tool can determine the design parameters for wind, rain, and temperature based on the future climatic projections provided by Environment and Climate Change Canada. This allows the projected wind loads to be easily incorporated into the roof design by engineers and practitioners. Overall it was found that the wind load predicted to be affected due to future climate variations will change the most for a city located in an "extreme" wind location for a global warming temperature increase of 2℃. This equates to an increase of 10% from the current wind pressure values and affects only 2% of the locations across Canada. The majority of locations will maintain in the "normal" classification which experiences an increase of less than 5% for a global warming magnitude of 2℃. This wind load needs to be included in the design of steep slope residential roofs that are classified under Part 9 of the NBCC. Incorporating such calculated wind load for the roof design accounts for variations due to the location and climate, which greatly affect the wind load acting on the roof.

The second step of the present HA is the evaluation of the roof's resistance. The resistance needs to be greater than the load. This resistance is achieved through the resistance chain of the roofing system with components interacting together. However, the resistance is as strong as the weakest link in a system. Thus the resistance of shingles as a component was analyzed and new, lab-stored, and field-aged shingles were quantified for the effect of weathering on the resistance. When the tests were performed on new shingles, they all met the performance requirements. However, after being exposed to weather conditions for a period of 15 years, this was no longer the case. The weather exposure was less than the number of years the warranty of the shingles were advertised for. The confusion between the shingle warranty and service life needs to be solved by providing more information to owners and by providing a new classification system that accounts for the roof performance as a system. The resistance of new roofing systems representing two zones of the roof, the edge and the field, were tested and they showed a better performance than the aged roof systems tested for the same zones.

The third step of the HA is the effect of the installation on the roof's service life. This was achieved through the creation of a number of quality assurance metrics based on the best practices and in consultation with the roofing industry.

7.2 Recommendations for Future Work

Based on the findings identified above one can develop a durability index for steep slope residential roof cladding. Such a developed durability index can be used to answer whether the roof will still meet the design performance requirements. This would also minimize the large discrepancy that exists between manufacturers' warranty and the years of service life of roofs.

The minimum requirement of overlap strength of the shingles needs to be included in CSA A123.5. The tear strength of the underlayment and its fastener pull-through resistance along with the peel strength of the eave protection all need to be determined under different climatic conditions. This would provide a better understanding of the system and component performance.

Future work would include evaluation of roof systems installed and aged for various Canadian cities weathering and under different climatic conditions. Periodic evaluations would also lead to the development of deterioration curves to quantify the strength under different aging and climatic combinations.

Currently, a resistance factor for steep slope asphalt roofs does not exist and needs to be determined in the future, to allow the application of the LRFD approach. By following the required steps outlined in Chapter 3 of this thesis the development of this factor, which is crucial in the implementation of the HA, can be determined.

The quality assurance measures for the installation of residential roofs can be further expanded. These requirements along with additional requirements for rain and temperature can compose a standard that would aid in increasing the steep slope residential roof's resiliency. "Climate-RCI" can also be further expanded to complete the calculations, similar to the calculation examples presented in Appendix B, which provides the wind uplift loads on the cladding of steep slope residential roof.

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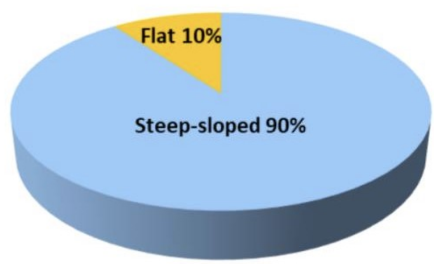
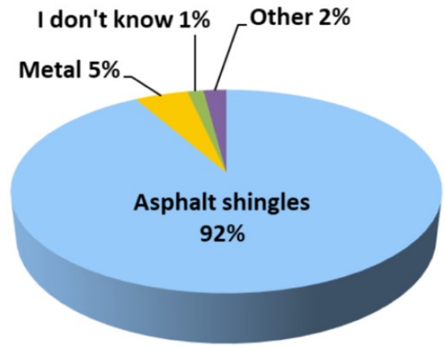
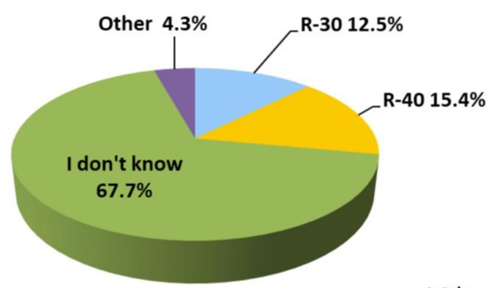
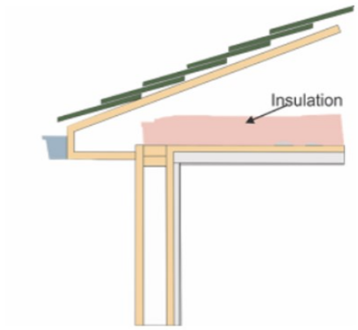
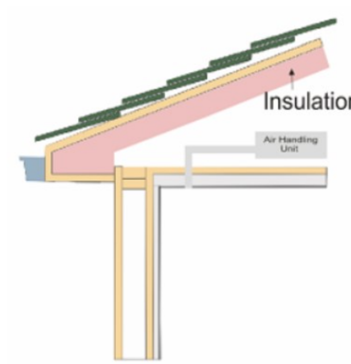
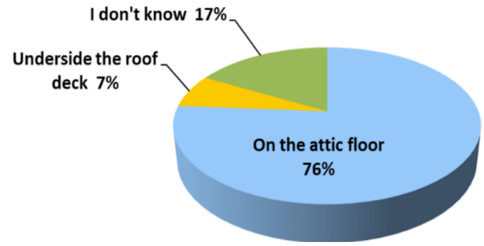
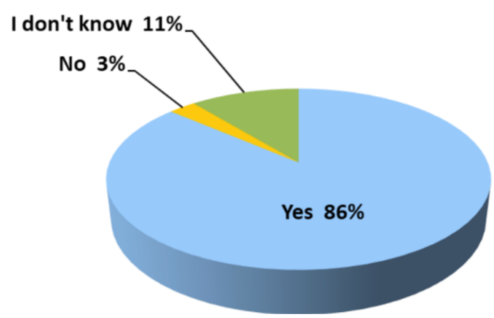
Appendix A: Residential survey questions

This appendix contains the questions from the National Research Council of Canada's survey for homeowners. Alongside each question, the answers received along with the total number of answers can be found.

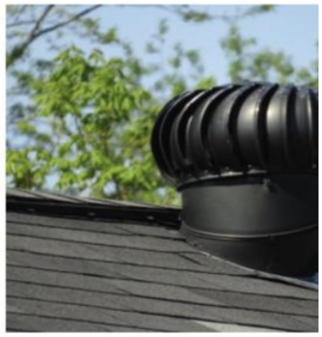
Number	Survey Question	Answer
1.	**Where is your house located?** **Please enter name of city (town, municipality, etc.) and province.**	Ontario – 1133 Quebec – 130 British Columbia – 33 Nova Scotia –17 New Brunswick – 8 Prince Edward Island – 5
2.	**Is the roof of your house:**  **Steep-sloped**  **Flat**	 Total answers: 1142

3.	**What type of roof covering does your roof have?** ○ Asphalt shingles  ○ Metal  ○ I don't know ○ Other (please specify)	 Total answers: 1029
4.	**What is the thermal resistance (R-value) of the roof insulation?** ○ R-30 ○ R-40 ○ I don't know ○ Other (please specify)	 Total answers: 927

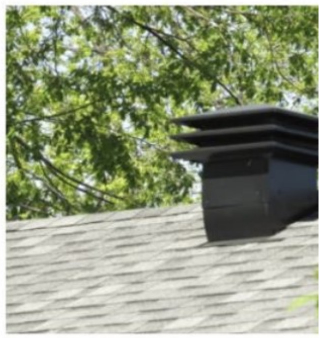
5.	**Where is the roof insulation located?** On the attic floor  Underside the roof deck  I don't know	 Total answers: 922
6.	**Is the attic vented to the outdoors?** Yes No I don't know	 Total answers: 922

7. **Please select the type of vents installed on your roof (please check all that apply):**

- ☐ Turbine vent

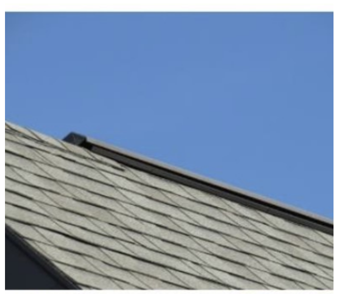


- ☐ Static vent type 1

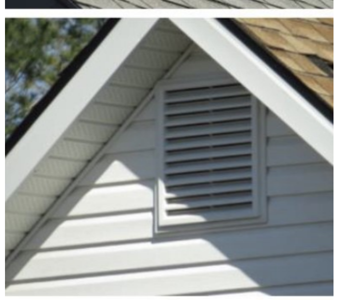


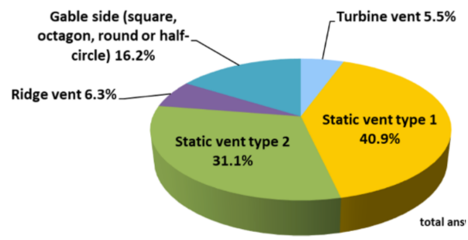
- ☐ Static vent type 2

- ☐ Ridge vent



- ☐ Gable side (square, octagon, round or half-circle)

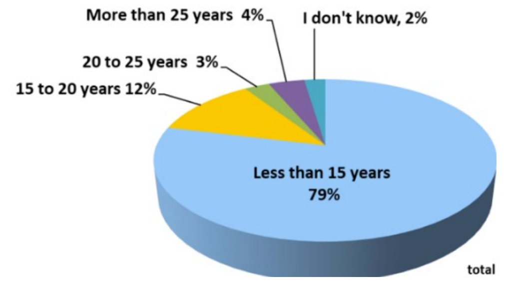




Total answers: 1120

8. **How old is your roof?**

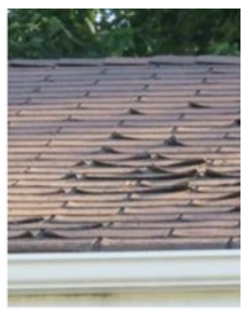
- ◉ Less than 15 years
- ○ 15 to 20 years
- ○ 20 to 25 years
- ○ More than 25 years
- ○ I don't know



Total answers: 886

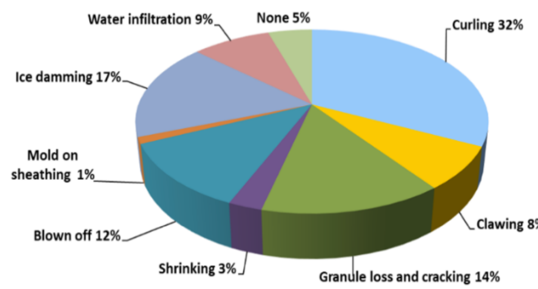
9. **Did your roof have any of the following damages? Please check all that apply.**

☐ Curling

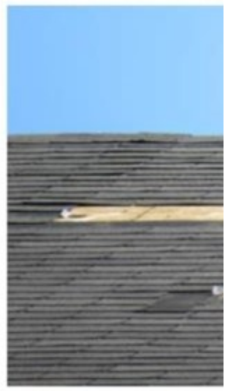
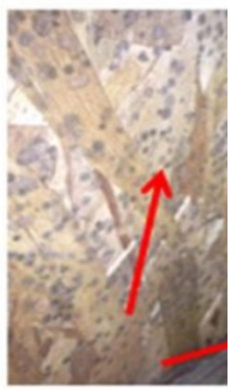


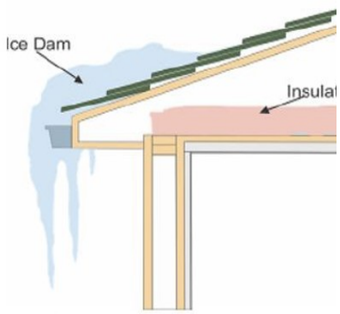
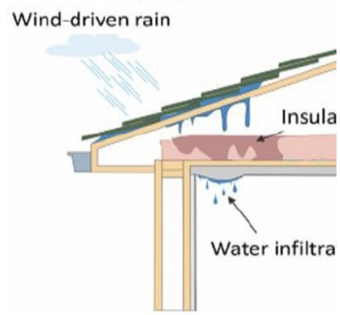
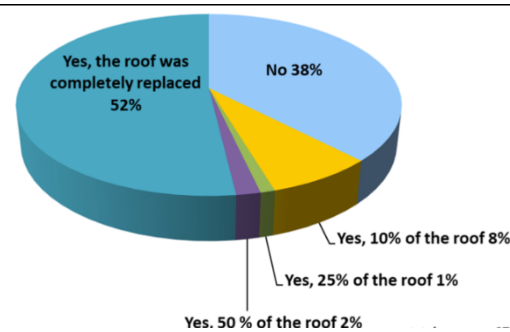
☐ Clawing

☐ Granule loss and cracking



Total answers: 873

	☐ Shrinking  ☐ Blown off  ☐ Mold on sheathing and truss 	

	☐ Ice damming  ☐ Water infiltration  Other (please specify):	
10.	**Have you ever repaired /replaced the roof of your house?** ○ No ◉ Yes, 10% of the roof ○ Yes, 25% of the roof ○ Yes, 50 % of the roof ○ Yes, the roof was completely replaced	 Total answers: 877
11.	Have you ever considered selecting a roof cover for your house with a 25, 40 or 50 years warranty, or similar? If yes, please comment here on your expectation:	
12.	If you have any other comments about your roof or this survey please enter them here:	
13.	Would you be open to a follow-up discussion regarding the roof of your house? If yes, please enter your name and your e-mail or telephone number here.	

Appendix B: Detailed calculations of wind pressures for roof cladding

This appendix shows detailed calculations to obtain the wind uplift pressures on the cladding of a hipped roof with a slope greater than 7° but less than 27°. Calculations were performed using both the reference wind pressure in the current NBCC as well as the future reference wind pressure assuming a global warming magnitude of 2°C. The results of the comparison are included in Chapter 3. The calculations shown follow the static procedure outlined in NBCC Part 4.1.7 and are summarized in Table B-1.

Table B - 1. Summary of wind load calculation procedure for roof cladding

Step Number	Step Description	Corresponding section in NBCC
1	Identify building specifications (width, length, height), locations	
2	Determine importance category (I_w)	Table 4.1.7.3
3	Define corner zone	
4	Determine reference velocity pressure based on building location (q)	Appendix C, Table C2, Column : Hourly Wind Pressures (1/50) (kPa)
5	Determine Topographic Factor (C_t)	4.1.7.4
6	Determine External Pressure Component ($C_pC_gC_e$)	
	Determine External Exposure Factor (C_e)	4.1.7.3-5
	Determine Combined Pressure and Gust Factor Coefficient (low-rise buildings) (C_pC_g)	4.1.7.6, Figures 4.1.7.6-A to 4.1.7.6-H
	Determine External Pressure Coefficient (high-rise buildings) (C_p)	4.1.7.5-3
	Determine Gust Factor (high-rise buildings) (C_g)	4.1.7.3-8
7	Determine Internal Pressure Component ($C_{pi}C_{gi}C_{ei}$)	
	Determine Internal Exposure Factor (C_{ei})	4.1.7.3-7
	Determine Internal Pressure Coefficient (C_{pi})	4.1.7.7
	Determine Internal Gust Factor (C_{gi})	4.1.7.3-10
8	Calculate Wind Uplift Pressure $P=I_wqC_t(C_pC_gC_e- C_{pi}C_{gi}C_{ei})$	4.1.7.3-2

A house is assumed to be located in Bella Cola, British Columbia with a height of 14 m, width of 10 m, and length of 16 m. The terrain around the building is rough and there are no hills or escarpments so the topographic factor (C_t) is equal to 1.The importance level of the building is normal while the building openings are assumed to be “Non-uniformly distributed openings of which none is significant or significant openings that are wind-resistant and closed during storms”.

Using current NBCC reference pressure:

Step 1 Identify building specifications (width, length, height), locations

Height=14 m; Width= 10 m; Length=16 m

Location= Bella Coola, British Columbia

Step 2 Determine importance category **(I_w)**

I_w=1 (Normal)

Step 3 Define corner zone

Corner Zone $a = min\begin{cases}0.1D\\0.4H\end{cases} = min\begin{cases}0.1*10\\0.4*14\end{cases} = min\begin{cases}1\\5.6\end{cases} = 1\,m$

Step 4 Determine reference velocity pressure based on building location **(q)**

$q = 0.39\ kpa$ based on $\frac{1}{50}$ probability (Appendix C, Table C-2, NBCC)

Step 5 Determine Topographic Factor **(C_t)**

C_t = 1 (no hills or escarpments)

Step 6 Determine External Pressure Component **($C_pC_gC_e$)**

Determine External Exposure Factor **(C_e)** (must be ≥ 0.7)

$$C_e = 0.7*\left(\frac{h}{12}\right)^{0.3} = 0.7*\left(\frac{14}{12}\right)^{0.3} = 0.73$$

Determine Combined Pressure and Gust Factor Coefficient (low-rise buildings) **(C_pC_g)** (Figure 4.1.7.6-E); tributary area assumed to be 1 m²

$$C_pC_g = \begin{cases}Corner = -5\\Edge = -3.6\\Field = -2.4\end{cases}$$

External Pressure Component $C_pC_gC_e = \begin{cases}Corner = -3.7\\Edge = -2.64\\Field = -1.76\end{cases}$

Step 7 Determine Internal Pressure Component **($C_{pi}C_{gi}C_{ei}$)**

Determine Internal Exposure Factor **(C_{ei})** (must be ≥ 0.7)

$$C_{ei} = 0.7*\left(\frac{\frac{h}{2}}{12}\right)^{0.3} = 0.7*\left(\frac{\frac{14}{2}}{12}\right)^{0.3} = 0.6 < 0.7\ \therefore \qquad C_{ei} = 0.7$$

Determine Internal Pressure Coefficient **(C_{pi})**

$$C_{pi} = -0.45\ to\ 0.30$$

Determine Internal Gust Factor **(C_{gi})**

$$C_{gi} = 2$$

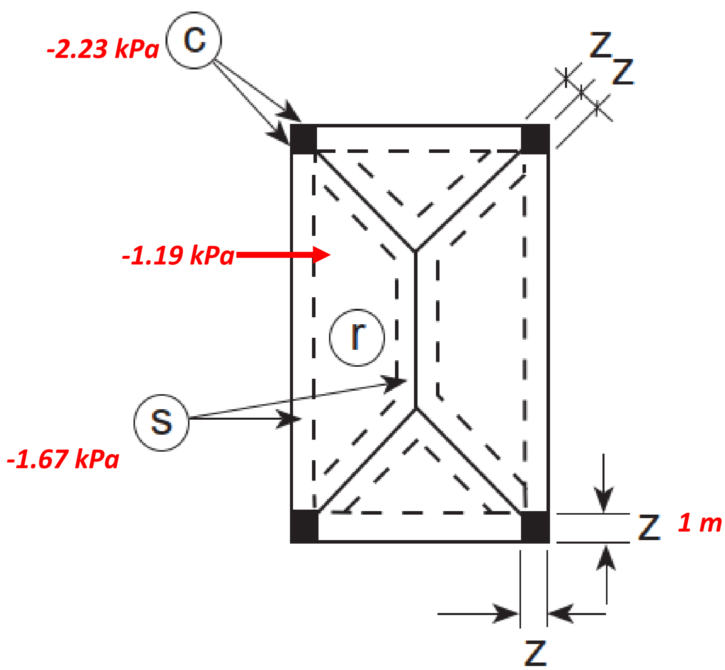
Internal Pressure Component $C_{pi}C_{gi}C_{ei} = -0.63\ to\ 0.42$

Step 8 Calculate Wind Uplift Pressure **$P=I_w q C_t \ (C_pC_gC_e - C_{pi}C_{gi}C_{ei})$**

$$P = q(External\ Pressure\ Component - Internal\ Pressure\ Component) = q(C_pC_gC_e - C_{pi}C_{gi}C_{ei})$$

$$P = \begin{cases} Corner = \max|[1x1x0.39(-3.7-(-0.63)); 0.39(-3.7-0.42)]| = \max|(-1.18;\ -1.59)| \\ Edge = \max|[1x1x0.39(-2.64-(-0.63)); 0.39(-2.64-0.42)]| = \max|(-0.78;\ -1.19)| \\ Field = \max|[1x1x0.39(-1.76-(-0.63)); 0.39(-1.76-0.42)]| = \max|(-0.44;\ -0.85)| \end{cases}$$

$$P = \begin{cases} Corner = -1.59kPa\ x\ 1.4 = -2.23\ kPa \\ Edge = -1.19kPa\ x\ 1.4 = -1.67kPa \\ Field = -0.85kPa\ x\ 1.4 = -1.19\ kPa \end{cases}$$



Using future NBCC reference pressure:

Step 1 Identify building specifications (width, length, height), locations

Height=14 m; Width= 19 m; Length=16 m

Location= Bella Coola, British Columbia

Step 2 Determine importance category **(I_w)**

I_w= 1 (Normal)

Step 3 Define corner zone

Corner Zone $a = min\begin{cases}0.1D\\0.4H\end{cases} = min\begin{cases}0.1*10\\0.4*14\end{cases} = min\begin{cases}1\\5.6\end{cases} = 1\,m$

Step 4 Determine reference velocity pressure based on building location **(q)**

$q = 0.45\ kpa$ based on $\frac{1}{50}$ probability (Climate-RCI)

Step 5 Determine Topographic Factor **(C_t)**

C_t= 1 (no hills or escarpments)

Step 6 Determine External Pressure Component **($C_pC_gC_e$)**

Determine External Exposure Factor **(C_e)** (must be ≥ 0.7)

$$C_e = 0.7*\left(\frac{h}{12}\right)^{0.3} = 0.7*\left(\frac{14}{12}\right)^{0.3} = 0.73$$

Determine Combined Pressure and Gust Factor Coefficient (low-rise buildings) **(C_pC_g)** (Figure 4.1.7.6-E); tributary area assumed to be 1 m^2

$$C_pC_g = \begin{cases}Corner = \ -5\\Edge = \ -3.6\\Field = \ -2.4\end{cases}$$

External Pressure Component $C_pC_gC_e = \begin{cases}Corner = -3.7\\Edge = \ -2.64\\Field = -1.76\end{cases}$

Step 7 Determine Internal Pressure Component **($C_{pi}C_{gi}C_{ei}$)**

Determine Internal Exposure Factor **(C_{ei})** (must be ≥ 0.7)

$$C_{ei} = 0.7*\left(\frac{\frac{h}{2}}{12}\right)^{0.3} = 0.7*\left(\frac{\frac{12}{2}}{12}\right)^{0.3} = 0.57 < 0.7\ \therefore \qquad C_{ei} = 0.7$$

Determine Internal Pressure Coefficient **(C_{pi})** $C_{pi} = -0.45\ to\ 0.30$

Determine Internal Gust Factor **(C_{gi})** $C_{gi} = 2$

Internal Pressure Component $C_{pi}C_{gi}C_{ei} = -0.63\ to\ 0.42$

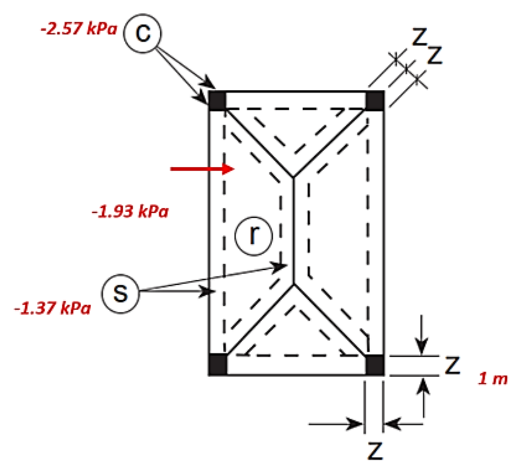
Step 8 Calculate Wind Uplift Pressure **$P=I_wqC_t\ (C_pC_gC_e - C_{pi}C_{gi}C_{ei})$**

$$P = q(External\ Pressure\ Component - Internal\ Pressure\ Component) = q(C_p C_g C_e - C_{pi} C_{gi} C_{ei})$$

Using Climate-RCI:

$$P = \begin{cases} Corner = \max\left|\left[1x1x0.45(-3.7-(-0.63)); 0.39(-3.7-0.42)\right]\right| = \max|(-1.37;\ -1.84)| \\ Edge = \max\left|\left[1x1x0.45(-2.64-(-0.63)); 0.39(-2.64-0.42)\right]\right| = \max|(-0.90;\ -1.38)| \\ Field = \max\left|\left[1x1x0.45(-1.76-(-0.63)); 0.39(-1.76-0.42)\right]\right| = \max|(-0.51;\ -0.98)| \end{cases}$$

$$P = \begin{cases} Corner = -1.84kPa\ x\ 1.4 = -2.57\ kPa \\ Edge = -1.38kPa\ x\ 1.4 = -1.93kPa \\ Field = -0.98kPa\ x\ 1.4 = -1.37kPa \end{cases}$$



Appendix C: Rain and temperature Severity Projected by Climate-RCI

This appendix shows how the rain and temperature design values found in Table C-2 of the NBCC were measured historically as well as the impact of climate change on them. The historical rain data was collected using a standard Canadian rain gauge placed level on a post, installed a minimum of two feet deep in an open terrain free of obstructions [70]. This was done to ensure an accurate and undisrupted measurement of rainfall. The apparatus is shown in Figure C-1 along with an automatic rain gauge [71], which sends an electronic impulse to a recording data acquisition every time 0.2mm of rain has been measured.

The rain load is calculated together with the snow load. Rain loads are tabulated in two categories in the NBCC: "15-Minute Rain" and "One-Day Rain". Both of these categories were first incorporated into the code in 1961 based on the data of about 100 stations. The concentration in this Appendix will be on the one-day rainfall which is used to ensure the structural capacity of the roof can withstand the load created in the event of a blockage of drainage systems. This is assumed as the worst-case scenario. The one-day rainfall values are standardized to express the maximum 1-day rainfall in a period of 1 in 50 years. This is based on daily observations from over 3500 stations across the country for over the past 10 years [10].

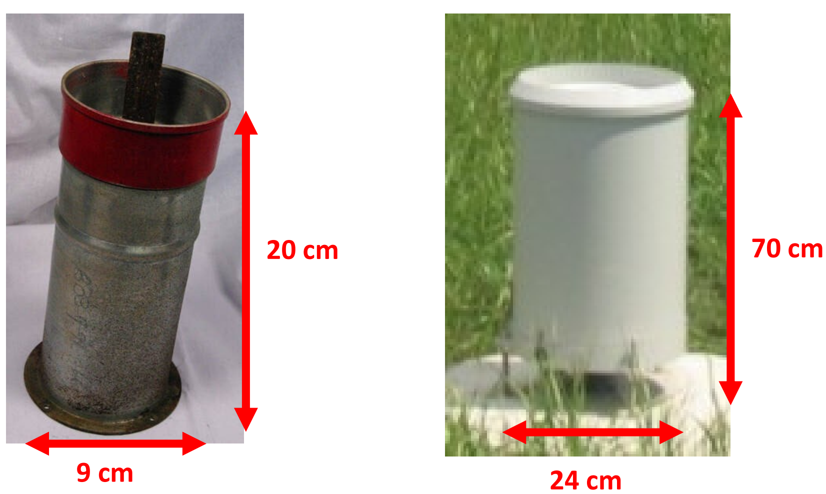


Figure C - 1. Rain gauges, historic (left) and current (right) [19], [20]

The continually changing climate is contributing to an ongoing increase in the amount of atmospheric moisture, and the frequency and intensity of extreme precipitations [72]. Therefore having reliable projected design values is critical for the design process of new roofs. As the global average temperature increases, it is clearly observed that the rain load in cities across Canada increased. This follows the general trend described in [53]. Climate change and Environment Canada have identified different global warming scenarios with 0.5˚C being the lowest and 3.5˚C being the highest. An example of this can be seen for three different cities across Canada in Figure C-2. There is a clear increase in the rain load for all three cities for all scenarios. From the 0.5˚C global warming magnitude to the 3.5˚C there is an increase of 36%, 32%, and 37% respectively for Ottawa, Fredericton, and Bella Coola. While the percent change in rainfall for two extreme scenarios compared to what is currently used in the code can be seen in Figure C-3 for all of Canada. All locations in Canada show an almost 40% increase in rain load. Table C-1 shows the severity classification for the rain as determined by "Climate-RCI".

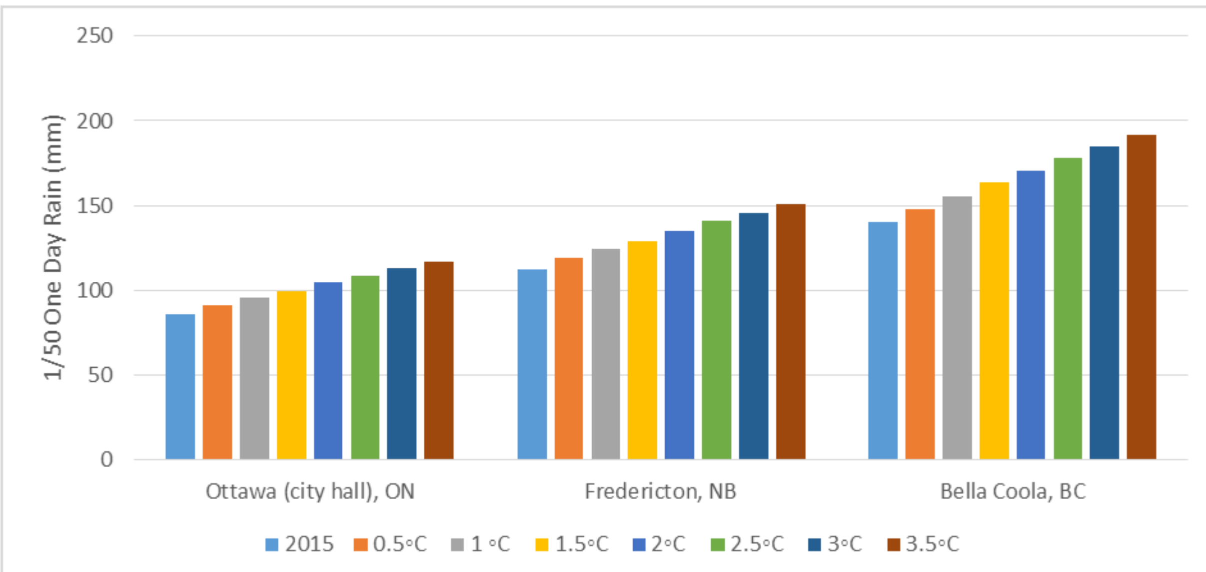


Figure C - 2. Rainfall comparison of three cities for each global warming magnitude

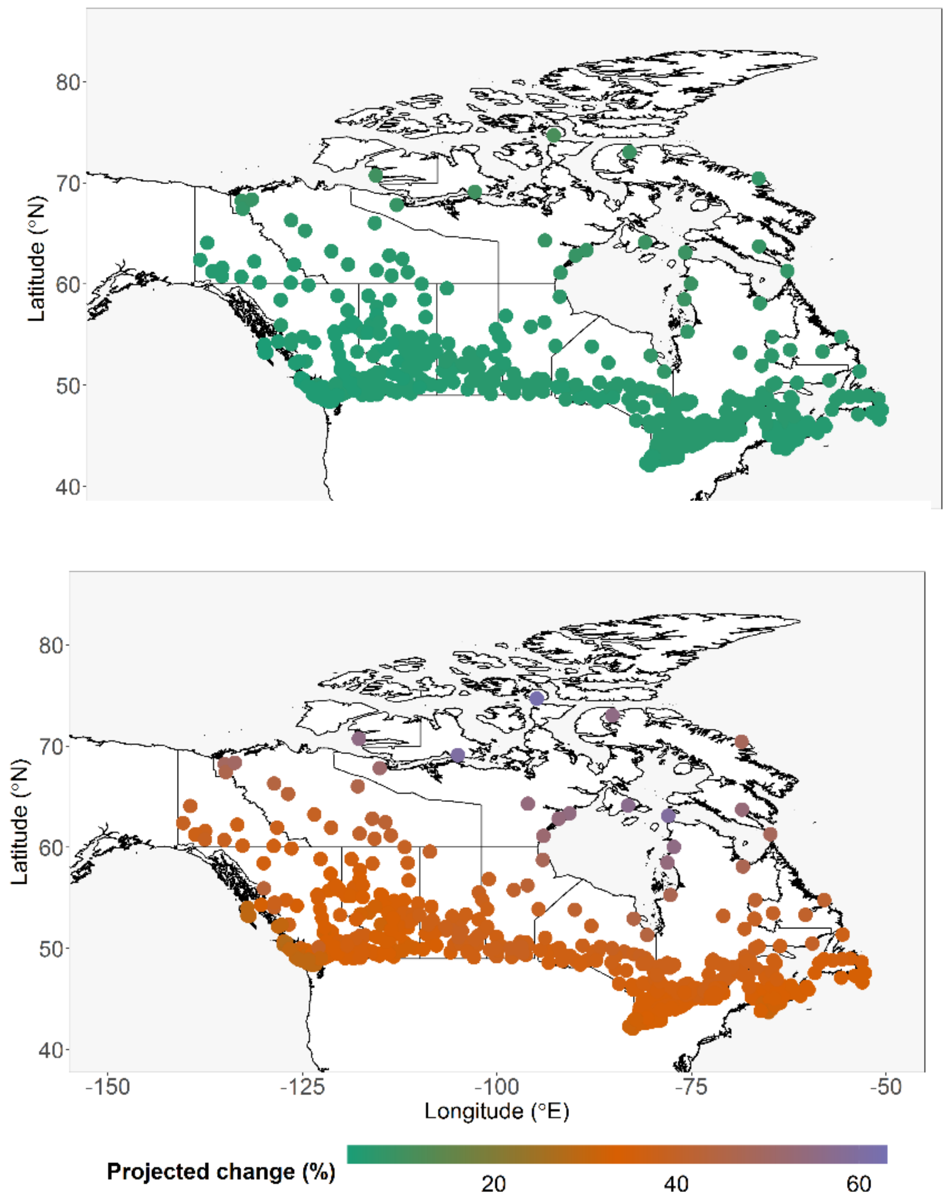


Figure C - 3. Percent rain increase under 0.5◦C (top) and 3.5◦C (bottom) global warming magnitude

Table C - 1. Severity Classification Ranges for Rain [32]

Range	Rain
Normal	Change < 20%
Severe	20% ≤ Change < 30%
Extreme	Change ≥ 30%

Temperature is used in the design of energy-efficient heating, ventilation, and cooling systems for buildings and the selection of appropriate materials and construction methods. The air temperatures listed in the code are measured using a Stevenson screen located 1.5 m above the ground in an open terrain, which protects the four different thermometers located in the box [73]. This records every 60 seconds but the actual values are presented as hourly averages for each hour, month, and year (Figure C-4).

The January design temperatures are typically used in the design of heating systems and 2.5% or 1%, are selected to be used depending on the temperature sensitivity of the area [10]. While the July design temperatures are used to design cooling systems to ensure the building maintains optimum temperature and humidity during the summer months [10].

Table C-2 of the NBCC provides four different temperature design temperatures, two for the month of January and two for the month of July. These two months are chosen as representatives of two extreme conditions of air temperature from 480 stations taking hourly measurements from the last 25 years [10]. Temperature design data was first incorporated in the code is in 1953 where the values were based on a 1 in 10 probability of occurrences. This led to the values being 5 to 10 degrees lower than those in the 1961 version of the code which contained data from a larger number of stations (about 100 stations). This led to the conclusion of the inaccuracy associated with the 1 in 10-year probability of occurrence method [74]. In both these versions of the code, only the winter design temperatures based on the month of January for both 2.5% and 1% were incorporated and no values were provided for the summer design temperature.

Figure C - 4.The setup for recording the temperature [20]

The temperature increase is one of the main indicators of climate change. It leads to the decrease of land and ocean reflectivity due to decreasing snow and ice cover, rising sea levels, droughts, increased precipitation, and overall more unpredictable weather [53]. As the global average temperature increases, the winter and summer temperatures are also increasing. This can be seen for Ottawa, Fredericton, and Bella Coola in Figure C-5 and Figure C-6 for January and July design temperatures, respectively. Table C-2 below shows the severity classification for the temperature calculated based on “Climate-RCI”.

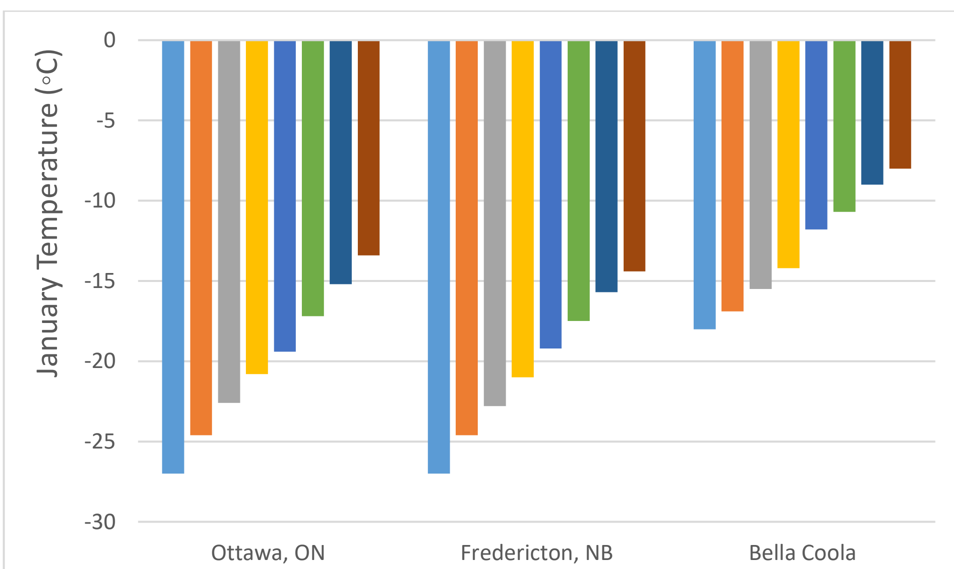


Figure C - 5. Temperature comparison- January Design Temperature

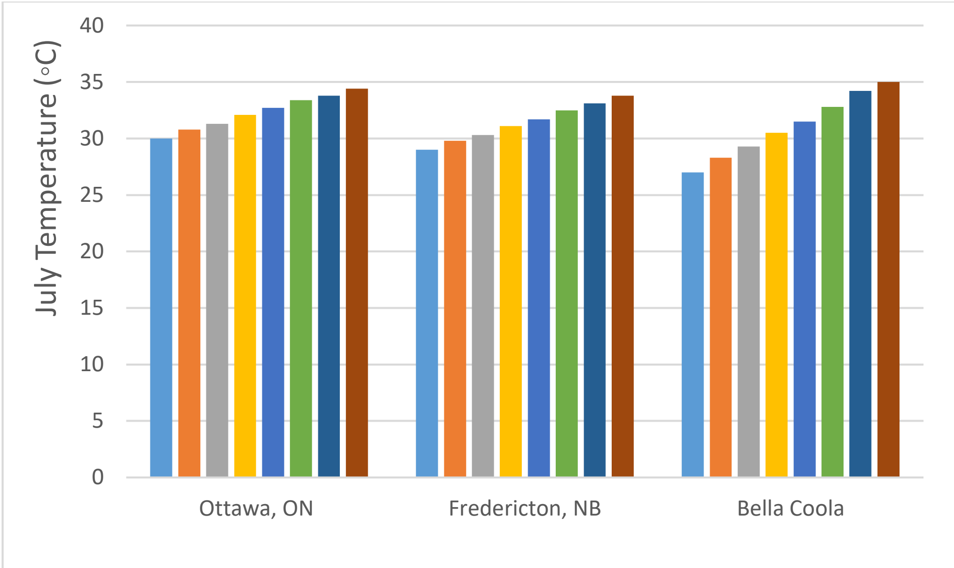


Figure C - 6. Temperature comparison- July Design Temperature

Table C - 2. Severity Classification Ranges for Temperature

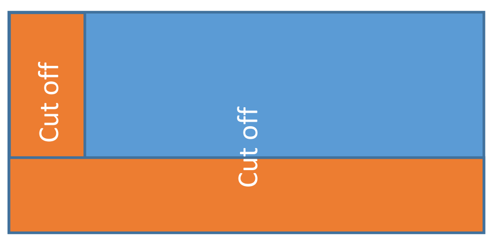
Range	Temperature
Normal	Change < 1°C
Severe	1°C ≤ Change <4°C
Extreme	Change ≥4°C

Appendix D: Construction of the mock-ups

This appendix shows the steps undertaken for the construction of the new mock-ups (New Edge 2020 and New Field 2020). The construction was performed inside the WDS under laboratory conditions following the procedure highlighted in Chapter 2.

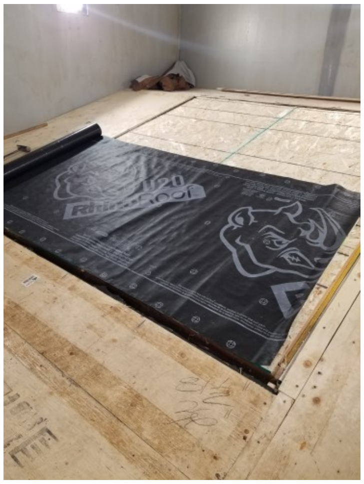
1. Roof trusses ready for the installation of the roof deck.

2. Installation of the roof deck on top of the roof trusses using 4 ft x 8 ft OSB sheets and screws.

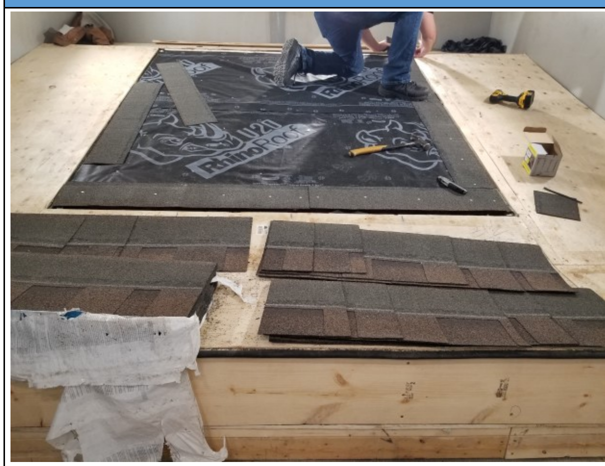


Initially OSB sheet 4ft x 8ft, was cut down to 3 ft x 6.3 ft to fit the mock-up size .

3. Once the roof deck has been installed, the underlayment is installed.



4. A row of starter shingles is installed before the shingles are installed from the edge up.



Appendix E: Component Variability

This appendix summarizes the results obtained for the component evaluation along with the standard deviation for each of the three properties evaluated for Type 1 and Type 2 shingles discussed in Chapter 5.

Property	Shingle Type	New 2005 (reference)		Lab 13 Years		Aged 13 Years		Aged 15 Years		New 2020	
		Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation
Tensile Strength (kN/m)	**1-MD**	**20.3**	**2**	**16.7**	**3.1**	**9.8**	**1.7**	**N/A**		**21.6**	**2**
	1-XD	**10**	**1.2**	**9.6**	**1**	**7**	**1.4**			**10.8**	**1.5**
	2-MD	**19.5**	**1.3**	**13.5**	**2.5**	**N/A**		**10**	**1.3**	**N/A**	
	2-XD	**12.2**	**1.3**	**9.7**	**1.1**			**7.7**	**2**		
Tear Strength (g)	**1**	**1721.6**	**267.7**	**1323.5**	**330.8**	**714.20**	**81.2**	**N/A**		**1780**	**243**
	2	**1804.8**	**110.1**	**1657.6**	**165.3**	**N/A**		**1198.5**	**170.5**	**N/A**	
Fastener Pull-through Strength (N)	**1**	**174.6**	**16**	**161**	**32.7**	**154**	**36.4**	**N/A**		**196.1**	**32.3**
	2	**N/A**						**83.2**	**5.4**	**N/A**	