

Increasing High Wind Safety for Canadian Homes: A Foundational Document for Low-Rise Residential and Small Buildings

By Dan Sandink, Gregory Kopp, Sarah Stevenson and Natalie Dale

April 2019



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Non-Technical Summary

High wind causes damage to homes and property, and results in risks to life and health across the country. A small number of communities in coastal and prairie regions of Canada may experience relatively frequent high wind events that result in damage to buildings; however, a significant area of central Canada is prone to tornadoes. It is expected that homeowners, insurers and decision makers will become increasingly interested in wind risk reduction as population increases result in more home construction in regions that may experience high wind and tornado events.

This report provides the basis for the development of a set of commonly acceptable, relatively straightforward wind risk reduction measures that can be incorporated into new single-family home construction and significant renovations to reduce risk to life, health and property. Measures presented in this document are intended to reduce risk from high winds associated with tornadoes, hurricanes, and other types of extreme weather events.

It is expected that the following groups may be interested in this report:

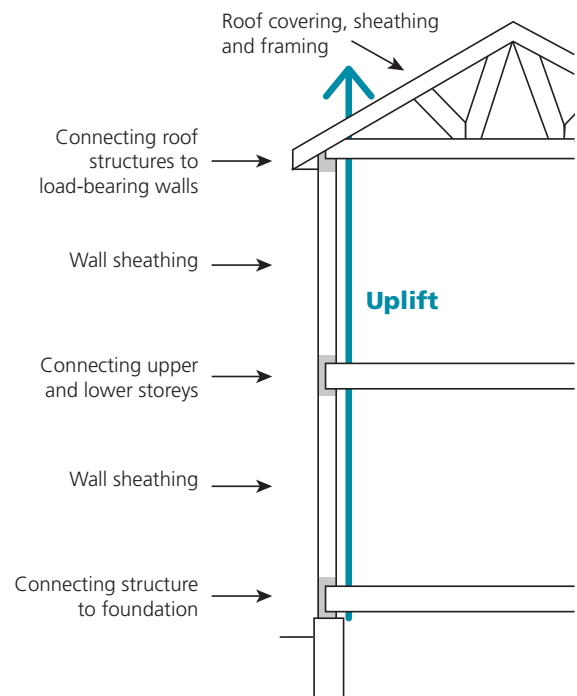
- Homeowners, particularly those who are in the process of buying or building a new home, or who are conducting significant structural changes/renovations to existing homes,
- Professionals involved in the building industry, including code officials, building material and component manufacturers and suppliers, code development agencies, builders' associations, and related professionals,
- Property and casualty (P&C) insurers, and
- Other stakeholders concerned with mitigating risk associated with high wind and extreme weather in general.

Measures presented in the report are based on research concerning performance of wood-frame buildings in high wind events, damage surveys following wind events and tornadoes in Canada and North America, best practices applied in North America, practical experience, and input from research, engineering, building and insurance professionals. Many of the measures presented here are already included in Canadian and provincial construction codes but may not be routinely applied to home construction in all regions that may be exposed to high wind events (e.g., regions prone to tornadoes).

The vast majority of tornado events in Canada are EF2 or less. Damages from these types of events are concentrated to roofs and roof structures. Damage surveys following tornado events in Canada have routinely identified damage to roofs, roof structures and connections between roofs and supporting walls. When roof structures fail, walls may also fail. Keeping roofs intact and securely fastened to the structure helps to reduce risk of wall failure. For these reasons, measures presented in this report emphasize protection of roofs and roof structures.

Additionally, an important strategy for increasing the safety of homes during high wind events is to ensure that homes have a secure "continuous vertical load path" (see Figure A). A continuous vertical load path requires that major structural systems – including roofs,

Figure A: Elements in the continuous vertical load path



walls and upper and lower storeys – are well connected, and that the entire structure is securely connected to the foundation. The aim of this approach is to ensure that uplift loads caused by high winds are transferred to the foundation.

High wind resistance for homes constructed according to Canadian construction codes remains an area of active research. While this report provides guidance on mitigating high wind risk, it is expected that best/recommended practice will evolve over time with improved knowledge, including collection of data from the field (damage surveys) and lab testing of building components constructed according to Canadian construction codes.

Roofs

Roofs are particularly exposed to damage from high winds. In general, roofs that are built using pre-fabricated, engineered trusses are considered to be more resistant to high wind impacts. When trusses are not used to frame roofs, as may be the case for custom homes with unique roof designs, an engineer who can take high wind loads into account should design the roof.

Hip roofs are also considered preferable for reducing exposure to high wind impacts (Figure B). When gable roofs are used, measures can be applied to ensure that gable end walls are reinforced and well connected to the structure, and therefore are capable of resisting wind loads.

Roof sheathing may be pulled off of buildings during high wind events, resulting in significant water intrusion into the attic and home. Slightly thicker roof sheathing (11.1 mm or 7/16" in lieu of 3/8" sheathing), combined with 63 mm (2.5") nails that are spaced 150 mm (6") apart along both the edges of the sheathing panel and along the interior supports, reduces the risk of damage associated with sheathing failure. Also, high wind events may result in damage to roofing materials (e.g., asphalt shingles), exposing the home to water damage. Shingles rated for high wind and secondary water penetration protection for roof decks, including sealing the seams between roof sheathing panels, can further reduce risk of damage.

One of the most common types of damage to homes experienced during high wind events is partial or complete removal of roof structures from supporting walls. Aside from directly damaging a home, debris from damaged roofs can become entrained in the wind field. When this debris impacts neighbouring buildings and punctures doors, windows and walls, affected buildings may become pressurized, causing the roof to be pushed up and off of the walls.

Wind-blown debris associated with roof failure may result in a domino effect of roof failure and damage to neighbouring buildings (Figure C). Flying debris is also a major cause of injury during tornado events. Application of measures to enhance the connection between the roof structure and the supporting walls serves to reduce the risk of this type of damage. Keeping roofs on homes helps reduce risk of wall collapse – a further reason to increase the strength of the connection between roofs and walls.

Figure B: Simple comparison of hip and gable roofs

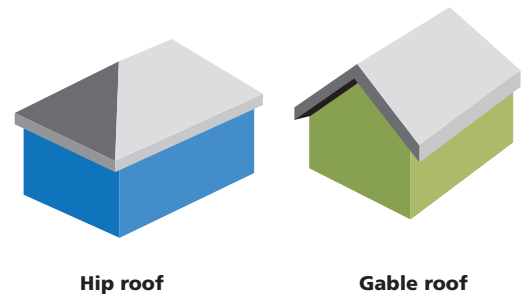
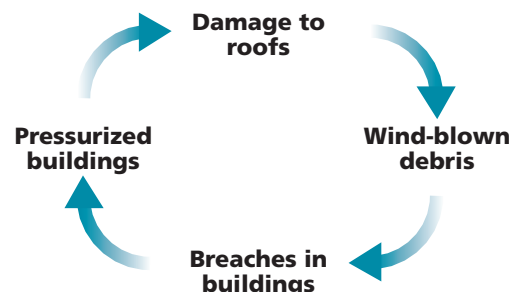


Figure C: Damage to roofs can result in flying debris, increasing risk of damage to neighbouring buildings



Walls and Upper and Lower Storey Connections

Further considerations covered in this report include application of methods for improving the rigidity of a building and nailing patterns that limit the risk that plywood or OSB sheathing will be removed from a building during high winds.

Wood wall sheathing can also contribute to a continuous load path when used to fasten upper and lower storeys together. Overlapping rim joists with upper- and lower-storey wall sheathing and fastening sheathing to rim joists can help tie the building together. Wood wall sheathing can also overlap and connect to the sill plate (a wood member that is fastened to the building's foundation), helping to tie the entire structure to the foundation. Where this approach is not practical, other measures, such as metal straps, may be used to enhance the vertical load path of a home, though the builder may have to consider alternative means of improving the rigidity or lateral bracing of the building.

Anchoring of the Building to the Foundation

Ensuring that the building is securely fastened to the foundation using anchor bolts will also contribute to the continuous vertical load path of the home. Anchor bolts may be installed based on existing building code requirements in much of Canada as a method to secure the continuous vertical load path of the building.

Additional Construction Detail: Post Connections and Garage Doors

Roofs that overhang exterior porches are prone to damage during high wind events. Ensuring that supporting posts are well connected to a foundation (e.g., concrete porch slab), and that robust connections are present at both the tops and bottoms of posts will help porch roofs remain in place during high wind events.

Failure of garage doors may result in pressurization of garages, causing roof failures. Garage doors rated for high winds, commonly used in regions of North America that are prone to hurricanes or tornadoes, can reduce the risk of garage door failure. Garage doors rated for high wind may be considered for "non-integral" garages, where the garage has its own roof with no living space above. Additionally, securing the continuous load path from the garage roof to the foundation will help reduce the risk of garage roof failure should a door fail during a storm.

Introduction

Extreme wind is a significant driver of disaster losses in Canada. Approximately 62% of all natural catastrophe events recorded by the Insurance Bureau of Canada between 1983 and 2016 were partially or fully caused by extreme wind.¹ It is further acknowledged in the Canadian P&C insurance industry that, after water damage, wind is the most significant driver of disaster losses. Experience in the Canadian P&C insurance industry indicates that non-engineered, residential structures drive the majority of losses during disasters. Damage to residential buildings during extreme wind events, including tornadoes, also creates life safety issues associated with flying debris and building collapse. Significant engineering knowledge exists that can be readily applied to reduce the risk of damage to buildings during high wind events.

The intent of this project is to develop a “seed” or “foundational” document that outlines a set of high wind/tornado risk reduction measures generally applicable to low-rise residential and small buildings, as defined in Article 1.3.3.3, Division A of the National Building Code of Canada (NBCC). These measures may be considered applicable in many regions of Canada, including areas not traditionally defined as exposed to high or extreme wind forces in the NBCC.

1.1. Overview

Prescriptive provisions related to high or extreme wind pressure exposure for NBCC Part 9 buildings apply in a relatively small number of Canadian regions where 1-in-50 year hourly wind pressure ($q_{1/50}$) is 0.8 kPa or greater, and less than 1.2 kPa. A significant portion of the country, including the majority of Canada’s heavily populated areas, however, has the potential to experience extreme wind events such as tornadoes between categories EF0-EF5 (Figure 1 and Table 1). Further, the majority of confirmed and probable tornado events in Canada have occurred outside of regions identified by the NBCC as exposed to high or extreme wind forces (Figure 1).

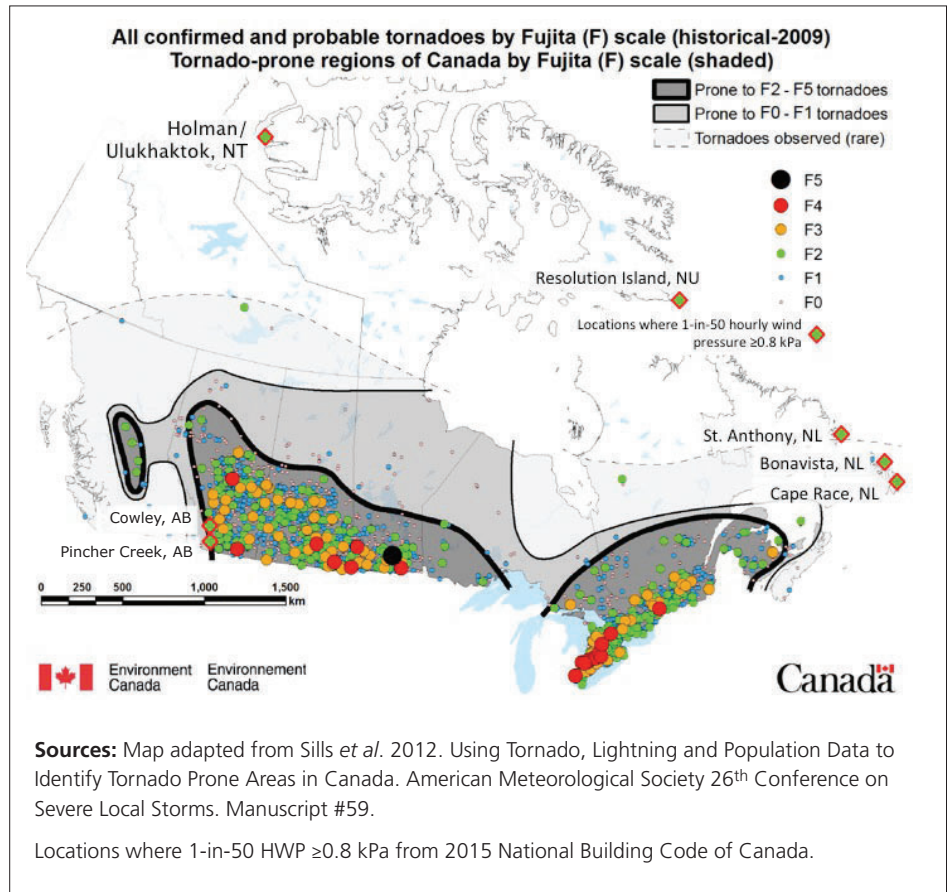
Risk reduction measures associated with high wind exposure presented in the NBCC are not intended to “...provide design solutions against the direct force of tornadoes.” A further national construction document related to NBCC Part 9 buildings, the Canadian Wood Council’s 2014 “Engineering Guide for Wood Frame Construction” (hereafter referred to as CWC 2014), applies a similar approach with respect to resistance to extreme wind and tornadoes. The prescriptive requirements in Part 9 of the NBCC were developed based on CWC 2014, although there are some differences. Generally, wind risk reduction is considered to be covered by Subsection 4.1.7 (Wind Load) of the NBCC (for engineered buildings).

For Part 9 buildings, key construction details designed to limit extreme wind risk, notably anchorage of building frames to foundations, are provided in the NBCC (see Article 9.23.6.1 and discussion in Appendix A).² NBCC Subsection 9.23.13 further includes provisions related to bracing to resist lateral loads for wind and earthquake. In areas where $0.8 \leq q_{1/50} < 1.2$, bracing to resist lateral loads is required to be constructed in accordance with NBCC Part 4, CWC 2014, or Articles 9.23.13.4 to 9.23.13.7 of the NBCC. As outlined in the NBCC, where $q_{1/50}$ is equal to or greater than 1.20 kPa, buildings must be engineered.

In the past few years, a number of Canadian and US jurisdictions have required or encouraged adoption of prescriptive wind risk reduction provisions for single-family homes. These jurisdictions include Moore, OK,³ Dufferin County, ON⁴ and Victoriaville, QC.⁵ In the case of Dufferin County and Victoriaville, wind risk reduction provisions have focused on strengthening the connections between the roof framing and supporting walls (hereafter referred to as roof-to-wall connections or RTWCs). The State of Oklahoma has also developed prescriptive measures aimed at increasing the resistance of wood-frame, low-rise residential buildings to tornado events.⁶ In the case of Moore, OK and the State of Oklahoma, measures have been implemented with the intent of reducing the impacts of EF2 tornadoes (in these instances, 135 mph [217 km/h] wind speeds were used to represent theoretical wind speeds associated with EF2 tornadoes – see Appendices J and K).⁷

In 2012, a change was introduced into the Ontario Building Code (OBC) requiring reduction of spacing of roof sheathing fasteners on intermediate supports from 300 mm to 150 mm (where supports are spaced at more than 406 mm o.c.).⁸ The change was specifically motivated by concerns related to resistance to high wind and tornadoes.⁹ Further, a project conducted by the Institute for Catastrophic Loss Reduction (ICLR) for the Region of Durham, Ontario in 2017 sought to develop a set of

Figure 1: Confirmed and probable tornadoes in Canada and 2015 NBCC wind hazard locations vs. tornado-prone regions



Box 1: A note on Figure 1 and F (Fujita) vs. EF (Enhanced Fujita) Scales:

Figure 1 is based on historical data, ending in 2009. Because Canada adopted the EF Scale in 2013, F-Scale data are presented in Figure 1.

The reader should note that the F and EF Scales are damage scales; therefore, while wind speeds associated with damages in the EF-Scale have changed, the damages associated with each EF category are reasonably consistent with the F-Scale. This characteristic of the EF-Scale reflected the desire to ensure that historical tornado damage databases were preserved when the EF-Scale was developed in the US in the mid-2000s. However, the damage ratings are not perfectly consistent in the two scales because the EF-Scale brought in several new damage indicators, while also rationalizing the damage observations (called Degrees of Damage, or DOD) for the existing damage indicators.

The compatibility of the F- and EF-Scales was tested during EF-Scale development. An independent set of professionals was asked to assess wind speeds based on the updated damage indicators. A correlation was then developed between F and EF Scale wind speeds, where the original F-Scale criteria were used to "...assign [F-Scale] categories...to the DOD's of the new EF Scale." Regression analysis revealed an R2 of 0.9118, indicating a very close relationship between F and EF scale damage estimates.

For more information, see: WSEC. 2006. A Recommendation for an Enhanced Fujita Scale. Wind Science and Engineering Centre, Texas Tech University.

prescriptive measures aimed at reducing wind damage risk for OBC Part 9 residential buildings,¹⁰ providing a basis for continued work at the national level.

Further to the above, a significant body of literature based on wind research at North American institutions, including Western University (London, ON), the Universities of Florida and Oklahoma, Colorado State University, Texas Tech, and other institutions, has served to identify key extreme wind vulnerabilities and risk reduction measures for non-engineered residential buildings. This work has relied on multiple methods, including wind tunnel, lab and field investigations. As outlined in this document, through the application of relatively simple, inexpensive and largely prescriptive construction measures, a more wind resistant structure can be achieved.

The following sections provide an introduction to the issue of extreme wind/tornado exposure in Canada and a discussion of measures related to NBCC Part 9 residential buildings that are expected to increase resistance to high wind. As discussed throughout this document, vulnerabilities may result from design approaches that do not incorporate high wind risk, as well as construction and inspection issues that lead to defects in key structural systems. It has been further noted that, in some instances, code officials may be unable to verify that specific code requirements have been implemented (e.g., proper toe-nailed RTWCs for certain residential building types). Additional measures related to protection of homes from relatively minor wind damages that increase exposure of buildings and contents to water damage (e.g., loss of roofing material) are also discussed here.

Measures presented in this document serve to reduce risk from high wind events caused by tornadoes, hurricanes, and other extreme weather events. As reported by the US Engineered Wood Association:

Whether caused by a tornado or a hurricane, high wind forces travel through the load path of a structure. Good connections that tie the floor, walls and roof together provide continuity in the load path and more reliable building performance.¹¹

Figure 2: Structural roof failures (Angus, Ontario 2014)



Source: Kopp, G. 2014. Presentation to ICLR

2. Issue Background

High wind regions identified in the NBCC include populated areas of Newfoundland and southwestern Alberta (where $q_{1/50} \geq 0.8$); however, a significant area of the country is prone to EF2-EF5 tornadoes (Figure 1).¹² During the period 1980 to 2009, it was estimated that roughly 60 to 70 tornadoes occurred on average each year in Canada, though it is recognized that many tornadoes occur in remote areas and are unreported. Estimations of tornado occurrence based on statistical analysis, population and meteorological factors suggests that as many as 150 to 230 tornadoes may occur on average each year.¹³ Increasing population densities and expansion of development into tornado prone regions will increase the likelihood that human populations will encounter tornado events.¹⁴

Given substantial engineering resources available for wood-frame, low-rise homes in North America, it is considered possible to strengthen wood-frame construction to reduce impacts of extreme wind events.¹⁵ Specifically, designing and building wood frame homes to resist EF2 tornadoes is reasonable given current wood frame home construction techniques in North America. For example, the Oklahoma Uniform Building Code Commission has adopted provisions that provide "...prescriptive based requirements for construction [residential structures] meeting or exceeding 135 mph [217 km/h] wind event corresponding to an EF2 tornado rating."¹⁶

Adoption of measures that provide resistance up to EF2 tornado damage would serve to significantly reduce tornado damage risk, since well over 90% of tornado events in Canada have been EF2 or less.¹⁷ Further, much of the damage that occurs during tornado events does not result from a "direct hit" with the highest intensity portions of tornado tracks – many buildings located on the periphery of tornado tracks are damaged by less intense winds. Thus, application of measures that would reduce risk from lower intensity tornadoes (EF2 or less) can serve to mitigate damage in the periphery of high intensity (EF3 and higher) tornado tracks.¹⁸

Tornado wind speeds are provided in Table 1. In Part 9 of the 2015 NBCC, low to moderate wind forces are associated with $q_{1/50} < 0.8$ kPa, high wind forces are associated with $0.8 \leq q_{1/50} < 1.2$ and extreme wind forces area associated with $q_{1/50} \geq 1.2$ kPa. As discussed by Gavanski *et al.* 2014, an Hourly Wind Pressure (HWP) of 0.8 kPa corresponds to a "...1-in-50 year annual maximum hourly-mean wind speed of 36 m/s (130 km/h), which represents a 3-second gust speed of about 200 km/h (120 mph)." Gavanski *et al.* 2014 further noted that an HWP of 1.2 kPa corresponds to a 3 s wind gust speed of ~240 km/h (150 mph). It was also noted that many of Canada's larger cities have $q_{1/50}$ in the range of 0.4 kPa (90 km/h) to 0.5 kPa (100 km/h), representing 3 s gusts of roughly 140 to 160 km/h (86-97 mph).¹⁹

Table 1: EF-Scale Wind Speeds²⁰

| EF-Scale rating | Wind speed (km/h)* |
|-----------------|--------------------|
| 0 | 90-130 |
| 1 | 135-175 |
| 2 | 180-220 |
| 3 | 225-265 |
| 4 | 270-310 |
| 5 | 315 or more |

* 3 s gust wind speeds at 10 m in open terrain.²¹

Table 2: 2015 NBCC 1-in-50 Hourly Wind Pressure and Corresponding 3 s Wind Gust Speeds²²

| $q_{1/50}$ (kPa)* | Corresponding 3 s gust speed | |
|-------------------|------------------------------|-----|
| | km/h | m/s |
| 0.40 | ~140 | 40 |
| 0.50 | ~160 | 44 |
| 0.80 | ~200 | 56 |
| 1.20 | ~240 | 67 |

* $q_{1/50}$ wind pressures at 10 m height in open terrain.

EF-Scale wind speeds are estimated based on post-event damage assessments, using a large set of damage indicators. Thirty-one damage indicator tables have been published for Canada, which cover a range of infrastructure and features that may be affected by tornado events (e.g., barns, schools, high-rise buildings, service station canopies, trees, electrical transmission lines).²³ Table 3 provides a subset of damage indicators used in Canada to estimate wind speeds based on damage to one- and two-storey homes.

Table 3: Degree of Damage (DOD) Descriptions and Expected Wind Speeds for One- and Two-Family Residences (100-500 m²), as Adopted in Canada²⁴

| Degree of damage | Damage description | Expected value (km/h)* | Lower bound (km/h)* | Upper bound (km/h)* |
|------------------|--|------------------------|---------------------|---------------------|
| 1 | Threshold of visible damage | 105 | 85 | 130 |
| 2 | Loss of roof covering material (less than 20%), gutters and/or awning; loss of vinyl or metal siding | 125 | 100 | 155 |
| 3 | Broken glass in doors and windows | 155 | 125 | 185 |
| 4 | Uplift of roof deck and loss of significant roof covering material (20% or more); collapse of chimney; garage doors collapse inward; failure of porch or carport | 155 | 130 | 185 |
| 5 | Entire house shifts off foundation | 195 | 165 | 225 |
| 6 | Large sections of roof structure removed; most walls remain standing | 195 | 165 | 230 |
| 7 | Exterior walls collapsed | 210 | 180 | 245 |
| 8 | Most walls collapsed, except small interior rooms | 245 | 205 | 285 |
| 9 | All walls collapsed | 275 | 230 | 320 |
| 10 | Destruction of engineered and/or well-constructed residence; slab swept clean | 320 | 265 | 355 |

* 3 s gust wind speeds at 10 m in open terrain.²⁵

The Degree of Damage (DOD) scale accounts for construction quality and includes upper and lower bounds of wind speeds associated with specific types of damage (Table 3). The expected wind speed values are associated with “traditional” construction quality, use of appropriate building materials, no noticeable “weak links” (discontinuities in the load path), compliance with local building codes and appropriate maintenance of the structure. Weak links may include inadequate fastening of roof sheathing, marginal RTWCs and inter-storey connections, and/or inadequate anchoring to the foundation.²⁶ For more information on the EF-Scale and to review the full set of damage indicators used in the assessment of potential tornado wind speeds, the reader is referred to WSEC 2006²⁷ and Environment Canada 2014.²⁸

2.1 Extreme Wind Damage of Non-Engineered Residential Buildings

Recent investigations following tornadoes have illustrated that relatively new homes are not exempt from damage associated with extreme wind events.²⁹ Observations following these events have indicated that RTWCs are often the weakest link in the vertical load path for wood-frame homes.³⁰

On August 20, 2009, a total of 19 tornadoes occurred across southern Ontario – the most prolific outbreak of tornadoes recorded in Canada to date. EF2 tornadoes struck both the communities of Woodbridge and Maple, located in Vaughan, ON. Media reports suggested that hundreds of homes were damaged during this event.³¹

A damage survey conducted by Western University researchers focused on 92 homes. Major structural damages observed included damage to masonry walls, wood framed roof structural members and sheathing, as well as shingle loss, and soffit and fascia failures. It was estimated that 40 of the 92 observed homes experienced major structural damage. Thirty of these homes experienced failures of RTWCs, 27 of which experienced loss of major portions of the roof. Ten additional homes experienced sheathing failure, with some homes losing 50% of their roof sheathing.³² It was estimated based on observed damages that gust speeds during this event were in the range of 56 m/s (~200 km/h), corresponding with an EF2 tornado event.³³

It was noted that two houses, side-by-side, experienced drastically different degrees of damage (Figure 3). These homes had plywood roof sheathing and were constructed at a time when RTWCs were consistent with current (2012) OBC requirements. Further details about the side-by-side homes:

- Both homes faced an open expanse in the direction from which the tornado travelled,
- Both homes were of similar shape,
- Both homes had roof overhangs above the porch on the windward side of the house, increasing potential for uplift, and
- Columns of the porch overhangs on both houses had support columns that likely added little uplift resistance, based on building code provisions at the time the homes were constructed.

Figure 3: Side-by-side damages – one home with damages consistent with DOD 1 or 2, the next with a global roof failure, consistent with DOD 6 damages³⁴



Images: Morrison, M.J., Kopp, G., Gavanski, E., Miller, C., and Ashton, A. 2014. Assessment of damage to residential construction from the tornadoes in Vaughan, Ontario, on 20 August 2009. *Canadian Journal of Civil Engineering*, 41, 550-558.

The primary difference between the two buildings was that there was a breach in the windward wall of the home that experienced loss of roof (specifically, the front doors blew in, determined by a discussion with the owners). The breach in the windward wall of the building (failure of double doors) would have increased net roof loads substantially in comparison with the neighbouring home, which did not experience a breach. The authors also noted that one of the roof trusses from the home that experienced total roof failure, recovered from the property near the home, had fewer toe-nails than required by code (see Figure 4). It was not clear, however, if all RTWCs in this particular home had missing toe-nails, though the authors argued that it was possible, based on previously conducted fragility assessments, that missing toe-nails on each of the connections could have significantly reduced the wind speed necessary to cause the roof failure on the home.³⁵

Breaches in buildings result in internal pressurization, contributing to risk of structural damages.³⁶ Observations following the Vaughan, Ontario tornadoes indicated that breaches in large windows and garage doors caused by flying debris were linked to structural roof failure. A “domino effect” of flying debris was noted:

When large openings are present in the building envelope, such as broken windows or doors, there can be internal pressurization of the structure, which can lead to substantial increases in the net wind loads on roofs... These increased loads can cause structural failures in buildings that might otherwise have remained undamaged.... This can then lead to additional debris becoming entrained in the wind field, which can cause further damage down wind. Such a chain of events was observed in Vaughan.³⁷

While it was noted that houses constructed according to NBCC Part 9 would not be expected to resist wind speeds experienced during the Vaughan event, it was argued that damage could have been considerably mitigated by enhanced RTWCs. Specifically, garage door failures caused by wind pressure and flying debris were considered a driver for damages, including roof failure. Of the 20 homes that experienced “dominant openings” (an opening of ~2% of the surface area of the wall or greater), 17 were associated with garage door failure, and 11 of these 17 homes experienced major roof failures. Non-integral garages (i.e., where the garage is not integrated into the main structure of the house with an upper storey/living space above the garage) were considered particularly vulnerable to roof failure, due to their small volume and failure of garage doors. It was noted that garage roofs would benefit from enhanced RTWCs because “... the large area of the door, coupled with the relatively small internal volume, will lead to particularly large internal pressures.”³⁸

Figure 4: Incorrect roof-to-wall connection (Vaughan, 2009)



Source: Kopp, G. 2014. Presentation to ICLR

Figure 5: Wall failure indicating issues with construction (Angus, 2014)



Source: Kopp, G. 2014. Presentation to ICLR

The June 17, 2014 Angus, Ontario EF2 tornado exemplified extreme wind damages for new home construction, as many of the homes damaged during this event were less than three years old. A total of 101 homes experienced some level of damage, ranging from shingle and siding failures to wall and roof structural failure. Eleven (10%) of the damaged homes lost their roofs. Ten of these roofs became completely detached, blowing off of the structure and impacting neighbouring homes. Nine homes experienced structural wall damage. It was further noted that all homes with complete roof failures had broken windows on one wall. Nine garage doors were observed to have failed, likely due to wind pressure.³⁹ Tornado translation speed was estimated to be 65 km/h (18 m/s), and damage width was estimated as ~200 m. Given tornado translation speed and damage width, it was estimated that high wind duration, beyond the threshold of damage for any given location, was maximum ~10 s.⁴⁰

Post-tornado and hurricane damage investigations in Canada and elsewhere have revealed recurring construction errors that have contributed to vulnerability of wood frame construction to high wind events. These errors include missing RTWCs and missing roof sheathing fasteners. Similarly, following the Angus tornado, the post-storm field survey indicated that “much of the structural roof and wall damage was associated with poor construction quality caused by missing toe-nails in the roof-to-wall connections and nails in the inter-storey wall-to-floor connections,” and that almost all of the toe-nailed RTWCs identified following the Angus tornado “...were below code requirements, with cases of zero, one, and two nails in the connections, rather than the code-required three.”⁴¹ Regardless of the existence of improper RTWCs and wall-to-floor connections observed during the Angus tornado inspections, damages corresponded with an EF2 tornado event.⁴²

As discussed by Morrison *et al.* 2012,⁴³ damaged RTWCs may not always be readily observable from the inside or outside of a home. For example, partial withdrawal of toe-nailed RTWCs (i.e., separation between top plates and rafters/trusses), may result in only hair-line fractures at the wall-to-ceiling drywall joint, with no observable evidence from the outside of the home. Therefore, there may have been more failed RTWCs than those observed during the abovementioned damage inspections.

With respect to tornado damage risk reduction, CWC 2014 provides the following commentary:

The wind design provisions in the National Building Code of Canada and the Engineering Guide for Wood Frame Construction are intended to simulate peak gusts in storms having a 1-in-50 probability of occurring every year. Studying the damage from hurricane force winds in other parts of the world provides insight into how wood frame construction behaves under high wind loads. Similar forms of damage have been reported in wood frame houses exposed to the direct paths of tornadoes.... The damage to housing in hurricanes and tornadoes shows that:

- 1. Sheathing attachments, gable end details and attachment of roof framing to walls are critical...*
- 2. When it occurs, structural damage is usually related to the roof system. Damage to walls and foundations are rarer...*
- 3. Major damage occurs at gradient wind speeds of 70 m/s or greater and minor damage occurs at gradient wind speeds below 50 m/s...⁴⁴*

Structural damages to Part 9 residential buildings may be attributed to lower wind speeds than those described above. For example, “first damaging peaks” observed during fluctuating load tests on toe-nailed RTWCs may occur at wind speeds of 25 m/s.⁴⁵ Further, the potential for flying debris, causing breaches in building envelopes, should be factored into decision making related to RTWCs in the context of extreme wind events. Breaches in the building envelope result in internal pressurization, contributing to roof failure (see Appendix A).

Roof structure failure is usually the precursor to wall collapse,⁴⁶ and roof failure causes downwind debris impacts on adjacent buildings. Further, common construction errors, including lack of installation of all required roof sheathing fasteners or fasteners that “miss” roof trusses, may result in sheathing failure during extreme wind events. These issues have been identified in several areas of Canada, including the Vaughan, 2009 and Bornham Ontario, 2007 tornadoes.⁴⁷

Literature on fatalities and injuries during tornado events in North America indicates that structural collapse is a major cause of serious injury and death, and the most frequently reported injuries are associated with flying debris.⁴⁸ Forensic analyses conducted in eastern Canada have revealed “... buildings in which more than 90% of the occupants were killed or seriously injured did not have anchorage of house floors into the foundation or anchorage of the roof to the walls.”⁴⁹ Canadian construction codes include optional provisions for use of anchor bolts to tie building frames to foundations.⁵⁰ Field observations and input from code officials have indicated, however, that when anchor bolts are applied for anchoring of frames to foundations, nuts and washers might not be installed by accident/omission. Shrinkage of the sill plate may also result in loosening of the connection (i.e., loosening of the nut). Ensuring that nuts and washers are in place is a critical aspect of ensuring life safety during tornado events. Further, as discussed in Appendix A, anchoring options including those that rely only on embedding members in concrete result in discontinuity in the vertical load path.

3. Overview of Wind Risk Reduction Measures

As discussed above, this document provides preliminary extreme wind risk reduction measures applicable to NBCC Part 9 residential, wood-frame buildings. The measures are adapted from those produced as part of the development of the Durham Region Climate Resilience Standard for New Houses (hereafter referred to as the Durham Standard – see Figure 6).⁵¹ High wind resistance for homes constructed according to Canadian construction codes remains an area of active research. While this report provides guidance on mitigating high wind risk, it is expected that best/recommended practice will evolve over time with improved knowledge, including collection of data from the field (damage surveys) and lab testing of building components constructed according to Canadian construction codes.

Figure 6: Relationship between Durham Standard and the Wind Seed Document



Measures presented in the report are intended to be relatively straightforward, achievable, and allow for consumer choice. With an emphasis on roofs and roof structures, measures are intended to assist in reducing structural damage associated with EF0 to EF2 tornadoes, which comprise the vast majority (>90%) of tornado events in Canada. As such, measures presented here are not meant to address risks associated with tornadoes rated/categorized as EF3 or higher, though they may assist in reducing damage in the periphery of these tornado events where wind speeds are lower.

Measures presented here are based on research concerning the performance of wood-frame buildings in high wind events, investigations following damaging wind events and tornadoes in Canada and North America, best practices applied in North America, practical experience, and input from an expert Stakeholder Committee (see Appendix C). Many of the measures are already included in Canadian and provincial building codes, but may not be routinely applied to non-engineered residential construction in all regions exposed to high wind events (e.g., regions prone to tornadoes). Stakeholder Committee members involved in this project further emphasized the importance of ensuring that wind risk reduction measures support other progressive Part 9 residential construction initiatives, including those related to energy efficiency, where possible.

CWC 2014 is referenced in the NBCC as a guide on “good engineering practice,” and is considered an alternative to constructing buildings in accordance with provisions in Part 4 of the NBCC.⁵² CWC 2014 provides multiple design details for Part 9 buildings, including construction detail to accommodate exposure to extreme wind. Both CWC 2014 and the NBCC highlight the need for engineering input when applying CWC 2014. Specifically, NBCC Appendix 9.4.1.1 states:

Design according to Part 4 or accepted good engineering practice, such as that described in CWC 2014 ‘Engineering Guide for Wood Frame Construction,’ requires engineering expertise. The CWC Guide contains alternative solutions and provides information on the applicability of the Part 9 prescriptive structural requirements to further assist designers and building officials to identify the appropriate design approach. The need for professional involvement in the structural design of a building, whether to Part 4 or Part 9 requirements or accepted good practice, is defined by provincial and territorial legislation.⁵³

In contrast to the provisions outlined in CWC 2014, the measures presented in Appendix A of this document are prescriptive (wherever possible) and are intended to be applicable and understandable by non-professionals and the public, including home-buying consumers. Further, there are notable technical differences between the measures presented here and those contained in CWC 2014 and the 2015 NBCC. Some of these differences include spacing of nails for intermediate supports, post base and cap connections, application of engineered RTWCs, and water penetration protection for roof decks, among other details (see Appendix A).

Measures related to non-structural components of homes discussed in the report include strategies for ensuring the roof remains sealed during a wind event (including provision of high wind-rated shingles and/or a secondary water barrier to protect the home in the event of roof covering failure), and provision of garage doors capable of resisting high winds. Other non-structural components of homes – including siding and windows – are not addressed in the report. Therefore, damage to windows, doors, siding, property outside of the home, as well as damage associated with falling trees/tree limbs, will not be mitigated through the application of measures described here. Further, because of multiple and complex site and building factors, the application of measures described in the report does not guarantee that homes will not be damaged during high wind events, including EFO to EF2 tornado events.

Measures presented here are meant to illustrate approaches that can generally increase resistance of low-rise/Part 9 buildings to high wind events. Alternatively, buildings can be designed according to NBCC Part 4 or good engineering practice based on high wind exposure (e.g., with minimum $q=0.8$ kPa replacing the specified velocity pressure from NBCC Appendix C for the given location), and/or the risk tolerance of homeowner, builder, or other concerned stakeholders.

Tiered Approach

As indicated in Figure 1, the NBCC identifies few areas of the country exposed to high wind hazards. The map provides an indication of where wind risk reduction measures may be appropriate; however, given varied exposure to wind hazards across Canada, a “tiered” approach related to application of measures may be appropriate. Basic measures (specifically, improved RTWCs and measures related to ensuring that roof covering remains in place), which can be completed at relatively low cost and address a recurring or high impact issue, may be applied throughout the country. Additional measures, including bracing options, would be applicable in regions exposed to higher wind hazards.

An issue with defining regions of the country where specific measures may be appropriate is availability of reliable data on wind hazards. For example, members of the Stakeholder Committee indicated that the tornado occurrence map presented in Figure 1 likely presents a bias toward more heavily populated areas where tornadoes have been observed and recorded. It is recognized that there are large regions in Canada where tornadoes are expected to occur, but that there are gaps in tornado observation data.

Ongoing research on tornado occurrence will shed additional light on wind hazards in Canada. For example, to understand occurrence of tornadoes in remote areas, the Northern Tornadoes Project will rely heavily on radar data analysis for storm prediction, and aerial photography for tornado path identification. As tornado data improves through projects like the Northern Tornadoes Project, it is expected that a larger area of the country may be considered exposed to tornado hazards. It was further noted that the National Research Council of Canada is pursuing updated climate data for the NBCC, which may be affected by updated wind hazard data. Availability of revised data will affect application of wind resistance measures and benefit-cost assessments.

Table 4: Overview of Wind Risk Reduction Measures. This table provides a high-level overview of measures presented in Appendix A. The reader should refer to Appendix A for additional detail.

| Measure/sub-measure | | | Overview | Type |
|--|-------------------------------------|---|---|------|
| Roofs and roof framing | Extreme wind resistant roof framing | A.1.1 | • Preference for hip roofs framed with pre-fabricated, engineered trusses. | 2 |
| | | | • Bracing and securing of gable end walls (where used). | 2 |
| | | | • Engineered roof framing where prefabricated, engineered trusses are not used. | 2 |
| | Roof-to-wall connections | A.1.2 | • Tie roof rafters, roof trusses, or roof joists to load-bearing wall framing in a manner that will result in a factored uplift load of 3 kN. | 1 |
| | | | • When engineered connectors are used, builders should request that truss manufacturers supply appropriate roof-to-wall connections along with trusses. | 4 |
| Roof sheathing, fasteners and fastener spacing | A.1.3 | <ul style="list-style-type: none"> • 11.1 mm (7/16") structural plywood or OSB roof sheathing. • Fasten with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails. • Fasteners spaced 150 mm o.c. on edge and intermediate supports. | 1 | |
| Sealing of roof deck | A.1.4 | • Application of measures to ensure that roof deck remains sealed, which may include application of shingles rated for high wind and/or application of measures to seal the roof deck. | 1 | |
| Walls | Bracing | A.2.1 | • Bracing of walls to resist lateral loads associated with high wind events. | 2 |
| | Floor-to-floor connections | A.2.2 | • Wall framing for upper and lower storeys should be connected to facilitate continuous vertical load path. | 2 |
| | Connections to sill plates | A.2.3 | • Connecting walls to sill plates to facilitate continuous vertical load path. | 2 |
| | Wall sheathing and fasteners | A.2.4 | <p>Where wood structural panels are applied for continuous load path, measures include:</p> <ul style="list-style-type: none"> • Continuous sheathing of exterior walls with structural wood sheathing. • Use of minimum 11.1 mm (7/16") plywood or OSB wall sheathing. • Fasten with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails. • 150 mm (6") fastener spacing along edges and intermediate supports. | 2 |
| Anchoring of building frames | | A.3 | • Anchorage of building frames should contribute to the continuous vertical load path. | 2 |
| Post base and cap connectors | | A.4 | <ul style="list-style-type: none"> • Use post base and cap connectors rated for 6.8 kN uplift. • Embed or fasten post base connections to concrete slabs for front and rear porch applications. • Use corrosion-resistant fasteners for post base connections. • Connectors should be visible for inspection. | 2 |
| Garage doors | | A.5 | • Garage doors rated for 200 km/h (125 mph) or above. | 3 |

Reflecting the above discussion, measures presented in this report are further presented in four categories. These categories include:

- Type 1: Measures that may be applied in any region of the country,
- Type 2: Measures that may be applied in higher hazard areas,
- Type 3: Measures that are considered optional, and
- Type 4: Measures that are outside of the scope of building codes, and relate to inspections, operational and management decisions.

Measures in Table 4 are presented in a “top-down” order, beginning with considerations for roofs and ending with anchoring to foundations. Table 4 provides a high-level overview of measures presented in the document. Detailed information on the measures, and discussion of purpose and benefits of the measures are provided in Appendix A and should be referred to by the reader. Where appropriate, 200 km/h has been selected to represent wind speeds associated with moderate EF2 tornado wind speeds in this report. Based on the best available knowledge, this figure represents gust speeds that would serve to reduce the structural impacts of the vast majority of damaging tornadoes in Canada.

A number of construction details that have been incorporated into practical wind risk reduction guidance documents elsewhere were not included in this document (e.g., protection of windows, entranceways). These details were not included where existing NBCC requirements were considered adequate, where the measures have not been identified in field and lab tests as critical vulnerabilities in home construction, or where Stakeholder Committee input indicated that benefits of the measures would not outweigh costs. Further, garage doors were included as an optional measure, as Stakeholder Committee members identified several logistical issues with application of this measure, and risk associated with garage door failure may be largely mitigated through improvements to garage continuous vertical load paths (see Appendix A).

While the majority of measures presented in Appendix A are prescriptive, performance measures are included where appropriate (e.g., options related to roof and wall design). This approach was applied due to the potential complexity of framing where pre-fabricated trusses are not used, and complexity with respect to bracing performance based on building design. Specifically, Technical Committee members noted during development of the Durham Standard that pre-fabricated trusses may not be used to frame custom home roofs, as they may limit opportunities for creation of unique roof designs. Further, there exists limited, widely applied prescriptive guidance on conventional (stick) roof construction to resist extreme wind forces.

3.1. Partially Constructed Homes

Work is currently underway at Western University to assess wind damage to residential structures observed during construction. Structural failures have been observed across several events, occurring at wind speeds not exceeding design wind speeds. Observed failures have occurred primarily through racking of first floor walls, where varying amounts and types of sheathing have been installed and roofs have been erected and sheathed. Wall failures, as such, are not predominant in complete homes and have been seldom observed in post-storm damage surveys following extreme wind events.

Figure 7: Wind damage to partially constructed homes in Waterloo (2017), Ontario. The image illustrates an intact roof resting on failed walls during construction.



Detailed analysis of partially constructed homes under high wind loads will include assessment of the critical load cases, to be applied to structural models representing different levels and types of lateral bracing. Current construction practice sees more houses being constructed without external structural sheathing, with rigid insulation being installed to the outer face of stud walls instead. Once these houses are complete, interior drywall acts as the stiffening membrane to provide horizontal load resistance; however, until construction is complete the stud walls themselves are insufficient in providing lateral resistance to large wind gusts. The effect of the wind on the lateral loading is also expected to be increased during different stages of construction, due to increased drag on exposed members or the increase of wind loaded area caused by sheathing the roof structure or applying rigid insulation to the walls.

Preliminary assessment of expected wind loads on partially constructed homes has identified that critical load cases are expected to occur when the roof is complete, but walls are either bare (studs exposed), partially sheathed with insulation, or fully sheathed with windows installed (enclosed). Calculation of the relevant wind pressures under each case will be carried out according to ASCE 7-16, with additional load factors introduced to account for the level of enclosure.⁵⁴

The second phase of this work is to model walls under in-plane loading. Experimental data representing the behaviour of nailed connections and shear walls has been obtained from the literature and will be used to approximate the relative strength and stiffness of sheathed and unsheathed walls. The critical wind loading cases will be identified and methods for bracing unsheathed walls will be tested and compared numerically.

It is recognized that mitigation of damage risk for partially constructed homes lies outside of the scope of the NBCC. A mechanism for implementing wind risk reduction measures in partially constructed homes may include agreements and contracts between insurers and builders. A similar approach has been recommended for fire safety on residential construction sites.⁵⁵

3.2. Discussion: Impact Assessment

Implementation costs and impacts were a central consideration in the identification of measures in this report. As discussed above, several measures that are commonly recommended for wind risk reduction were not included in this document due to the potential for unfavourable benefit-cost ratios in much of the country,⁵⁶ and where enforcement may be considered impractical. For example, due to practical/implementation issues, high wind resistant garage doors are considered an optional measure (see discussion in Appendix A).

Further to the above discussion, an initial, high-level assessment of costs and qualitative benefits of identified measures is provided here. Included in this high-level assessment are relevant examples from jurisdictions applying various wind risk reduction measures and cost estimates from available datasets. In several instances contractors and suppliers were consulted to assess a potential range of implementation costs. Benefits and purposes of each of the proposed measures are summarized in Table 5 (see Appendix A for additional detail and discussion). In addition to the benefits identified below, each of the measures, aside from garage doors and sealing of roof decks, directly contribute to the continuous vertical load path.

Measures including use of engineered trusses for roof framing (Measure A.1.1), use of continuous structural/wood sheathing (Measure A.2.4), anchoring the structure to the foundation using bolts (Measure A.3), and overlapping/fastening rim joists and sill plates with wall sheathing to enhance the continuous vertical load path (Measures A.2.2 and A.2.3) were considered common practice in Part 9 construction in many instances, and would likely not contribute substantially to the cost of construction. Hip roofs are a design element (Measure A.1.1), and therefore do not contribute incrementally to the cost of construction. Requesting that connections be supplied along with engineered trusses (included as part of Measure A.1.2) may require education and improved communication between the building industry and suppliers, but would likely not result in incremental construction costs.

Consultations with home building industry stakeholders have indicated that above-grade walls are increasingly being sheathed with continuous exterior insulation products, often without structural exterior sheathing. In these instances, addition of wood structural panel sheathing would add to the cost of construction.⁵⁷ Additional sheathing may also result in increased foundation wall thickness, further contributing to cost of construction. As identified in Appendix A, alternative methods of enhancing the continuous vertical load path are available, including use of straps and proprietary truss screws. Bracing measures outlined in NBCC 9.23.13* (see sidebar) provide options that do not rely on exterior wood structural panel sheathing, though wood structural panel sheathing is a widely recommended method for increasing lateral and uplift resistance for high wind loads.

With respect to nailing patterns for sheathing (Measures A.1.3 and A.2.4), a change was made to the 2012 OBC, such that 150 mm x 150 mm nailing patterns are required across the province for roof sheathing (where support spacing is 406 mm o.c. or greater). The code change request presented by the Ontario Ministry of Municipal Affairs and Housing indicated that the proposal would "...lead to a minimal cost increase in the construction of new buildings" (see Appendix E) and the change was accepted for OBC Part 9 construction.

Estimates related to incremental costs of increasing sheathing thickness (Measures A.1.3 and A.2.4) ranged from no change to \$0.08/ft².⁵⁸ Engineered roof framing (i.e., in the case of "stick-framed" roofs) (see Measure A.1.1) may add to the cost of construction; however, it was also noted that stick-roof framing is typically applied in custom homes, so the incremental cost may not be significant. Reinforcement of gable end walls (Measure A.1.1) would add to the cost of construction, though material costs are considered to be low (i.e., options presented in the report include use of metal straps, 2 x 4s for bracing). Use of more resistant (and visible) post base and cap connections (Measure A.4) meeting the specific uplift loads identified in this report may add marginally to the cost of construction.⁵⁹

*While NBCC 9.23.13 may imply that options other than wood sheathing on exterior walls are permitted, CWC 2014 (Part C) does not permit sheathing other than wood, recognizing the significant difference in strength resistance between OSB or plywood and other materials. (Pers. Communication, R. Jonkman, CWC, January 2019)

With respect to application of wood sheathing to increase lateral and uplift resistance, see also:

American Wood Council. 2013. Guide to Wood Construction in High Wind Areas for One and Two-Family Dwellings. 130 MPH Exposure. B. Leesburg, VA: American Wood Council.

APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light Wood Frame Construction. Tacoma, WA: APA – The Engineered Wood Association.

City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.

Insurance Institute for Business and Home Safety (IBHS). 2015. High Wind Standards. Tampa, FL: Insurance Institute for Business and Home Safety.

Table 5: Benefits of Proposed Measures

| # | Measure | Qualitative Benefits |
|-------|--|---|
| A.1.1 | Extreme wind resistant roof framing | Relative to the remainder of the structure, roofs and roof structures experience the greatest forces during high wind events. Measures include application of roof framing, roof design, and sheathing measures to increase the resistance of roof structures to forces associated with high wind events. |
| A.1.2 | Enhanced roof-to-wall connections (RTWCs) | Roof failures during high wind events typically begin at the RTWCs. By adequately connecting rafters, joists or trusses to wall framing, the resistance of the connection to uplift forces during windstorms is increased, decreasing the risk of structural damage. This measure also provides resistance to internal pressure in the event of envelope breaches during high wind events. |
| A.1.3 | Roof sheathing, fasteners and fastener spacing | Increased sheathing thickness provides improved roof bracing. Additionally, the fastener measures reduce risk of sheathing panel failure during high wind events, as sheathing is better able to withstand uplift forces. |
| A.1.4 | Sealing of roof deck | It has been previously identified that much of the damage caused to residential buildings during extreme wind events results from water penetration into buildings. This measure would offer enhanced protection to the building from water damage by reducing risk of roof covering failure and/or providing additional protection in the event of roofing failure (e.g., when shingles are blown off). |
| A.2.1 | Bracing | Bracing to resist lateral loads associated with high wind events. |
| A.2.2 | Floor-to-floor connections | Contributes to continuous vertical load path. |
| A.2.3 | Connection between walls and sill plates | Contributes to continuous vertical load path. |
| A.2.4 | Wall sheathing and fasteners | Continuous structural sheathing provides improved bracing and assists in achieving continuous vertical load path measures. Fastener spacing at 150 mm along edges and intermediate supports increases resistance to negative wind pressure. The measure assists in enhanced RTWCs (Measure A.1.2) by securing the top plate to the load-bearing wall and transferring loads to the foundation. |
| A.3 | Anchoring of building frames | Contributes to continuous vertical load path. |
| A.4 | Post base and cap connectors | By adequately attaching porch roof support beams to their posts, and posts to their foundation, the resistance of the posts to uplift forces during windstorms is increased and the risk of structural damage is decreased. Currently, porch columns are often toe-nailed to foundations, which provides insufficient uplift capacity. Further, use of visible connectors (e.g., connections that extend above the base of posts) increases the ability to inspect post base and cap connections. |
| A.5 | Garage doors rated for high wind | Reduces risk of creating a “dominant opening” during high wind events, which results in pressurization of building and contributes to roof failure. Considered an optional measure. |

With respect to the optional high wind rated garage door measure (Measure A.5), observations following the implementation of wind safety provisions in Moore, OK indicated that high wind rated garage doors (rated to 135 mph) resulted in a roughly \$600 increase in construction costs.⁶⁰ Consultations with suppliers and installers in Canada and the US revealed a range of potential prices for high wind rated garage doors, from ~\$600 to ~\$1,200 (standard non-insulated 9' x 7' single garage door, rated for 130 mph, based on a range of four different models from two Canadian suppliers and one supplier in Moore, OK). A widely used builder costing database indicated that average costs for a standard, sectional overhead 9' x 7' garage door would be ~\$1,200.⁶¹ Another Canadian supplier estimated that wind rating would likely add ~\$200 in hardware alone. Additional Canadian suppliers were unable to provide quotes due to lack of experience with installation of high wind rated doors in Canada.

Tiered Approach

Cost-benefit analyses have been conducted for similar sets of wind risk reduction measures in other regions of North America. Studies have found various levels of cost efficiencies, depending on wind hazard exposure.⁶² For example, an analysis of the Moore, OK tornado risk reduction provisions, as applied across the state of Oklahoma, was conducted by Simmons *et al.*⁶³ The analysis was based on an expected cost of \$1.00/ft² (\$10.75/m²) for mitigation measures, average home size of 2,000 ft², an expected home lifespan of 50 years, and a 2.5% discount rate. The authors identified a payback ratio for the measures of 3.2:1. A number of meetings were held with home builders in Moore before the local code changes were made, and it has been reported that resistance to the code changes has not been significant.⁶⁴

Sutter *et al.*⁶⁵ assessed costs and benefits associated with application of four prescriptive tornado damage reduction measures in an Oklahoma case study. The measures included anchoring of the structure to the foundation using anchor bolts, hurricane ties, altered roof nailing patterns, and use of OSB sheathing. The total estimated cost for the measures was ~\$500, based on the experience of an Oklahoma builder who had been applying these measures for roughly 10 years. The authors noted a previous study that estimated costs of altered nailing patterns and use of hurricane ties at \$100 in South Carolina.⁶⁶ In order to meet the cost-benefit threshold, it was estimated that the package of risk reduction measures should result in a 30% to 50% reduction in damages (i.e., positive net present value). The results indicate the mitigation measures were economically viable in tornado prone states.⁶⁷ Further, the recent US Multihazard Mitigation Council report of benefit cost ratios (BCR) for natural hazard mitigation measures revealed an average BCR of 5:1 for hurricane wind risk reduction measures, as developed by the Insurance Institute for Business and Home Safety (IBHS Fortified Bronze and Silver compliance).⁶⁸

Given the potential for varied wind hazard exposure in much of Canada, and further to the above considerations for costs of implementation, a "tiered" approach, including identification of basic and more advanced risk reduction options, is presented in the report (Table 4). Basic measures, which may be completed at relatively low cost and address recurring issues, may be applied throughout the country. These included enhanced RTWCs (Measure A.1.2), and ensuring that roof decks remain sealed during high wind events (Measure A.1.4). Additional measures identified in the report (e.g., bracing, anchoring options) would be applied in regions exposed to higher wind hazards.

Multiple options are available for ensuring that roof decks remain sealed, ranging from application of ice-and-water shield over the entire roof deck, to a lower-cost option of taping seams between sheathing panels (Appendix A). Application of high wind rated shingles may also serve as an option for ensuring that roof covering stays in place during high wind events.

Consultations with nine Ontario roofing contractors suggested a considerable range of incremental costs associated with the installation of laminate shingles rated for higher wind speeds, as costs may vary significantly based on roof characteristics (e.g., design/complexity, shape, pitch, accessibility of roof). Overall, incremental cost estimates ranged from ~\$2 to ~\$33 per square (100 ft²).⁶⁹ A comparison of moderate shingle costs available in a national database suggested an estimated incremental cost of \$63.93 per square (\$0.639/ft²) to install laminate shingles relative to standard strip three-tab (~27% increase in cost).⁷⁰ It was further noted by several contractors that, when homes are being re-roofed, basic, three-tab shingles are rarely used as better warranties are often offered with improved shingle quality, and that laminate shingles are easier to install and may be overall of higher quality.

As an alternative to shingles rated for high wind, providing a secondary water barrier for the roof deck through one of a variety of means would serve to limit the impact of roof covering failure. Ice-and-water shield may be considered an option with respect to performance (costs estimated at \$164.78/square or \$1.65/ft²).⁷¹ A less costly measure includes taping of seams between roof sheathing panels (see Table A.2, Option 1). Consultations with manufacturers and suppliers indicated that, for existing proprietary sheathing systems that incorporate sealing of gaps between sheathing panels,⁷² application of tape is roughly \$0.10/ft² for material and \$0.10 per linear ft for labour. These figures translate to ~\$200 for a typical home with a ~1,200 ft² hip roof.⁷³

There is variability in cost estimates for available methods to enhance RTWCs. Commonly available hurricane ties, which have historically been applied most often for enhanced RTWCs, retail for ~\$0.70 to \$0.80, not including nails required to fasten the tie to the truss and top plate. Other products, including truss screws, may retail for a similar price, and would have the advantages of not requiring nails for installation and reduced installation time. Where raised heel trusses are applied with structural wood sheathing, an enhanced RTWC may be achieved with no incremental cost.

Estimates provided by Dufferin County as part of its Hurricane Clip Rebate Program indicate a material cost per hurricane tie of ~\$1.00,⁷⁴ translating to, on average, \$50 to \$60 material cost per home. The code change proposal presented by Ontario's Building and Development Branch indicated that additional material costs for a typical house would be less than \$200. It was further estimated by builders that labour costs would vary based on the complexity and size of roofs, and would be at minimum \$500 (see Appendix F). Labour costs for home builders may be substantially higher in regions where collective bargaining agreements exist with relevant trades.

Home builders in Dufferin County are provided access to an incentive of \$4.50 per installed hurricane tie to account for labour and material costs.⁷⁵ Considering a scenario where there are 60 RTWCs in a home, the \$4.50 subsidy would translate to a cost of \$270. The former CBO of Dufferin County reported that the program has received a positive response from home builders.

4. Additional Topics Raised by Stakeholder Committee

Detailed notes from the Stakeholder Committee workshop, held at Western University on June 28, 2018, are available in Appendix C. The majority of the discussion and decisions from the workshop were integrated into this report. This section highlights additional topics that were raised during the workshop. Specifically, discussion related to use of the term “resilience,” the importance of homeowner representation on future work related to NBCC Part 9 wind risk reduction work, issues related to scoping and application of the proposed set of measures, and issues related to inspections and enforcement of measures identified in the report are highlighted below.

Resilience: The term “resilience,” as applied in academia, is more nuanced than as presented in initial versions of the report. In practice, and in the context of recent Canadian disaster risk reduction programs, the term is being used widely and interchangeably to represent actions (social, physical, etc.) that are related to climate change adaptation, emergency management, and/or disaster risk reduction/mitigation. The purpose of this document is to contribute to resilience by enhancing the wind resistance of non-engineered, residential buildings in the Canadian context.

Homeowner representation on future committees: No formal homeowner representation was included in the Stakeholder Committee for this project. It was generally acknowledged that homeowner representation on this type of committee would be helpful, and this should be pursued should a National Standard of Canada (NSC) be developed on the topic of wind risk reduction for non-engineered residential buildings. Various strategies to increase homeowner involvement were discussed, including inclusion of knowledgeable homeowners (with no affiliation), knowledgeable homeowners who have been directly affected by damaging wind events, and/or inclusion of a “consumer representative” on future technical committees.

Scoping and application: Currently, the NBCC identifies few areas of the country exposed to high wind hazards. Figure 1 is meant to give an indication of where the example measures presented in this document may be appropriate. It should be noted that current availability of reliable data on wind hazards is limited. Availability of improved wind hazard data would affect application of wind-resistance measures and benefit-cost assessments. It is further noted that regions where $q_{1/50}$ is 1.2 kPa and higher are out of scope, as the NBCC requires that buildings in these regions be engineered. With respect to applicability, Stakeholder Committee members also highlighted a need to clarify that this report is a seed document that may serve as the basis for the development of a voluntary, “code-plus” NSC.

Inspections: The Stakeholder Committee generally agreed that inspection issues should be discussed/highlighted in the report. Specifically, it was noted that all measures outlined in the report can be incorporated into plans that are approved by municipalities, but it is not possible under any scenario for every inspection to be completed. Inspection issues exist for non-structural measures discussed in the report because these elements may not always be included in normal inspection schedules (e.g., specifically, roof deck sealing and garage doors).

Committee members noted that inspections are always difficult to complete – including for existing, basic code requirements. This difficulty, however, should not preclude pursuance of the types of measures identified in the report. It was discussed that one of the most effective means of overcoming inspections issues is to train tradespeople on these types of measures early in their careers (e.g., incorporate into trade school curriculum). Code officials present at the workshop suggested that construction practices are more likely to be applied when trades are aware of the intent and purpose of specific measures.

Appendix A: Wind Risk Reduction Measures

A.1. Roofs and Roof Framing

A.1.1. Roof Structures and Gable End Walls

Preferred option: Hip roofs framed with prefabricated, engineered truss systems.

If prefabricated, engineered trusses **are not used**, it is recommended that an engineered design of the roof be completed following NBCC 2015 Part 4 or the Canadian Wood Council's Engineering Guide for Wood Frame Construction 2014.

And

Where gable end walls are used, they should be appropriately braced and secured to resist extreme wind forces.

Options to Achieve Measure A.1.1:

Preferred option:

- a) Use hip roofs framed using prefabricated, engineered trusses.

Alternative to hip roofs:

- b) Gable roofs should adhere to the following:
 - i) Gable end walls should be tied to the supporting wall assemblies and roof framing. Connections should be present at tops and bottoms of end walls. Options may include steel connection plates or straps.⁷⁶
 - ii) Sheathing and fastener methods for gable end walls should comply with Measure A.2.4.
 - iii) Gable end wall sheathing should fully overlap the top plate of the supporting wall.⁷⁷ Sheathing should be fastened to the top plate with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails using 150 mm (6") o.c. spacing.

Alternative to truss roof framing (performance):

- c) Conventional roof framing should be engineered and capable of resisting wind loads as specified in Part 4 of the National Building Code (NBCC), with (minimum) $q=0.8$ kPa replacing the specified velocity pressure from NBCC Appendix C.

Note: Performance goals and options are provided as part of Measure A.1.1 to allow flexibility with respect to roof design and framing methods. It should be noted that, based on lab and field research results in Canada, hip roofs framed with engineered trusses are considered the preferred roof framing and design method to increase resilience to high wind and tornado events.

Purpose:

- Apply roof framing, roof design, and sheathing measures to increase the resistance of roof structures to forces associated with high wind events.
- Contributes to continuous vertical load path.

Notes:

Engineered trusses:

- Engineered trusses perform better than conventionally framed roof construction during tornado events, as they are capable of accommodating both compression and tension.

Hip roofs:

- Hip roofs with engineered trusses have demonstrated higher resistance to uplift and failure of roof sections.⁷⁸

Engineered roof framing:

- The measure related to roof framing using Part 4 of the NBCC applies where engineered trusses are not used for roof framing.^a

Bracing and fastening gable end walls, structural sheathing for gable end walls:

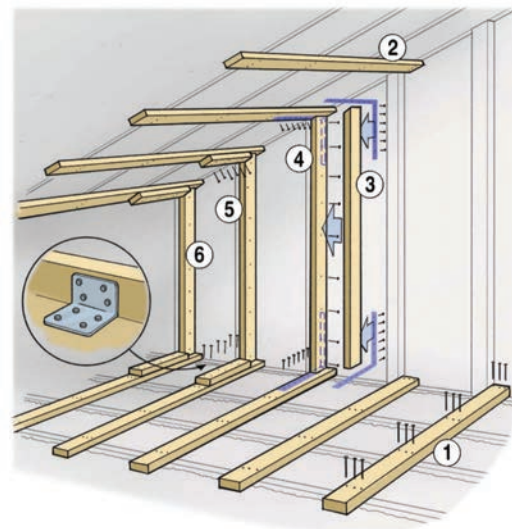
- Fastening of gable end walls results in end walls that are better able to resist suction forces during tornado events.
- Structural sheathing improves bracing and can be applied to contribute to the continuous vertical load path.
- See Appendix B for example methods applicable to Measure A.1.1.b) i).
- See “Gable End Bracing” report for additional information on common vulnerabilities for gable end walls and principles for bracing and securing gable end walls.⁷⁹

Note that gable end wall details provided in Appendix B do not include connections to the roof diaphragm. These connections are not included because these guides assume truss-built roofs. It should be noted that different roof types and configurations (e.g. stick built roofs, open roofs, roofs with dormers, etc.) require different gable end wall bracing and connection options. Additional guidance on these roof types is available from multiple US sources (see Figure A.1).⁸⁰

Roof framing:

- Due to the varied and complex nature of conventional roof framing, roofs should be engineered if they are not constructed with pre-fabricated, engineered trusses.
- The design wind pressure figure presented here (minimum 0.8 kPa) is based on the National Building Code of Canada design wind pressure for a moderate EF2 (200 km/h) tornado. In this instance, a 3 second gust associated with the EF2 tornado event was converted to hourly average wind pressure.

Figure A.1: Example of connecting gable end wall to ceiling and roof diaphragms for an open roof⁸¹



^a It was noted during development of the Durham Standard that a considerable portion of homes in the Greater Toronto Area are custom homes, with unusual/original rooflines and architectural detail that are typically achieved using alternatives to engineered roof trusses. Given the potential for variability in roof design in these scenarios, prescriptive provisions were not considered appropriate.

A.1.1: Discussion and Context

Roofs experience the highest wind loads during extreme wind events and have been repeatedly shown to be vulnerable to the impacts of hurricanes, tornadoes and downbursts.⁸² Sheathing, RTWCs and roofing materials (e.g., shingles) have all been found to be vulnerable to damages during severe storms.⁸³ Roofs are particularly prone to significant structural damage when breaches occur on windward walls, due to pressurization of the building.⁸⁴ Breaches are common during high wind events due to wind-borne debris. Aside from reducing wind speeds at which roofs are likely to fail, breaches causing failure of roof elements significantly increase damages associated with water infiltration – a major driver of property damage during extreme wind events.⁸⁵ Further, roof failures present a life safety risk to occupants, notably in wood-frame structures, as walls are more susceptible to collapse following roof failure.⁸⁶

Prefabricated, engineered truss roofs are considered a preferred means of roof framing. Higher wind speeds (i.e. DOD upper bound limits for DOD 4 and 6 – see Table 3) are required to result in uplift of hip roof deck and removal of sections of hip roofs. Kopp *et al.* 2017 also observed that hip roofs require median 50 km/h higher wind speeds for failure when compared to gable roofs – the equivalent of moving up one category in the EF scale.⁸⁷

Where prefabricated, engineered trusses are not used to frame roofs, it is recommended that roofs be engineered according to NBCC Part 4 or CWC 2014. There exists limited prescriptive guidance with respect to high wind resistant stick roof framing. Members of the Technical Committee involved in the development of the Durham Standard noted that there is potential for more “variability” in stick roof framing – that is, improvisations may be made in the field that result in construction that does not necessarily follow the design specifications.

A.1.2. Roof-to-Wall Connections (RTWCs)

- a) Tie roof rafters, roof trusses or roof joists to load-bearing wall framing in a manner that will resist a factored uplift load of 3 kN.⁸⁸

This measure requires adequate connection of the top plate to the supporting wall studs, combined with adequate continuous vertical load path. If continuous structural wall sheathing (see Measure A.2.4) is not applied, then a top-to-bottom inspection to address all potential weak links in the continuous vertical load path using additional ties, straps or related measures should be applied.

And

- b) When engineered connectors are used, builders should request that truss manufacturers supply appropriate roof-to-wall connections along with trusses.

Purpose:

- In the context of high wind events, the potential for flying debris causing breaches in building envelopes should be factored into decision making related to RTWCs. Breaches in the building envelope result in internal pressurization, contributing to the need for enhanced RTWCs.
- Requesting truss manufacturers to supply roof-to-wall connections ensures that appropriate connections are provided along with trusses.

Notes:

- Options may include engineered connectors, use of a combination of raised-heel and continuous structural wood wall sheathing adequately fastened to the heel of the truss, or other methods that meet the specified uplift load. Connections may also be engineered to meet the referenced factored uplift load (3 kN).
- See manufacturers' specifications for products that are capable of resisting a factored uplift load of 3 kN.
- CWC 2014 does not incorporate prescriptive provisions for engineered RTWCs, but does provide guidance and details for completing an engineered design for such a connection.
- Enhanced RTWC options, including use of hurricane ties, increases ease of inspection of roof-to-wall connections, when compared to toe-nail connections.
- Stakeholder Committee members noted increased use of raised-heel trusses, which allow for full-depth insulation throughout the attic. Raised-heel trusses also allow for an additional RTWC option when used in combination with continuous structural wood sheathing, as the wall sheathing can be directly fastened to trusses, and therefore offer a co-benefit between thermal efficiency and wind resistance (see Figure A.2).⁸⁹

Figure A.2: Fastening wall sheathing to raised-heel truss⁹⁰



Image reproduced with permission from APA – The Engineered Wood Association

A.1.2. Discussion and Context

Roof failures during high wind events typically begin at the RTWCs,⁹¹ and tornado damage investigations, including those discussed above, indicate that toe-nailed RTWCs are frequently not compliant with code requirements. Input from Canadian code officials involved in the development of this draft and the Durham Standard reiterated that toe-nailed RTWCs are often not code compliant. It has also been noted in the literature that diagonal nailing of toe-nails through rafters may split lumber.⁹² Damage to trusses (e.g., splitting of bottom chord of truss, damage to gusset plates) associated with three 12d fastener toe-nail connections was further noted by reviewers of this paper,⁹³ and would serve to significantly reduce uplift capacity of toe-nailed connections. It was further noted by a member of the Stakeholder Committee that toe-nailed connections are difficult to enforce in several instances (e.g., where minimum end-bearing is applied – see Figure A.3).

The uplift capacity of individual toe-nailed RTWCs varies considerably based on characteristics of connections. For example, static loading tests have revealed uplift capacity of toe-nails varies between 1.1 kN and 2.8 kN, depending on type and number of nails (nails in these studies ranged from 8d to 16d), age of connection, type of wood, and moisture content of wood.⁹⁴

Figure A.3: An example of a roof-to-wall connection where provision of toe-nails is difficult (roof truss, minimum end bearing, southern Ontario)



Photo credit: D. Potter, Town of Newmarket, June 2018

Tests of NBCC Part 9 compliant, toe-nail RTWCs (comprised of three 12d twisted-shank nails, at 45° angles, installed with nail gun, accounting for variability in wood properties and construction by testing 21 samples), indicated that these connections have a mean failure capacity of 2.8 kN. Non-code compliant toe-nailed connections are considerably less able to resist uplift loads. Specifically, when the nail is missing on the single-nail side of the connection, mean failure capacity is 1.9 kN. When a nail was missing on the two-nail side of the connection, mean failure capacity was 2.2 kN (Table A.1). These values compare with mean resistance for hurricane ties of 5.84 kN (single H2.5 tie).⁹⁵

Table A.1: Toe-Nail, Mean Failure Capacity⁹⁶

| Defect | Mean failure capacity (kN) | Standard deviation (kN) | #Split/#Pull outs |
|-----------|----------------------------|-------------------------|-------------------|
| No defect | 2.8 | 0.6 | 22/41 |
| Defect #1 | 1.9 | 0.46 | 11/5 |
| Defect #2 | 2.2 | 0.48 | 0/16 |

Note: Defect #1 includes a defect where one nail is missing from the single-nail side of the toe-nail connection. Defect #2 includes a scenario where one nail is missing from the two-nail side of the toe-nail connection. Tests conducted using static loading (ramp tests).

Fluctuating wind load test methods applied at Western University also revealed a mean failure capacity of 2.8 kN for toe-nail RTWCs (no defects), and a fifth percentile capacity for toe-nailed connections of 1.9 kN (no defects). Similar to ramp loading tests, pull out was identified as the most common failure mechanism representing 76% of failures in the fluctuating wind load tests.⁹⁷

An important difference between the fluctuating and static testing methods was identification of multiple “damaging peaks” resulting in partial failure of RTWCs under the fluctuating wind load tests. A damaging peak was defined as a “...peak load that causes a permanent partial withdrawal of the nail from the top plate.”⁹⁸ These initial damaging peaks were found to have occurred at loads as little as 56% of the maximum applied loads.⁹⁹ As noted above, it may be difficult to identify partially damaged toe-nail connections. Indeed, testing conducted at Western University revealed that no evidence of partial failure may exist on the exterior of the home, and interior evidence may consist of hairline cracks in the wall-to-ceiling drywall, which may be indistinguishable from “normal” cracking in drywall joints.¹⁰⁰

Kopp *et al.* 2017 noted the impact of improper RTWCs on wind speeds associated with roof failure. Specifically, for gable roofs the wind speed (at the median failure probability) is reduced from ~200 km/h for code-compliant connections (i.e., three nails per connection) to roughly 160-170 km/h when two nails are missing from every connection. For hip roofs, the wind speed associated with median failure probability, with the same RTWC scenario, would result in a reduction from a wind speed of 260 km/h to 180-200 km/h.¹⁰¹

Testing of a full-scale home with NBCC compliant RTWCs using fluctuating wind loads by Morrison *et al.* 2012 revealed that the uplift resistance of an entire roof is greater than would be predicted based on individual tests of RTWCs, due to load sharing amongst the RTWCs.¹⁰² The authors stated that "...it seems unlikely that the complete roof would fail under design wind conditions in any region of North America unless there was a breach in the building envelope."¹⁰³ The authors further stated:

*The current test house was found to be able to withstand design wind loads for almost all wind regions in the United States and Canada, provided that internal pressures are not considered. Internal pressures due to a dominant opening drastically reduce the failure wind speeds. So, on the one hand, toe-nailed connections may be adequate for most of North America, at least for roofs with the current lateral stiffness, provided that dominant openings are not allowed to occur at any time. On the other hand, the current results suggest that complete roof failures observed in the field (except for tornadoes) almost certainly initiate as the result of internal pressurization of the structure due to a breach in the building envelope.*¹⁰⁴

Reflecting the above statements, the potential for breaches in the building envelope during extreme wind events should be factored into decision making related to RTWCs. It has been noted that even relatively minor wind damage to buildings, including loss of shingles, results in flying debris that has enough energy to result in building envelope breaches. Breaches can significantly reduce the wind speeds at which RTWCs fail,¹⁰⁵ contributing to a "domino effect" of downwind damages (see Vaughan case study, above). Heavier debris associated with roof failure also presents a life safety risk. Further, loss of roof structure is typically a precursor to wall collapses,¹⁰⁶ which can cause death or injury during wind storms.

Further, fluctuating wind load testing on the full roof system revealed a first damaging peak associated with wind speeds of 25 m/s (90 km/h). The connection at which this first damaging peak was observed experienced a load of 1.97 kN, which was consistent with the peak load that caused initial damage during testing of the individual toe-nails, as described above.¹⁰⁷

Details related to fastening of roof sheathing are provided in A.1.3. Kopp *et al.* 2017 provided discussion about the relative failure capacities of roof sheathing compared to toe-nailed RTWCs. Specifically, using a scenario that includes 8d nails fastening the sheathing at 300 mm spacing along the intermediate supports and 150 mm spacing along outside edges of sheathing, RTWCs for gable roofs are more likely to fail than roof sheathing. In the case of hip roofs, RTWC failure probabilities are similar to sheathing failure probability when 8d nails are used (under same fastener spacing scenario described above).¹⁰⁸

With respect to estimates of withdrawal resistance for toe-nailed RTWCs, CSA O86-14 calculations may be considered preferable to the lab test data because they are fifth-percentile values rather than mean test values, and the specimens as tested in the lab would have been built to a relatively high standard (code values attempt to account for material variations, some errors in construction, etc.). Values calculated based on CSA O86-14 are made additionally conservative by overall resistance factor (ϕ), which varies based on variability of the particular connection type being assessed.

CSA O86-14 Wood Design Code (12.9.5) calculations indicate that NBCC compliant toe-nailed RTWCs (composed of three 82.5 mm nails) have a minimum factored withdrawal resistance of 340 N (567 N, unfactored), assuming S-P-F members. Use of DF-L members, similar to specimens

used in the Morrison and Kopp 2011¹⁰⁹ study summarized above, would increase the factored withdrawal resistance to 476 N (793 N, unfactored).¹¹⁰ Thus, toe-nailed connections, with design capacities ranging from ~340 N (factored) are considered sufficient in NBCC 9.23.3.4 for hourly wind pressures up to 0.8 kPa. Beyond this pressure, ties with 3 kN resistance are required (per NBCC 9.23.3.4.(3)).

Roof 9 in CWC 2014 further indicates that a factored uplift resistance for toe-nail connections consisting of three 3.5" common nails of 0.83 kN. It is further noted that the engineering guide allows one to use a load duration factor of 1.25 instead of 1.15, so the uplift capacity is increased by 1.09 in the guide. The guide provides the following notes related to minimum top plate width based on CSA O86 table 12.9.2.1: "140 mm for D. Fir-L and Hem-Fir, 89 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements for CSA O86 Table 12.9.2.1."

A.1.3. Roof Sheathing and Roof Sheathing Fasteners

Install minimum 11.1 mm (7/16") structural plywood or OSB roof sheathing.¹¹¹

And

Fasten with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails.

And

Where roof sheathing supports are spaced at more than 406 mm o.c., the maximum spacing of fasteners for roof sheathing should be 150 mm along edges and intermediate supports.

Purpose:

- 11.1 mm (7/16") plywood or OSB roof sheathing provides improved roof bracing.
- Fastener measures reduce risk of sheathing panel failure during high wind events, as sheathing is better able to withstand uplift forces.

Notes:

- Failure of sheathing fasteners typically originates at interior fasteners, which have the largest effective tributary area on the panel (see below discussion).
- Fastener spacing requirement has been incorporated into the 2012 OBC, based on a 2010 code change request submitted by ICLR/Western University (see Appendix E). See OBC 9.23.3.5(5) for roof sheathing nailing pattern provision (150 mm [6"] spacing on edge and intermediate supports, when roof sheathing supports are placed at more than 406 mm [16"] o.c.).
- NBCC 9.23.3.5 does not include the above sentence. See NBCC wording in Appendix D for detail on roof sheathing fastening requirements. NBCC wording requires tighter nailing patterns only at the edges of roof sheathing in areas exposed to high winds (i.e., where $q_{1/50} \geq 0.8$ kPa). A similar code change request to the OBC change cited above has been made for the NBCC (see Appendix H).
- NBCC 9.23.16.7-A: Currently, sheathing thickness varies from 7.5 mm to 12.7 mm depending on spacing of supports and sheathing type.
- This measure assumes that roof trusses/rafters are spaced 600 mm (24") o.c.¹¹²

A.1.3. Discussion and Context

Capacity of roof sheathing to withstand high wind events depends on several factors, including sheathing material and thickness, roof trusses, fastener types (e.g., nails, staples), fastener sizes, and fastener patterns.¹¹³ As noted by Gavanski *et al.* 2014:¹¹⁴

Roof sheathing is one of the critical components of the roof structure, since it ties the rafters or trusses together, allowing them to act as a system critical to the integrity of the overall structure. In addition, sheathing provides the support for the roof cover and is a critical element in the environmental control provided by the cladding, which keeps the rain out.

In a study that applied wind exposure values presented in the NBCC and sheathing requirements associated with 11.1 mm OSB sheathing, Gavanski *et al.* 2014 assessed the adequacy of prescriptive NBCC requirements related to roof sheathing fasteners. The authors applied a failure probability target of less than 5×10^{-5} (target reliability index used for calibrating the wind load provisions in NBCC) for any individual sheathing panel on a roof. The following observations were made:¹¹⁵

- Building height (e.g., 3 storey vs. 2 storey) and roof shape (e.g., slope, gable vs. hip) affect failure probabilities, where higher buildings and gable roofs experience higher probability of failure of sheathing panels.
- Dominant openings, associated with occurrence of wind-borne debris, causing increased pressure inside of attics, significantly increased failure probability for sheathing panels.
- Minimum NBCC requirements related to use of 6d (51 mm) common nails with 150 mm edge spacing and 300 mm interior spacing were considered inadequate for many regions of Canada, regardless of roof shape and existence of dominant openings in walls. However, 6d spiral nails were considered adequate up to HWPs of ~0.5 kPa.
- Small differences in fastener type, for example use of 8d (63 mm) spiral nails in place of 6d (51 mm) spiral nails, reduced failure probabilities considerably. Specifically, use of 8d spiral nails with a fastening pattern of 150 mm x 300 mm was adequate to lower the failure probability to an acceptable level in HWP 0.4 kPa areas, even when dominant openings were present.
- Specialized nails, notably 63 mm HurriQuake nails with a 150 mm x 150 mm fastener pattern, were considered adequate even where HWP reached 1.0 kPa.
- It was noted that capacities for panels increased with increasing nail lengths, or use of ring shank nails, and increased number of fasteners on intermediate supports.
- The authors made the following recommendations:
 - For locations with wind pressures of $0.4 \text{ kPa} < \text{HWP} \leq 0.8 \text{ kPa}$, it is recommended that fastener types specified in NBCC table 9.23.3.5.B be used with a nail spacing of 150 mm for both the interior and edge supports.
 - Table 9.23.3.5.A of NBCC... should be modified to apply only for $\text{HWP} \leq 0.4 \text{ kPa}$, not the current range of $\text{HWP} < 0.8 \text{ kPa}$.
 - For $\text{HWP} > 0.8 \text{ kPa}$, engineering guidance should be obtained for design.

Henderson *et al.* 2013 provided an assessment of fasteners for roof sheathing under high wind loads. The authors noted that, with twist shank nails, there was an incremental failure progression, similar to earlier results for toe-nails noted by Morrison and Kopp 2011.¹¹⁶ Incremental failure has also been noted for wall sheathing in fluctuating load tests. Under these incremental failure scenarios, there were multiple nail slips of roughly 1-2 mm, resulting in failures typically associated with nail pull-out. Ring-shank nails, coated ring-shank nails and HurriQuake nails did not withdraw incrementally. Typically, these fasteners exhibited more sudden failures associated with pull-over, though small displacements of a few millimetres before failures were observed. For both failure mechanisms (i.e., pull-out/incremental and pull-over/sudden), the failures originated at interior fasteners, which have the largest effective tributary area on the panel.¹¹⁷

It was further noted that twist-shank nails experienced smaller impacts associated with missing fasteners, when compared to ring shank nails. Specifically, Henderson *et al.* 2013 noted:

*For twisted-shank nails, the reduction of mean capacity when there are two nails missing is about 23%, while for the ring-shank...nails, it is about 38%. This difference is attributed to the greater load sharing facilitated by the incremental failure mechanism compared to panell/fastener combinations where the sudden failure occurs.*¹¹⁸

A.1.4. Roof Covering and Sealing of the Roof Deck

Application of measures to ensure that roof covering remains in place, which may include application of shingles rated for high wind and/or application of measures to seal the roof deck.

Options:

Roof covering rated to withstand wind speeds of 200 km/h (or ~130 mph) would serve to meet the intent of this measure. Roof covering material should meet appropriate standards (ASTM D7158 Class G or equivalent).

The Insurance Institute for Business and Home Safety (IBHS) identifies multiple options for sealing of the roof deck. A summary of the measures is provided below (Table A.2). Additional installation measures are available from IBHS. Reference material should be consulted before installation.¹¹⁹

Note regarding applicability of this measure:

Proprietary roof sheathing systems that include a sealed roof deck system composed of wood structural panel with integrated factory-bonded underlayment and field applied seam sealing tape may preclude need for additional secondary water barrier/deck sealing options.

Purpose:

- It has been previously identified that much of the damage caused to residential buildings during extreme wind events results from water penetration into the building.¹²⁰
- This measure offers enhanced protection to the building from water damage by reducing risk of roof covering failure, and/or providing additional protection in the event of roofing failure (e.g., when shingles are blown off during wind events).

Notes:

- Follow manufacturers' instructions for installation of measures identified in Table A.2.
- With respect to cold weather application, specifications for products compliant with Option 1 in Table A.2 indicated minimum application temperatures as low as -29°C. Multiple products had a minimum application temperature of -17°C and under (for additional detail and initial pricing comparison, see end note).¹²¹
- Note that CWC 2014 includes provisions for a 2-3 mm gap between roof sheathing panel edges and end joints, and states that the gap "...can be sealed appropriately to form an air barrier."¹²²
- This measure is meant to apply to steep slope roofs (3:12 or greater).¹²³

Table A.2: Roof Deck Sealing Options*

| Roof type | Option |
|---------------------------|---|
| Asphalt and metal roofs | <p>1. Taping seams between roof sheathing panels:</p> <p>1(a). Apply an ASTM 1970 compliant self-adhering polymer-modified bitumen flashing tape at least 100 mm (4") wide directly to the roof deck to seal the horizontal and vertical joints in the roof deck.</p> <p>1(b). Apply an AAMA 711, Level 3 (for exposure up to 80°C/176°F) compliant self-adhering flexible flashing tape at least 95 mm (3¾") wide directly to the roof deck to seal the horizontal and vertical joints in the roof deck.</p> <p>Both options include application of a code-compliant #30 ASTM D226 Type II or ASTM D4869 Type IV underlayment over the self-adhering tape.*</p> |
| | <p>2. Installation of two layers of ASTM D 226 Type II (#30) or ASTM D 4869 Type IV (#30) underlayment in a shingle-fashion, lapped 480 mm (19") on horizontal seams (915 mm or 36" roll), and 150 mm (6") on vertical seams.</p> |
| | <p>3. Cover the entire roof deck with a full layer of self-adhering polymer-modified bitumen membrane meeting ASTM D1970 requirements.</p> |
| | <p>4. Apply a reinforced synthetic roof underlayment that has an International Code Council approval as an alternate to ASTM D226 Type II felt paper.</p> |
| Concrete, clay tile roofs | <p>1. Cover the entire roof deck with a full layer of self-adhering polymer-modified bitumen membrane meeting ASTM D1970 requirements.</p> |
| | <p>Taping seams between roof sheathing panels:</p> <p>2(a). Apply an ASTM 1970 compliant self-adhering polymer-modified bitumen flashing tape at least 100 mm (4") wide directly to the roof deck to seal the horizontal and vertical joints in the roof deck.</p> <p>2(b). Apply an AAMA 711-13, Level 3 (for exposure up to 80°C/176°F) compliant self-adhering flexible flashing tape at least 95 mm (3¾") wide directly to the roof deck to seal the horizontal and vertical joints in the roof deck.</p> <p>Both options include application of a code-compliant #30 ASTM D226 Type II or ASTM D4869 Type IV underlayment over the self-adhering tape.*</p> <p>As a final step:*</p> <ul style="list-style-type: none"> i) Apply a self-adhering polymer-modified bitumen cap sheet complying with ASTM D1970 over this underlayment. <p>Or</p> <ul style="list-style-type: none"> ii) Hot mop the underlayment using hot asphalt and apply a #90 mineral surface cap sheet. |
| | <p>3. Installation of two layers of ASTM D 226 Type II (#30) or ASTM D 4869 Type IV (#30) underlayment in a shingle-fashion, lapped 480 mm (19") on horizontal seams (915 mm or 36" roll), and 150 mm (6") on vertical seams.</p> |

* For additional detail and installation guidance, see Insurance Institute for Business and Home Safety (IBHS). 2015. High Wind Standards. Insurance Institute for Business and Home Safety.

A.1.4. Discussion and Context

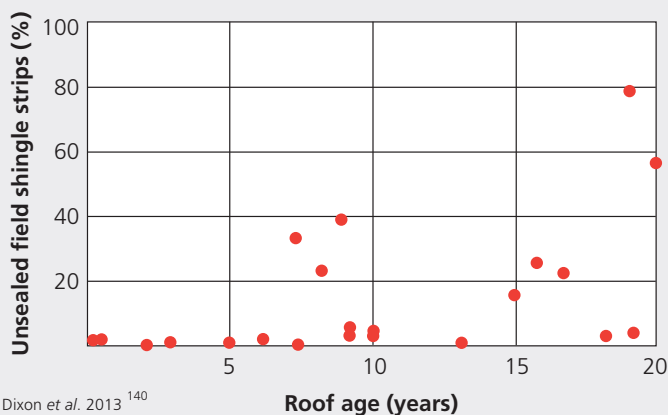
Asphalt shingle roof coverings can be lifted, cracked, torn, or completely blown off from uplift forces caused by wind.¹²⁴ Shingle resistance to uplift depends on several factors including the installation of the correct number and position of fasteners, the quality of the connection between the thermally activated sealant strip and the lower shingle, and physical properties such as tear resistance, pliability, stiffness, and fastener pull-through resistance.¹²⁵ Shingle uplift is the result of negative pressure that develops as wind flows over roof systems,¹²⁶ which occurs as the flow separates at the leading edge of the shingle.¹²⁷ Several post-hurricane damage assessments determined that the performance of asphalt shingles was highly variable, and wind damage primarily resulted from the underperformance of the adhesive sealant strip, followed by the presence of misplaced fasteners.¹²⁸

Sealant strips are the primary vertical load paths that transfer wind uplift forces from the surface of the shingle to the building structure.¹²⁹ The sealant strip on the leading edge of asphalt shingles prevents water penetration and physical lifting, and reduces the surface area exposed to the maximum uplift pressures.¹³⁰ The risk of asphalt shingle roof failure is strongly influenced by the condition of the sealant strips.¹³¹

Uplift resistance is significantly reduced when adhesive sealant strips are not fully adhered on field, hip, and ridge regions of asphalt shingle roof cover.¹³² Because the acting pressure on the underside of the shingle increases if the sealant is partially unsealed, the strip's adhesion can further reduce and the risk of blow-off increases.¹³³ Partial unsealing of the strips has been shown to occur naturally across the roof field and hips and ridges as roof systems age,¹³⁴ resulting in roof coverings that are more vulnerable to damage caused by wind loading.¹³⁵ Additionally, post-event damage assessments have observed that fully sealed shingles only become susceptible to wind damage when directly adjacent to failed partially unsealed shingles.¹³⁶

Studies have shown a statistically significant increase in the percentage of partially unsealed shingles for older roofs compared to roofs less than six years old (see Figure A.4), regardless of shingle manufacture and installation.¹³⁷ If dust deposition on the strip occurs before the sealant becomes thermally activated, imperfect sealant bonds that are susceptible to failure may form.¹³⁸ Although the mode of sealant strip failure over time remains largely unknown, post-damage assessments have repeatedly reported sealant strip failures on damaged shingles.¹³⁹

Figure A.4: Percent of unsealed shingle strips located in the field of the roof versus roof age



Adapted from Dixon *et al.* 2013¹⁴⁰

Metal roof sheathing is usually made from cold-formed steel, and can become susceptible to failures including structural deformation, material fatigue, and loose fasteners resulting from environmental exposure.¹⁴¹ Changing temperatures, radiation, hail, wind, snow, rain, and atmospheric pollution can all degrade metal roofing over time.¹⁴² The presence of moisture may promote the corrosion of metal roofing.¹⁴³ Metal roof claddings can become susceptible to metal fatigue, or the development of cracks in areas of stress concentration that can ultimately result in roof failure caused by wind loading.¹⁴⁴ Fatigue failure results from the application of repeated fluctuating loads lower than the material's design load.¹⁴⁵ Metal roofing that has been degraded can become susceptible to wind load failure even if design loads are not met.¹⁴⁶

Loss of roofing components during storm events can result in significant entrance of rainwater, damaging contents and significantly contributing to property damages.¹⁴⁷ It is further noted that roofing materials (e.g., shingles) may fail at relatively low wind speeds (e.g., wind speeds associated with DOD 2-4), exposing buildings to water damage risk. Asphalt shingles' resistance to wind uplift depends on a variety of factors including shingle type, design, quality of manufacture and installation, and degree of weathering.¹⁴⁸ Increasing a roof's resilience to high winds typically involves the installation of heavier, stiffer shingles that are more resistant to uplift,¹⁴⁹ and increasing the number of nails fastening each shingle to the roof deck.¹⁵⁰ The most commonly installed types of asphalt shingles are strip (i.e., 3-tab) and laminate shingles,¹⁵¹ which are composed of multiple layers of material.¹⁵² Standard strip shingles are generally rated for lower wind speeds than laminate shingles.¹⁵³

A.2. Walls

A.2.1. Bracing

Wall framing should be capable of resisting lateral loads as specified in Subsection 9.23.13 of the National Building Code of Canada (NBCC), with (minimum) $q=0.8$ kPa replacing the specified velocity pressure from NBCC Appendix C.

Purpose:

Provides enhanced bracing for specified wind load.

Notes:

- Where deemed appropriate, bracing may comply with NBCC 9.23.13 and provisions therein, including options for engineered design.
- Per NBCC 9.23.13, where $q_{1/50}$ is equal to or greater than 1.20 kPa, the building must be engineered in accordance with NBCC Part 4 or good engineering practice (e.g., CWC 2014).
- Measure A.2.1 applies where enhanced protection is required in the form of bracing to resist lateral loads, based on a minimum wind pressure of 0.8 kPa.

A.2.2. Floor-to-Floor Connections (Multi-Storey Construction)

Load-bearing wall framing for upper and lower storeys should be connected to facilitate continuous vertical load path. Connections should achieve a factored uplift load of 4 kN/m.

Options may include:

- a) Upper and lower storey wall sheathing should be fastened to common rim joist with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails using a 150 mm (6") o.c. spacing along both the top and bottom edges of the rim joist.¹⁵⁴

Or

- b) Installation of metal straps to connect upper and lower storeys by:
 - (i) Connecting wall studs from the wall above to wall studs in the wall below,¹⁵⁵
or
 - (ii) Connecting wall studs above to the rim joist and from the rim joist to the wall studs below.¹⁵⁶ When this approach is applied, straps connected to upper-storey studs should extend to the bottom of the rim joist, and straps from the lower-storey studs should extend to the top of the rim joist.

For options b(i) and b(ii):

Care should be taken when installing the connector to ensure that the connector does not buckle due to shrinkage of lumber after being installed.

Purpose:

Contributes to continuous vertical load path.

Notes:

- Options for achieving continuous load path may include use of wood sheathing, straps, proprietary “truss screws,” or other measures capable of achieving the specified minimum uplift load.
- Note that use of continuous wood sheathing provides improved bracing as well as providing options for enhancing the continuous vertical load path.
- See CWC 2014 detail on connecting sheathing to rim joist (shearwall applications) in Appendix L.
- CWC 2014 states that sheathing should lap the connecting floor framing member (rim joists or blocking) by not less than 50 mm. Nails driven into the rim joist should be staggered (see Wall 11).
- CWC 2014 advises 3 mm gap between upper and lower storey sheathing panels.
- Building design factors may limit viability of fastening upper and lower storey sheathing to common rim joist. Use of straps provides an alternative to use of sheathing to resist uplift.
 - Metal straps should be provided to achieve equal or greater factored uplift resistance provided by measure outlined in A.2.3.(a) (4 kN/m – see Wall 11, CWC 2014, Appendix L).
 - Note additional strap requirements for openings are 0.91 m (3') wide and over.¹⁵⁷
 - Note that, where the bottom wall plate or sole plate of an exterior wall is not nailed to floor joists, rim joists or blocking in conformance with NBCC Table 9.23.3.4., NBCC Subclause 9.23.3.4.(2)(b) requires that straps be spaced no more than 1.2 m apart (see Appendix D for full Subclause).
 - Strap spacing may also be modified to increase uplift capacity.
 - See manufacturers' detail for strap uplift capacity.
- Straps may be installed on each stud or at some other convenient spacing not to exceed 2.44 m (8');¹⁵⁸ however, to facilitate improved load sharing, tighter strap application (e.g., 1.22 m or less), should be applied. Tighter strap placing would also allow for use of less expensive straps (e.g., thinner/shorter straps that would still accommodate the specified 4 kN/m load).

Note: See CWC 7 in Appendix L of this report for an illustration of the above-noted measure related to use of structural wall sheathing to tie together upper and lower storeys.

A.2.3. Stud-to-Sill Plate Connections

Studs should be connected to sill plate to facilitate continuous vertical load path. Connections should achieve a factored uplift load of 4 kN/m.

Options may include:

- a) Structural wall sheathing extended to fully lap the sill plate.¹⁵⁹ Where this option is applied:
 - i) Sheathing should be fastened to sill plate with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails using 150 mm (6") o.c. fastener spacing.
And
 - ii) Fasten wall sheathing to rim joist (if present) with 8d 3.3 mm x 63 mm (0.13" x 2.5") nails using 150 mm (6") o.c. spacing along both top and bottom edges of the rim joist.¹⁶⁰
- b) Installation of metal straps or connectors to connect lower storey wall studs to the sill plate.

Purpose:

Contributes to continuous vertical load path.

Notes:

- See CWC 2014 detail on connecting sheathing to sill plate (shearwall applications – CWC 9 in Appendix L).
- Where wood structural panels are not used to fasten walls to sill plates, metal connectors should be spaced at 1.22 m or less to facilitate load sharing.

A.2.4. Wall Sheathing and Fastening

This measure applies where wood structural panel wall sheathing is used to achieve continuous load path (see Measures A.1.2., A.2.2. and A.2.3.) and reinforce gable end walls (see Measure A.1.1.(b)).

Where appropriate, continuously sheath all walls with structural sheathing (OSB or plywood),¹⁶¹ applied/installed with the following details:

(a) Minimum thickness of wall sheathing should be 11.1 mm (7/16”).

And

(b) Sheathing should be fastened with 8d 3.3 mm x 63 mm (0.13” x 2.5”) nails.¹⁶²

And

(c) Fasteners should be spaced 150 mm (6”) along edges and intermediate supports.

Purpose:

- Use of structural wood panels provides opportunity for application of wall sheathing to achieve enhanced continuous vertical load path, including floor-to-floor connections and sill plate connections.
- Specifically, continuously sheathing all walls with structural sheathing (OSB or plywood)¹⁶³ assists in improved RTWCs (Measure A.1.2) by securing the top plate to the load-bearing wall and transferring loads to the foundation.
- Fastener type and spacing increases resistance to negative wind pressure.¹⁶⁴

Notes:

- This measure applies to both walls with stud spacing of 406 mm o.c. (16”) and 610 mm o.c. (24”) with 150 mm (6”) o.c. fastener spacing along edges and intermediate supports.¹⁶⁵
- Several provisions outlined in CWC 2014 meet or exceed those provided here. For example, see 12.5 mm sheathing provision noted in Table C1 a (Wood Sheathed Braced Wall Panel Construction Details for Wind or Seismic Loads) of CWC 2014 where $q_{1/50} \leq 1.2$.¹⁶⁶
- CWC 2014 Wall 1 sheathing fastening provisions may meet or exceed this measure depending on wind pressure, spacing of roof framing (e.g., interior zone fastener spacing varies between 75 and 300 mm).
- 8d (63 mm) nails exceed provisions outlined in CWC 2014 (e.g., Wall 1 in CWC 2014 references 2” Common Nails or Larger).

Discussion:

Discussion amongst members of the Stakeholder Committee suggested a need to clarify that use of continuous wood wall sheathing should not preclude use of continuous exterior insulation, which is increasingly being applied in Part 9 home construction across Canada. A review of prescriptive guidance documents indicated that continuous plywood or OSB wall sheathing would not preclude application of continuous exterior insulation (for example, see Figure A.5).¹⁶⁷

Figure A.5: Example of use of continuous exterior insulation and 7/16" OSB sheathing produced as part of a guide to help homebuilders and designers achieve BC Energy Step Code requirement of R22+ thermal performance in walls¹⁶⁸

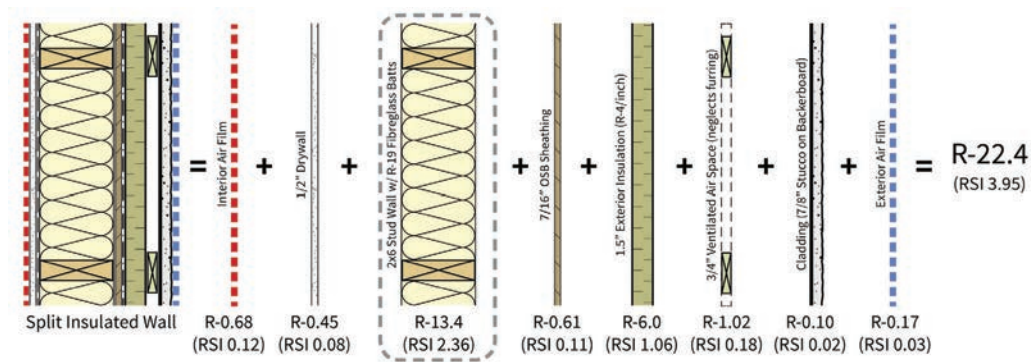


Image source: Ricketts, L. (RDH Building Science Inc). 2017. Illustrated Guide to R22+ Effective Walls in Wood-Frame Construction in British Columbia. Victoria: BC Housing. Used with permission from BC Housing.

Within the building science community there is evolving understanding on the durability implications of installing exterior foam insulation over wood sheathings.¹⁶⁹ While this topic is generally outside of the scope of this report, it is recognized that the durability of support components is an important element to ensuring buildings maintain bracing through the service lives of the building. Findings of research related to this topic should be monitored and solutions considered in future wind risk reduction guidance documents. Note that CWC offers a free web tool that provides a durability rating and effective thermal insulation value for wood walls, including walls with rigid foam plastic insulation over wood structural panel sheathing.¹⁷⁰

A.3. Anchorage of Building Frames

Anchorage of building frames should contribute to the continuous vertical load path.

Purpose:

The building frame should be anchored to the foundation in a manner that contributes to the continuous vertical load path. Anchoring, at a minimum, should comply with NBCC Clause 9.23.6.1.(2)(b).

Notes:

- NBCC Clause 9.23.6.1.(2)(a) (embedding the ends of the first floor joists in concrete) results in a discontinuity in the vertical load path.
- Anchoring provisions in some provincial codes result in discontinuous vertical load path. Specifically: Alberta Building Code 9.23.6.1.(2)(c) “embedding in concrete two 38 mm x 89 mm sill plates placed on edge and separated by blocking 1.2 m o.c.”

A.3. Discussion and Context

Anchoring provisions in the NBCC include options for anchoring of building frames to foundations using anchor bolts, and CWC 2014 provides additional prescriptive guidance on anchor bolts (See Table C5 in CWC 2014) where $0.8 \leq q_{1/50} < 1.2$ kPa. Stakeholder Committee members, however, identified building anchoring methods permitted by provincial codes that result in discontinuities in continuous load paths.

Specifically, the Alberta Building Code (ABBC) contains the following anchoring options (emphasis added):

9.23.6.1. Anchorage of Building Frames

- 1) Except as required by Sentence 9.23.6.3.(1), building frames shall be anchored to the foundation unless a structural analysis of wind and earthquake pressures shows anchorage is not required.
- 2) Except as provided in Sentences (3) and (5), anchorage shall be provided by
 - a) embedding the ends of the first floor joists in concrete,
 - b) fastening the sill plate to the foundation with not less than 12.7 mm diam anchor bolts spaced not more than 2.4 m o.c., or
 - c) *embedding in concrete two 38 mm x 89 mm sill plates placed on edge and separated by blocking spaced 1.2 m o.c.*

Buildings with anchorage details complying with Clause (c) may be fastened to the embedded “ladder” (see Figure A.6) using two 82 mm nails per floor joist or blocking, in accordance with ABBC Article 9.23.3.4. It was noted by Stakeholder Committee members that almost every foundation in central Alberta incorporates anchoring methods in accordance with ABBC 9.23.6.1.(2)(c).

Figure A.6: Alberta Building Code 9.23.6.1(2)(c), as applied in central Alberta

ABBC 9.23.6.1.(2)(c) results in a discontinuous vertical load path. All uplift on the sill plates would be resisted only by friction or shear bond between the face of the members and the concrete. It should not be assumed, however, that a shear bond will be present as, after a number of expansion and contraction cycles of the wood members, there may no longer be contact between the members and the concrete. Thus, a conservative assessment suggests that the uplift resistance provided by this anchoring option would be non-existent, and only the weight of the building would provide downward resistance. Upward movement of the frame by 89 mm (3.5") (height of the wood members) could result in dislocation of the frame from the foundation. Further, Figure A.6 indicates that a limited amount of concrete is present between the wood members. This concrete could be of poor quality, as it is the least-settled part of the pour at the very top of the foundation.



Photo credit: B. Baumgarten, Red Deer County, 2018

With respect to vertical load path, the anchoring provision outlined in NBCC 9.23.6.1.(2)(a), requiring that the ends of the first floor joists are embedded in concrete, is similar to ABBC 9.23.6.1.(2)(c). An additional issue related to these provisions includes increased risk of degradation of embedded wood (associated with water accumulation/poor drainage).¹⁷¹ Stakeholder Committee members noted that for ABBC 9.23.6.1.(2)(c), where the ladder is located more than 150 mm above grade, there may not be a requirement for treated lumber in the embedded ladder.¹⁷²

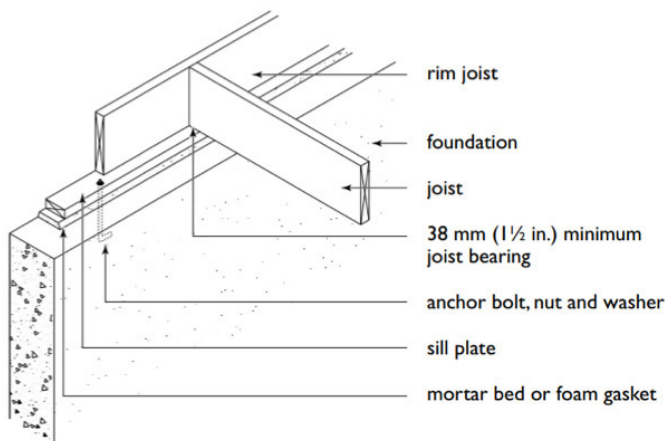
It is important to also consider the tie between the sill plate, which is anchored (or cast-in according to ABBC 9.23.6.1.(2)(c)), to the floor joists and structure above. Using anchor bolts provides good resistance between the foundation and the sill plate; however, the NBCC specifies toe-nailing methods for floor joists. Table A.3 indicates how toe-nailed connections compare to the anchorage methods – the weak link in the bottom of an anchored wall will be the toe-nailed connections. This is improved, however, by lapping the sill plate with wall sheathing, and application of other fastening measures identified in this document.

Table A.3: Floor Anchorage Detail

| | Uplift Resistance[N] | Shear Resistance[N] |
|--|--|--------------------------------|
| Bottom of Wall Framing (for comparison with anchorage values to identify which link is weakest) | | |
| Floor Joist to Sill Plate | | |
| 2-82.5 mm spiral nails | 339.96 per 600 mm | 1019 per 600 mm |
| Ridge Board to Sill Plate | | |
| 1-82.5 mm spiral nail | 169.98 per 150 mm | 509.5 per 150mm |
| Combined (best case) | | |
| 2-82.5/600mm + 1-82.5/150mm | 1.7 N/mm | 5.1 N/mm |
| Normalized to 2.4 m (8-ft spacing of anchors) | 4.08 kN | 12.2 kN |
| Anchorage | | |
| Basic Anchorage Requirement (Simpson Strong-Tie® Anchor Designer™ Software) ¹⁷³ | | |
| 5/8" (~13.7mm) J-bolt | 12.8 kN, Decreases under combined shear and uplift (interaction rate) | 22 kN (strong), 13kN (weak) |
| Members Cast-in | | |
| Sill plate cast into foundation wall | Negligible, relying only on self-weight | Shear strength of concrete |

Note: Values are all factored resistances.

Figure A.7: Method of anchoring floor system to concrete walls, showing anchor bolt for wood sill ¹⁷⁴



Source: Canada Mortgage and Housing Corporation (CMHC)

A.4. Post Base and Cap Connections

- a) Post base and cap connections rated for at least 6.8 kN (1,536 lbs) allowable uplift loads should be used.

And

- b) Post base connections should be embedded in or fastened to concrete slabs for front and rear porch applications.

And

- c) Fasteners used for post base connectors should be hot-dipped galvanized or stainless steel.

And

- d) Post and cap connections must be visible for the purposes of inspection.

Purpose:

- By adequately attaching porch roof support beams to their posts, and posts to their foundation, the resistance of the posts to uplift forces during windstorms is increased, decreasing the risk of structural damage.
- Porch columns are often toe-nailed to foundations, which provides insufficient uplift capacity.
- Use of visible connectors (e.g., connections that extend above the base of posts) increases the ability to inspect post base and cap connections.

Notes:

- With respect to porch roofs, Table C8 of CWC 2014 outlines connections between end-joist and built-up columns. In lieu of nails, straps capable of resisting a minimum factored tensile and compressive force of 4.6 kN are permitted.¹⁷⁵ However, no specific prescriptive provisions in 2015 NBCC or CWC 2014 related to uplift capacity of post base and cap connections are provided.¹⁷⁶
- The measure includes “visible” connectors to ease inspections.
- See NBCC 9.23.6.2 for post base and post cap connections for attached structures.¹⁷⁷ Uplift capacity is not provided for in this Article.
- NBCC 9.35.4.3 (1) contains provisions for anchorage of garage and carport walls and columns. An uplift capacity for anchorage is not specified.
- The 6.8 kN figure is based on the following assumptions (see NBCC code change request in Appendix I):¹⁷⁸
 - 2.44 m (8') wide porch,
 - 2.44 m (8') between posts,
 - Porch weight of 0.48 kPa (10 psf),
 - Open terrain wind exposure, and
 - 1/50 year wind exposure of 0.8 kPa.
- Enhanced corrosion resistance may be required based on environmental conditions (e.g., stainless steel may be necessary in coastal areas, where there is high exposure to salt).
- See manufacturers' catalogues for uplift ratings of post base and cap connectors.
- Install connectors according to manufacturers' instructions.

A.5. Garage Doors Rated for High Wind (Optional)

Garage doors should be rated for minimum 200 km/h wind speed.¹⁷⁹

Purpose:

Garage door failures result in increased internal pressure during tornadoes, resulting in roof failure and increasing wind-borne debris.

Notes:

- High wind-rated garage doors are considered an optional measure as risk can be mitigated through provision of improved continuous vertical load path (e.g., adequate connection of roof and walls to foundation).
- Peak wind speed roughly translates to pressure values of 1.86/-1.95 kPa (39.1/-41.2 psf) for single garage doors (see discussion below).
- Applicable for non-integral garages (e.g., where garages are not integrated into the main structure of the house, where floors/living space are not present above the garage).
- Garage doors are a weak point in the building envelope and breaches of garage doors have been observed to result in roof failure in post-tornado damage assessments in Canada. Breaches in buildings result in internal pressurization, contributing to damage of contents associated with wind and water, and increasing risk of structural damages.¹⁸⁰

A.5. Discussion

The International Residential Code (2018), ASCE 7-16 Minimum Design Loads for Buildings and Other Structures (2017), and the Florida Building Code (2017) include standard methods for calculating the applicable wind loads for garage doors. Buildings are categorized based on the risk to human life in the event of a failure.¹⁸¹ Site-specific wind speeds are identified using a regional wind speed map,¹⁸² and a building's exposure to wind is determined by ground surface roughness of the surrounding environment.¹⁸³ For most Canadian residential applications, the surrounding ground surface roughness is urban, suburban, or wooded (i.e., Exposure B).¹⁸⁴ A topographic factor that accounts for wind speed-up over surrounding hills, ridges, and escarpments may be included in the wind load calculation method.¹⁸⁵ The location of the garage door within the structure can play a role in the effective wind loads,¹⁸⁶ and is considered in several of the standard wind load calculation methods. Additional factors such as wind directionality, ground elevation, and the effect of both external and internal pressures are reflected in the ASCE wind load calculation.¹⁸⁷ Once calculated, wind loads are adjusted for variations in roof height, door dimension, and exposure from the surrounding environment.¹⁸⁸

The Door and Access Systems Manufacturers Association International (DASMA) has published resources outlining approaches for determining garage door wind load requirements across North America according to various jurisdictions' building codes.¹⁸⁹ Guidance documents include spreadsheet tools where building, door, and exposure data can be inputted to generate pressure values for garage doors.¹⁹⁰ Further, a wind load guide was developed according to the specified wind load calculations in the 2010 NBCC.¹⁹¹

The garage door wind load guide was developed using the Static Procedure calculation method for determining the loads exerted on the surfaces of low-rise residential buildings exposed to Canadian wind speeds.¹⁹² The location's $q_{1/50}$, the dimensions of the garage door and its position within the wall, the size of the building, the position of the garage within the building's structure, the mean roof height and slope, and the surrounding terrain are used to calculate the wind load requirements for garage doors using the 2010 NBCC calculation method.¹⁹³ The steps used to develop the guide's wind load values and an example calculation are included in the technical sheet published by DASMA.¹⁹⁴

Several significant technical changes have been made in the 2015 edition of the NBCC, including the incorporation of the wind load commentary section into the body of the code and the introduction of a topographic factor for calculating wind loads.¹⁹⁵ Despite these changes, DASMA's method of applying NBCC wind load calculations to garage doors is consistent across editions. Tables A.4 and A.5 provide wind load values for garage doors calculated using the wind load specifications of the 2010 NBCC. For additional information, see DASMA's Technical Data Series, including Technical Data Sheet #155u.¹⁹⁶

Table A.4: Wind load values (psf) for garage doors calculated using parameters outlined in the 2010 NBCC¹⁹⁷

| Mean roof height | Door size | Design pressures (psf) | | | | |
|-------------------------------|----------------------|------------------------|-------|-------|-------|-------|
| | | | | | | |
| Up to 25 ft. | Single (9' x 7') | 32.7 | 39.1 | 46.0 | 53.3 | 61.1 |
| | | -34.4 | -41.2 | -48.5 | -56.2 | -64.4 |
| | Double (16' x 7') | 32.0 | 38.3 | 45.2 | 52.4 | 60.0 |
| | | -33.7 | -40.3 | -47.7 | -55.3 | -63.4 |
| Mean hourly wind speeds (mph) | | 77 | 84 | 91 | 98 | 105 |
| Fastest mile | | 100 | 110 | 120 | 130 | 140 |
| Peak gust | | 116 | 127 | 138 | 149 | 159 |

Table A.5: Wind load values (kPa) for garage doors calculated using parameters outlined in the 2010 NBCC for design pressures exceeding 0.80 kPa¹⁹⁸

| Mean roof height | Door size | .80 kPa | .90 kPa | 1.00 kPa | 1.10 kPa | 1.20 kPa |
|-------------------------------|------------------------------|---------|---------|----------|----------|----------|
| | | | | | | |
| Up to 7.62 m | Single 2.74 m x 2.13 m | 1.65 | 1.86 | 2.06 | 2.27 | 2.48 |
| | | -1.74 | -1.95 | -2.17 | -2.39 | -2.61 |
| | Double 4.88 m x 2.13 m | 1.61 | 1.81 | 2.02 | 2.22 | 2.42 |
| | | -1.70 | -1.91 | -2.12 | -2.34 | -2.55 |
| Mean hourly wind speeds (m/s) | | 35 | 37 | 39 | 41 | 43 |

While it is considered important to manage the risk of occurrence of breaches in the building envelope that may lead to pressurization of buildings causing roof failure, it was generally agreed by the Stakeholder Committee that high wind rated garage doors should be included as an optional measure. Factors considered in reaching this decision included the following:

- Garage doors are not considered structural, creating logistical issues with building inspections,
- It would be difficult to regulate garage door replacement. For example, a permit requirement would have to be implemented by local authorities, which was considered impractical, and
- Effectiveness of garage doors requires that they be closed during wind events.

It was further acknowledged that measures that contribute to the continuous load path, including those discussed in this report, would serve to limit risk of roof failure. For non-integral garages, other measures, including application of improved RTWCs and adequate continuous vertical load path, would serve to limit risk of roof failure in the event that garage doors fail. Garage doors rated for high wind would not be necessary for integral garages (e.g., where there is an upper storey/living space above the garage).

Appendix B: Gable End Wall Support and Bracing Options

Lateral Support at Gable End Walls¹⁹⁹

The Canadian Wood Council's *Engineering Guide for Wood Frame Construction* (2014 Edition) provides the following guideline for lateral support at gable end walls:

Lateral Support at Gable End Walls

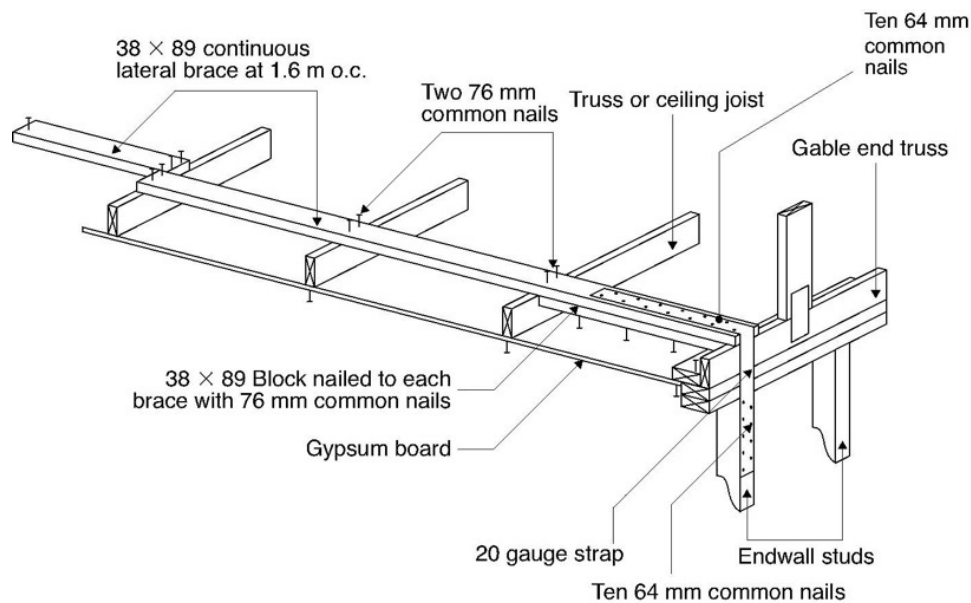
These guidelines are additional to the Part 9 prescriptive requirements of the NBCC

These guidelines for lateral support of gable end wall apply in high wind areas where $0.8 \leq q_{1/50} \leq 1.2$.

Gable end walls are braced as shown in [below figure] by:

- 38 x 89 mm continuous lateral bracing spaced 1.6 m o.c. nailed with two – 76 mm common nails into each truss bottom chord or each ceiling joist.
- 38 x 89 mm blocks between the first truss or ceiling joist and the gable end truss nailed to the continuous lateral bracing with four – 76 mm common nails.
- 20 gauge strapping nailed to the continuous lateral bracing and endwall studs using ten 64 mm common nails at each end.

Figure B.1



The following excerpts of images and text are from Building Component Safety Information (BCSI) Canada. 2014. **Guide to Good Practice for Handling, Installing, Restraining and Bracing of Metal Plate Connected Wood Trusses**. Produced by Structural Building Components Association (SBCA), Truss Plate Institute (TPI), and Truss Plate Institute of Canada (TPIC). For more information, visit sbcindustry.com.

Building Designer Responsibilities for Gable End Frame Bracing²⁰⁰

The building designer, knowing the intended flow of Loads for the entire building, is responsible for taking the resultant Loads that exist within the Gable End Frame and safely transferring the Loads through additional Bracing from the Gable End Frame to the roof and Ceiling Diaphragms.

Gable End Frame Bracing is designed by considering a number of factors including:

- The length, spacing, species and size of the Gable End Frame studs
- Gravity Loads
- Lateral Loads (wind and seismic)

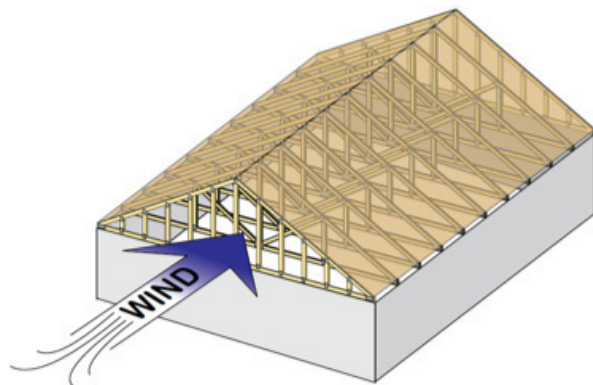
The Building Designer, through detailing in the Construction Documents, is responsible for all Gable End Frame Bracing, including the Bracing member size and locations, attachment to Trusses, gable end sheathing, and fastener size and locations including any mechanical Connectors required.

Other factors the Building Designer shall consider include:

- Thickness and type of roof, wall and ceiling sheathing
- Transfer of Load between the Gable End Frame Bottom Chord and wall below
- Attachment of Structure Sheathing to the wall/Gable End Frame interface and attachment of wall to foundation to resist uplift and lateral Loads

In service, Gable End Frames also experience lateral Loads parallel and perpendicular to their plane. The Gable End Frame shall be incorporated into the wall design by the Building Designer.

Figure B.2



Truss Designer Responsibilities for Gable End Frame Reinforcement

The Truss Designer must note on the [Truss Design Drawing] for the Gable End Frame the type and location of Permanent Individual Truss Restraint (PITMR) required to resist the vertical Loads assumed in the design of the frame. Examples include single or double L, T, U, Scab, horizontal L or any other means of reinforcement deemed appropriate to restrain the out-of-plane buckling on the vertical "studs."

The Truss Designer is responsible for indicating the loading and environmental design assumptions used in the design of the Gable End Frame to conform to the Loads specified in the Construction Documents.

Contractor Responsibilities for Gable End Frame Bracing

The Contractor is responsible for properly installing the Gable End Frame as detailed in the Construction Documents and within the Truss Submittal Package.

Gable End Frame Bracing/Reinforcement Requirements

If the lateral Load is large enough, and the vertical studs are long enough, the Gable End Frame may require Bracing to prevent it from rotating at the Gable End Frame/end wall interface, along with Diagonal Bracing and/or Web Reinforcement to prevent the vertical Webs from bending excessively. Serviceability failures often occur if the Gable End Frame is not properly braced.

Gable End Frame Bracing/reinforcement helps prevent these types of serviceability failures and safely transfers forces from the Gable End Frame into the associated Diaphragms.

Typical Gable End Frame Bracing/reinforcement details include Blocking at the ceiling and roof level Diaphragms, gable stud reinforcement, horizontal reinforcement and/or Diagonal Bracing, mechanical Connectors/straps and specific fastener size and frequency schedules.

Figure B.3: Lateral force transfer to roof and ceiling diaphragms

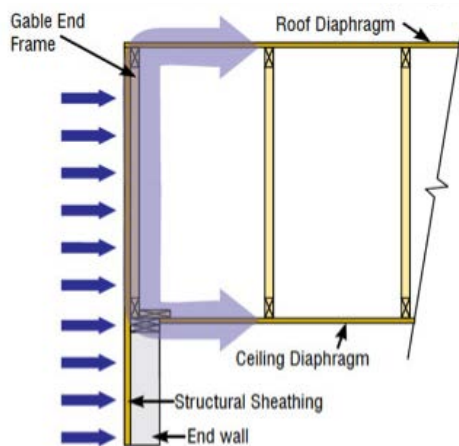


Figure B.4: Potential modes of failure

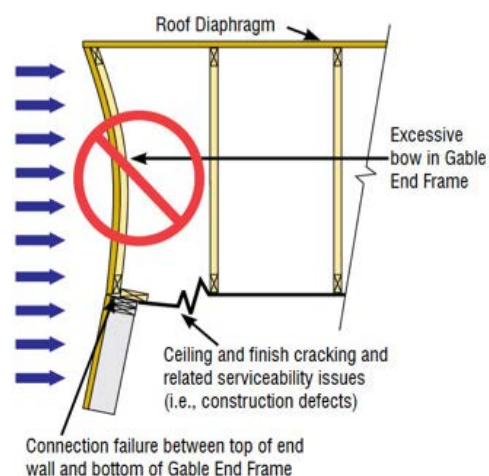


Figure B.5

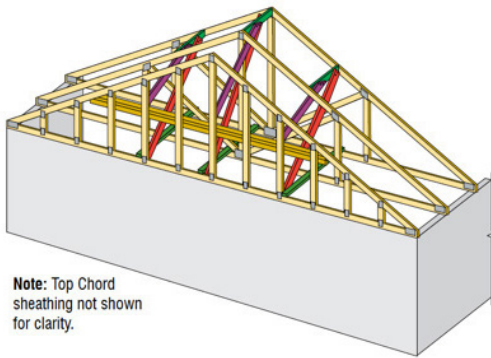
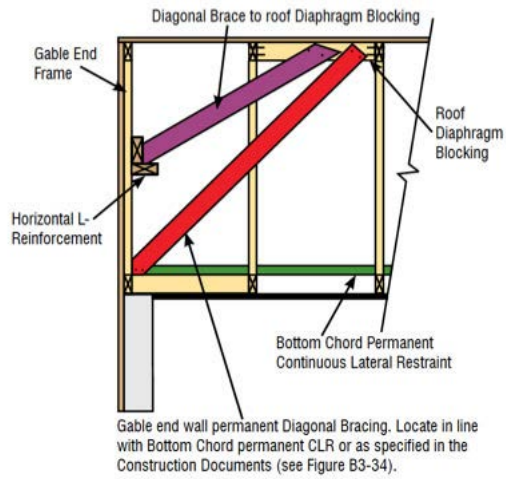
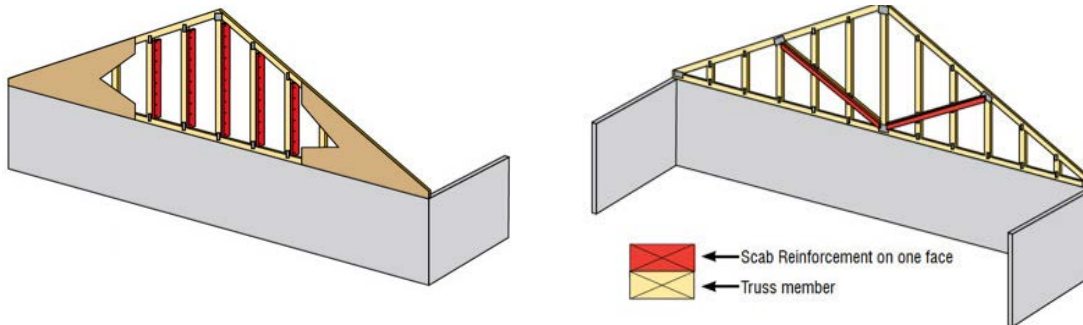


Figure B.6



Note: The Diagonal Brace from the top of the end wall to the top chord of the Truss will impart a vertical force to the Truss Top Chord. This is in addition to any uplift forces the roof sheathing will impart to the Truss from wind. The Loads from this brace must be considered in the design and attachment of the supporting Truss.

Figure B.7: Examples of gable end frame web reinforcement



Appendix C: Stakeholder Workshop

Agenda

Wind Seed Document Stakeholder Committee Workshop
8:30 AM to 4:30 PM
June 28, 2018
Spencer Engineering Building
Western University
London, Ontario

- **Part 1: Lab Tours**
 - Insurance Research Lab for Better Homes
 - Boundary Layer Wind Tunnel Laboratory
- **Part 2: Introductory Presentations:**
 - Introduction to SCC's infrastructure program. Kala Pendakur, Sector Specialist, Strategic Policy and Sector Engagement, Standards Council of Canada.
 - Wind risk reduction research, findings from lab and field work. Gregory Kopp, Professor, Civil and Environmental Engineering, Western University.
 - Dufferin County experience with implementing wind resilience measures. Michael Giles, Chief Building Official, Adjala-Tosorontio, Ontario (formerly of Dufferin County).
- **Part 3: Discussion of Seed Document**
 - Overview, discussion moderated by Dan Sandink, Institute for Catastrophic Loss Reduction.



Figure C.1: Stakeholder Committee members participate in a tour of the Boundary Layer Wind Tunnel, led by Prof. Gregory Kopp



Workshop Notes & Major Decisions

General Comments & Decisions:

Resilience: The term “resilience” is more nuanced than as presented here. Ensure understanding of the scope of the document. In practice, and in the context of recent Canadian disaster risk reduction programs, “resilience” is being used widely and interchangeably to represent any action (social, physical, etc.) that is related to climate change adaptation and disaster risk reduction. The purpose of this document is to contribute to resilience by enhancing the wind resistance of non-engineered, residential buildings in the Canadian context. This will be clarified in the report.

Homeowner representation on future committees: No formal homeowner representation is currently on the Stakeholder Committee. It was generally acknowledged that homeowner representation on this type of committee would be helpful, and this should be pursued in the standard development phase (i.e., the phase that will follow the seed document project). Various strategies were discussed, including inclusion of knowledgeable homeowners (with no affiliation), knowledgeable homeowners who have been directly affected by damaging wind events, and/or inclusion of a “consumer representative” on the standard development committee.

Scoping & application: Include discussion of where measures presented in the report should be applied. Currently NBCC identifies few areas of the country exposed to high wind hazards. The map included in the draft was meant to give an indication of where these types of measures may be appropriate.

Include discussion of a “tiered” approach. Basic provisions (specifically, improved roof-to-wall connections and measures to ensuring that roof covering remains in place), which can be completed at low cost and address a recurring issue, may be applied throughout the country. Additional provisions identified in the report (e.g., bracing, anchoring options) would be applied in regions exposed to higher wind hazards.

Current availability of reliable data on wind hazards is a limitation. It was further noted that NRC is pursuing updated climate data, which may be affected by updated tornado, convective cell and hurricane wind data. Availability of revised data will affect application of wind-resistance measures and benefit-cost assessments.

Note that regions where 1 in 50 HWP is 1.20 kPa and higher are out of scope, as NBCC requires that buildings in these regions be engineered.

Leading to development of voluntary standard: Clarify that this report is a seed document that is meant to serve as the basis for the development of a voluntary, “code-plus” standard.

Benefit-cost discussion: An initial/high level discussion on costs and benefits of identified measures is appropriate for the report (as outlined in the presentation). Include references to available literature, examples from other jurisdictions, cost estimates from available/recommended datasets. Discussion of benefits will be qualitative at this stage.

Detailed/Technical Comments and Decisions:

Sealing the roof deck: Agreement that alternatives to ice-and-water shield should be identified in the report. Reference IBHS Fortified recommendations related to sealing of roof decks. Provide discussion of potential inspections issues (e.g., inspections schedules, access to roof deck may inhibit inspection of roof deck sealing). Include reference to high wind rated roof covering (e.g., high wind rated shingles).

Floor-to-floor connections: Include option of use of metal straps for floor-to-floor connections.

Garage doors:

- Agreed that garage doors should be included as a “second tier” measure:
 - Garage doors are not considered structural, creating logistical issues with inspections.
 - It would be difficult to regulate garage door replacement. For example, a permit requirement would have to be implemented, which was considered impractical.
 - Effectiveness of garage doors requires that they are closed during wind events.
 - Other factors would serve to limit risk of roof failure. For example, garage doors rated for high wind would not be necessary for integral garages (where there is an upper storey above the garage). For non-integral garages, other measures, including application of improved RTWCs, would serve to limit risk of roof failure in the event that garage doors fail.
- Where garage doors are presented as an option, pressure values should be used to classify doors rather than wind speed. Include discussion of this approach and how it has been applied in the US for selection of doors. Where pressure values are used, they should be related to/presented in relevant NBCC units (1 in 50 year HWP, kPa).

Gable end wall bracing, reinforcement & connections:

- Discussion of gable end wall diagrams included in the report (as published by CWC, BCSI and US guides):
 - Connections to the roof diaphragm are not incorporated into the Canadian diagrams because these guides assume truss-built roofs.
 - Note in the report that different roof types and configurations (e.g. stick built roofs, open roofs, roofs with dormers, etc.) require different gable end wall bracing and connection options.
 - Reference the available Canadian and US guidance literature.
- It was noted that the BCSI bracing option identified in the report has been effective in southern Ontario. Specifically, farm buildings without BCSI-type bracing were observed to have failed more frequently than those with this approach applied.

Sheathing:

- Nominal 7/16” structural wood sheathing should be adequate to meet the design goals identified in the report (for both roofs and walls).
- Work with committee members to ensure that industry-appropriate language is applied with respect to sheathing measures.

Post base, cap connections:

- Potential issues associated with inspections of these connections were discussed (i.e., inspection schedules often result in missing inspections of these connections). Mention availability of products that may assist in improved inspections (e.g., visible post base connections that can be installed after pouring of concrete).

Nails/fasteners: Mention importance of nail length, head size and shank diameter.

Partially constructed homes: Discussion of partially constructed homes is warranted in the report. Include discussion of S. Stevenson's results, as available.

Wood sheathing and continuous insulation/thermal efficiency:

- Generally, with respect to thermal efficiency vs. wind risk reduction, it was discussed that use of wood sheathing does not preclude use of continuous exterior insulation. There are several bracing options that would assist in meeting both wind risk reduction/bracing goals and continuous insulation goals. Included in the final report will also be discussion of work by S. Stevenson, which will include modelling of various sheathing scenarios.
- It was noted that thermal efficiency measures and bracing measures do not "conflict," and this type of terminology should not be used in the report.
- Eaves/overhangs: It was further noted that longer eaves as discussed during the meeting/in the draft report provide multiple benefits and would not exacerbate wind risk provided that adequate RTWCs are used.

Inspections:

- It was generally agreed that inspection issues should be discussed/highlighted in the report. Highlights of the discussion included:
 - All provisions outlined in the report can be incorporated into plans that are approved by municipalities, but it is not possible under any scenario for every inspection to be completed. Inspections are always difficult to complete, but this should not preclude pursuance of the types of measures identified in the report.
 - Inspection issues exist for measures discussed in the report because they may not be part of normal inspection schedules (e.g., roof deck sealing, garage doors).
 - One of the most effective means of overcoming inspections issues is to train trades people on these types of measures early in their careers (e.g., incorporate into trade school curriculum).

Roof-to-wall connections:

- Reword RTWC provision to allow for use of alternative products/methods that provide the same uplift capacity as hurricane ties (e.g., product identified by B. Bunting, fastening wall sheathing to raised-heel trusses).
- Raised-heel trusses: Increasing use/encouragement of use of raised-heel trusses (for the purposes of improved attic insulation) also allows for an additional RTWC option, as the wall sheathing can be directly fastened to trusses. This should be identified as a thermal efficiency/wind resistance co-benefit.

Meeting Attendance:

Mike Giles, Township of Adjala
Tosorontio, ON

Tony Muscedere, Municipality of
Leamington, ON

Gregory Kopp, Western University
Sarah Stevenson, Western University
Brad Baumgarten, Red Deer County, AB

Natalie Dale, U of T/ICLR

Dave Dean, City of Windsor, ON

Dave Hiscock, Town of Bonavista, NL

Kevin Law, TD Insurance

Harshan Radhakrishnan, Engineers
and Geoscientists BC

Kevin Rocchi, Witzel Dyce Engineering

Brent Bunting, Simpson Strong-Tie

Robert Jonkman, Canadian Wood
Council

Kala Pendakur, Standards Council of
Canada

John van de Lindt, Colorado State
University (via webinar/phone)

David Potter, Town of Newmarket, ON

Dan Sandink, ICLR

***Not present (all provided written
comments):***

David O. Prevatt, University of Florida

Chris Rol, Insurance Bureau of Canada

David Foster, Canadian Home Builders'
Association

Paul Holmes, City of Red Deer, AB

Randy Van Straaten, RDH Building
Science Laboratories

Appendix D: 2015 NBCC Part 9 References Related to Measures Presented in Appendix A

NBCC Reference Related to Measure A.1.2.

NBCC Table 9.23.3.4.

- Roof rafter, roof truss or roof joist to plate – toe nail
 - Minimum length of nails (mm): 82
 - Minimum number of nails: 3
 - See note 3, which refers to sentence 9.23.3.4.(3).

NBCC 9.23.3.4.(3): Where the 1-in-50 hourly wind pressure is equal to or greater than 0.8 kPa, roof rafters, joists or trusses shall be tied to the wall framing with connectors that will resist a factored uplift load of 3 kN.

NBCC 9.23.3.4.(4): Galvanized-steel straps are deemed to comply with Sentence (3), provided they are

- a) 50 mm wide,
- b) not less than 0.91 mm thick, and
- c) fastened at each end with at least four 63 mm nails.

NBCC Reference Related to Measure A.1.3.

Relevant NBCC wording related to fasteners, fastener length and fastener spacing:

NBCC Article 9.23.3.5. Fasteners for Sheathing or Subflooring. Summary of provisions relevant to this document:

NBCC 9.23.3.5.(1) Except as provided in Sentences (2) to (4), fastening of sheathing and subflooring shall conform to Table 9.23.3.5.-A (applies where 1-in-50 hourly wind pressure (HWP) is <0.8 kPa):

Summary of provision: For plywood [or OSB]...over 10 mm and up to 20 mm thick, minimum length of ring thread nails is 45 mm, and maximum spacing of fasteners is 150 mm o.c. along edges and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(2) Fastening of **roof sheathing** and sheathing in required braced wall panels shall conform to Table 9.23.3.5.-B (applies where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa):

Summary of provision: For plywood [or OSB]...up to 20 mm thick⁽¹⁾, minimum length of ring thread nails is 63 mm, maximum spacing of fasteners: 150 mm o.c. and 300 mm o.c. along intermediate supports.

Roof Sheathing: 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.

NBCC 9.23.3.5.(3) Fastening of **roof sheathing** and sheathing in *braced wall panels* shall conform to Table 9.23.3.5.-C (applies where (a) the where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa and the spectral response acceleration, $S_a(0.2)$, is not more than 1.8, or (b) the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.90 and not more than 1.8):

Plywood [or OSB]...up to 20 mm thick, ring thread nail minimum length is 63 mm and the maximum spacing of fasteners is 75 mm o.c. along edges and 300 mm o.c. along intermediate supports.

Roof Sheathing: For roof sheathing where the 1-in-50 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.

9.23.3.5.(4) Fastening of sheathing shall conform to Part 4 (a) where the 1-in-50 HWP is equal to or greater than 1.2 kPa...

Note that common and spiral nails are permitted in all of the above NBCC 9.23.3.5 provisions. Fasteners placed at 150 mm o.c. for sheathing panel intermediate supports exceeds the above NBCC 9.23.3.5 provisions.

2015 NBCC Roof Sheathing Thickness Section

NBCC 9.23.16.7. Thickness or Rating

- 1) The thickness or rating of roof sheathing on a flat roof used as a walking deck shall conform to either Table 9.23.15.5.-A or Table 9.23.15.5.-B for subfloors.
- 2) The thickness or rating of roof sheathing on a roof not used as a walking deck shall conform to either Table 9.23.16.7.-A or Table 9.23.16.7.-B
- 3) Asphalt-coated or asphalt-impregnated fibreboard not less than 11.1 mm thick conforming to CAN/ULC-S706, "Wood Fibre Insulating Boards for Buildings," is permitted to be used as a roof sheathing over supports spaced not more than 400 mm o.c. provided the roofing consists of
 - a. a continuous sheet of galvanized steel not less than 0.33 mm in thickness, or
 - b. a continuous sheet of aluminum not less than 0.61 mm in thickness.
- 4) All edges of sheathing described in Sentence (3) shall be supported by blocking or framing.

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 | | | | Lumber |
|---------------------------------|-----------------------------|-------------------|------------------------------|-------------------|--------|
| | Plywood, and OSB, O-2 Grade | | OSB, O-1 Grade..., R-1 Grade | | |
| | 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 |

Table 9.23.16.7.-B.: Rating for Roof Sheathing When Applying CSA O325
Forming Part of Sentence 9.23.16.7.(2)

| Maximum Spacing of Supports, mm | Panel mark | |
|---------------------------------|-----------------|-------------------|
| | Edges Supported | Edges Unsupported |
| 400 | 2R16 | 1R16 |
| 500 | 2R20 | 1R20 |
| 600 | 2R24 | 1R24 |

NBCC Reference Related to Measure A.1.4.

*2015 NBCC: 9.26.6. Underlay Beneath Shingles*²⁰¹

9.26.6.1. Materials

- 1) Except as required in Sentence (2), when underlay is used beneath shingles, it shall be
 - a) asphalt-saturated sheathing paper weighing not less than 0.195 kg/m², or
 - b) No. 15 plain or perforated asphalt-saturated felt.
- 2) Underlay beneath wood shingles shall be breather type.

9.26.6.2. Installation

- 1) When used with shingles, underlay shall be installed parallel to the eaves with head and end lap of not less than 50 mm.
- 2) The top edge of each strip of underlay referred to in Sentence (1) shall be fastened with sufficient roofing nails to hold it in place until the shingles are applied.
- 3) The underlay referenced in Sentence (1) shall overlap the eave protection by not less than 100 mm. (see Article 9.26.10.2. for underlay beneath wood shakes).

NBCC Article 9.26.10.2:

9.26.10. Cedar Roof Shakes

9.26.10.2. Underlay

- 1) Where eave protection is not provided, an underlay conforming to the requirements in Article 9.26.6.1. for wood shingles shall be laid as a strip not less than 900 mm wide along the eaves.
- 2) A strip of material similar to that described in Sentence (1) not less than 450 mm wide shall be interlaid between each course of shakes with the bottom edge of the strip position above the butt line at a distance equal to double the exposure of the shakes.
- 3) Interlaid strips referred to in Sentence (2) shall be lapped not less than 150 mm at hips and ridges in a manner that will prevent water from reaching the roof sheathing.

NBCC Reference Related to A.2 Measures

NBCC 9.23.17. Wall Sheathing

9.23.17.1. Required Sheathing

- 1) Exterior walls and gable ends shall be sheathed when the exterior cladding requires intermediate fastening between supports or if the exterior cladding requires solid backing.

NBCC wording related to fasteners, fastener length and fastener spacing:

NBCC Article 9.23.3.5. Fasteners for Sheathing or Subflooring. Summary of provisions relevant to this document:

NBCC 9.23.3.5.(1) Except as provided in Sentences (2) to (4), fastening of sheathing and subflooring shall conform to Table 9.23.3.5.-A (applies where 1-in-50 hourly wind pressure is <0.8 kPa):

Summary of provision: For plywood [or OSB]...over 10 mm and up to 20 mm thick, minimum length of ring thread nails is 45 mm, and maximum spacing of fasteners is 150 mm o.c. along edges and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(2) Fastening of roof sheathing and sheathing in required braced wall panels shall conform to Table 9.23.3.5.-B (applies where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa):

Summary of provision: For plywood [or OSB]...up to 20 mm thick⁽¹⁾, minimum length of ring thread nails is 63 mm, maximum spacing of fasteners: 150 mm o.c. and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(3) Fastening of roof sheathing and sheathing in *braced wall panels* shall conform to Table 9.23.3.5.-C (applies where (a) the where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa and the spectral response acceleration, $S_a(0.2)$, is not more than 1.8, or (b) the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.90 and not more than 1.8):

Plywood [or OSB]...up to 20 mm thick, ring thread nail minimum length is 63 mm and the maximum spacing of fasteners is 75 mm o.c. along edges and 300 mm o.c. along intermediate supports.

9.23.3.5.(4): Fastening of sheathing shall conform to Part 4 (a) where the 1-in-50 HWP is equal to or greater than 1.2 kPa.

NBCC wording related to stud spacing:

NBCC 9.23.10 Wall Studs

NBCC 9.23.10.1 Size and Spacing of Studs (Summary of provisions in Table):

- Exterior wall supporting roof with or without attic storage plus 1 floor – with 38 mm x 89 mm (2 x 4s) spacing max is 400 mm (16”), with 38 mm x 140 mm (2 x 6s), spacing max is 600 mm (24”).
- Exterior wall supporting roof with or without attic storage plus 2 floors – with 38 mm x 89 mm (2 x 4s) spacing max is 300 mm (12”), with 64 mm x 89 mm, spacing max is 400 mm (16”), with 38 mm x 140 mm (2 x 6s), spacing max is 400 mm (16”).
- Exterior wall supporting roof with or without attic storage plus 3 floors – with 38 mm x 140 mm (2 x 6s), spacing max is 300 mm (12”).

NBCC wording related to bracing, lateral support

NBCC 9.23.10.2. Bracing and Lateral Support

- 1) Where *loadbearing* interior walls are not finished in accordance with Section 9.29., blocking or strapping shall be fastened to the studs at mid-height to prevent sideways buckling.

NBCC 9.23.13. Bracing to Resist Lateral Loads Due to Wind and Earthquake

See note A-9.23.13. (A-9.23.13. Bracing for Resistance to Lateral Loads. Subsection 9.23.14 along with Articles 9.23.3.4., 9.23.3.5., 9.23.6.1., 9.23.9.8., 9.23.15.5., 9.29.5.8., 9.29.5.9., 9.29.6.3., and 9.29.9.3.²⁰² provide explicit instructions to address resistance to wind and earthquake loads in higher wind and earthquake regions of Canada.)

NBCC 9.23.13.1. Requirements for Low to Moderate Wind and Seismic Forces

See Note A-9.23.13.1²⁰³

- 1) This Article applies in locations where...the 1-in-50 hourly wind pressure is less than 0.8 kPa.
- 2) Bracing to resist lateral loads shall be designed and constructed as follows:
 - a) exterior walls shall be
 - i) clad with panel-type cladding in accordance with Section 9.27
 - ii) sheathed with plywood, OSB...fibreboard, gypsum board or diagonal lumber sheathing complying with Subsection 9.23.16. [sic] and fastened in accordance with Table 9.23.3.5.-A, or
 - iii) finished on the interior with a panel type material in accordance with the requirements of Section 9.29., or
 - b) in accordance with
 - i) Articles 9.23.13.4. to 9.23.13.7.²⁰⁴,
 - ii) Part 4, or
 - iii) good engineering practice such as that provided in CWC 2014, “Engineering Guide for Wood Frame Construction.”

NBCC 9.23.13.2. Requirements for High Wind and Seismic Forces

- 1) Except as provided in Article 9.23.13.1, this Article applies in locations where...
 - b) the 1-in-50 hourly wind pressure is less than 1.20 kPa.
- 2) Bracing to resist lateral loads shall be designed and constructed in accordance with
 - a) Articles 9.23.13.4. to 9.23.13.7.,
 - b) Part 4, or
 - c) good engineering practice such as that provided in CWC 2014, "Engineering Guide for Wood Frame Construction."

NBCC 9.23.13.3. Requirements for Extreme Wind and Seismic Forces

- 1) Except as provide in Articles 9.23.13.1. and 9.23.13.2., this Article applies in locations where...
 - b) the 1-in-50 hourly wind pressure is equal to or greater than 1.20 kPa.
- 2) Bracing to resist lateral loads shall be designed and constructed in accordance with
 - a) Part 4, or
 - b) good engineering practice such as that provided in CWC 2014, "Engineering Guide for Wood Frame Construction."

NBCC Appendix: A-9.23.13.1. states:

Bracing to Resist Lateral Loads in Low Load Locations

Of the 679 locations identified in Appendix C, 614 are locations where the seismic spectral response acceleration, $S_a(0.2)$, is less than or equal to 0.70 and the 1-in-50 hourly wind pressure is ***less than 0.80 kPa. For buildings in these locations, Sentence 9.23.13.1.(2) requires only that exterior walls be braced using the acceptable materials and fastening specified.***

There are no spacing or dimension requirements for braced wall panels in these buildings. (Emphasis added).

Structural Design for Lateral Wind and Earthquake Loads

In cases where lateral load design is required, CWC 2014 "Engineering Guide for Wood Frame Construction," provides acceptable engineering solutions as an alternative to Part 4. The CWC Guide also contains alternative solutions and provides information on the applicability of Part 9 prescriptive structural requirements to further assist designers and building officials to identify the appropriate design approach.

Relevant NBCC wording related to fasteners, fastener length and fastener spacing:

NBCC Article 9.23.3.5. Fasteners for Sheathing or Subflooring. Summary of provisions relevant to this document:

NBCC 9.23.3.5.(1) Except as provided in Sentences (2) to (4), fastening of sheathing and subflooring shall conform to Table 9.23.3.5.-A (applies where 1-in-50 hourly wind pressure (HWP) is <0.8 kPa):

Summary of provision: For plywood [or OSB]...over 10 mm and up to 20 mm thick, minimum length of ring thread nails is 45 mm, and maximum spacing of fasteners is 150 mm o.c. along edges and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(2) Fastening of roof sheathing and sheathing in required braced wall panels shall conform to Table 9.23.3.5.-B (applies where the 1-in-50 HWP is greater than or equal to 0.8 kPa and less than 1.2 kPa):

Summary of provision: For plywood [or OSB]...up to 20 mm thick⁽¹⁾, minimum length of ring thread nails is 63 mm, maximum spacing of fasteners: 150 mm o.c. and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(3) Fastening of roof sheathing and sheathing in *braced wall panels* shall conform to Table 9.23.3.5.-C (applies where (a) the where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa and the spectral response acceleration, $S_a(0.2)$, is not more than 1.8, or (b) the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.90 and not more than 1.8):

Plywood [or OSB]...up to 20 mm thick, ring thread nail minimum length is 63 mm and the maximum spacing of fasteners is 75 mm o.c. along edges and 300 mm o.c. along intermediate supports.

9.23.3.5.(4): Fastening of sheathing shall conform to Part 4 (a) where the 1-in-50 HWP is equal to or greater than 1.2 kPa.

NBCC Wording related to lapping rim joist:

NBCC 9.23.3.4 Nailing of Framing

- 2) Where the bottom wall plate or sole plate of an exterior wall is not nailed to floor joists, *rim joists* or blocking in conformance with Table 9.23.3.4., the exterior wall is permitted to be fastened to the floor framing by
 - a) having plywood [or OSB]...sheathing extend down over floor framing and fastened to the floor framing by nails or staples conforming to Article 9.23.3.5., or
 - b) tying wall framing to the floor framing by galvanized-metal strips
 - i) 50 mm wide,
 - ii) not less than 0.41 mm thick,
 - iii) spaced not more than 1.2 m apart, and
 - iv) fastened at each end with at least two 63 mm nails.

Relevant NBCC wording related to fasteners, fastener length and fastener spacing:

NBCC Article 9.23.3.5. Fasteners for Sheathing or Subflooring. Summary of provisions relevant to this document:

NBCC 9.23.3.5.(1) Except as provided in Sentences (2) to (4), fastening of sheathing and subflooring shall conform to Table 9.23.3.5.-A (applies where 1-in-50 hourly wind pressure (HWP) is <0.8 kPa):

Summary of provision: For plywood [or OSB]...over 10 mm and up to 20 mm thick, minimum length of ring thread nails is 45 mm, and maximum spacing of fasteners is 150 mm o.c. along edges and 300 mm o.c. along intermediate supports.

NBCC 9.23.3.5.(2) Fastening of roof sheathing and sheathing in required *braced wall panels* shall conform to Table 9.23.3.5.-B (applies where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa):

Summary of provision: For plywood, [or OSB]...up to 20 mm thick⁽¹⁾, minimum length of ring thread nails is 63 mm, maximum spacing of fasteners: 150 mm o.c. and 300 mm o.c. along intermediate supports..

NBCC 9.23.3.5.(3) Fastening of roof sheathing and sheathing in *braced wall panels* shall conform to Table 9.23.3.5.-C (applies where (a) the where the 1-in-50 HWP is greater or equal to 0.8 kPa and less than 1.2 kPa and the spectral response acceleration, $S_a(0.2)$, is not more than 1.8, or (b) the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.90 and not more than 1.8):

Plywood [or OSB]...up to 20 mm thick, ring thread nail minimum length is 63 mm and the maximum spacing of fasteners is 75 mm o.c. along edges and 300 mm o.c. along intermediate supports.

9.23.3.5.(4): Fastening of sheathing shall conform to Part 4 (a) where the 1-in-50 HWP is equal to or greater than 1.2 kPa.

NBCC Wording related to lapping rim joist:

NBCC 9.23.3.4 Nailing of Framing

- 2) Where the bottom wall plate or sole plate of an exterior wall is not nailed to floor joists, *rim joists* or blocking in conformance with Table 9.23.3.4., the exterior wall is permitted to be fastened to the floor framing by
 - a) having plywood [or OSB]...sheathing extend down over floor framing and fastened to the floor framing by nails or staples conforming to Article 9.23.3.5., or
 - b) tying wall framing to the floor framing by galvanized-metal strips
 - i) 50 mm wide,
 - ii) not less than 0.41 mm thick,
 - iii) spaced not more than 1.2 m apart, and
 - iv) fastened at each end with at least two 63 mm nails.

NBCC Reference Related to Measure A.3.

9.23.6.1 Anchorage of Building Frames

- 1) Except as required by Sentence 9.23.6.3.(1), *building* frames shall be anchored to the *foundation* unless a structural analysis of wind and earthquake pressures show anchorage is not required.
- 2) Except as provided in Sentences (3) to (6), anchorage shall be provided by
 - a) embedding the ends of the first floor joists in concrete, or
 - b) fastening the sill plate to the *foundation* with not less than 12.7 mm diam. anchor bolts spaced not more than 2.4 m o.c.

- 3) For *buildings* with 2 or more floors supported by frame walls that are in areas where the seismic spectral response acceleration, $S_a(0.2)$, is not greater than 0.70 or the 1-in-50 hourly wind pressure (HWP) is equal to or greater than 0.8 kPa but not greater than 1.20 kPa, anchorage shall be provided by fastening the sill plate to the *foundation* with not less than two anchor bolts per braced wall panel, where the anchor bolts used are
- not less than 15.9 mm in diameter, located within 0.5 m of the end of the *foundation*, and spaced not more than 2.4 m o.c. or
 - not less than 12.7 mm in diameter, located within 0.5 m of the end of the *foundation*, and spaced not more than 1.7 m o.c.
- 4) For buildings supported by frame walls that are in areas where the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.70 but not greater than 1.8 and the 1-in-50 hourly wind pressure (HWP) is not greater than 1.20 kPa, anchorage shall be provided by fastening the sill plate to the *foundation* with not less than two anchor bolts per *braced wall panel* located within 0.5 m of the end of the *foundation* and spaced in accordance with Table 9.23.6.1.

Table 9.23.6.1.: Anchor Bolt Spacing where the 1-in-50 HWP 1.20 kPa and $0.70 < S_a(0.2) \leq 1.8$
Forming Part of Sentence 9.23.6.1(4)

| Anchor Bolt Diameter, mm | $S_a(0.2)$ | Maximum Spacing of Anchor Bolts Along Braced Wall Band, m | | | | | |
|--------------------------|-----------------------------|---|-----|-----|-----------------------------------|-----|--|
| | | Light Construction | | | Heavy Construction ⁽¹⁾ | | |
| | | Number of Floors Supported ⁽²⁾ | | | | | |
| | | 1 | 2 | 3 | 1 | 2 | |
| 12.7 | $0.70 < S_a(0.2) \leq 0.80$ | 2.4 | 2.3 | 1.8 | 2.4 | 2.0 | |
| | $0.80 < S_a(0.2) \leq 0.90$ | 2.4 | 2.3 | 1.8 | 2.4 | 2.0 | |
| | $0.90 < S_a(0.2) \leq 1.0$ | 2.4 | 2.2 | 1.5 | 2.4 | 1.8 | |
| | $1.0 < S_a(0.2) \leq 1.1$ | 2.4 | 2.1 | 1.4 | 2.4 | 1.6 | |
| | $1.1 < S_a(0.2) \leq 1.2$ | 2.4 | 2.0 | 1.3 | 2.4 | 1.5 | |
| | $1.2 < S_a(0.2) \leq 1.3$ | 2.4 | 1.9 | 1.3 | 2.4 | 1.5 | |
| | $1.3 < S_a(0.2) \leq 1.35$ | 2.4 | 1.8 | 1.2 | 2.3 | 1.4 | |
| | $1.35 < S_a(0.2) \leq 1.8$ | 2.4 | 1.8 | 1.1 | 2.3 | 1.4 | |
| 15.9 | $0.70 < S_a(0.2) \leq 0.80$ | 2.4 | 2.4 | 2.2 | 2.4 | 2.4 | |
| | $0.80 < S_a(0.2) \leq 0.90$ | 2.4 | 2.4 | 2.2 | 2.4 | 2.4 | |
| | $0.90 < S_a(0.2) \leq 1.0$ | 2.4 | 2.4 | 2.1 | 2.4 | 2.3 | |
| | $1.0 < S_a(0.2) \leq 1.1$ | 2.4 | 2.4 | 1.9 | 2.4 | 2.3 | |
| | $1.1 < S_a(0.2) \leq 1.2$ | 2.4 | 2.4 | 1.9 | 2.4 | 2.2 | |
| | $1.2 < S_a(0.2) \leq 1.3$ | 2.4 | 2.4 | 1.8 | 2.4 | 2.1 | |
| | $1.3 < S_a(0.2) \leq 1.35$ | 2.4 | 2.3 | 1.7 | 2.4 | 2.0 | |
| | $1.35 < S_a(0.2) \leq 1.8$ | 2.4 | 2.2 | 1.6 | 2.4 | 1.9 | |

Notes to Table 9.23.6.1.:

(1) See Note A-9.23.13.2.(1)(a)(i).

(2) All constructions include support of a roof load in addition to the indicated number of floors.

- 5) Anchor bolts referred to in Sentences (2) to (4) shall be
 - a) fastened to the sill plate with nuts and washers,
 - b) embedded not less than 100 mm in the *foundation*, and
 - c) so designed that they may be tightened without withdrawing them from the *foundation*.
- 6) Where the seismic spectral response acceleration, $S_a(0.2)$, is greater than 1.8 or the 1-in-50 hourly wind pressure is equal to or greater than 1.2 kPa, anchorage shall be designed according to Part 4.

NBCC Reference Related to Measure A.4.

NBCC 9.23.6. Anchorage

NBCC 9.23.6.2 Anchorage of Columns and Posts

- 1) Except as provided in Sentences (2) and (3), exterior columns and posts shall be anchored to resist uplift and lateral movement.
- 2) Except as provided in Sentence (3), where columns and posts support balconies, decks, verandas or other exterior platforms, and the distance from finished ground to the underside of the joists is not more than 600 mm,
 - a) the columns or posts shall be anchored to the foundation to resist uplift and lateral movement, or
 - b) the supported joists or beams shall be directly anchored to the ground to resist uplift.
- 3) Anchorage is not required for platforms described in Sentence (20) that
 - a) are not more than 1 *storey* in height,
 - b) are not more than 55 m² in area,
 - c) do not support a roof, and
 - d) are not attached to another structure, unless it can be demonstrated that differential movement will not adversely affect the performance of the structure to which the platform is attached.

NBCC 9.35. Garages and Carports

NBCC 9.35.4. Walls and Columns

NBCC 9.35.4.3.

- 1) Garage or carport walls and columns shall be anchored to the foundation to resist wind uplift in conformance with Subsection 9.23.6., except where that garage is supported on the surface of the ground, ground anchors shall be provided to resist wind uplift.

NBCC Reference Related to Measure A.5.

No specific NBCC provision for garage door wind resistance in Section 9.35 – Garages and Carports.

NBCC 9.35. Garages and Carports

NBCC 9.35.1.2. Construction Requirements

- 1) The construction of a garage or carport shall conform to the requirements for other buildings in this Part, except as provided in this Section.

9.7.3. Performance of Windows, Doors and Skylights

9.7.3.1. General Performance Expectations

- 1) Except as provided in Sentences (2) to (4), windows, doors and skylights and their components separating conditioned space from unconditioned space or the exterior shall be designed, constructed and installed so that, when in closed position, they
 - a) resist the ingress of precipitation into interior space (see Note A-9.7.4.2.(1)),
 - b) resist wind loads,
 - c) control air leakage,
 - d) resist the ingress of insects and vermin,
 - e) where required, resist forced entry, and
 - f) are easily operable when not intended to be fixed.

Appendix E: 2010 OBC Code Change Request – Roof Sheathing Fastener Spacing

Source: Ministry of Municipal Affairs and Housing, Building and Development Branch. September 2010. Toronto: Queen’s Printer for Ontario.

Ontario’s Building Code

| Ontario only | PROPOSED CHANGE TO THE 2006 BUILDING CODE | | |
|--|---|-----------------------|-------------------------------------|
| Change number | B-09-23-06 | Code reference | Div. B/ 9.23.3.5.(5) new |
| Description of proposed amendment | Reduce maximum spacing of fasteners for roof sheathing from 300 mm to 150 mm along intermediate supports. | | |

Existing 2006 Building Code Provision(s):

9.23.3.5. Fastening for Sheathing or Subflooring

- (1) Fastening of sheathing and subflooring shall conform to Table 9.23.3.5.
- (2) Staples shall not be less than 1.6 mm (1/16 in) in diameter or thickness, with not less than a 9.5 mm (3/8 in) crown driven with the crown parallel to framing.
- (3) Roofing nails for the attachment of fibreboard or gypsum sheathing shall not be less than 3.2 mm (1/8 in) in diameter with a minimum head diameter of 11.1 mm (7/16 in).
- (4) Flooring screws shall not be less than 3.2 mm (1/8 in) in diameter.

Table 9.23.3.5.: Fasteners for Sheathing and Subflooring
Forming Part of Sentence 9.23.3.5.(1)

| Element | Minimum Length of Fasteners, mm (in) | | | | Minimum Number or Maximum Spacing of Fasteners |
|--|--------------------------------------|-----------------------------|---------------|---------|--|
| | Common or Spiral Nails | Ring Thread Nails or Screws | Roofing Nails | Staples | |
| Board lumber 184 mm (7¼ in) or less wide | 51 (2) | 45 (1¾) | N/A | 51 (2) | 2 per support |
| Board Lumber more than 184 mm (7¼ in) wide | 51 (2) | 45 (1¾) | N/A | 51 (2) | 3 per support |
| Fibreboard sheathing up to 13 mm (½ in) thick | N/A | N/A | 44 (1¾) | 28 (1⅛) | 150 mm (5⅞ in) (o.c.) along edges and 300 mm (11¾ in) (o.c.) along intermediate supports |
| Gypsum sheathing up to 13 mm (½ in) thick | N/A | N/A | 44 (1¾) | N/A | |
| Plywood, OSB or waferboard up to 10 mm (⅜ in) thick | 51 (2) | 45 (1¾) | N/A | 38 (1½) | |
| Plywood, OSB or waferboard from 10 mm (⅜ in) to 20 mm (13/16 in) thick | 51 (2) | 45 (1¾) | N/A | 51 (2) | |
| Plywood, OSB or waferboard over 20 mm (13/16 in) thick | 57 (2¼) | 51 (2) | N/A | N/A | |
| Column 1 | 2 | 3 | 4 | 5 | 6 |

PROPOSED CODE CHANGE:

Add new Sentence 9.23.3.5.(5) as follows:

- (5) The maximum spacing of fasteners for roof sheathing shall be 150 mm along edges and intermediate supports.

RATIONALE FOR CHANGE:

Problem/General Background

The proponent states that the current Code provision with maximum fastener spacing of 300 mm along intermediate supports for roof sheathing has proven to be insufficient in cases of severe wind conditions which occurred in recent tornadoes in Ontario.

A study based on a test house showed that nails in roof sheathing were improperly fastened or were missing. It was observed that for each roof sheathing panel, at least one nail was missing or improperly fastened resulting in a decrease of about 5% – 10% in the mean uplift capacity.

Justification/Explanation

The proponent states that by reducing the intermediate fastener spacing from 300 mm to 150 mm, the minimum number of nails will increase to 45 from 33 (for framing spaced 600 mm o.c.), but that uplift capacity will be approximately doubled.

Advanced finite element modeling, probabilistic analysis and simulation technique was used in the numerical analysis. The analysis was based on a model roof sheathing panel 4' x 8' x 3/8" (3-ply) plywood spaced at 600 mm on centre. Common 8d nails were used (2-1/2" length with 0.133" diameter). The study focused on nail withdrawal rather than nail punching failure mode.

Supporting Material: A draft manuscript authored by W. He and H.P. Hong, and two additional papers related to NBCC (2005).

He, W. and Hong, H.P. (2010) Probabilistic Characterizations of Roof Panel Uplift Capacity Subjected to Wind Loading, To be submitted for publication.

Bartlett, F.M., Hong, H.P. and Zhou, W. 2003. Load factor calibration for NBCC 2004: Companion action load combinations Canadian Journal of Civil Engineering, 30 (2) 429-439.

Bartlett, F.M., Hong, H.P. and Zhou, W. 2003. Load factor calibration for NBCC 2004: Statistics of loads and load effects, Canadian Journal of Civil Engineering, 30 (2) 440-448.

Cost/Benefit Implications

This will lead to a minimal cost increase in the construction of new buildings.

Enforcement Implications

Can be enforced using current resources.

Who is Affected

Designers, insurers, builders, building occupants, owners and building officials.

Objective Based Analysis

Unchanged.

Appendix F: 2016 OBC Code Change Request – Roof-to-Wall Connections

Source: Ministry of Municipal Affairs, Building Development Branch. 2017. Toronto: Queen’s Printer for Ontario.

PROPOSED CHANGE TO THE 2012 BUILDING CODE O. REG. 332/12 AS AMENDED

CHANGE NUMBER: 2-CC-B-09-23-01

SOURCE: Ontario

CODE REFERENCE: Div. B, 9.23.3.4. DESCRIPTION OF THE PROPOSED AMENDMENT

The proposed change requires roof rafter, roof truss or roof joist to be tied to the wall framing with engineered connectors to resist higher uplift loads.

EXISTING 2012 BUILDING CODE PROVISION(S)

Section 9.23. Wood Frame Construction

9.23.3. Fasteners

9.23.3.4. Nailing of Framing

- (1) Except as provided in Sentence (2), nailing of framing shall conform to Table 9.23.3.4.
- (2) Where the bottom wall plate or sole plate of an exterior wall is not nailed to joists or blocking in conformance with Table 9.23.3.4., the exterior wall may be fastened to the floor framing by,
 - (a) having plywood, OSB or waferboard sheathing extend down over floor framing and fastened to the floor framing by nails or staples conforming to Article 9.23.3.5., or
 - (b) tying the wall framing to the floor framing by 50 mm wide galvanized-metal strips,
 - (i) not less than 0.41 mm in thickness,
 - (ii) spaced not more than 1.2 m apart, and
 - (iii) fastened at each end with at least two 63 mm nails.

Excerpt from: Table 9.23.3.4. Nailing for Framing

Forming Part of Sentence 9.23.3.4.(1)

| Construction Detail | Minimum Length of Nails, mm | Minimum Number or Maximum Spacing of Nails |
|---|-----------------------------|--|
| Roof rafter, roof truss or roof joist to plate – toe nail | 82 | 3 |
| Column 1 | 2 | 3 |

PROPOSED CODE CHANGE

Revised Article 9.23.3.4 as follows:

Section 9.23. Wood Frame Construction

9.23.3. Fasteners

9.23.3.4. Nailing of Framing

- (1) Except as provided in Sentence (2) and (3), nailing of framing shall conform to Table 9.23.3.4.
- (2) Where the bottom wall plate or sole plate of an exterior wall is not nailed to joists or blocking in conformance with Table 9.23.3.4., the exterior wall may be fastened to the floor framing by,
 - (a) having plywood, OSB or waferboard sheathing extend down over floor framing and fastened to the floor framing by nails or staples conforming to Article 9.23.3.5., or
 - (b) tying the wall framing to the floor framing by 50 mm wide galvanized-metal strips,
 - (i) not less than 0.41 mm in thickness,
 - (ii) spaced not more than 1.2 m apart, and
 - (iii) fastened at each end with at least two 63 mm nails.
- (3) Roof rafter, roof truss or roof joist shall be tied to loadbearing walls framing with engineered connectors that will resist a factored uplift load of 3 kN. (See Appendix A.)
- (4) Galvanized-steel straps are deemed to comply with Sentence (3), provided they are:
 - (a) 50 mm wide,
 - (b) not less than 0.91 mm thick, and
 - (c) fastened at each end with at least four 63 mm nails.

Excerpt from: Table 9.23.3.4. Nailing for Framing

Forming Part of Sentence 9.23.3.4.(1)

| Construction Detail | Minimum Length of Nails, mm | Minimum Number or Maximum Spacing of Nails |
|---|-----------------------------|--|
| Roof rafter, roof truss or roof joist to plate – toe nail | 82 See Sentence (3) | 3 See Sentence (3) |
| Column 1 | 2 | 3 |

A-9.23.3.4.(3) Establishing uplift resistance.

The factored uplift resistance shall be established using general guidance laid out in Section 12.10 “Joist Hangers” of CSA O86 “Engineering Design in Wood”, as applicable to engineered roof-to-wall tie-down engineered connectors in lieu of specific procedures for those products.

RATIONALE FOR CHANGE

Problem/General Background

Wind loads impose both lateral and uplift forces upon roof structures. Concerns have been raised about the resilience of buildings to high winds during extreme weather events. The Building Code requires roof structures on buildings to be attached to the supporting walls in such a manner that they can resist these up-lift forces. In the case of buildings designed under Part 4 of the Building Code, an engineer would calculate the uplift force based on the design wind load and the shape and configuration of the roof. In the case of Part 9 buildings, the Building Code prescribes toe nailing connection. Article 9.23.3.4 of the Building Code currently requires roof structures to be attached to the supporting wall system using toe nailing as prescribed in Table 9.23.3.4.

Justification/Explanation

The model National Building Code (Sentence 9.23.3.4.(3)&(4)) has for some time required the use of a metal connector for roof-to-wall connections for areas with high wind speeds (over 127 kph). Areas subject to frequent high winds are found only in parts of the Maritime Provinces and parts of Alberta.

A similar approach, with a few modifications, has been proposed for Ontario. The intent of the proposal is to require the use of galvanized steel straps in order to provide a higher level of consistency and quality control than the currently prescribed toe nailing. Galvanized steel straps provide significantly greater resistance to uplift forces than toe nailing.

The proposed Code change includes a prescriptive option, as well as a performance based option.

- The prescriptive option would require that the roof structure be attached to the wall system with galvanized steel straps that are at least 50 mm wide and 0.91mm thick, and that can be attached with 4 nails of a specified length (63mm). Under 9.23.3.4 (2), Ontario's Building Code currently includes a similar prescriptive option allowing wall framing to be attached to the floor assembly using galvanized metal straps.
- The performance option would require that the galvanized steel strap would be capable of resisting a factored uplift load of at least 3kN (675 lbs).

Cost/Benefit Implications

For a typical house additional material costs are estimated at less than \$200. Builders estimate that the additional labour costs would be at least \$500 and potentially more, depending on the complexity and size of the roof.

Benefits include more consistent construction quality (compared to toe nailing) that provides greater assurance that Part 9 buildings can resist the one in thirty year design wind loads that Part 9 buildings are required to resist. A requirement for galvanized steel connectors can also promote quality control as their presence can be more easily ascertained. This promotes more streamlined and effective inspections by enforcement officials.

Enforcement Implications

None expected other than facilitating streamlined and effective inspections as noted above.

Who is Affected

Designers (including licensed architects and professional engineers), builders, contractors and installers, product manufacturers and enforcement officials are affected.

Table F.2: Objective Based Analysis

| Provision | Objective/Functional Statement |
|----------------------|--|
| Div. B, 9.23.3.4.(1) | F20-OP2.1, OP2.5] [F22-OP2.4, OP2.5] [F20, F22-OP2.3] Applies to elements that support or are part of an environmental separator. [F20, F22-OS1.2] Applies to assemblies required to provide fire resistance. [F20-OS2.1] [F20, F22-OS2.5] [F20, F22-OS2.3] Applies to elements that support or are part of an environmental separator. [F22-OS3.1] Applies to floors and elements that support floors. [F22-OS3.7] Applies to walls, and elements that support walls, that contain doors or windows required for emergency egress. [F20, F22-OH1.1, OH1.2, OH1.3] Applies to elements that support or are part of an environmental separator. [F22-OH4] Applies to floors and elements that support floors. |
| Div. B, 9.23.3.4.(3) | [F20-OS2.1] [F20, F22-OS2.3] [F20,F22-OS2.5] [F20-OP2.1,OP2.5] [F20,F22-OP2.3] [F22-OP2.4,OP2.5] [F20,F22-OH1.1,OH1.2,OH1.3] [F20,F22-OS1.2] Applies to assemblies required to provide fire resistance. |
| Div. B, 9.23.3.4.(4) | [F20-OS2.1] [F20, F22-OS2.3] [F20,F22-OS2.5] [F20-OP2.1,OP2.5] [F20,F22-OP2.3] [F22-OP2.4,OP2.5] [F20,F22-OH1.1,OH1.2,OH1.3] [F20,F22-OS1.2] Applies to assemblies required to provide fire resistance. |

OTHER SUPPORTING MATERIALS

References:

- Nateghi-A, F. (1996). "Assessment of wind speeds that damage buildings", *Natural Hazards*, July 1996, Volume 14, Issue 1, pp. 73-84. (See section 3.1 on p 78)
- Kopp, G., Oh, J., and Incullet, D. (2008). "Wind-Induced Internal Pressures in Houses.", *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2008)134:7(1129), 1129-1138.
- Morrison, M.J., Kopp, G.A., Gavanski, E., Miller, C., and Ashton, A. (2012). "Damage to Residential Construction from the Tornadoes in Vaughan, Ontario, on August 20, 2009", *Forensic Engineering 2012*: pp. 831-840.
- Morrison, M.J., Henderson, D.J., and Kopp, G.A. (2012). "The response of a wood-frame, gable roof to fluctuating wind loads", *Engineering Structures*, Vol. 41, Aug. 2012, 498–509.

Appendix G: 2013 NBCC Code Change Request – Roof-to-Wall Connections

Code Reference: NBCC 9.23.3.4

Subject: Nailing of framing

Requested Change/Addition:

Change: Table 9.23.3.4.

| Construction Detail | Minimum Length of Nails, mm | Minimum Number or Maximum Spacing of Nails |
|--|-----------------------------|--|
| Roof rafter, roof truss or roof joist to plate – toe nail ⁽²⁾ | 82 N/A | 3 N/A |

Change: 9.23.3.4

3) Where the 1-in-50 hourly wind pressure is equal or less than 0.8 kPa, Roof rafters, joists, or trusses shall be tied to the wall framing with connectors that will resist a factored uplift load of 3 kN.

Problem:

The capacity of toe-nails connections has been demonstrated to not meet the capacity necessary for design conditions for houses in Canada. This is a significant inconsistency in the code presenting a life safety issue.

Justification/Explanation:

By adequately connecting roof rafters, joists, and/or trusses to wall framing, the resistance of the connection to uplift forces is uplift forces during windstorms conditions is increased, decreasing the risk of structural damage.

The design uplift force on trusses can be calculated using the Static Procedure defined in Commentary I of the code as follows:

Assuming a house with the following characteristics;

- code limit for low/medium wind exposure 0.8 kPa
- internal pressure coefficient of 0.3,
- 10 m wide,
- two storey, 6 m reference height,
- gable roof,
- common 4/12 pitch,
- no overhang,
- 0.61m (2') truss spacing with load sharing between 3 trusses,
- roof weighs 0.48 kPa (10 psf), and
- open terrain wind exposure,

results in an uplift force of 5.2 kN per connection.

Testing has demonstrated that the capacity of the toe-nail connections is 1.14 kN (Morrison and Kopp (2011) using 5th percentile ultimate strength with 0.6 resistance factor applied as per CSA O86 Engineering Design in Wood). This is about half the required capacity for a basic house in the Canadian locations with the lowest wind speeds.

As a consequence, design wind events could cause damage to the roof structure, or even the separation of the roof structure from walls when using toe nails. Loss of roof structure is usually the precursor to wall collapses (Nateghi 1996), and wall collapses can cause of death or injury in wind storms.

Objective(s):

NBCC OP2.1 Loads bearing on the building elements that exceed their load-bearing capability.

Cost/Benefit Implications:

Cost Estimate: \$200

Enforcement Implications:

The requested addition/change does not add any additional burden to the inspection process and can be easily enforced.

Other Comments:

References:

Morrison, M., Kopp, G., "Performance of toe-nail connections under realistic wind loading", Engineering Structures, 33(1) pp 69-76, 2011

Nateghi-A, F., "Assessment of Wind Speeds that Damage Buildings", Natural Hazards, 14 pp 73-84, 1996. (see section 3.1 on pg 78)

Appendix H: 2013 NBCC Code Change Request – Fasteners for Roof Sheathing

Code Reference: NBCC 9.23.3.5

Subject: Roof Sheathing Nailing

Requested Change/Addition:

Table 9.23.3.5A

| 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 | |
| Plywood, OSB or waferboard up to 10 mm thick | <u>51, use 57 for roofs</u> | <u>45, use 51 for roofs</u> | n/a | <u>38, use 51 for roofs</u> | 150 mm (o.c.) along edges and 300 mm (o.c.) along intermediate supports, and for roof sheathing, 150mm (o.c.) along intermediate supports where supports are spaced at more than 406 mm o.c. |
| Plywood, OSB or waferboard over 10 mm and up to 20 mm thick | <u>51, use 63 for roofs</u> | <u>45, use 51 for roofs</u> | n/a | <u>51, use 63 for roofs</u> | |
| Plywood, OSB or waferboard over 20 mm and up to 25 mm thick | <u>57, use 63 for roofs</u> | <u>51, use 57 for roofs</u> | n/a | n/a | |

Table 9.23.3.5B

| Element | Minimum Length of Fasteners, mm | | | Minimum Number or Maximum Spacing of Fasteners |
|---|---------------------------------|-----------------------------|-----------------|--|
| | Common or Spiral Nails | Ring Thread Nails or Screws | 14-Gage Staples | |
| Plywood, OSB or waferboard over 10 mm and up to 20 mm thick | 63 | 51 | 63 | 150 mm (o.c.) along edges and 300 mm (o.c.) along intermediate supports, and for roof sheathing, 150mm (o.c.) along intermediate supports where supports are spaced at more than 406 mm o.c. and where HWP is equal to or greater than 0.8 kPa d less than 1.2kPa, 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 | |

Problem:

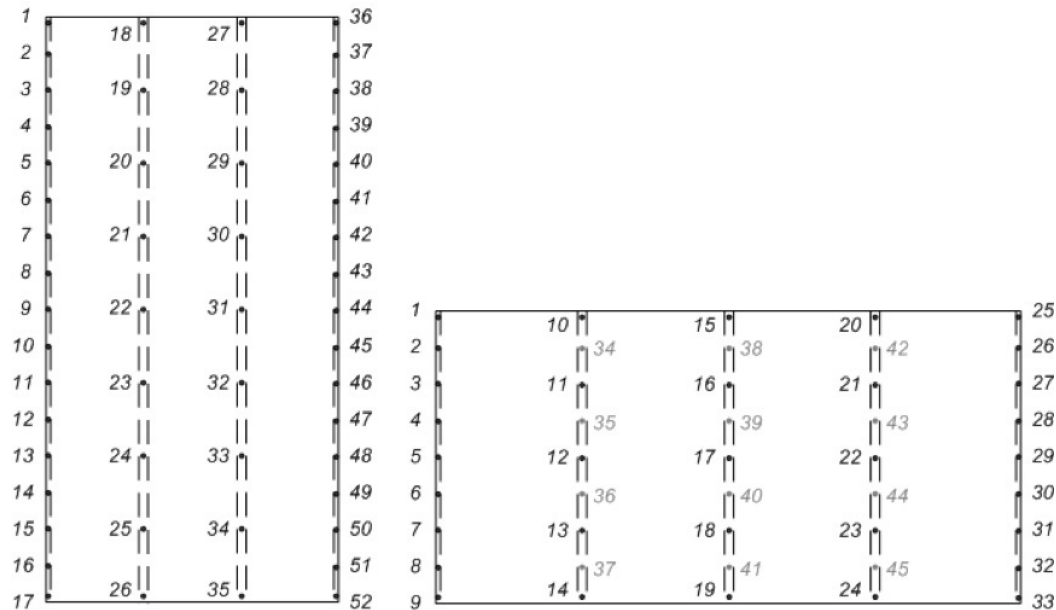
Roof sheathing sees significantly greater uplift pressures than wall sheathing, but due to typical horizontal installation (versus vertical installation for wall sheathing) it is installed with 19 fewer fasteners (i.e. 33 fasteners instead of 52). This inconsistency represents a life safety and property damage risk as uplift failure of roof sheathing is a common damage mechanism in wind events.

Justification/Explanation:

To ensure wall and roof sheathing requirements provide a uniform level of safety for Canadian homeowners, it is necessary to align wall and roofing sheathing attachment requirements. Without this change, code requirements for roof sheathing fasteners do not afford the same level of protection against wind damage as wall sheathing requirements. Roof sheathing is a common damage resulting from windstorms (Morrison et. al. 2009 see figure 7, 14, and 19).

Currently roof sheathing code requirements result in 33 nails, whereas walls require 52 (see illustration below). A requirement that fasteners are installed at 150 mm (o.c.), rather than 300 mm (o.c.) along intermediate edges would increase the number of nails to 45, which is 7 less than the current requirement for walls.

Typical wall (left) and roof (right) sheathing installation over studs/rafters and nail count (additional proposed nails shown in grey)



Since the roof sheathing will still have less nails than the wall sheathing, the size of the nails also need be increased.

This change is justified because suction forces on roof is much greater than the walls (see $C_p C_g$ in Figures I-7 thru 14 for low rise buildings in Appendix I of the code). As a consequence, uplift failure of roof sheathing is a common damage mechanism in wind events. Often roof sheathing becomes debris that can damage adjacent homes during such wind events.

In 2012, the OBC adopted changes similar to these. As a consequence, NBCC language under section 9.23.3.5 is inconsistent with the new Ontario language and does not afford a uniform level of safety for all Canadians.

OBC added to section 9.23.3.5: "5) Where roof sheathing supports are spaced at more than 406 mm o.c., the maximum spacing of fasteners for roof sheathing shall be 150 mm along edges and intermediate supports."

Objective(s):

The requested change is aimed at addressing OS2, OH1, and OP5.

Cost/Benefit Implications:

The cost of additional nails for each panel is very small (12 additional nails for each panel).

Enforcement Implications:

The requested addition/change does not add any additional burden to the inspection process, and can be easily enforced.

Photos of sheathing loss after wind event (GTA area houses).



Appendix I: 2013 NBCC Code Change Request – Anchorage for Columns and Posts

Code Reference: NBCC 9.23.6.2

Subject: Anchorage of Columns and Posts

Requested Change/Addition:

9.23.6.2 Anchorage of Columns and Posts

- 1) Except as provided in Sentences (2) and (3), exterior columns and posts shall be anchored to resist uplift and lateral movement **using hot-dipped galvanized or stainless steel post and base connection rated for at least 6.8 kN allowable uplift loads.**
- 2) Except as provided in Sentence (3), where columns or posts support balconies, decks, verandas or other exterior platforms, and the distance from finished ground to the underside of the joists is not more than 600 mm,
 - a) the columns or posts shall be anchored to the foundation to resist uplift and lateral movement, or
 - b) the supported joists or beams shall be directly anchored to the ground to resist uplift.
- 3) Anchorage is not required for platforms described in Sentence (2) that
 - a) are not more than 1 storey in height,
 - b) are not more than 55 m² in area,
 - c) do not support a roof, and
 - d) are not attached to another structure, unless it can be demonstrated that differential movement will not adversely affect the performance of the structure to which the platform is attached.

Problem:

Uplift forces applied to porch roofs and raised decks during design winds conditions can cause support posts to be lifted off of their supports causing structural damage to the building. The anchoring requirements are not currently provided in a prescriptive format leading to inadequately anchored installations in the field.

Justification/Explanation:

By adequately attaching porch roof support beams to their posts, and posts to their foundation, the resistance of the posts to uplift forces during windstorms is increased, decreasing the risk of structural damage.

The design uplift force on trusses can be calculated using the Static Procedure defined in Commentary I of the code as follows:

Assuming a house with the following characteristics;

- located in lowest 1-in-50 wind exposure locations of 0.3 kPa (Dryden ON),
- 2.44 m (8') wide porch,
- 2.44 m (8') between posts,
- porch weighs 0.48 kPa (10 psf), and
- open terrain wind exposure,

results in an external uplift force of 1.8 kN per column. Hence, relying on the weight of the porch roof is inadequate and a fastener is needed to anchor the bottom and top of the column to resist uplift forces for all locations in Canada. The design wind pressure for low and medium exposure is 0.8 kPa which would result in a 6.8 kN (1536 lb) uplift load. For the highest wind load of 1.05 (Cape Race, NFLD) 9.3 kN (2099 lb) per column would be required.

Currently porch columns are often toe nailed to foundations which provides insufficient uplift capacity.

Objective(s):

NBCC OP2.1 Loads bearing on the building elements that exceed their load-bearing capability.

Cost/Benefit Implications:

The cost estimate is between \$100-200. This cost is substantially outweighed by the benefit of improving life and safety protection through better anchorage of columns and posts.

Enforcement Implications:

Improved anchorage of columns and posts can be visually inspected.

Appendix J: Moore, Oklahoma – High Wind Resistance Residential Construction Requirements

City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.²⁰⁵

The following additions are hereby included in the dwelling code for the purposes of establishing minimum regulations governing residential construction for high wind resistance:

1. Roof sheathing (OSB or plywood) shall be nailed with 8d ring shank (0.131" × 2.5") or 10d (0.148" × 3") nails on 4" on center along the edges and 6" on center in the field. Dimensional lumber decking is not allowed.
2. Maximum spacing for roof framing shall be 16 inches on center. Minimum nominal sheathing panel size shall be 7/16. Minimum wood structural panel span rating shall be 24/16.
3. Connections for roof framing shall be designed for both compression and tension, and may include nail plates or steel connection plates. Connections for roof framing shall include connections on rafters, web members, purlins, kickers, bracing connections, and the connections to interior brace wall top plates or ceiling joists.
4. Gable end walls shall be tied to the structure, and may include steel connection plates or straps. The connections shall be made at the top and bottom of the gable end wall.
5. Structural sheathing panel (OSB or plywood) shall be required for gable end walls.
6. Hurricane [tie] or framing anchor shall be required on all rafter to wall connections.
7. The upper and lower story wall sheathing shall be nailed to the common rim board.
8. All walls shall be continuously sheathed with structural sheathing (OSB or plywood) using the CS-WSP method. Garage doors shall be framed using the sheathed portal frame method CS-PF. No form of intermittent bracing shall be allowed on an outer wall. Intermittent bracing may only be used for interior braced wall lines.
9. Nailing of wall sheathing (OSB or plywood) shall be increased to 8d ring shank (0.131" × 2.5") or 10d (0.148" × 3") nails on 4" on center along the edges and 6" on center in the field.
10. Structural wood sheathing shall be extended to lap the sill plate and nailed to the sill plate using a 4" on center along the edges. Structural wood sheathing shall be nailed to rim board if present with 8d ring shank (0.131 × 2.5") or 10d (0.148" × 3") nails on 4" on center along both the top and bottom edges of the rim board.
11. Garage doors shall be rated to 135 mph wind or above.
12. Exterior wall studs shall be 16" on center.

Appendix K: Oklahoma Amendments to IRC

Oklahoma Uniform Building Code Commission. 748 – Uniform Building Code Commission. Adopted Codes International Residential Code®, 2015 Edition (IRC®, 2015) 748:20-5-1 through 748:20-5-28. Appendix Y. Recommendations made in this section of the building requirements align with requirements currently in force in the City of Moore, Oklahoma (see Appendix H).

748:20-5-28. Appendix Y, Residential Tornado Provisions

- (a) This appendix has been newly created and entitled “Residential Tornado Provisions.” The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance or order.
- (b) Y101 Scope. This section heading has been added to specify the sections of this appendix that deal with the Scope of the appendix. This section header has been added to read: Y101. Scope.

(1) Section Y101.1 General.

This section has been added to clarify the provisions shall be applicable for new construction. This section has been added to read: Y101.1 General.

These provisions shall be applicable for new construction where residential tornado provisions are required. This appendix provides prescriptive based requirements for construction of a residential structure meeting or exceeding a 135 mph wind event corresponding to an EF-2 tornado rating. The single most important objective in protecting a structure against high wind is achieving a continuous load path from the roof to the foundation. Based on the findings of studies and failures associated with various construction types, a group of 11 building practices (each associated with a different aspect of the structure) are summarized in this section.

(2) Section Y101.2 Application.

This section has been added to clarify the administrative provisions of this appendix are applicable in the administrative and building planning and construction requirements in Chapters 1 through 10 of this code. The section has been added to read: Section Y101.2 Application. In addition to the general administration requirements of Chapter 1, the administrative provisions of this appendix shall also apply to the building planning and construction requirements of Chapters 1 through 10.

(3) Section Y101.3 Wind design criteria.

This section has been added to clarify that if Section R301.2.1 is modified, the buildings and portions thereof shall be constructed in accordance with the code and the ultimate wind speed design of 135 mph. This section has been added to read: Y101.3 Wind design criteria. Modifying section R301.2.1 buildings and portions thereof shall be constructed in accordance with the wind provisions of this code using the ultimate design wind speed 135 mph.

(4) Section Y101.4 Lumber sheathing.

This section has been added to address the permitted forms of lumber sheathing. This section has been added to read: Y101.4 Lumber sheathing.

Only OSB or plywood sheathing is permitted. Dimensional lumber sheathing may not be used. Allowable spans and attachment for lumber used as roof or exterior wall sheathing shall conform to the following:

- (A) Section Y101.4.1 Sixteen Inch Framing. For rafter, stud, or beam spacing of 16 inches, the minimum nominal sheathing panel thickness will be 7/16 inch, the minimum wood structural panel span rating 24/16, to be nailed with 8d ring shank (0.131 inch x 2.5 inch) or 10d (0.148 inch x 3 inch) nails on 4 inches on center along the edges and 6 inches on center in the field.
- (B) Y101.4.2 Section Twenty-four Inch Framing. For rafter, stud or beam spacing of 24 inches, the minimum nominal sheathing panel thickness will be 23/32 inch, the minimum wood structural panel span rating 24/16 to be nailed with 8d ring shank (0.131 inch x 2.5 inch) or 10d (0.148 inch x 3 inch) nails on 4 inches on center along the edges and 4 inches on center in the field.

(5) Section Y101.5 Ceiling joist and rafter connections.

This section has been added to require ceiling joists and rafters to be nailed to each other in a manner to achieve a connection that can transfer a 500 pound force in both compression and tension across the connections. This section has been added to read: Y101.5 Ceiling joist and rafter connections.

In addition to the provisions of Chapter 8, ceiling joists and rafters shall be nailed to each other in a manner to achieve a connection that can transfer a 500 pound force in both compression and tension across the connection.

(6) Section Y101.6 Rafter uplift resistance.

This section has been added to require individual rafters to be attached to supporting wall assemblies by connections capable of resisting uplift forces of 500 pounds. This section has been added to read: Y101.6 Rafter uplift resistance.

Individual rafters shall be attached to supporting wall assemblies by connections capable of resisting uplift forces of 500 pounds.

(7) Section Y101.7 Gable end walls.

This section has been added to clarify connections and sheathing for gable end walls. This section has been added to read: Y101.7 Gable end walls.

Gable end walls will be sheathed per Y101.4 and will have connections to both

- a.) supporting wall assemblies and
- b.) roof framing by connections capable of resisting uplift forces of 500 pounds in both compression and tension across the connection.

(8) Section Y101.8 Exterior wall bracing.

This section has been added to clarify sheathing methods to be utilized to brace exterior walls and prohibit intermittent bracing on exterior walls. This section has been added to read: Y101.8 Exterior wall bracing. Only continuous sheathing methods per R602.10.4.2 may be used to brace exterior walls. Frame garage doors using the sheathed portal frame method CS-PF. Lumber sheathing and attachment per Y101.4. Any form of intermittent bracing is not allowed on an exterior wall. Intermittent bracing may only be used for interior braced wall lines.

(9) Section Y101.9 Multi story construction.

This section has been added to require nailing upper and lower story wall sheathing to a common rim board. This section has been added to read: Y101.9 Multi story construction. Nail upper and lower story wall sheathing to common rim board in order to maintain continuity between stories.

(10) Section Y101.10 Wood floor above crawl space construction.

This section has been added to require extending structural wood sheathing to lap the sill plate. This section has been added to read: Y101.10 Wood floor above crawl space construction. Extend structural wood sheathing to lap the sill plate. Nail to sill plate at 4 inches on center along the edges. Nail to rim board if present with 8d ring shank (0.131 inch x 2.5 inch) or 10d (0.148 inch x 3 inch) nails at 4 inches on center along both the top and bottom edges of the rim board.

Appendix L: Detail and Diagrams from CWC 2014

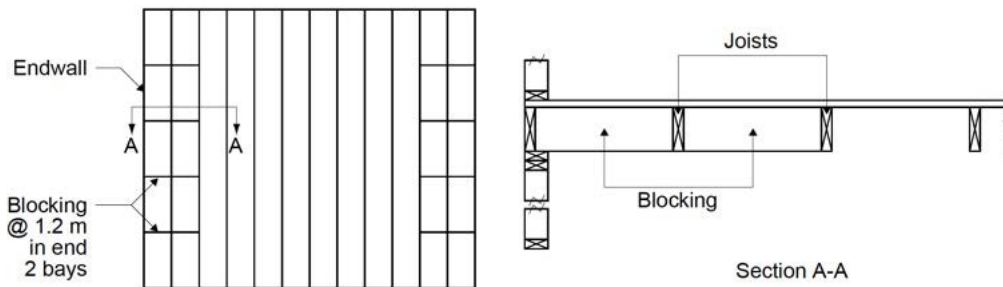
Source: Canadian Wood Council. 2014. Engineering Guide for Wood Frame Construction. Ottawa: Canadian Wood Council. Used with permission.

CWC 1: Bracing for Walls. Floor Bracing Endwall, Ceiling Bracing Gable Endwall. Related to Measure A.1.1 in Appendix A of this document.

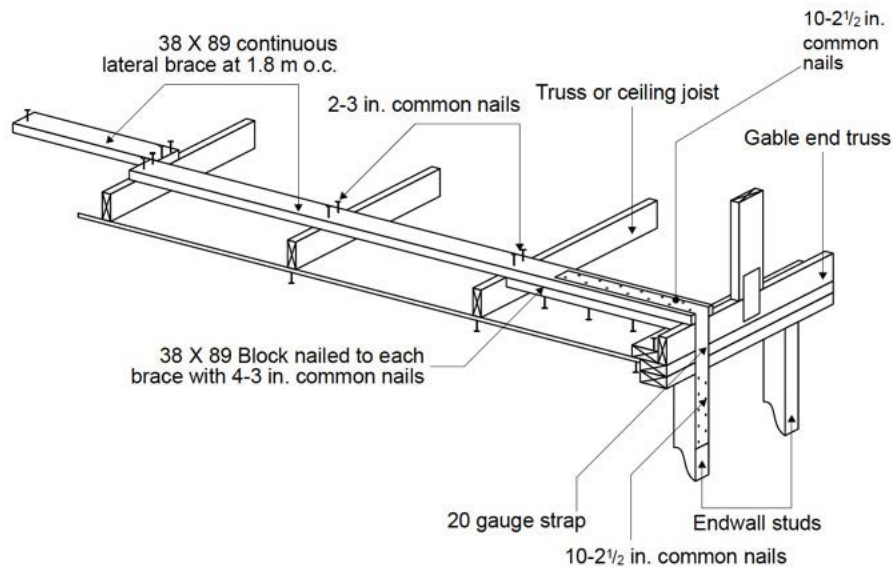
Wall 27

Bracing for walls

Floor bracing endwall



Ceiling bracing gable endwall



Notes:

1. Ceiling to be sheathed with structural wood panels and detailed as a diaphragm.
2. Table Load 10 may be used, conservatively, to determine the loads on the ceiling diaphragm.
3. Diaphragm to be designed using Tables Diaphragm 1-7
4. When studs are continuous from the floor diaphragm to the roof diaphragm and connected in accordance with Table Wall 11, the ceiling does not need to be designed to brace the endwall.

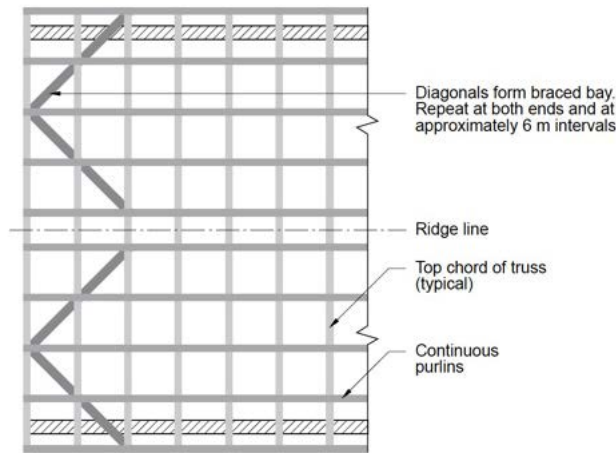
CWC 2: Roof 11, Permanent Bracing for Trusses. Related to Measure A.1.1 in Appendix A of this document. Note that BCSI guidance related to truss bracing provision is provided in Appendix B of this document.

Roof 11

Permanent bracing for trusses

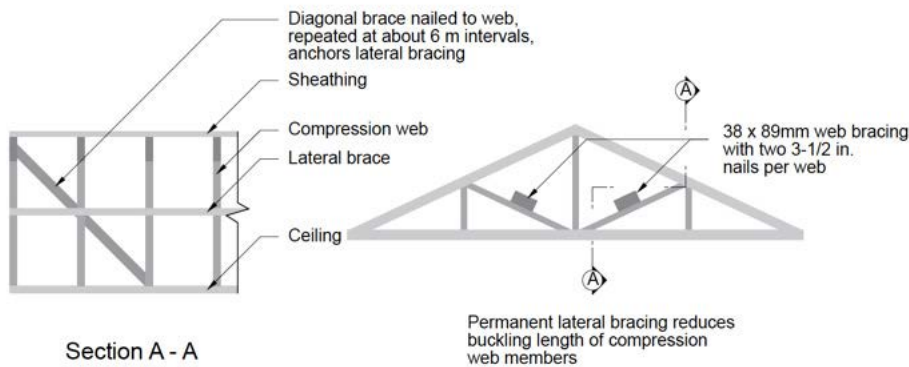
Permanent bracing of top chord plane (large buildings)

If plywood floor or roof sheathing is properly applied with staggered joints and adequate nailing, a continuous diaphragm action is developed to resist lateral movement at the top chord, and additional bracing in the plane is generally not required. Some metal roofing materials may act as a diaphragm when properly lapped and nailed but selection and use of these materials is at the discretion of the building designer. If purlins are used, spaced not to exceed the buckling length at the top chord, diagonal bracing should be applied to the underside of the top chord to prevent lateral shifting of the purlins. The diagonal bracing should be installed on both sides of the ridge line in all end bays. If the building exceeds 18 m in length, this bracing should be repeated at intervals not exceeding 6 m.



Permanent lateral bracing to web member or bottom chord (all buildings)

Permanent bracing in web and bottom chord planes is usually applied as temporary bracing. Lateral bracing of compression web members is a typical method to prevent buckling. Bottom chord bracing installed to maintain truss spacing, also can resist buckling caused by stress reversal. Multiple-bearing or cantilevered trusses can result in compressive forces in bottom chords.

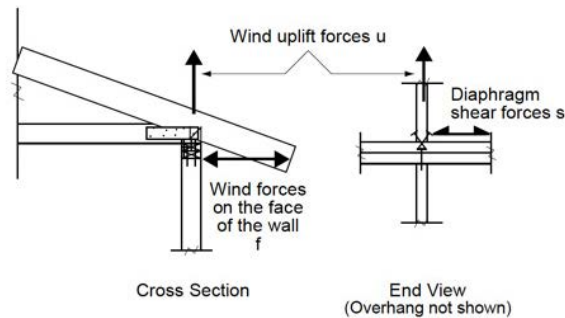


Note:

1. Where lateral bracing is shown on the truss design drawings, it must be installed so that the truss will support design loads.

CWC 3: Roof 9, Roof Framing Connection at Exterior Wall. Related to Measure A.1.2 in Appendix A of this document.

Roof framing connection at exterior wall



Design toe-nail connections for the most critical of uplift load “u”, face load “f” or shear force “s”. The three loads may be considered independently. In lieu of toe-nails, proprietary framing anchors may be used to transfer the loads.

Factored Uplift Load u (kN)

| Factored roof uplift loads ¹ kN/m | Roof framing spacing (mm) | | |
|---|---------------------------|-----|-----|
| | 300 | 400 | 600 |
| 3 | 0.90 | 1.2 | 1.8 |
| 4 | 1.2 | 1.6 | 2.4 |
| 5 | 1.5 | 2.0 | 3.0 |
| 6 | 1.8 | 2.4 | 3.6 |
| 7 | 2.1 | 2.8 | 4.2 |
| 8 | 2.4 | 3.2 | 4.8 |
| 9 | 2.7 | 3.6 | 5.4 |
| 10 | 3.0 | 4.0 | 6.0 |

Notes:

1. See Table Load 15 for wind uplift loads.
2. Factored uplift load is calculated as factored roof uplift load x framing spacing/1000.

Factored Uplift Resistance (kN)

| Common nail length in. | Framing toe-nailed to wall plate. Capacity of 3 toe-nails ¹ | Framing toe-nailed to wall plate. Capacity of 4 toe-nails ¹ |
|---------------------------|---|---|
| | 3.25 ³ | 0.70 |
| 3.5 ⁴ | 0.83 | 1.1 |
| 4 ⁵ | 1.1 | 1.5 |

Notes:

1. Table applies to S-P-F lumber. For D. Fir-L and Hem-Fir multiply the resistance by 1.2. For Northern Species multiply the resistance by 0.7.
2. In lieu of toe-nails, special framing anchors may be used.
3. Minimum Top Plate Width: 140 mm for D. Fir-L and Hem-Fir, 89 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
4. Minimum Top Plate Width: 140 mm for all Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
5. Minimum Top Plate Width: 184 mm for D. Fir-L and Hem-Fir, 140 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
6. Toe-nail resistance calculated as per 9.5.3 with “ θ ” 30 degrees.

Roof framing connection at exterior wall

Factored Lateral Load f (kN)

| Factored wind lateral loads ¹ kN/m | Roof framing spacing (mm) | | |
|--|---------------------------|------|------|
| | 300 | 400 | 600 |
| 0.5 | 0.15 | 0.20 | 0.30 |
| 1 | 0.30 | 0.40 | 0.60 |
| 1.5 | 0.45 | 0.60 | 0.90 |
| 2 | 0.60 | 0.80 | 1.2 |
| 2.5 | 0.75 | 1.0 | 1.5 |
| 3 | 0.90 | 1.2 | 1.8 |
| 4 | 1.2 | 1.6 | 2.4 |
| 5 | 1.5 | 2.0 | 3.0 |
| 6 | 1.8 | 2.4 | 3.6 |

Notes:

- See Table Load 17 for factored wind lateral loads on the face of the wall. Use half of the wall height as the tributary width in Table Load 17.
- Factored lateral load is calculated as factored wind lateral load x framing spacing/1000.

Factored Shear Force s (kN)

| Factored diaphragm shear force ¹ kN/m | Roof framing spacing (mm) | | |
|---|---------------------------|-----|-----|
| | 300 | 400 | 600 |
| 2 | 0.60 | 0.8 | 1.2 |
| 4 | 1.2 | 1.6 | 2.4 |
| 6 | 1.8 | 2.4 | 3.6 |
| 8 | 2.4 | 3.2 | 4.8 |
| 10 | 3.0 | 4.0 | 6.0 |
| 12 | 3.6 | 4.8 | 7.2 |
| 14 | 4.2 | 5.6 | 8.4 |
| 16 | 4.8 | 6.4 | 9.6 |
| 18 | 5.4 | 7.2 | 11 |
| 20 | 6 | 8 | 12 |

Notes:

- See Table Diaphragm 1 for factored diaphragm shear force.
- Factored shear force is calculated as factored diaphragm shear force x framing spacing/1000.

Factored Lateral Resistance (kN)

| Common nail length in. | Framing toe-nailed to wall plate. Capacity of 3 toe-nails ¹ | Framing toe-nailed to wall plate. Capacity of 4 toe-nails ¹ |
|---------------------------|--|--|
| | 3.25 ³ | 2.2 |
| 3.5 ⁴ | 2.7 | 3.5 |
| 4 ⁵ | 3.7 | 4.9 |

Notes:

- Table applies to S-P-F lumber. For D. Fir-L and Hem-Fir multiply the resistance by 1.1. For Northern Species multiply the resistance by 0.9.
- In lieu of toe-nails, special framing anchors may be used.
- Minimum Top Plate Width: 140 mm for D. Fir-L and Hem-Fir, 89 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Minimum Top Plate Width: 140 mm for all Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Minimum Top Plate Width: 184 mm for D. Fir-L and Hem-Fir, 140 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Lateral resistance calculated as per clause 12.9.4 of the CSA O86.

Factored Shear Resistance (kN)

| Common nail length in. | Framing toe-nailed to wall plate. Capacity of 3 toe-nails ¹ | Framing toe-nailed to wall plate. Capacity of 4 toe-nails ¹ |
|---------------------------|--|--|
| | 3.25 ³ | 0.97 |
| 3.5 ⁴ | 1.1 | 1.5 |
| 4 ⁵ | 1.5 | 2.1 |

Notes:

- Table applies to S-P-F lumber. For D. Fir-L and Hem-Fir multiply the resistance by 1.2. For Northern Species multiply the resistance by 0.7.
- In lieu of toe-nails, special framing anchors may be used.
- Minimum Top Plate Width: 140 mm for D. Fir-L and Hem-Fir, 89 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Minimum Top Plate Width: 140 mm for all Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Minimum Top Plate Width: 184 mm for D. Fir-L and Hem-Fir, 140 mm for S-P-F and Northern Species to conform to minimum nail spacing requirements of CSA O86 Table 12.9.2.1.
- Toe-nail resistance calculated as per 9.5.3 with "Θ" 60 degrees.

CWC 4: Roof 1, Roof Sheathing Attachment. Related to Measure A.1.3 in Appendix A of this document.

Roof sheathing attachment

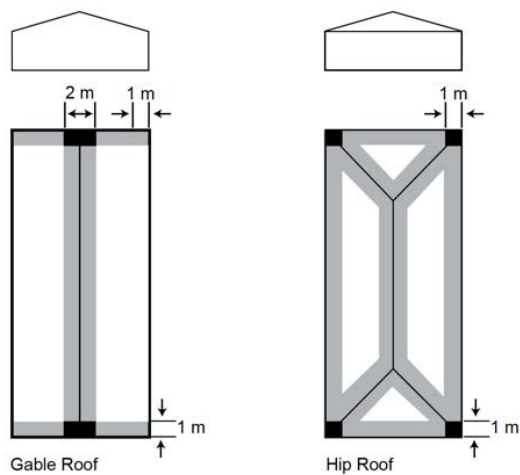
Sheathing Attached with 2 in. Common Nails or Larger

Nail Spacings, mm¹

| Sheathing location on roof | Roof framing spacing mm | Building in a suburban setting ^{2,3} q _{1/50} wind loads (kPa) | | | Building in an open setting ⁴ q _{1/50} wind loads (kPa) | | |
|--------------------------------------|-------------------------|---|------|------|--|------|------|
| | | 0.65 | 0.75 | 0.85 | 0.65 | 0.75 | 0.85 |
| 1 m corner zones ⁵ | 300 | 150 | 100 | 100 | 100 | 75 | 75 |
| | 400 | 100 | 100 | 75 | 75 | 50 | 50 |
| | 600 | 75 | 50 | 50 | 50 | 35 | 35 |
| 1 m perimeter edge zone ⁵ | 300 | 150 | 150 | 150 | 150 | 100 | 100 |
| | 400 | 150 | 100 | 100 | 100 | 75 | 75 |
| | 600 | 100 | 75 | 75 | 75 | 50 | 50 |
| Interior zone | 300 | 300 | 150 | 150 | 150 | 150 | 150 |
| | 400 | 150 | 150 | 150 | 150 | 150 | 100 |
| | 600 | 150 | 100 | 100 | 100 | 100 | 75 |

Notes:

- Nail spacings in the table are for nails along intermediate supports. Maximum nail spacing at the panel edge is 150 mm.
- Nail spacings also apply to buildings in a wooded setting or in the centre of a large town.
- C_e value of 0.7 for suburban settings.
- C_e value of 1.0 for open settings.
- See the diagram for the description of corner zones and edge zones. If the minimum horizontal dimension of the roof is greater than 10 m, use the edge zone values for the lesser of 10% of the least horizontal dimension or 40% of the eave height.
- Tabulated values are valid for the following conditions:
 - 1, 2 or 3 storey buildings.
 - External wind pressure coefficient C_pC_g of -5.0 for corner zones, -3.8 for edge zone and -2.4 for interior zone.
 - Roof rafters or roof trusses constructed with D. fir-L, Hem-Fir or S-P-F lumber.



■ Corner zone ■ Edge zone

CWC 5: Wall 1, Wall Sheathing Attachment. Related to Measure A.2.4 in Appendix A of this document.

Wall sheathing attachment

Sheathing Attached with 2 in. Common Nails or Larger

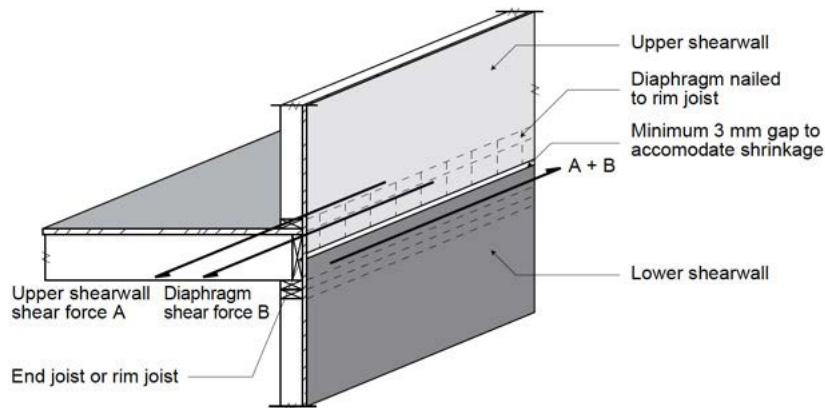
| Nail Spacings, mm ¹ | | Building in a suburban setting ^{2,3} | | | Building in an open setting ⁴ | | |
|--------------------------------------|----------------------------|---|------|------|--|------|------|
| Sheathing location on wall | Wall framing spacing mm | q _{1/50} wind loads (kPa) | | | q _{1/50} wind loads (kPa) | | |
| | | 0.65 | 0.75 | 0.85 | 0.65 | 0.75 | 0.85 |
| 1 m perimeter edge zone ⁵ | 300 | 300 | 300 | 150 | 150 | 150 | 150 |
| | 400 | 150 | 150 | 150 | 150 | 150 | 100 |
| | 600 | 150 | 150 | 100 | 100 | 100 | 75 |
| Interior zone | 300 | 300 | 300 | 300 | 300 | 150 | 150 |
| | 400 | 300 | 150 | 150 | 150 | 150 | 150 |
| | 600 | 150 | 150 | 150 | 150 | 100 | 100 |

Notes:

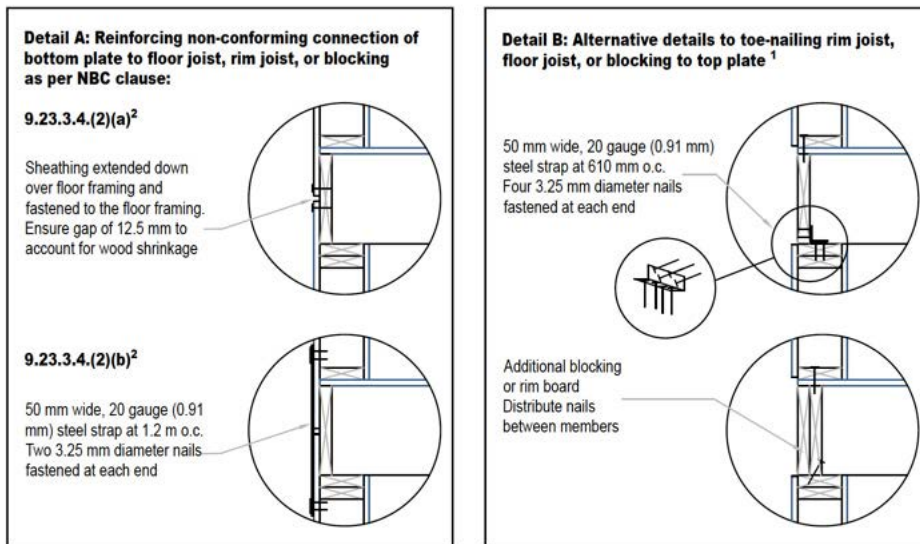
- Nail spacings in the table are for nails along intermediate supports. Maximum nail spacing at the panel edge is 150 mm.
- Adjustment factors also apply to buildings in a wooded setting or in the centre of a large town.
- C_e value of 0.7 for suburban settings.
- C_e value of 1.0 for open settings.
- For wall sheathing within 1 m of the perimeter of the building edge, use the end zone requirements. If the minimum horizontal dimension of the building is greater than 10 m, use the edge zone values for the lesser of 10% of the least horizontal dimension or 40% of the eave height.
- Tabulated values are valid for the following conditions:
 - 1, 2 or 3 storey buildings.
 - External wind pressure coefficient C_pC_g of -2.1 for edge zone and -1.75 for interior zone.
 - D. Fir-L, Hem-Fir and S-P-F wall framing lumber.

CWC 6: Detail related to connection of exterior sheathing to common rim joist. Related to Measure A.2.3 in Appendix A of this document.

Figure 10.4.2 A: Shear transfer where sheathing extends across floor framing



CWC 7: Detail Figure C5a. Related to Measure A.2.3 in Appendix A of this document.



Notes:

1. Other connectors may be used provided the capacity of the connector is capable of transferring equivalent lateral, uplift, and shear forces of 82 mm toe-nails at 150 mm on centre.
2. Reinforcing non-conforming connection of bottom plate to floor joist, rim joist, or blocking connection applies to walls that meet the requirement of bottom plate nailing in the NBC Table 9.23.3.4 (400 mm o.c) but does not conform to the braced wall requirements of 150 mm o.c. Alternative details following NBC 9.23.3.4.(2)(a) and 9.23.3.4.(2)(b) are required to ensure transfer of lateral loads between storeys.

CWC 8: Wall 11, Stud to wall plate connections at exterior walls. Lapping the connecting floor framing member (rim joists or blocking) by not less than 50 mm. Nails driven in to the rim joist should be staggered. Related to Measure A.2.3 in Appendix A of this document.

Factored uplift resistance (kN/m)

Where wall sheathing used to resist uplift¹

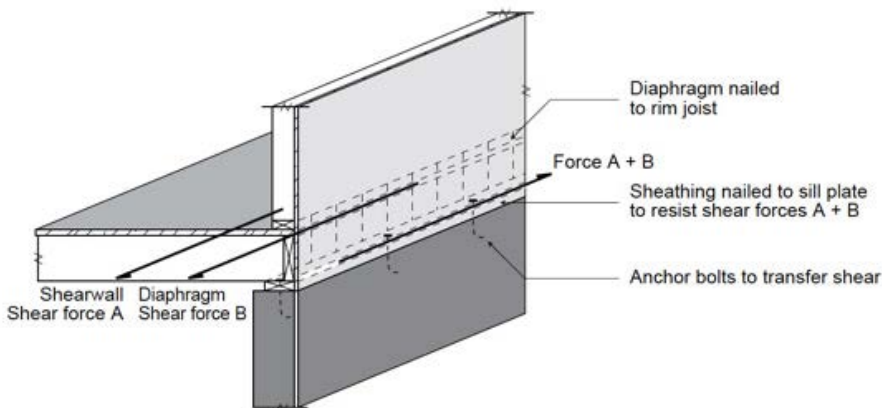
| Common nail length in. | Spacing of nails in the end or rim joist ³ | |
|---------------------------|---|--------|
| | 150 mm | 100 mm |
| 2 | 3.2 | 4.7 |
| 2.5 | 4.0 | 6.0 |
| 3 | 4.7 | 7.0 |

Notes:

1. Upper storey sheathing and lower storey sheathing shall each overlap the connecting floor framing member (rim joists or blocking) by not less than 50 mm. Nails driven into the rim joist should be staggered.
2. A gap of 3 mm minimum shall be left between the sheathing panels to accommodate shrinkage of the floor framing
3. The spacing of nailing shall not be less than required for shearwall capacity. See Tables Shearwall 10 and Shearwall 14 Case A for shearwall adjustment factors for shearwalls subject to combined shear and uplift.
4. Resistance values are for 38 mm or thicker rim joists. For proprietary rim joists see manufacturer's literature.
5. Table applies to S-P-F and Hem-Fir lumber. For D. Fir-L multiply the resistance by 1.05. For Northern Species multiply the resistance by 0.95.
6. Lateral resistance calculated as per 12.9.4 of the CSA O86.

CWC 9: Detail that provides an example of fastening wall sheathing to sill plate. Related to Measure A.2.3 in Appendix A of this document.

Figure 10.4.3 A: Shear transfer where sheathing extends to sill plate



End Notes & References

- 1 Out of a total of 204 catastrophe events recorded by the Insurance Bureau of Canada between 1983 and 2016, 127 were at least partially caused by storm, hurricane, wind, windstorm, or tornado. Source: IBC. 2017. Facts of the Property and Casualty Insurance Industry. Toronto, ON: IBC.
- 2 Referencing wind risk reduction provisions in the National Building Code of Canada (NBCC), CWC 2014 states:

Neither the National Building Code nor this guide is intended to provide design solutions against the direct force of tornadoes. Based on observations made in Canadian tornadoes, the Structural Commentaries of the National Building Code state, 'it is generally not economical to design buildings for tornadoes beyond what is currently required by NBCC Subsection 4.1.7 because of the low risk of loss to individual owners. It is, however, important to provide key construction details for the safety of building occupants...anchorage of home floors, is essentially covered by NBCC Article 9.23.6.1 for normal housing with permanent foundations. (Canadian Commission on Building and Fire Codes, 2010).' Source: Canadian Wood Council. 2014. Engineering Guide for Wood Frame Construction, 2014 Edition. Ottawa, ON: Canadian Wood Council. See pages A-6 to A-7.
- 3 City of Moore, OK. 2014. High Wind Resistance Residential Construction Requirements. Moore, OK: City of Moore.
- 4 See Dufferin County. 2017. Hurricane Clip Rebate Program. Dufferin County, ON: Dufferin County Building Inspection Department and the Institute for Catastrophic Loss Reduction. Accessed August 2018 from <https://www.dufferincounty.ca/files/content-pdf/hurricane-clip-rebate-program-brochure.pdf>
- 5 It should be noted that Victoriaville's program has been adopted by eight additional municipalities in the Province of Quebec. Ville de Victoriaville. 2013. Habitation Durable. Victoriaville, QC: Ville de Victoriaville. Accessed November 2018 from <http://www.habitationdurable.com/victoriaville/nouvelle-construction/construire-durable>
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- 8 Ministry of Municipal Affairs. 2012. Ontario Building Code. Toronto, ON: Ministry of Municipal Affairs. See Sentence 9.23.3.5.(5).
- 9 Ministry of Municipal Affairs and Housing. 2010. Proposed Change to the 2006 Ontario Building Code B-09-23-06. Toronto, ON: Ministry of Municipal Affairs and Housing. See Appendix E.
- 10 ICLR/Durham Region. Durham Region Climate Resilience Standard for New Houses. Whitby, ON: Durham Region.
- 11 APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA – The Engineered Wood Association.

- 12 Sills, D., Cheng, V., McCarthy, P., Rousseau, B., Waller, J., Elliott, L., Klassen, J., and Auld, H. 2012. Using tornado, lightning and population data to identify tornado prone areas in Canada. 26th Conference on Severe Local Storms, Nashville, TN, American Meteorological Society, Pg. 59.
- For more information on the EF scale and the compatibility of the F and EF scales, see: Wind Science and Engineering Center. 2006. A Recommendation for an Enhanced Fujita Scale (EF-scale). Lubbock, TX: Wind Science and Engineering Centre, Texas Tech University.
- 13 Cheng, V.Y., Arhonditsis, G. B., Sills, D. M., Auld, H., Shephard, M. W., Gough, W. A., and Klaassen, J. 2013. Probability of tornado occurrence across Canada. *Journal of Climate*, 26(23), 9415-9428. See page 9415.
- Sills, D., Cheng, V., McCarthy, P., Rousseau, B., Waller, J., Elliott, L., Klassen, J., and Auld, H. 2012. Using tornado, lightning and population data to identify tornado prone areas in Canada. 26th Conference on Severe Local Storms, Nashville, TN., American Meteorological Society, Pg. 59.
- 14 Newark, M.J. 1991. A design basis tornado. *Canadian Journal of Civil Engineering*, 18(3), 521–524. doi:10.1139/191-063.
- 15 Newark, M.J. 1991. A design basis tornado. *Canadian Journal of Civil Engineering*, 18(3), 521–524. doi:10.1139/191-063.
- Ramseyer, C., Holliday, L., and Floyd, R. 2016. Enhanced residential building code for tornado safety. *Journal of Performance of Constructed Facilities*, 30(4), 04015084.
- 16 Oklahoma Uniform Building Code Commission. 2018. Oklahoma Uniform Building Code Commission Rules, Title 748 - Uniform Building Code Commission, Chapter 20 – Adopted Codes, Subchapter 20 – International Residential Code®, 2015 Edition (IRC®, 2015), 748:20-5-1 through 748:20-5-28. See Appendix Y, Residential Tornado Provisions. Recommendations made in this section of the building requirements align closely with requirements currently in force in the City of Moore, Oklahoma.
- 17 Ninety-three percent of tornadoes recorded in the Canadian National Tornado Database – Verified Tracks were classified F2 or less. Source: Environment and Climate Change Canada. 2017. Canadian National Tornado Database: Verified Tracks (1980-2009) – Public GIS EN. Accessed October 2018 from <https://open.canada.ca/data/en/dataset/65658050-7a80-4da3-9a09-da137c203a34>
- See Box 1 for a description of F and EF tornado scales.
- Similarly, it has been reported that in the US, 95% of tornado events are EF2 or less. Source: APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA Wood – The Engineered Wood Association.
- 18 APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA – The Engineered Wood Association.
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- For wind force levels, see 2015 National Building Code of Canada, Section 9.23.13 Bracing to Resist Lateral Loads Due to Wind and Earthquake.
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50 See 2015 NBCC 9.23.6. Anchorage

51 Sandink, D., & Kopp, G. 2018. Part B: Tornado/Extreme Wind Protection. In Durham Region Climate Resilience Standard, Draft for Consultation. Durham Region and Institute for Catastrophic Loss Reduction.

52 See for example NBCC Sentence 9.4.1.1.(1) (part of Subsection 9.4.1. Structural Design Requirements and Application Limitations), and specifically Sentences 9.23.13.1.(2), 9.23.13.2.(2), and 9.23.13.3.(2), which include provisions for Part 9 buildings that are exposed to low-moderate (1-in-50 HWP less than 0.8 kPa) wind forces, high (where 1-in-50 HWP is less than 1.20 kPa) wind forces, and extreme (where 1-in-50 HWP is equal to or greater than 1.20 kPa).

For example: 9.23.13.1. Requirements for Low to Moderate Wind and Seismic Forces (where 1-in-50 HWP is less than 0.8 kPa), which states that "Bracing to resist lateral loads shall be designed and constructed...in accordance with (a) Articles 9.23.13.4 to 9.23.13.7., (b) Part 4, or (c) good engineering practice such as that provided in CWC 2014 "Engineering Guide for Wood Frame Construction."

NBCC references CWC 2014 in the appendices:

A-9.4.1.1

A-9.23.13.1

53 NBCC 2015.

54 Kopp, G.A., Galsworthy, J.K., and Oh, J.H. 2010. Horizontal wind loads on open-frame, low-rise buildings. *Journal of Structural Engineering*, 136(1), 98-105.

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56 See also Ramseyer, C., Holliday, L., and Floyd, R. 2016. Enhanced residential building code for tornado safety. *Journal of Performance of Constructed Facilities*, 30(4), 04015084.

57 OSB sheathing cost/installation: 7/16" = \$1.48/sq. ft. (pneumatic nailed), \$1.66/sq. ft. (hand-nailed)

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58 From RSMeans, RSMO Residential Cost Data, Quarter 3 2018. Copyright RSMeans, Rockland, MA 781-422-5000; All rights reserved:

Plywood:

3/8" = \$1.72/sq. ft. (pneumatic nailed), \$1.90/sq. ft. (hand-nailed)

1/2" = \$1.81/sq. ft. (pneumatic nailed), \$1.99/sq. ft. (hand-nailed)

Total cost increase = \$0.09/sq. ft. (pneumatic and hand nailed) when increasing from 3/8" to 1/2"

OSB:

7/16" = \$1.48/sq. ft. (pneumatic nailed), \$1.66/sq. ft. (hand-nailed)

1/2" = \$1.56/sq. ft. (pneumatic nailed), \$1.66/sq. ft. (hand-nailed)

Total cost increase = \$0.08 and \$0.00/sq. ft. (pneumatic and hand nailed, respectively) when increasing from 3/8" to 1/2"

Note also material/labour costs associated with OSB vs. plywood sheathing provided below (comparison provided for 1/2" sheathing):

Roof: 1/2" OSB = \$1.39/sq. ft. vs. 1/2" plywood = \$1.65/sq. ft.

Walls: 1/2" OSB = \$1.57/sq. ft. vs. 1/2" plywood = \$1.78/sq. ft.

- 59 Costs of cap and base connectors would vary based on construction specifics (e.g., size of supporting post, depth of slab). For the purposes of illustration, specific products are referenced here (based on manufacturer MSRPs):

A post base connector for 4" x 4" post that can be embedded in concrete (6" concrete slab), which is capable of resisting 9.4 kN uplift, and is visible after installation, retails at ~\$78.00. This compares with a more basic connector, capable of receiving a 4" post and accommodating 4.5 kN uplift, which may retail for ~\$23.00. Using this example, for a home with two porch columns, the incremental cost would be \$110.00.

A post base connector for 6" x 6" post that can be embedded in concrete (6" concrete slab), which is capable of resisting 9.4 kN uplift, and is visible after installation, retails at ~\$208.00. This compares with a more basic connector, capable of receiving a 6" post and accommodating 5.3 kN uplift, which may retail for ~\$58.00. Using this example, for a home with two porch columns, the incremental cost would be \$300.00.

- 60 It was further reported that 16" o.c. roof framing increased price of construction as well (~\$900), however this provision is not included in the set of provisions presented here. Source: Ramseyer, C., Holliday, L., and Floyd, R. 2016. Enhanced residential building code for tornado safety. *Journal of Performance of Constructed Facilities*, 30(4), 04015084.
- 61 From RSMMeans, RSMO Residential Cost Data, Quarter 3 2018. Copyright RSMMeans, Rockland, MA 781-422-5000; All rights reserved.
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- Multihazard Mitigation Council. 2017. Natural Hazard Mitigation Saves 2017 Interim Report: An Independent Study. Principal Investigator Porter, K.; co-Principal Investigators Scawthorn, C., Dash, N., Santos, J. and Schneider, P., Director, Multihazard Mitigation Council. Washington, DC: National Institute of Building Sciences.
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- 66 Applied Research Associates. 2002. Analysis of costs and loss reduction benefits of windborne debris protection—South Carolina coast exposure C residential buildings, ARA Report #1082. Raleigh, NC: Applied Research Associates. Cited in Sutter, D., DeSilva, D., and Kruse, J. 2009. An economic analysis of wind resistant construction. *Journal of Wind Engineering and Industrial Aerodynamics*, 97, 113-119.
- 67 Sutter, D., DeSilva, D., and Kruse, J. 2009. An economic analysis of wind resistant construction. *Journal of Wind Engineering and Industrial Aerodynamics*, 97, 113-119.
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- Multihazard Mitigation Council. 2017. Natural Hazard Mitigation Saves 2017 Interim Report: An Independent Study. Principal Investigator Porter, K.; co-Principal Investigators Scawthorn, C., Dash, N., Santos, J., and Schneider, P., Director, Multihazard Mitigation Council. Washington, DC: National Institute of Building Sciences.

69 Nine contractors in Ontario were asked what the change in cost is for the installation of architectural laminate shingles compared to traditional 3-tab shingles, and estimates were provided by six contractors (estimates per square assume 3 bundles/square).

| Contractor | Cost increase | | Notes |
|------------|--|-------------------------------------|--|
| | Materials | Installation | |
| 1 | \$4 increase per bundle | \$3.50-7.00 per bundle | Cost depends on roof pitch, ease of access, roof complexity, etc. |
| | Increase ranges from \$7.50-11.00 per bundle Total = \$22.50-\$33.00 increase per square | | |
| 2 | Similar costs, but ineffective warranties for 3-tab shingles 3-tab must be properly aligned, time consuming (can result in increases in installation – situation dependent) | | 3-tab rarely installed anymore, customers want better warranties. ~1% of jobs use 3-tab – e.g. patch work, customer bought shingles independently, insurance jobs |
| 3 | Minimal, range of \$0.65-\$1.50 increase per bundle | Minimal | Cost depends on roof pitch, ease of access, roof complexity, etc. |
| | Range of \$0.65-\$1.50 increase per bundle plus minimal installation cost Total = \$1.95-\$4.50 plus minimal installation cost increase per square | | |
| 4 | Minimal, additional \$1.00 per bundle | Minimal | 3-tab rarely installed anymore, customers want better warranties |
| | Increase of \$1.00 per bundle plus minimal installation cost Total = \$3.00 plus minimal installation cost increase per square | | |
| 5 | No major difference, total increase of ~\$2.00 per bundle Total = ~\$6.00 increase per square | | 3-tab rarely installed anymore, customers want better warranties. Industry standard basically architectural |
| 6 | Additional \$3.00-\$4.00 per bundle | Additional \$1.00-\$2.00 per bundle | 3-tab rarely installed anymore, customers want better warranties. No comparison between the two types, architectural much better |
| | Range of \$4.00-\$6.00 per bundle Total = \$12.00-\$18.00 increase per square | | |
| 7 | – | | Architectural easier to install, less experience required |
| 8 | – | | 3-tab rarely installed anymore, customers want better warranties |

Estimates of cost increase involved in installing architectural shingles compared to traditional 3-tab shingles produced a range of \$1.95-\$33.00 per square (assuming 3 bundles per square).

- 70 From RSMMeans, RSMO Residential Cost Data, Quarter 3 2018. Copyright RSMMeans, Rockland, MA 781-422-5000; All rights reserved.
- 71 From RSMMeans, RSMO Residential Cost Data, Quarter 3 2018. Copyright RSMMeans, Rockland, MA 781-422-5000; All rights reserved.
- 72 Generic term for the product: A sealed roof deck system composed of wood structural panel with integrated factory-bonded underlayment with field applied seam sealing tape.
- 73 Estimate based on consultations with manufacturers and suppliers. Based on an average 2,000 ft² home with a hip roof.
- Consumer retailers report a cost per roll of ~\$50 for the tape product in question. Use of the \$50 figure would increase costs of installation to ~\$350 for the roof type/size scenario, assuming application of same figures for labour.
- Note that the tape in question is code recognized in ESR-2227 as a pressure-sensitive, self-adhering, cold-applied tape to be used as flashing around windows, door frames, wall penetrations and roof penetrations.
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- Huber Engineered Woods. 2018. ZIP System Roof Sheathing & Wall Sheathing. Accessed September 2018 from <http://www.huberwood.com/zipsystem/products/zip-system-wall>
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- 76 Adapted from City of Moore, OK. 2014. High Wind Resistance Residential Construction Requirements. Moore, OK: City of Moore.
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- 77 APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA – The Engineered Wood Association.
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- 80 See:
- Florida Div. of Emergency Management. Gable End Bracing. Accessed August 2018 from <https://apps.floridadisaster.org/hrg/content/roofs/bracing.asp>
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- 81 Florida Division of Emergency Management. Hurricane Retrofit Guide – Gable End Bracing. Accessed September 2018 from <https://apps.floridadisaster.org/hrg/content/roofs/bracing.asp>. Used with permission.
- 82 Morrison, M. 2010. Response of a Two-Story Residential House Under Realistic Fluctuating Wind Loads. PhD Thesis, Department of Civil and Environmental Engineering, University of Western Ontario.
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108 For 150 mm edge and 300 mm centre field nailing patterns, and for gable roofs, if 6d nails are used for sheathing, sheathing failures would be more likely than RTWC failures. However, if 8d nails are used, complete gable roof failure is more likely to occur than sheathing failure. Thus – when toe-nailed connections are used for gable roofs, “the RTWCs are the weak link in the vertical load path, not the sheathing panels.” Further: “For hip roofs, failure of 6d-nailed sheathing is most probable.” However, for hip roofs, “RTWC failure probabilities are similar to those for 8d-nailed sheathing and one may expect to see similar numbers for both.” It is further noted that “median probability of failure for sheathing fastened with 8d nails is about 230 km/h for gable roofs and 260 km/h for hip roofs, assuming no missing nails. With perfect RTWCs – median value for gable roofs is about 200 km/h, for hip roofs it’s about 260 km/h.”

Further: “When the fragility curves for the RTWC and sheathing panel failures of gable roofs are compared, the curves for RTWC failure locate between the two curves for 6d-nail and 8d-nail sheathing panel failures, regardless of the neighbourhood configuration (except at high values of failure probability). Thus, if 6d nails are used for the sheathing, one would expect to see greater numbers of sheathing failures than RTWC failures (or, perhaps, RTWC failures with panels missing from the roof). However, if 8d nails are used for the sheathing, complete (gable) roof failure is more likely... Thus, when toe-nailed connections on gable roof houses are used, the RTWCs are the weak link in the vertical load path, not the sheathing panels.... For hip-roofs, failure of 6d-nailed sheathing is most probable. However, RTWC failure probabilities are similar to those for 8d-nailed sheathing and one may expect to see similar numbers of both.”

Source: Kopp, G., Hong, E., Gavanski, E., Stedman, D., and Sills, D. 2017. Assessment of wind speeds based on damage observations from the August (Ontario) tornado of 17 June 2014. *Canadian Journal of Civil Engineering*, 44, 37-47.

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110 NBCC Requirement for toe-nailing:

| | |
|---|---------------------|
| Trusses and Stick-Frame Rafters (outer) | 3-82 mm (12d) nails |
|---|---------------------|

CSA O86-14 Wood Design Code (12.9.5):

| |
|---|
| $P_{rw} = \phi Y_w L_p n_f J_A J_B$ |
| $\phi = 0.6$ |
| Y_w is withdrawal resistance per unit mm embedded (in second member) ϕY_w is tabulated in CWC WDM for common grades of lumber and nail types |
| L_p is length of nail penetration in main member. NBCC requires at least half of nail length into top plate, so L_p is taken as minimum of $0.5 * L_{fastener}$ |
| $n_f = 3$ (number of nails in connection) |
| $J_A = 0.67$ (reduction factor for toe-nailing) |
| $J_B = 1.0$ (increase allowed in connection with clinched nails) |

| Common Spiral Nails (lowest resistance) | | | ϕY_w [N/mm] | |
|---|---------|-----|-------------------|-----------------|
| Length | | | Douglas Fir-Larch | Spruce-Pine-Fir |
| 3" | 76.2 mm | 10d | 5.2 | 3.7 |
| 3 ¼" | 82.5 mm | 12d | 5.7 | 4.1 |
| 3 ½" | 88.9 mm | 16d | 6.2 | 4.4 |

SPF (minimum):

$$P_{rw} = 4.1 * (0.5 * 82.5) * 3 * 0.67 * 1.0 = 339.94 \text{ N}$$

Unfactored: $P_{rw} = 566.6 \text{ N}$

Douglas Fir-Larch (Same as Morrison & Kopp 2011):

$$P_{rw} = 5.7 * (0.5 * 82.5) * 3 * 0.67 * 1.0 = 475.81 \text{ N}$$

Unfactored: $P_{rw} = 793.0 \text{ N}$

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| Provision type (compliance) | Product | Minimum application temperature | Price/sq. ft. reported by retailers (online and personal communication) |
|------------------------------|---------|---|---|
| Modified bitumen (ASTM 1970) | 1 | 5°C | \$1.05 |
| | 2 | 4.4°C | \$0.78 |
| | 3 | 1.7°C | \$1.00 |
| | 4 | 7.2°C (primer can be used for colder temp. applications) | \$4.40 |
| Butyl (AAMA 711, Level 3) | 1 | -28.9°C | \$2.14 |
| | 2 | -3.9°C (primer can be used for colder temp. applications) | \$0.58 |
| | 3 | -3.9°C | \$3.37 |
| | 4 | -1.1°C | \$1.75 |
| Acrylic (AAMA 711, Level 3) | 1 | -6.7°C (to be reduced to -17°C in 2019) | \$1.48 |
| | 2 | -17.8°C | \$2.12 |
| | 3 | 4.4°C | \$1.35 |
| | 4 | -28.9°C | \$0.98 |

- 122 Canadian Wood Council. 2014. Engineering Guide for Wood Frame Construction. 2014 Edition. Ottawa, ON: Canadian Wood Council. See page B-19: "Note: A 2-3 mm gap (the same size as a typical sheathing nail diameter) between all panel edge and end joints is required to minimize the potential for panel buckling due to wood sheathing's moisture related to expansion and shrinkage. This gap can be sealed appropriately to form an air barrier."
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- 150 Canadian Asphalt Shingle Manufacturers' Association. 2016. Technical Bulletin No. 11 – Wind Protection of Asphalt Shingle Roofs. East Montreal, QC: Canadian Asphalt Shingle Manufacturers' Association.
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- 153 DeLeon, M.A. and Pietrasik, P.C. 2009. Assessing Wind Damage to Asphalt Roof Shingles. Plano, TX: Nelson Architectural Engineers.
- 154 Adapted from:
- City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.
- The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA – The Engineered Wood Association.
- 155 Adapted from Insurance Institute for Business and Home Safety. 2015. High Wind Standards. Tampa, FL: Insurance Institute for Business and Home Safety.
- 156 Adapted from Insurance Institute for Business and Home Safety. 2015. High Wind Standards. Tampa, FL: Insurance Institute for Business and Home Safety.
- 157 See Insurance Institute for Business and Home Safety. 2015. High Wind Standards. Tampa, FL: Insurance Institute for Business and Home Safety.
- 158 Adapted from Insurance Institute for Business and Home Safety. 2015. High Wind Standards. Tampa, FL: Insurance Institute for Business and Home Safety.

- 159 Adapted from:
City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.
APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light Wood Frame Construction. Tacoma, WA: APA – The Engineered Wood Association.
- 160 Adapted from City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.
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- 162 APA – The Engineered Wood Association. 2018. Building for High Wind Resistance in Light-Frame Wood Construction. Tacoma, WA: APA – The Engineered Wood Association.
- 163 Adapted from City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.
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- 166 For example, notes for Table C1a in CWC 2014 state “for unblocked walls with 600 mm stud spacing, use 12.5 mm wood sheathing in lieu of 9.5 or 11 mm wood sheathing shown in table”
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Baker, P. 2013. External Insulation of Masonry Walls and Wood Framed Walls. Somerville, MA: Building Science Corporation.
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Finch, G., Wang, J. and Ricketts, D. 2014. Guide for Designing Energy-Efficient Building Enclosures. Vancouver, BC: FPInnovations and RDH Building Engineering Inc.
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Kochkin, V. and Wiehagen, J. 2017. Next Generation High Performance Walls – Climate Zones 3-5, Part 2: 2x4 Walls with 1”-1.5” Exterior Insulative Sheathing. Upper Marlboro, MD: Home Innovation Research Labs.
Ricketts, L., Higgins, J. and Finch, G. 2017. Illustrated Guide to R22+ Effective Walls in Wood-Frame Construction in British Columbia. Victoria, BC: BC Housing, City of Vancouver and City of New Westminster.
- 168 Ricketts, L., Higgins, J. and Finch, G. 2017. Illustrated Guide to R22+ Effective Walls in Wood-Frame Construction in British Columbia. Victoria, BC: BC Housing, City of Vancouver and City of New Westminster. Used with permission.

169 See, for example:

EIFS Council of Canada. 2013. EIFS Practice Manual. Richmond Hill, ON: Exterior Insulation Finish Systems Council of Canada.

Kochkin, V. and J. Wiehagen. 2017. Next Generation High Performance Walls – Climate Zones 3-5, Part 2: 2 x 4 Walls with 1”-1.5” Exterior Insulative Sheathing. Upper Marlboro, MD: Home Innovation Research Labs.

Ricketts, L. (RDH Building Science Inc). 2017. Illustrated Guide to R22+ Effective Walls in Wood-Frame Construction in British Columbia. BC Housing.

Yeh, B., Herzog, B., and Glass, S. 2014. Wood Structural Panel and Foam Insulation Systems: Hygrothermal Behavior and Lateral Load Resistance – Experimental Studies. Tacoma, WA: APA – The Engineered Wood Association.

170 See: Canadian Wood Council. 2018. Effective R Calculator. Ottawa: CWC. Available from www.effectiver.ca

171 Sentence NBCC 9.3.2.9.(3) Structural wood elements shall be pressure-treated with a preservative to resist decay,

- a) where the vertical clearance between structural wood elements and the finished ground level is less than 150 mm (also see Articles 9.23.2.2. and 9.23.2.3.), or
- b) where
 - i) the wood elements are not protected from exposure to precipitation,
 - ii) the configuration is conducive to moisture accumulation, and
 - iii) the moisture index is greater than 1.00.

(See Note A-9.3.2.9.(3).)

Note A-9.3.2.9.(3) Protection of Structural Wood Elements from Moisture and Decay

There are many above-ground, structural wood systems where precipitation is readily trapped or drying is slow, creating conditions conducive to decay. Beams extending beyond roof decks, junctions between deck members, and connections between balcony guards and walls are three examples of elements that can accumulate water when exposed to precipitation if they are not detailed to allow drainage.

172 Personal Communication, Brad Baumgarten, Red Deer County, September 19, 2018.

173 Available from <https://www2.strongtie.com/software/anchordesigner.html>

174 Image source: Canada Mortgage and Housing Corporation. 2014. Canadian Wood-Frame House Construction. Ottawa: CMHC.

175 CWC 2014 guidelines for construction of roof projections apply to wood frame porches with no walls or with sun rooms or other window walls that do not have lateral load resistance, where $0.8 \leq q_{1/50} < 1.2$.

176 CWC 2014 includes a design provision related to anchorage of columns: Part B 8.4 COLUMN ANCHORAGE 8.4.1 Resistance to Uplift: “Where a column supports a roof, connections shall be provided at the top and the base of the column to resist the factored wind uplift loads minus the factored dead load calculated using a principal load factor of 0.9”.

177 NBCC 9.23.6.2. Anchorage of Columns and Posts

- (1) Except as provided in Sentences (2) and (3), exterior columns and posts shall be anchored to resist uplift and lateral movement.
- (2) Except as provided in Sentence (3), where columns or posts support balconies, decks, verandas or other exterior platforms, and the distance from the finished ground to the underside of the joists is not more than 600 mm,
 - (a) the columns or posts shall be anchored to the *foundation* to resist uplift and lateral movement, or
 - (b) the supported joists or beams shall be directly anchored to the ground to resist uplift.
- (3) Anchorage is not required for platforms described in Sentence (2) that,
 - (a) are not more than 1 *storey* in height,
 - (b) are not more than 55 m² in area,
 - (c) do not support a roof, and
 - (d) are not attached to another structure, unless it can be demonstrated that differential movement will not adversely affect the performance of the structure to which the platform is attached.

178 Additional justification and detail for this recommendation:

Uplift forces applied to porch roofs and raised decks during design winds conditions can cause support posts to be lifted off of their supports causing structural damage to the building. The anchoring requirements are not currently provided in a prescriptive format leading to inadequately anchored installations in the field.

By adequately attaching porch roof support beams to their posts, and posts to their foundation, the resistance of the posts to uplift forces during windstorms is increased, decreasing the risk of structural damage.

The design uplift force on trusses can be calculated using the Static Procedure defined in Commentary I of the code as follows:

Assuming a house with the following characteristics;

- located in lowest 1-in-50 wind exposure locations of 0.3 kPa (Dryden ON),
- 2.44 m (8') wide porch,
- 2.44 m (8') between posts,
- porch weighs 0.48 kPa (10 psf), and
- open terrain wind exposure,

results in an external uplift force of 1.8 kN per column. Hence, relying on the weight of the porch roof is inadequate and a fastener is needed to anchor the bottom and top of the column to resist uplift forces for all locations in Canada. The design wind pressure for low and medium exposure is 0.8 kPa which would result in a 6.8 kN (1536 lb) uplift load. For the highest wind load of 1.05 (Cape Race, NFLD) 9.3 kN (2099 lb) per column would be required.

Currently porch columns are often toe-nailed to foundations which provides insufficient uplift capacity.

179 Adapted from City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.

180 Riley, M., and Sadek, F. 2003. Experimental Testing of Roof to Wall Connections in Wood Frame Houses. Gaithersburg, MD: Building and Fire Research Laboratory, National Institute for Standards and Technology.

- 181 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- Florida Building Commission. 2017. Florida Building Code. Tallahassee, FL: Florida Building Commission, Florida Department of Business and Professional Regulation;
- International Code Council. 2018. International Residential Code for One and Two-Family Dwellings. Washington, DC: International Code Council.
- 182 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- Florida Building Commission. 2017. Florida Building Code. Tallahassee, FL: Florida Building Commission, Florida Department of Business and Professional Regulation.
- 183 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- Florida Building Commission. 2017. Florida Building Code. Tallahassee, FL: Florida Building Commission, Florida Department of Business and Professional Regulation;
- International Code Council. 2018. International Residential Code for One and Two-Family Dwellings. Washington, DC: International Code Council.
- 184 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- Florida Building Commission. 2017. Florida Building Code. Tallahassee, FL: Florida Building Commission, Florida Department of Business and Professional Regulation.
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- 185 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
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- 186 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- International Code Council. 2018. International Residential Code for One and Two-Family Dwellings. Washington, DC: International Code Council.
- 187 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- 188 American Civil Society of Engineers. 2017. ASCE/SEI Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers.
- Florida Building Commission. 2017. Florida Building Code. Tallahassee, FL: Florida Building Commission, Florida Department of Business and Professional Regulation; International Code Council. 2018.
- International Residential Code for One and Two-Family Dwellings. Washington, DC: International Code Council.
- 189 DASMA Commercial and Residential Garage Door Division. 2018. Technical Data Sheet #155 – Residential and Commercial Wind Load Guides. Cleveland, OH: DASMA.
- 190 DASMA. n.d. Wind Load Calculator, ASCE7-10. Accessed August 2018 from <http://www.dasma.com/dasma-pages/DASMA-tehnical-data-sheets.asp>

- 191 DASMA Commercial and Residential Garage Door Division. 2018. Technical Data Sheet #155u – Residential and Commercial Wind Load Guide: Based on the 2010 National Building Code of Canada. Cleveland, OH: DASMA.
- 192 National Research Council of Canada, Institute for Research in Construction. 2010. User's Guide – NBC 2010, Structural Commentaries (Part 4 of Division B). Ottawa: National Research Council of Canada, Institute for Research in Construction.
- National Research Council of Canada, Institute for Research in Construction. 2010. NBC 2010, Volume 2, 4.1.7. Ottawa: National Research Council of Canada, Institute for Research in Construction.
- 193 DASMA. 2014. Garage Door and Commercial Door Wind Load Guide Based on the 2010 National Building Code of Canada. Cleveland, OH: DASMA.
- 194 DASMA. 2014. Garage Door and Commercial Door Wind Load Guide Based on the 2010 National Building Code of Canada. Cleveland, OH: DASMA.
- 195 National Research Council of Canada. 2018. 2015 editions of Codes Canada publications: Significant Technical Changes. Accessed September, 2018 from https://www.nrc-cnrc.gc.ca/eng/solutions/advisory/codes_centre/technical_changes_2015.html
- 196 Accessed August, 2018 from <http://www.dasma.com/dasma-pages/DASMA-tehcnical-data-sheets.asp>
- 197 DASMA Commercial and Residential Garage Door Division. 2017. Technical Data Sheet #155u – Garage Door and Commercial Door Wind Load Guide Based on the 2010 National Building Code of Canada. Cleveland, OH: DASMA.
- 198 DASMA Commercial and Residential Garage Door Division. 2017. Technical Data Sheet #155u – Garage Door and Commercial Door Wind Load Guide Based on the 2010 National Building Code of Canada. Cleveland, OH: DASMA.
- 199 Image and text from Canadian Wood Council. 2014. Engineering Guide for Wood Frame Construction, 2014 ed. Ottawa, ON: Canadian Wood Council. Used with permission.
- 200 Excerpts of images and text from Building Component Safety Information (BCSI) Canada. 2014. Guide to Good Practice for Handling, Installing, Restraining and Bracing of Metal Plate Connected Wood Trusses. Produced by Structural Building Components Association (SBCA), Truss Plate Institute (TPI), and Truss Plate Institute of Canada (TPIC). For more information, visit sbcindustry.com. Used with permission.
- 201 Note that the 2015 NBCC does not have the same Appendix sentence referring to requirement of roof underlayment:
- OBC A-9.26.6.(1) Underlay Beneath Shingles states: "While underlayment has not traditionally been required by the Code, some shingle manufacturers require its use beneath their products."
- 202 9.23.6.1 Anchorage of Building Frames
- 1) Except as required by Sentence 9.23.6.3.(1), *building* frames shall be anchored to the *foundation* unless a structural analysis of wind and earthquake pressures show anchorage is not required.
 - 2) Except as provided in Sentences (3) to (6), anchorage shall be provided by
 - a) embedding the ends of the first floor joists in concrete, or
 - b) fastening the sill plate to the foundation with not less than 12.7 mm diam anchor bolts spaced not more than 2.4 m o.c.

- 3) For *buildings* with 2 or more floors supported by frame walls that are in areas where the seismic spectral response acceleration, $S_a(0.2)$, is not greater than 0.70 or the 1-in-50 hourly wind pressure (HWP) is equal to or greater than 0.8 kPa but not greater than 1.20 kPa, anchorage shall be provided by fastening the sill plate to the *foundation* with not less than two anchor bolts per *braced wall panel*, where are the anchor bolts used are
- not less than 15.9 mm in diameter, located within 0.5 m of the end of the *foundation*, and spaced not more than 2.4 m o.c, or
 - not less than 12.7 mm in diameter, located within 0.5 m of the end of the *foundation*, and spaced not more than 1.7 m o.c.
- 4) For *buildings* supported by frame walls that are in areas where the seismic spectral response acceleration, $S_a(0.2)$, is greater than 0.70 but not greater than 1.8 and the 1-in-50 hourly wind pressure (HWP) is not greater than 1.20 kPa, anchorage shall be provided by fastening the sill plate to the *foundation* with not less than two anchor bolts per *braced wall panel* located within 0.5 m of the end of the *foundation* and spaced in accordance with Table 9.23.6.1.

Table 9.23.6.1.

Anchor Bolt Spacing where the 1-in-50 HWP ≤ 1.20 kPa and $0.70 < S_a(0.2) \leq 1.8$

Forming Part of Sentence 9.23.6.1(4)

| Anchor Bolt Diameter, mm | $S_a(0.2)$ | Maximum Spacing of Anchor Bolts Along <i>Braced Wall Band</i> , m | | | | |
|--------------------------|-----------------------------|---|-----|-----|-----------------------------------|-----|
| | | Light Construction | | | Heavy Construction ⁽¹⁾ | |
| | | Number of floors supported ⁽²⁾ | | | | |
| | | 1 | 2 | 3 | 1 | 2 |
| 12.7 | $0.70 < S_a(0.2) \leq 0.80$ | 2.4 | 2.3 | 1.8 | 2.4 | 2.0 |
| | $0.80 < S_a(0.2) \leq 0.90$ | 2.4 | 2.3 | 1.8 | 2.4 | 2.0 |
| | $0.90 < S_a(0.2) \leq 1.0$ | 2.4 | 2.2 | 1.5 | 2.4 | 1.8 |
| | $1.0 < S_a(0.2) \leq 1.1$ | 2.4 | 2.1 | 1.4 | 2.4 | 1.6 |
| | $1.1 < S_a(0.2) \leq 1.2$ | 2.4 | 2.0 | 1.3 | 2.4 | 1.5 |
| | $1.2 < S_a(0.2) \leq 1.3$ | 2.4 | 1.9 | 1.3 | 2.4 | 1.5 |
| | $1.3 < S_a(0.2) \leq 1.35$ | 2.4 | 1.8 | 1.2 | 2.3 | 1.4 |
| | $1.35 < S_a(0.2) \leq 1.8$ | 2.4 | 1.8 | 1.1 | 2.3 | 1.4 |
| 15.9 | $0.70 < S_a(0.2) \leq 0.80$ | 2.4 | 2.4 | 2.2 | 2.4 | 2.4 |
| | $0.80 < S_a(0.2) \leq 0.90$ | 2.4 | 2.4 | 2.2 | 2.4 | 2.4 |
| | $0.90 < S_a(0.2) \leq 1.0$ | 2.4 | 2.4 | 2.1 | 2.4 | 2.3 |
| | $1.0 < S_a(0.2) \leq 1.1$ | 2.4 | 2.4 | 1.9 | 2.4 | 2.3 |
| | $1.1 < S_a(0.2) \leq 1.2$ | 2.4 | 2.4 | 1.9 | 2.4 | 2.2 |
| | $1.2 < S_a(0.2) \leq 1.3$ | 2.4 | 2.4 | 1.8 | 2.4 | 2.1 |
| | $1.3 < S_a(0.2) \leq 1.35$ | 2.4 | 2.3 | 1.7 | 2.4 | 2.0 |
| | $1.35 < S_a(0.2) \leq 1.8$ | 2.4 | 2.2 | 1.6 | 2.4 | 1.9 |

Notes to Table 9.23.6.1.:

(1) See Note A-9.23.13.2.(1)(a)(i).

(2) All constructions include support of a roof load in addition to the indicated number of floors.

- 5) Anchor bolts referred to in Sentences (2) to (4) shall be
 - a) fastened to the sill plate with nuts and washers,
 - b) embedded not less than 100 mm in the *foundation*, and
 - c) designed so that they may be tightened without withdrawing them from the *foundation*.
- 6) Where the seismic spectral response acceleration, $S_a(0.2)$, is greater than 1.8 or the 1-in-50 hourly wind pressure is equal to or greater than 1.2 kPa, anchorage shall be designed according to Part 4.

NBCC 9.23.14. Roof and Ceiling Framing

9.23.14.1. Continuity of Rafters and Joists

- 1) Roof rafters and joists shall be continuous or shall be spliced over vertical supports that extend to a suitable bearing.

9.23.14.2. Framing around Openings

- 1) Roof and ceiling framing members shall be doubled on each side of openings greater than 2 rafter or joist spacings wide.

9.23.14.3. End Bearing Length

- 1) The length of end bearing of joists and rafters shall not be less than 38 mm.

9.23.14.4. Location and Attachment of Rafters

- 1) Rafters shall be located directly opposite each other and tied together at the peak, or may be offset by their own thickness if nailed to a ridge board not less than 17.5 mm thick.
- 2) Except as permitted in Sentence (3), framing members shall be connected by gusset plates or nailing at the peak in conformance with Table 9.23.3.4.
- 3) Where the roof framing on opposite sides of the peak is assembled separately, such as in the case of factory-built houses, the roof framing on opposite sides is permitted to be fastened together with galvanized-steel strips not less than 200 mm by 75 mm by 41 mm thick spaced not more than 1.2 m apart and nailed at each end to the framing by at least two 63 mm nails.

9.23.14.5. Shaping of Rafters

- 1) Rafters shall be shaped at supports to provide even bearing surfaces and supported directly above the exterior walls.

9.23.14.6. Hip and Valley Rafters

- 1) Hip and valley rafters shall not be less than 50 mm greater in depth than the common rafters and not less than 38 mm thick, actual dimensions.

9.23.14.7. Intermediate Support for rafters and Joists

- 1) Ceiling joists and collar ties of not less than 38 mm by 39 mm lumber are permitted to be assumed to provide intermediate support to reduce the span for rafters and joists where the roof slope is 1 in 3 or greater.
- 2) Collar ties referred to in Sentence (1) more than 2.4 m long shall be laterally supported near their centres by not less than 19 mm by 89 mm continuous members at right angles to the collar ties.
- 3) Dwarf walls and struts are permitted to be used to provide intermediate support to reduce the span for rafters and joists.
- 4) When struts are used to provide intermediate support they shall not be less than 38 mm by 89 mm material extending from each rafter to a *loadbearing* wall at an angle of not less than 45° to the horizontal.

- 5) When dwarf walls are used for rafter support, they shall be framed in the same manner as *loadbearing* walls and securely fastened top and bottom to the roof and ceiling framing to prevent over-all movement.
- 6) Solid blocking shall be installed between floor joists beneath dwarf walls referred to in Sentence (5) that enclose finished rooms.

9.23.14.8. Ridge Support

- 1) Except as provided in Sentence (4), roof rafters and joists shall be supported at the ridge of the roof by
 - a) a loadbearing wall extending from the ridge to suitable bearing, or
 - b) a ridge beam supported by not less than 89 mm length of bearing.
- 2) Except as provided in Sentence (3), the ridge beam referred to in Sentence (1) shall conform to the sizes and spans shown in Span Table 9.23.4.2.-L, provided
 - a) the supported rafter or joist length does not exceed 4.9 m, and
 - b) the roof does not support any concentrated loads.
- 3) The ridge beam referred to in Sentence (1) need not comply with Sentence (2) where
 - a) the beam is not less than 38 mm by 140 mm material, and
 - b) the beam is supported at intervals not exceeding 1.2 m by not less than 38 mm by 89 mm members extending vertically from the ridge to suitable bearing.
- 4) When the roof slope is 1 in 3 or more, ridge support need not be provided when the lower ends of the rafters are adequately tied to prevent outward movement.
- 5) Ties required in Sentence (4) are permitting to consist of tie rods or ceiling joists forming a continuous tie for opposing rafters and nailed in accordance with Table 9.23.14.8.

Table 9.23.14.8.

Rafter-to-Joist Nailing (Unsupported Ridge)

Forming Part of Sentences 9.23.14.8.(5) and (6)

| Roof slope | Rafter Spacing, mm | Minimum Number of Nails not less than 76 mm Long | | | | | | | | | | | |
|------------|--------------------|--|-----|-------------|------------------------------------|-----|-------------|----------------------------------|-----|-------------|------------------------------------|-----|-------------|
| | | Rafter Tied to every Joist | | | | | | Rafter Tied to Joist every 1.2 m | | | | | |
| | | <i>Building Width, up to 8 m</i> | | | <i>Building Width, up to 9.8 m</i> | | | <i>Building Width, up to 8 m</i> | | | <i>Building Width, up to 9.8 m</i> | | |
| | | Roof Snow Load, kPa | | | Roof Snow Load, kPa | | | Roof Snow Load, kPa | | | Roof Snow Load, kPa | | |
| | | 1.0 or less | 1.5 | 2.0 or more | 1.0 or less | 1.5 | 2.0 or more | 1.0 or less | 1.5 | 2.0 or more | 1.0 or less | 1.5 | 2.0 or more |
| 1 in 3 | 400 | 4 | 5 | 6 | 5 | 7 | 8 | 11 | – | – | – | – | – |
| | 600 | 6 | 8 | 9 | 8 | – | – | 11 | – | – | – | – | – |
| 1 in 2.4 | 400 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 10 | – | 9 | – | – |
| | 600 | 5 | 7 | 8 | 7 | 9 | 11 | 7 | 10 | – | – | – | – |
| 1 in 2 | 400 | 4 | 4 | 4 | 4 | 4 | 5 | 6 | 8 | 9 | 8 | – | – |
| | 600 | 4 | 5 | 6 | 5 | 7 | 8 | 6 | 8 | 9 | 8 | – | – |
| 1 in 1.71 | 400 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 7 | 8 | 7 | 9 | 11 |
| | 600 | 4 | 4 | 5 | 5 | 6 | 7 | 5 | 7 | 8 | 7 | 9 | 11 |
| 1 in 1.33 | 400 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 6 | 5 | 6 | 7 |
| | 600 | 4 | 4 | 4 | 4 | 4 | 5 | 4 | 5 | 6 | 5 | 6 | 7 |
| 1 in 1 | 400 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 |
| | 600 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 |

- 6) Ceiling joists referred to in Sentence (5) shall be fastened together with at least one more nail per joist splice than required for the rafter to joist connections shown in Table 9.23.14.8.
- 7) Members referred to in Sentence (6) are permitted to be fastened together either directly or through a gusset plate.

9.23.14.9. Restraint of Joist Bottoms

- 1) Roof joists supporting a finished ceiling, other than plywood [or OSB]...shall be restrained from twisting along the bottom edges by means of furring, blocking, cross bridging or strapping conforming to Article 9.23.9.3.

9.23.14.10. Ceiling Joists Supporting Rook Load

- 1) Except as permitted in Sentence (2), ceiling joists supporting part of the roof load from the rafters shall be not less than 25 mm greater in depth than required for ceiling joists not supporting part of the roof load.
- 2) When the roof slope is 1 in 4 or less, the ceiling joists referred to in Sentence (1) shall be determined from Span Tables 9.23.4.2.-C to 9.23.4.2.-F and 9.23.4.2.-L for roof joists.

9.23.14.11. Roof Trusses

- 1) Roof trusses which are not designed in accordance with Part 4 shall
 - a) be capable of supporting a total ceiling load (*dead load plus live load*) of 0.35 kPa plus two-thirds times the specified live roof load for 24 h, and
 - b) not exceed the deflections shown in Table 9.23.14.11. when loaded with the ceiling load plus one and two-thirds times the specified roof snow load for 1 h.

Table 9.23.14.11.

Maximum Roof Truss Deflections

Forming Part of Sentence 9.23.14.11.(1)

| Truss Span | Type of Ceiling | Maximum Deflection |
|---------------|------------------------------------|--------------------|
| 4.3 m or less | Plaster or gypsum board | 1/360 of the span |
| | Other than plaster or gypsum board | 1/180 of the span |
| Over 4.3 m | Plaster or gypsum board | 1/360 of the span |
| | Other than plaster or gypsum board | 1/240 of the span |

- 2) The joint connections used in trusses described in Sentence (1) shall be designed in conformance with the requirements in Subsection 4.3.1. (See Note A-9.23.14.11.(2).)
- 3) Where the length of compression web members in roof trusses described in Sentence (1) exceeds 1.83 m, such web members shall be provided with continuous bracing to prevent blocking.
- 4) Bracing required in Sentence (3) shall consist of not less than 19 mm by 89 mm lumber nailed at right angles to the web members near their centres with at least two 63 mm nails for each member.
- 5) Where the ability of a truss design to satisfy the requirements of Sentence (1) is demonstrated by testing, it shall consist of a full scale load test carried out in conformance with CSA S307-M, "Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings."
- 6) Where the ability of a truss design to satisfy the requirements of Sentence (1) is demonstrated by analysis, it shall be carried out in accordance with good engineering practice such as that described in TPIC 2014, "Truss Design Procedures and Specifications for Light Metal Plate Connected Wood Trusses (Limit States Design)."

Subsections and Articles noted in Appendix A-9.23.13 but are not directly referenced for measures presented in this draft:

NBCC 9.23.15. Subflooring

NBCC 9.23.15.5. Subfloor Thickness or Rating

NBCC 9.23.9. Floor Joists

NBCC 9.23.9.8. Support of Walls

NBCC 9.29.5. Gypsum Board Finished (Taped Joints)

NBCC 9.29.5.8. Spacing of Nails

NBCC 9.29.5.9. Spacing of Screws

NBCC 9.29. Interior Wall and Ceiling Finishes

NBCC 9.29.6 Plywood Finish

NBCC 9.29.6.3. Nails and Staples

NBCC 9.29.9. Particle Board, OSB or Waferboard Finish

NBCC 9.29.9.3. Nails

203 Bracing to Resist Lateral Loads in Low Load Locations:

Of the 679 locations identified in Appendix C, 614 are locations where the seismic spectral response acceleration, $S_a(0.2)$, is less than or equal to 0.70 and the 1-in-50 hourly wind pressure is less than 0.80 kPa. For buildings in these locations, Sentence 9.23.13.1.(2) requires only that exterior walls be braced using the acceptable materials and fastening specified. There are no spacing or dimension requirements for braced wall panels in these buildings.

Structural Design for Lateral Wind and Earthquake Loads:

In cases where lateral load design is required, CWC 2014 "Engineering Guide for Wood Frame Construction," provides acceptable engineering solutions as an alternative to Part 4. The CWC Guide also contains alternative solutions to further assist designers and building officials to identify the appropriate design approach.

204 Subsections and Articles referenced here:

NBCC 9.23.13 Bracing to Resist Lateral Loads Due to Wind and Earthquake (see Note A-9.23.13)

NBCC 9.23.13.4. Braced Wall Bands

NBCC 9.23.13.5. Braced Wall Panels in Braced Wall Bands

NBCC 9.23.13.6. Martials in Braced Wall Panels

NBCC 9.23.13.7. Additional System Considerations

205 City of Moore, OK. 2014. High wind resistance residential construction requirements. Moore, OK: City of Moore.

City of Moore wind provisions referenced in Multihazard Mitigation Council. 2017. Natural Hazard Mitigation Saves 2017 Interim Report: An Independent Study – Summary of Findings. Principal Investigator Porter, K.; co-Principal Investigators Scawthorn, C., Dash, N., Santos, J., and Schneider, P., Director, MMC. Washington, DC: National Institute of Building Sciences



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