Prepared for:

SMITHS FALLS

RISE AT THE FALLS

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Heritage Building Structural Pathology Engineering Report

25 Old Mill Road
Town of Smiths Falls

A HERITAGE BUILDING SIMCOE, ONTARIO

HERITAGE BUILDING STRUCTURAL PATHOLOGY ENGINEERING REPORT

25 OLD MILL ROAD, SMITHS FALLS, ON

EXECUTIVE SUMMARY

BACKGROUND

A structure fire occurred on May 8, 2023, at the heritage designated Waterworks Building Complex at 25 Old Mill Road in Smiths Falls, Ontario. An engineer's report recommended immediate demolition. VALLEE was retained to explore preservation considerations and options for the heritage buildings. The exterior walls were inspected and samples of stone, brick and mortar were taken, for material analysis. All available drawings and reports were provided by Town staff for the analysis of the stability of the structure.

ENGINEERING ANALYSIS

Engineering calculations were made to understand the global stability of the stone masonry walls under applied environmental live loads (i.e.: wind, seismic). This was achieved by determining the strength of the walls compared to the considered applied loads. The governing failure mode is projected to be overturning of the upper portion of the stone masonry wall due to horizontal failure caused by wind induced pressure.

All else remaining equal, based on the data available, our method of analysis, and the validity of our assumptions, it is our structural engineering opinion the upper portion of the three-storey stone masonry wall has an approximate 20% probability of failure (collapse) during a wind event that exceeds a 1:50 probability of occurring. In the event that lateral bracing is installed, these probabilities approach zero.

Therefore options for preservation are available for consideration.

RECOMMENDATIONS

Our research revealed that demolition is less costly than restoration, but of a similar order of magnitude. In addition to the option of demolition, our recommendations for a stabilizing the walls based on our analysis would include:

- Install exterior and interior lateral supports by temporary scaffolding or other means.
- Commence fire damage abatement and any hazardous material removals if required.
- Perform stone masonry and brick masonry repairs (i.e. repointing with lime mortar).

It is recommended that the above course of action be implemented during calmer summer months.

The repair course of action would provide structural stability and preservation of the heritage valued buildings while development options are considered.

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Consulting Engineers, Architects & Planners

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TABLE OF CONTENTS

1.0 INTRODUCTION	**1**
2.0 BACKGROUND	**2**
3.0 APPROACH	**3**
4.0 METHODOLOGY	**4**
5.0 ANALYSES	
5.1 Photographic	**6**
5.2 Material	**12**
5.3 Resistance	**14**
5.4 Wind	**15**
5.5 Seismic	**16**
6.0 DISCUSSION	
6.1 Factor of Safety	**17**
6.2 Analysis Results	**17**
6.3 Cost Estimates	**18**
7.0 RECOMMENDATIONS	**20**
8.0 CLOSURE	**21**

APPENDICES

APPENDIX A – EDIFICE Material Report

APPENDIX B – Photographic Log

APPENDIX C – Ottawa International Airport Wind Data

1.0 INTRODUCTION

The Waterworks Building Complex located at 25 Old Mill Road in Smiths Falls, Ontario, is a designated heritage site (*reference: Bylaw 4350-77).* The complex of buildings comprises of several adjoining structures that experienced a fire on May 8, 2023.

Immediately following the fire, an inspection and report was prepared by DFA Engineering Services (DFA) at the request of the building insurer (*reference: DFA File Report: DFA23-176C, dated May 10, 2023).* The conclusion of the DFA report recommended immediate demolition.

G. Douglas Vallee Limited (VALLEE) was retained by the Town of Smiths Falls to provide professional structural engineering expertise to explore preservation considerations and options for the heritage buildings. VALLEE immediately attended the site to visually inspect the structures and obtain material samples. Subsequent to the site visit, material analysis and structural engineering analysis was undertaken.

The subject of this study is focused on the 3-storey 1868 stone masonry structure (Building 2) and the 1886 brick masonry addition (Building 3), which experienced significant fire damage. The adjoining 2-storey 1924/1955 brick masonry addition (Building 4) and a single-storey masonry block workshop (Building 1) were not heavily damaged and were not included in the scope of this assignment.

The complex comprises of a total of seven adjoined distinct buildings; this report will look at the primary 4 buildings noted, with a focus on Buildings 2 & 3.

The buildings as seen in Figures 1 & 2 consist of the following:

- ***Building 1:*** single-storey block masonry workshop;
- ***Building 2:*** three-storey 1868 stone masonry mill;
- ***Building 3:*** three storey 1886 brick masonry waterworks pumping station addition;
- ***Building 4:*** two-storey 1924 filtration plant addition with 1952-55 renovations and modifications.

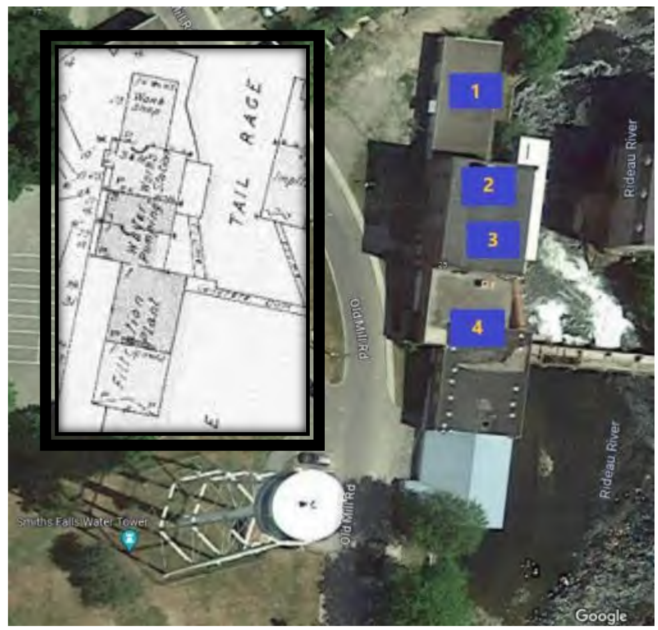


Fig 1. Site Plan view *(inset: 1959 site plan).*

Fig 2. Elevation view from a time when the facility was still active *(Google 'street-view' June 2009).*

2.0 BACKGROUND

The original three-storey stone masonry building (Building 2) was built by Mr. Jason Gould in 1868 to serve as a grist mill. The adjoining three-storey brick masonry building (Building 3) was added by Capt. Adam Foster in 1886 to serve as a pumping station, which was acquired by the Town around 1910. A filtration plant was added in 1924 (Building 4) which underwent significant modifications between 1952-1955.

The Waterworks Building Complex was designated as a heritage property in 1977 for its historic and architectural value. The facility was in full operation as the Town of Smiths Falls Water commission until 2010, providing over a century of water service to the Town.

Fig 3. Datestone set above main entrance

The building has been shuttered since being vacated following the relocation of the Waterworks facilities post-2010.

The fire that occurred on May 8, 2023 consumed the majority of the interior floors and roof of Building 2 and the roof and third floor of Building 3, with some damage occurring to Building 4, and little to no damage occurring to Building 1.

VALLEE was retained by the Town on Friday, May 19, 2023 and immediately mobilized and attended the site on Tuesday, May 23, 2023, with the intention of providing a report on options within a month.

The purpose of the visit was to physically inspect and observe the condition of the buildings, and to obtain samples of stone, brick, and mortar for further study and analysis.

The site was considered an active crime scene at the time of the visit, under investigation from both the Police and Fire Services. In addition to access restrictions imposed by that classification, the building interior was also deemed unsafe due to the present conditions. Access for the inspection of the exterior walls and overhead interior viewing was facilitated using the Smiths Falls FS aerial access platform vehicle.

Fig 4. The west façade as seen on May 23, 2023.

Town staff was also able to provide record drawings of the buildings dating back to the 1924 filtration plant addition. The most useful drawing set proved to be the 1992-1994 interior renovations of Buildings 2 & 3 which contained detailed information regarding the structures and their geometry.

VALLEE would like to extend sincere gratitude to the Smiths Falls Fire Service, the Smiths Falls Police Service, and municipal staff with the Town of Smiths Falls – all of which without whom this structural engineering analysis and report would not have been possible.

3.0 APPROACH

The approach to this project includes a fundamental ‘first-principles’ calculation of the structural stability of the masonry walls supplemented with engineering calculation methods provided by various codes, standards, guidelines, research, and good engineering judgement.

In general, the overall approach included the following tasks:

- Perform a site visit to observe and photograph site conditions, and obtain material samples of stone, brick, and mortar.
- Deliver the sampled material to EDIFICE (sub-consultant) for visual review and physical analysis.
- Research background information, drawings, current and historical site photographs and reports, 19th century masonry characteristics, and multiple analysis techniques.
- Calculate anticipated ranges of external lateral loads (i.e.: wind, seismic) applied to the structure based on local data and OBC design requirements.
- Calculate anticipated ranges of masonry strength (i.e.: load resistance) characteristics.
- Develop an analytical calculation model of the structure for an iterative analysis utilizing ranges of applied loads and material and structural strength characteristics.
- Review analysis result to determine governing failure mode(s) and calculate theoretical *Factors of Safety** to determine the nature of the existing stability or instability of the structures.
- Report the analysis result with consideration of available options and recommendations.
- Review the order of magnitude of potential cost of demolition compared to stabilization and repair.

Each of the above noted tasks within our approach are described in greater detail throughout this report. In discussion of the available options, third party contractors with specific qualifications and experience with heritage building restoration or demolition were consulted to gain an understanding of the magnitude of costs for considered options.

It is important to reiterate that VALLEE was retained by the Town of Smiths Falls to provide professional structural engineering expertise to explore preservation considerations and options for the heritage buildings. The analysis will utilize engineering principles, methods, codes, standards, and guidelines – however the mandate to be satisfied is not for a forensic engineering report. The work undertaken is not to determine a cause but rather simply to better understand the current stability of the building and if options other than immediate demolition can be considered.

[*]Factor of Safety refers to a ratio of strength over resistance. For example, a beam that is twice as strong as the worst-case effect of a load applied to it, is considered to have a Factor of Safety of 2 (FS = Resistance / Load) .

4.0 METHODOLOGY

The methods used in the engineering analysis for this assignment are taken from various codes, standards, and guidelines, accepted structural engineering practices, and supplemented by academic research and resources, and good engineering judgement.

A list of reference documents required for this structural engineering analysis includes, but is not limited to:

Codes & Standards

- Ontario Building Code (updated 2020)
  - Part 4 Structural Design
  - Supplementary Standard SB1 – Climate Design Data
  - Supplementary Standard SB1 – Seismic Data
- Design of Masonry Structures (CSA S304)

Record Drawings

- Smiths Falls Filter Plant – 1924 (Building 4)
- Smiths Falls Water Treatment Plant – 1954 (Building 4)
- Smiths Falls Water Commission - 1955 (Building 2)
- Smiths Falls Water Commission – 1994 (Building 2 & 3)

Third Party Reports & Reference Material

- Engineering Assessment - Structural Assessment Following a Fire (DFA23-176C)
  Prepared by DFA Engineering Services Inc., Hamze Mankal, P,.Eng., dated May 10, 2023
- Report on Materials Analysis – 25 Old Mill Rd, Smiths Falls, ON.
  Prepared by: EDIFICE, Dr. Christopher Cooper, Ph.D., dated June 9, 2023.
- Seismic History and Data – Engineering Characteristics of Ground Motion Records of the Val-des-Bois, Quebec, Earthquake of June 23, 2010 by Lan Lin and John Adams (Geological Survey of Canada, Natural Resources Canada, Ottawa, Ontario, Canada)
- Wind Speed at Macdonald-Cartier International Airport: Years 2012-2022 inclusive
  Source: www.weatherspark.com
- Google Street View: 25 Old Mill Road at June 2009, Sept 2012, June 2014, May 2019, Aug 2021
  Source: www.google.com/maps/@44.898,-76.022
- Potsdam Sandstone: Composition and Qualities, James Carl,
  Source: http://potsdampublicmuseum.org/subpages/95/109/20/composition-and-qualities

Research & Academic Papers

- Porosity and Bulk Density of Sedimentary Rocks – United States Department of the Interior
  Prepare by G. Edward Manger, published Washington, 1963
- A Comparative Study of the Durability and Behaviour of Fat Lime and Feebly-Hydraulic Lime Mortars
  Prepared by S. Pavia, E. Treacy, published University of Dublin-Trinity College, UK, 2006

Site Visit Observation Notes & Photographs

- A site visit was conducted on Tuesday, May 23, 2023, by the author in attendance from 10am until 4pm in the presence of the Smiths Falls Police Service & Fire Service (SFPS & SFFS respectively).
- With the aid of the SFFS aerial platform vehicle and staff, the exterior walls of Building 2 & 3 were visually and physically inspected with samples of stone, brick, and mortar obtained.
- Over 150 photographs and videos were obtained during the site visit. The record of photographs is logged in the appendices.

Photograph Reference:

The log of photographs found in the appendix are referenced by their location on Buildings 2 & 3 in accordance with the following grid and level key plan:

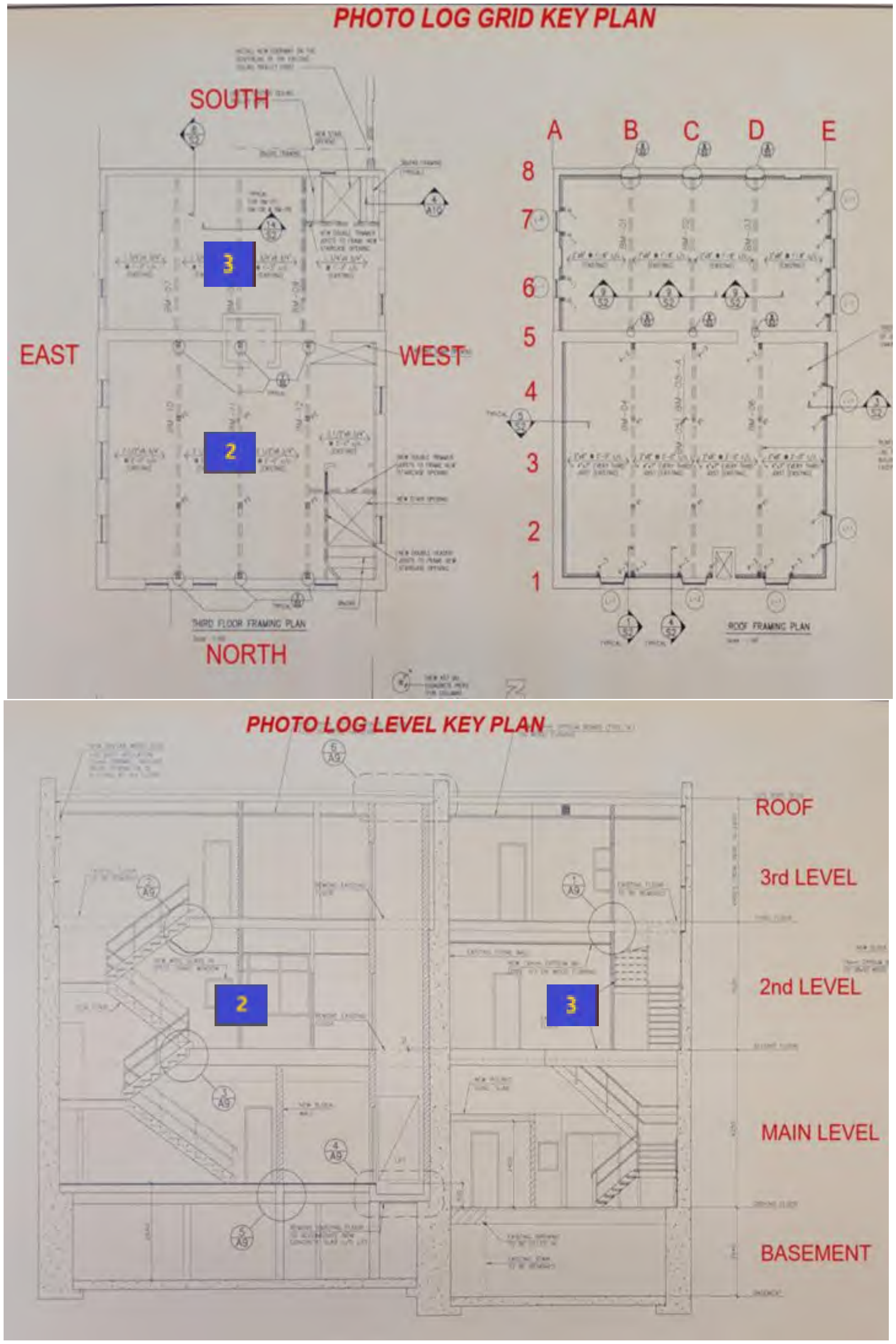


Fig 5. Photograph Log Plan and Level Keyplans.

5.0 ANALYSES

5.1 Photographic Analysis

To gain a better understanding of the condition of the structure prior to the fire, several historic photographs were reviewed in comparison to the photographs taken during the site visit. We are interested in understanding if masonry cracks and deteriorations observed at the time of the site visit were evident prior to the fire.

To visualize the rate of change in condition for the vacated building, compare Fig. 6 when the building was in active use to Fig. 8 below. A chronology of the west façade over the last decade is illustrated below.

Fig 6. Google street view from September 2009.

Fig 7. Google street view from June 2014.

Fig 8. Google street view from August 2021.

The following photographs illustrate the typical cracks seen in the front (west) face of the building. Areas that experienced direct exposure to flame heat and thermal shock from firefighting water experienced spalling of surficial repointed mortar. Parapet masonry typically experiences advanced deterioration due to exposure.

Fig 9. Note the appearance of post-fire cracks at RE2, 3E2, RE4 and parapet damage at RE8.

Fig 10. Note that mortar cracks at RE2, 3E2, 2E2, and RE4 existed before the fire.

Historical photographs of the same areas as seen in Figs. 9 & 10 were reviewed to determine if the majority of cracks existed prior to the fire, or if the fire had caused additional damage. Pyramidal cracks in the exterior face were observed during the site visit at various locations, as well as areas of mortar repointing.

The photo comparison below is a crack located above the north window on the front face of the building at the roof level: gridpoint location 3E2 to RE2.

Fig 9a. Crack at 3E2-RE2 before…

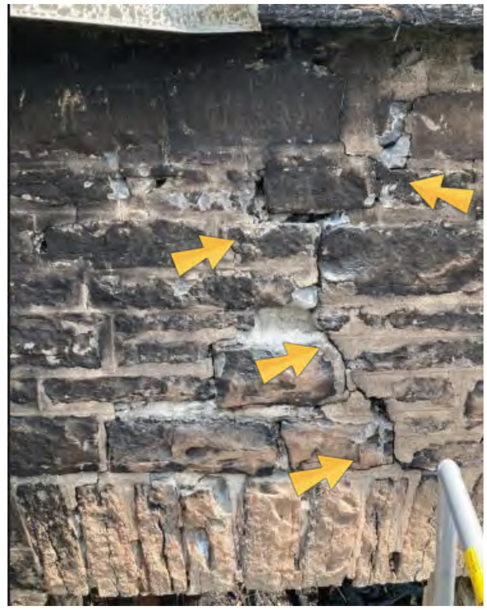


Fig 9b. … and after.

This is an example of a pre-existing crack that had been repointed in a previous repair but remains in similar condition post-fire. Our general observations of the mortar indicate that the previous mortar repairs suffered the most damage as the previous bond to the original stone and mortar was likely compromised in areas such as this prior to the fire; thermal shock accelerated spalling of poorly bonded surficial repointed mortar.

Fig. 10 Concave parapet at RE3.

Fig.11 Convex parapet at RB1.

Slight parapet concave and convex undulations noted at RE3 & RB1 in Figs. 10 & 11 appeared to be original to the building construction and not a result of the fire.

As seen below in Fig 12, the entirety of the timber floor and roof is destroyed by fire. The face of the wall exhibits similar degradation as noted on exterior faces that experienced direct exposure to flame heat and thermal shock from firefighting. The surficial mortar and parging experienced varying degrees of spalling in areas exposed to high thermal gradient, exposing the original lime mortar behind the surface. Arguably, the highest heat and thermal gradient shock would have occurred on the interior face of the rear (east) wall in the area from gridpoints A2-A4 at the 3rd level and roof.

Fig 12. Interior face of stone wall 3A2-3A4.

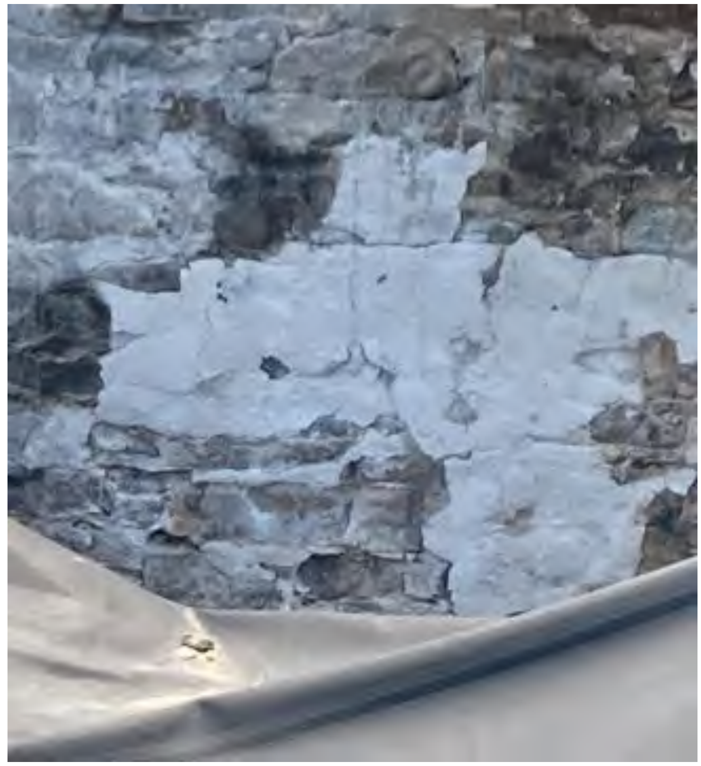


Fig 13. Plaster parging example at 3A5-3C5.

It is unclear if the white colour noted on some of the interior walls was due surficial mortar flushed down the wall face by firefighting or if it is remnant plaster, parging, or gypsum board. Plaster and parging was observed on the walls in various locations such as 3A5-3C5 and 3A8-3E8; gypsum drywall board is indicated in the provided drawings as part of the 1994 renovations.

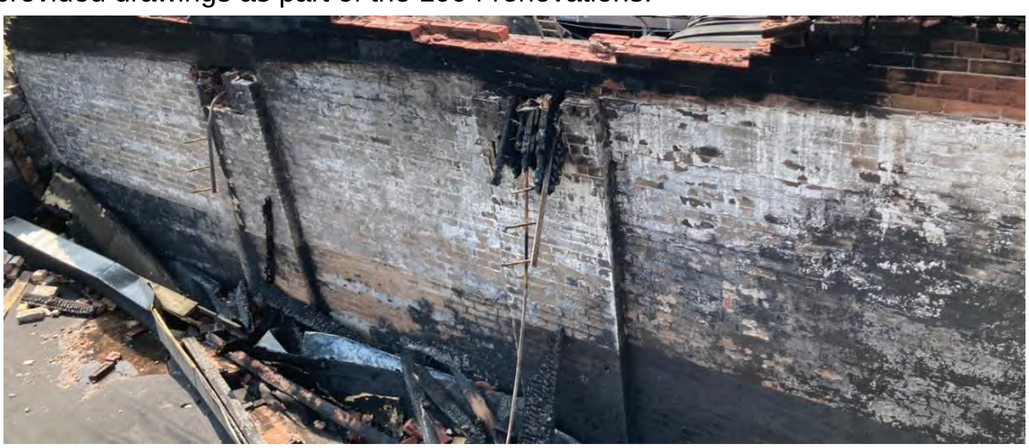


Fig 14. Plaster parging evident on the interior face of 3A8-3E8.

Depending on the location observed, the white residue is likely of these substances or a combination thereof.

The interior face of the brick wall of the 3rd level of Building 3 appears to have had a plaster parging as did other interior walls observed during the site visit.

At various locations on the exterior faces of both Building 2 & 3, there are exterior plates with through-bolt tie-backs. In many cases for buildings of this vintage, these types of connections are typically used as a post-tensioned tie-back to prevent or arrest masonry wall displacements. In effect, the opposite function appears to the case here, whereas the interior floor structure renovations were tied into the stone walls for additional stability and anchorage. Upon examination of the provided structural design drawings, these appear to function more as anchorage for the interior renovation to the solid stone and masonry walls.

Fig 15. Through-bolt wall anchor at 3E2.

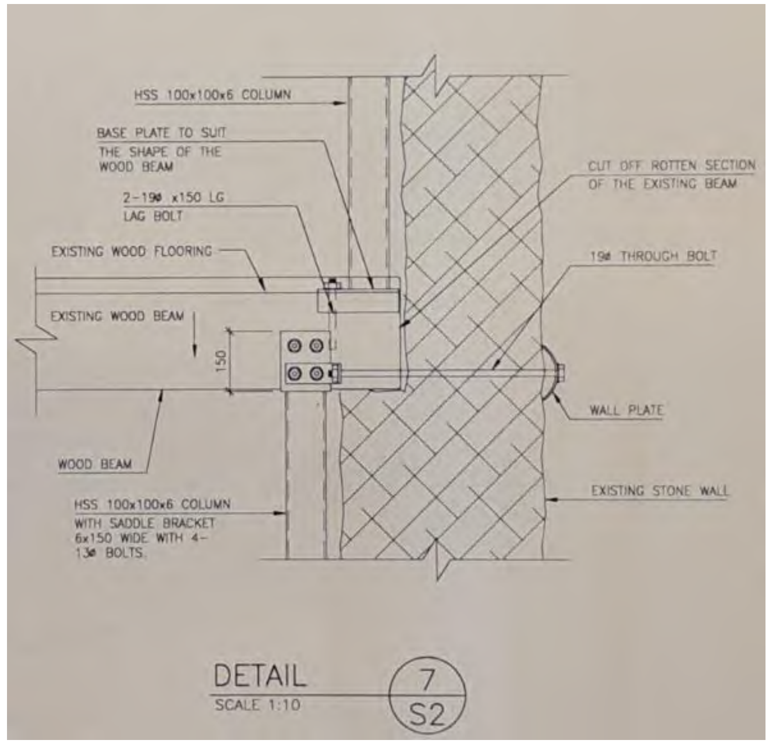


Fig 16. Typical stone wall anchor detail from 1994 work.

Areas that experienced high thermal gradients typically exhibit spalled repointed mortar. Areas of wall face that were not severely exposed to flame heat and thermal shock appear to remain in sound condition. This is evident in a large portion of the exterior wall faces that include stone and brick masonry in both buildings.

Fig 17. Brick voussoir and sill at 3E6.5.

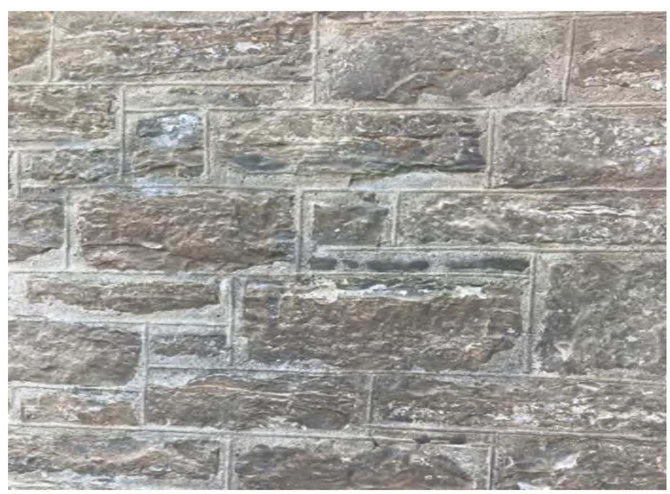


Fig 18. Typical stone face in good condition.

Fig 19. Building 3 is triple wythe brick masonry.

The damaged parapet at the southwest corner of Building 3 (1886 brick masonry) provided an opportunity to review the nature of the triple wythe masonry work as well as the condition and quality of the exterior and interior bricks. Iron (red) pigmented exterior mortar was observed in contrast to the interior mortar. The accumulated effects of over a century of freeze-thaw deterioration for exposed parapets weaken the mortar bond. The damage observed at RE8 and along the rear wall from RA5-RA8 would not be unexpected given the force applied by water pressure during firefighting. The physical characteristics of the mortars will be discussed in more detail in *Sub-Section 5.2 Material Analysis* and in the reference material analysis report from EDIFICE in the appendices.

The parapet of the rear wall of Building 3 suffered extensive damage (RA5-RA8). The masonry arch lintels remain intact. Exterior views taken from Google Street View indicate the pre-fire condition of the rear wall.

Fig 19. View of Building 3 rear wall at 3rd level along the roof line (RA5-RA8).

Fig 20a. Isometric view of rear of buildings 2, 3, & 4.

Fig 20b. Rear elevation view of Buildings 2&3

5.2 Material Analysis

Masonry Characteristics

Physical characteristics of the stone, brick, and mortar are essential to the engineering analysis. Physical properties such as mass, density and compressive strength are required to calculate the stability of the walls. Typical values found in reference documents were used for the initial analysis, which were then corroborated by physical analysis results provided by Dr. Christopher Cooper (refer to the EDIFICE report in appendices for additional details).

Ranges of values were used (i.e.: minima, median, maxima) for structural characteristics such as material density and mortar strength, given the variation in conditions observed, researched material characteristics, and corroborating material analysis results from EDIFICE. The range of values was used to gain an understanding of the sensitivity that certain properties, such as stone & mortar density, would have on overall stability. In doing so, a range of Factors of Safety were calculated. The lowest calculated Factor of Safety representing the *governing failure mode* will be detailed further in *Section 6.0 Discussion*.

Sandstone

It is important to note that the stone masonry blocks of the original mill are **not limestone** but are actually **sandstone**. Physical and textural observations of the stone include quartz mineral content as well as sedimentary striations which are typically not observed in limestone. This was confirmed in the EDIFICE analysis with physical observations and a wet acid digestion test. The wet acid digestion test is used to determine the composition and proportion of cement, lime, and aggregate in a given sample; it is also useful to determine lime content in a rock sample. It is our opinion that the stone of Building 2 is Potsdam/Nepean Formation sandstone that is typically found in Northern New York, Vermont, Quebec and Eastern Ontario. Construction in the 1868 era would use readily available local materials. This Cambrian era sandstone is a well cemented rock of nearly pure quartz* which is more resilient to fire compared to heat sensitive limestone.

In the 19th century, Potsdam/Nepean Sandstone was highly regarded as a building material. Properties of the rock that give it value as a building material include high compressive strength, attractive coloring, and resistance to weathering. Potsdam Sandstone resists spalling when exposed to fire, making it highly suitable for use as a refractory for lining iron furnaces. Buildings constructed with this rock include portions of Canada's Parliament Buildings (original Centre Block and Library of Parliament) in Ottawa.

Fig 21. Aerial view of Building 2 interior.

Petrophysical changes anticipated to fire-heated sandstone include an increase in stone porosity due to the consumption of clay minerals at high heat. Due to the high quartz content and low clay mineral content of this type of local sandstone, the effect is anticipated to be a difference in appearance due to change in colour and a minor increase in moisture absorption of the stone due to the increased porosity. Research has indicated however that the compressive strength of sandstone of this nature remains robust as corroborated by the EDIFICE analysis.

**James Carl, [Potsdam Sandstone: Composition and Qualities], Potsdam Public Museum website.*

Stone Masonry Density & Strength

The density of the stone masonry wall was estimated using typical individual densities for sandstone and lime-based mortar and approximate proportions of sandstone to mortar (Figs. 21 & 22). This method was used at a number of locations throughout the building exterior and interior. This was done to determine an average composite density for the walls to be used in calculations of wall mass. This value was then compared to EDIFICE reported results and researched typical material densities. The proportions of sandstone to mortar were approximated by reviewing 1m^2 sections of the walls throughout the structure (Fig. 22) and calculating the average composition percentage of sandstone and mortar (lime based). The average composition of the typical stone masonry was approximated to be **78% sandstone and 22% mortar.**

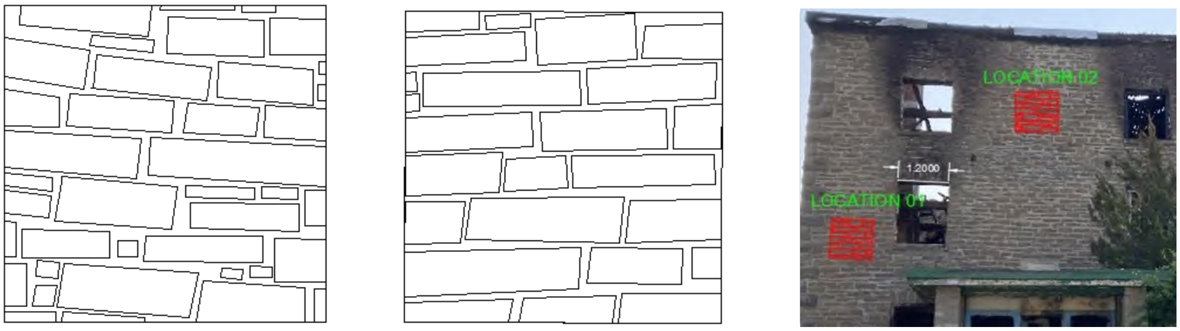


Fig 21. Example of typical stone masonry composition determination method.

The flexural and compressive strength of the stone masonry walls were approximated using a first-principles engineering approach which assumes the wall is solid unreinforced masonry with a running-bond course and no out-of-plane interlocking. This is considered a conservative approach given that the stone masonry walls were constructed with a "snecked" masonry configuration which does include out-of-plane masonry interlocking between wythes. The flexural tensile strength (Fig. 23) and assembly compressive strength (Fig. 24) of the masonry were assumed using conservative low-range values from CSA S304.1 since stone masonry is not considered in modern design standards. These low value estimates are typically exceeded by actual compressive strengths for stone masonry which was corroborated through testing by EDIFICE.

Material	**Typical Density**	**Density corroborated by EDIFICE**
Sandstone (Potsdam/Nepean)	2.44 g/cm^3	2.14 g/cm^3
Mortar (Lime Based)	1.76 g/cm^3	1.70 g/cm^3 (Average)

Fig 22. Material density comparative values used.

Design Value Source	**Flexural Tensile Strength**
CSA S304.1 (Table 5)	0.1 MPa
Corroborated by EDIFICE	0.17 MPa

Fig 23. Flexural tensile strength design values used.

Table 4
Specified compressive strength normal to the bed joint, f'_m, for concrete block masonry, MPa
(See Clauses 5.1.3.3, 5.1.3.5.2, and D.6.1.)

Specified compressive strength of unit (average net area)*, MPa	Type S mortar		Type N mortar	
	Hollow	Solid† or grouted	Hollow	Solid† or grouted
40 or more	22	17	14	10.5
30	17.5	13.5	12	9
20	13	10	10	7.5
15	9.8	7.5	8	6
10	6.5	5	6	4.5

Fig 24. Compressive Strength CSA S304.1

5.3 Resistance Analysis

Engineering calculations were made to understand the global stability of the stone masonry walls with respect to bearing, overturning, and flexure (bending). This was achieved by determining the strength of the walls, or resistance to: bearing, overturning, and flexure, in comparison to the considered applied lateral loads of wind and seismic ground movements.

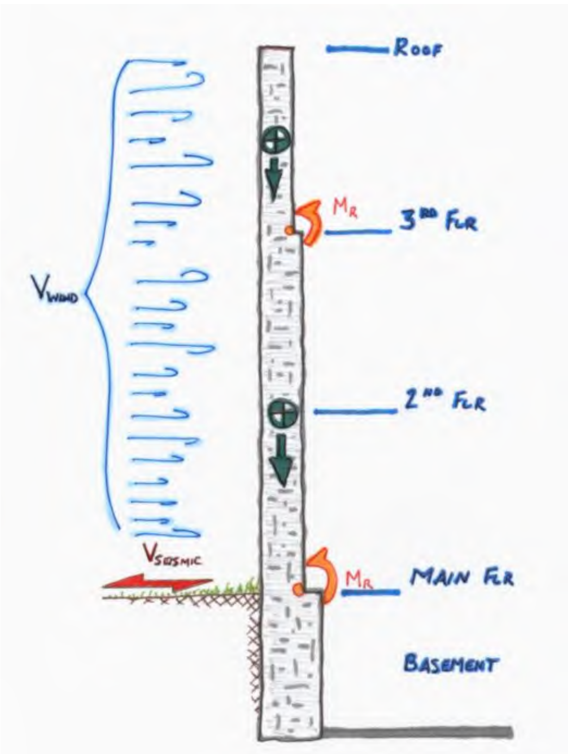


Fig 25. Schematic cross-section of wall.

Building Geometry
The detailed drawings indicated the geometry of the area of greatest concern – Building 2, the original 1868 stone masonry mill. Of particular note, are the details of the wall construction geometry, which are dimensioned approximately 1.0m (3'3") thick for the perimeter of the foundation, 0.6m (2') thick for the main and second levels, and 0.5m (1'8") thick for the third level and roof parapet. These wall geometry characteristics were confirmed in the field as well as in the photographs appended.

The resistance analysis assumed that the interior timber floor structures cannot be relied upon to provide lateral bracing to the vertical walls. The analysis also assumed conservatively and confirmed that the mortar in the worst-case scenario could not be relied upon to provide tensile adhesion nor friction between stones (i.e.: at pyramidal crack locations). This approach was considered conservative because a wall failure or collapse would need to overcome not only any remaining mortar strength (not unrealistic) but friction between stone units as well. This approach to the resistance to overturning provides a more conservative (lower risk) data set for comparison with applied loads, predicted failure modes, and associated factors of safety.

It was determined that the primary contributing factor to the overturning resistance of the wall is the self-weight of the stone masonry. The self-weight, due to the force of gravity and acting normal (perpendicular) to applied lateral load, provides resistance to overturning. This resistance is also called a resisting 'moment' (i.e. M_R) or resisting torque, that acts to oppose the applied lateral load. This resistance is directly proportional to wall mass and thickness.

In simpler terms, the heavier the wall and/or the thicker the wall, the more resistance there is to overturning.

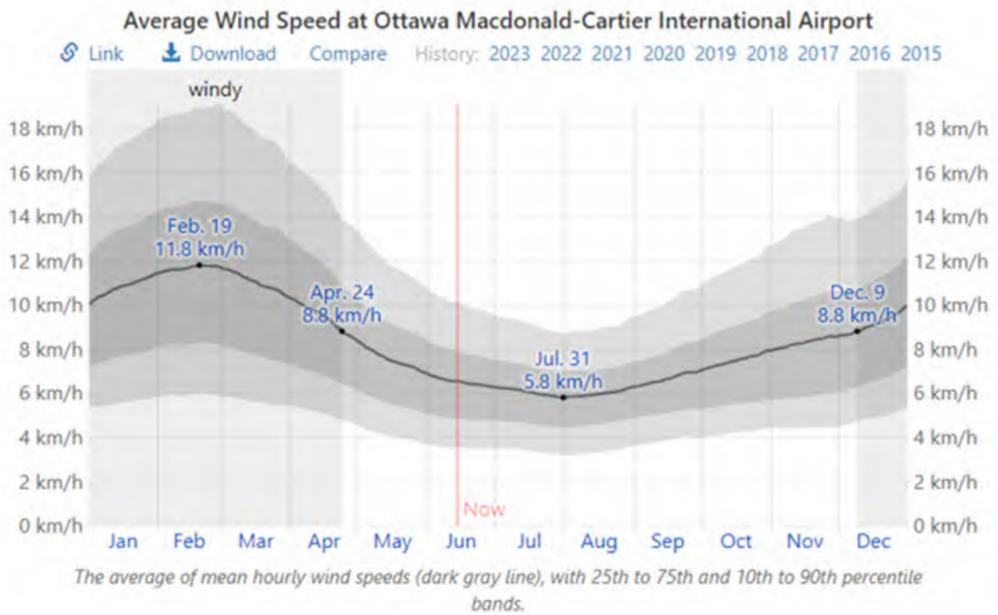
The cross section of the stone masonry was visible at openings, where wall thickness and interlocking stones between the interior and exterior faces can be observed.

Fig 26. Cross-section of wall at 2E2 opening.

5.4 Wind Analysis

The Ontario Building Code (OBC) prescribes a design wind pressure that is based on a mean hourly wind speed with the probability of 1:50 of being exceeded per year (i.e.: Smiths Falls hourly wind pressure from OBC SB-1 Table 2: q_{50} = 0.41kPa). The external and internal (windward and leeward) wind forces on the structure were calculated in accordance with the OBC. The wind force calculation considered the following boundary conditions and environmental considerations as stipulated within Section 4.1.7 of the OBC:

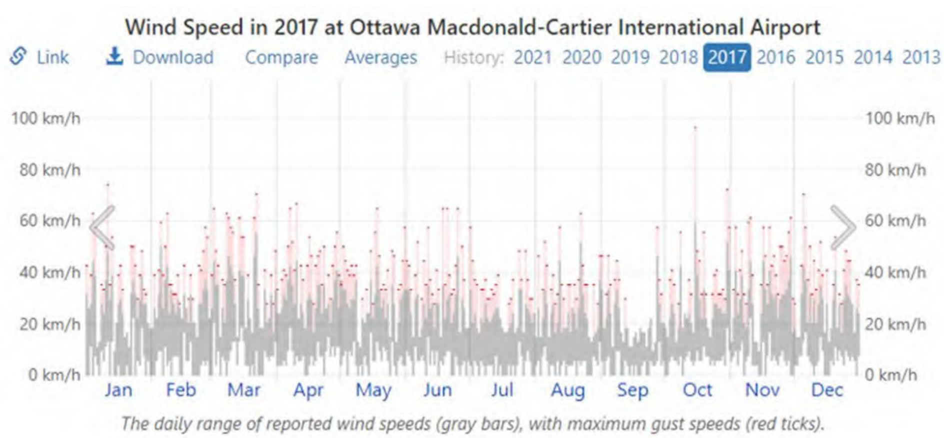
- “Normal” Building Importance (OBC Table 4.1.7.3);
- Situated in “Rough Terrain” or an urban/suburban setting (OBC Section 4.1.7.3.5.a);
- Large openings and open structure when considering internal wind pressures (OBC Table 4.1.7.7).



This prescribed wind load from the OBC for current design requirements represents an approximate **wind speed of 85-95 km/h** (e.g.: gale to strong gale force wind).

For comparison, wind data from the Ottawa MacDonald Cartier International Airport reported wind speeds of **< 20 km/h** for more than **90% of the year**.

Fig 27. Average wind speed is typically less than 20km/h 90% of the time.



Data obtained from the same source also indicates the number of occurances that gusts have exceeded the wind threshold value. Wind speeds in excess of 80km/h have occurred approximately 14 times over the course of the previous decade (since 2012).

The wind data obtained from the Ottawa Airport is included in the appendices.

Fig 28. Wind gusts exceeding 80km/h have occurred 14 times since 2012.

5.5 Seismic Analysis

The Ontario Building Code (OBC) prescribes a seismic design spectrum that has a probability of exceedance **of 2% in 50 years (0.04% annual occurrence)** that is used to determine the loads and effects of earthquakes that a structure may experience. The design response spectral acceleration (g) for periods (T_a) ranging from 0 seconds to 4 seconds for the structure was calculated in accordance with the most recent version of the OBC and plotted with the resulting loads on the structure.

The existing 19th century structures obviously were not designed in accordance with the most recent version of the OBC yet remain standing. Records of seismic activity for this region have been kept since the early 20th century; the OBC design event has not yet occurred since the recording of seismic activity began around 1914. It was determined that this structure would have a high probability of failure if the OBC seismic design event occurred. This would have also been the analysis result if the fire had not occurred.

The equivalent seismic lateral load that could cause potential failure of the building was calculated based on the calculated resistance of the stone masonry wall. This is illustrated as the *"Structure Critical Spectrum"* (Refer to the red line on Fig. 29). Seismic events above the red line would likely cause failure. For comparison, the OBC design requirement (black line on Fig 29) and the seismic activity recorded during the Val-des-Bois earthquake (Refer to the colour data lines on Fig. 29) that occurred on June 23, 2010, are shown together on Fig 29. The recent (2010) Val-des-Bois event produced the strongest ground shaking felt in Ottawa since records were started in 1914.

When the response spectra from Val-des-Bois is compared to the *"Structure Critical Spectrum"* and the *Design Spectrum (OBC)* shown in Fig. 29 at the natural period of the structure (T_a = 0.310sec.), the forces experienced during Val-des-Bois were lower by a factor of approximately 3.9 and 5.4, respectively.

Based on this analysis, for the purpose of this study, we have assumed that the likelihood of a critical wind event far exceeds the probability of a critical seismic event.

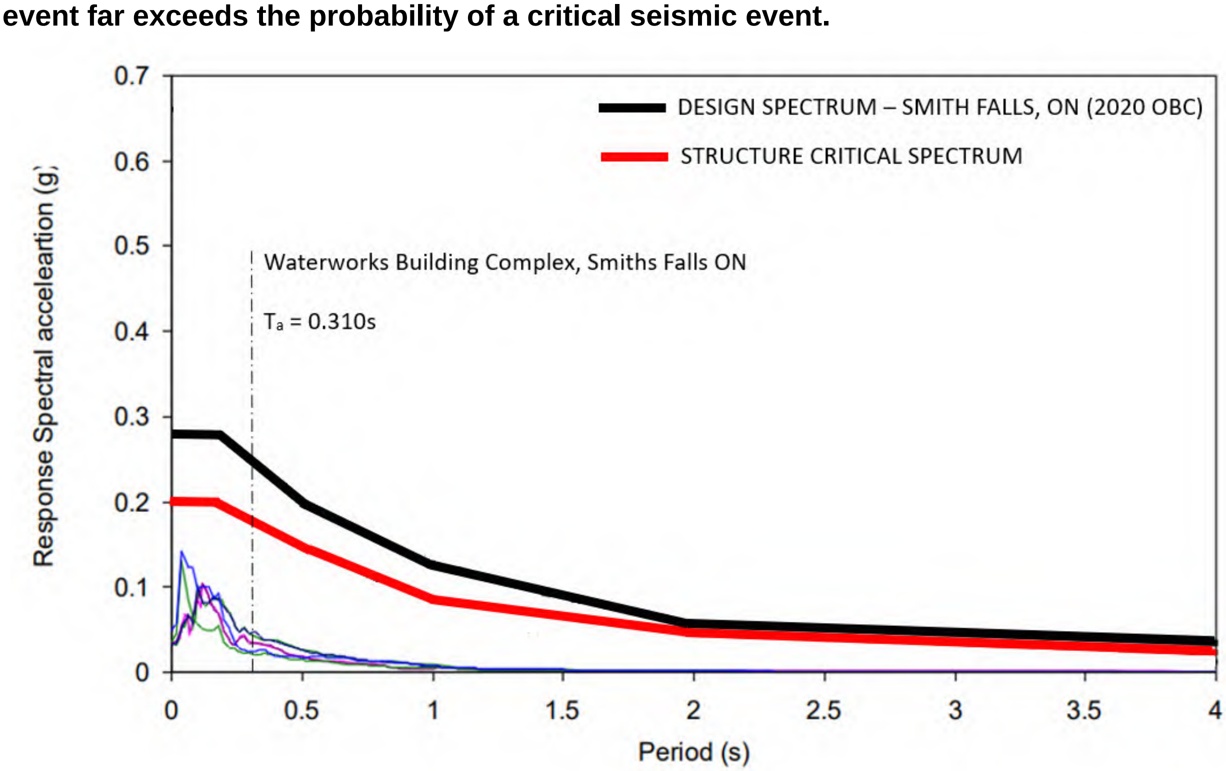


Fig 29. Val-des-Bois Response Spectra v. Design & Critical Spectra.

6.0 DISCUSSION

6.1 Factor of Safety

Values for Factors of Safety (FS) were calculated for the numerous considered failure modes. A Factor of Safety is a ratio of strength or resistance to an applied load. For example, if a beam is twice as strong as what is needed to carry a given load, it would have a Factor of Safety of 2 (i.e.: FS = Resistance / Load). If a beam is not adequate, the FS would be less than 1.0, indicating the applied load is greater than the members resisting strength. An FS value of > 1.0 indicates the wall is stable; < 1.0 the wall is unstable.

A further consideration with respect to the factors of safety is the inclusion of various strength reduction factors and load amplification factors used throughout the analysis. For example, detrimental applied loads such as wind are amplified (i.e. load factor 1.25) whereas the load resisting self-weight of the building is reduced (i.e.: load factor 0.9) as an added measure to provide a conservative result.

This approach improves the reliability of the result and reduces the probability of failure in the event of unforeseen variances or inaccurate assumptions.

6.2 Analysis Results

In consideration of the seismic load, the OBC design event would cause collapse. This load is significantly higher than the highest recorded seismic activity in the region over the past century. The statistical probability of a critical seismic event occurring this year is estimated at 0.04%, whereas the statistical probability of a critical wind event occurring is approximately 2% per annum. While both probabilities are low, the critical wind event has occurred about 14 times in the last decade.

Based on the above, if we neglect a potential collapse due to a seismic event because of the low probability we can focus on the more likely critical wind event occurring.

Therefore, the governing failure mode was determined to be overturning of the upper portion of the stone masonry wall due to horizontal failure caused by wind induced pressure. This failure mode has the lowest calculated FS ranging from 0.8 to 1.0 for the OBC q_{50} wind pressure. The range of FS value is due to the range of reliable data used in the analysis (i.e.: stone and mortar density variations, etc.).

All else remaining equal, based on the data available, our method of analysis, and the validity of our assumptions, it is our structural engineering opinion that the upper portion of the three-storey stone masonry wall has an approximate 20% probability of failure (collapse) during a wind event that exceeds a 1:50 probability of occurring. In the event that lateral bracing is installed, the factors of safety are significantly improved (i.e.: 2.5 < FS < 5.0) where probability of failure begins a downward approach toward zero.

In simple terms, the upper portion of the stone masonry wall could fail in the event of a gale force wind if left unbraced. Winds of this magnitude have a low probability of occurrence, but the risk is not zero. In the event that the walls are braced, the factors of safety are significantly increased, and the probability of failure is substantially reduced.

Therefore, there are options for preservation available for consideration.

Options under consideration would include: do nothing, demolition, or structural stabilization.

6.3 Cost Considerations

The options under consideration include: do nothing, demolition, and structural stabilization & masonry restoration. At this stage, with no design or detailed terms of reference, any estimate of costs should be considered a Class D (Order of Magnitude) estimate that would carry a range of +50% to -25% until the scope of work can be further refined. This level of estimate is valuable for comparing options and determining feasibility from a budgetary standpoint.

An opinion of the cost of implementing these options was obtained from:

Demolition

- Bill Weiske, A1 Demolition, Ancaster, ON.
- Darrell Hiltz, GSC, G. Douglas Vallee Limited, Simcoe, ON.

Heritage Building Specialists

- P.V. Hoad, BA, CAHP, Robertson Restoration Inc., Brantford, ON.
- C. Mayberry, Tacit Brick & Stone, London, ON.

The estimated costs received were based on the same information provided to each individual to help ensure that their opinions would be comparable. Once the terms of reference are more clear, refined cost estimates can be prepared.

Hazardous Materials

There is potential for hazardous materials, such as asbestos, to be present within the buildings. In order to quantify the cost of abatement of hazardous materials, an investigation and report would be required. This was not within the scope of work for this assignment and was not possible at the time of our site visit due to access restrictions.

If the walls are stabilized, and access is made safe, a specialist fire abatement crew is capable of selective removals during demolition in compliance with applicable regulations. If the building is to be demolished, hazardous material remediation becomes more complex as it is difficult to separate and remove from rubble. In the event that hazardous materials cannot be identified and removed prior to demolition, the option to treat all material as hazardous can be incorporated but is costly.

For the purpose of the cost considerations of this assignment, we will assume that the cost of hazardous material abatement for either option is similar. This work would then produce no net difference in the initial Class D cost estimation comparison, but should be considered for the project budget.

Do Nothing

The option to do nothing is always considered but rarely is feasible or responsible. This option comes with arguably the lowest initial financial cost, but only delays the selection of either demolition or structural stabilization. If ‘do nothing’ is selected by the choice of delay or neglect, the availability of structural stabilization and restoration as a viable option will diminish. To do nothing would leave an unabated vacant industrial complex as a significant liability to the Town. This option is not considered feasible or responsible and is not recommended.

Demolition

The demolition option would remove Buildings 2 & 3, while leaving the site with Building 4 remaining as well as the east foundations adjacent to the Rideau River.

The initial scope of work considered for demolition of Building 2 & 3 would include:

- Professional services (Heritage) required to obtain a Cultural Heritage Evaluation Report (CHER), a Cultural Heritage Impact Assessment (CHIA) for the demolition option to review alternatives (i.e. salvage, commemoration, etc.), and full documentation prior to demolition
- Professional services (Ecologist) required to submit a species-at-risk mitigation report to determine requirements related to aquatic and avian species that may be affected by the work.
- Professional services (Engineer) to obtain a demolition report required for demolition permit.
- Installation of waterway protection to prevent debris from entering watercourse.
- Partial demolition of Building 1 to facilitate lay down area and demolition of Building 2.
- Pulling down east face of Buildings 2 & 3 in order to gain access to interior.
- Selective removal of interior materials in order to install debris mitigation and waterway protection.
- Collapse remaining walls into foundation space, remove into bins/trucks for legal disposal.
- Selective demolition and make-good work around interface wall with Building 4 to protect structure that is to remain.

The opinion of cost for this demolition work was approximately $300k with a range of **$450k high to $225k low** with a mean **value of $340k** considering the Class D range of +50% to -25%.

Structural Stabilization & Masonry Restoration

The structural stabilization option would leave the site with Buildings 1 & 4 as-is, with Buildings 2 & 3 secured and stabilized. The load bearing exterior masonry walls of Buildings 2 & 3 would be restored and available for redevelopment.

The initial scope of work considered for the structural stabilization of Buildings 2 & 3 would include:

- Professional services (Ecologist) required to submit a species-at-risk mitigation report to determine requirements related to aquatic and avian species that may be affected by the work.
- Professional services (Engineer) required for restoration and shoring plans necessary to obtain a building permit and Heritage Committee approval.
- Installation of exterior scaffold on all elevations to secure walls and prevent falling debris.
- Secure masonry and cap exposed walls.
- Selective removals of interior fire damage to facilitate interior scaffold, shoring, & bracing.
- Stone masonry restoration & brick masonry restoration.

The opinion of cost for this structural stabilization and masonry restoration work was approximately $400k with a range of **$600k high to $300k low** with a mean **value of $450k** considering the Class D range of +50% to -25%.

Both of the above options do not consider hazardous material abatement, since that scope of work is currently undefined, but is also relatively equivalent to both options. This being the case however, it should be considered for budgeting purposes in addition to the estimates noted above.

7.0 RECOMMENDATIONS

The result of our site visit and subsequent research and analysis is that the Town of Smiths Falls **does** have options that can be considered in addition to demolition. Our analysis indicates that the risk of collapse may not be as certain or immediate as initially thought. While the risk is not zero, as there are environmental events that could cause partial collapse of the stone masonry wall, there is an available option to stabilize and restore the masonry walls.

The option to 'do nothing' is not safe or feasible, and it only delays the decision to demolish. Given the current condition of the building, any significant delays will diminish the opportunity available for restoration. For this reason, to 'do nothing' is therefore not recommended.

The option to **demolish** would require meeting regulatory requirements under the Ontario Heritage Act. This option would improve the safety of the site, though Building 4 would remain. The cost of this option is similar (same order of magnitude) but less than the cost of stabilization and restoration. A meticulous program of selective sympathetic demolition would likely be required. This process would promote the salvage and potential adaptive re-use of heritage features, particularly the hand-hewn stones. This option is feasible and economically responsible in the short term if the **heritage value of the site is not considered**.

The option to stabilize and **restore** the masonry improves the safety of the site while protecting the heritage integrity of the buildings. This option is similar (same order of magnitude) yet understandably more expensive than demolition. If there is a foreseeable vision for the future development of this site and the will to preserve the historic structure, this option is feasible and economically responsible in the long term if the **heritage value of the site is considered**. This option is available on the condition that the decision is made without delay so that work can begin during calmer summer months, and the walls are secured before winter. The potential for adverse weather over the winter months will cause further deterioration and risk.

Based on all of the above considerations and the heritage value of the site, it is our recommendation that the Town consider setting a budget and timeline to commence stabilization and restoration of the 1868 stone masonry and 1886 brick masonry buildings as soon as possible.

CLOSURE

We trust that the Heritage Building Structural Pathology Inspection Engineering Report satisfies the scope of work required by this assignment.

It has been a pleasure to visit and work with the Town of Smiths Falls. We would like to extend our sincere appreciation and thanks to the Town Council for allowing the opportunity to perform this important work, Town staff, the Smiths Falls Police Service, and especially to the Smiths Falls Fire Service for not only the use of the aerial platform vehicle but for their staff and time.

Please do not hesitate to contact the undersigned to discuss any aspect of this Engineering Report.

Respectfully submitted,

LICENSED PROFESSIONAL ENGINEER
June 26 '23
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A. Ryan Elliott, P.Eng., BDS, Consulting Engineer
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G. DOUGLAS VALLEE LIMITED
Consulting Engineers, Architects & Planners

HERITAGE BUILDING STRUCTURAL PATHOLOGY ENGINEERING REPORT

25 OLD MILL ROAD, SMITHS FALLS, ON

APPENDICES

APPENDIX A – EDIFICE Material Report

APPENDIX B – Photographic Log

APPENDIX C – Ottawa International Airport Wind Data

G. DOUGLAS VALLEE LIMITED
Consulting Engineers, Architects & Planners

HERITAGE BUILDING STRUCTURAL PATHOLOGY ENGINEERING REPORT

25 OLD MILL ROAD, SMITHS FALLS, ON

APPENDIX A

EDIFICE Material Report

G. DOUGLAS VALLEE LIMITED
Consulting Engineers, Architects & Planners

Prepared For:
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Consulting Engineers,
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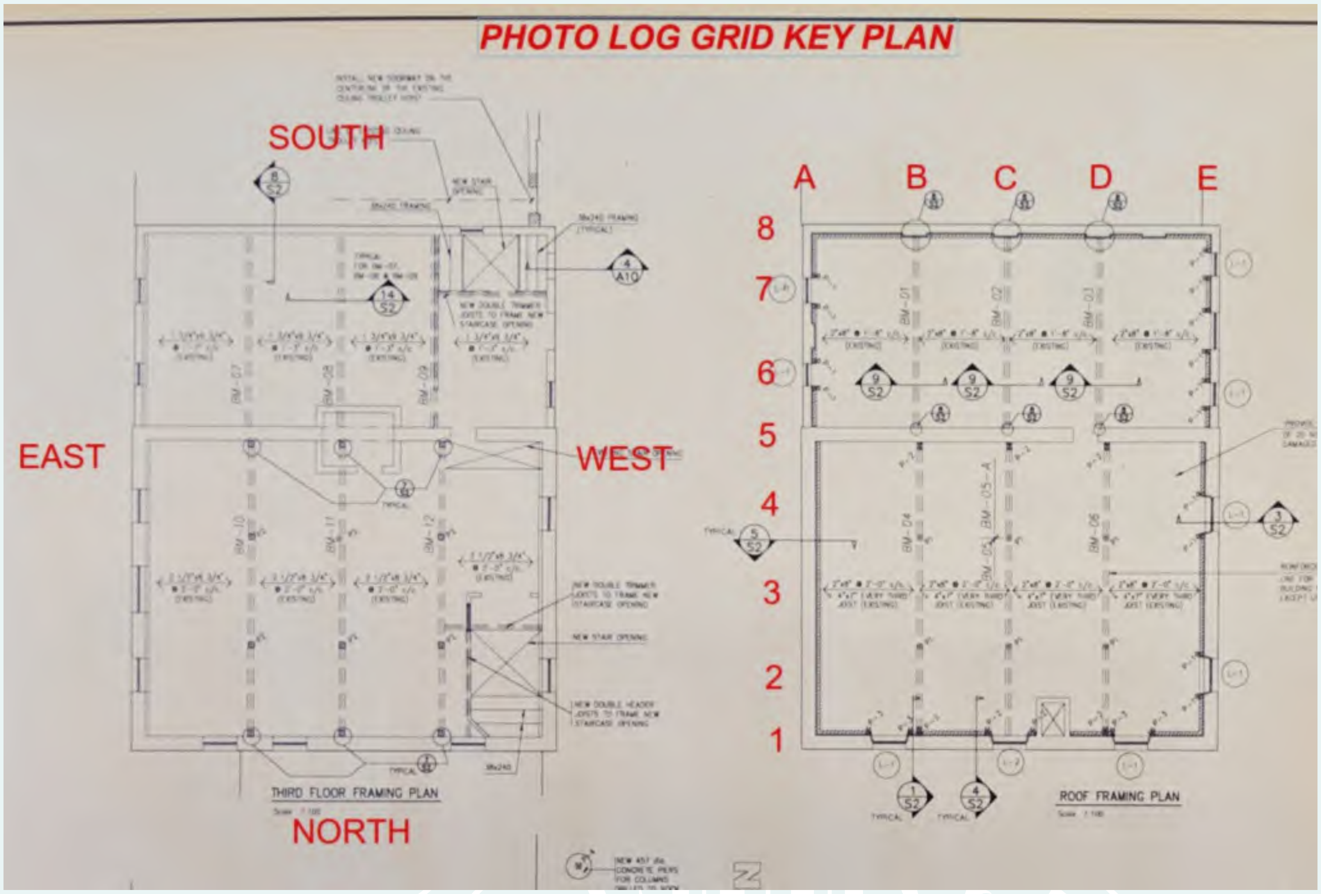
2 Talbot Street North, Simcoe, ON
[redacted]

Report On Materials Analysis

25 Old Mill Road, Smiths Falls

Prepared By Dr. Christopher Cooper

Date: June 09, 2023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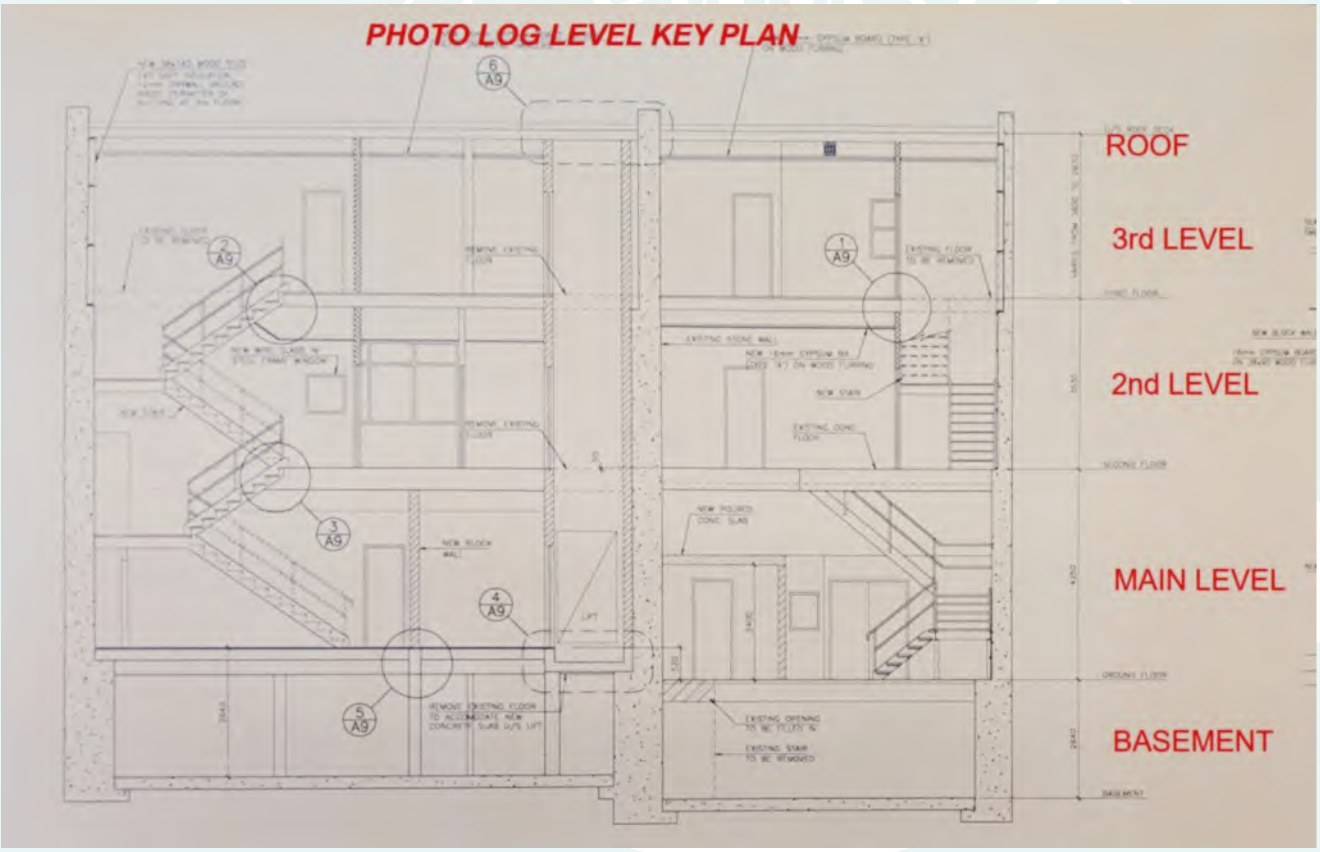
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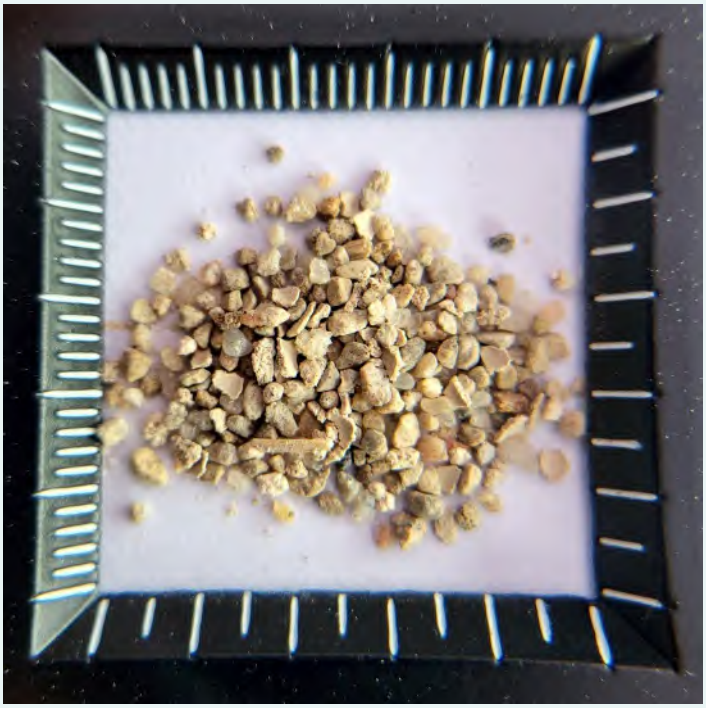


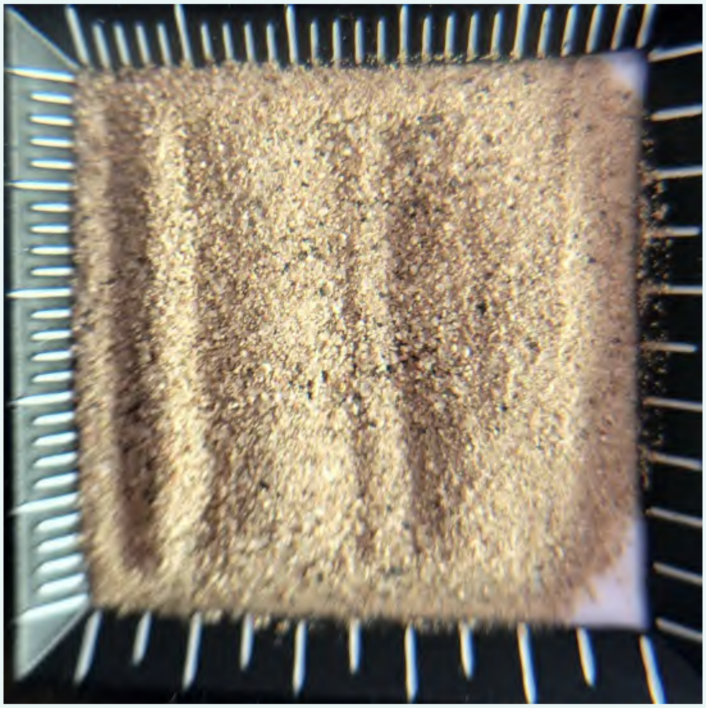
ANALYTICAL PROCEDURES WET ACID DIGESTION TESTS

The purpose of a basic Wet Acid Digestion Mortar Analysis is to determine the approximate proportions of three principal components of historic mortars: aggregate, binder, and fines. Certain additives may also be detected via this method, but their proportions may not be accurately determined. A basic mortar analysis is primarily used to help ascertain general details about composition of a mortar for the purpose of recreating a historic blend or as a prelude to further instrumental analysis. Thus, this test is most useful for identifying whether cement, lime, and sand are present and in what quantities.

This test protocol is useful for distinguishing general characteristics associated with different binders, it is important to note however that the test is subjective. It is based on the interpretation of data and physical properties, rather than unequivocal. Interpretation relies not only on the data produced while testing, but also on observed physical characteristics such as colour, texture, hardness, cohesiveness, and visual properties of aggregate.

Additional clarification on specific properties or additives of a mortar, such as additional pigments, modifying additives, cement type, or mineralogy, would require further instrumental analysis (X-Ray Diffraction, SEM-XEDS, Petrography, and other tests) which can be arranged at a client's request for fees to be determined on a case-by-case basis. It is important to note that testing cannot determine several other important factors in mortar which are difficult or impossible to accurately ascertain (due to a lack of records), including original water mix, mixing and pointing method, rate of drying, or original condition/origin of aggregate.

25 Old Mill Road,
Smiths Falls, Ontario



ANALYTICAL PROCEDURES WET ACID DIGESTION TESTS CONT...

The selected sample of material was dried to a constant weight and examined under a loop at x10 magnification.

Quicklime, also known as calcium oxide (CaO), is a base. It is a strong alkali that has a pH of about 12.3. Both the 1868 sandstone structure and the 1886 red brick structure would have used (in antiquity) a hot mixed lime mortar (Calcium Oxide CaO and Sand) on site in a basic ratio of 3 parts sand to 1 part lime.

An assessment of the binder type was made by evaluating the physical characteristics of the mortar based on our 43 years of knowledge, experience and understanding of lime-based materials.

Application of 45% acetic acid to the sample resulted in dissolution of the binder enabling relative proportions of lime to aggregate to be determined; where appropriate, proportions of insoluble binder were determined and factored into this calculation. Subsequent aggregate characterisation was undertaken by means of dry sieve rated from fines 0.063 mm to 8.00 mm to analysis and microscopic analysis.

The analysis results and interpretations made from it, providing information on the composition and characteristics of the mortar sample(s) received by Edifice. Provided samples were representative of the “Original” mortar generally, the analysis will give a reasonable indication of the original materials and provide a basis for specification of repair mortars. If more detailed information is required (for example, for purposes of historic research) more sophisticated analytical procedures can be undertaken.

The mortar samples provided by VALLEE were categorized into four categories.

ONE: The original carbonated lime mortar on the 1868 Goulds Mills building.

TWO: The 1886 Red Brick Italianate style building. Pigmented carbonated (red iron oxide) and natural carbonated (grey/light brown).

THREE: Black (pigmented) strap Portland cement repair pointing on the 1868 Goulds Mills building. These samples were tested to ascertain the composition of the mortar samples taken. As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1930s) further testing is not required. Notwithstanding this, this type of mortar with a Portland Cement content should never be used on these buildings in any amount.

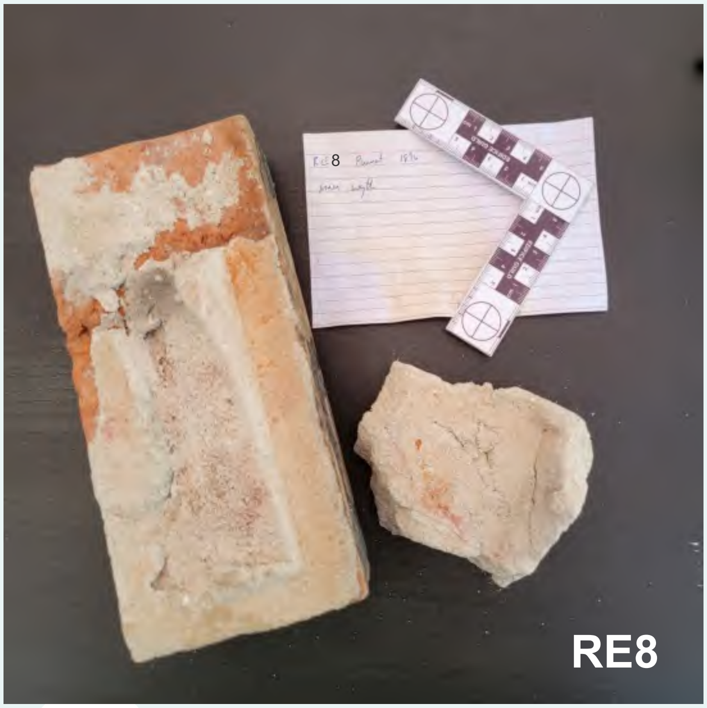
FOUR: Natural colour Portland cement surface repair pointing on both buildings. Portland cement repair pointing on the 1868 Goulds Mills building and the 1886 Red Brick Italianate Structure. These samples were tested to ascertain the composition of the mortar samples taken. As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1950s) further testing is not required. Notwithstanding this, this type of mortar with Portland Cement should never be used on these buildings in any amount.

Original Lime Mortar Bonding Stone & Brick:

The samples were received as fully carbonated intact pieces of mortar plus a high proportion of fines. The strength of the mortar was difficult to accurately assess owing to the size of the received samples and high proportion of fines; however, in relation to sample size, the "original" mortars appear to be generally firm and moderately friable, with the samples disrupted under moderate to heavy pressure. There did not appear (in the samples provided) to be any effect from the fire event to change what would be the expected characteristics of fully carbonated mortar of the period.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 11 mm in length.

* This mortar analysis report is NOT intended as a repair specification. Details of repair specifications based on information from this report should also take account of prevailing site conditions, including stone type and condition, location, and function of the new mortar, building details, exposure, seasonal working etc.



RE8 Inner Sample: Before WADT = 11.3g After WADT = 8.7g

The mix ratio of the sample is approximately 23.01% moderately hydraulic quicklime to 76.99% aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively low proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 1.98 mm in length. This sample indicates a very highly graded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced moderate water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 52.36% by weight. The mortar is very well graded. Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts. This appears to be the same vernacular aggregate as the 1868 portion – simply well graded and sieved prior to mixing.

RE8 Inner Mortar Sample Specifications:

Weight = 1.651 grams per cubic cm

Compressive Strength = 1.543 MPa

* **Flexural Strength** = 0.122 MPa

* Samples very small Inconclusive

Mortar Bed Height = Inconclusive field inspection required for averages

Range of Mortar Height intact on samples = 12.32 mm to 18.55 mm (inconclusive)

25 Old Mill Road,
Smiths Falls, Ontario

RE8 Ext. Sample Red: Before WADT = 14.7g Post WADT = 11.2g

The mix ratio of the sample is approximately 23.81% moderately hydraulic quicklime to 76.19% aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight

The sample contains generally indiscernible very fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, clay inclusions, coal fragments and a relatively low proportion of lime inclusions (unburnt lime clinkers). This sample indicates a very well contained sub-angular to rounded graded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced extremely fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 3mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 54.5%. The Munsell Soil Colour Charts could not be used on this sample due to the red iron oxide colouring of the mortar. This appears to be the same vernacular aggregate as the 1868 portion, simply well graded and sieved prior to mixing.

RE8 Outer Mortar Sample Specifications:

Weight = 1.731 grams per cubic cm

Compressive Strength = 1.682 MPa

***Flexural Strength** = 0.135 MPa

* Samples very small Inconclusive

Range of Mortar Height intact on samples = 12.32 mm to 18.55 mm

*Note: The Exterior Bricks sample (tinted red mortar) were very well bedded in mortar including the frogs (the indentation in the centre of the brick) filled. However, the interior brick wythes showed signs of less desirable bedding and partial frog filling.

RE2 Sample Black: Before WADT = 9.7g Inconclusive Portland Cement

Black (pigmented) strap Portland cement repair pointing on the 1868 Goulds Mills building. These samples were tested to ascertain the composition of the mortar samples taken. They were found (an approximation through magnification) to be 1:1:2 (1 Lime, 1 Portland, 2 Sand). As this mortar does not make up any of the “Original” bedding or head mortar material and only used as a surficial repair solution in antiquity (c1930s) further testing is not required. Notwithstanding this, this type of mortar with a Portland Cement content should never be used on these buildings in any amount.

As this re-pointing is not an integral part of the make up of the two structures in question, and the Wet Acid Digestion Test obtained inconclusive results in exacting measurement of Portland Cement, Aggregate and Lime we can only provide a result of standard mixes during that time period and microscopic observation. Further testing with an electron microscope can be made, mapping and charting the grains of cement to aggregate. However, this mortar has no effect on the existing structures whatsoever, other than the possibility of a negative effect on spalling both stone and brick due to its (the mortar) inability to be permeable.

RC4 Sample Black: Before WADT = 13.1g Inconclusive Portland Cement

Black (pigmented) strap Portland cement repair pointing on the 1868 Goulds Mills building. These samples were tested to ascertain the composition of the mortar samples taken. They were found (an approximation through magnification) to be 1:1:2 (1 Lime, 1 Portland, 2 Sand). As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1930s) further testing is not required. Notwithstanding this, this type of mortar with a Portland Cement content should never be used on these buildings in any amount.

As this re-pointing is not an integral part of the make up of the two structures in question, and the Wet Acid Digestion Test obtained inconclusive results in exacting measurement of Portland Cement, Aggregate and Lime we can only provide a result of standard mixes during that time period and microscopic observation. Further testing with an electron microscope can be made, mapping and charting the grains of cement to aggregate. However, this mortar has no effect on the existing structures whatsoever, other than the possibility of a negative effect on spalling both stone and brick due to its (the mortar) inability to be permeable.

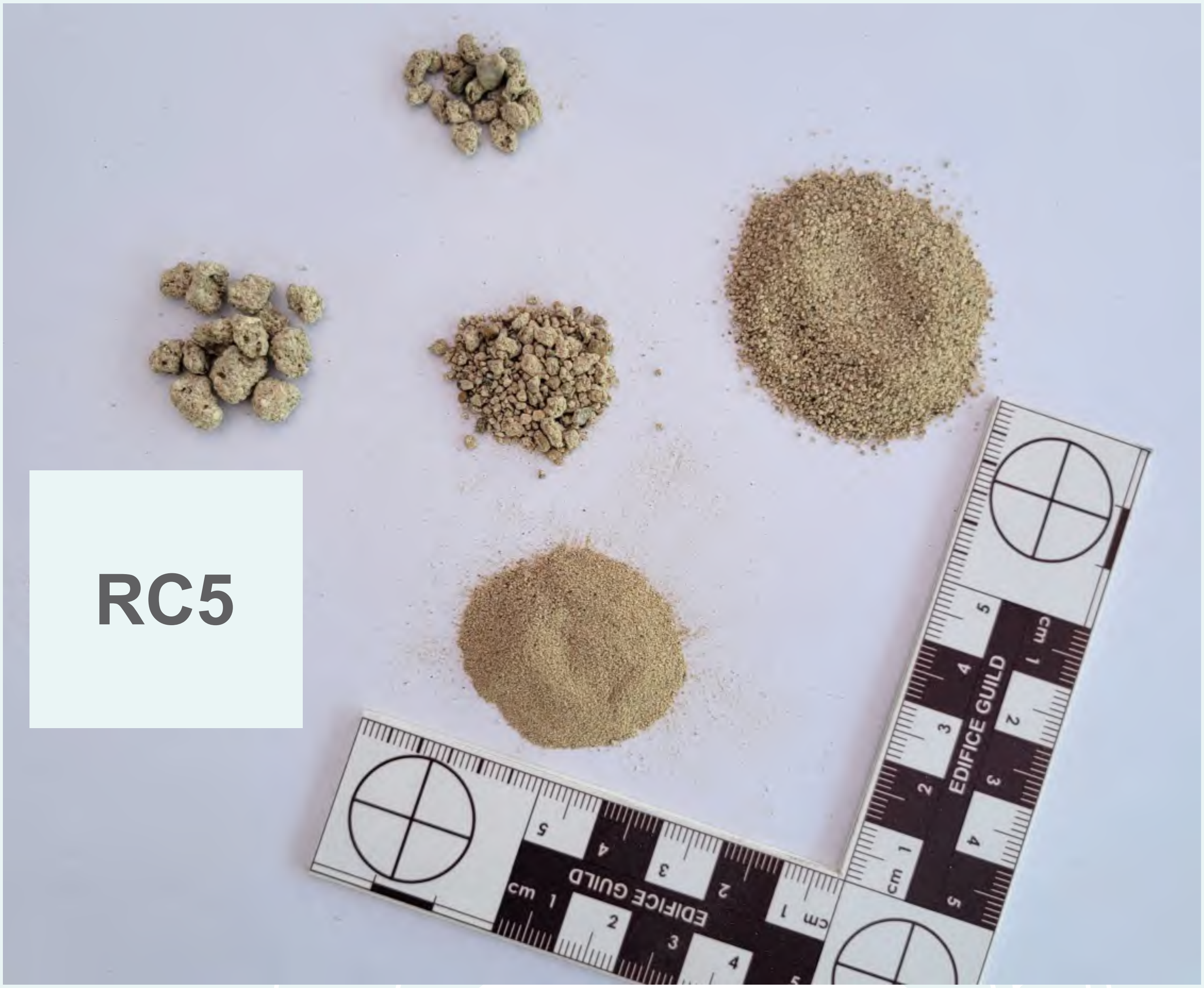
RC5 Sample: Before WADT = 15.6g Post WADT = 11.8g

The mix ratio of the sample is approximately 24.36% moderately hydraulic quicklime to 75.64% aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 9.25 mm in length. This sample indicates a very ungraded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 31.5% by weight. The mortar is very ungraded, Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts.



RC5 Mortar Sample Specifications:

Weight = 1.538 grams per cubic cm

Compressive **Strength** = 1.737 MPa

* **Flexural Strength** = 0.168 MPa

* Samples very small Inconclusive

Mortar Bed Height = Inconclusive field inspection required for averages

Range of Mortar Height intact on samples = 17 mm to 55 mm (inconclusive)

RD1 Sample Black: Before WADT = 12.5g Inconclusive Portland Cement

Black (pigmented) strap Portland cement repair pointing on the 1868 Goulds Mills building. These samples were tested to ascertain the composition of the mortar samples taken. They were found (an approximation through magnification) to be 1:1:2 (1 Lime, 1 Portland, 2 Sand). As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1930s) further testing is not required. Notwithstanding this, this type of mortar with a Portland Cement content should never be used on these buildings in any amount.

As this re-pointing is not an integral part of the make up of the two structures in question, and the Wet Acid Digestion Test obtained inconclusive results in exacting measurement of Portland Cement, Aggregate and Lime we can only provide a result of standard mixes during that time period and microscopic observation. Further testing with an electron microscope can be made, mapping and charting the grains of cement to aggregate. However, this mortar has no effect on the existing structures whatsoever, other than the possibility of a negative effect on spalling both stone and brick due to its (the mortar) inability to be permeable.

RA2 Sample: Before WADT = 16.4g Post WADT = 11.9g

The mix ratio of the sample is approximately 27.44 % moderately hydraulic quicklime to 72.56 % aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 11.35 mm in length. This sample indicates a very ungraded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 39.13% by weight. The mortar is very ungraded. Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts.

RA2 Mortar Sample Specifications:

Weight = 1.537 grams per cubic cm

Compressive Strength = 1.791 MPa

* **Flexural Strength** = 0.17 MPa

* Samples very small Inconclusive

Mortar Bed Height = Inconclusive field inspection required for averages

Range of Mortar Height intact on samples = 17 mm to 55 mm (inconclusive)

2E6.5 Sample: Before WADT = 7.5g Post WADT = 4.6g

The mix ratio of the sample is approximately 38.67 % moderately hydraulic quicklime to 61.33 % aggregate (by weight). Approximately 1 Part Lime to 2.5 Parts Aggregate by Weight.

The sample contains generally indiscernible very fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, clay inclusions, coal fragments and a relatively low proportion of lime inclusions (unburnt lime clinkers). This sample indicates a very well contained sub-angular to rounded graded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced extremely fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 53.2% by weight. The Munsell Soil Colour Charts could not be used on this sample due to the red iron oxide colouring of the mortar. This appears to be the same vernacular aggregate as the 1868 portion, simply well graded and sieved prior to mixing.

2E6.5 Outer Mortar Sample Specifications:

Weight = N/A

Compressive Strength = N/A

Flexural Strength = N/A

Range of Mortar Height intact on samples = N/A

Not enough sample to perform tests see RE8 Ext. Sample Red for Specifications.

*Note: The Exterior Bricks sample (tinted red mortar) were very well bedded in mortar including the frogs (the indentation in the centre of the brick) filled. However, the interior brick wythes showed signs of less desirable bedding and

2B1 Sample: Before WADT = 7.2g Post WADT = 5.2g

The mix ratio of the sample is approximately 27.78 % moderately hydraulic quicklime to 72.22 % aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 4.2 mm in length. This sample indicates a very ungraded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced fast water absorption rates when subjected to the water droplet test [1]. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 53.7% by weight. The mortar is very ungraded. Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts.

2B1 Mortar Sample Specifications:

Weight = 1.916 grams per cubic cm

Compressive Strength = 1.983 MPa

* **Flexural Strength** = 0.187 MPa

* Samples very small Inconclusive

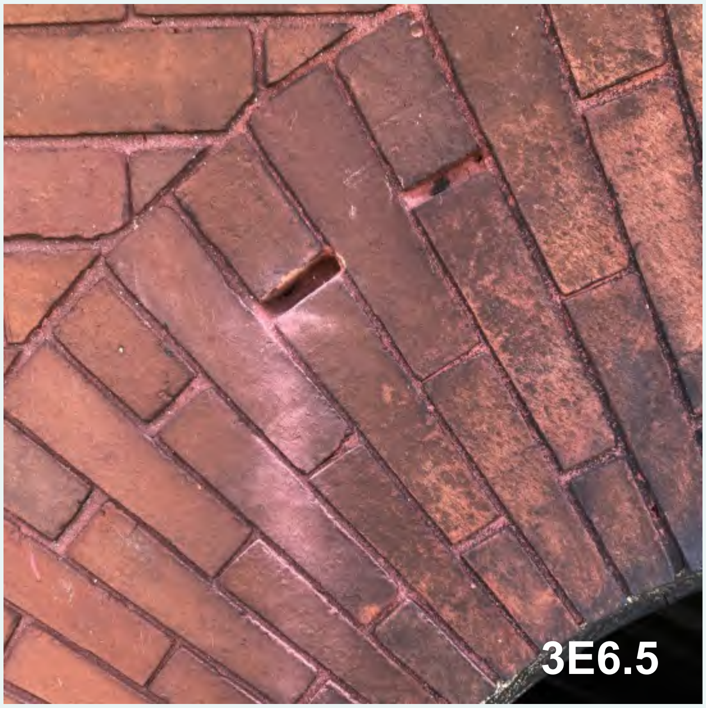
Mortar Bed Height = Inconclusive field inspection required for averages

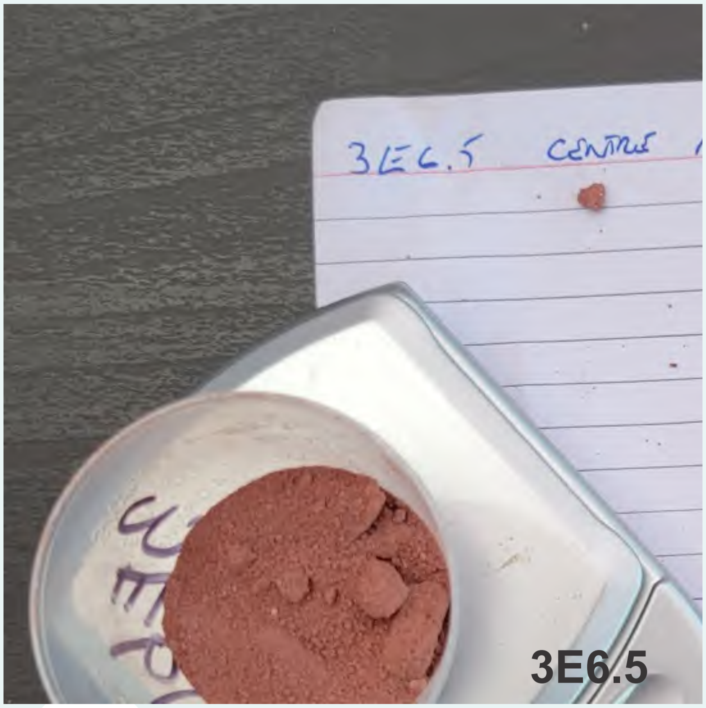
Range of Mortar Height intact on samples = 17 mm to 55 mm (inconclusive)

3B1 Sample: Before WADT = 9.3g Inconclusive Portland Cement

Natural colour Portland cement surface repair pointing on both buildings. Portland cement repair pointing on the 1868 Goulds Mills building and the 1886 Red Brick Italianate Structure. These samples were tested to ascertain the composition of the mortar samples taken. They were found to be 1:3 (1 Portland, 3 Sand). As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1950s) further testing is not required. Notwithstanding this, this type of mortar with Portland Cement should never be used on these buildings in any amount.

Again, this re-pointing is not an integral part of the make up of the two structures in question, and the Wet Acid Digestion Test obtained inconclusive results in exacting measurement of Portland Cement to Aggregate we can only provide a result of standard mixes during that time period and microscopic observation. Further testing with an electron microscope can be made, mapping and charting the grains of cement to aggregate. However, this mortar has no effect on the existing structures whatsoever, other than the possibility of a negative effect on spalling both stone and brick due to its (mortar) inability to be permeable.





3E6.5 Sample Red: Before WADT = 12.2g Post WADT = 9.7g

The mix ratio of the sample is approximately 20.50% moderately hydraulic quicklime to 79.50% aggregate (by weight). Approximately 1 Part Lime to 3.25 Parts Aggregate by Weight. As the colourant is red iron oxide the weight can be skewed.

The sample contains generally indiscernible very fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, clay inclusions, coal fragments and a relatively low proportion of lime inclusions (unburnt lime clinkers). This sample indicates a very well contained sub-angular to rounded graded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced extremely fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 48.1% by weight. The Munsell Soil Colour Charts could not be used on this sample due to the red iron oxide colouring of the mortar. This appears to be the same vernacular aggregate as the 1868 portion, simply well graded and sieved prior to mixing.

25 Old Mill Road,
Smiths Falls, Ontario

3E6.5 Outer Mortar Sample Specifications:

Weight = N/A

Compressive Strength = N/A

Flexural Strength = N/A

Range of Mortar Height intact on samples = N/A

Not enough sample to perform tests see RE8 Ext. Sample Red for Specifications.

3E2 Sample: Before WADT = 8.1g Post WADT = 6.0g

The mix ratio of the sample is approximately 25.93 % moderately hydraulic quicklime to 74.07 % aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 5.5 mm in length. This sample indicates a very ungraded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 51.5% by weight. The mortar is very ungraded. Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts.

25 Old Mill Road,
Smiths Falls, Ontario

3E2 Mortar Sample Specifications:

Weight = 1.858 grams per cubic cm

Compressive Strength = 2.003 MPa

* **Flexural Strength** = 0.198 MPa

* Samples very small Inconclusive

Mortar Bed Height = Inconclusive field inspection required for averages

Range of Mortar Height intact on samples = 17 mm to 55 mm (inconclusive)

BE5 Sample Lime: Before WADT = 11.4g Post WADT = 8.5g

The mix ratio of the sample is approximately 25.44 % moderately hydraulic quicklime to 74.56 % aggregate (by weight). Approximately 1 Part Lime to 3 Parts Aggregate by Weight.

The sample contains generally indiscernible fine-grained aggregate that includes quartz grains, weathered crystalline lithic fragments, sedimentary stone grains, orange to brown coloured clay inclusions, coal fragments and a relatively high proportion of lime inclusions (unburnt lime clinkers), with the largest measuring ~ 5.5 mm in length. This sample indicates a very ungraded sub-angular to rounded vernacular aggregate.

The fines experienced a moderate water absorption rate, while fully intact sub-samples of mortar experienced fast water absorption rates when subjected to the water droplet test. This fast water absorption rate within the intact sub-samples indicates an interconnected internal pore network that permits the fast and efficient absorption and transportation of moisture through the thickness of the mortar.

This sand has been preserved from a sieve mesh 4mm to 0.075 mm, with the highest percentage of aggregate having been retrieved from sieve mesh size 0.75mm with 51.5% by weight. The mortar is very ungraded. Once dried, the mortar was found to be 10YR 7/3 'very pale brown' to 10YR 6/6 'pale brownish Yellow' when assessed against the Munsell Soil Colour Charts.

BE5 Mortar Sample Specifications:

Weight = N/A

Compressive Strength = N/A

Flexural Strength = N/A

Range of Mortar Height intact on samples = N/A

Not enough sample to perform tests.

BE5 Sample Black: Before WADT = 15.15g Inconclusive Portland Cement

Black (pigmented) strap Portland cement repair pointing on the 1868 Goulds Mills building. These samples were tested to ascertain the composition of the mortar samples taken. They were found (an approximation through magnification) to be 1:1:2 (1 Lime, 1 Portland, 2 Sand). As this mortar does not make up any of the "Original" bedding or head mortar material and only used as a surficial repair solution in antiquity (c1930s) further testing is not required. Notwithstanding this, this type of mortar with a Portland Cement content should never be used on these buildings in any amount.

As this re-pointing is not an integral part of the make up of the two structures in question, and the Wet Acid Digestion Test obtained inconclusive results in exacting measurement of Portland Cement, Aggregate and Lime we can only provide a result of standard mixes during that time period and microscopic observation. Further testing with an electron microscope can be made, mapping and charting the grains of cement to aggregate. However, this mortar has no effect on the existing structures whatsoever, other than the possibility of a negative effect on spalling both stone and brick due to its (the mortar) inability to be permeable.

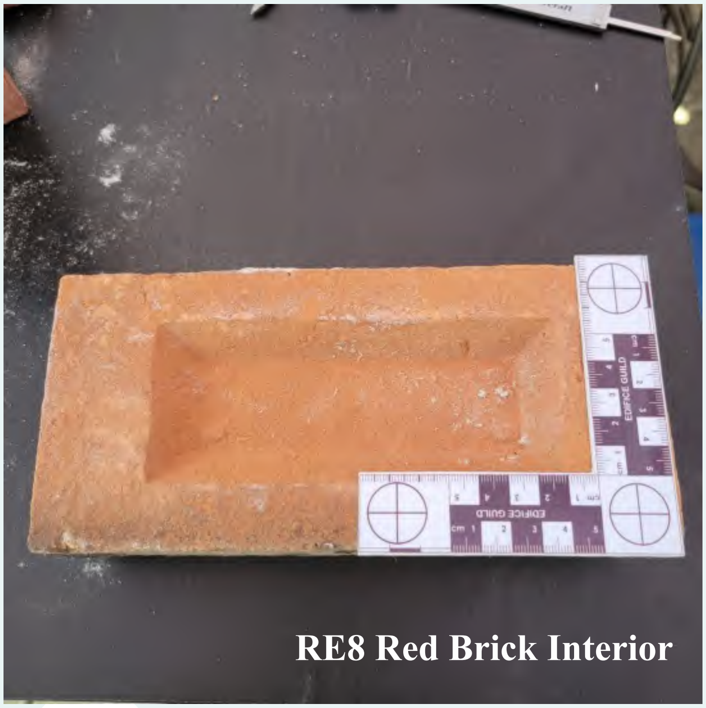
1886 Unit Brick Analysis Compressive Strength

ASTM C 67 requires that the specimen be full height and width, and approximately one-half of a brick in length, plus or minus 25 mm. For example, a 200 mm long brick may be tested using a piece of brick with a length between 75 and 125 mm.

The requirement in ASTM C 67 that the specimen be centered under the spherical upper bearing block within 1.6 mm is not a capricious one. The introduction of an eccentric load, if the specimen is not carefully centered, can result in a lower apparent compressive strength for the test specimen. It should be understood, however, that this requirement assumes that the specimen is symmetrical about both horizontal axes and its center of gravity. For symmetrical specimens, the center of gravity will be the geometrical center of the unit.

The two sample bricks for these tests were dried and cut in half. Half of each brick were submerged in water to test absorption rate and “wet compression test”. A 20-tonne press was used with flatbed appliances the same size as the bricks (both wet and dry) producing a uniform loading over the cross section of the specimens. The compressive strength was obtained by dividing the maximum load of the press by the width and depth of the brick.


RE8 Red Brick Interior



RE8 Red Brick Interior

RE8 Red Brick Sample Interior:

Weight = 2,185 grams

Length = 220.31 mm

Width = 100.23 mm

Height = 64.44 mm

Frog = 58.22 mm width x 144.48 mm length x 11.54 mm depth, angled at 44.2 degrees off face. It appears through photographic evidence all Frogs are down.

Absorption rate 48 hours = 20.23%

Half Brick Compressive **Strength Wet** = 11.996 MPa

Half Brick Compressive **Strength Dry** = 14.547 MPa


RE8 Red Brick Exterior


RE8 Red Brick Exterior

RE8 Red Brick Sample Exterior:

Weight = 2,112 grams

Length = 218.59 mm

Width = 99.102 mm

Height = 64.55 mm

Frog = 58.31 mm width x 143.96 mm length x 11.512 mm depth, angled at 44.56 degrees off face. It appears through photographic evidence all Frogs are down.

Absorption rate 48 hours = 23.45%

Half Brick Compressive Strength Wet = 12.785 MPa

Half Brick Compressive Strength Dry = 15.207 MPa

Nepean Sandstone

Quoted Research: [2] "The colour change in sandstones corresponds with the dehydration of iron compounds, as noted in the samples provided. This requires a temperature of 250-300 °C. Brown or buff coloured sandstone changes colour to reddish brown but the change may not be apparent until the stone has been heated to temperatures above 400 °C. This colour change can persist after heating to 1000 °C."

Notwithstanding this, the samples provided including ones (heavily scorched) showed reddish brown colour change from 2 to 15 mm deep from the face.

[2] "Heating sandstones to greater than 573 °C usually causes internal rupturing of the quartz grains thus leading to weakening of the stone which tends to become friable."

In the samples provided we did not find this to be the case (not able to crumble the samples by hand), the stone would only successfully fracture with a 2-pound mash hammer.

[2] Effects of fire damage on natural stonework in buildings. Birbhushan Charkrabarti, Tim Yates and Andrew Lewry 6.10.1995.

The Sandstone samples were tested in the same manner as the bricks, however, all Sandstone samples were cut to an approximate size of 30 mm x 30 mm x 30 mm. A 20-tonne press was used with flatbed appliances the same size as the sandstone specimens producing a uniform loading over the cross section of the specimens. The compressive strength was obtained by dividing the maximum load of the press by the width and depth of the sandstone specimens.

As we did not have large enough specimens for mortar and sandstone flexural strength testing the results should be considered inaccurate. Typically, flexural strengths is gathered in the same manner as compressive strength, except the specimens are bridged on two supporting points from below the sample with pressure (from the press) placed on the centre of the specimen from above. Specimens should always be regular in size and shape preferably rectangular. Edifice, does not possess the necessary heavy equipment to cut a hard sandstone sample for testing (a large rectangular bar of specimen material is required). Most flexural testing will require at least three specimens of uniform size and shape. Therefore, flexural testing noted in this report were taken with small and very irregular size and shaped specimens.

Three 30 mm x 30mm x 30mm samples were cut and polished from samples at RC5 (two sample areas) and MA2. All three samples reacted similarly with a difference of plus or minus 0.10 MPa we have noted on preceding pages the minimum compressive strength gathered from the three specimen sample.

Sandstone Sample:

Average Weight = 2.141 grams per cubic cm

*[1] **Compressive Strength** = 39.965 MPa

*[1] **Crack formed** (inelastic fracture)

*[2] **Flexural Strength** = 9.376 MPa (inelastic fracture)

*[2] Not enough sample to make accurate hypothesise

A homogenous, small particle material (the subject sandstone) will exhibit the most regular fracturing or inelasticity, and will form very sharp edges.

Note: Elasticity vs Inelasticity

The prominent factor affecting the fracture mechanics of rocks is their elasticity. Elasticity refers to the tendency of a material to rebound from the effects of a force applied to it. Inelasticity is the inverse of elasticity.

Wet Acid Digestion Test on Sandstone Sample:

A Wet Acid Digestion Test was performed on a small 12.1 g (as weighed) sample of sandstone collected from the building at RC5. The sample was subjected to a Wet Acid Digestion Test utilizing 45% acetic acid for 48 hours. The sample was dried and weighed. The resulting weight was 11.80 g which is to be expected from sandstone. The 0.30 g difference can be attributed to surface fines, which were present in the specimen receptacle. Therefore, all samples provide by VALLEE from the 1868 Goulds Mills building appear to be Sandstone.

NOTE: Many times, during an analysis of limestone, we will perform the same Wet Acid Digestion Test to confirm the samples are indeed limestone. Most limestone samples will lose up to 15 - 25% in total weight when subjected to acetic acid for a period of 48 hours. Therefore, all samples provide by VALLEE from the 1868 Goulds Mills building appear to be Sandstone.

END THIS REPORT

HERITAGE BUILDING STRUCTURAL PATHOLOGY ENGINEERING REPORT

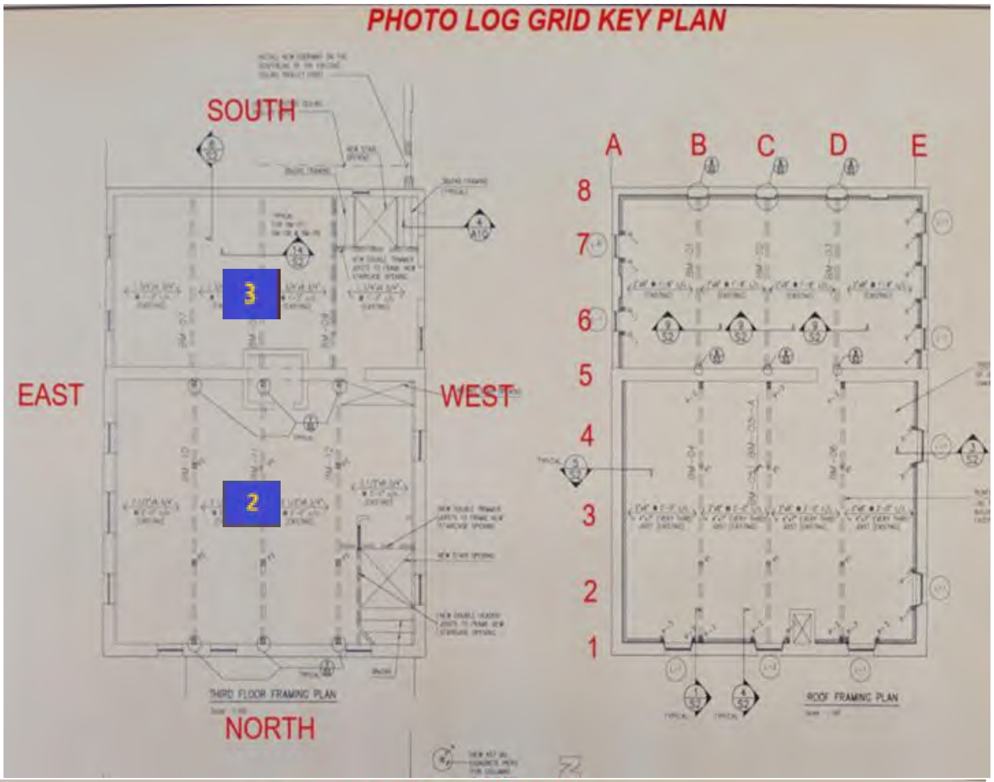
25 OLD MILL ROAD, SMITHS FALLS, ON

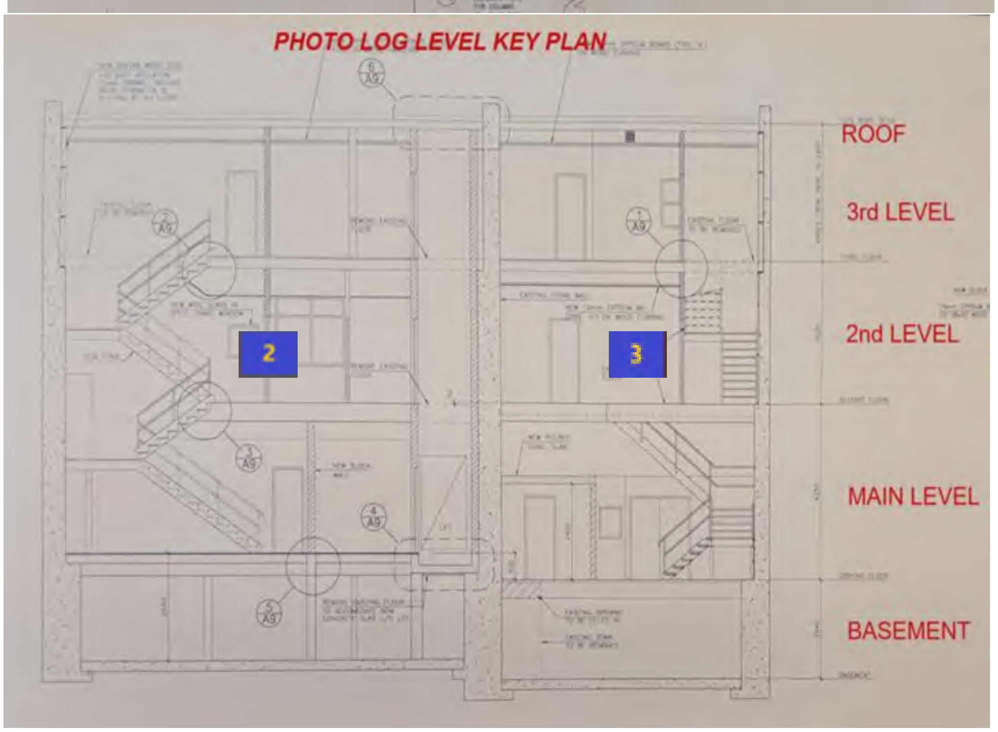
APPENDIX B

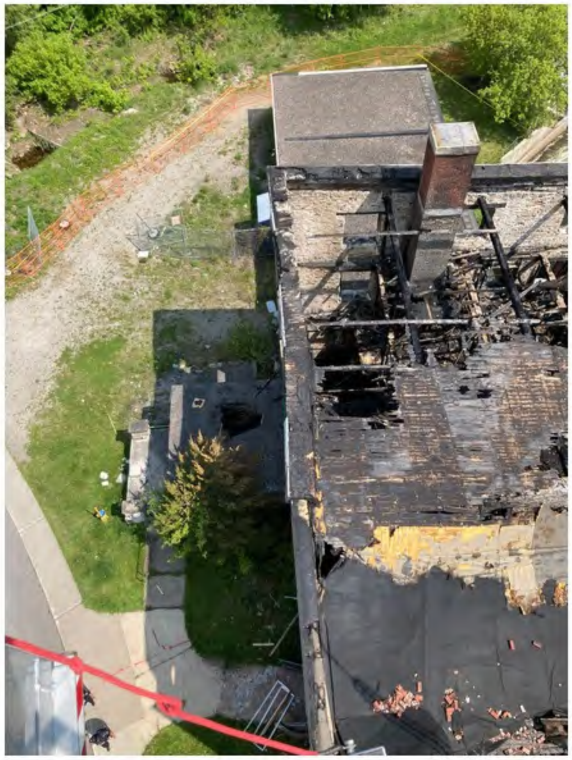
Photographic Log

G. DOUGLAS VALLEE LIMITED
Consulting Engineers, Architects & Planners

The log of photographs found in the appendix are referenced by their location on Buildings 2 & 3 in accordance with the following grid and level key plan:







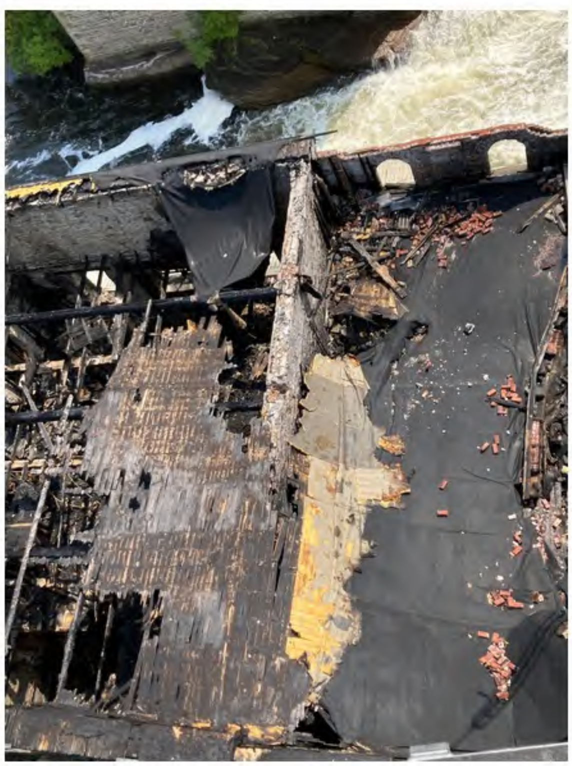
Aerial E1-E8

Aerial View 1

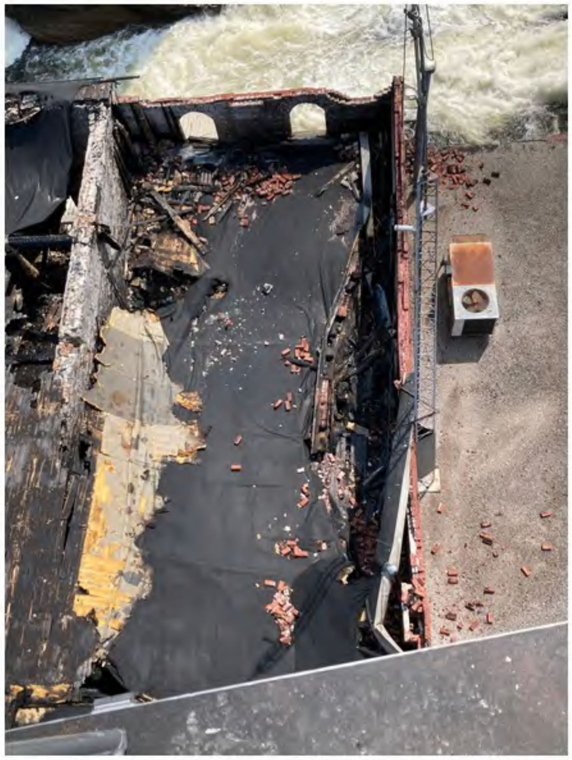
Aerial View 3

Aerial View 4

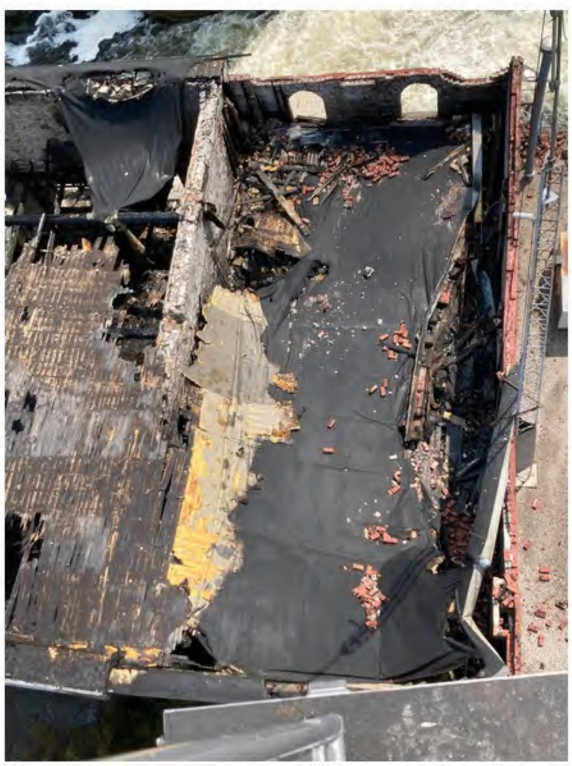
Aerial View 5



Aerial View 6



Aerial View 7



Aerial View 8

Aerial View 9

Aerial View 1868 Stone Bdg Roof

Aerial View 1896 Brick Bdg Roof

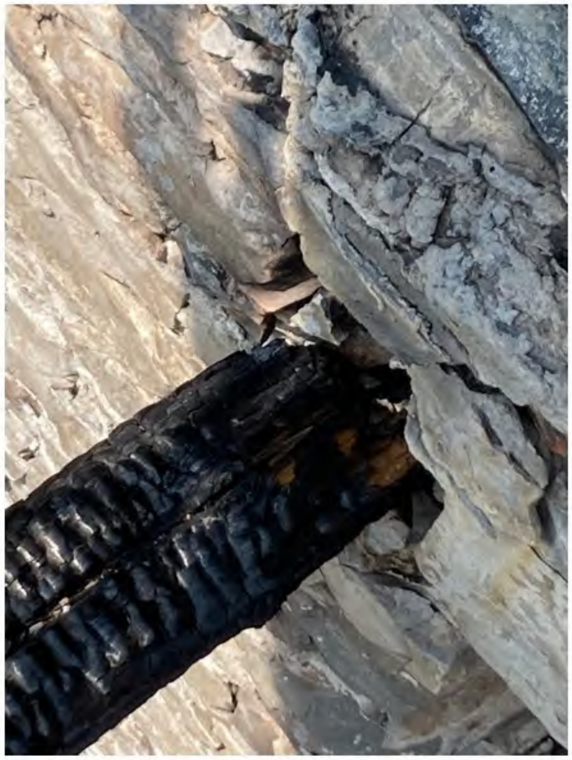
Aerial View 1924 Bdg Roof

RA4-RA5 roof membrane

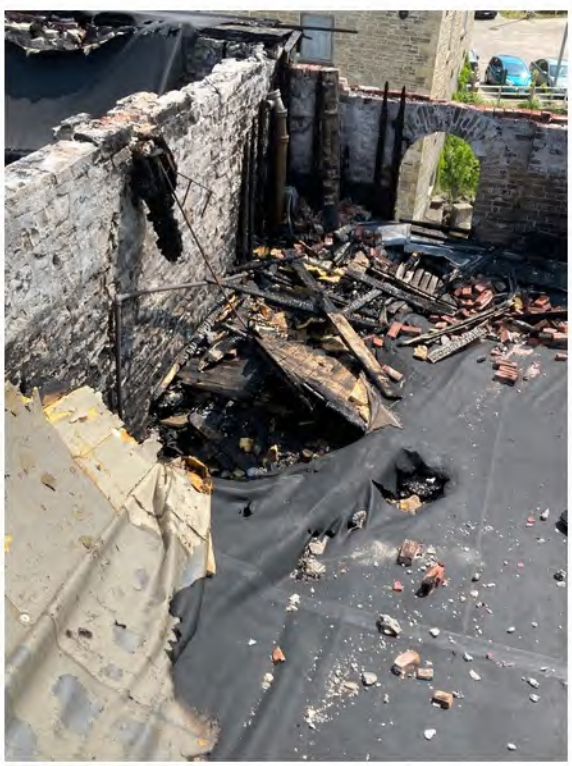
RA4-RB4 roof membrane and 3A5-3B5 parging

RA5 to RE8 Panoramic

RAB1 exterior

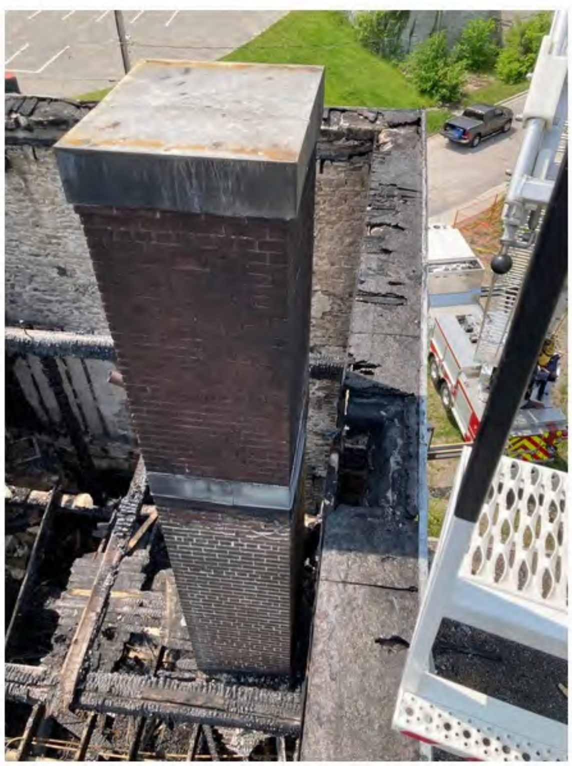


RB1 beam pocket



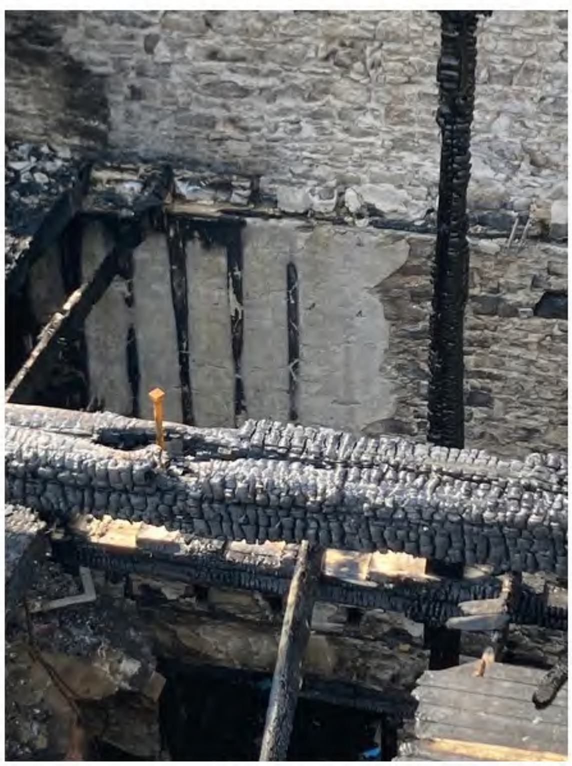
RB5-RA6

RC1 chimney



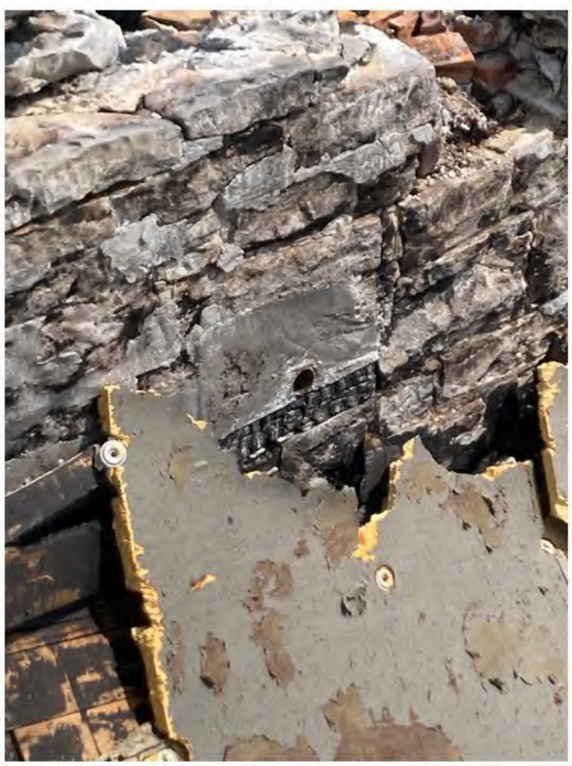
RC1 East face of chimney

RC1-RB1 to 2C1-2B1 interior

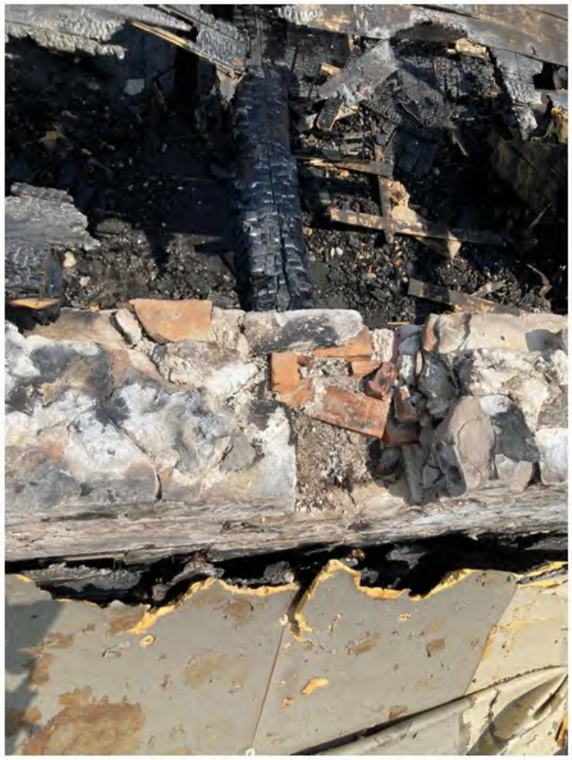


RC2 interior

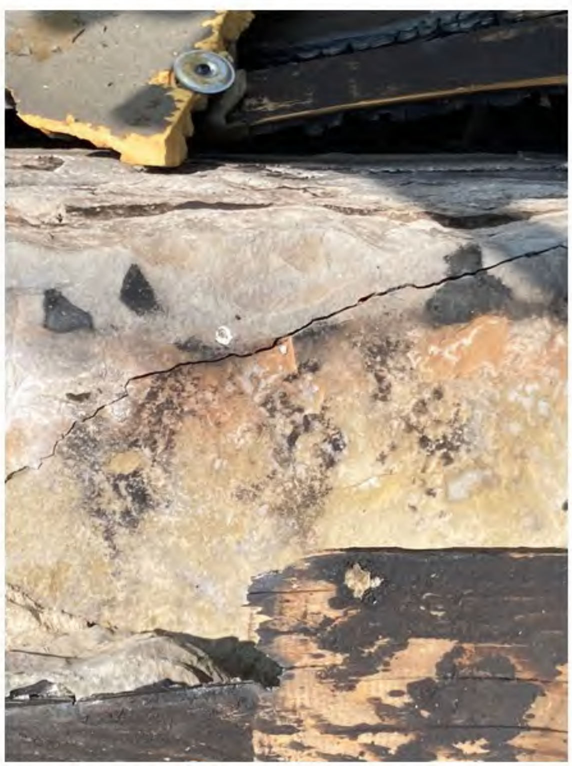
RC2 spike in timber



RC5 1868 exterior 1896 interior



RC5 Parapet sample location



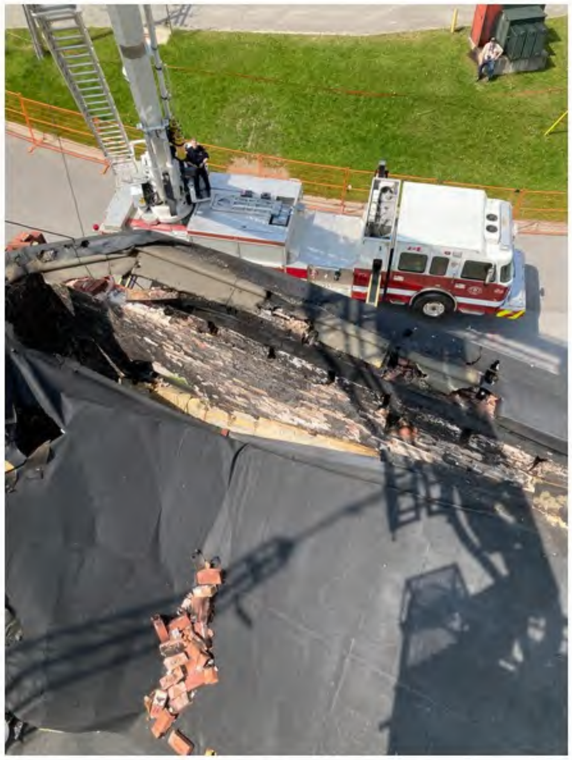
RC5 Parapet sandstone fissure

RC5 Parapet sandstone sample location

RC5 Parapet

RC5 Timber roof beam

RC5-RA5 1868 exterior side



RE&-RE8 parapet interior

RE1 exterior

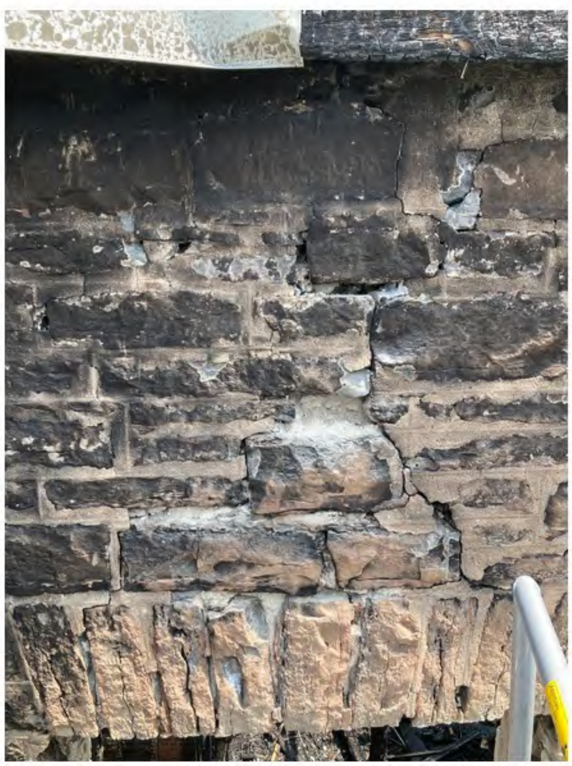
RE1 pyramidal crack

RE1-RA1 to 2E1-2A1 interior

RE1-RD1 to 2E2-2D2 interior

RE2 mortar powder

RE2 mortar sample location



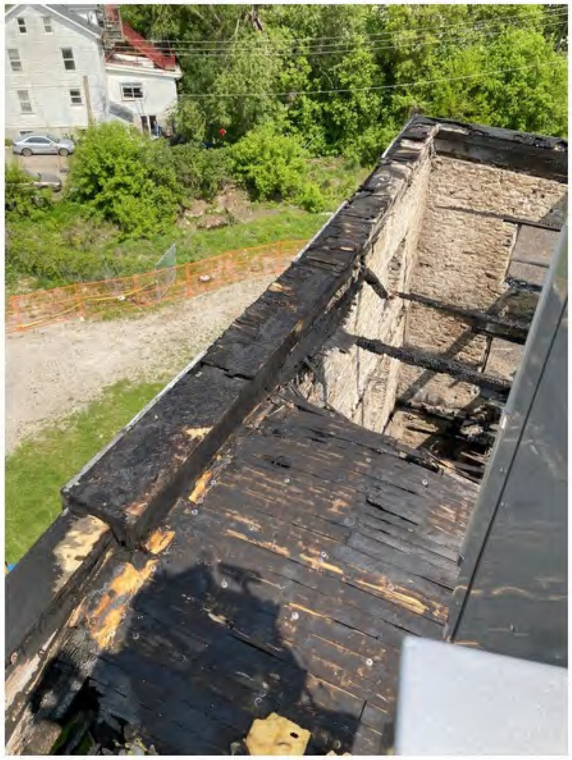
RE2 pyramidal crack

RE4 above window

RE4 mortar sample locn

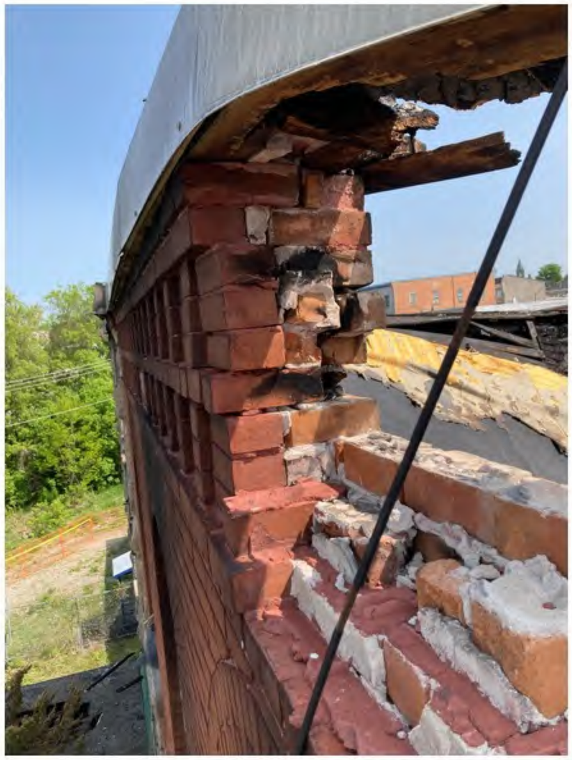
RE4 to RA5 Panoramic

RE5 exterior



RE5-RE1 aerial interior

RE8 Brick Parapet sample locn



RE8 Brick Parapet section

RE8 Brick Parapet

West face 2E2-RE2

West face 2E3-RE3

3A1 interior corner

3A1 interior face looking down

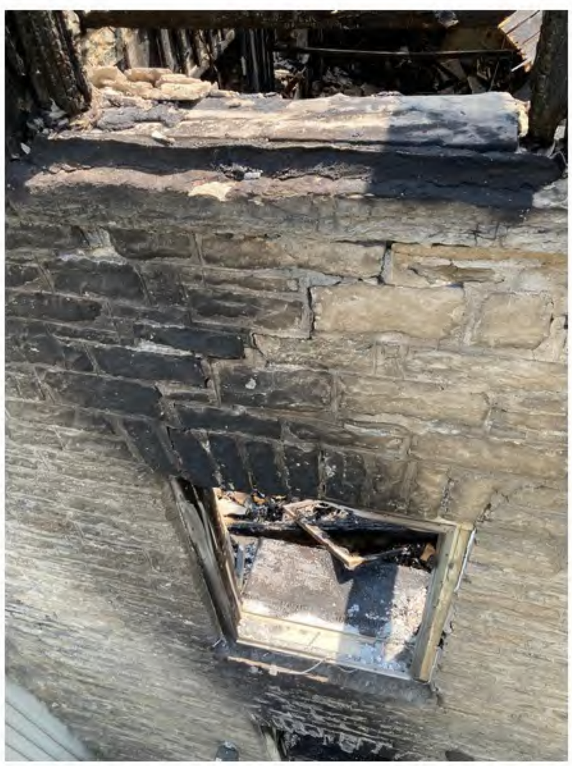
3A1 spalled out-of-bed sandstone Sample locn

3A1-3A3 interior face

3A5-3A8 interior

3A5-3B5 parging

3AB1 window



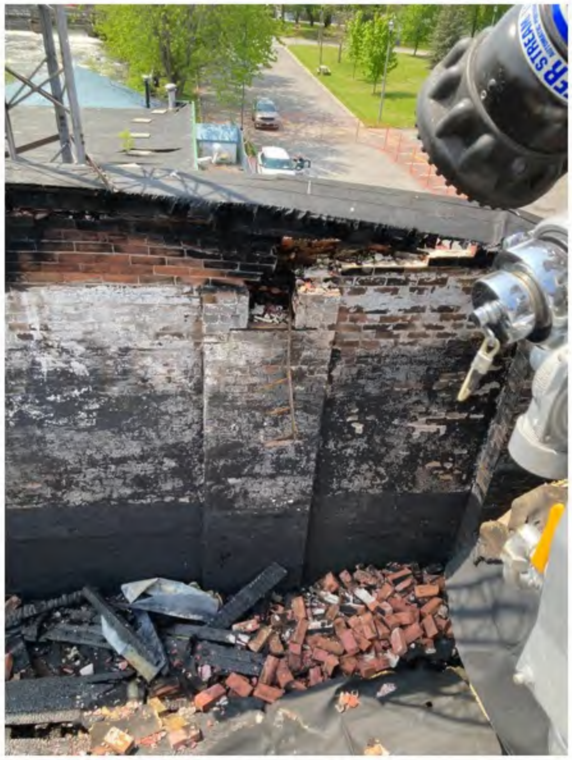
3B1 window sill 2B1 voussoir

3B2 interior

3B3 interior

3B8-3D8 interior

3D1-3A1 exterior



3D8 interior

3E1 exterior

3E1-3D1 interior

3E2 above 2E2 below

3E2 exterior

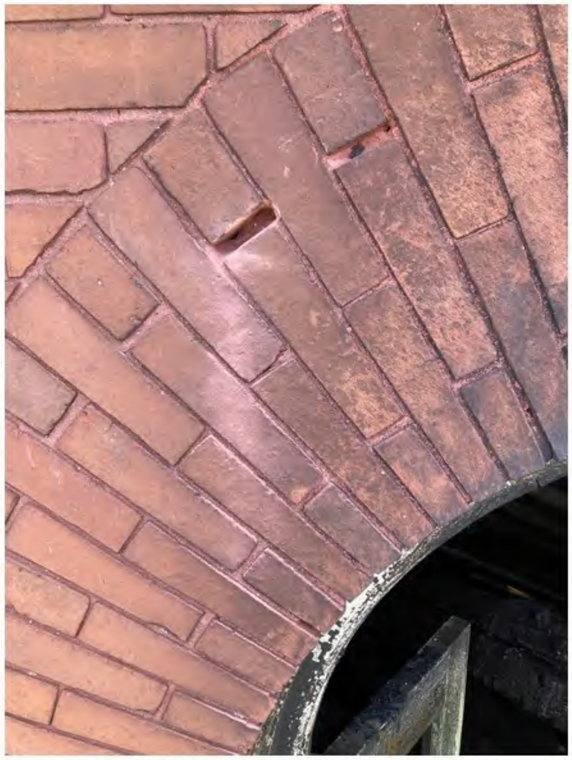
3E2 jamb wall section

3E2 view inside window

3E2 voussoir lintel section

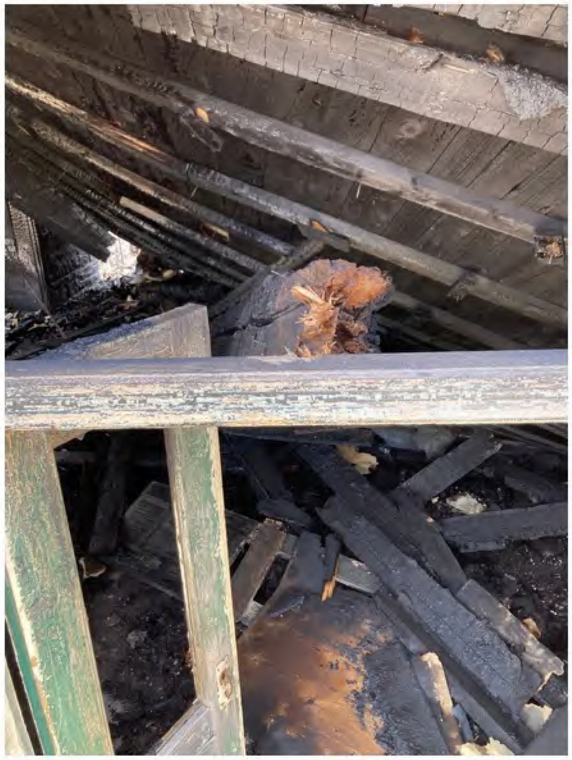
3E2 window sill @E2 voussoir

3E2 window sill detail

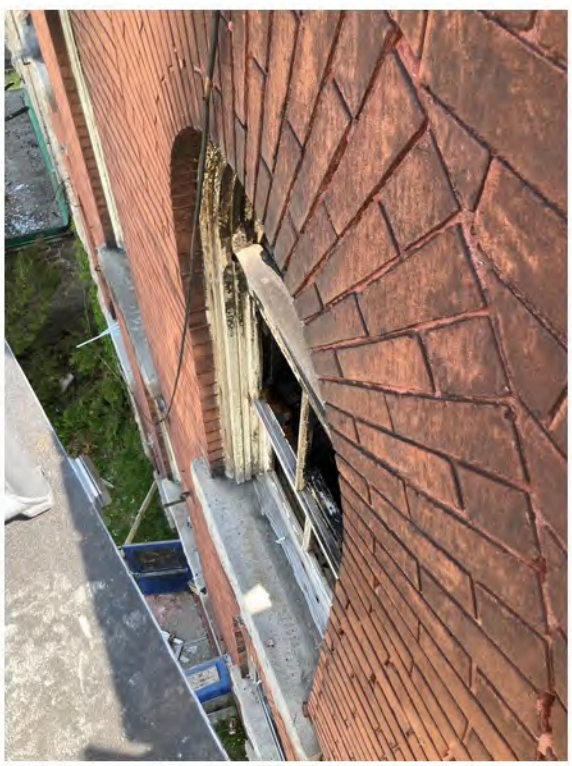


3E6.5 red brick mortar sample locn

3E6.5 Voussoir



3E6.5 window frame supporting failed roof beam

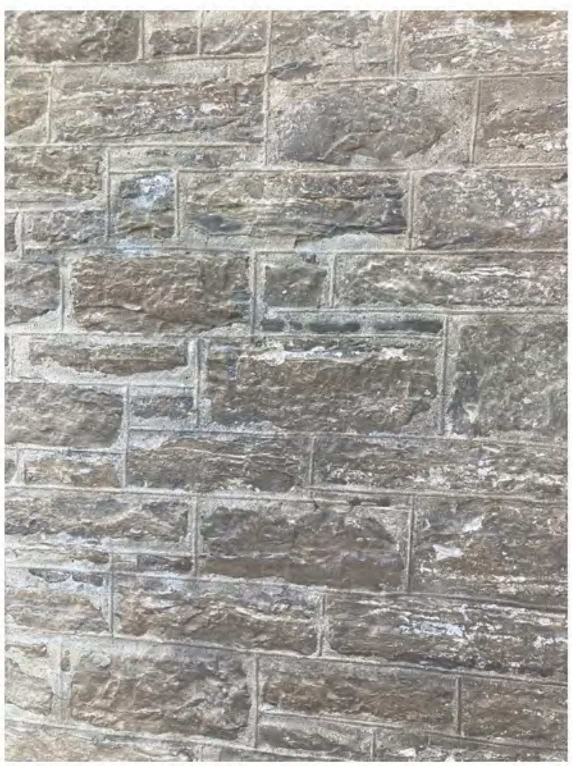


3E7 Brick Arch Window

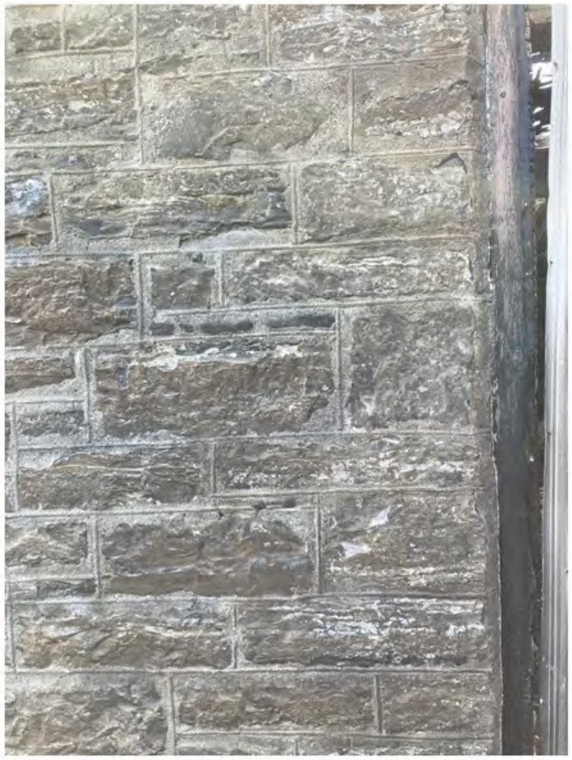
2A2-2A3 interior

2A3 interior

2B1 interior view

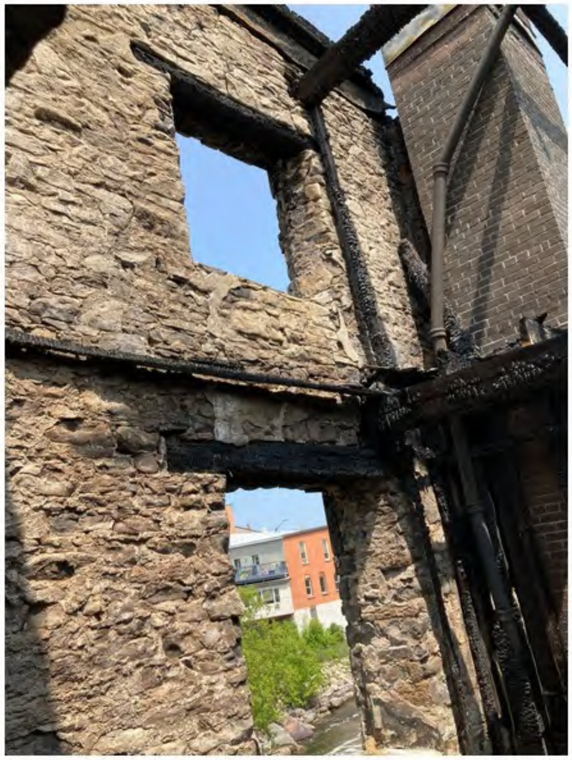


2D1 exterior good condition



2D1 exterior

2D1-3D1 exterior



2D1-3D1 interior

2D1-MD1 exterior screenshot

2E1 exterior

2E1-2D1 interior screenshot

2E1-ME1 exterior

2E1-RE4 exterior

2E2 pyramidal cracking

2E2 window jamb section

2E2 window sill

2E6.5 voussoir

2E7 brick sample locn

2E7 mortar sample locn

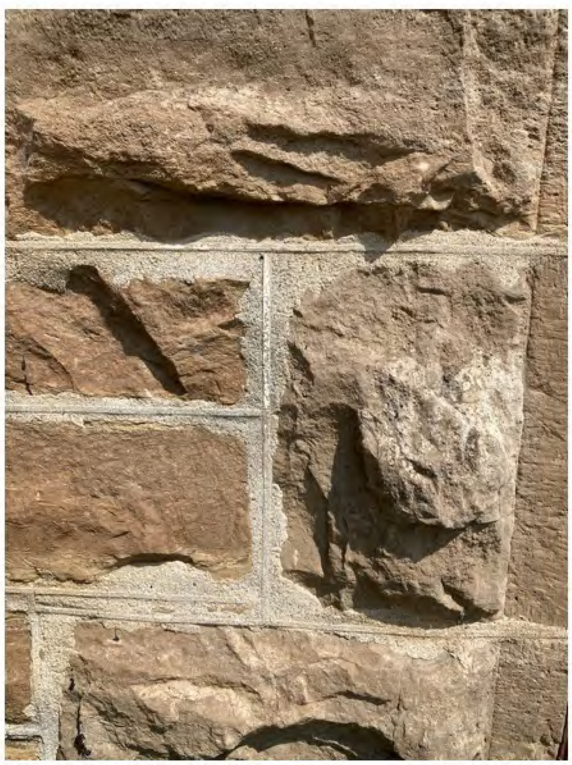
IMG_1294

MB1 exterior

MB1 window view to main and basement

MB1 window voussoir Spalled lintel stone sample locn

ME5 exterior Sample locn

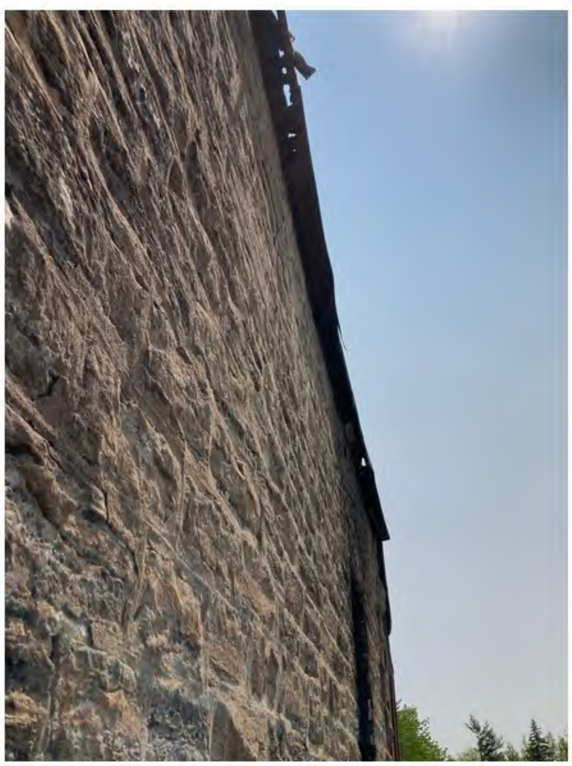


ME5 quoins

1924 Roof

Date Block iso

Date Block



E2-E4 parapet concave

E3 bell tie-back at 3rd floor elevation

E8-D8 Brick Parapet

Elevation West

Entrance Addition north elevation and roof

IMG_1195

IMG_1227

West face iso

West face looking south

HERITAGE BUILDING STRUCTURAL PATHOLOGY ENGINEERING REPORT

25 OLD MILL ROAD, SMITHS FALLS, ON

APPENDIX C

Ottawa International Airport Wind Data

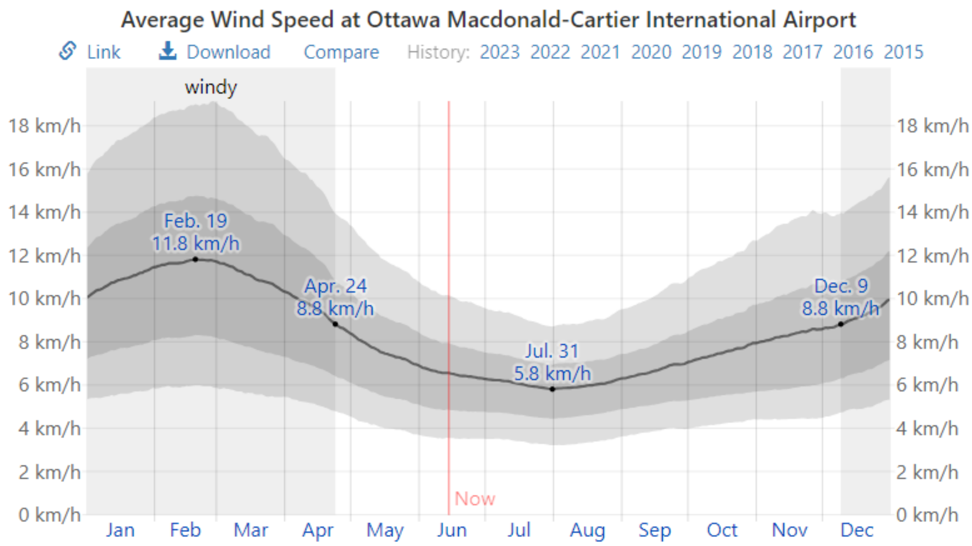
Wind

This section discusses the wide-area hourly average wind vector (speed and direction) at 10 metres above the ground. The wind experienced at any given location is highly dependent on local topography and other factors, and instantaneous wind speed and direction vary more widely than hourly averages.

The average hourly wind speed at Ottawa Macdonald-Cartier International Airport experiences significant seasonal variation over the course of the year.

The windier part of the year lasts for 4.5 months, from December 9 to April 24, with average wind speeds of more than 8.8 kilometres per hour. The windiest month of the year at Ottawa Macdonald-Cartier International Airport is February, with an average hourly wind speed of 11.7 kilometres per hour.

The calmer time of year lasts for 7.5 months, from April 24 to December 9. The calmest month of the year at Ottawa Macdonald-Cartier International Airport is August, with an average hourly wind speed of 6.0 kilometres per hour.

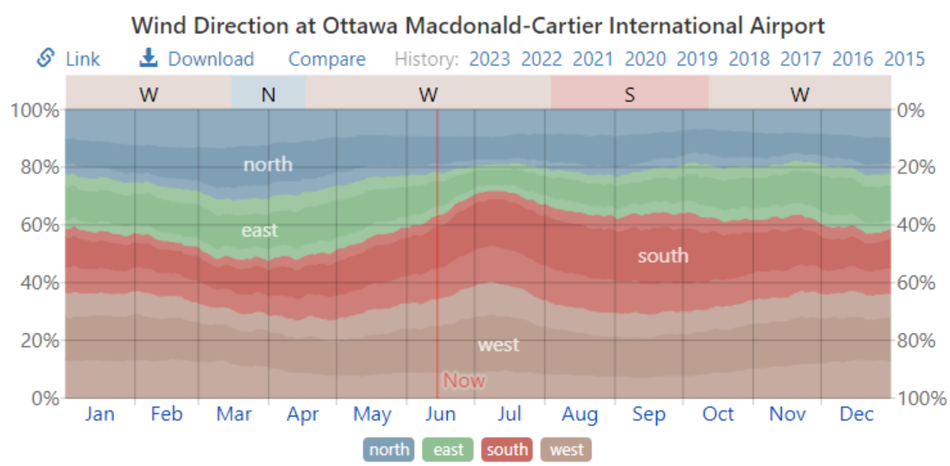


The average of mean hourly wind speeds (dark gray line), with 25th to 75th and 10th to 90th percentile bands.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Wind Speed (kph)	10.8	11.7	11.0	9.4	7.5	6.5	6.0	6.0	6.7	7.5	8.3	9.2

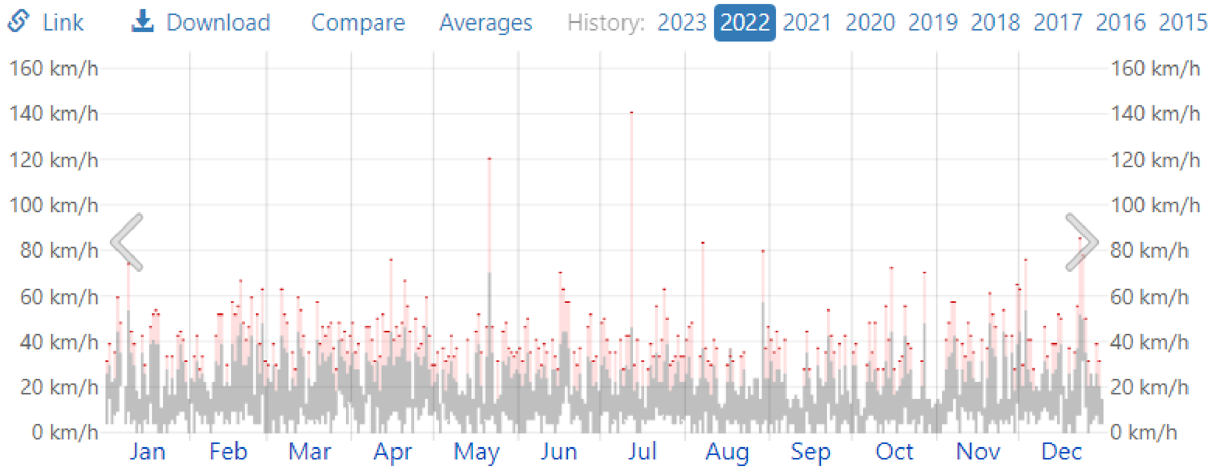
The predominant average hourly wind direction at Ottawa Macdonald-Cartier International Airport varies throughout the year.

The wind is most often from the north for 1.1 months, from March 15 to April 17, with a peak percentage of 31% on March 16. The wind is most often from the west for 3.5 months, from April 17 to August 3 and for 5.1 months, from October 12 to March 15, with a peak percentage of 41% on July 8. The wind is most often from the south for 2.3 months, from August 3 to October 12, with a peak percentage of 35% on September 18.



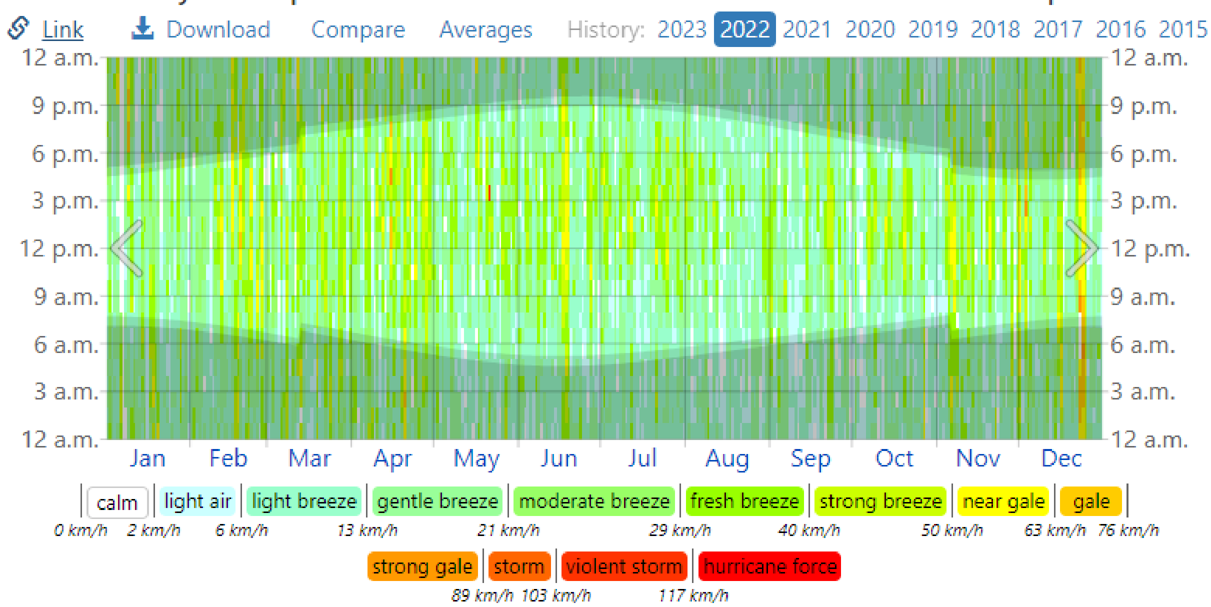
The percentage of hours in which the mean wind direction is from each of the four cardinal wind directions, excluding hours in which the mean wind speed is less than 1.6 km/h. The lightly tinted areas at the boundaries are the percentage of hours spent in the implied intermediate directions (northeast, southeast, southwest, and northwest).

Wind Speed in 2022 at Ottawa Macdonald-Cartier International Airport



The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

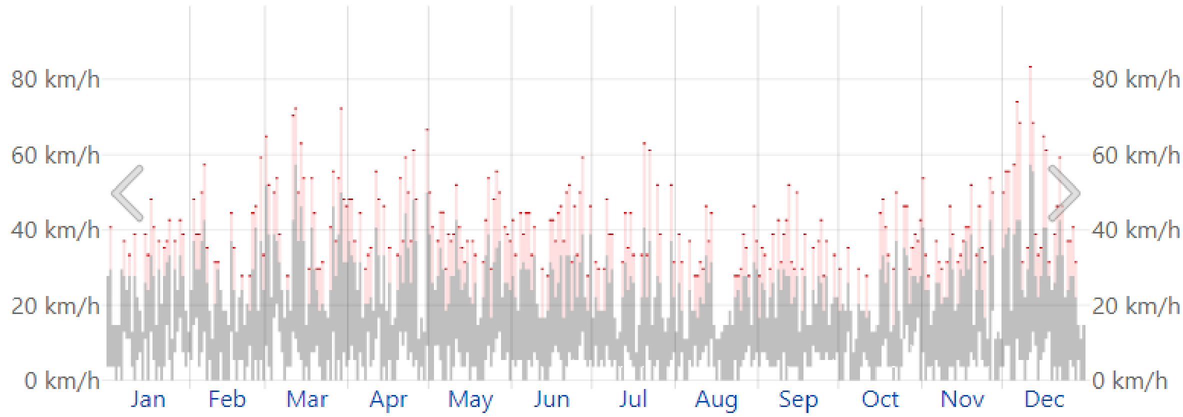
Hourly Wind Speed in 2022 at Ottawa Macdonald-Cartier International Airport



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2021 at Ottawa Macdonald-Cartier International Airport

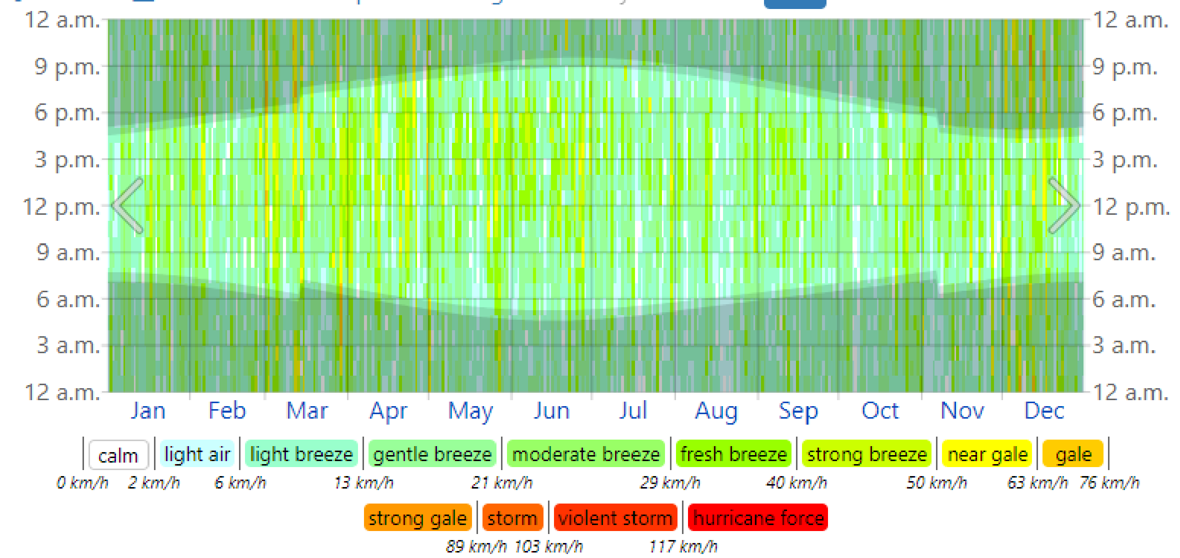
Link Download Compare Averages History: 2023 2022 2021 2020 2019 2018 2017 2016 2015



The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

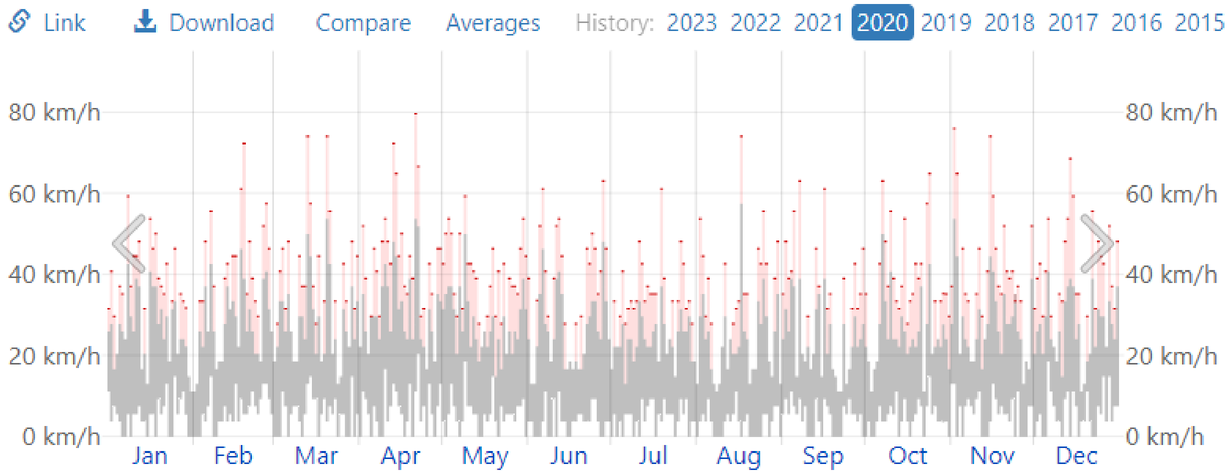
Hourly Wind Speed in 2021 at Ottawa Macdonald-Cartier International Airport

Link Download Compare Averages History: 2023 2022 2021 2020 2019 2018 2017 2016 2015



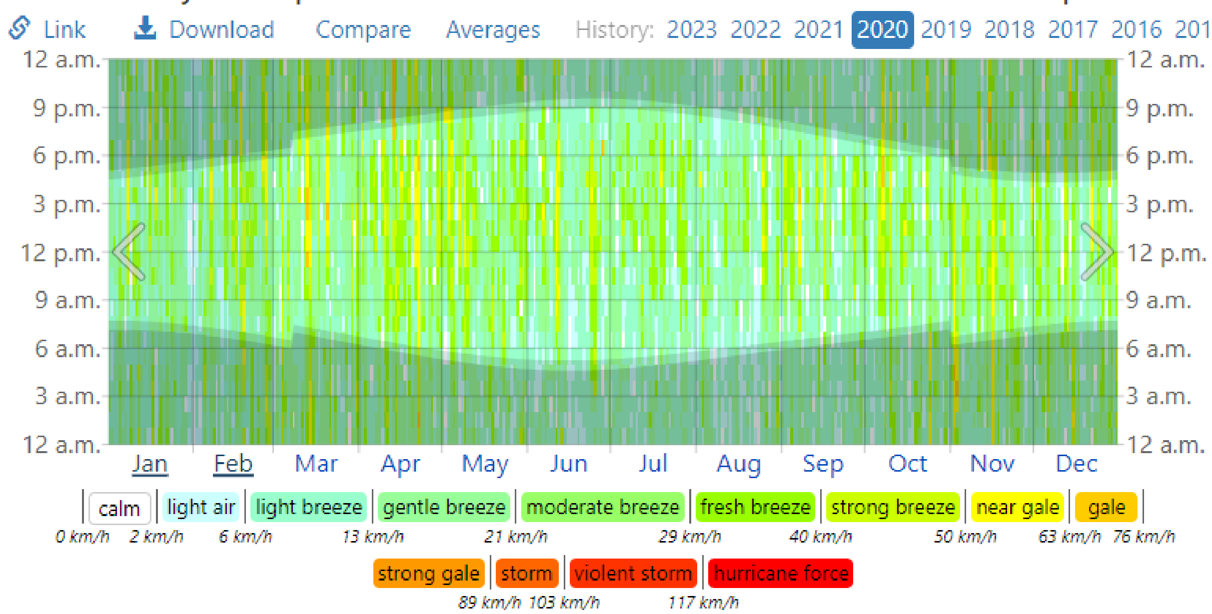
The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2020 at Ottawa Macdonald-Cartier International Airport



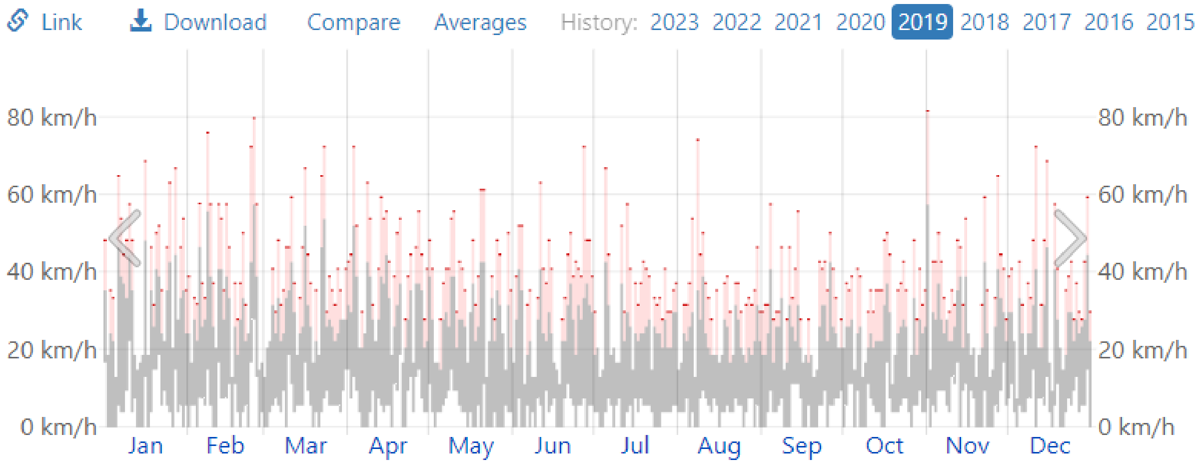
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

Hourly Wind Speed in 2020 at Ottawa Macdonald-Cartier International Airport



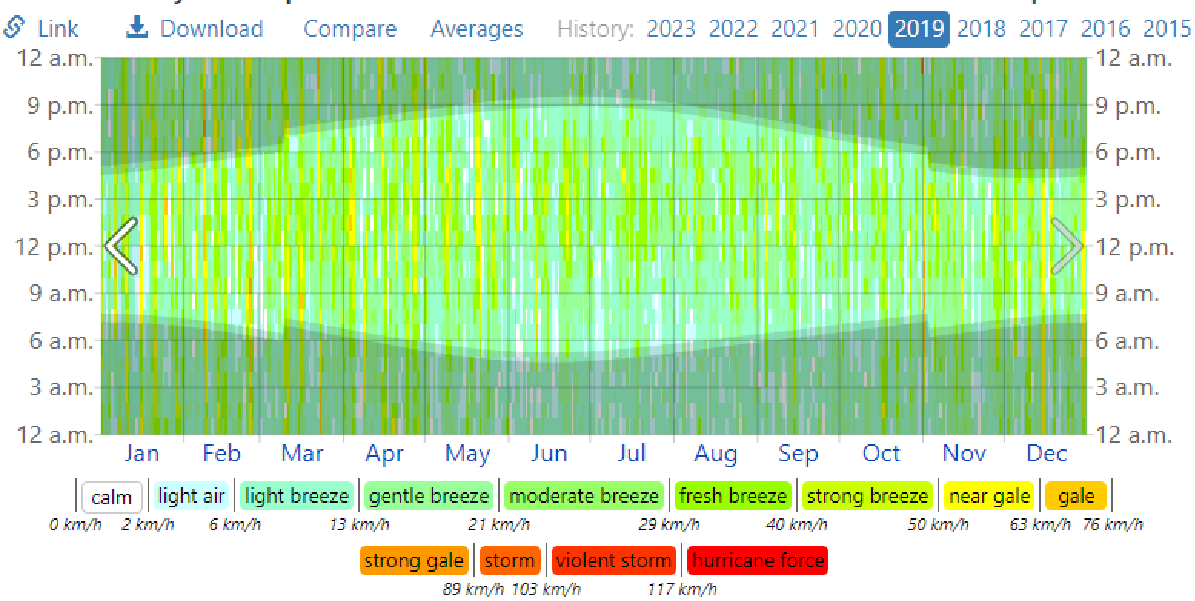
The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2019 at Ottawa Macdonald-Cartier International Airport



The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

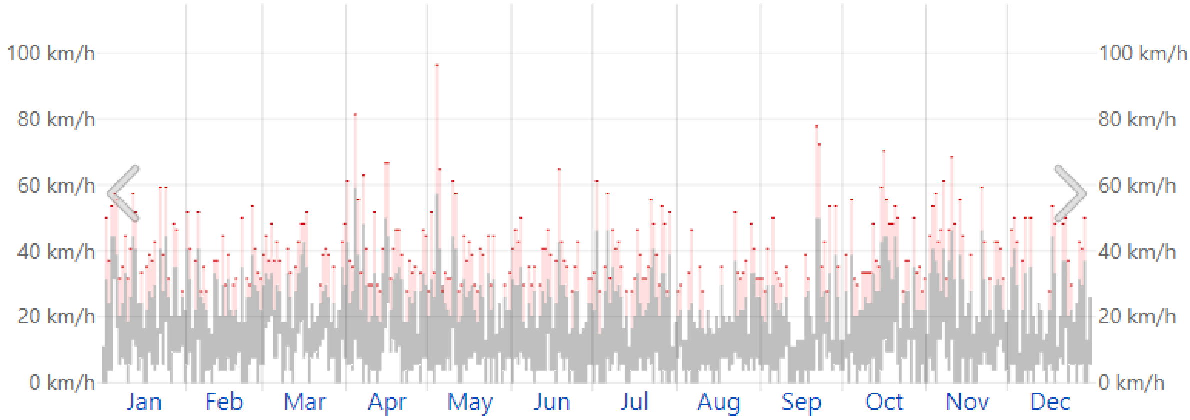
Hourly Wind Speed in 2019 at Ottawa Macdonald-Cartier International Airport



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2018 at Ottawa Macdonald-Cartier International Airport

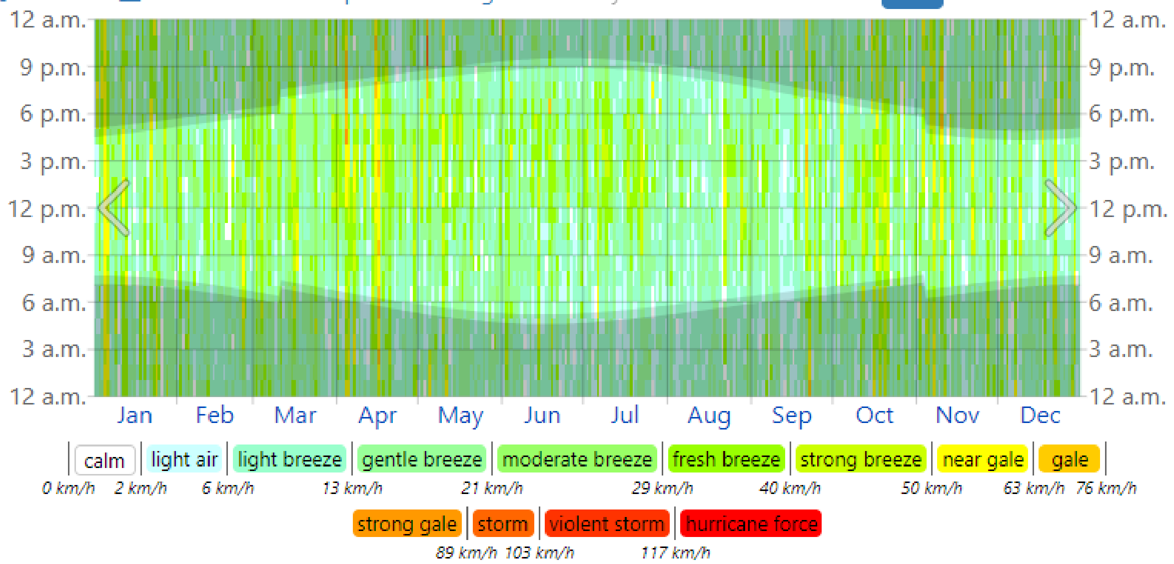
Link Download Compare Averages History: 2022 2021 2020 2019 2018 2017 2016 2015 2014



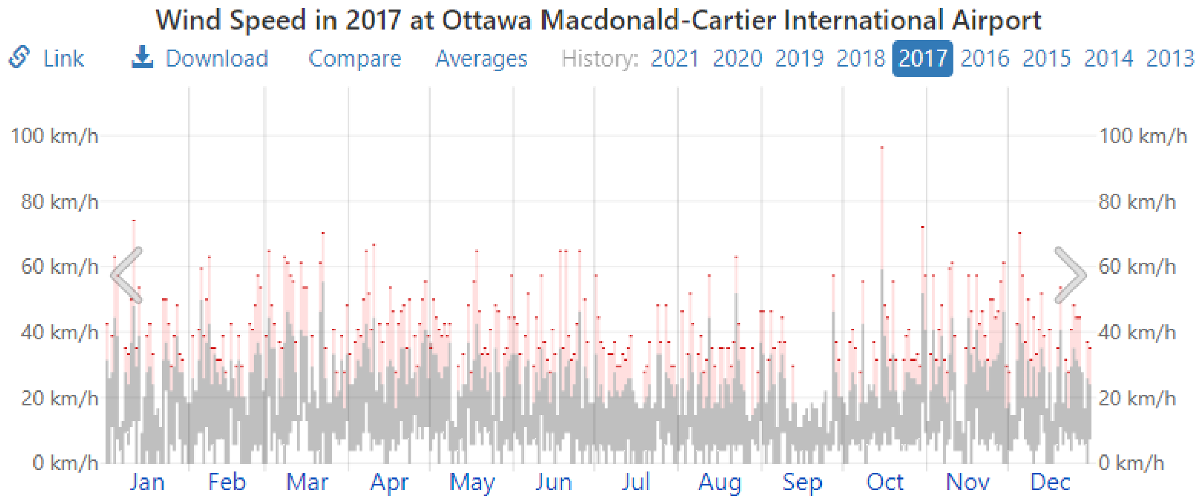
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

Hourly Wind Speed in 2018 at Ottawa Macdonald-Cartier International Airport

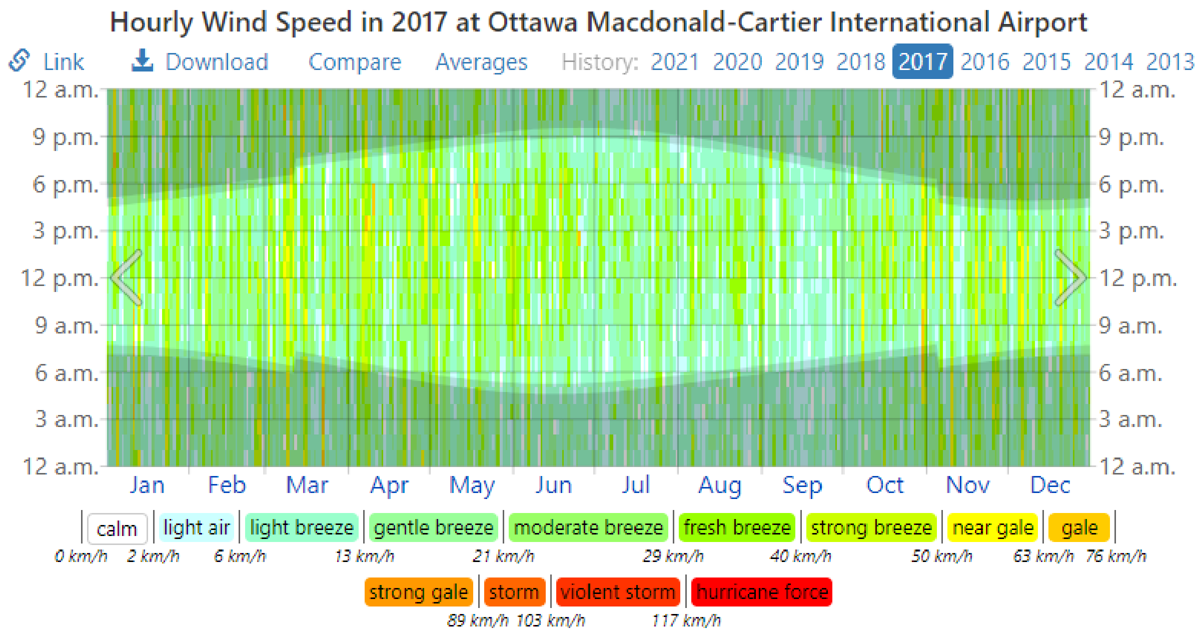
Link Download Compare Averages History: 2022 2021 2020 2019 2018 2017 2016 2015 2014



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.



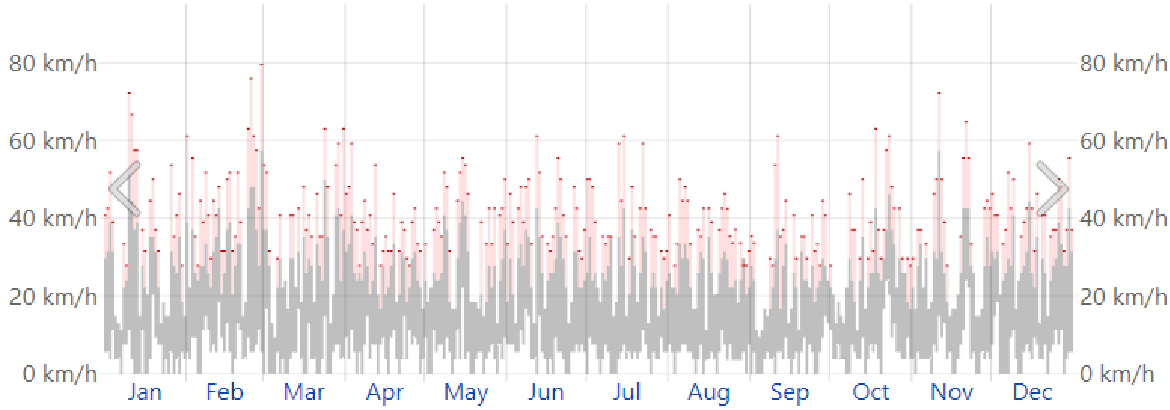
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2016 at Ottawa Macdonald-Cartier International Airport

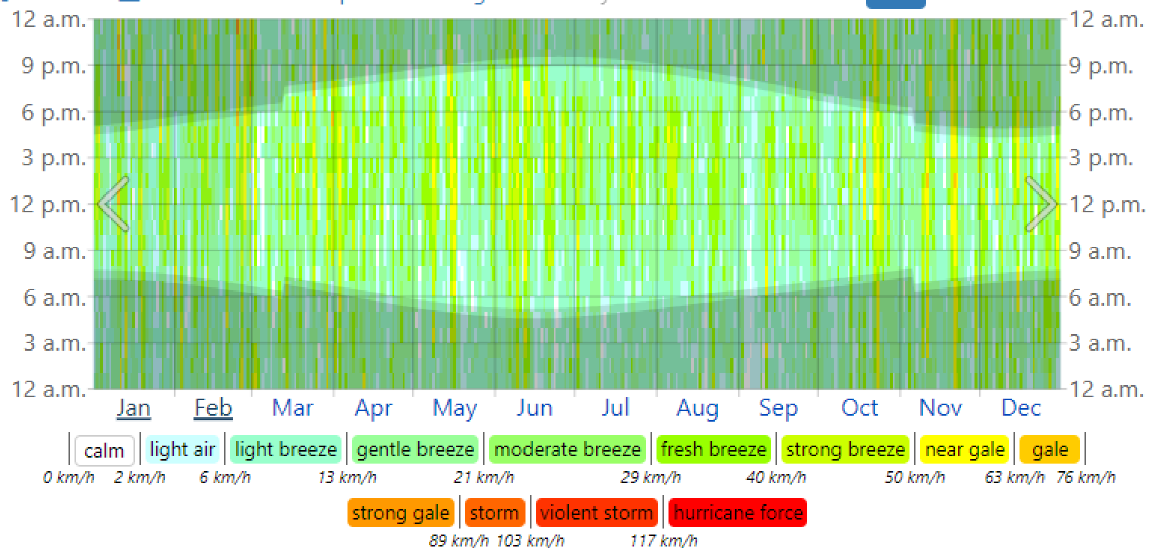
Link Download Compare Averages History: 2020 2019 2018 2017 2016 2015 2014 2013 2012

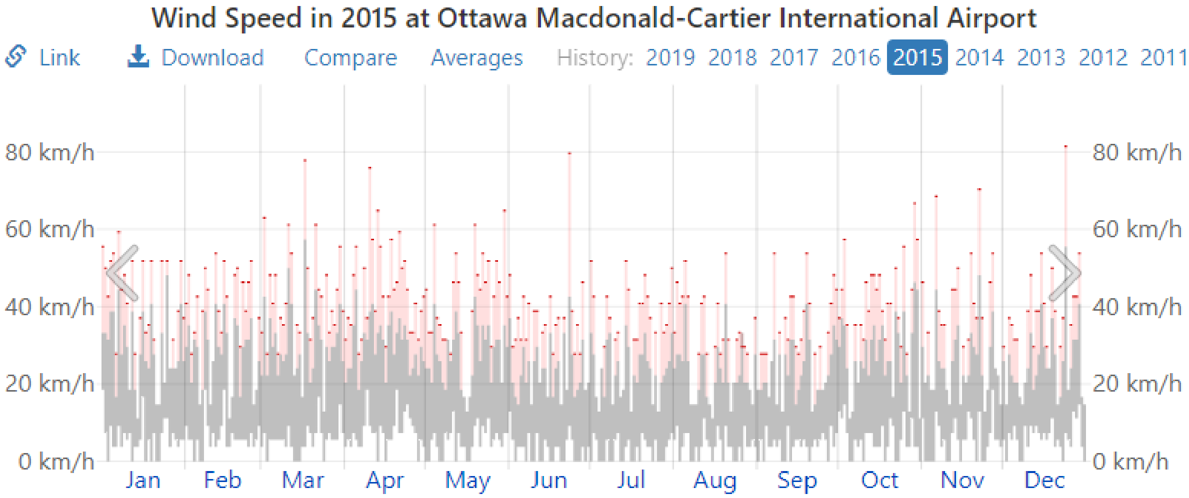


The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

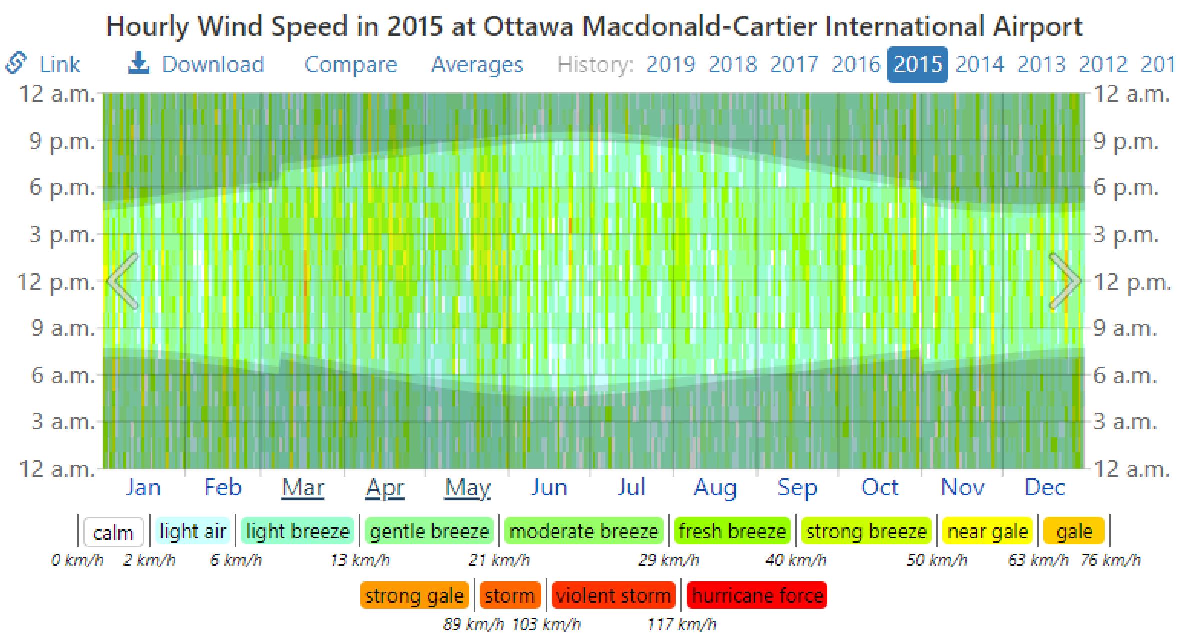
Hourly Wind Speed in 2016 at Ottawa Macdonald-Cartier International Airport

Link Download Compare Averages History: 2020 2019 2018 2017 2016 2015 2014 2013 2012





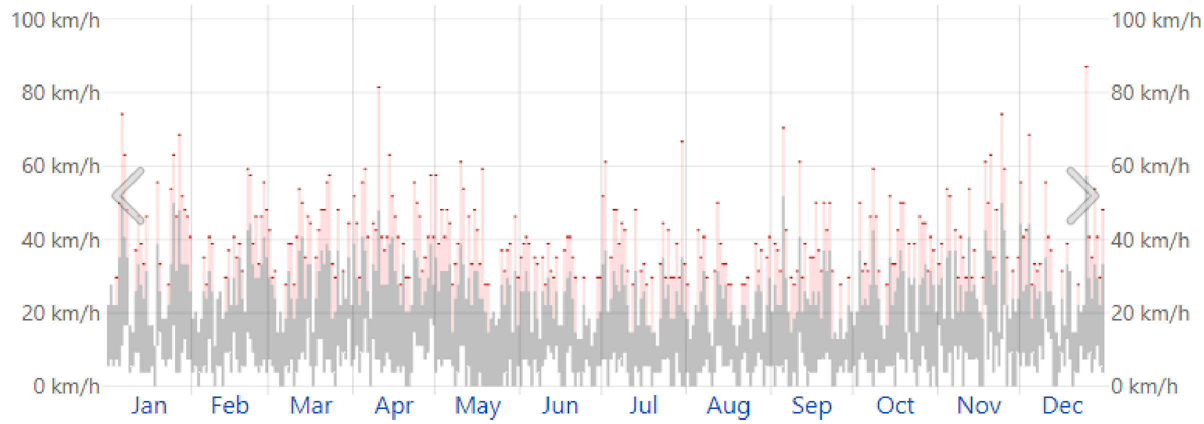
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.

Wind Speed in 2014 at Ottawa Macdonald-Cartier International Airport

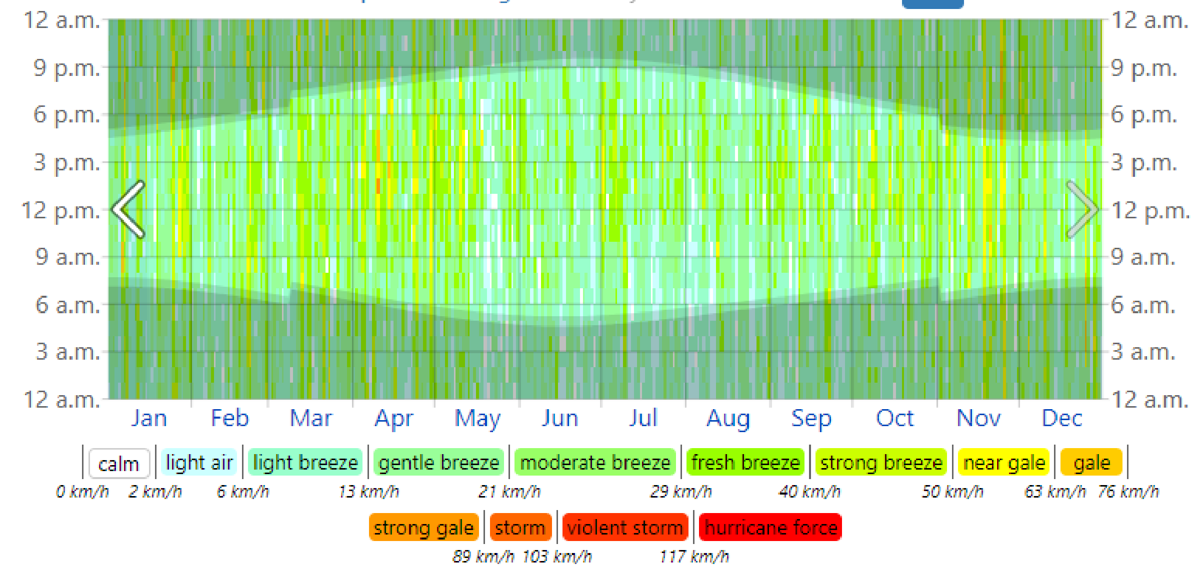
Link Download Compare Averages History: 2018 2017 2016 2015 2014 2013 2012 2011 2010



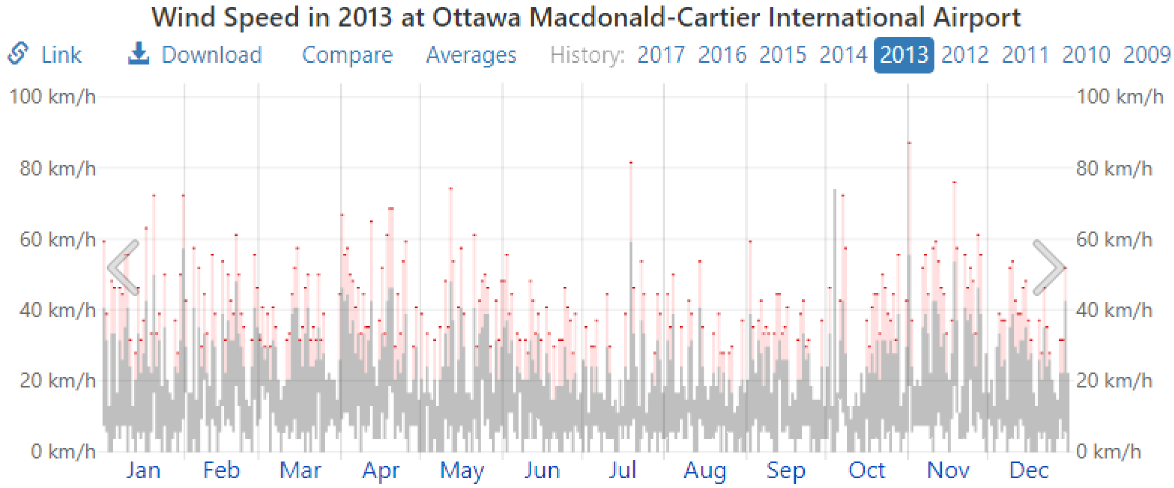
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).

Hourly Wind Speed in 2014 at Ottawa Macdonald-Cartier International Airport

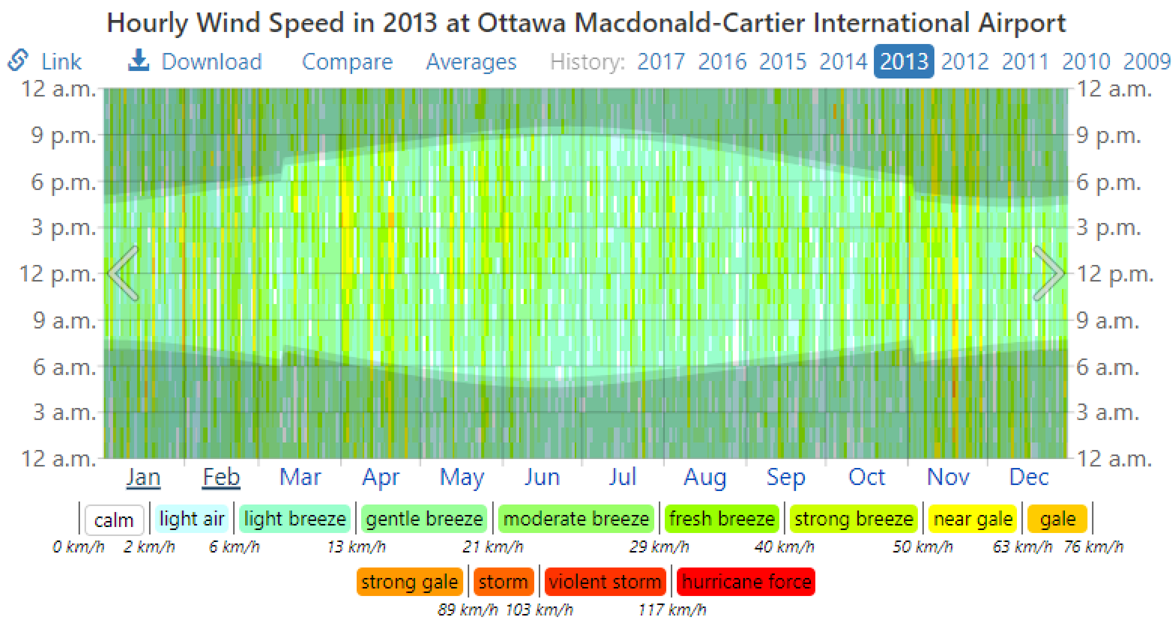
Link Download Compare Averages History: 2018 2017 2016 2015 2014 2013 2012 2011 2010



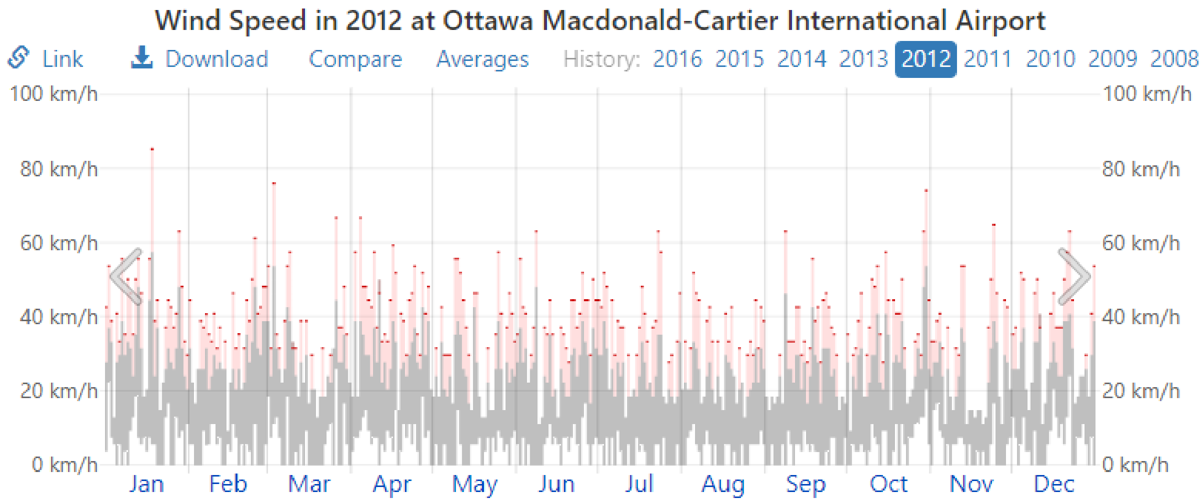
The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.



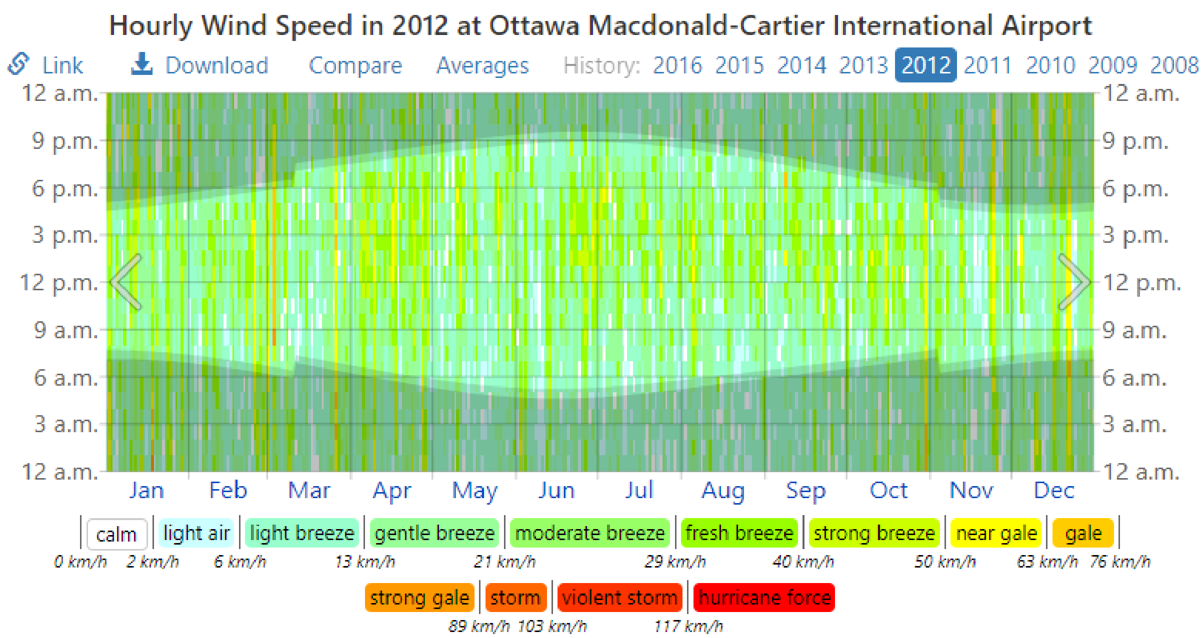
The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.



The daily range of reported wind speeds (gray bars), with maximum gust speeds (red ticks).



The hourly reported wind speed, color coded into bands according to the Beaufort scale. The shaded overlays indicate night and civil twilight.