Framework for Ensuring the Ongoing Maintenance of Existing Buildings
Applied to a National Capital Commission Case Study

by

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ABSTRACT

All buildings deteriorate over time. Factors that cause decay range from everyday occurrences, such as weathering, to purposeful destruction, such as vandalism. If buildings are neglected, the severity of their deterioration increases and the amount of repair work required also increases. This means that more time and money would be needed to bring the buildings back to a suitable working condition. The deterioration of buildings, however, can be slowed with ongoing maintenance.

This research created an informative, user-friendly, and uniform workflow that can ensure the ongoing maintenance of buildings. Header House, a National Capital Commission owned building located in Major Hill Park, was used as a case study to showcase the workflow. The first step of any maintenance plan is research. This means not only looking into past defects and issues, but also the history and significance of the building. From there, the current condition of the building needs to be documented.

A survey of the building was conducted to produce as-built and condition assessment drawings to indicate the areas of concern. A Building Information Model is also made to keep track of the various elements of a building. Finally, a seismic analysis was conducted to assess the structure's performance. As there were many access limitations in surveying the building, the seismic analysis incorporated a range of values and conditions to determine a possible range in seismic performance. The drift of the building was not sensitive to these range cases, while the acceleration and base

shear were affected by the change in stiffness. The results of this analysis determined that Header House's performance falls under the damage and serviceability limits for masonry structures. Therefore, no seismic interventions are required.

A monitoring program was then suggested to accurately determine the causes of deterioration. The final maintenance plan begins with corrective maintenance to repair the current decayed elements. From there, a preventive maintenance plan was created, which includes small tasks that can be conducted to reduce the rate of deterioration. The workflow is then repeated at five year intervals to ensure that the building continues to be maintained.

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GLOSSARY

- α - constant equal to 0.7
- α_0 - zero moment coefficient
- $\alpha_v(\lambda)$ - pier shear ratio
- b - corrective coefficient connected to the distribution of stress on the section depending on the geometric aspect ratio of the wall
- β - constant equal to 0.3
- f_b - compressive strength of the brick unit
- f_m - compressive strength of mortar
- $f_{M,c}$ - overall compression strength of masonry wall
- $f_{M,t}$ - diagonal tensile strength of masonry
- F_p - minimum of $f_{M,t}$ and $0.4f_{M,c}t_{sp}h_{sp}$
- f_{vo} - initial shear strength of masonry
- h_0 - height of top of pier from the ground
- h_{pr} - height of pier
- $h_{pr,0}$ - location along the height of the pier where the moment is equal to zero
- h_{sp} - height of the spandrel
- κ - peak compressive strength reduction factor
- l_{pr} - pier length
- $l_{pr,c}$ - length of the compressed section of the pier
- l_{sp} - length of the spandrel
- λ - pier slenderness ratio
- $M_{Pr,r}$ - moment rocking failure strength of pier

- $M_{Sp,r}$ - moment rocking failure strength of spandrel
- μ - friction coefficient
- N - compressive axial load at failure of the wall
- ξ - masonry cohesion coefficient
- σ_0 - normal compressive stress on the pier due to vertical loading
- σ_d - average vertical stress over compressed part of wall
- $\sigma(N)$ - mean compressive stress in the pier due to the axial force N
- t_{pr} - pier thickness
- t_{sp} - thickness of the spandrel
- τ_0 - shear strength of the masonry at zero compressive stress
- ψ - corrective factor based on boundary conditions
- $V_{Pr,r}$ - rocking failure equivalent shear strength of piers
- $V_{Pr,s}$ - diagonal shear cracking failure strength of piers
- $V_{Pr,t}$ - shear sliding failure strength of piers
- $V_{Sp,r}$ - rocking failure equivalent shear strength of spandrels
- $V_{Sp,s}$ - shear failure strength of spandrels

1 – INTRODUCTION

In David Watt's book, *Surveying Historic Buildings*, he states that "Much of the deterioration and disrepair diagnosed in surveys comes as a direct result of inadequate maintenance."¹ Repairing damaged buildings is often significantly more costly than conducting preventive maintenance work. However, many owners and occupiers often fail to see the importance and cost-benefits of ongoing maintenance work.² In addition, the work done on a building can often be hard to track as reports can vary in style and organisation. This can make it more difficult to determine the causes of deterioration, which preventive maintenance strives to monitor and control. By understanding these sources of deterioration, preventive interventions can be implemented to decrease their effects. These interventions, for example, can help to mitigate the damage during a future seismic event, as older buildings were often not designed specifically to withstand earthquakes.³

The steps required to determine the appropriate maintenance of a structure and how to implement it can often seem complicated and time-consuming. What is missing is an informative, user-friendly, and uniform workflow that can ensure the ongoing maintenance of buildings.

¹ David Watt, *Surveying Historic Buildings*, 2nd ed. (Donhead St. Mary: Donhead Publishing Ltd, 2010), 348.

² Ibid.

³ G. Vasconcelos and P. B. Lourenço, "In-Plane Experimental Behavior of Stone Masonry Walls under Cyclic Loading," *Journal of Structural Engineering* 135, no. 10 (2009): 1269.

1.1. Problem Statement

The motivation for conducting this research came from discussions with the National Capital Commission (NCC). The NCC is currently undergoing a major change in tenant contracts, so they need a new method of planning and conducting maintenance. Before this change, the contracts lasted for decades and the tenants were in charge of maintaining their properties. As the contracts came to a close, NCC noticed that the condition of most of their properties was not up to the suggested standard. They are now looking into conducting the maintenance themselves and, therefore, need a workflow that can be used for their sites, which make up 11% of all land in the Ottawa Capital region.⁴

The aim of this thesis is to research current practices for documenting and maintaining existing buildings and to synthesize the process into an easy to follow framework that can be used not only by the NCC, but by every building owner. This framework will further develop the process of applying the *Standards and Guidelines for the Conservation of Historic Buildings in Canada* for maintenance. In addition to this framework, attention is given to determining how to conduct a seismic analysis of an existing building when various properties are unknown. This research will determine the affect of varying mechanical properties and varying connectivity cases on the drift, displacement, residual drift, acceleration, and base shear of a structure.

⁴ National Capital Commission, *National Capital Commission* (2018). <<http://ncc-ccn.gc.ca/>> [20 January 2018], para. 3.

1.2. Case Study

To showcase the framework, a case study was conducted on Header House as recommended by the NCC. Header House was built from 1901 to 1902 as a potting house for the Major's Hill Park greenhouse complex. The Chief Architect's Office of the Department of Public Works prepared the design of the building under the supervision of Chief Architect David Ewart. When the greenhouse complex was removed in 1937, the National Capital Commission took ownership of Header House, converting it into a maintenance building for the park. The garage on the west-end of the building was added in 1960 and matches the architectural elements of the original structure.⁵ Figure 1 is a photograph of the Header House in 1988.

Figure 1: Header House in 1988.⁶

⁵ Canada's Historic Places, *Former Potting House (now Maintenance Building)*, (01 August 1990). <[http://www.historicplaces.ca/media/10589/1987-128\(e\)formerpottinghouse\(nowmaintenancebuilding\).pdf](http://www.historicplaces.ca/media/10589/1987-128(e)formerpottinghouse(nowmaintenancebuilding).pdf)> [26 February 2018], 1.

⁶ Canada's Historic Places, *Maintenance Building* (2008). <<http://www.historicplaces.ca/en/rep-reg/place-lieu.aspx?id=9947>> [20 January 2018], para. 1.

In recent years, Header House has been rented out to various commercial tenants. The first tenant was Blink Gallery, a non-profit artist collective that used the space to showcase work from artists in the Ottawa region and beyond.⁷ Currently, Header House is home to Tavern on the Hill, a seasonal outdoor canteen, patio, and ice cream shop. Figure 2 is a picture of Header House taken on September 25, 2017.

Figure 2: Header House on September 25, 2017.

Header House was formally recognized as a Historic Place on July 7, 1988 by the Federal Heritage Buildings Review Office (FHBRO) and was added onto the

⁷ National Capital Commission, *Major's Hill Park*, (2017). <<http://archives.ncc-ccn.ca/places-to-visit/parks-paths/majors-hill-park>> [26 February 2018], para. 5.

Canadian Register of Historic Places on August 19, 2008. Its heritage value is characterized by its historical, architectural, and environmental significance.⁸ Header House's historical value is closely associated with Major's Hill Park. The park was first the home of famous English military engineer John By, also known as Colonel By. In the early 20th century, the Ottawa Improvement Commission, currently the National Capital Commission, launched a city beautification project, which included transforming the area into Ottawa's first municipal park.⁹ Header House, as a potting shed, was constructed for this project, alongside renovations to an existing greenhouse, and the construction of a new conservatory, a two-storey summerhouse, and new fountains and walkways.¹⁰ Header House's architectural value resides primarily in its picturesque domestic forms, which are outlined by its Baronial style. The use of limestone on the exterior walls and brick on the interior showcase how the building's use was taken into consideration for its design as they provide the necessary cool temperatures and shade required for potting plants. The building also showcases quality craftsmanship through its coursed, rough-cut limestone with corner quoins and the use of a plain dressed canted stone band to separate the basement and upper walls.¹¹ Header House's environmental value stems from its compatibility with the municipal park setting. Its presence is not only familiar to visitors, but to those appreciating the view from Parliament Hill and the Ottawa River. The natural

⁸ Ibid.

⁹ Margaret Meek, *History of the city beautiful movement in Canada, 1890-1930* (Vancouver: University of British Columbia, 1979), 117-18.

¹⁰ Canada's Historic Places, *Maintenance Building*, para. 3.

¹¹ Ibid., para. 4.

materials on the building's exterior fit in with the picturesque natural character of Major's Hill Park.¹²

1.3. Thesis Outline

Section 2 presents a literature review on building deterioration, maintenance, and all aspects of historic structure reports. The latter covers preliminary research, documentation through surveying, data processing and analysis, monitoring strategies, and maintenance programming. Section 3 covers the methodology of the framework and the work done for the case study of the National Capital Commission owned Header House, while section 4 showcases the results and products of the case study. Finally, section 5 presents the conclusion of the research and recommendations for future work.

¹² Ibid., para. 5.

2 – LITERATURE REVIEW

To better understand how to maintain existing buildings, it is important to understand the issues buildings can face, the methods to identify them, and how they can be mitigated. This literature review will discuss the six factors that cause deterioration in buildings and the two types of maintenance used to repair and prevent them. Reports on existing buildings will be reviewed to understand current practices. This includes methods of documenting and recording buildings and their defects. Various data processing techniques and outputs will be discussed to showcase the use of surveying in the ongoing maintenance. As many existing buildings are subject to severe damage during a seismic event, seismic analysis procedures will also be covered. Finally, monitoring methods and techniques for producing maintenance plans will be explored.

2.1. Building Deterioration

All buildings decay over time, but this deterioration does not necessarily mean that the building has failed.¹³ The failure of a building is measured by its defects. These defects are characterized as an element of the building, such as a material or component, that no longer performs to an accepted criterion.¹⁴ This can include

¹³ Samuel Harris, *Building Pathology: Deterioration, Diagnostics, and Intervention* (Toronto: John Wiley & Sons, Inc., 2001), 13.

¹⁴ Azree Mydin et al., “Factors of Deterioration in Building and the Principles of Repair,” *Analele Universității "Eftimie Murgu" Reșița: Fascicola I, Inginerie*. XIX (2012): 345.

failing in function, performance, statutory regulations, or specific user requirements. Defects can affect the structure, fabric, and services of a building.¹⁵

2.1.1. What Causes Building Defects

Building defects can be a result of poor design, but they can also manifest over time due to the presence of decaying factors. The main factors that cause decay are excess moisture, structural changes, fungal and insect infestation, inappropriate interventions, and poor performance choices in the use of the building.¹⁶ Of these factors, excess moisture is the greatest source of deterioration due to its abundance and large damaging effects.¹⁷ Although these factors are separated into different categories, they are not independent of each other. For example, fungal and insect infestations are likely to occur if there is water present. These infestations tend to eat away the moist wooden material thus decreasing its structural performance.¹⁸ These decaying factors can be presented as a set of failure mechanisms. David Watt categorises these mechanisms into six groups: atmospheric and climatic action; excess moisture; chemical, physical and biological action; movement; fire; and human factors. Like the main factors of decay, these failure mechanisms are not necessarily independent from one another.

¹⁵ David Watt, *Building Pathology*, 2nd ed. (Oxford: Blackwell Publishing Ltd, 2007), 96.

¹⁶ Watt, *Surveying Historic Buildings*, 81.

¹⁷ Canada's Historic Places, *Heritage Building Maintenance Manual*, (30 December 2016). <https://www.gov.mb.ca/chc/hrb/pdf/maintenace_for_heritage_bldgs.pdf> [10 November 2017], 19.

¹⁸ Watt, *Surveying Historic Buildings*, 153.

2.1.1. Atmospheric and Climatic Action

The deterioration effects of atmospheric and climatic action vary greatly depending on the geographic location, situation, exposure, design, materials, condition, and usage of the building.¹⁹ These climatic actions can range from everyday weather to severe natural disasters, while atmospheric actions take into account the makeup of the air and its pollutants.²⁰ Natural disasters, such as avalanches, earthquakes, volcano eruptions, floods, tidal waves, and tornadoes, may not happen regularly, however may cause considerable damage to buildings when they do.²¹ Climate change increases the occurrence and intensity of most of these disasters thus causing increasingly more damage over the years.²² However, more common weather conditions can also cause damage to buildings. One of the more damaging weather phenomena is lightning. If lightning were to hit a building, the building could catch on fire, metals could crack and/or melt, and the electrical services could overload.²³ However, lightning hitting a building is still not an everyday occurrence. Temperature differences, sun, rain, and wind can also cause damage to a building over time. This takes the form of weathering. Weathering is the “breakdown and alteration of materials by mechanical and chemical processes.”²⁴ One of the more destructive weathering forces in colder

¹⁹ Ibid., 131.

²⁰ Ibid., 132.

²¹ Watt, *Building Pathology*, 106.

²² Katherine Richardson et al., *Climate Change: Global Risks, Challenges, and Decisions* (Cambridge: Cambridge University Press, 2011), 117.

²³ Watt, *Building Pathology*, 107.

²⁴ Ibid.

climates is frost damage. Frost damage is commonly seen on masonry and occurs when water within a material freezes. This causes it to expand quicker than the material can, causing fracturing and disintegration.²⁵ Figure 3 showcases the spalling of clay brick due to frost damage.

Figure 3: Chimney bricks affected by frost damage.²⁶

The air itself can also be considered a failure mechanism. This is because the air can bring various pollutants on and into a building. These pollutants can be hostile to certain materials and people or cause discomfort to the occupants. Some of the more hostile pollutants are: sulphur dioxide, nitrogen oxides, ozone, radon, and excess carbon dioxide.²⁷ The pollutants can react with the building materials causing a variety of deterioration effects. Other pollutants can decrease the indoor air quality of

²⁵ B. Kralj et al., “On the Mechanics of Frost Damage to Brick Masonry,” *Computers and Structures*, 41, no 1 (1991): 56.

²⁶ New Buck Chimney Services, *Freeze Thaw Damage Affects All Masonry*, (2017).
<<https://newbuckchimney.com/freeze-thaw-damage-affects-masonry/>> [7 November 2017].

²⁷ Watt, *Building Pathology*, 111.

a building if they find their way in through cracks or openings in building envelopes. This can affect the health and comfort of the occupants, creating the ‘sick building syndrome’.²⁸ These pollutants can include: airborne dusts, mineral fibres, microbial pollutants, soot, and smoke.²⁹

2.1.2. Excess Moisture

Excess moisture is categorized as both a failure mechanism and a decaying factor. When seeing it as a failure mechanism, the methods of building up the moisture are considered. This includes penetrating damp or moisture infiltration, rising damp, and surface and interstitial condensation.³⁰ Determining the root cause of a moisture problem can be at times straightforward and other times more complex, especially since the moisture can travel far from its source before becoming evident to the occupants.³¹ Penetrating damp and moisture infiltration both relate to moisture moving into a building. Penetrating damp is a more conductive method of moisture transfer, where the moisture is absorbed by a porous material until it evaporates on the other side.³² Moisture infiltration can include penetrating damp, but also more convective methods, including air infiltration. Air infiltration can take place through intentional and unintentional opening in the building envelope. Some unintentional

²⁸ Chuck Yua and Jeong Kim, “Building Pathology, Investigation of Sick Buildings – VOC Emissions,” *Indoor Built Environment* 19, no 1 (2010): 31.

²⁹ Watt, *Building Pathology*, 113.

³⁰ Watt, *Surveying Historic Buildings*, 133-139.

³¹ James Douglas and Edward Noy, *Building Surveys and Reports*, 4th ed. (Hoboken: Wiley-Blackwell, 2011), 120.

³² Watt, *Building Pathology*, 117.

openings include: cracks, gaps, and joints in the structure.³³ The infiltration of moisture occurs between areas of differing moisture concentrations. The moisture in the high concentration zones will move to the areas of lower concentration to try and balance the concentrations.³⁴

Another excess moisture mechanism is rising damp. Rising damp is categorized by the upward movement of water in a wall from the ground through capillary action.³⁵ This failure mechanism is most commonly found in, but not limited to, historic masonry buildings. It is also the most common failure mechanism that affects buildings and it can cause degradation of materials, unsuitable indoor thermal–hygrometric conditions, poor thermal insulation efficiency in exterior walls, and a decrease in the mechanical performance of the wall.³⁶ Rising damp’s symptoms can often be confused with those of penetrating damp or condensation and can be misdiagnosed.³⁷ That is why finding the root cause of the excess moisture is key. Figure 4 demonstrates the capillary action that causes rising damp in masonry.

³³ Christopher Gorse et al., *A Dictionary of Construction, Surveying and Civil Engineering* (Oxford: Oxford University Press, 2012), 13.

³⁴ John Constance, “Calculating Building Moisture Infiltration,” *Plant Engineering*, 44, no. 13 (1990): 80.

³⁵ Gorse et al., 353.

³⁶ Elisa Franzoni, “Rising damp removal from historical masonries: A still open challenge,” *Construction and Building Materials*, 54 (2014): 124.

³⁷ Watt, *Surveying Historic Buildings*, 136.

Figure 4: Rising damp, caused by capillary action, in masonry.³⁸

Condensation, which is defined as the process of gas cooling down and becoming a liquid, can increase the water present on materials in a building and can be divided into surface and interstitial condensation”³⁹ Surface condensation refers to the condensation that forms on the surface of the material. To do so, the air at the surface of the material first needs to be at 100% relative humidity and the surface needs to be colder than the ambient air to form the water droplets. Interstitial condensation is when the dew point is reached within assemblies, such as a wall. The dew point is

³⁸ Franzoni, 125.

³⁹ Gorse et al., 88.

when the temperature in the air cools enough for condensation to form. For this to occur, the assembly has to have insulating materials and a change in temperature from one side of the assembly to the other that goes past the dew point for the given relative humidity.⁴⁰ High relative humidity within a building is often caused by the occupants.⁴¹ This can occur through activities like washing, cooking, and bathing. Natural or mechanical ventilation is needed to reduce the moisture in the air, therefore reducing both the chance of condensation and the excess moisture present in a building.⁴²

2.1.3. Chemical, Physical, and Biological Action

The third failure mechanism categorized by David Watt is chemical, physical, and biological action. This mechanism refers to the changes in materials due to external forces, such as other substances and even the natural environment.⁴³ Chemical actions are identified by the chemical reaction between a material and another substance. Reactions that cause decay can include corrosion, sulphate attack, and carbonation.⁴⁴ Corrosion can be described as “the undesirable deterioration' of a metal or alloy.”⁴⁵ It is the chemical reaction that turns a metal or alloy into rust or another substance, which can flake off or erode, thus reducing the structural and aesthetic performance

⁴⁰ Watt, *Building Pathology*, 118.

⁴¹ Douglas and Noy, 132.

⁴² Watt, *Surveying Historic Buildings*, 137.

⁴³ Watt, *Building Pathology*, 119.

⁴⁴ Ibid., 120-124.

⁴⁵ L.L. Shreir et al., *Corrosion*, 3rd ed, Vol. 1-2 (Oxford: Butterworth-Heinemann, 2000), 1:5.

of the material.⁴⁶ Metals may corrode due to exposure to a wide variety of substances. Atmospheric gases, pollutants, soil, water, acids, and alkalines are some of the external conditions that can cause corrosion.⁴⁷ Sulphate attack can affect clay, stone, cement, and concrete materials. The tri-calcium aluminate in these materials reacts with water-soluble sulphates to form hydrated calciumsulphoaluminate, also called ettringite, which is larger than tri-calcium aluminate.⁴⁸ This expansion leads to the breakdown and failure of the material, which in turn decreases its structural strength and deteriorates its finish.⁴⁹ Carbonation affects concrete materials, which contain calcium hydroxide. The calcium hydroxide makes the concrete alkaline, which acts as a chemical protection for the embedded steel reinforcement.⁵⁰ Carbonation occurs when acidic atmospheric oxides, such as carbon dioxide, come in contact with the calcium hydroxide through pores and cracks in the material. The reaction between these two elements decreases the alkalinity of the concrete, decreasing the protection of the embedded steel. The steel can then start to corrode, which in turn causes cracking and spalling in the concrete.⁵¹ Figure 5 is an image of a corroded steel rebar that has caused significant spalling in the concrete.

⁴⁶ Watt, *Building Pathology*, 120-121.

⁴⁷ J.R. Davis, *Corrosion: Understanding the Basics* (Materials Park: ASM International, 2000), 193.

⁴⁸ Watt, *Surveying Historic Buildings*, 146.

⁴⁹ Gorse et al., 429.

⁵⁰ Watt, *Surveying Historic Buildings*, 142.

⁵¹ Watt, *Building Pathology*, 124.

Figure 5: Corroded exposed steel rebar in concrete.⁵²

In contrast to chemical actions, physical actions do not produce new compounds, but instead change the physical characteristics and performance of the material. The most common of these are thermal movement, moisture movement, and the crystallisation of soluble salts. Other less common physical actions include light, sound, electricity, and magnetism.⁵³ Thermal movement relates to the expansion and contraction of a material due to temperature changes. Most materials expand with heat and contract with cold temperatures. These changes in temperature can occur from day to night and over seasons.⁵⁴ Moisture movement relates to the change in size of a material due to its moisture content. The more moisture a material absorbs, the larger it will expand. Materials generally then contract as the moisture evaporates.⁵⁵ Both thermal

⁵² Aali Alizadeh, “A New Technique for Rebar Corrosion Rate Measurement in Concrete,” *LinkedIn*, (5 July 2016). <<https://www.linkedin.com/pulse/new-technique-rebar-corrosion-rate-measurement-aali-r-alizadeh>> [8 November 2017].

⁵³ Watt, *Building Pathology*, 125.

⁵⁴ *Ibid.*

⁵⁵ *Ibid.*, 127.

and moisture movement can cause cracks in the material due to these expansion and contraction cycles. Within a building, these cracks tend to form at the weakest points of the structure, such as the door and window frames.⁵⁶ Frost damage, which was described under atmospheric and climatic action, can also be considered a combination of moisture and thermal movement.⁵⁷

The crystallisation of soluble salts can be divided into external and internal crystallisation. Efflorescence is the external crystallisation of soluble salts. It occurs when water containing soluble salts within a material starts to evaporate on its surface. As the water evaporates, the salts are left behind, forming the growth of crystals on the surface.⁵⁸ Subflorescence, sometimes referred to as crypto-efflorescence, is the internal crystallisation of soluble salts.⁵⁹ These crystals form within porous materials, when the evaporation of water is much faster than the movement of the soluble salts to the surface of the material. The creation of crystals within the material creates internal stresses and can cause the failure and disintegration of the surface layer.⁶⁰ Subflorescence can be hard to find, but efflorescence can be an indicator that it may be present. The most common soluble salts are nitrates, chlorides, and sulphates that typically come from the ground or groundwater. Chlorides are also found within de-icing salts that are used in Canadian winters to melt the ice on pavements and

⁵⁶ Douglas and Noy, 82.

⁵⁷ Watt, *Surveying Historic Buildings*, 148.

⁵⁸ Harris, 391.

⁵⁹ Monuments and Sites XV, *ICOMOS-ISCS: Illustrated Glossary on Stone Deterioration Patterns*, (Champigny: Monuments and Sites XV, 2008), 48.

⁶⁰ Watt, *Building Pathology*, 129.

walkways.⁶¹ Some soluble salts are also hygroscopic. This means that they can attract moisture from the air, increasing the moisture content within a material.⁶² Figure 6 showcases efflorescence on sandstone masonry.

Figure 6: Efflorescence on sandstone.⁶³

Biological actions refer to effects of living organisms on building materials. These range from small microscopic organisms, like moulds, to larger organisms, such as mammals and trees.⁶⁴ However, the main culprits of decay are fungi and insect infestation, both of which affect timber. Fungi infestation starts off with its spores landing on a damp timber surface. It then starts to feed on the timber causing it to rot and decay.⁶⁵ There are five conditions that are needed for timber rot to occur. They

⁶¹ Watt, *Surveying Historic Buildings*, 147.

⁶² Watt, *Building Pathology*, 129.

⁶³ *Monuments and Sites XV*, 49.

⁶⁴ Watt, *Surveying Historic Buildings*, 148.

⁶⁵ *Ibid.*, 153.

are the presence of fungi, moisture, oxygen, heat, and for the timber to be free of toxins. If one of these is removed, the timber rot is stopped.⁶⁶

There are two types of timber rot: dry and wet rot. Though it is called dry rot, the timber still needs to have a moisture content of about 20% for the rot to occur.⁶⁷ Wet rot is common with high moisture, typically over 25%, and occurs in localised areas since the fungi causing the destruction cannot penetrate past masonry.⁶⁸ The type of rot can then be classified further as brown and white rot. Brown rot is more common and causes the timber to crack and dry out, while the white rot typically attacks hardwoods that have a higher moisture content and leaves behind white fibres.⁶⁹

Wood-boring insects start off with their eggs being laid on or near timber. The larva then feeds on the timber decreasing its structural performance. It can spend from one to eleven years inside the timber before it forms a pupal chamber at the surface and emerges as an adult beetle. As it emerges, it leaves behind holes in the timber and wood dust.⁷⁰ Apart from beetles, insects such as weevils, wasps, borers, and termites can damage both structural and non-structural timbers.⁷¹ Figure 7 showcases the damage to roof timber by deathwatch beetles.

⁶⁶ Harris, 22.

⁶⁷ Watt, *Building Pathology*, 130.

⁶⁸ Douglas and Noy, 138.

⁶⁹ Watt, *Surveying Historic Buildings*, 153.

⁷⁰ Ibid., 156.

⁷¹ Watt, *Building Pathology*, 134.

Figure 7: Roof timber damaged by deathwatch beetles.⁷²

Plants, both small and large, can also pose a risk to buildings. This can include damaging foundations and underground services due to penetrating root growth and damaging the exterior by direct contact with braches, or through the roots of climbing plants and creepers.⁷³ Wild birds and mammals can also cause destruction to the building fabric. These animals can chew on electric wiring and thermal insulation, they can block drainage systems, and they can destroy structural components for nesting purposes. They can also stain the building's appearance through their excrement and harm the occupants since they can carry diseases.⁷⁴ Microscopic organisms, such as mould, can also cause harm to the occupants that come into

⁷² Ibid., 136.

⁷³ Ibid., 130.

⁷⁴ Chartered Institute of Environmental Health, *Pest minimisation: Best practice for the construction industry* (n.d.) <http://www.urbanpestsbook.com/downloads/Best_practice_for_the_construction_industry.pdf> [10 November 2017], 3.

contact with them. They can also be a sign of excess moisture issues since they tend to grown on damp internal surfaces.⁷⁵

It is also important to note that combining various materials during the construction of a building that have different physical and chemical properties and characteristics may cause unwanted reactions. For example, if a mortar is used in a masonry wall that is much more rigid than the stone, it can cause the stone to crack or break off when it expands and contracts at a different rate than the mortar. It is important to think about the compatibility and appropriateness of the materials in the selection, specification and application process. Though these effects are not often seen in the short-term, they can cause considerable long-term deterioration.⁷⁶

2.1.4. Movement

The movement of buildings can be caused by a number of factors. This can include movement on a molecular level, movement caused by the building's own features and loads, and movement due to external factors.⁷⁷ One of the smallest forms of movement is caused by temperature changes and moisture levels, as explained previously under physical action.⁷⁸ On a larger scale, the change of use of a building can bring about increased loadings, which may cause movement due to the now

⁷⁵ Watt, *Building Pathology*, 132.

⁷⁶ Watt, *Surveying Historic Buildings*, 158.

⁷⁷ Watt, *Building Pathology*, 137.

⁷⁸ Watt, *Surveying Historic Buildings*, 160-161.

inadequate structural elements.⁷⁹ An increase in loading can also cause settlement to occur, which is the downwards movement or compression of the soil underneath the building foundations. Settlement is most common during the early life of a building, but may occur with the addition of loads on or in the building. However, loadings are not the only cause of settlement; it may also occur if the groundwater level lowers. As it lowers, the support it gave to the surrounding soil diminishes and may cause it to redistribute, shifting the support for the building as well.⁸⁰

Another downward movement of a building and its foundation can be caused by subsidence. Subsidence is caused by “the loss of support of the site beneath the foundations,” which is typically associated with “volumetric changes in the subsoil.”⁸¹ Damage to a building by subsidence is often caused by different rates of settlement across the foundation of a building. This difference causes cracks and, at times, the collapse of the structure.⁸²

Movement of a building due to movements of the ground does not always happen in a downwards direction. Heave is the “volumetric expansion of the ground beneath part or all of the building.”⁸³ Some causes of this expansion can include the upwards force

⁷⁹ Watt, *Building Pathology*, 137.

⁸⁰ Gorse et al., 382.

⁸¹ Watt, *Building Pathology*, 138.

⁸² Gorse et al., 428.

⁸³ Watt, *Building Pathology*, 139.

of growing adjacent tree roots⁸⁴ or the freezing of the subsoil in the winter.⁸⁵ The ground itself may also slope over time, sloping the building as well, causing what is called landslip. This sloping movement may be gradual over time or sudden.⁸⁶ Some causes of landslip include groundwater regime changes, unsuitable geometry, and additional loading. The four common types of landslips are falls, transitional slides, rotational slips, and flows.⁸⁷

The movement caused by earthquakes can cause liquefaction of the soil. Liquefaction is when the movement of the soil causes it to soften up and act more like a liquid than a solid. This can drastically move a building if it affects the soil supporting it.⁸⁸

Finally, a building can also exhibit movement deformation due to the failure of its own foundation. This can be due to the failure of the footing, foundation arches, timber piles, or a chemical attack on the concrete.⁸⁹ Figure 8 shows a residential home in Florida that has fallen victim to extreme subsidence.

⁸⁴ Watt, *Surveying Historic Buildings*, 165.

⁸⁵ Gorse et al., 175.

⁸⁶ Watt, *Building Pathology*, 139.

⁸⁷ Gorse et al., 245.

⁸⁸ Jonathan Bray and Shideh Dashti, “Liquefaction-induced building movements,” *Bulletin of Earthquake Engineering*, 12 (2014): 1129.

⁸⁹ Watt, *Building Pathology*, 139.

Figure 8: Extreme subsidence of residential home in Florida⁹⁰

2.1.5. Fire

Fire has been a significant destructive factor for buildings throughout history. Design choices involving the use of non-combustible materials and fire-resistant forms of construction have been implemented to try to diminish the severity of damage caused by fire.⁹¹ Fire occurs when fuel or a combustible material reacts with heat, at the ignition temperature or higher, and oxygen. It begins with ignition, then goes through the smouldering stage, before beginning the flaming stage of development.⁹² Most accidental fires begin with electrical defects.⁹³ This can include exposed wiring and overloaded circuits. These defects can cause a spark, giving the needed momentum to

⁹⁰ National Geodetic Survey, *Subsidence* (25 February 1997).
<<https://www.ngs.noaa.gov/GRD/GPS/Projects/CB/SUBSIDENCE/subsidence.html>> [10 November 2017].

⁹¹ Watt, *Building Pathology*, 139.

⁹² Watt, *Surveying Historic Buildings*, 165-166.

⁹³ J. Stanley Rabun and Richard Kelso. *Building Evaluation for Adaptive Reuse and Preservation*, (Hoboken: John Wiley & Sons, 2009), 103.

begin a fire.⁹⁴ Other methods of starting a fire can be the use of appliances that overheat, and arson.⁹⁵

Fires damage buildings through direct burning, indirect effects of heat and smoke, and through fire fighting actions.⁹⁶ The main decaying result of fires is structural damage.⁹⁷ This is caused by the breakdown of combustible materials, changes in loading distribution, and distortion of structural elements, mainly due to the heat of the fire. The smoke of a fire can also cause staining and possible chemical reactions with building elements.⁹⁸ One of the ways that fire fighting actions cause deterioration is through the use of water to extinguish the fire. This water can bring about excess moisture in the building and poses a risk for fungal infestation and mould growth, even after repairing damages.⁹⁹ Firefighters may also cause damage to the building through forced entry, clearing access routes, and removing materials from the building.¹⁰⁰ Figure 9 demonstrates the damage that fire can do in a residential kitchen.

⁹⁴ Canada's Historic Places, *Heritage Building Maintenance Manual*, 12.

⁹⁵ Watt, *Surveying Historic Buildings*, 165.

⁹⁶ Watt, *Building Pathology*, 140.

⁹⁷ Rabun and Kelso. 34.

⁹⁸ Watt, *Surveying Historic Buildings*, 166.

⁹⁹ Watt, *Building Pathology*, 140.

¹⁰⁰ Watt, *Surveying Historic Buildings*, 167.

Figure 9: Fire damage after cooking oil ignites.¹⁰¹

2.1.6. Human Factors

Humans' activities and behaviours have significant effects on buildings.¹⁰² Humans can cause deterioration by direct action or by neglect, such as: vandalism, poor-quality interventions, and the lack of maintenance.¹⁰³ Purposeful actions, like vandalism, are often hard to control and to avoid due to their unpredictability.¹⁰⁴ On the other hand, inappropriate interventions and poor choices in building materials are

¹⁰¹ Molly Hottle, "Elephant ears to blame for Clark County house fire that displaced family." *The Oregonian* (16 January 2011). < http://www.oregonlive.com/clark-county/index.ssf/2011/01/elephant_ears_to_blame_for_clark_county_house_fire_that_displaced_family.html > [11 November 2017].

¹⁰² Watt, *Building Pathology*, 142.

¹⁰³ Rocío Ortiz and Pilar Ortiz, "Vulnerability Index: A New Approach for Preventive Conservation of Monuments," *International Journal of Architectural Heritage* 10 (2016): 1086.

¹⁰⁴ Watt, *Building Pathology*, 142.

typically not done purposefully and may not become evident immediately. However, they can open the doors to long term deterioration.¹⁰⁵

All buildings deteriorate over time, but through maintenance work this process can be significantly slowed. However, people's attitude towards maintenance is often negative, as it is perceived as additional work for little to no visible results. Maintenance is often conducted to repair significant damage, rather than to proactively reduce potential decay.¹⁰⁶ If people could shift their views on maintenance, buildings would last longer, and less money would need to be spent.

2.2. Types of Maintenance

Maintenance is defined as “the combination of all technical and associated administrative actions intended to retain an asset in or bring it to a state in which it can perform its required function”.¹⁰⁷ In simpler terms, it is about maintaining a building in a chosen condition over a long period of time.¹⁰⁸ Since all buildings deteriorate over time, maintenance needs to be a continuous process to be able to maximize the life of a building.¹⁰⁹ Regular maintenance can also cut costs in the long term by reducing occurrence of heavily decayed building elements which require

¹⁰⁵ Watt, *Surveying Historic Buildings*, 130.

¹⁰⁶ Watt, *Building Pathology*, 142.

¹⁰⁷ Royal Institution of Chartered Surveyors, *Building maintenance: strategy, planning and procurement*, 2nd ed. (Coventry: Royal Institution of Chartered Surveyors, 2009), 6.

¹⁰⁸ Housing and Building Research Institute, “Chapter 5: Maintenance Management, Repairs, Retrofitting and Strengthening of Buildings,” *Bangladesh National Building Code 2012* (2012). <http://pwd.gov.bd/document/library/Final_Part_7-Chap_5_new.pdf> [14 November 2017], 67.

¹⁰⁹ Canada's Historic Places, *Heritage Building Maintenance Manual*, 3.

costly repair work to mitigate. Though maintenance is not free, the modest spending required ends up being less than that for large repair work.¹¹⁰

Maintenance can be broken down into two categories: preventive and corrective. At a glance, these categories can be summarized as retaining and restoring work, respectively.¹¹¹ Koenraad van Balen compares the maintenance of buildings with medical services for people. As such, corrective maintenance can be compared to things like taking medication for an illness or treating a cut or broken arm, while preventive maintenance can be compared to living a healthy lifestyle by eating well and exercising.¹¹²

2.2.1. Corrective Maintenance

Corrective maintenance is the work needed to repair a deteriorated building or material to an accepted level of performance.¹¹³ This type of maintenance is performed after an element has become defective. One must first see the defect before the corrective maintenance work can be decided upon. The objective is to remove the defect so that the element can function as intended.¹¹⁴ At times, repair of the element

¹¹⁰ Ibid., 5.

¹¹¹ Royal Institution of Chartered Surveyors, 6.

¹¹² Koenraad Van Balen, “Preventive conservation of historic buildings,” *Restoration of Buildings and Monuments: Materials science, Conservation of architectural heritage, Sustainable construction* (n.d.) <http://www.changes-project.eu/wp-content/uploads/2016/01/RBM-2015_Preventive-Conservation-of-historic-buildings.pdf> [11 September 2017], 2.

¹¹³ Canada’s Historic Places, *Heritage Building Maintenance Manual*, 5.

¹¹⁴ Christer Stenström et al., “Preventive and corrective maintenance – cost comparison and cost–benefit analysis,” *Structure and Infrastructure Engineering*, 12, no. 5 (2016): 603.

is not necessarily needed. In such cases, the building element must still be stabilized to stop further deterioration. This still falls under corrective maintenance work.¹¹⁵

There is also some corrective maintenance that could be categorized as ‘emergency maintenance’. This work covers repair jobs that need to be done immediately for health, safety, and security factors or that could cause rapid decay of the structure or fabric of a building.¹¹⁶ These can be done after vital features of a building break, after a large storm or natural disaster, or after discovering an issue during an inspection. Inspection related issues can include finding fire hazards, such as improper wiring or overloaded electrical circuits, or hazards that could cause personal injury, such as broken steps, loose building elements, and asbestos.¹¹⁷

2.2.2. Preventive Maintenance

Preventive maintenance, sometimes referred to as planned maintenance, is work done at predetermined increments that aim “to reduce the probability of failure or the degradation of items.”¹¹⁸ This means that it must be planned, yet constantly updated to ensure that new information, often found through inspections and monitoring, can be incorporated in the maintenance decisions.¹¹⁹

¹¹⁵ Canada’s Historic Places, *Heritage Building Maintenance Manual*, 10.

¹¹⁶ *Ibid.*, 5.

¹¹⁷ Canada’s Historic Places, *Heritage Building Maintenance Manual*, 12.

¹¹⁸ Christer Stenström et al., 603.

¹¹⁹ Watt, *Building Pathology*, 252.

PRECOMOS (Preventive Conservation, Monitoring and Maintenance of Monuments and Sites), which is under UNESCO (United Nations Educational, Scientific and Cultural Organization) agency, splits preventive maintenance into primary, secondary, and tertiary prevention. Primary prevention works “to avoid the causes of the unwanted effect”, such as loss of heritage value or damage.¹²⁰ Secondary prevention covers the monitoring of symptoms associated with ‘unwanted effects’ to allow for early detection of the issue.¹²¹ Finally, tertiary prevention works to stop the spread of the ‘unwanted effects’ and to stop the generation of new unwanted or side effects.¹²²

2.3. Historic Structure Report

The identification of defects and the creation of maintenance plans for buildings are recorded and typically presented in a historic structure report for historic buildings. The purpose of such a report is to provide “documentary, graphic, and physical information about a property's history and existing condition.”¹²³ Therefore, the production of a historic structure report is typically the first phase of work, preceding design and construction work on an existing building. These reports are often commissioned in anticipation of work done to a building. This can include repair work, constructing additions, or changing the use of a site. Depending on the reason,

¹²⁰ PRECOMOS, *What is Preventive Conservation?* (n.d.). <<http://precomos.org/>> [7 September 2017].

¹²¹ Koenraad Van Balen, 4.

¹²² PRECOMOS.

¹²³ Deborah Slaton, “The Preparation and Use of Historic Structure Reports,” 43 *Preservation Briefs*. (April 2005). <<https://www.nps.gov/tps/how-to-preserve/preservedocs/preservation-briefs/43Preserve-Brief-StructureReports.pdf>> [7 September 2017], 1.

the report can hone in on building elements that would be affected by the decided work.¹²⁴ The basis of creating these reports is conservation-based research and analysis. This entails “the research, analysis, survey and investigation necessary to understand the significance of a building and its landscape, and thus inform decisions about repair, alteration, use and management.”¹²⁵

Figure 10: Diagram showcasing the cycle and relationship of heritage information.¹²⁶

The cycle of such a project typically follows a common pattern. Robin Letellier highlights this cycle with the flowing stages: initiating the project; planning the

¹²⁴ Ibid., 4.

¹²⁵ Watt, *Surveying Historic Buildings*, 23.

¹²⁶ Robin Letellier et al., *Recording, Documentation, and Information Management for the Conservation of Heritage Places: Guiding Principles*, (Los Angeles: Getty Conservation Institute, 2007), <http://www.getty.edu/conservation/publications_resources/pdf_publications/pdf/guiding_principles.pdf> [11 September 2017], 12.

project and studying options; selecting an option and preparing contractual documents; executing and controlling the work; closing the project; and monitoring and evaluating the project.¹²⁷ Figures 10 and 11 showcase variations of this cycle and Figure 11 briefly describes the necessary documents at each stage.

Figure 11: Diagram showcasing the general heritage conservation process.¹²⁸

¹²⁷ Ibid., 22.

¹²⁸ Ibid., 26.

In general, historic structure reports begin with a brief introduction of the site and a summary of the overall findings. The report is then typically split into two sections. The first section begins with the history of the building, its physical characteristics, its existing condition, and a description on its significance. The second section then focuses on the objectives of the work to be conducted and some recommendations of the treatment work to be done. The report then ends with a conclusion, references, and appendices.¹²⁹ However, though this is the general pattern, reports tend to format the information differently depending on the company that writes them. Though they tend to all have the same information, the order or headings may differ. For example, James Douglas and Edward Noy showcase a format for the report where the headings are:

1. Introduction
 - a. Instructions
 - b. Description of Property
 - c. Survey/investigation details
2. Investigation and Analysis
 - a. Condition
 - b. Diagnosis
 - c. Prognosis
3. Conclusions and Recommendations
 - a. Conclusions
 - b. Recommendations

¹²⁹ Slaton, 14.

4. Appendixes
 - a. Location plan of property
 - b. Photographs of defects
 - c. Schedule of costs¹³⁰

Deborah Slaton showcases another format for the report with the following headings:

1. Introduction
 - a. Study summary
 - b. Project data
2. Development History
 - a. Historical background and context
 - b. Chronology of development and use
 - c. Physical description
 - d. Evaluation of significance
 - e. Condition assessment
3. Treatment and Work Recommendations
 - a. Historic preservation objectives
 - b. Requirements for work
 - c. Work recommendations and alterations
4. Notes, Bibliography and Appendixes
5. Supplemental Record of Work Performed.¹³¹

¹³⁰ Douglas and Noy, 317-318.

¹³¹ Slaton, 15.

2.4. Preliminary Work

The preliminary work required for a historic structure report includes historical research and writing a statement of significance for the site. This work is typically done in the office before going on site, though there are times when a brief site visit can assist with these tasks.

2.4.1. Historical Research

Historical research helps to understand the building and the site. As such, it is the first step of a project and involves reviewing and verifying existing documentation and reports.¹³² Ideally, all documentation should be examined before surveying the building to understand its significance, its evolution, and the past work conducted. It can also help to understand the possible tasks that may be required for the project.¹³³ However, such an inclusive research task may be too overbearing for the scope of the project. It is at times more efficient to review selective information on the evolution of the structure, its significance, and the justification for the possible treatment options.¹³⁴ When researching the building's significance, it is important to include social, political, economic, and cultural factors that affect the site and its surroundings.¹³⁵ When gathering information to be used to write the Statement of Significance, it is important to look into the historical, architectural, engineering, and

¹³² Letellier et al., 13.

¹³³ Douglas and Noy, 24.

¹³⁴ Slaton, 7.

¹³⁵ Letellier et al., 11.

cultural significance of the property, as well as the construction, use, and occupants that have been associated with it.¹³⁶

2.4.2. Statement of Significance

To best plan the work to be done on a building, whether it is new construction or maintenance, it is vital to understand the value a building may hold and what characterizes that value. This information helps to identify what should be protected in order to minimize the possible damage.¹³⁷ A Statement of Significance helps to organise this information and it can also be used as a planning and property management tool for both the property owners and the designers.¹³⁸ Though people tend to believe that only registered buildings have value, structures and places of all kinds can gain value or character over time through their intrinsic quality, continued existence, and familiarity. As such, it is good to record identifying and unique features of a building, as well as its use over time.¹³⁹ Though a Statement of Significance is required for a building to be added to the Canadian Register of Historic Places, it can also be used as a format for recording the value of a site and building.¹⁴⁰ If a building is already registered, its Statement of Significance should

¹³⁶ Slaton, 10.

¹³⁷ Canada's Historic Places, *Heritage Building Maintenance Manual*, 9.

¹³⁸ Parks Canada, *Canadian Register of Historic Places: Writing Statements of Significance*, (2016). <<http://www.historicplaces.ca/media/5422/sosguideen.pdf>> [19 November 2017], 4.

¹³⁹ Department of Arts, Heritage and the Gaeltacht, *Architectural Heritage Protection: Guidelines for Planning Authorities*, (Dublin: Stationary Office, 2011), 13.

¹⁴⁰ Parks Canada, 1.

already be available.¹⁴¹ Then all that needs to be done is to review the statement to ensure that information is valid and up-to-date.¹⁴² Statements of Significance follow a values-based approach and are also used in Australia, New Zealand, Scotland, Ireland, and England.¹⁴³

A Statement of Significance is broken up into three sections: Description of Historic Place, Heritage Value, and Character-defining Elements. When following the specific requirements of the Canadian Register of Historic Places, the first two sections are to be written in narrative form, while the Character-defining Elements section should be in point form. The overall statement should also not exceed 4000 characters.¹⁴⁴ However, if the statement is not written for the purpose of registration, these rules can be taken rather as guidelines. In the Description of Historic Place section, the site and building's details are stated. It is not meant to be a chronology of the building's life, but the main time periods of interest, such as original construction date and registration date, and a brief description of its uses over time can help to get a sense of the physical character of the place. The Heritage Value section focuses on what makes the building valuable in the grand scheme of things.¹⁴⁵ This should not be small aspects of the building, but rather significant themes or events associated with it. Areas of interest can be categorised under the following values: architectural,

¹⁴¹ Canada's Historic Places, *Heritage Building Maintenance Manual*, 9.

¹⁴² Slaton, 11.

¹⁴³ Parks Canada, 4.

¹⁴⁴ *Ibid.*, 4-5.

¹⁴⁵ *Ibid.*, 10.

historical, archaeological, artistic, cultural, scientific, technical, and social.¹⁴⁶ The specific items that contribute to the building's heritage value are listed in the Character-defining Elements section. This section is most vital to the owner and to people, such as managers and designers, as it gives a key list of what should be protected.¹⁴⁷

If the building is not designated and its character-defining elements are not listed, a brief physical survey can be conducted to determine them. What should be looked at are the building's overall features, such as its form and scale, the immediate site and landscaping that can affect the building, the materials used both on the exterior and interior, and the features, finishes, and details of the building.¹⁴⁸ It is also good to consider the typical categories that character-defining elements are listed under in Statements of Significance. They are: style, scale, massing and composition; interior layout or exterior spatial configuration; functional features; materials and craftsmanship; the interaction of the historic place and its surroundings; the use of the building; and customs and traditions that are or were associated with it.¹⁴⁹ J. Stanley Rabun and Richard Kelso also made a list of elements that add architectural character to the building. They are: the footprint on the site; form, massing, rhythm, and symmetry; materials, primary and secondary, type, texture, and color; craftsmanship; degree of articulation, details, and finishes; color and finishes; roof type; windows,

¹⁴⁶ Department of Arts, Heritage and the Gaeltacht, 24.

¹⁴⁷ Parks Canada, 15.

¹⁴⁸ Canada's Historic Places, *Heritage Building Maintenance Manual*, 9.

¹⁴⁹ Parks Canada, 15.

also looking into their patterns and reveals; solids-to-voids ratio of walls; and doors.¹⁵⁰

2.5. Surveying

The surveying aspect of a project is about getting data on the building in question. Though there are times that the survey would focus on a specific element of a building, such as windows or roof coverings, there is an increasing need to consider the building as a whole rather than as separate parts.¹⁵¹ One reason why surveys are conducted is for posterity; if existing or historic buildings were to collapse or be demolished, there would still be a document of some sort on the properties and construction details of the building.¹⁵² Other times, it is just to record the current condition of the building either for upcoming maintenance or construction projects. The purpose of a survey is to prepare measured drawings of the site, prepare a condition report describing the defects present, and to advise on possible repair work needed for the upkeep of the building.¹⁵³ When conducting a survey of a building, the following items need to be covered: exterior and interior materials, features, and finishes; structural systems; interior spaces; mechanical, electrical, and plumbing systems; and fire detection and security systems.¹⁵⁴ A typical survey consists of the following steps: preliminary walk through of the building, recording the building

¹⁵⁰ Rabun and Kelso, 21.

¹⁵¹ Watt, *Surveying Historic Buildings*, 346.

¹⁵² Letellier et al., vii.

¹⁵³ Douglas and Noy, 9.

¹⁵⁴ Slaton, 8.

through photography, determining the current condition, creating measured drawings, determining the appropriate treatment for the building, and developing a set of work recommendations for the building.¹⁵⁵ Figure 12 showcases the main stages of a survey identified by James Douglas and Edward Noy.

Figure 12: Diagram showcasing the main stages of a survey.¹⁵⁶

¹⁵⁵ Ibid., 6.

¹⁵⁶ Douglas and Noy, 23.

2.5.1. Types of Surveys

Surveys can be either intensive or extensive. Intensive surveys are conducted at a high-resolution level. These are needed when a building requires detailed investigations or if it is about to change. Extensive surveys are conducted at a low-resolution level. The purpose of these surveys is to document a group of buildings to give a general description of their condition and required repair and maintenance work.¹⁵⁷ Besides dividing based on resolution, surveys can be divided by type. The four main types of surveys are: measured, valuation, archaeological, and building surveys. As its title suggests, measured surveys produce measured drawings on the building's layout, construction, and appearance. Valuation surveys focus on the value of a building in a variety of sectors. This can include heritage, acquisition, compensation, disposal, investment, insurance, and taxation. Archaeological surveys cover remains, both human and material, of earlier societies. They can be found in standing structures, ruins, and below ground. Finally, building surveys focus on the structure and fabric of the building itself.¹⁵⁸ The final way to categorise a survey is based on duration. This can be split into synchronic and diachronic surveys. A synchronic survey can be defined as "a snapshot assessment of a building and the way it all fits together at a particular moment in time."¹⁵⁹ One type of survey that is synchronic is a condition survey, as it captures the condition of the building at one

¹⁵⁷ Watt, *Surveying Historic Buildings*, 8.

¹⁵⁸ *Ibid.*, 1.

¹⁵⁹ Douglas and Noy, 3.

point in time.¹⁶⁰ A diachronic survey can be defined as “a way of studying buildings in terms of how they change or evolve over time.”¹⁶¹ A common diachronic survey is the quinquennial survey that is done every five years. The purpose of these surveys is to conduct condition assessments to monitor the changes and needs of the structure and its services.¹⁶² As such, diachronic surveys also include maintenance surveys and conservation plan inspections.¹⁶³

2.5.2. Inspection Process

The intent of an inspection is to get a grasp and an understanding of the property. This helps to later plan the recording process and the objectives of the project.¹⁶⁴ The inspection should first start with reconnaissance. This includes understanding the basics of the building, such as the hazards, access limitations, and the building layout. From there, the inspection can begin. David Watt suggests that the order of inspection should begin with the site and environment.¹⁶⁵ This should be followed by looking at the exterior of the building, before moving onto the interior. The final stage of the investigation is to look at the building services. The same approach to the inspection should be carried out for each item. For example, if for one façade the inspection begins on the left hand side, all the other façade inspections should also begin on the

¹⁶⁰ Royal Institution of Chartered Surveyors, 24.

¹⁶¹ Douglas and Noy, 3.

¹⁶² Watt, *Surveying Historic Buildings*, 23.

¹⁶³ Douglas and Noy, 3.

¹⁶⁴ Canada's Historic Places, *Heritage Building Maintenance Manual*, 10.

¹⁶⁵ Watt, *Surveying Historic Buildings*, 75.

left hand side. This insures consistency, which makes it easier to organise the findings and ensure that nothing has been missed during the inspection.¹⁶⁶

2.5.3. Identifying Defects

Identifying defects is the first step in a condition assessment and in determining the treatment of the defects. The process includes conducting a pathology of the defects and a damage analysis.¹⁶⁷ The definition of pathology is “the systematic study of diseases with the aim of understanding their causes, symptoms and treatment.”¹⁶⁸ In the case of building pathology, the diseases are the defects within its fabric. Building pathology is founded on “detailed knowledge of how a building is designed, constructed, used and changed, and the various mechanisms by which its structural, material and environmental conditions can be affected.”¹⁶⁹ As so many factors come into play in building pathology, a ‘cookbook’ type method of conducting interventions is not necessarily the answer. Having a list of what treatments have worked for various symptoms can be a useful start. However, without properly delving into why the symptoms are occurring, the issue may not be mitigated and, at times, may get worse.¹⁷⁰ David Watt states that for defects to be properly identified, one must have a “good observation to discover symptoms and establish the nature of

¹⁶⁶ Rabun and Kelso. 34.

¹⁶⁷ Aziliz Vandesande, *Preventive Conservation Strategy: For Built Heritage Aimed At Sustainable Management And Local Development*, (Leuven: KU Leuven, 2017), 74.

¹⁶⁸ Watt, *Building Pathology*, 1.

¹⁶⁹ Ibid., 7.

¹⁷⁰ Harris, 39.

the construction, and a thorough understanding of the mechanisms of deterioration and decay.”¹⁷¹ When identifying the defects in a building, it is imperative to find any possible defects or issues that could pose current or future safety risk, the loss of historic elements, or a decrease in the building’s or element’s performance.¹⁷²

When assessing the building for defects, the findings can be split into two functional groups: passive and active. Passive elements focus more on the physical fabric of the building and can be broken down in structural, horizontal, and vertical system categories. Active elements focus on changing elements and can be broken down into utility, environmental, and protective system categories.¹⁷³ Defects can become evident to occupants through the following indications: visual, physical, olfactory, aural, and tactile.¹⁷⁴ Once the symptoms of a possible defect are detected, an investigation should be conducted to determine the cause of the defect. The inspection ends with a defect diagnosis of the issue. David Watt has highlighted some typical questions that should be asked during the defect identification process.¹⁷⁵ They are broken up in the three stages and are as follows:

- “Symptoms
 - How does the defect manifest itself?
 - Do the symptoms change or are they constant?

¹⁷¹ Watt, *Surveying Historic Buildings*, 79.

¹⁷² Slaton, 11.

¹⁷³ Harris, 28.

¹⁷⁴ Watt, *Building Pathology*, 164.

¹⁷⁵ *Ibid.*, 166.

- Are the symptoms getting worse?
- Investigation
 - What is the extent of the defect and could it affect other parts of the building?
 - Are the symptoms relevant to one or more possible defects?
 - Could the cause of the defect be remote from the symptom?
- Diagnosis
 - What is the cause of the defect?
 - Can the defect be attributed to two or more causes?
 - Is the defect static or progressive?
 - Is further action required to diagnose the defect?”¹⁷⁶

Figure 13 showcases a decision tree on how to identify and diagnose a building defect.

¹⁷⁶ Ibid.

Figure 13: "Decision tree for the diagnosis of defects."¹⁷⁷

¹⁷⁷ Ibid., 167.

2.5.4. Data Acquisition

Documenting a building is an essential stage in creating a historic structure report and provides the basis for planning maintenance, monitoring, and management strategies.¹⁷⁸ The first step in documenting is data acquisition. This entails recording the as-found condition of the building.¹⁷⁹ Recording can be broken down into three levels: reconnaissance, preliminary, and detailed recording. These levels dictate the level of detail needed. Reconnaissance recording is the lowest level of recording and its typical output is a photo survey with sketched plans. This type of recording is best used to get an idea of how the site is configured and what are its important characteristics. A higher level of detail can be achieved through preliminary recording, since its output includes measured drawings. The accuracy of measurements for a preliminary recording varies depending on the type of drawing. For plan, elevation, or cross-section drawings, the accuracy should be ± 10 cm, while for structural and other detailed elements, it should be ± 2 cm.¹⁸⁰ The highest level of detail is obtained in a detailed recording. This type of recording is most commonly used when a structure will undergo some type of intervention. It provides an accurate record of the site's physical configuration, condition, and significant features. The accuracy of plan,

¹⁷⁸ Jessica Kottke et al., "Terrestrial Laser Scanning: Imaging, Quantifying, and Monitoring Microscale Surface Deterioration of Stone at Heritage Sites," *Change Over Time* 1, no. 2 (2011): 269.

¹⁷⁹ Letellier et al., 27.

¹⁸⁰ Ibid., 36.

elevation, and cross-section drawings should fall within the range of $\pm 10\text{-}25$ mm, and $\pm 2\text{-}5$ mm for building element details.¹⁸¹

What framework is recommended for the development of levels of heritage recording?

Levels of Heritage Recording			
	A Reconnaissance Record Low Accuracy	B Preliminary Record Midrange Accuracy	C Detailed Record High Accuracy
Purpose of Recording	Reconnaissance Initial inventory Initial planning Reference data	Planning Initial condition Investigation Stabilization Pre-design Reference data	As-found condition Design Construction As-built record Maintenance/monitoring Posterity
Accuracy of Drawings	Not to scale	Plans and elevations ± 5.0 in. (± 10 cm) Details ± 1.0 in. (± 2 cm)	Plans and elevations ± 0.5 in. (± 1 cm) Details ± 0.1 in. (± 2 mm)
Results	Photographic report Photo-key plan Initial condition Descriptive sketches	Measured drawings Asset description/condition Observations Photographic report	Measured drawings Asset description/condition Observations Photographic report
Cost (will vary in absolute terms with scale and complexity of site)	Low (a few days on site by recording team)	Moderate (several weeks or more on site by heritage recording team and input by conservation professionals)	Moderate to high (extensive and possibly ongoing activity on site by recording team and increased input by conservation professionals)

Figure 14: Comparison between reconnaissance, preliminary, and detailed recording levels.¹⁸²

As seen in Figure 14, photography plays a pivotal role in all levels of recording. It is one of the oldest non-destructive imaging approaches that is still heavily used not only in the heritage sector, but in everyday life.¹⁸³ Photography is an essential tool in

¹⁸¹ Ibid., 38.

¹⁸² Ibid., 40.

¹⁸³ Efstratios Stylianidis and Fabio Remondino, ed, *3D Recording, Documentation and Management of Cultural Heritage*, (Dunbeath: Whittles Publishing, 2016), 127.

creating a visual record of a building and its site. Its use can range from showcasing the context of a building to showcasing close-up details of an element at a particular point in time.¹⁸⁴

As the level of detail increases, the need for measurements becomes vital. The oldest method is through hand measurements. This technique is best used on small and non-complex buildings and element details. Though the cheapest method of obtaining measurements, it is time-consuming, has no method of creating a coordinate reference system, and is limited in the precision that can be acquired.¹⁸⁵ Another method of obtaining measurements is through the use of a total station. It is a surveying instrument that is used to determine the x, y, and z position of a point within a set reference system. This system must first be georeferenced, which involves inputting the position and orientation of the total station before using it to obtain coordinates.¹⁸⁶ The total station can also be combined with a computer on-site to draw sections, profiles, and 3D wire frame-models directly into AutoCAD, with high absolute point accuracy.¹⁸⁷

¹⁸⁴ Watt, *Surveying Historic Buildings*, 66.

¹⁸⁵ Naif Adel Haddad, "From Hand Survey to 3D Laser Scanning: A Discussion for Non-Technical Users of Heritage Documentation," *Conservation and Management of Archaeological Sites*, 15, no. 2 (2013): 214.

¹⁸⁶ Milan Horemuž and Patric Jansson, "Optimum Establishment of Total Station," *Journal of Surveying Engineering* 143, no. 2 (2017): 06016004-1.

¹⁸⁷ Haddad, 214.

Another non-contact method of obtaining detailed measured data is through the use of laser scanning.¹⁸⁸ UNESCO defines this method as “the act of acquiring, processing, presenting and recording the necessary data for the determination of the position and the actual existing form, shape and size of a monument in the 3D space at a particular given moment in time.”¹⁸⁹ Some have compared laser scanning to a robotic total station that is set to scan and record the coordinates of points within a specific area at regular intervals. Though similar, the laser scanner’s speed of recording is faster at a rate of at least 1,000 points per second.¹⁹⁰ The final output from a laser scanner is a set of millions of points with Cartesian coordinates that can be viewed through computer programs as a point cloud to showcase the 3D object’s form.¹⁹¹ The accuracy of these measured points depends on the system, technique, and selected point density used, but typically falls between 0.5 and 25 mm.¹⁹² The use of laser scanners reduces the amount of time needed on site. This helps improve safety when working in more dangerous environments, like ruined or unstable buildings.¹⁹³

¹⁸⁸ David Barber et al., “Laser Scanning for Architectural Conservation,” *Journal of Architecture Conservation* 12, no. 1, (2006): 35.

¹⁸⁹ Odutola Amans et al., “The Need for 3D Laser Scanning Documentation for Select Nigeria Cultural Heritage Sites,” *European Scientific Journal* 9, no. 24 (2013): 76.

¹⁹⁰ Barber et al., 36.

¹⁹¹ Gulhan Benli and Derya Ozer, “Use Of Laser Scanning For Cultural Heritage Documentation,” *International Journal of Electronics, Mechanical and Mechatronics Engineering* 3, no. 1 (2013): 447.

¹⁹² Barber et al., 36.

¹⁹³ Ibid., 45.

Scanners can be categorized by the type of technology they use. The three types are: triangulation, phase-based, and time-of-flight scanners.¹⁹⁴ Triangulation scanners are best used for smaller objects and are based on the optical triangulation method. They are made up of a laser and two CCD (charge-coupled device) cameras, one placed on either side of the laser. The specific distance of the cameras from the laser is vital information for the triangulation process. The way it works is the scanner sends out a laser beam to the surface of an object and the cameras detect how far the laser went before it hit the object.¹⁹⁵ Phase-based scanners, sometimes referred to as phase comparison scanners, measure points using predefined wavelengths. The waves are emitted from the scanner and the fraction of the whole wavelength that is returned is used to determine the distance of the point from the scanner. Phase-based scanners output high precision data at up to 500,000 points per second, but only work for distances less than 30 metres.¹⁹⁶

Time-of-flight scanners are best for long distances and large objects, making it the ideal scanner for documenting heritage buildings. This method emits a laser light pulse and the distance is determined by the time it takes for the laser to return to the instrument, thus giving it the name ‘time-of-flight’.¹⁹⁷ These scanners can scan up to several hundred metres while maintaining an accuracy within a few millimeters to

¹⁹⁴ Haddad, 216.

¹⁹⁵ Naci Yastikli, “Documentation of cultural heritage using digital photogrammetry and laser scanning,” *Journal of Cultural Heritage* 8 (2007): 423.

¹⁹⁶ Brian Romsek, *Terrestrial Laser Scanning: Comparison of Time-Of-Flight and Phase Based Measuring Systems*, (West Lafayette: Purdue University, 2008), 5.

¹⁹⁷ *Ibid.*, 4.

centimeters, depending on the distance to the object.¹⁹⁸ Laser scanners can also be airborne or terrestrial based. Airborne laser scanners are often attached to aircrafts to capture data not accessible from the ground. Terrestrial scanners, or on-the-ground scanners, are the most common and are typically perched on top of a tripod.¹⁹⁹ Table 1 summarizes the categories of laser scanning.

Table 1: Summary of the categories of laser scanning hardware²⁰⁰

Scanning system		Key points	Typical accuracies /operating range	Example manufacturers websites
Triangulation	Rotation stage	Objects have to be taken to the scanner so best suited to small objects (due to limitations of range) where high accuracy is required.	50 microns/ 0.1–1 m	www.3dscanners.co.uk www.callidus.de www.cyberware.com
	Arm mounted	Can be taken to site or used in a studio setting. Not suited to recording entire buildings and structures, but suitable for subjects up to several metres in size.	50 microns/ 0.1–1 m	www.faro.com www.3dscanners.co.uk www.cyberware.com
	Portable/tripod mounted	Site portable and offering high accuracy. Not suited for entire buildings (due to limitations of range), but ideal for smaller architectural features providing access is available.	0.1 mm/ 0.1–25 m	www.konicaminolta-3d.com www.trimble.com
Time of flight		Portable systems that take around 5–10 minutes per scan and are suitable for building façades and interiors.	3–6 mm at ranges up to 100 m.	www.trimble.com www.leica-geosystems.com www.callidus.de www.optech.ca
Phase comparison		Portable systems that capture very large amounts of data (typically gigabytes) quickly with an almost photographic effect.	5 mm at ranges up to 25 m.	www.zf-laser.com www.leica-geosystems.com www.faro.com www.callidus.de

2.5.5. Data Processing

The second step of documenting is processing the data collected during the recording session. Using the raw data, final products, such as architectural drawings, 3D models, and information systems, can be produced in the office.²⁰¹ One type of data

¹⁹⁸ Yastikli, 423.

¹⁹⁹ Stylianidis and Remondino, 29.

²⁰⁰ Barber et al., 39.

²⁰¹ Benli and Ozer, 447.

processing is photogrammetry. Photogrammetry uses a set of over 3 photographs to create a 3D model, which can be used to extract metric data.²⁰² These photographs must have sufficient overlap, generally over 60%, to be able to process. By using photogrammetry software, such as AgiSoft's PhotoScan, the photographs can be aligned and then developed into dense point clouds and 3D textured meshes in an automatic process.²⁰³ As photographs do not possess any metric information of their own, the addition of a scale bar or Cartesian points, typically obtained with a total station, is necessary to scale and orientate the photogrammetric model. This data processing method helps to achieve rapid and accurate 3D information with less time spent on data acquisition.²⁰⁴

The use of laser scanning also requires data processing to register the point clouds. Registering refers to aligning the individual data sets to form one continuous point cloud.²⁰⁵ There are three types of registration: target-based, surface-to-surface, and cloud-to-cloud registration. Target-based registration works by matching targets that can be found on more than one scan. As such, targets should be found on each individual scan.²⁰⁶ These targets can be artificial targets placed on the building and its surroundings, or natural features. To position the scan within a specific reference

²⁰² Stylianidis and Remondino, 127.

²⁰³ Geert Verhoeven, "Taking Computer Vision Aloft – Archaeological Three-dimensional Reconstructions from Aerial Photographs with PhotoScan," *Archaeological Prospection* 18 (2011): 68.

²⁰⁴ Haddad, 216.

²⁰⁵ Benli and Ozer, 448.

²⁰⁶ Kottke et al., 271.

system, the targets' positions should be recorded, preferably with a total station.²⁰⁷

The surface-to-surface registration method uses the surfaces of the object to align the point clouds. This is done by using “an algorithm to estimate the Euclidian distances between surface patches according to their best fit and refined matches.”²⁰⁸ Finally, cloud-to-cloud registration aligns the point clouds based on the overlapping areas between scans. The software begins an automated process that finds what is common between the scans to be matched together and relatively positioned to best fit one another.²⁰⁹

Another process that can be done to a point cloud is the addition of colour. Some laser scanners now include a camera to take coloured photographs to colour the point cloud automatically.²¹⁰ However, the camera does not always take the best quality photographs. Therefore, at times it is necessary to use photographs taken with a higher resolution digital camera to colour the point cloud. These 2D images can be projected onto the 3D point cloud to add the true colour values to the points.²¹¹ In addition to photographs, the point clouds from the laser scanner can also be combined with the photogrammetric models to add colour and texture.²¹² Figure 15 illustrates a

²⁰⁷ Barber et al., 40.

²⁰⁸ Kottke et al., 272.

²⁰⁹ Barber et al., 40.

²¹⁰ Amans et al., 78.

²¹¹ Riccardo Fanti et al., “Terrestrial laser scanning for rockfall stability analysis in the cultural heritage site of Pitigliano (Italy),” *Landslides* 10 (2013): 413.

²¹² Haddad, 218.

photograph projected onto a point cloud, and Figure 16 showcases a colourised point cloud of Pitigliano Cliff.

Figure 15: Photograph projected onto point cloud²¹³

Figure 16: Coloured point cloud of Pitigliano Cliff²¹⁴

²¹³ Amans et al., 78.

The finished point cloud can be used for further data processing, whether in its current state or processed into a mesh or textured model.²¹⁵ In software meant for photogrammetry or registering point clouds, it is possible to create an orthophoto of the object. An orthophoto is a 2D representation of the point cloud or mesh. Orthophotos can be used to measure 2D elements and assist in the creation of other 2D data. Orthophotos also allow the 3D data to be easier showcased to others, as the 2D monoscopic image can be shown through more accessible programs and through more traditional formats, such as a printed picture.²¹⁶

Drawings are one outcome of data processing that has always been used in architecture, engineering, construction and operation. Whether they are drawings of small details or a map of the site, drawings help to convey the object's form and characteristics. Drawings, from as simple as a sketch to as complicated as a detailed structural plan, are used throughout the documenting and conservation process.²¹⁷ For more detailed recordings, there is a need for measured drawings. Measured drawings can include: floor plans, site plans, structural plans, elevations, cross-sections, and details.²¹⁸

These drawings are typically produced in Computer-Aided Design (CAD) systems. The advantage that CAD systems have is that there are no longer any "limits on

²¹⁴ Fanti et al., 413.

²¹⁵ Kottke et al., 272.

²¹⁶ Haddad, 217.

²¹⁷ Stylianidis and Remondino, 113.

²¹⁸ Kottke et al., 272.

precision drawing that were once inherent in the use of scaled drawings.”²¹⁹ Hand measurements, total stations, point clouds, meshes, and orthophotos can all be used to provide the necessary data to create accurate measured drawings.²²⁰ The accuracy and the ability to draw details are directly related to the data used to produce them. For example, the density ratio of a point cloud will limit the scale of detail that can be viewed, and therefore drawn.²²¹ Figure 17 showcases the point cloud of Stoke Rochford Hall, and Figure 18 showcases the elevation drawing produced from the point cloud.

Figure 17: Point cloud of Stoke Rochford Hall²²²

²¹⁹ Haddad, 217.

²²⁰ Barber et al., 44.

²²¹ Benli and Ozer, 448.

²²² Barber et al., 44.

Figure 18: Elevation drawing of Stoke Rochford Hall prepared using point cloud in Fig. 15²²³

Aside from the 2D data processing, information can also be processed in 3D in the form of a model. One of the more common modeling formats today is Building Information Modeling (BIM), which requires defining the building by its objects. This tool is primarily used for new construction as a management tool for the building's whole life cycle.²²⁴ It combines the 3D representation of the building with a database to store information on each individual object within it.²²⁵ When creating such a model, it is important to consider the level of detail and the level of development. The level of detail refers to the extent that an element within the BIM

²²³ Ibid., 45.

²²⁴ Ramona Quattrini, "Conservation-Oriented HBIM. The BIMexplorer Web Tool," *The International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences* XLII-5/W1 (2017): 275.

²²⁵ K. Pramod Reddy, *BIM for Building Owners and Developers Making a Business Case for Using BIM on Projects*, (Hoboken: John Wiley & Sons, Inc, 2012), 5.

will be detailed; the more detailed the more information, but also the more time intensive. The level of development considers how well the object's geometry and the associated information may be relied on, focusing more on how enriched the data for the object are.²²⁶

The application of BIM for historic or existing buildings is relatively new. The term Historic Building Information Modeling (HBIM) has been used for such applications. The main issue of conducting a BIM for existing buildings rather than new construction is the difficulty of fitting the object's current conditions, including its deteriorated elements, into the model.²²⁷ BIM, however, can be utilized for planned maintenance. The database can be used to store information on each object's condition, which can later be imported or linked with other maintenance or management software. It can also include inspection dates and work done to help organize and schedule additional work or visits. The information can then be benchmarked with similar building(s) to compare performance and deterioration.²²⁸

There are also newer methods of presenting the recorded information. They take the form of virtual and augmented reality. Virtual reality allows for the viewer to immerse themselves in or around the building itself. It can act as a 'site visit' or preliminary inspection to reduce the number of people actually visiting the structure. Augmented reality works by virtually projecting information by means of computer

²²⁶ Stylianidis and Remondino, 118.

²²⁷ Ibid., 114.

²²⁸ Reddy, 36.

and camera based on the surrounding environment. Both of these methods help to present information and can be used as a learning tool.²²⁹

2.6. Structural Analysis

In addition to the survey of the building itself, a structural analysis should be conducted to assess an existing building's strength and deformation potential. The results of the structural analysis will determine if interventions are required to properly support the current or future loads on the building and to decrease the damage during a seismic event. The *National Building Code of Canada* (NBCC) provides a list of load combinations to consider during a structural analysis. The five load combination cases are as follows:

Case 1: $1.4D$

Case 2: $(1.25D \text{ or } 0.9D) + 1.5L + (0.5S \text{ or } 0.4W)$

Case 3: $(1.25D \text{ or } 0.9D) + 1.5S + (0.5L \text{ or } 0.4W)$

Case 4: $(1.25D \text{ or } 0.9D) + 1.4W + (0.5L \text{ or } 0.5S)$

Case 5: $1.0D + 1.0E + 0.5L + 0.25S$

where D is the dead load, L is the live load, S is the snow load, W is the wind load, and E is the earthquake load.²³⁰ The dead load is taken as the weight of the building and its furniture that a structural element has to support. The live load represents the

²²⁹ Stylianidis and Remondino, 20.

²³⁰ National Research Council Canada, *National Building Code of Canada 2015* (Ottawa: National Research Council Canada, 2015), Division B 4-5.

expected weight of the occupants by room use. The snow load is calculated using the following equation from the NBCC:

$$S = I_s [S_s (C_b C_w C_s C_a) + S_r] \quad (2.1)$$

where I_s is the importance factor for the snow load, S_s is the 1 in 50 year ground snow load, C_b is the basic roof snow load factor, C_w is the wind exposure factor, C_s is the slope factor, C_a is the shape factor, and S_r is the 1 in 50 year ground snow load that is not greater than $S_s (C_b C_w C_s C_a)$.²³¹ The wind load represents the force of the wind on the structure and is calculated as follows:

$$W = I_w q C_e C_g C_p \quad (2.2)$$

where I_w is the importance factor for wind load, q is the reference velocity pressure, C_e is the exposure factor, C_g is the gust effect factor, and C_p is the external pressure coefficient.²³²

Cases 1 to 4 primarily look into gravity loads affecting the building. During a structural analysis, these cases are compared to determine the governing load combination for the building. Structural members' ability to support this governing load is then assessed in the structural analysis. Case 5 is different from the other cases as it is the only load combination that takes into account earthquake loads. This case is assessed through a seismic analysis.

²³¹ Ibid., 4-14.

²³² Ibid., 4-16.

A seismic analysis assists in planning possible interventions to mitigate the damage during a future seismic event. Though buildings in high seismic hazard zones carry the highest risk, those within low and moderate zones are still susceptible to damage.²³³ Unreinforced masonry structures are the most prone to damage during an earthquake. This is mainly due to their constituent material properties, which include low tensile strength, high mass, and limited ductility.²³⁴ In addition to these material limitations, historic unreinforced masonry buildings were typically constructed through the use of practical rules, that were created based on centuries of experience with little consideration of theoretical support. These empirical rules were weak in terms of seismic design since builders had a tendency to forget the effects, and subsequent countermeasures, of earthquakes within a few generations and in between seismic events.²³⁵

As the construction of historic masonry structures did not focus on the theoretical behaviour of the materials, information on the mechanical behaviour of unreinforced masonry is limited in comparison to the information that is available for other materials. Non-destructive in-situ testing, laboratory testing, and the use of numerical tools have all been used to better understand masonry structures.²³⁶ A major

²³³ Vasconcelos and Lourenço, 1269.

²³⁴ Mingke Deng and Shuo Yang, "Cyclic testing of unreinforced masonry walls retrofitted with engineered cementitious composites," *Construction and Building Materials* 177 (2018): 395.

²³⁵ Manuel Berti et al., "Unreinforced masonry walls with irregular opening layouts: reliability of equivalent-frame modelling for seismic vulnerability assessment," *Bulletin of Earthquake Engineering* 15 (2017): 1214.

²³⁶ Paulo Lourenço, "Computations on historic masonry structures," *Progress in Structural Engineering and Materials* 4 (2002): 301.

complication in modeling historic masonry structures under seismic loads is the difficulty of obtaining information on the masonry structure itself. There can be a lot of unknown information on existing masonry buildings, such as missing geometric data, missing information of the inner core structural elements, unknown construction sequence information, and unknown existing damage to the building's structural components. Current building regulations and codes may also not be applicable as they focus more on new construction rather than existing and historic buildings.²³⁷ It can also be very difficult and expensive to determine and characterize the exact mechanical properties of the masonry. These properties are heavily dependant on the properties of its constituents, which are the brick and mortar. These can vary greatly within a structure due to both the workmanship and the inherent property variance of the materials.²³⁸

One property required for modeling masonry is its compressive strength. This can be calculated using the compressive strength of the bricks and the compressive strength of the mortar. *Eurocode 6* suggests the following equation to calculate the compressive strength of the masonry wall:

$$f_{M,c} = K f_b^\alpha f_m^\beta \quad (2.3)$$

²³⁷ Ibid.

²³⁸ M. Holicky et al. "Assessment of masonry strength in a heritage building," In *Heritage Masonry Materials and Structures*, S. Syngellakis (Southampton: Wessex Institute of Technology Press, 2014), 34.

where α and β are constants 0.7 and 0.3, respectively, f_b and f_m are the compressive strength of the brick and mortar in N/mm^2 , respectively, and K is a constant selected using Table 2.²³⁹ This equation is the most commonly used equation in practice, but it is not the only one. For example, the *Code for Seismic Design of Buildings* in China suggests calculating the compressive strength using $f_{M,c} = 0.78 f_b^{0.5} (1 + 0.07 f_m)$.²⁴⁰ Other properties required for the analysis of a masonry wall are the tensile strength, the modulus of elasticity, and shear modulus. The tensile strength can be assumed to be 5 to 10% of the wall's compressive strength when laboratory tests are not possible.²⁴¹ When experimental values are unavailable, the modulus of elasticity can be calculated as $1000f_{M,c}$, and the shear modulus can be assumed to be 40% of the modulus of elasticity under the design assumption of a 0.25 Poisson's ratio.²⁴²

²³⁹ European Committee For Standardization, *Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures* (Brussels: European Committee For Standardization, 2005), 35-36.

²⁴⁰ Deng and Yang, 397.

²⁴¹ Robert Drysdale and Ahmad Hamid, *Masonry Structures Behaviour and Design*. Canadian ed. (Mississauga: Canada Masonry Design Centre, 2005), 138.

²⁴² Meta Kržan et al., "Acquiring reference parameters of masonry for the structural performance analysis of historical buildings," *Bulletin of Earthquake Engineering* 13 (2015): 214.

Table 2: Values of K for varying masonry unit materials and mortar properties.²⁴³

Masonry Unit		General purpose mortar	Thin layer mortar (bed joint ≥ 0,5 mm and ≤ 3 mm)	Lightweight mortar of density	
				$600 \leq \rho_d$ ≤ 800 kg/m ³	$800 < \rho_d$ ≤ 1 300kg/m ³
Clay	Group 1	0,55	0,75	0,30	0,40
	Group 2	0,45	0,70	0,25	0,30
	Group 3	0,35	0,50	0,20	0,25
	Group 4	0,35	0,35	0,20	0,25
Calcium Silicate	Group 1	0,55	0,80	‡	‡
	Group 2	0,45	0,65	‡	‡
Aggregate Concrete	Group 1	0,55	0,80	0,45	0,45
	Group 2	0,45	0,65	0,45	0,45
	Group 3	0,40	0,50	‡	‡
	Group 4	0,35	‡	‡	‡
Autoclaved Aerated Concrete	Group 1	0,55	0,80	0,45	0,45
Manufactured Stone	Group 1	0,45	0,75	‡	‡
Dimensioned Natural Stone	Group 1	0,45	‡	‡	‡
‡ Combination of mortar/unit not normally used, so no value given.					

Modelling has often been used to simulate the response of a structure to various loads. Masonry walls can be modeled under seismic loads to determine their performance. The detail level of modeling masonry can be broken up into micro- and macro-modeling. Micro-modeling individually identifies the distinct components of the masonry. These are the brick units, the mortar, and their interfaces. This type of modeling is very computationally demanding, so it is only realistically suitable for small structural elements. On the other hand, macro-modeling defines the masonry as a “fictitious homogeneous but orthotropic continuum material where the tensile and

²⁴³ European Committee For Standardization, 37.

compressive strengths are very different.”²⁴⁴ Due to its lower computational demand, macro-modeling is often better suited for conducting a seismic analysis of full building structures.

Among the macro-modeling methods, the equivalent frame approach is the most used within the industry, and is endorsed by some building codes, such as *Eurocode 8* and the *Italian Building Code*.²⁴⁵ The equivalent frame approach converts the masonry wall into a set of piers and spandrels that are connected with rigid joints. The joints are assumed to be rigid so that only the effective lengths of the spandrels and piers are taken into consideration in the model. The piers and spandrels, modeled as one-dimensional beam elements, represent the vertical and horizontal structural components, respectively. Vertical and lateral loads in an equivalent frame model are only applied at the rigid joint nodes of the structure.²⁴⁶ Figure 19 showcases two examples of equivalent frame models for a wall with two openings; one with aligned openings and one where they are misaligned.

²⁴⁴ Paola Miglietta et al., “Finite/discrete element modelling of reversed cyclic tests on unreinforced masonry structures,” *Engineering Structures* 138 (2017): 159.

²⁴⁵ Berti et al., 1216.

²⁴⁶ Simon Petrovčič and Vojko Kilar, “Seismic failure mode interaction for the equivalent frame modeling of unreinforced masonry structures,” *Engineering Structures* 54 (2013): 9.

Figure 19: Equivalent frame model for aligned (a) and misaligned (b) openings.²⁴⁷

Masonry pier elements have three possible forms of failure: rocking or toe-crushing failure, diagonal shear cracking failure, and shear sliding failure. Rocking failure occurs when the bed-joints crack in tension due to an increased horizontal load or displacement demand. The final failure is achieved when the wall overturns causing the crushing of the compressed corner, thus giving it the nickname toe-crushing failure.²⁴⁸ Rocking tends to be the governing failure mode for slender piers.²⁴⁹ Diagonal shear cracking failure is characterized by the diagonal or cross-shaped cracking of the wall.²⁵⁰ These cracks usually start at the centre of the pier and work their way to the corners, typically involving both the mortar joints and the masonry units.²⁵¹ Finally, shear sliding failure is caused by a tensile horizontal crack in the bed-joints. This failure mode is possible in walls with low vertical loads and with low

²⁴⁷ Berti et al., 1225.

²⁴⁸ Guido Magenes and Gian Michele Calvi, "In-Plane Seismic Response of Brick Masonry Walls," *Earthquake Engineering And Structural Dynamics* 26 (1997): 1092.

²⁴⁹ Petrovčič and Kilar, 11.

²⁵⁰ Magenes and Calvi, 1092.

²⁵¹ Petrovčič and Kilar, 11.

friction coefficients between the brick and the mortar.²⁵² This failure mode most often occurs in very squat piers, either in a single mortar bed-joint or in a stepwise pattern.²⁵³ Figure 20 illustrates each failure mode of pier elements.

Figure 20: Typical failure modes for piers in an equivalent frame model.²⁵⁴

Figure 21: Idealized hysteresis for each pier failure mode.²⁵⁵

²⁵² Magenes and Calvi, 1092.

²⁵³ Petrovčič and Kilar, 11.

²⁵⁴ Yasuto Nakamura et al., "Equivalent Frame Modelling of an Unreinforced Masonry Building with Flexible Diaphragms – A Case Study," *Bulletin of the New Zealand Society for Earthquake Engineering* 49, no. 3 (2016): 234.

²⁵⁵ Joonam Park et al., "Seismic fragility analysis of low-rise unreinforced masonry structures," *Engineering Structures* 31 (2009): 128.

Figure 21 showcases the idealized hysteresis for each pier failure mode. The idealized rocking failure of a pier is considered purely ductile as the pier moves back and forth due to the moment at the base of the wall. On the other hand, bed-joint sliding is considered highly non-linear since, once it fails, the pier relies purely on friction to withstand the ongoing forces. Diagonal tension cracking's idealized hysteresis falls in-between the other failure modes as its post failure behaviour imitates a combination of the two. Within the context of a wall, especially perforated ones, the behaviour of the elements becomes much more complex due to its reactions with the other components.²⁵⁶

Figure 22: Equivalent frame model geometric parameters.

²⁵⁶ Ibid.

To determine which failure will govern, the pier's ability to resist each mode needs to be calculated and compared. The lowest strength out of the three failure modes will govern for the pier. Figure 22 illustrates the geometric parameters that are required to calculate the failure strength. One issue when comparing the strengths is that rocking failure depends on the moment exerted on the pier, while diagonal cracking and sliding failure depend on the shear force. The moment strength associated with rocking failure can be calculated, as indicated in *Eurocode 8* and the *Italian Building Code*, as:

$$M_{Pr,r} = \frac{1}{2} N_{pr} \left(1 - \frac{N}{0.85 f_{M,c} t_{pr} l_{pr}} \right) \quad (2.4)$$

where N is the compressive axial load at failure of the wall, l_{pr} is the pier length, t_{pr} is the pier thickness, and $f_{M,c}$ is the compression strength as shown previously in Eq(2.3).²⁵⁷

To properly compare the values, the rocking failure strength needs to be converted into an equivalent shear force. The equivalent rocking shear strength can be calculated as:

$$V_{Pr,r}(N, \lambda) = l_{pr} t_{pr} \frac{\sigma(N)}{2\alpha_V(\lambda)} \left(1 - \frac{\sigma(N)}{\kappa f_{M,c}} \right) \quad (2.5)$$

²⁵⁷ Berti et al., 1226.

where $\sigma(N)$ is the mean compressive stress in the pier due to the axial force N , κ is the peak compressive strength reduction factor with a value of 0.85, and $\alpha_v(\lambda)$ is the pier shear ratio equal to the pier slenderness ratio (λ) multiplied by the absolute value of the zero moment coefficient (α_0). λ is calculated by dividing the height of the pier (h_{pr}) by l_{pr} . The parameter α_0 can be calculated as $h_{pr,0}/h_{pr}$ when $h_{pr,0}$ is greater than or equal to half of h_{pr} or as $1 - (h_{pr,0}/h_{pr})$ when $h_{pr,0}$ is less than half of h_{pr} ; $h_{pr,0}$ is the location along the height of the pier where the moment is equal to zero.²⁵⁸ The rocking strength can be also be calculated as follows:

$$V_{Pr,r} = \psi \frac{\sigma_0 t_{pr} l_{pr}^2}{2h_0} \left(1 + \frac{\sigma_0}{f_{M,c}} \right) \quad (2.6)$$

where ψ is the corrective factor based on boundary conditions and is equal to 1 for cantilever walls and equal to 2 for both fixed ends, σ_0 is normal compressive stress on the pier due to vertical loading, and h_0 is the height of top of the pier from the ground. This equation can be found in the *NTC (2008)*, *EC8-3*, and *FEMA 306* building codes.²⁵⁹ The diagonal shear cracking failure strength can also be calculated in two ways. The first calculation, which includes the mean compressive stress due to the axial force, is as follows:

²⁵⁸ Petrovčič and Kilar, 11.

²⁵⁹ Kržan et al., 221.

$$V_{Pr,t}(N, \lambda) = l_{pr} t_{pr} \frac{f_{M,t}}{b} \sqrt{\left(1 + \frac{\sigma(N)}{f_{M,t}}\right)} \quad (2.7)$$

where $f_{M,t}$ is the diagonal tensile strength of masonry and can be approximated as 5 to 10% of the masonry's compressive strength, and b is the “corrective coefficient connected to the distribution of stress on the section depending on the geometric aspect ratio of the wall” and is equal to h_{pr}/l_{pr} , but not greater than 1.5 or smaller than 1.0.²⁶⁰ The other method, which is also supported by the previously stated building codes, can be calculated as followed:

$$V_{Pr,t} = l_{pr} t_{pr} \frac{f_{M,t}}{b} \sqrt{\left(1 + \frac{\sigma_0}{f_{M,t}}\right)} \quad (2.8)$$

where the mean compressive stress due to the axial force was substituted with the normal compressive stress.²⁶¹ For the final failure mode, shear sliding, the experimental data based equation is expressed as:

$$V_{Pr,s}(N, \lambda) = l_{pr} t_{pr} \left(\frac{f_{M,t} + \mu \sigma(N)}{1 + 3\tau_0 \alpha_v(\lambda) / \sigma(N)} \right) \quad (2.9)$$

where μ is the friction coefficient that ranges from 0.4 to 0.8, and τ_0 is shear strength of the masonry at zero compressive stress, which is calculated by multiplying the

²⁶⁰ Petrovčič and Kilar, 11.

²⁶¹ Kržan et al., 221.

masonry's compression strength by the masonry cohesion coefficient (ζ) that ranges from 0.01 to 0.1.²⁶² The shear sliding strength equation for when there is no experimental data exhibits the largest change and can be expressed as:

$$V_{Pr,s} = l_{pr,c} t_{pr} (f_{vo} + \mu \sigma_d) \quad (2.10)$$

where $l_{pr,c}$ is length of the compressed section of the pier, σ_d is the average vertical stress over compressed part of wall, and f_{vo} is the initial shear strength of masonry that ranges between 0.29 to 0.90 MPa.²⁶³

Spandrels also have rocking and shear cracking failure, but unlike piers, the shear is not split into two failure modes since there is no sliding shear failure mode for the spandrels. As such, the governing strength is determined by comparing only two calculated failure strengths. Once again, rocking failure is dependant on the moment of the structure. A spandrel's moment strength is calculated as:

$$M_{Sp,r} = F_p h_{sp} (1 - F_p / 0.85 f_{M,c} t_{sp} h_{sp}) / 2 \quad (2.11)$$

where t_{sp} is the thickness of the spandrel, h_{sp} is the height of the spandrel, and F_p is the minimum of $f_{M,t}$ and $0.4 f_{M,c} t_{sp} h_{sp}$.²⁶⁴ This moment strength needs to be converted

²⁶² Petrovčič and Kilar, 11.

²⁶³ Kržan et al., 221.

²⁶⁴ Berti et al., 1227.

into an equivalent shear force to be compared with the shear cracking failure mode's strength. The equivalent shear strength for rocking failure is calculated as:

$$V_{Sp,r} = \frac{F_p h_{sp}}{l_{sp}} \left(1 - \frac{F_p}{\kappa f_{M,c} h_{sp} t_{sp}} \right) \quad (2.12)$$

where l_{sp} is the length of the spandrel. The shear strength of the spandrel can be calculated using the following equation:²⁶⁵

$$V_{Sp,s} = \tau_0 h_{sp} t_{sp} \quad (2.13)$$

Table 3 summaries all the equations to calculate the equivalent strength for each failure mode.

²⁶⁵ Petrovčič and Kilar, 14.

Table 3: Summary of strength equations for each failure mode.

Element	Failure Mode	Equivalent Strength Equations	Source
Pier	Rocking	$M_{Pr,r} = \frac{1}{2} N l_{pr} \left(1 - \frac{N}{0.85 f_{M,c} t_{pr} l_{pr}} \right)$	Berti et al., 1226.
		$V_{Pr,r}(N, \lambda) = l_{pr} t_{pr} \frac{\sigma(N)}{2\alpha_v(\lambda)} \left(1 - \frac{\sigma(N)}{\kappa f_{M,c}} \right)$	Petrovčič and Kilar, 11.
		$V_{Pr,r} = \psi \frac{\sigma_0 t_{pr} l_{pr}^2}{2h_0} \left(1 + \frac{\sigma_0}{f_{M,c}} \right)$	Kržan et al., 221.
	Shear Sliding	$V_{Pr,t}(N, \lambda) = l_{pr} t_{pr} \frac{f_{M,t}}{b} \sqrt{1 + \frac{\sigma(N)}{f_{M,t}}}$	Petrovčič and Kilar, 11.
		$V_{Pr,t} = l_{pr} t_{pr} \frac{f_{M,t}}{b} \sqrt{1 + \frac{\sigma_0}{f_{M,t}}}$	Kržan et al., 221.
	Diagonal Shear Cracking	$V_{Pr,s}(N, \lambda) = l_{pr} t_{pr} \left(\frac{f_{M,t} + \mu \sigma(N)}{1 + 3\tau_0 \alpha_v(\lambda) / \sigma(N)} \right)$	Petrovčič and Kilar, 11.
		$V_{Pr,s} = l_{pr,c} t_{pr} (f_{vo} + \mu \sigma_d)$	Kržan et al., 221.
Spandrel	Rocking	$M_{Sp,r} = F_p h_{sp} (1 - F_p / 0.85 f_{M,c} t_{sp} h_{sp}) / 2$	Berti et al., 1227.
		$V_{Sp,r} = \frac{F_p h_{sp}}{l_{sp}} \left(1 - \frac{F_p}{\kappa f_{M,c} h_{sp} t_{sp}} \right)$	Petrovčič and Kilar, 14.
	Shear	$V_{Sp,s} = \tau_0 h_{sp} t_{sp}$	Petrovčič and Kilar, 14.

$f_{M,c}$: compression strength; $f_{M,t}$: diagonal tensile strength of masonry; f_{vo} : initial shear strength of masonry; τ_0 : shear strength of the masonry at zero compressive stress; N : compressive axial load at failure of the wall; $\sigma(N)$: mean compressive stress in the pier due to the axial force N ; σ_0 : normal compressive stress on the pier due to vertical loading; σ_d : average vertical stress over compressed part of wall; κ : peak compressive strength reduction factor; ψ : corrective factor based on boundary conditions; α_0 : zero moment coefficient; b : corrective coefficient connected to the distribution of stress on the section depending on the geometric aspect ratio of the wall; μ : friction coefficient; ζ : masonry cohesion coefficient; $\alpha_v(\lambda)$: pier shear ratio; λ : pier slenderness ratio; $l_{pr,c}$: length of the compressed section of the pier; $h_{pr,0}$: location along the height of the pier where the moment is equal to zero; h_0 : height of top of the pier from the ground; h_{pr} : height of pier; l_{pr} : pier length; t_{pr} : pier thickness; t_{sp} : thickness of the spandrel; h_{sp} : height of the spandrel; l_{sp} : length of the spandrel; F_p : minimum of $f_{M,t}$ and $0.4 f_{M,c} t_{sp} h_{sp}$

With the ultimate governing strengths of both the piers and spandrels calculated, the next step is to define the hysteresis shape and behaviour. This represents the load versus displacement behaviour of the wall elements, which governs the behaviour of the piers and spandrels during the seismic analysis. The components that make up a hysteresis are the envelope curves and the different forms of degradation per cycle.²⁶⁶ The envelope of the hysteresis curve is heavily influenced by the governing failure mode. The end result will be similar to the idealized hysteresis previously showcased in Figure 21. As the cycles continue, the hysteresis curve will be influenced by the three deterioration factors affecting unloading stiffness, the strength developed at the maximum and minimum deformation demands, and the strength achieved at increasing deformation demands.²⁶⁷ These factors are often referred to as unloading stiffness degradation, reloading stiffness degradation, and strength degradation, respectively.²⁶⁸ Figures 23 to 25 showcase samples of experimental hysteretic curves for unreinforced masonry walls with no openings. Figure 26 showcases all of the components that define the hysteretic curve of unreinforced masonry and their degradation effects on the hysteresis shape.

²⁶⁶ Laura Lowes et al., *A Beam-Column Joint Model for Simulating the Earthquake Response of Reinforced Concrete Frames* (Berkeley: University of California, 2003), 31.

²⁶⁷ Ibid., 31-32.

²⁶⁸ Eva Raka et al., “Advanced frame element for seismic analysis of masonry structures: model formulation and validation,” *Earthquake Engineering and Structural Dynamics* 44 (2015): 2495.

Figure 23: Experimental hysteretic curves of clay brick masonry under cyclic loading.²⁶⁹

Figure 24: Experimental hysteretic curves of irregular stone masonry under cyclic loading.²⁷⁰

Figure 25: Experimental hysteretic curves of rubble stone masonry under cyclic loading.²⁷¹

²⁶⁹ Deng and Yang, 405.

²⁷⁰ Vasconcelos and Lourenço, 1272.

²⁷¹ Ibid., 1273.

Figure 26: Components of a hysteresis: (a) loading and reloading envelopes, (b) unloading stiffness degradation, (c) reloading stiffness degradation, and (d) strength degradation.²⁷²

The envelope curves are comprised of four points in both the loading and reloading zones. The points in the reloading zone are simply the negative counterparts of those in the loading zone. These four points are characterized as (γ_1, V_1) , (γ_2, V_2) , (γ_3, V_3) , and (γ_4, V_4) , where the γ values represent the displacement and the V values represent the load. These points are often based on experimental data. However, they can be approximated as fractions of the governing failure mode strength and the overall wall height. The four loading values correspond to a fraction of the governing failure mode strength. V_1 , V_2 , V_3 , and V_4 can be represented as 66%, 99%, 100%, and 80% of the

²⁷² Raka et al., 2495.

governing strength, respectively.²⁷³ The displacement values correspond to fractions of the overall height of the masonry wall under analysis. γ_1 , γ_2 , γ_3 , and γ_4 can be 0.035%, 0.075%, 0.4%, and 1.0% of the wall height, respectively.²⁷⁴ The remaining degradation factors are solely derived from experimental results.

With the model complete, the seismic analysis can commence. This analysis can be conducted through two methods: a non-linear static pushover test and a non-linear dynamic time-history analysis. A pushover test involves applying a gradually increasing horizontal load to the model until it collapses. It outputs a hysteresis that shows the displacement of the structure versus the load. Though the pushover test is not as reliable, time-history analyses are more complex and require more time and computational resources. Therefore, a pushover test is often preferred by practitioners as it is much simpler while remaining an effective tool to assess the strength and ductility of a structure.²⁷⁵ An example of the results of a pushover test can be seen in Figure 27. The figure shows the displacement of a masonry tower with different support constraints.

²⁷³ Kržan et al., 219.

²⁷⁴ Raka et al., 2495.

²⁷⁵ Massimiliano Bocciarelli and Gaia Barbieri, “A numerical procedure for the pushover analysis of masonry towers,” *Soil Dynamics and Earthquake Engineering* 93 (2017): 162.

Figure 27: Pushover results of a masonry tower case study by Bocciarelli and Barbieri (2017) with differing support constraints.²⁷⁶

For the time-history analysis, seismic data needs to be imported to test the building's structural performance. In the appendix of the *National Building Code of Canada's* commentary, it suggests using no less than 11 records for each source. These records must then be scaled to equal or exceed the target spectrum for the site, $S_T(T)$, over the scenario-specific period range, T_{RS} . This involves a two step process. The first step is to scale each ground motion individually, and, the second, is to scale all the records so that the mean response spectrum is not below 10% of the target spectrum.²⁷⁷

The response of the model by the ground motions and its behaviour should be documented. This includes the accelerations, the base shear of the model, and the drift

²⁷⁶ Ibid., 168.

²⁷⁷ National Research Council Canada, J114-J115.

both during and at the end of the seismic event. There is no one set way to quantify the amount of damage the modeled masonry structure endured during the tested seismic events. However, a commonly used method is to examine the model's maximum drift ratio. Proposed by *FEMA 356*, this method looks into the inter-storey drift as a relative lateral displacement, which is then expressed as a percentage of the story height at the floor. Based on those percentages, the building performance, or damage level, can be approximated.²⁷⁸ Table 4 states what drift ratios correspond to four different levels of damage.

Table 4: Structural performance levels for unreinforced masonry structures.²⁷⁹

	Structural performance levels			
	Complete damage	Extensive damage	Moderate damage	Slight damage
Overall damage descriptions	Structure has collapsed or is in imminent danger of collapse due to failure of the walls. Approximately 15% of the total area is expected to be collapsed.	In buildings with relatively large area of wall openings, most walls have suffered extensive cracking. Some parapets and gable end walls have fallen. Beams or trusses may have moved relative to their supports	Most wall surfaces exhibit cracks; some of the walls exhibit larger cracks; masonry walls may have visible separation from diaphragms; significant cracking of parapets; some masonry may fall from walls or parapets.	Diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings; movements of lintels; cracks at base of parapets.
Drift ratio	3.5%	1.5%	0.6%	0.3%

Another important drift to consider is the residual drift, which corresponds to the horizontal deformation of a building present at the end of an earthquake. The residual drift can dictate a structure's ability to remain in use after a seismic event. This can be split into safety and functionality. A building is considered safe if the residual drift is

²⁷⁸ Park et al., 131.

²⁷⁹ Ibid., 132.

under 1.0%; however, residual drifts over 0.5% are known to amplify the drift during a second earthquake.²⁸⁰ In terms of functionality, it was found that people can perceive the inclinations of a residual drift between 0.5% and 0.6%. A residual drift of 0.8% or greater deem the building to be not functional since the deformation leads to headaches, dizziness, and an overall hindrance to the occupants.²⁸¹ Overall, for a building's use to be unaffected after an earthquake, the residual drift should be under 0.5%.

Current equivalent frame model case studies are aimed at comparing the empirical masonry wall hysteresis behaviour to the numerical, and for determining the areas of failure within a wall. One example of a case study that compares the experimental wall hysteresis behaviour to that of the equivalent frame models was conducted by G. Rinaldin et al. (2017). Their case study focused on the behaviour of unreinforced and strengthened masonry spandrels.²⁸² Figure 28 illustrates the wall configuration and area of interest alongside the equivalent frame model deformation. Figure 29 showcases the experimental and numerical hysteresis curves for the URM walls.

²⁸⁰ Jason McCormick et al., "Permissible Residual Deformation Levels for Building Structures Considering Both Safety And Human Elements," *The 14th World Conference on Earthquake Engineering* Beijing, China (2008): 5.

²⁸¹ Ibid.

²⁸² G. Rinaldin et al., "Review of experimental cyclic tests on unreinforced and strengthened masonry spandrels and numerical modelling of their cyclic behaviour," *Engineering Structures* 132 (2017): 609.

Figure 28: Rinaldin et al. masonry perforated wall with highlighted section of interest (a), and the deformation of the spandrels in the equivalent frame model.²⁸³

Figure 29: Experiment vs. numerical hysteresis curves for URM walls.²⁸⁴

An example of a case study that compared the experimental failure of a wall with the drift ratios calculated by an equivalent frame model was conducted by Yasuto

²⁸³ Ibid., 612.

²⁸⁴ Ibid., 621-622.

Nakamura et al. (2016). The drift ratios for the rocking and shear failures for both the piers and the spandrels were compared with ultimate drift capacities of the failure modes. The calculated capacities for the case study were 0.6% and 0.3%, respectively. Sections that had a ratio larger than the capacity were estimated to fail. If both failure modes were higher than their capacities, the larger ratio compared to the capacity governed.²⁸⁵ Figure 30 showcases the experimental wall crack patterns and the numerically calculated failures for piers and spandrels for one of their model cases.

Figure 30: Experimental wall crack patterns vs. numerical failure sections by drift ratios.²⁸⁶

²⁸⁵ Nakamura et al., 240-241.

²⁸⁶ Ibid., 241.

2.7. Monitoring

Monitoring aims to detect changes and decay in a building and to determine why they are present, if they are active, the speed of which they are occurring, and the urgency of remedial action.²⁸⁷ To determine the exact cause of decay or failure, the symptoms need to undergo long-term observation, which usually involves some degree of absolute or relative measurement.²⁸⁸ The methods used to monitor will vary depending on the reasons and resources for the project. Some of the reasons to monitor a building are to record an already occurring condition or to detect the conditions or changes before there is damage to the element. The method will also be influenced by the ability to access the site, the overall condition of the building or element, financial resources, and “the need to obtain a standard and continual sequence of information.”²⁸⁹

Inspections are a fundamental part of monitoring. The National Trust in England conducts quinquennial surveys to monitor their buildings’ condition. Quinquennial surveys refer to the fact that they are held every five years and they “provide a detailed appraisal of condition of the building and clearly identify present and future needs in order to maintain the structure and services in perpetuity.”²⁹⁰ These inspections look at each individual component of the building to assess their

²⁸⁷ Letellier et al., 14.

²⁸⁸ Watt, *Surveying Historic Buildings*, 90.

²⁸⁹ Watt, *Building Pathology*, 178.

²⁹⁰ Watt, *Surveying Historic Buildings*, 23.

condition, and often include a checklist or summary table to present the data in a way that can be easily compared with previous inspections. It is also important that during these surveys, any renovations, alterations, or additions done to the building in the past five years are recorded.²⁹¹ The guidelines for a quinquennial survey can be seen in Figure 31. Keeping a record of inspection reports is also vital to the monitoring process, as it keeps track of the building's history of changes, deterioration issues, and repair work completed. Therefore, it can help to identify the chronic problems of the building and to understand what remedial work could be done to mitigate the issue(s).²⁹²

²⁹¹ Mydin et al. 351.

²⁹² Canada's Historic Places, *Heritage Building Maintenance Manual*, 10.

Introduction	
Schedule of buildings included in survey	Heating installations
Brief description of the building/s	Environmental and humidity control
Description of construction and materials	Electricity supply
Reference to inaccessible parts of building	Electrical installation
Recommendations for further opening up	Lightning conductor installation
Brief recommendations for improving energy efficiency of structure and services	Gas supply
Recommendations for further specialist advice	Gas installation
Record of work undertaken since last survey	Above-ground foul drainage
Record of outstanding work from last survey	Below-ground rainwater disposal system
Summary of general structural condition	Below-ground foul drainage and disposal system, including effluence sampling facilities
Description of defects	
Exterior	
Chimneys and high-level features	Fire precautions
Roof structures and roof coverings	Separate checklist
Gutters and above-ground rainwater disposal systems	Security precautions
Walls	Weaknesses or observed defects
Doors, windows and external joinery	Site features
Metalwork	Boundary walls, gates, drives, paths, fences, etc.
External paintwork	Summary of recommendations with estimated costs
Pavings	For immediate action
Interior	
Roof spaces	For completion within two years
Ceilings	For completion within five years
Walls and partitions	For completion within ten years
Floors and floor structures	Observations
Lintels	Suggested improvements
Internal joinery	Monitoring of defects
Staircases	Compliance with health and safety legislation
Glazing	Compliance with the Disability Discrimination Act
Fittings	Routine maintenance
Metalwork	Adequacy of routine maintenance
Decorations	Overall comparison of maintenance levels relative to last quinquennial inspection
Asbestos installations	Drawings and photographs
Services	
Water supply	
Hot and cold water installations	

Figure 31: The guidelines for quinquennial surveys of National Trust buildings.²⁹³

²⁹³ Watt, *Surveying Historic Buildings*, 21.

2.7.1. Preventive Conservation

Preventive conservation is the monitoring and controlling of causes of deterioration. It works “to ensure the practical and cost-effective use and aftercare of sensitive or valuable buildings and their contents.”²⁹⁴ The main aim of preventive conservation is to gain knowledge of the potential and current threats to a historic building and its current condition to be able to implement strategies to reduce the deterioration, thus increasing the building’s service life.²⁹⁵ These strategies can take two forms. The first is through continuous action that responds to ongoing decaying factors that come with time. The second works to mitigate the damage caused by isolated events, such as fire and earthquakes.²⁹⁶

2.7.2. Structural Health Monitoring

Structural Health Monitoring is the method of detecting and assessing damage and deterioration through the use of strategically placed sensors on a building.²⁹⁷ These sensors allow for a continuous diagnosis of the current condition of materials and of the full assembly.²⁹⁸ When utilizing Structural Health Monitoring, it is important to consider the intervals of data collection, “the types of sensors to be used, the locations

²⁹⁴ Watt, *Building Pathology*, 254.

²⁹⁵ Koenraad Van Balen, 1.

²⁹⁶ Ortiz and Ortiz, 1078.

²⁹⁷ Structural Health Monitoring Canada, *Services* (2017). <<http://shmcanada.com/services/>> [4 January 2018], 1.

²⁹⁸ Daniel Balageas et al., *Structural Health Monitoring*, (London: ISTE Ltd, 2006), 13.

where the sensors should be placed, the number of sensors to be used, and the data acquisition/storage/transmittal hardware.”²⁹⁹ Overall, it allows for the monitoring of building using non-destructive evaluations.³⁰⁰

2.8. Maintenance

The *Standards and Guidelines for the Conservation of Historic Places in Canada* define maintenance as the “routine, cyclical, non-destructive actions necessary to slow the deterioration of an historic place. It entails periodic inspection; routine, cyclical, non-destructive cleaning; minor repair and refinishing operations; replacement of damaged or deteriorated materials that are impractical to save.”³⁰¹ The aim of maintenance is to increase the life of a building while optimising its service potential and minimising its ‘whole-of-life’ costs.³⁰² When creating a maintenance plan, it is important to ensure that it is neither too complicated nor onerous as to ensure that the work is carried out.³⁰³ It is also important that the maintenance work is decided upon after determining the cause of deterioration to ensure that no improper and aggressive treatments are done that could cause permanent damage.³⁰⁴

²⁹⁹ Hoon Sohn, *A Review of Structural Health Monitoring Literature 1996 – 2001*, (Cambridge: Los Alamos National Laboratory, 2002), 3.

³⁰⁰ Balageas et al., 13.

³⁰¹ Canada’s Historic Places, *Standards and Guidelines for the Conservation of Historic Places in Canada*, 2nd ed. (Gatineau: Parks Canada, 2010), 254.

³⁰² Department of Housing and Public Works, *Scope of Building Scope of Building Maintenance*, (2017) <<http://www.hpw.qld.gov.au/SiteCollectionDocuments/MMFScopePan.pdf>> [9 November 2017], 1.

³⁰³ Canada’s Historic Places, *Heritage Building Maintenance Manual*, 5.

³⁰⁴ Department of Arts, Heritage and the Gaeltacht, 241.

There is no one maintenance plan that can be used universally. Each plan must be adapted to fit the project at hand.³⁰⁵ Though the specific plans vary, they tend to follow the same steps. The steps are: identify the heritage value and character-defining elements, review the *Standards and Guidelines* to understand how to approach the work to be done, conduct an inspection of the building, determine and conduct any corrective or emergency maintenance on the building, and then develop a preventive, or cyclical, maintenance plan.³⁰⁶ To determine the exact maintenance needs of a building, it is important to collect and examine information on the building's current and past state. This can be done through: condition surveys, existing maintenance plans, discussing issues with the building's users, existing building and service records, and referencing other existing buildings that have similar features or issues to that of the project.³⁰⁷

2.8.1. Determining Corrective Maintenance Work

Corrective maintenance should be executed before creating a preventive maintenance plan to decrease the rate of deterioration on decayed elements, all while keeping the level of intervention to a minimum as not to alter or disturb historical features.³⁰⁸ When assessing the condition of building elements, it is common to base the decision of whether to repair on a condition rating system. Each item can be ranked in terms

³⁰⁵ Royal Institution of Chartered Surveyors, 8.

³⁰⁶ Canada's Historic Places, *Heritage Building Maintenance Manual*, 9-13.

³⁰⁷ Royal Institution of Chartered Surveyors, 23.

³⁰⁸ Mydin et al. 350.

of: building materials, causes of deterioration, how much the element is affected, its priority, the frequency of treatment, any visual identification, and the workers needed to complete the task.³⁰⁹ When assessing a building, it is usually broken down into its components. However, there is no one way to break them down. For example, Monumentenwacht, a conservation program based in Belgium, splits their buildings into 8 modules: roofs; gutters and outer drainage; roof construction and attic; support structure and spans; joinery and windows; finishes and furnishings; technical installations; and the site and environment.³¹⁰ On the other hand, the Maintenance Management Framework, under the Department of Housing and Public Works in Queensland, Australia, splits their building under 6 categories, which are: substructure, superstructure, finishes, fittings, services, and external services.³¹¹

When judging the priority of corrective maintenance work, it is usually done through a rating system based on the urgency of the repair. As with the component breakdown, the priority rating systems varies. For example, the Royal Institution of Chartered Surveyors uses ‘urgent’ for work to be done within 24 hours, ‘high priority’ for work to be done within a week, and ‘as and when’ for when the repair work is “not critical to the function of the building.”³¹² The Defences Estates organisation, which operates under the Ministry of Defence in the United Kingdom, uses instead ‘unavoidable’ for urgent tasks, ‘essential’ for work to be done within 3 to 6 months, ‘urgent’ for within

³⁰⁹ Vandesande, 77.

³¹⁰ Ibid., 77-78.

³¹¹ Department of Housing and Public Works, 2-8.

³¹² Royal Institution of Chartered Surveyors, 27.

one year, and 'desirable' when the work is not critical.³¹³ The use of the term 'urgent' is present in both rating systems, yet refers to very different time deadlines. This can cause confusion when comparing projects. No matter the rating system used, the criteria for determining the priority of corrective maintenance work are based on the speed of decay, the importance of the element to the functionality of the building, and the civil requirements that dictate what is reasonable for a building to be in usable condition. However, items with low priority should not be ignored as, if they are left to decay, the small repair work can easily evolve into a bigger issue.³¹⁴ Figure 32 showcases a sample of a corrective maintenance rating system.

Title: *Polkney Mill*
 Parish/Address: *Reedham*
 Date of last inspection: *04.05.05*
 Keys/access: *NMP7 Key*
 Owner: *NMPT*

File Number: *EW.WP.51*
 Date of Inspection: *05.02.06*
 Inspected by: *DEW*
 Weather conditions: *fine and windy*
 Photographic record: *colour slide*

 Condition of Element: C1=Good; C2=Fair; C3=Poor; C4=Very bad
 Priority of Work: P1=Desirable; P2=Essential; P3=Urgent
 N/A = Not Applicable (do not leave blank if relevant)
 F/I = Further Investigation (detailed examination of element/defect required)
 A/I = Additional information (notes/sketches on separate sheets)

GENERAL BUILDING FABRIC *C = 14 / P = 11*

*** Exterior	C1	C2	C3	C4	P1	P2	P3	N/A	F/I	A/I
1.0 GENERAL STRUCTURE										
1.1 Settlement, cracking, movement or loss of stability										
1.2 Failure of structural members										
1.3 Excessive deterioration of fabric										
1.4 Damp penetration										
1.5 General condition										
1.6 Additional information										

ivy growth to both sides of entrance door to east side. Open joints and cracking noted in last inspection report now faulted. Vegetation and elders growing up around tower and in channel.

Figure 32: Sample of a survey form showcasing a corrective maintenance rating system.³¹⁵

³¹³ Watt, *Surveying Historic Buildings*, 111.

³¹⁴ Royal Institution of Chartered Surveyors, 30-31.

³¹⁵ Watt, *Surveying Historic Buildings*, 356.

When determining the corrective maintenance plan, there are 6 general intervention approaches to consider. They are: abstention, mitigation, reconstruction, substitution, circumvention, and acceleration. Abstention refers to no intervention and leaves the building element as is. Mitigation works on changing the environment around the element rather than the element itself, in order to reduce deterioration. The opposite of mitigation is reconstitution, which exclusively acts on the fabric of the building element. Substitution is, as its name implies, the replacement of a material for another that enhances the element's performance. Specifically, the new material will match the original's placement and configuration. Circumvention is similar to substitution, but it does not match the original material's placement and configuration. This is more common for structural elements where, for example, the support system of the original is no longer sufficient for the building's loads. Finally, acceleration refers to demolition. This approach should only be chosen when there is no other alternative.³¹⁶

2.8.2. Determining a Preventive Maintenance Plan

A preventive maintenance plan is made to assign a set of jobs to be conducted at predetermined intervals or following prescribed criteria to reduce the chances of a failure or decreased performance of an element. This means that the work needs to not only be planned, but also to be constantly revised to take into account changes to

³¹⁶ Harris, 40-44.

these elements and new found information.³¹⁷ The tasks reflect each individual building element's likely maintenance cycle, and inspections should be made at regular intervals to observe any deterioration progression.³¹⁸

The tasks in a preventive maintenance plan can be broken up into four groups: day-to-day tasks, planned maintenance, cyclical maintenance, and preventive maintenance. Day-to-day tasks are typically small repair jobs, such as rectifying breakages and failures, and are often low value. Planned maintenance refers to the periodic routine task required to maintain the services of a building in a safe and efficient operating condition. Cyclical maintenance is the work done at irregular intervals that relate more to the buildings fabric. This can included cleaning, both internal and external, and decorating. Finally, preventive maintenance works with the issues identified in a condition assessment and “takes place to suit the pattern of deterioration of a building and the availability of funds for the purpose.”³¹⁹ To make the tasks easier to comprehend, they can be placed in a short checklist where they are divided into weekly, monthly and annual tasks.³²⁰ It is also important to document all maintenance and repair work, to keep as a reference if any issues to the building elements and fabric were to occur.³²¹ It should be noted that no matter how well a preventive

³¹⁷ Watt, *Building Pathology*, 252.

³¹⁸ Housing and Building Research Institute, 69.

³¹⁹ Royal Institution of Chartered Surveyors, 39-40.

³²⁰ Watt, *Surveying Historic Buildings*, 122.

³²¹ Canada's Historic Places, *Heritage Building Maintenance Manual*, 35.

maintenance plan works, there are always unexpected events that can cause disastrous damage to a building.³²²

The inspection component of preventive maintenance can be categorized in three groups. The first type of inspection is routine. This encompasses regular observations undertaken by the occupants of the building. The second type is general inspection. This is a visual or walk-through inspection of the main elements in a building conducted by suitably qualified personnel on an annual basis. The final inspection type is detailed, which includes a full condition assessment and should not exceed a 5 year period.³²³

2.8.3. People's Outlook on Maintenance

Typically, the owner or occupier of a building is in charge of conducting maintenance work. At times, some form of authority can provide assistance in the maintenance process, whether through financial aid, materials, equipment, or through services.³²⁴ Though many owners and occupiers are aware of the benefits of maintenance, they “seem to see it as something either beyond their means or beneath their dignity.”³²⁵ Another issue is that building maintenance is not yet a service that can be effectively traded as strategic market management, as it does not provide identical or

³²² Royal Institution of Chartered Surveyors, 27.

³²³ Housing and Building Research Institute, 74.

³²⁴ Department of Arts, Heritage and the Gaeltacht, 241.

³²⁵ Watt, *Surveying Historic Buildings*, 348.

interchangeable services, nor is it a standardised commodity.³²⁶ Another issue among owners and occupiers can be the inability to secure adequate funding, especially if the building had been neglected.³²⁷ “Accurate, detailed and well-presented maintenance plans and budget estimates are therefore essential in presenting an effective case for the maintenance budget.”³²⁸

2.9. Summary

The six factors that cause deterioration were identified as: atmospheric and climatic action; excess moisture; chemical, physical, and biological action; movement; fire; and human factors. The different types of surveys were discussed. Surveys can be divided by resolution quality, categorised as intensive or extensive, duration, categorised as synchronic or diachronic, and by type. The types are measured, valuation, archaeological, and building surveys. The three detail levels of recording, from lowest to highest detail, are reconnaissance, preliminary, and detailed recording. The use of photography was prevalent in all levels of detail. Hand-measuring, total station, and laser scanning documentation techniques were explored as options for obtaining measurements from a building. Photogrammetry, registering point clouds, producing drawings in AutoCAD, and creating Building Information Models were discussed as possible data processing worked that could be done after a survey. Equivalent frame modeling techniques for the seismic analysis of masonry structures

³²⁶ Vandesande, 66.

³²⁷ Femi Olubodun and Trevor Mole, “Evaluation of defect influencing factors in public housing in the UK,” *Structural Survey*, 17, no 3 (1999): 170.

³²⁸ Royal Institution of Chartered Surveyors, 32.

were presented along with equations to determine the strengths of the wall, such as compression, tension, and failure in both rocking and shear. Various monitoring methods were presented before delving into the procedures of determining both the corrective and preventive maintenance required for a building.

The information obtained from the literature review will be used to create a workflow for ensuring the ongoing maintenance of existing buildings. This workflow aims to compile the various steps presented into one easy to follow guide.

3 – METHODOLOGY

The proposed framework for ensuring the ongoing maintenance of existing buildings is split into five main sections: preliminary work, surveying and recording, data processing and analysis, identifying the maintenance plan, and selecting a monitoring strategy. The first step, ‘preliminary work’, establishes the scope of project. In this step, research is conducted on the history of the building to understand its significance and past deterioration conditions. A walk-through inspection is also conducted to better understand the building and its context. A plan is then conceived to determine how the surveying of the site will be addressed.

The following surveying and recording step works to obtain information on the building. This includes geometric data and its condition. The method of surveying depends on the scope of the project and the available resources. Hand measurements, the use of a total station, or the use of a laser scanner are all possible methods for documenting the geometric properties of the site. The areas of deterioration should be properly documented during the condition assessment of the site. Deterioration type, the size, and the location of each condition should be noted. Photography plays an essential role in the survey as it helps to provide context of the building and can showcase the building’s details, such as its character-defining elements and its decay.

Once the survey and recording is complete, the data needs to be processed and analyzed. Outputs from the data processing step must include as-built drawings and condition drawings. The as-built drawings must accurately reflect the current

geometry of the building, including deformations. The condition drawings should then use the as-built drawings as a base to illustrate the deterioration within the site. Depending on the scope of the project, a Building Information Model can be constructed to have a 3D representation of the site and its components. A structural analysis should be conducted to assess the building's performance. This analysis is split into a gravity load analysis and a seismic analysis. The gravity load analysis is used to determine whether the building's current condition can support the loads on the structure. This is important to assess since the structural elements may have decayed over time and the loads on the building may have changed. Also, as older buildings were typically not designed to withstand earthquake loads, a seismic analysis of the building should be conducted. This analysis is especially important for unreinforced masonry structures. Interventions should then be suggested and conducted if the building is deemed to be underperforming. The suggested interventions from both the gravity load analysis and seismic analysis should not interfere with the character-defining elements of the structure.

The information gathered from the previous steps should then be used to create a maintenance plan. The plan should include both corrective and preventive tasks. The corrective tasks work to repair the deterioration in the building, while the preventive tasks work to mitigate the possible sources of deterioration. Special attention should be given to ensure that none of the tasks in the maintenance plan will have adverse affects that can result in the loss of a building's character-defining elements.

The last step is selecting a monitoring strategy. Depending on the decay found within the building, various monitoring techniques can be set up to better understand the source of the issue. Aside for the case by case strategies, one monitoring method that should be implemented for all projects is the use of quinquennial surveys. These surveys are conducted every five years to document the changes to the building. This includes physical change, change of use, and new decay. The process of conducting quinquennial surveys is the same as the original framework process. However, the amount of overall work will be reduced as there is a basis to work from. For example, as-built drawings do not need to be produced from scratch, but instead can be updated every five years. The maintenance plan will also be updated every five years to account for any new corrective work required. The preventive maintenance plan may also need to be modified to reflect any new knowledge obtained about the sources of deterioration in the building. Since any new decay will be addressed early on, the amount of repair work required after each quinquennial survey will also be much less than the original survey, saving time and money. A workflow summarizing the framework steps can be seen in Figure 33.

To showcase the framework for the ongoing maintenance of buildings, Header House, under the recommendation from the NCC, was used as a case study. The following sections will discuss the specific methodology used for Header House.

Figure 33: Workflow for ensuring the ongoing maintenance of existing buildings.

3.1. Surveying Header House

The first steps of the workflow are research, inspection, and recording. Surveying acts as the basis of creating a maintenance plan. It provides the necessary information for making decisions on how to maintain and monitor a building. For this case study, the synchronic survey focused more on Header House itself rather than its surrounding environment.

3.1.1. Background and Research

Research on Header House and its site, Major's Hill Park, was conducted to better understand its heritage value, physical history, and its history of decay. The research began by reading through the documents provided by the NCC. They included a report made in 1987 on the site and building's history, as well as detailed architectural drawings produced in 2015 for a proposed renovation that has yet to be fully carried out. Aside from the information sent by NCC, there was little information specifically on Header House. Most of the other documents focused more on the history and significance of Major's Hill Park. As the building is recognised as one of Canada's Historic Places, it possesses a Statement of Significance, which aids in the building's research. However, as it is listed simply as 'Maintenance Building', rather than Header House, and it was last updated in 2008, it is evident that the statement is not up-to-date.³²⁹ As such, the Statement of Significance was reviewed

³²⁹ Canada's Historic Places, *Maintenance Building*, para. 1.

and updated. The updated Statement of Significance of the Header House, along with all its character-defining elements, can be found in Appendix A.

3.1.2. Inspection

The initial inspection was conducted to better understand the building and its context, starting with the exterior and working towards the interior. Further investigations were carried out to document visible deterioration and to create a plan for recording the site.

3.1.3. Recording

A detailed recording level was selected out of the level of detail options covered in the literature review to provide valuable information required for as-built drawings and maintenance planning. Photographs of the building, site, and detailed elements were taken with both a Nikon COOLPIX P520 camera and a Nikon D800 digital SLR camera. Images of detailed elements were corrected for white balance in Adobe Photoshop with the help of an IFRAO Standard Scale, as seen in Figure 34.

Figure 34: IFRAO Standard Scale

A FARO Focus 3D X 330 laser scanner was used to survey Header House. For exterior scans, the 'Outdoor 20m...' factory predefined scan profile with a 4x quality setting was used, while for interior scans, where the elements required to be scanned are much closer to the scanner, the 'Indoor ..10m' profile also with a 4x quality setting was used. The time required for each type of scan was about 10 minutes and 5 minutes, respectively. A series of four recording sessions was conducted. The first recording session was of the exterior. However, as the session was held in the evening, the images from the laser scanner were very dark, so the session ended early and was postponed to another day. A total of 8 scans were taken within the 1.5 hour long second session of the exterior. The third recording session was of the interior. As that session coincided with the closure of the building for the season, a time limit for site access was put into place and the whole interior could not be recorded. The final recording session consisted of scanning the whole of the interior with a total of 12 scans in the 1.5 hour long session. Spherical targets were used for both the exterior and interior to assist with registering the scans.

3.1.4. Limitations

There were a number of access limitations when surveying the site, specifically the interior. Though owned by the National Capital Commission, Header House is currently rented out to Tavern on the Hill, a gourmet hot dog canteen. Site visits had to be coordinated to not interfere with the business' services. In addition, the canteen closed for the season on November 1st, 2017, thus restricting access to the interior. According to the NCC, Header House is only used as storage when not in season. The

building is heated, but no snow removal is conducted on the exterior steps to provide access to the building. Access into the newly constructed wooded shed adjacent to Header House was also restricted.

Aside from site access limitations, the current use of the interior also posed some surveying restrictions. The garage is currently used as the cooking station, so it is full of cooking equipment and appliances. This made surveying the space difficult, as there was little room to move around and to position the laser scanner. Both the attic and basement of the Header House are currently used as storage spaces. This meant that some walls and part of the floor could not be surveyed. In addition, there is a large freezer in the basement that obscures a corner of the space. The furniture and items on the main floor were in a constant state of change, as they were preparing for the closure for the season. This meant that scans would differ significantly from one site visit to the other.

The final surveying restriction was that only non-destructive surveying techniques could be utilized. The tenants did not want any surveying work to interfere with the use of the space. As such, walls and floors could not be opened up to inspect the structural and building envelope elements. This posed the biggest issue for the roof, as the structural systems were covered with gypsum. Since most of the structural elements and their connections could not be properly surveyed and assessed, a high degree of uncertainty is present for the structure's overall condition. Further investigation is required to obtain the proper overall condition of Header House.

3.2. Header House's Current Condition

The next step in the workflow is to present the current condition of the site. This presentation provides the backbone for determining the course of action for the maintenance of the building. For Header House, this information was presented through as-built and condition drawings, a Building Information Model, and a structural analysis.

3.2.1. Photogrammetry and Point Cloud Registration

Before creating the architectural drawings of Header House, the point cloud scans needed to be registered to accurately showcase the building as a whole. The registering process was conducted through FARO's SCENE program and it began with importing the scans into the software. The 'Process Scans' function was then used to perform an automatic registration of the scans using the 'Top View and Cloud to Cloud' method. To save processing time, the 'Colorize Scans' feature was disabled. After the process was complete, it was evident that using just the 'Top View and Cloud to Cloud' method was not enough to register all the scans. This was most likely caused by an insufficient amount of overlap between the scans.

To counteract the insufficient overlap, targets were marked between the scans. The first targets that were marked were the spheres that were placed during the recording stage. As there were a limited number of spheres when recording the site, other targets needed to be marked to be able to register the scans together. These targets

were in the form of a single point or as a plane. These elements were identified using natural points visible on two or more point clouds. A giveaway that there were enough markers between the scans to properly register them was when the software automatically identified and named the corresponding targets.

With the targets in place, it was time to process the remaining scans again, but this time using the ‘Target Based’ registering method. The ‘Verify Registration’ process was utilized to check if the scans had been correctly positioned. The final overall mean point error was 0.9 mm. The last step to finalize the point cloud was to use the ‘Process Scans’ function with only the ‘Colorize Scans’ feature enabled to add the colours from the images taken by the laser scanner to the point cloud itself.

Since architectural drawings represent the 3D data in 2D, there was no need to bring the entire point cloud of the Header House into AutoCAD; orthophotos were sufficient. To create the orthophotos, the scanned environment surrounding Header House first needed to be out of view. This was done through the use of a clipping box created in the 3D view of the scan. This clipping box was created to be in-line with the exterior walls of the building. The camera mode was then changed to ‘Orthographic’ to view the point cloud as it would appear in the orthophoto, rather than isometrically. The point cloud and its clipping box were then rotated to capture each described view of the building. The clipping box was also manipulated to cut through the building for the cut-section drawings. The final orthophotos of each chosen view were created using the ‘Create orthophoto’ function when the point

cloud was properly aligned to the view on the screen. These images were exported as TIFF files. Figure 35 showcases a screenshot of the registered point cloud in SCENE.

Figure 35: Screenshot of registered point cloud in SCENE.

Although the final point cloud, as seen in Figure 35, provides accurate measurements for Header House as a whole, the quality of the scans do not showcase the details of the building, such as the mortar and stone connection, in sufficient detail to be traced in AutoCAD. To counter this lack of quality, photogrammetry was used to capture the detailed elements. Agisoft PhotoScan was the program used to create the photogrammetric models. The models were separated by area since the point clouds from the laser scanner already provide the relationship between the selected areas of Header House. The areas chosen were broken down into the exterior, basement, attic, garage, and the main level. Each model followed the same workflow. The first step was to import the photographs into PhotoScan using the ‘Add Photos’ command. For the exterior model, the images had to then be masked to isolate the building. Masking

refers to a selected area that will not be processed to create the photogrammetric model. This was first done by marking out the area to mask on each individual image through the use of the 'Intelligent Scissors' tool in PhotoScan. By masking the images for the exterior model, the processing time was reduced since not every element of the image was considered for the model. The mask ensured that the final model consisted only of the building itself, rather than its environment. The interior models did not need to be masked, as all the information provided in the photographs was of use for the final model.

Once the images were imported and masked, they were used to create a sparse point cloud with the 'Align Photos' command. This sparse point cloud provides the relative form of the building, but does not have a scale or orientation connected with it. As such, point locations were added to scale and orient the model to that of the point cloud produced with the laser scanner. The positions of these points were taken directly from the laser scan's point cloud in Autodesk's ReCap, after importing the scans into the program and switching to RealView mode. RealView mode allows the user to view the 3D images taken by the laser scanner. This allows for a more precise selection of a point compared to the default view, since in the default view there is a possibility of selecting a point behind or in front of what is desired by accident. In RealView, what you see is what you get, and so there is no way of accidentally selecting a point in a different plane. The position of the point can be viewed in the bottom left corner of the screen as the mouse moves around the image. The use of the annotation function in ReCap was used to name and identify the chosen natural point locations. With the known locations, the points were added in PhotoScan through the

use of markers. Markers were created with the same name as the points and the x, y, and z locations were inputted in the ‘Reference’ tab. The next step was to add the markers to the images that contained the point. However, not every image that contained the point needed a marker. Three markers were placed per point to secure its location on the point cloud. The rest of the images showcased the approximate location of where that point was located. With the markers in position, the ‘Optimize Cameras’ tool was used to orientate and scale the model in reference to the point locations. The accuracy and error of each point can be viewed under the ‘Reference’ tab in PhotoScan. Table 5 showcases these values for the points used in the exterior photogrammetric model of Header House.

Table 5: Markers’ location, accuracy, and error for the exterior model

Marker	X (m)	Y (m)	Z (m)	Accuracy (m)	Error (m)
101	993.1197	1009.0460	998.3739	0.0050	0.003593
102	989.0381	1002.522	998.6547	0.0050	0.008768
103	988.8916	1002.6006	1001.3384	0.0050	0.006759
104	993.0165	1009.2001	1001.3412	0.0050	0.005507
105	990.9179	1005.8661	1004.0458	0.0050	0.006177

With the sparse point cloud scaled and orientated, the ‘Building Dense Cloud’ command, under the ‘Workflow’ tab, was used to create a denser point cloud. The final photogrammetric model was created using first the ‘Build Mesh’ command, followed by the ‘Build Texture’ command, creating a textured mesh. To generate the orthophotos that would later be used to create the architectural drawings, the ‘Build Orthomosaic’ command, also found under the ‘Workflow’ tab, was used. The

orthophotos were then exported as a TIFF file. The final textured mesh model can be seen in Figure 36.

Figure 36: Screenshot of textured mesh model of Header House's exterior in PhotoScan.

3.2.2. As-Built and Condition Drawings

Using the orthophotos created in SCENE and in PhotoScan, the as-built drawings were produced in AutoCAD. The images were imported at a 1:1 scale to be traced to create the final architectural drawings. The as-built drawings consist of a roof plan, three floor plans, four elevation drawings, and six cut-sections to showcase the whole building. The orthophotos from SCENE were first used to produce the outlines of the Header House's features and form. Once completed, the higher quality PhotoScan orthophotos were placed in their corresponding positions and used to draw the finer details of each drawing. Some older drawings of the building that were produced in 2015 and were provided by the NCC were used as a reference for areas that were not

captured in enough detail. However, the drawings did not showcase the entirety of the building and Header House had undergone some changes since those drawings were produced. This was most evident in the old basement floor plan that included partitions that no longer exist, and lacked others that are now present. The as-built drawings can be found in Appendix B, and Figure 37 showcases the elevation drawing of the south-east façade of Header House as an example.

Once the as-built drawings were complete, the condition drawings were produced. These condition drawings exhibit the building's various deterioration conditions on the as-built drawings. These conditions were identified during the investigation process and added to the architectural drawings in their respective locations. A legend was created to help identify what each pattern and line colour represents in the drawings. The condition drawings can be found in Appendix D and Figure 38 showcases a snapshot of the condition drawing of the south-east elevation as an example. In addition to these drawings, a glossary of deterioration, found in Appendix C, was created to better describe all the deterioration present in Header House.

Figure 37: South-East elevation drawing of Header House.

Figure 38: Snapshot from the condition drawing of the south-east elevation of Header House, where pink marks missing elements, blue marks mortar spalling, red marks cracks, and mustard marks a stain in the masonry.

3.2.3. Building Information Model

When discussing the scope of the case study project with the National Capital Commission, having a Building Information Model for the Header House was set to have the lowest priority. This was due to two factors. The first was that NCC

primarily works with 2D drawings in AutoCAD for their projects. The other factor was that they have few computers able to run Revit, a 3D modeling software that is most commonly used to create Building Information Models, and they have few employees that understand how to use the software. A Building Information Model was still created for Header House, however, a simplified model was constructed to be used as a reference whenever Revit and Building Information Models become more prominently used in NCC or, specifically, Header House projects.

The BIM was constructed using the point cloud as a reference. The point cloud was inserted into the Revit file and a section box was created to focus on the building itself. The next step was to create the guidelines for the floor levels and wall placements. In the elevation view, the floor levels were manually placed in Revit to match those on the point cloud. The final levels, from bottom to top, were: Basement, Level 1 Garage, Mini Riser Level, Level 1 Main, Level 2 Garage, Main Floor Roof Line, Level 2 Main, Garage Roof Line, and Roof Line. In the plan views at each level, grid lines were drawn to indicate where the main walls of the building would be placed. The next step was to set-up the walls. Due to restrictions against destructive testing, the exact wall configuration could not be determined. As such, the main building exterior walls were assumed to be made of only two wythes: a 280 mm limestone exterior and a 180 mm clay brick interior with no air space in between. The garage walls, which were constructed at a later time and have a thickness of 150 mm, were constructed as a limestone exterior, a wood framing core, and a gypsum interior, as per the existing drawings. These walls were then placed following the previously set up grid guidelines. Header House has exterior limestone wall sweeps on both the

main building and the garage. These sweeps were defined to match the dimensions of the point cloud and placed accordingly. With the walls completed, the next step was to add the openings. Window models that were similar to those found in the building were imported and adjusted to accurately match the true form of the wooden window frames. The limestone ledges under the windows were also added at this point. The garage door, main entrance door, interior doors, and the door frames models were also created by adjusting imported models to fit the specifications needed to match the point cloud.

After the walls and openings were complete, the roofs were modeled. The garage roof slopes at a 30° angle towards the North and South walls. The rooflines of the complex main building roof were modeled in sections, all at a slope of 40°. These sections were then connected using the 'Join Roof' command under the 'Modify', 'Roofs', and 'Geometry' tabs. The small main entrance segmental roof was also added at this time. A new object file, or 'family', was created and modeled for the fascias that were placed along the roof's circumference. The chimney cap was also modeled using a custom family for the copper chimney exhausts. The concrete floor footprints for the garage and basement, as well as the wooden floors of the main and attic floors were then drawn and modeled. The stair runs were then created for the entrance porch and inter-storey floors. The interior staircases and railings were made up of wood, while the exterior porch staircase was made of limestone with black iron railings. A custom object family was created to construct the Canada Centennial logo located on the front railing of the porch.

With the building elements completed, plumbing fixtures and appliances were added to the model. The appliances that were added were the walk-in freezer in the basement, and the revolving door freezer and glass fridge in the garage. A rough landscape was also modeled to showcase the building's relationship with its surroundings. The newly constructed wooden shed at the East face and the wooden deck at the back of the building were not modeled as they are temporary construction and are not connected to Header House. Figures 39 and 40 are renderings of the east and west views of the Header House Building Information Model in Revit.

The Header House BIM also assisted in creating the models for the structural analysis. The BIM could not be used as an import for the structural model due to variations in the software between the programs, but it did help with understanding the geometric information required for the structural analysis. The dimensions of the BIM exterior unreinforced masonry walls were implemented in the OpenSees model to assess Header House's seismic performance.

Figure 39: Header House BIM west view.

Figure 40: Header House BIM east view.

3.3. Structural Analysis

The structural analysis of a building helps to determine if any interventions are required to ensure that the building is safe to use. The first step of the structural analysis is to determine the loads on the structure and assess the governing load combinations. The loads required are dead load, live load, snow load, wind load, and earthquake load. The mass and dead loads of the structure were approximated using values from the Canadian Wood Council's *Wood Design Manual*.³³⁰ The mass and dead loads of the brick and limestone are per millimeter thickness. As the thicknesses of these materials are unknown, they also had to be approximated. The overall thickness of the wall, taken from the survey is 460 mm. The point where the brick and stone layer meet within the wall are unknown since on one side of the door and window frames there is stone and on the other there is brick. Under the assumption

³³⁰ Canadian Wood Council, *Wood Design Manual 2010* (Ottawa: Canadian Wood Council, 2010), 653-654.

that they connect about half way through the door frame, the brick and limestone thicknesses are assumed to be 180 mm and 280 mm, respectively. The other materials in the building were determined using existing drawings produced by NCC in 2015.³³¹ They are the cedar shingles, the 38 mm by 89 mm wood joists spaced at 400 mm for the attic floor, and the 38 mm by 235 mm wood joists spaced at 400 mm for the ground floor. When the previous drawings were produced, it seems as though they also had survey limitations as the roof truss members are unknown. As the attic floor is connected at the top of the masonry wall, and assumed to also be connected with the roof truss system, it was assumed that the roof also uses 38 mm by 89 mm wood rafters spaced at 400 mm. The species of wood used in Header House is unknown. For the sole purpose of determining their mass from the *Wood Design Manual*, D.Fir-L was assumed. When surveying Header House, new vinyl flooring had just been added, so its mass was also taken from the *Wood Design Manual*. Table 6 contains the assumed mass and dead load values.

The live loads on the building were also taken from the NBCC. The ground floor was given as live load of 4.8 kPa as it can be characterized as an assembly area without fixed seating. Since the attic floor is assessable by stairway, the live load was taken as 1.4 kPa.³³² Finally, the roof also has a live load of 1.0 kPa.³³³

³³¹ National Capital Commission, *Major's Hill Park, Header House Universal Accessibility/ Full Renovation* (Ottawa: National Capital Commission, 2015), 1-7.

³³² National Research Council Canada, Division B 4-8.

³³³ *Ibid.*, 4-10.

Table 6: Assumed mass and dead load of materials.³³⁴

Material	Mass (kg/m²)	Dead Load (N/m²)
Brick	1.8 per mm Thickness	17.7 per mm Thickness
Limestone	2.7 per mm Thickness	26.3 per mm Thickness
38x89 D.Fir-L at 400 mm spacing	-	50
38x235 D.Fir-L at 400 mm spacing	-	120
Cedar Shingles	10	100
Vinyl Tile Flooring	7	70

The snow load on the building was calculated using the following equation from the NBCC: $S = I_s [S_s (C_b C_w C_s C_a) + S_r]$, where I_s is the importance factor for the snow load, S_s is the 1 in 50 year ground snow load, C_b is the basic roof snow load factor, C_w is the wind exposure factor, C_s is the slope factor, C_a is the shape factor, and S_r is the 1 in 50 year ground snow load that is not greater than $S_s (C_b C_w C_s C_a)$.³³⁵ The values of S_s and S_r are taken from Table C-2 in the NBCC and are 2.4 kPa and 0.4 kPa, respectively.³³⁶ As Header House has a ‘normal’ importance, I_s is 1.0. The remaining factors C_b , C_w , C_s , and C_a were determined using the NBCC and are 0.8, 1.0, 1.25, and 0.75, respectively.³³⁷ Therefore, the final snow load is 2.2 kPa. Finally, the wind load was calculated using the following equation from the NBCC: $W = I_w q C_e C_g C_p$,

³³⁴ Canadian Wood Council, 653-654.

³³⁵ National Research Council Canada, 4-14.

³³⁶ Ibid., C-26.

³³⁷ Ibid., 4-14 – 4-15.

where I_w is the importance factor for wind load, q is the reference velocity pressure, C_e is the exposure factor, C_g is the gust effect factor, and C_p is the external pressure coefficient.³³⁸ The value of q was taken from Table C-2 in the NBCC for Ottawa as 0.41 kPa.³³⁹ As Header House has a ‘normal’ importance, I_w is 1.0. The remaining factors C_e , C_g , and C_p were determined and calculated using the NBCC and are 0.9, 2.0, and 0.72, respectively.³⁴⁰ Therefore, the final wind load is 0.53 kPa. The earthquake loads used for the seismic analysis will be discussed later on in the methodology.

The structural analysis is split into a gravity load analysis and a seismic analysis. The gravity load analysis deals with load combination cases 1 to 4, as introduced in section 2.6, and the seismic analysis deals with load combination case 5. For Header House, an accurate gravity load analysis could not be conducted due to the surveying limitations and restrictions. The condition of most of the structural elements is unknown so their performance cannot be properly assessed. Though the gravity load analysis was not conducted, the governing load combination cases for each of the exterior unreinforced masonry walls were calculated. For the south-east and north-west walls, case 2 governed with 619 kN and 613 kN, respectively. For the south-west and north-east walls, case 3 governed with 348 kN and 357 kN, respectively.

³³⁸ Ibid., 4-16.

³³⁹ Ibid., C-26.

³⁴⁰ Ibid., 4-16 – 4-17.

For the seismic analysis, the equivalent frame modeling approach, as introduced in section 2.6., was used for the four main unreinforced structural masonry walls of Header House. They are the south-east, south-west, north-west, and north-east exterior walls of the original building. The walls were renamed for the model as front, left, back, and right walls, respectively, to help organise the component names. Figure 41 showcases the ground floor plan of Header House with the corresponding model wall names. The four walls that were modeled are those that define the retail space.

Figure 41: Ground floor plan of Header House showcasing the model names of each wall.

These walls were chosen as they pose a significant damage risk during an earthquake. Other walls, such as those in the garage addition, are constructed from light frame timber and are much less stiff than the masonry walls. This lower stiffness will not significantly affect the structural dynamics of the building. For simplicity, these walls were omitted from the model. Only the mass of the roof, along with the masonry walls in the attic and the masonry chimney, and the mass of the floor were considered for the model. The masonry attic exterior walls are relatively small and hold much

less weight than the walls below them, so they have a much lower vulnerability during an earthquake. The chimney, on the other hand, is more slender than the attic walls, so there is a larger possibility that it could sustain substantial damage. However, due to the investigation constraints previously mentioned, very little geometrical information is known about the chimney, such as the thickness of the stone and brick, so it could not be accurately modeled.

The seismic model of Header House was created in OpenSees. OpenSees is an object-oriented open source software framework, primarily written in C++, and is used for the nonlinear analysis of structural and soil systems.³⁴¹ Each wall was assessed on an individual basis. Two model files were created for the two parallel directions; one for the front and back walls and another for the left and right walls. Each wall was modelled using two equivalent frames. Within each model file, the four frames are placed at different horizontal locations so that they can be assessed individually. These two frames represent the limestone and clay brick layers of the wall. Due to the surveying limitations, the connection between these two layers is unknown. The two layers could work together as one unit, as individual walls, or only partially work together. In addition, the effect of the floors on the two opposing walls is also unknown. The floors are supported with wood beams that are inserted into cut-outs in the masonry wall. It is unknown how deep these beams are within the walls and whether they only reside in the clay brick layer or also in the limestone exterior. The

³⁴¹ Frank McKenna, "OpenSees: A Framework for Earthquake Engineering Simulation." *Computing in Science & Engineering* July/August (2011): 61.

floor's rigidity is also unknown. It could be flexible enough to allow the two opposing walls to move independently of each other or rigid enough to make the walls move together as one. To counter these unknowns, the walls were modelled in two different ways: completely independent, representing a flexible diaphragm, and completely connected, representing a rigid diaphragm. In terms of the wall layer connectivity, the walls were either modeled as working together as one unit or as two separate walls with no connection. To simplify the number of model cases, the diaphragm rigidity cases and the wall layer connectivity cases were grouped together to produce a bounded analysis of the two extreme scenarios. Therefore, the two cases are: two cohesive limestone and brick walls with a rigid diaphragm connection and four individual walls, two brick and two stone, with a flexible diaphragm connection.

In addition to the unknown connectivity of the structure, the surveying limitations meant that the mechanical properties of the masonry walls are unknown. The walls could not be tested to determine necessary properties of the wall, such as its compressive strength. To model the structure, a bound analysis was conducted to determine the possible properties that the walls and their components could have. Similarly to the connectivity ranges, the lowest and highest values in the range were used to conduct the seismic analysis. Overall, four cases were modeled for the Header House seismic analysis: lower bound strength with a flexible diaphragm, lower bound strength with a rigid diaphragm, upper bound strength with a flexible diaphragm, and upper bound strength with a rigid diaphragm.

One of the most important values needed to analyse an unreinforced masonry wall is its compressive strength. As mentioned earlier, this can be calculated using the strength of the wall's masonry units and mortar. The equation used for this model was *Eurocode 6's* $f_{M,c} = K f_b^\alpha f_m^\beta$.³⁴² During a conversation with Serge Morin, a Construction Supervisor at NCC, he mentioned that based on his experience with other NCC owned buildings built around the same time with similar materials, the exterior walls of the Header House are composed of clay brick, lime mortar, and limestone that was most likely quarried in the Ottawa-Gatineau area.³⁴³ Through a literature review, it was found that the compressive strengths of historic clay brick, limestone, and lime mortar vary from 4 to 32 MPa, 20 to 180 MPa, and 0.5 to 10 MPa, respectively.³⁴⁴ Also, in North America, clay brick, both historic and new, fall within the compressive strength range of 12 to 200 MPa.³⁴⁵ Ken Trischuk, an engineer at the National Research Council of Canada, stated that limestone quarried in the Ottawa-Gatineau area has a 100 to 140 MPa compressive strength range and that 4 MPa was quite low for clay bricks found in historic buildings in Ottawa.³⁴⁶ Therefore, the final range of compressive strengths used in the analysis are 12 to 32 MPa, 100 to 140 MPa, and 0.5 to 10 MPa for the clay brick, limestone, and lime mortar, respectively.

³⁴² European Committee For Standardization, 35.

³⁴³ Personal communication with Serge Morin on March 2, 2018.

³⁴⁴ Kržan et al., 212-213.

³⁴⁵ Robert Drysdale and Ahmad Hamid, *Masonry Structures Behaviour and Design*. Canadian ed. (Mississauga: Canada Masonry Design Centre, 2005), 137.

³⁴⁶ Personal communication with Ken Trischuk on July 21, 2018.

Other necessary value ranges needed for the seismic analysis were determined through a literature review. Diagonal tensile strength of masonry ranges from 5 to 10% of the compressive strength,³⁴⁷ the friction coefficient between the mortar and masonry unit ranges from 0.4 to 0.8,³⁴⁸ the masonry cohesion coefficient ranges from 0.01 to 0.1,³⁴⁹ and the initial shear strength of masonry ranges from 0.29 to 0.90 MPa.³⁵⁰ Table 7 summarises the ranges used for the seismic analysis.

Table 7: Summary of the range of material properties for Header House.

Variable	Range
Clay Brick Compressive Strength ($f_{b, brick}$)	12 to 32 MPa
Limestone Compressive Strength ($f_{b, stone}$)	100 to 140 MPa
Lime Mortar Compressive Strength (f_m)	0.5 to 10 MPa
Diagonal Tensile Strength ($f_{M,t}$)	5 to 10% of $f_{M,c}$
Friction Coefficient (μ)	0.4 to 0.8
Masonry Cohesion Coefficient (ζ)	0.01 to 0.1
Initial Shear Strength (f_{vo})	0.29 to 0.90 MPa

These material properties and calculated loads were used to create the model in OpenSees. The piers and spandrels in the equivalent frames were modeled using elastic beam column members with zero length members to simulate the behaviour of the masonry walls during an earthquake. The height of the spandrels over arched

³⁴⁷ Kržan et al., 221.

³⁴⁸ Magenes and Calvi, 1095.

³⁴⁹ Petrovčič and Kilar, 11.

³⁵⁰ Kržan et al., 221.

openings was assumed to run from the top of the arch to the top of the wall. The rigid joints were modeled as joint offsets of the elastic beam column members. Details of each component will be explained later in the section. Figure 42 shows the roof plan of Header House and the corresponding model wall names. Figures 43 to 46 illustrate the equivalent frame models of the four walls.

Figure 42: Roof plan of Header House showcasing the model names of each wall.

Figure 43: Front wall equivalent frame model.

Figure 44: Left wall equivalent frame model.

Figure 45: Back wall equivalent frame model.

Figure 46: Right wall equivalent frame model.

The first step is to create the node locations. The nodes, as seen in Figures 43 to 46, connect two elements together. The locations of the pier and spandrel elements are in the centre of the wall area for which they are equivalent to. The two nodes required for each zero length element, were placed in the same location in the centre of the pier or spandrel. There are three types of nodes: base nodes located at the bottom of the wall, zero length member nodes, and the rigid joint nodes. The rigid joint nodes are all other nodes that are not at the base of the wall or at zero length members.

With the nodes in place, their fixity needs to be defined. In this case, the only nodes that need to be fixed are those at the base. Since it is a fixed end connection, the nodes were fixed in the x direction, y direction, and for rotation. The next step was to define the diaphragm constraints. All of the wall models have constraints between the nodes for each individual zero length member. For piers, they are constrained together in the y direction and in rotation, to only allow x-direction movement in the zero-length element. For spandrels, they are constrained together in the x direction and in rotation. For the model case where the brick and stone layers act together and the floor between the walls is assumed to be very rigid, extra constraints are added between the rigid joint nodes of the walls. For these interlayer connections, they are constrained in both the x and y direction. To model the rigid floor, x direction constraints are added to all the rigid joint nodes, which represent the location of the floors, and in the y direction for the rigid joint nodes at the top of the walls between the two brick walls. The last step for the nodes is to define each one's mass. These masses differ from the gravity loads, which will be added in a later step. The nodal mass takes into account the mass of the structure plus 25% of the snow load, as per

NBCC clause 4.1.3.2.³⁵¹ As the mass distribution between the two wall layers was unknown, assumptions had to be made to determine the mass at each node. The roof load was distributed to the walls by thickness ratio for the stone and brick walls. The floor loads were transferred only to the brick walls as the depth that the floor joists penetrate the exterior walls was also unknown. These assumptions can change the final behaviour of the walls, especially for the flexible diaphragm model case, as the walls act independently from one another.

The next step in creating the model was to define the material properties. The material properties in OpenSees represent the behaviour of an element. In this case, a major part of the material properties is the strength of the piers and spandrels, as previously discussed in section 2.6. Table 8 identifies the equations used to calculate the strength of each failure mode for both types of elements. The governing failure mode and strength of each pier and spandrel in the model, for both the lower and upper bounds, are compiled in Tables 9 and 10.

³⁵¹ National Research Council Canada, Division B 4-5.

Table 8: Equations used to calculate the strength of each failure mode.

Element	Failure Mode	Equivalent Shear Strength Equation
Pier	Rocking	$V_{Pr,r} = \psi \frac{\sigma_0 t_{pr} l_{pr}^2}{2h_0} \left(1 + \frac{\sigma_0}{f_{M,c}} \right)$
	Shear Sliding	$V_{Pr,t} = l_{pr} t_{pr} \frac{f_{M,t}}{b} \sqrt{1 + \frac{\sigma_0}{f_{M,t}}}$
	Diagonal Shear Cracking	$V_{Pr,s} = l_{pr,c} t_{pr} (f_{vo} + \mu \sigma_d)$
Spandrel	Rocking	$V_{Sp,r} = \frac{F_p h_{sp}}{l_{sp}} \left(1 - \frac{F_p}{\kappa f_{M,c} h_{sp} t_{sp}} \right)$
	Shear	$V_{Sp,s} = \tau_0 h_{sp} t_{sp}$

Table 9: Governing failure modes and strengths of the front and back walls.

Wall	Material	Element	Lower Bound		Upper Bound	
			Governing Failure	Equivalent Shear Strength (kN)	Governing Failure	Equivalent Shear Strength (kN)
Front	Limestone	Pier - Left	Rocking	15.38	Rocking	15.41
		Pier - Right	Rocking	15.38	Rocking	15.41
		Spandrel - Top	Shear	10.80	Rocking	64.24
		Spandrel - Bottom	Shear	28.28	Rocking	440.66
	Brick	Pier - Left	Rocking	8.51	Rocking	8.57
		Pier - Right	Rocking	8.51	Rocking	8.57
		Spandrel - Top	Shear	1.57	Rocking	9.36
		Spandrel - Bottom	Shear	4.12	Rocking	64.22
Back	Limestone	Pier - Top Left	Rocking	1.88	Rocking	1.88
		Pier - Top Middle	Rocking	15.25	Rocking	15.36
		Pier - Top Right	Rocking	1.80	Rocking	1.80
		Pier - Bottom Left	Rocking	20.09	Rocking	20.15
		Pier - Bottom Middle	Rocking	73.50	Rocking	74.05
		Pier - Bottom Right	Rocking	20.05	Rocking	20.11
		Spandrel - Top	Shear	10.80	Rocking	67.32
		Spandrel - Bottom	Shear	34.19	Shear	839.89
	Brick	Pier - Top Left	Rocking	1.29	Rocking	1.29
		Pier - Top Middle	Rocking	7.31	Rocking	7.49
		Pier - Top Right	Rocking	1.24	Rocking	1.24
		Pier - Bottom Left	Rocking	9.81	Rocking	9.91
		Pier - Bottom Middle	Rocking	33.67	Rocking	34.52
		Pier - Bottom Right	Rocking	9.82	Rocking	9.92
		Spandrel - Top	Shear	1.57	Rocking	9.81
		Spandrel - Bottom	Shear	4.98	Shear	122.39

Table 10: Governing failure modes and strengths for the left and right walls.

Wall	Material	Element	Lower Bound		Upper Bound	
			Governing Failure	Equivalent Shear Strength (kN)	Governing Failure	Equivalent Shear Strength (kN)
Left	Limestone	Pier - Left	Rocking	7.83	Rocking	7.85
		Pier - Right	Rocking	7.83	Rocking	7.85
		Spandrel - Top	Shear	10.80	Rocking	68.26
		Spandrel - Bottom	Shear	28.28	Rocking	468.20
	Brick	Pier - Left	Rocking	3.76	Rocking	3.80
		Pier - Right	Rocking	3.76	Rocking	3.80
		Spandrel - Top	Shear	1.57	Rocking	9.95
		Spandrel - Bottom	Shear	4.12	Rocking	68.23
Right	Limestone	Pier - Top Left	Rocking	7.67	Rocking	7.69
		Pier - Top Right	Rocking	7.67	Rocking	7.69
		Pier - Bottom Left	Rocking	37.36	Rocking	37.54
		Pier - Bottom Right	Rocking	37.36	Rocking	37.54
		Spandrel - Top	Shear	10.80	Rocking	63.82
		Spandrel - Bottom	Shear	34.19	Shear	839.89
	Brick	Pier - Top Left	Rocking	4.41	Rocking	4.47
		Pier - Top Right	Rocking	4.41	Rocking	4.47
		Pier - Bottom Left	Rocking	18.82	Rocking	19.16
		Pier - Bottom Right	Rocking	18.82	Rocking	19.16
		Spandrel - Top	Shear	1.57	Rocking	9.30
		Spandrel - Bottom	Shear	4.98	Shear	122.39

For unreinforced masonry structures, the *Pinching4* material has been used to accurately model the wall's hysteresis behaviour.³⁵² This material was used for the zero length members. *Pinching4* is “a uniaxial material that represents a 'pinched' load-deformation response and exhibits degradation under cyclic loading.”³⁵³ The code for constructing this material, and its needed inputs, is as follows:

```
uniaxialMaterial Pinching4 $matTag $ePf1 $ePd1 $ePf2 $ePd2 $ePf3 $ePd3 $ePf4 $ePd4 <$eNf1
    $eNd1 $eNf2 $eNd2 $eNf3 $eNd3 $eNf4 $eNd4> $rDispP
    $rForceP $uForceP <$rDispN $rForceN $uForceN > $gK1
    $gK2 $gK3 $gK4 $gKLim $gD1 $gD2 $gD3 $gD4 $gDLim
    $gF1 $gF2 $gF3 $gF4 $gFLim $gE $dmgType
```

where \$matTag is the integer tag identifying the material, variables with ePf and eNf are the positive and negative values defining the force points on the response envelope, as shown in Figure 47, variables with ePd and eNd are the positive and negative values defining deformation points on the response envelope, \$rDispP and \$rDispN are the values defining the ratio of the deformation at which reloading occurs to the maximum and minimum historic deformation demand, \$rForceP and \$rForceN are the values defining the ratio of the force at which reloading begins to force corresponding to the maximum and minimum historic deformation demand, \$uForceP and \$uForceN are values defining the ratio of strength developed upon unloading from negative load to the maximum and minimum strength developed under monotonic loading, variables with gK are values controlling the unloading

³⁵² Lowes et al., 34.

³⁵³ OpenSees, *Pinching4 Material* (7 April 2012).
http://opensees.berkeley.edu/wiki/index.php/Pinching4_Material [8 August 2018].

stiffness degradation, variables with gD are values controlling the reloading stiffness degradation, variables with gF are values controlling the strength degradation, \$gE is a value used to define maximum energy dissipation under cyclic loading, and \$dmgType is a string to indicate the type of damage, with the options ‘cycle’ and ‘energy’.³⁵⁴

The force points are calculated using the percentages of the governing strength of the member, V_{ult} , and the deformation points are calculated using the percentages of the wall height, in this case 3.59 m, to produce the desired envelope as mentioned in section 2.6. The rest of the values are typically determined after experimental tests. Due to the limitations set in place at Header House, no such experiments could be done on the building’s materials. As such, other experiments, in a similar context had to be utilized to determine their values. In a 2013 thesis by Arno Barandun, experiments were conducted on unreinforced masonry walls to record their horizontal quasi-static cyclic displacement history.³⁵⁵ This information was then used as a basis to create models for the walls in OpenSees. The *Pinching4* was used in their model, and the remaining variables for Header House models match those used in Barandun’s model.³⁵⁶ Table 11 summarizes the *Pinching4* material inputs for the Header House models.

³⁵⁴ Ibid.

³⁵⁵ Arno Barandun, *Seismic Behavior of Unreinforced Masonry Walls with Soft-Layer Strip Bearings* (Zürich: Institut für Baustatik und Konstruktion, 2013), 54.

³⁵⁶ Ibid.

Figure 47: Hysteresis envelope for the *Pinching4* material.³⁵⁷

Table 11: *Pinching4* material inputs.

Variable Name	Value	Variable Name	Value	Variable Name	Value
ePf1	$0.66V_{ult}$	eNd3	-0.0144 m	gKLim	0.75
ePd1	0.0013 m	eNf4	$-0.8V_{ult}$	gD1	0.1
ePf2	$0.9V_{ult}$	eNd4	-0.0359 m	gD2	0.2
ePd2	0.0027 m	rDispP	1.0	gD3	0.2
ePf3	V_{ult}	rForceP	1.0	gD4	0.05
ePd3	0.0144 m	uForceP	0.05	gDLim	0.3
ePf4	$0.8V_{ult}$	rDispN	1.0	gF1	0.0
ePd4	0.0359 m	rForceN	1.0	gF2	0.4
eNf1	$-0.66V_{ult}$	uForceN	0.05	gF3	0.4
eNd1	-0.0013 m	gK1	0.4	gF4	0.3
eNf2	$-0.9V_{ult}$	gK2	0.42	gFLim	0.6
eNd2	-0.0027 m	gK3	0.6	gE	10
eNf3	$-V_{ult}$	gK4	0.15	dmgType	energy

³⁵⁷ OpenSees, *Pinching4* Material.

The elastic beam column members do not need a material definition, but instead rely on the cross-sectional area of the member, the modulus of elasticity, and the moment of inertia. The modulus of elasticity is calculated as 1000 times the compressive strength of the masonry wall, as discussed in section 2.6. Both the cross-sectional area and the moment of inertia use the width, height, and thickness of the wall in the pier and spandrel areas.

After the material properties are defined, the geometric transformations need to be defined. During this step, the joint offsets are created. The offsets in the y direction for the pier elements and offsets in the x direction for the spandrel elements are determined based on the distance from the node of the rigid joint to where the element should begin. The last step to create the model is to add the elements. The elastic beam column elements and zero length members are defined by what nodes they connect and their material properties.

With the model in place, the next step is to define the recorders. Recorders are used to save information about your model during the analysis. For the Header House model, records for the structure's drift, displacement, base shear, and accelerations are defined.

The first step of the analysis process is to apply the gravity loads. In OpenSees, the mass at the nodes is assessed differently than the loads. At this step the dead load, 50% of the live load, and 25% of the snow load are added to the corresponding nodes. The remaining earthquake load will be added and assessed in a later step.

Two methods were used to conduct the seismic analysis: a non-linear pushover test and a nonlinear time history seismic analysis. The non-linear pushover test is a static modeling method for determining the strength and ductility of a structure. The main output of the test is a hysteresis showcasing the displacement caused by a gradually increasing horizontal load. The test is conducted by increasing the load of the structure until it collapses, essentially pushing it over.³⁵⁸

For the nonlinear time history seismic analysis, the earthquake data used is mostly made up of simulated data. This is due to the lack of strong shaking historical ground motion records for Eastern North America.³⁵⁹ The last strong earthquake to hit Ottawa was the magnitude 5.0 Val-des-Bois earthquake on June 23, 2010, with a 22.4 km source depth.³⁶⁰ The earthquake recording from Val-des-Bois that was used for the Header House model was taken from a station in Ottawa where the soil condition was rock.³⁶¹ This coincides with Header House's soil site class A conditions.³⁶² The accelerations and total duration of the Val-des-Bois (VDB) earthquake are illustrated in Figure 48.

³⁵⁸ Bocciarelli and Barbieri, 162.

³⁵⁹ Dominic Michaud and Pierre Léger, "Ground motions selection and scaling for nonlinear dynamic analysis of structures located in Eastern North America," *Canadian Journal of Civil Engineering* 41 (2014): 233.

³⁶⁰ Lan Lin and John Adams, *Strong Motions from Nearby Seismometer Records of the Val-des-Bois, Québec, Earthquake of June 23, 2010* (Ottawa: Geological Survey of Canada, 2010), 3.

³⁶¹ *Ibid.*, 8.

³⁶² *Interactive Vs30 Google Map for the City of Ottawa*, map (2011) <http://mypage.science.carleton.ca/~dariush/Microzonation/vs30-map/google_map.html> [8 August 2018].

The remaining 12 earthquake records used for the seismic analysis are simulated data for eastern Canada. Gail Atkinson created these time histories in 2009 to be compatible with the 2005 NBCC uniform hazard spectrum.³⁶³ The simulated data is broken up into four data sets. The first data set is for earthquakes with a magnitude of 6 at a fault distance from 10 to 15 km. The second data set has the same magnitude, but at a distance of 20 to 30 km. The third and fourth data sets are for earthquake with a magnitude of 7 with fault distance from 15 to 25 km and 50 to 100 km, respectively. The names of these data sets are 6a1, 6a2, 7a1, and 7a2, respectively, where the first number is the magnitude, the 'a' represents site class A conditions, and the second number is the data set number per magnitude. Each data set contains 45 simulated records.³⁶⁴ Three from each data set were selected to represent various distances from the earthquake source in the seismic analysis of Header House.

Table 12 summarises the properties of the earthquakes used in the analysis. Figures 49 to 60 show the acceleration time histories for each simulated record. The duration of the earthquake data in the figures was clipped to better highlight the acceleration pattern. For the rest of the duration, the acceleration is approximately zero.

³⁶³ Gail Atkinson, *Earthquake Time Histories Compatible with the 2005 NBCC Uniform Hazard Spectrum* (2009) <http://www.seismotoolbox.ca/TimeHistoriesNBCC2005/Atkinson2009_cjce.pdf> [8 August 2018], 1.

³⁶⁴ Ibid, 11.

Table 12: Summary of earthquakes used for the seismic analysis of Header House.

Source	Earthquake ID	Magnitude	Station Location	Distance from Source (km)	Data Time Step (s)
6a1	6a1_1	6	-	12.8	0.002
	6a1_2	6	-	10.7	0.002
	6a1_3	6	-	17	0.002
6a2	6a2_1	6	-	20.8	0.002
	6a2_2	6	-	16.9	0.002
	6a2_3	6	-	30.7	0.002
7a1	7a1_1	7	-	20.6	0.002
	7a1_2	7	-	17	0.002
	7a1_3	7	-	25.8	0.002
7a2	7a2_1	7	-	41.6	0.002
	7a2_2	7	-	70.2	0.002
	7a2_3	7	-	100.2	0.002
Val-des-Bois	VDB	5	Ottawa	56	0.01

Figure 48: 2010 Val-des-Bois earthquake acceleration time history recorded in Ottawa.

Figure 49: 6a1_1 earthquake acceleration time history from 20 to 24 seconds.

Figure 50: 6a1_2 earthquake acceleration time history from 20 to 24 seconds.

Figure 51: 6a1_3 earthquake acceleration time history from 20 to 24 seconds.

Figure 52: 6a2_1 earthquake acceleration time history from 20 to 27 seconds.

Figure 53: 6a2_2 earthquake acceleration time history from 20 to 27 seconds.

Figure 54: 6a2_3 earthquake acceleration time history from 20 to 27 seconds.

Figure 55: 7a1_1 earthquake acceleration time history from 18 to 37 seconds.

Figure 56: 7a1_2 earthquake acceleration time history from 18 to 37 seconds.

Figure 57: 7a1_3 earthquake acceleration time history from 18 to 37 seconds.

Figure 58: 7a2_1 earthquake acceleration time history from 18 to 40 seconds.

Figure 59: 7a2_2 earthquake acceleration time history from 18 to 40 seconds.

Figure 60: 7a2_3 earthquake acceleration time history from 18 to 40 seconds.

Before these ground motion time histories can be used in the seismic analysis, they need to be scaled to fit the target spectrum of the site. The target spectrum differs from the corresponding site class design spectrum as it is modified to fit the target of a specific location. The target spectral acceleration was determined using the 2015 National Building Code of Canada seismic hazard calculator. The calculator takes in the latitude and longitude coordinates of a site and outputs its spectral ($S_a(T)$) and peak ground acceleration (PGA) values.³⁶⁵ Figure 61 showcases the target spectral acceleration during a period of 10 s for Header House.

Figure 61: Target spectral acceleration for Header House's soil class A conditions.

Scaling method A from the NBCC 2015 Commentary J Appendix was used to scale the earthquake records. Method A involves scaling the earthquake records only for a select period range of the target $S_a(T)$ that significantly contributes to the building's

³⁶⁵ Natural Resources Canada, *Determine 2015 National Building Code of Canada seismic hazard values* (2016) <http://www.seismescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php> [8 August 2018].

dynamic response needs to be considered.³⁶⁶ The first step in determining this period was to conduct a modal analysis of the OpenSees models. The periods and responses at the rigid joint nodes calculated by the modal analysis were recorded and used to determine the period range. The lower bound value of the period range is determined as the period that achieves 90% mass participation, but not higher than 0.15 times the first mode period. The upper bound value of the period range is “greater than or equal to twice the first mode period, but not less than 1.5 s.”³⁶⁷ The calculated period ranges for each model case can be found in Table 13, and the final period range was determined using the lowest lower bound value and the highest upper bound value.

Table 13: Calculated period range for each wall scenario.

Walls	Low or High Range Property Values	Rigid or Flexible	T _{min} (s)	-	T _{max} (s)
Front & Back	Low	Flexible	0.0288	-	1.5
		Rigid	0.0004	-	1.5
	High	Flexible	0.0286	-	1.5
		Rigid	0.0002	-	1.5
Left & Right	Low	Flexible	0.0280	-	1.5
		Rigid	0.0012	-	1.5
	High	Flexible	0.0278	-	1.5
		Rigid	0.0280	-	1.5
Overall			0.0002	-	1.5

The scaling process is broken up into two steps. The first step involves individually scaling the records so that each record’s response spectrum equals or exceeds the

³⁶⁶ National Research Council Canada, J-110.

³⁶⁷ Ibid.

target spectrum on average over the specified period range. Once individually scaled, the records are then all scaled together so that the mean response spectrum of the suite does not fall 10% below the target spectrum.³⁶⁸ The calculated scaling factors and their standard deviation are compiled in Table 14. The NBCC does caution that ground motion records that require a scaling factor less than 0.5 or larger than 4.0 may not be compatible with the seismic hazard level.³⁶⁹ This occurs for the VDB earthquake. Therefore, the VDB data will may be considered separately if it varies drastically from the data produced from the simulated ground motion records.

Table 14: Calculated individual and suite scaling factors and standard deviations.

Earthquake ID	Scaling Factor - Individual		Scaling Factor - Suite	
	Mean	St. Dev.	Mean	St. Dev.
6a1_1	0.58	0.38	1.12	0.43
6a1_2	0.61	0.34	1.12	0.39
6a1_3	0.82	0.22	1.12	0.24
6a2_1	1.23	0.30	1.12	0.34
6a2_2	1.22	0.31	1.12	0.35
6a2_3	2.14	0.24	1.12	0.27
7a1_1	0.68	0.27	1.12	0.30
7a1_2	0.50	0.22	1.12	0.25
7a1_3	0.82	0.20	1.12	0.23
7a2_1	1.20	0.16	1.12	0.17
7a2_2	2.74	0.15	1.12	0.17
7a2_3	3.04	0.16	1.12	0.17
VDB	24.14	0.41	1.12	0.46

³⁶⁸ Ibid., J-115.

³⁶⁹ Ibid..

The unscaled, individually scaled, and final scaled spectral accelerations are compared with the target values in Figures 62 to 64. The final scaled earthquakes are then used to conduct the seismic analysis of the four model cases. Each earthquake is assessed individually and the model is reconstructed after each analysis to ensure no residual damage is retained.

Figure 62: Target Spectral Acceleration (Blue) versus Unscaled Earthquake Data (Grey)

Figure 63: Target Spectral Acceleration (Blue) vs. Individually Scaled Earthquake Data (Grey)

Figure 64: Target Spectral Acceleration (Blue) vs. Final Scaled Earthquake Data (Grey)

3.4. Maintenance Program

The final step of the workflow is to establish a maintenance program. Maintenance programs take into account the historic information gathered on a site and its current condition. The aim of the maintenance program is to repair the current deterioration of a building and to plan simple tasks that can help to reduce or slow down future deterioration. The maintenance program is split into two parts, corrective and preventive maintenance.

3.4.1. Determining Corrective Maintenance

The first part of a maintenance program is identifying and conducting corrective maintenance. This process begins with evaluating the current condition of the building and its components. For Header House, the focus of the evaluation was on the architectural and structural components rather than the mechanical and electrical elements. As a reference, Christopher Dziwinski from NCC provided a Building

Assessment Report of their Sussex properties that was produced in 2008 and contains an evaluation of their condition. The rating system for the existing conditions used in that report is based on a rating from 1 to 5. These five ratings represent ‘Urgent Attention Required’, ‘Poor - Considerable Repairs Required’, ‘Fair - Average Condition - Some Repairs Required’, ‘Good - Minor Repairs Required’, and ‘Excellent - No Repairs Needed’, respectively. This rating system does a good job in describing the amount of repair work needed, and so it was also used for the Header House to help keep consistency between NCC evaluations. In addition to the condition rating system, the evaluation in the Building Assessment Report also includes a rating system for the building element’s heritage value, in terms of high, medium, and low value. Since a significant portion of NCC’s properties are designated for their heritage value, this added rating system adds an extra layer to help prioritize the repair work. This is most beneficial when allocating funds for the corrective maintenance work.

In terms of categorizing the building components, the Building Assessment Report simply breaks them down into architectural, structural, mechanical systems, and electrical systems. A category missing from this list is services and utilities. Due to the scope of this project, the Header House’s corrective maintenance plan will focus solely on the architectural and structural categories. It should also be noted that due to the investigation restrictions on site, not all the structural components could be evaluated. The condition survey form for Header House can be found in Appendix E.

With the condition of the building determined, areas that are in subpar condition will need to be corrected. In the next chapter, corrective work for Header House will be suggested in order of highest to lowest priority.

3.4.2. Creating a Preventive Maintenance Plan

The second step in creating a maintenance program is establishing a preventive maintenance plan. This plan lays out simple scheduled tasks that aim to increase the longevity of the building by decreasing or slowing the decay of its components. To make the plan easier to understand, the tasks will be laid out in a table in order of frequency. The frequency of these tasks will range from minuscule daily tasks to more work intensive yearly tasks. Header House's preventive maintenance plan will take into account the materials of the building, the use of the building both in season and during its off season, and the Ottawa climate.

Though the preventive maintenance tasks for Header House will be established in the next chapter, it is not set in stone. Included with the list of tasks will be a plan for quinquennial surveys. As the name suggests, these surveys will take place every five years. The purpose of these surveys is to monitor the building. This includes any physical changes, new deterioration issues, and changes of use. These surveys will follow a similar workflow to the initial survey of the building, but with less pre-site visit desk work to be completed, as the first site visit already processed most of the historical information. This monitoring helps to establish if more corrective maintenance work needs to be conducted and to update the current preventive

maintenance plan. This updating includes adding additional tasks, if deemed necessary, and to verify if the previous tasks produce the expected results or if they, in turn, actually do more harm than good.

3.5. Summary of Header House Case Study

The case study of Header House followed the suggested framework for ensuring the ongoing maintenance of existing buildings. The process began with researching the building's history and a walk-through inspection. Surveying was then conducted to obtain geometric data, take photography, and conduct a condition assessment. That information was then processed to create as-built drawings, condition drawings, and a Building Information Model. To assess Header House's performance, a structural analysis was conducted. The maintenance plan and monitoring strategies were then determined based on all the information gathered on the building. However, due to surveying limitations and restrictions, not all the steps within the workflow were fully completed. Figure 65 shows the workflow with the partially completed tasks highlighted.

Figure 65: Workflow used for Header House case study with highlighted tasks that were only partially completed.

The first task that could not be fully completed was the condition assessment of Header House. Though all visible areas were assessed, significant portions of the building, including most of the structural elements were not able to be surveyed and their condition is unknown. Therefore, there is a high degree of uncertainty for overall condition of Header House and further investigation is required. In addition, service and utility equipment were not assessed. This meant that the complete BIM of Header House could not be constructed. The current BIM does not include the services and utilities that are present in the building and only focuses of the architectural elements. The final task that was fully completed for the Header House case study was the structural analysis. Since most of the structural members' condition is unknown, a gravity load analysis could not be conducted. Once again, further investigation is required to assess Header House's structural performance.

The following section presents the results of the Header House case study. The results include the condition assessment, the seismic analysis results, the maintenance plan, and the selected monitoring strategy.

4 – RESULTS AND DISCUSSION

This section showcases the results of Header House’s condition assessment, the results of the seismic analysis, and the final maintenance plan. The aspects of the condition assessment include the rated condition of the building and the various decay and deterioration present. The results of the seismic analysis outline the maximum reactions of Header House for each earthquake and the expected damage. The final maintenance plan outlines the corrective maintenance tasks, the preventive maintenance plan, and the selected monitoring program.

4.1. Condition Assessment

The overall condition assessment of Header House was conducted during the surveying step. However, due to surveying limitations, a high degree of uncertainty is present for the overall condition of Header House. Most notably, most structural elements, such as the timber trusses and floor joists, and their connections could not be surveyed, yet their condition is most important for the overall safety and use of the building. The six decay conditions identified were: cracking, efflorescence, missing part, corrosion, spalling, and staining. Figures 66 and 67 show a sample of Header House’s condition drawings. The full collection can be found in Appendix D. The cracks in Header House are predominantly found on the lower half of the exterior limestone façade. These cracks could be caused by either settlement or by freeze-thaw cycle as they are close to the ground and more susceptible to moisture. The largest cracks are found on a cornerstone on the entrance stairway on the south-east façade.

Figure 66: Condition drawing for south-east elevation.

Figure 67: Condition drawing of south-east cut-section.

Efflorescence is found in large concentrations on the clay brick interior walls in the basement of Header House. Figure 68 showcases a sample of clay brick with efflorescence in the basement. This large concentration is probably caused by two factors. The first factor is moisture. As these are basement walls, the moisture and salts from the surrounding soil can be absorbed by the masonry if no protective layer is present. The moisture and soluble salts move through the wall until they evaporate on the other side, leaving the salts behind.

Figure 68: Efflorescence on clay brick in the basement of Header House.

The other factor that could contribute to the large scale efflorescence is the excess heating in the basement. The main building of Header House is comprised of three levels: the attic, ground floor, and basement. Out of all these levels, the basement was by far the hottest. This is most likely due to improperly placed HVAC ducts that do

not circulate the heated air throughout the building. This excess heat aids in the evaporation of water, which can cause the soluble salts to crystallize.

The third decay condition is represented by a missing part or component. The missing parts take the form as the missing triangle in the Canada Centennial logo and the various holes found at Header House. In the railing on the entrance stairwell, there is a Canada Centennial logo detail. This detail is currently missing one of its triangles that contribute to its geometric maple leaf motif. The holes are mostly found in or leading to the basement. Prominent examples include the two holes in the chimney wall in the basement and the hole where a floor joist once rested, but has since been cut off to allow access to the basement by stairs.

Another condition found solely in the basement is corrosion. This affects the exposed regions of the steel columns and the metal door opening in the chimney wall. This corrosion is most likely amplified by the excess moisture allowed into the basement by the hole in the window opening. This hole was created to allow the electrical wires and plumbing to access the equipment for the patio bar. This hole was left wide open with no protective layer or coating to separate the outdoor and indoor spaces. Figure 69 shows a photograph of the window with its opening.

Figure 69: Hole in plywood covered basement window.

The condition that is most predominate in Header House is spalling. Spalling is present both on the exterior, as limestone and mortar spalling, and on the interior, as brick and paint spalling. The limestone spalling is isolated to one case on the south-west façade, but mortar spalling is found on three out of four elevations and mostly on the lower half of the wall. This mortar spalling on the lower half of the wall is most likely due to the excess moisture caused by snow build up on the ground in winter. The paint spalling, mostly found on the ground floor, is most likely caused by past renovations. The interior ground floor walls were originally painted, but were later covered with gypsum during the time that Header House hosted an art gallery. The new tenants have since removed the gypsum, which most likely damaged the paint underneath. Figure 70 shows a close up of the paint spalling.

Brick spalling is concentrated in the basement walls of the Header House alongside the efflorescence. This spalling could be caused by subflorescence that produced

enough internal stress to cause disintegration of the surface layer. Brick spalling near the basement stairwell can be seen in Figure 71.

Figure 70: Close up of paint spalling on ground floor.

Figure 71: Brick spalling in the basement stairwell.

The last condition is iron staining, which is only found on a portion of limestone under the railing of the front entrance stairwell. The survey and condition assessment of Header House show no signs of subsidence. This may indicate that the building rests directly on bedrock. As the site is classified as soil class A for hard rock, it is not an unlikely scenario.

A glossary describing the conditions affecting Header House, along with photographs of each condition, can be found in Appendix D. During the condition assessment process, a condition survey form, which can be found in Appendix E, was filled out to summarize the condition of the building's components. Condition drawings were also produced to showcase the location of the deteriorations on the as-built drawings. They can be found in Appendix C.

4.2. Seismic Analysis Results

The first method of seismic analysis was conducted through a pushover test. The results for the rigid diaphragm can be seen in Figure 72, where the cases were split by wall orientation and by lower and upper bound material strengths. The results show that the base shear is much higher for the front-back orientation than the left-right. This is due to the larger size and mass of the front and back walls. The results also show a higher base shear for the upper bound model case. This is most likely due to the increased stiffness in the wall.

Figure 72: Base shear vs. displacement of the pushover analysis of the rigid model case.

The second seismic analysis method was conducted through a non-linear time-history analysis. The drift, displacement, residual drift, base shear, and absolute acceleration reaction by each of the thirteen earthquakes were recorded for each model case of each wall. The four model cases were: lower bound strength with a rigid diaphragm, lower bound strength with a flexible diaphragm, upper bound strength with a rigid diaphragm, and upper bound strength with a flexible diaphragm.

A compilation of sample data was graphed to show some of the behaviours in the walls. Figures 73 and 74 show the hysteresis behaviour of the left pier in the front stone wall with lower bound strength and a rigid diaphragm under 7a1_2 and VDB earthquakes, respectively. Figures 75 and 76 show the hysteresis behaviour of the top spandrel in the same wall under the same earthquake loads. Figures 77 and 78 show

the overall wall behaviour for the front stone wall with lower bound strength and a rigid diaphragm, still under the same loads. These base shear versus displacement graphs show that the piers govern for the overall wall for the Header House. Though the spandrels exhibit some movement, the rocking of the piers completely dominates for the model. These graphs also showcase the difference between the simulated data and the Val-des-Bois record. This is also evident in the drift over time graphs of the two wall cases, shown in Figures 79 and 80. Though they follow a similar shape, the VDB earthquake causes a much larger displacement due to its ground motion forces. This displacement is not recuperated, resulting in a much larger displacement and residual drift.

The relationship between the brick and stone layers were also compared for the flexible diaphragm case. Figures 81 and 82 demonstrate the difference between the limestone and brick layers of the front wall with a flexible diaphragm. Figure 81 shows the wall's hysteresis behaviour under the 7a1_2 earthquake, while Figure 82 shows the drifts over time. The brick layer, which has a lower stiffness due to its smaller thickness and material properties, also has a much smaller residual drift than the limestone layer. Its smaller thickness also means that it has a smaller base shear since the brick walls support less load than its limestone counterpart.

Figure 73: Base shear vs. displacement graph for the left pier of the front stone wall by 7a1_2 with lower bound values and a rigid diaphragm.

Figure 74: Base shear vs. displacement graph for the left pier of the front stone wall by VDB with lower bound values and a rigid diaphragm.

Figure 75: Base shear vs. displacement graph for the top spandrel of the front stone wall by 7a1_2 with lower bound values and a rigid diaphragm.

Figure 76: Base shear vs. displacement graph for the top spandrel of the front stone wall by VDB with lower bound values and a rigid diaphragm.

Figure 77: Base shear vs. displacement graph for the overall front stone wall by 7a1_2 with lower bound values and a rigid diaphragm.

Figure 78: Base shear vs. displacement graph for the overall front stone wall by VDB with lower bound values and a rigid diaphragm.

Figure 79: Drift time history for 7a1_2 earthquake on the front stone wall with lower bound values and a rigid diaphragm.

Figure 80: Drift time history for VDB earthquake on the front stone wall with lower bound values and a rigid diaphragm.

Figure 81: Base shear vs. displacement by 7a1_2 of brick and stone front walls with lower bound values and a flexible diaphragm.

Figure 82: Drift vs. time by 7a1_2 of brick and stone front walls with lower bound values and a flexible diaphragm.

It is also important to note that the VDB earthquake did produce significantly different results for some of the walls and model cases. This difference correlates with the large scaling factor the data required to meet the target spectrum, as the NBCC hinted at the possibility that the earthquake was not compatible with the seismic hazard level.

To better compare the results, the maximum drift, displacement, acceleration, and residual drift for each wall and earthquake were averaged by model case. These results can be viewed in Table 15, which exhibits the mean value and the mean value plus the standard deviation in the square brackets.

Table 15: Mean values for the maximum drift, displacement, absolute acceleration, and residual drift for each model case.

Diaphragm Connectivity	Bound Value	Mean Value [Mean + Standard Deviation]			
		Max Drift	Max Displacement (mm)	Max Acceleration (g)	Max Residual Drift
Rigid	Lower	0.17% [0.26%]	5.73 [8.92]	0.98 [1.15]	0.11% [0.21%]
	Upper	0.17% [0.26%]	5.73 [8.91]	0.98 [1.15]	0.11% [0.22%]
Flexible	Lower	0.18% [0.28%]	6.16 [9.33]	1.47 [1.66]	0.14% [0.24%]
	Upper	0.18% [0.27%]	6.10 [9.28]	1.14 [1.30]	0.13% [0.24%]

A similar process was followed for the base shear, though the data was split by wall orientation. These results can be found in Table 16.

Table 16: Mean values for the maximum base shear for each model case by wall orientation.

Diaphragm Connectivity	Bound Value	Mean Value [Mean + Standard Deviation]	
		Max Base Shear Front-Back (kN)	Max Base Shear Left-Right (kN)
Rigid	Lower	356 [398]	194 [222]
	Upper	384 [423]	216 [243]
Flexible	Lower	349 [391]	163 [193]
	Upper	352 [394]	165 [196]

Tables of all the maximum values for each wall case and each earthquake load can be found in Appendix F.

The maximum drift and displacement of the walls did not differ significantly between the four model cases. The maximum displacement difference occurs between the upper bound rigid diaphragm case and the lower bound flexible diaphragm case. The latter is only 0.43 [0.42] mm larger than the former. This means that the building's response is not sensitive to these parameters. The drift values can then be used to approximate the damage caused by the earthquake. Table 4, in section 2.6, stated the drift limits for each damage case. The lowest limit accounts for a drift of 0.3% or higher that correlates with slight damage and includes: diagonal, stair-step hairline cracks on masonry wall surfaces; larger cracks around door and window openings; movements of lintels; and cracks at the base of parapets.³⁷⁰ Since the absolute maximum drift of Header House is only 0.28%, the walls should incur little to no

³⁷⁰ Park et al., 132.

damage during an earthquake. The maximum residual drift also plays a key factor in the functionality of the space following the seismic event. One of the residual drift limits, discussed in section 2.6, was 1.0% for determining if the structure is safe. Another limit along the same lines was for the amplification of the drift if a second earthquake were to occur. This limit is 0.5% and above, but the absolute maximum residual drift of 0.24% is well under these two limits. The last limit corresponds to occupant comfort. As stated in section 2.6, a residual drift of 0.8% or greater leads to headaches, dizziness, and an overall hindrance to the occupants.³⁷¹ Occupants also start to feel the displacement if the residual drift is over 0.5%. Once again, the 0.24% residual drift is well under the limits and should not affect the occupants. Therefore, no interventions are required.

The acceleration increases significantly from a rigid diaphragm case to a flexible one. This showcases the lower stiffness in the latter case. The lower stiffness allows for easier oscillation of the walls. The lower bound values further decrease the walls' stiffness, increasing the acceleration for the flexible diaphragm case as well. The maximum acceleration of 1.47 g is quite high. However, this acceleration is for the top of the wall, which only affects the attic floor. Since the attic floor is only used for storage, occupants typically are not affected.

The base shears of the non-linear time-history analysis match up with those determined by the pushover test. The front-back orientation walls have a much higher

³⁷¹ McCormick et al., 5.

base shear than those for the left-right orientation. The larger wall size and overall loads create a larger stiffness in the wall, thus increasing the base shear force. This increase in stiffness with increase base shear relationship is also evident for the rigid and flexible diaphragm cases and the upper and lower bound cases. The largest base shear is found in the front-back orientation, rigid diaphragm, and upper bound wall, should also have the highest stiffness.

The base shear values would affect the foundation of the building. However, they were not modeled, so its reaction is currently unknown. Since the diaphragms themselves were also not modelled, their reaction to the wall movements and their performance are also unknown. Further research should be done to assess their affects on the building's overall seismic performance.

It is also important to note that these results correlate with rocking failure. It would be interesting to see how walls with other governing failure modes would react under the same seismic loads.

4.3. Finalised Maintenance Plan

This section covers the selected maintenance plan for Header House. The maintenance plan is broken up into corrective tasks, a preventive maintenance plan, and the monitoring program to keep track of the ongoing changes to the building and its condition.

4.3.1. Corrective Maintenance Tasks

The most deteriorated components in Header House are the basement clay brick walls. These walls have holes, are spalling, and have a considerable amount of efflorescence. The first step to repairing these walls is to remove the efflorescence. The first step is to remove any flaky efflorescence through dry brushing. Once removed, desalination can commence. Desalination is the process of “saturation and poulticing with an absorbent clay to try to reduce the level of potentially damaging soluble salts concentrated within the surface of decaying stone.”³⁷² The process begins by saturating the wall with mists of clean water for several days. During this stage, temporary gutters should be constructed to collect the water run-off. Once the wall has either wet for either the full depth or a specified depth, an absorbent clay or diatomaceous earth is added to clean water to produce a soft and sticky paste, known as clay poultice.³⁷³ When the poultice has been mixed enough to be free of lumps, it is plastered onto the soaked wall in a layer of 20 to 25 mm. To keep the poultice in place, a light-gauge galvanized wire mesh is placed on top and tacked into the joints carefully with galvanized staples.³⁷⁴ During the poultice treatment, the plastered area should be protected from any heat source to reduce the risk of rapid drying. The poultice will draw out the salts from the masonry wall as it dries. The drying conditions of the area will affect the contact time, which can range from a few days to

³⁷² John Ashurst and Nicola Ashurst, *Practical Building Conservation English Heritage Technical Handbook*, vol. 1 (Avon: Gower Technical Press Ltd, 1988), 69.

³⁷³ Ibid.

³⁷⁴ Ibid., 70.

weeks. When the poultice has fully dried, it will have lightened in colour, cracked, shrunken, and detached itself from the wall. The staples can then be removed along with the wire mesh and dried clay poultice. Any remain poultice on the wall may be removed through dry brushing. This desalination process is then repeated until the salts have been reduced to a predefined acceptable level.³⁷⁵ Once the efflorescence has been cleaned off, the bricks should be reassessed to determine their condition. If the bricks are in very poor condition, they should be replaced along with the bricks affected by spalling. During this step, any holes in the brick wall should also be filled with the new brick. The new bricks should be the same size and shape as the original bricks. After the replacement, any damaged mortar should be repointed with a lime based product.

Another largely affected component is the exterior limestone masonry façade. This façade has limestone spalling, mortar spalling, cracking, and staining deterioration. The limestone spalling is isolated in a small area on the south-west wall. The affected area should be treated with a Dutchman repair. A Dutchman repair is the partial replacement of the damaged material with a new unit.³⁷⁶ Since the exterior masonry façade is a character-defining element of the building, the new unit must match the pre-existing limestone. Serge Morin, from NCC, advises that new limestone units for Header House should be Champlain limestone that is quarried from the Ottawa-

³⁷⁵ Ibid., 71.

³⁷⁶ Anne Grimmer, *A Glossary of Historic Masonry Deterioration Problems and Preservation Treatments* (Washington: Department of the Interior National Park Service Preservation Assistance Division, 1984), 56.

Gatineau area that matches the colour of the current units. The areas affected by mortar spalling should be repointed with a lime-based mortar that is similar in colour with the rest of the current mortar. The cracked limestone units should be treated based on crack width. Very narrow cracks can be either filled with an epoxy or repointed.³⁷⁷ If the cracks are very large, as present in the cornerstone of the entrance stairwell, the stone should be replaced. Once again, the new stone should be a Champlain limestone quarried from the local area. The final corrective maintenance work required for the exterior limestone façade is the cleaning of the iron staining on the entrance stairwell. This cleaning process begins by crating a mixture of 7 parts glycerine, 1 part sodium citrate, and 6 parts warm water. Attapulgitic clay is then added into the solution until it forms a smooth paste. The paste is then applied to the stain and left to dry. Once dry, it is removed with a wooden or other non-metallic spatula. This process is repeated until the stain has lifted completely or to a satisfactory lightened state.³⁷⁸

The remaining condition affecting the structural masonry walls of the Header House is paint spalling on the interior brick on the ground floor. The walls should be repainted to provide a protective layer for the clay brick. Another condition found in Header House is the corrosion of steel elements. The elements affected are the exposed sections of the steel column in the basement and the metal door opening in the basement chimney wall. The first step is to scrape away the corroded areas on the

³⁷⁷ Alexander Newman, *Structural Renovation of Buildings: Methods, Details, and Design Examples* (New York: MacGraw-Hill, 2001), 540.

³⁷⁸ Ashurst and Ashurst, 62.

members. The element's performance should then be assessed based on the remaining metal thickness, especially for the steel column case. If they are deemed unacceptable, they should be reinforced or replaced. A protective coating should then be applied to the steel to protect it from further corrosion.³⁷⁹ The last condition present in Header House is the missing part on the Canada Centennial logo on the railing of the entrance stairway. The missing triangle piece should be replaced with the same material as the rest of the logo.

4.3.2. Preventive Maintenance

Some of the preventive maintenance tasks should work to mitigate the source of the deterioration in the basement. One task is to seal the hole in the basement window to reduce the moisture and contaminants from the outdoor air to enter the interior space. Another task is to reduce the amount of heating in the basement. This can be achieved in two ways. One is to redirect some of the heated air to the rest of the building. The ductwork should be assessed to determine the best course of action. The second way is to simply reduce the temperature that the HVAC system heats to. A lower temperature would reduce the rate of evaporation on the surface of the clay brick.

For the exterior, a system of rain gutters and downspouts should be added to reduce the water runoff from the roof to the limestone walls. Since the exterior façade is a factor of Header House's architectural value, the added gutter system should blend

³⁷⁹ Newman, 540.

with the roof line and the downspouts should be placed in less visible areas of the structure as to not obstruct the view of the character-defining elements. Once in place, the gutters should be cleaned every year to ensure that they continue to function effectively.

4.3.3. Selected Monitoring Program

Quinquennial surveys were selected as the best monitoring method not only for Header House, but for all buildings. These surveys are repeated every five years to document the changes in the building. Having a record of the past and current conditions of a building help to better understand what the source of the deterioration is. The workflow for the quinquennial surveys is the same as the proposed workflow in Figure 25 in the methodology chapter. However, the amount of work required for the subsequent surveys will be less than that for the original survey. Instead of conducting thorough research on a building's history, only past reports and the last five years will need to be reviewed. Since the as-built drawings and Building Information Models were already produced before, they will just need to be updated with each new survey. If a building has not changed physically or in use since the last survey, a seismic analysis would not need to be conducted. These quinquennial surveys are focused on finding any new deterioration that has become visible in the past five years and repairing the damage. The accumulation of decay within these five year intervals should be minimal and easy to repair ensuring that the building is kept to a proper serviceable condition.

One condition that is important to document in subsequent surveys of Header House is efflorescence. If efflorescence continues to persist after the corrective and preventive maintenance work at high volumes, further in-depth investigations are required to determine the root cause.

5 – CONCLUSIONS AND RECOMMENDATIONS

The proposed framework for ensuring the ongoing maintenance of existing buildings is broken up into five main sections: preliminary work, surveying and recording, data processing and analysis, identifying the maintenance plan, and selecting a monitoring strategy. The preliminary work section aims to determine the building history of deterioration and its significance. Geometric information is gathered and the building's current condition is assessed during the surveying and recording step. For the Header House case study, the geometrical information was gathered by a FARO Focus 3D X 330 laser scanner. The condition of Header House is quite poor in the basement due to large amounts of efflorescence and brick spalling. These conditions were most likely amplified due to the excess heating in the basement and the hole that allows moisture and contaminants from outdoors to enter the basement. The required data processing outputs are as-built drawings and condition drawings. A Building Information Model can also be created to have a 3D record of the building and its components. As older buildings were typically not designed to resist earthquake loads, a seismic analysis should be conducted.

A seismic analysis of Header House was conducted for the four exterior unreinforced masonry structural walls. These walls were modeled as equivalent frames in OpenSees. Surveying limitations meant that the material properties of the walls were unknown. To combat this, four models were selected to create a bounded analysis of the building. The four cases were: lower bound strength with a flexible diaphragm, lower bound strength with a rigid diaphragm, upper bound strength with a flexible

diaphragm, and upper bound strength with a rigid diaphragm. The upper and lower bound values correspond to the material property ranges selected. The flexible diaphragm cases model the brick and stone layers and the opposing walls independently. The rigid diaphragm cases model the brick and stone layers as one unit, and the opposing walls also act together as one. Thirteen earthquakes were selected for the seismic analysis. Twelve were simulated records for magnitude 6 and 7 earthquakes. The last record was from the magnitude 5 June 23, 2010 Val-des-Bois earthquake recorded from a station in Ottawa. The results of the analysis showed that an increase in stiffness due to the rigid diaphragm and upper bound values resulted in larger base shear forces. On the opposite end, the lower stiffness in the lower bound with flexible diaphragm cases resulted in higher accelerations. The drift of the walls was relatively unaffected by the model cases. The walls should incur little to no damage since the highest drift of 0.28% falls under the slight damage limit of 0.3%. The maximum residual drift of 0.14% falls well under the limits for occupant discomfort and for amplified displacements in the case of secondary earthquake.

The final steps of the framework are to create a maintenance plan and to determine the monitoring strategies. The maintenance plan includes both corrective maintenance work to repair the existing damage and preventive maintenance work to mitigate further deterioration. Monitoring methods should be determined on a case by case basis. However, quinquennial surveys should be conducted to monitor the change in the building every five years. The process for these surveys match that of the overall framework.

Recommendations for future work include further investigation of Header House, more research on bounded seismic analyses, and creating a user-friendly app to assist in the framework tasks. As there were significant limitations when surveying Header House, the overall condition of the building could not be assessed. Further investigation is required to fully understand the current state of Header House and to be able to conduct a gravity load analysis of the structure.

Further research should be done on the effects of a bounded seismic analysis on other structures. This includes not only larger buildings, but also buildings where the governing failure mode is not rocking. In addition, in the Header House seismic analysis, the floors and the roof were only considered as mass. Further research should be done to assess their effects on the building during a seismic event. Also, only in-plane failure was considered for Header House's analysis. Future research should look into the effects of out-of-plane failure.

Finally, future work on ensuring the ongoing maintenance of existing buildings should include the creation of a user-friendly app to assist with the suggested framework. This app would make it easier to follow the framework for not only large corporations and government agencies, but also for average homeowners.

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APPENDIX A – STATEMENT OF SIGNIFICANCE

HEADER HOUSE – STATEMENT OF SIGNIFICANCE

Description of Historic Place

Header House is a modest Baronial style building, with an irregular, picturesque roof and rough cut, irregularly coursed limestone walls, that was constructed from 1901 to 1902. It is located in the north-west tip of Major's Hill Park, in view from Parliament Hill and the Ottawa River. The designation of the site is confined to the footprint of the building.³⁸⁰

Heritage Value

Header House is a Recognized Federal Heritage Building due to its historical associations, and its architectural and environmental value.

Historical Value

Header House's historical value is closely associated with that of Major's Hill Park as a whole. The park was first the home of famous English military engineer John By, also known as Colonel By. The land underwent a transformation during the city's beautification phase of the development in the early 20th century, under the Ottawa Improvement Commission, currently known as the National Capital Commission, and

³⁸⁰ Canada's Historic Places, *Maintenance Building*, para. 1.

became Ottawa's first municipal park.³⁸¹ Header House was originally constructed as a potting house to accompany the turn-of-the-century renovations to an existing greenhouse, and the construction of a new conservatory, a two-storey summerhouse, and new fountains and walkways.³⁸² These were built in accordance with plans developed by Public Works to enhance the attractiveness of the park, under the supervision of Canada's Chief Dominion Architect, David Ewart, whose portfolio includes many of the capital's most notable and iconic buildings, such as the Royal Canadian Mint and Connaught Building.³⁸³ With the dismantling of the green-house complex in 1937, the building served as a maintenance shed.³⁸⁴ The building is currently used as a commercial property, with Tavern on the Hill, a seasonal outdoor canteen, patio, and ice cream shop, as its tenant.

Architectural Value

Header House's character resides primarily in the picturesque domestic forms, outlined by its Baronial style, and in the detailing which delineates these forms. The decision to use stone for the exterior walls and brick for interior partitions was made to ensure the cool temperatures and shaded conditions required for successful potting. The stone walls, built up in irregularly coursed, rough-cut limestone with corner

³⁸¹ Meek 117-18.

³⁸² Canada's Historic Places, *Maintenance Building*, para. 3.

³⁸³ Canada's Historic Places, *Former Potting House (now Maintenance Building)*, 1.

³⁸⁴ Canada's Historic Places, *Maintenance Building*, para. 3.

quoins, and the basement and upper walls, which are separated by a plain dressed canted stone band, are indicative of the building's very good quality craftsmanship.³⁸⁵

Environmental Value

Header House is compatible with the picturesque character of its municipal park setting and is familiar to residents and those who visit the park.³⁸⁶

Character-Defining Elements

Its very good aesthetic design, functional design and very good materials and craftsmanship, for example:

- the modest, domestically scaled massing, the hip and cross gable roof, offset chimney and the attached one-storey garage;
- the stone walls, built up in irregularly coursed rough-cut limestone, and the basement and upper walls separated by a plain dressed canted stone band;
- the stone quoins at the corners, window jambs, lintels and sills, the rough-cut voussoirs and the stone front porch with iron railing;
- the two-over-two wood sash, the semi-elliptical window, and the front door;
- Stuart Ash's 1967 centennial logo motif moulded into the front porch railing;
- the brick interior partitions.

³⁸⁵ Canada's Historic Places, *Maintenance Building*, para. 4.

³⁸⁶ *Ibid.*, para. 5.

The manner in which the Header House reinforces the picturesque character of its municipal park setting and is familiar within the immediate area, as evidenced by:

- its picturesque domestic forms and elements of its design, which harmonize with its landscaped surroundings of Ottawa's, Major's Hill Park;
- its visibility, given its location in a park highly frequented by visitors and residents of the nation's capital.

APPENDIX B – AS-BUILT DRAWINGS

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: SITE PLAN		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:500	DRAWING NO.: A00 of A14	

NOTES:

- THIS RECORD WAS PRODUCED USING THE INTERNATIONAL SYSTEM OF MEASUREMENTS.
- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:

- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE	
DRAWING: ROOF PLAN	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018
SCALE: 1:100	DRAWING NO.: A01 of A14

NOTES:

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LEGEND:	
---	- ESTIMATED OUTLINE
	- NOT SCANNED AREA
	- CUT-SECTION
	- GROUND
	- BRICK

PROJECT:		HERITAGE RECORDING - HEADER HOUSE	
DRAWING:		ATTIC FLOOR PLAN	
DRAWN BY:	RALUCA DOBROTESCU	DATE:	JANUARY 12, 2018
SCALE:	1:100	DRAWING NO.:	A02 of A14

NOTES:
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LEGEND:
--- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE	
DRAWING: GROUND FLOOR PLAN	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018
SCALE: 1:100	DRAWING NO.: A03 of A14

NOTES:

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LEGEND:	
---	ESTIMATED OUTLINE
	NOT SCANNED AREA
	CUT-SECTION
	GROUND
	BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE	
DRAWING: BASEMENT FLOOR PLAN	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018
SCALE: 1:100	DRAWING NO.: A04 of A14

NOTES:

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---	ESTIMATED OUTLINE
	NOT SCANNED AREA
	CUT-SECTION
	GROUND
	BRICK

CUT-SECTION NO.

A

B

DRAWING PAGE NO.

PROJECT: HERITAGE RECORDING - HEADER HOUSE

DRAWING: SOUTH-EAST ELEVATION

DRAWN BY: RALUCA DOBROTESCU

DATE: JANUARY 12, 2018

SCALE: 1:100

DRAWING NO.: A05 of A14

NOTES:

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LEGEND:

- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: SOUTH-WEST ELEVATION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A06 of A14	

NOTES:

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LEGEND:

- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: NORTH-WEST ELEVATION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A07 of A14	

- NOTES:
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LEGEND:	
---	- ESTIMATED OUTLINE
	- NOT SCANNED AREA
	- CUT-SECTION
	- GROUND
	- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE	
DRAWING: NORTH-EAST ELEVATION	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018
SCALE: 1:100	DRAWING NO.: A08 of A14

NOTES:
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LEGEND:
--- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE	
DRAWING: NORTH-WEST CUT-SECTION	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018
SCALE: 1:100	DRAWING NO.: A09 of A14

NOTES:

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LEGEND:

- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: SOUTH-EAST CUT-SECTION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A10 of A14	

NOTES:
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LEGEND:
--- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: NORTH-EAST CUT-SECTION (GARAGE)		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A11 of A14	

NOTES:

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- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:

- - ESTIMATED OUTLINE
- - NOT SCANNED AREA
- - CUT-SECTION
- - GROUND
- - BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: NORTH-EAST CUT-SECTION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A12 of A14	

- NOTES:
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 - THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
 - THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

- LEGEND:
- - ESTIMATED OUTLINE
 - NOT SCANNED AREA
 - CUT-SECTION
 - GROUND
 - BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: SOUTH-WEST CUT-SECTION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A13 of A14	

NOTES:
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LEGEND:
--- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

PROJECT: HERITAGE RECORDING - HEADER HOUSE		
DRAWING: SOUTH-WEST CUT-SECTION (GARAGE)		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 12, 2018	
SCALE: 1:100	DRAWING NO.: A14 of A14	

NOTES:
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- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:
--- - ESTIMATED OUTLINE
- NOT SCANNED AREA
- CUT-SECTION
- GROUND
- BRICK

APPENDIX C – GLOSSARY OF DECAY

Cracks

Cracks are formed when sufficient force is applied to cause a material to split apart. These forces can be caused by a variety of factors, such as settlement, thermal changes, and the freezing of water in the pores of the material.³⁸⁷ In Header House, cracks can be found in the exterior limestone, interior clay brick, and the concrete floors in the basement. Figure 83 showcases a crack in the cornerstone located on the staircase leading to the front entrance on the south-east façade.

Figure 83: Cracked cornerstone on the entrance staircase.

³⁸⁷ Douglas and Noy, 82.

Efflorescence

Efflorescence is the external crystallisation of soluble salts. The salts are either already present within the material or are introduced by the moisture in its surroundings. When the water containing the soluble salts starts to evaporate, the salts are left behind, forming the growth of crystals on the surface.³⁸⁸ Efflorescence in Header House is mostly found on the clay brick in the basement, but it is also present near bottom of the walls on the ground floor. Figure 84 is a close up photograph of the efflorescence on the clay brick.

Figure 84: Efflorescence on clay brick in the basement of Header House.

³⁸⁸ Harris, 391.

Missing Part

Missing part are characterised as an “empty space, obviously located in the place of some formerly existing” element or component.³⁸⁹ In Header House, this takes two forms. One missing part is located on the railing on the south-east façade. Within the railing is the Canada Centennial logo. However, the current detail is missing a triangle to properly form the maple leaf inspired shape. Figure 85 showcases the Canada Centennial logo detail with the missing part.

Figure 85: The Canada Centennial logo with missing part (photograph by Shiraz Uri).

Missing part in Header House is also characterized by its holes. These holes are either formed by deterioration, intentionally, or a mix of both. Most of the holes in Header

³⁸⁹ Monuments and Sites XV, 36.

House can be found in the basement. One example of this is found in the stairwell leading towards the basement. A hole is present in the clay brick wall where a floor joist once rested, as seen in Figure 86. The floor joist was cut to allow access to the basement, but the hole was never filled. Another set of holes are located on the chimney wall located in the basement. These holes can be seen in Figure 87. One example of an intentional hole is located in the plywood covering one of the basement windows. This hole was made to allow electrical wires to be run from the inside of the building to the patio space outside. A photograph of this hole is presented in Figure 88.

Figure 86: Hole in clay brick in stairwell towards the basement.

Figure 87: Holes in chimney wall in the basement of Header House.

Figure 88: Hole in plywood covered basement window.

Corrosion

Corrosion is the rusting of a metal or alloy. Once corroded, the material can flake off or erode, reducing the structural and aesthetic performance of the material.³⁹⁰ In Header House, corrosion can be found on metal elements in the basement. This includes the steel column and the metal opening in the chimney wall, as seen in Figures 89 and 90, respectively.

Figure 89: Corroded exposed column in Header House basement.

³⁹⁰ Watt, *Building Pathology*, 120-121.

Figure 90: Corroded metal opening in chimney wall.

Spalling

Spalling is the detachment of a material as a scale or a stack of scales parallel to the surface.³⁹¹ In Header House, there are four types of spalling present: brick spalling, limestone spalling, mortar spalling, and paint spalling.

Figure 91 showcases brick spalling in the basement stairwell, Figure 92 shows limestone spalling on the south-west façade, and mortar spalling can be seen in Figure 93. Figures 94 and 95 showcase paint spalling on the ground floor interior walls.

³⁹¹ Monuments and Sites XV, 26.

Figure 91: Brick spalling in basement stairwell.

Figure 92: Limestone spalling on south-west façade.

Figure 93: Mortar spalling on south-east façade.

Figure 94: Paint spalling on interior ground floor walls.

Figure 95: Close up of paint spalling on ground floor.

Staining

Staining is a form of discolouration, which is the change in one to three of the colour parameters: hue, value, and chroma. Hue corresponds to the most prominent characteristic of a colour, value corresponds to the darkness or lightness of a colour, and chroma corresponds to the purity of a colour.³⁹² Iron staining can be found on the limestone entrance stairwell of Header House, as seen in Figure 96.

Figure 96: Iron stains on limestone entrance stairwell on south-east façade.

³⁹² Ibid., 46.

APPENDIX D – CONDITION DRAWINGS

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE

DRAWING: SOUTH-EAST ELEVATION

DRAWN BY: RALUCA DOBROTESCU

DATE: JANUARY 18, 2018

SCALE: 1:150

DRAWING NO.: B01 of B07

NOTES:

- THIS RECORD WAS PRODUCED USING THE INTERNATIONAL SYSTEM OF MEASUREMENTS.
- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:

- CRACK
- MORTAR SPALLING
- MISSING COMPONENT
- HOLE
- STAIN

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE		
DRAWING: SOUTH-WEST ELEVATION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 18, 2018	
SCALE: 1:150	DRAWING NO.: B02 of B07	

NOTES:
- THIS RECORD WAS PRODUCED USING THE INTERNATIONAL SYSTEM OF MEASUREMENTS.
- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:
— - CRACK
— - MORTAR SPALLING
■ - HOLE
■ - STONE SPALLING

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE	
DRAWING: NORTH-EAST ELEVATION	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 18, 2018
SCALE: 1:150	DRAWING NO.: B03 of B07

NOTES:
- THIS RECORD WAS PRODUCED USING THE INTERNATIONAL SYSTEM OF MEASUREMENTS.
- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:
- CRACK
- MORTAR SPALLING

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE

DRAWING: NORTH-WEST CUT-SECTION

DRAWN BY: RALUCA DOBROTESCU

DATE: JANUARY 18, 2018

SCALE: 1:150

DRAWING NO.: B04 of B07

NOTES:

- THIS RECORD WAS PRODUCED USING THE INTERNATIONAL SYSTEM OF MEASUREMENTS.
- THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
- THE DRAWING ACCURACY IS OF $\pm 50\text{mm}$.

LEGEND:

- CRACK
- HOLE
- BRICK SPALLING
- PAINT SPALLING
- RUST

CUT-SECTION NO.

DRAWING PAGE NO.

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE

DRAWING: SOUTH-EAST CUT-SECTION

DRAWN BY: RALUCA DOBROTESCU

DATE: JANUARY 18, 2018

SCALE: 1:150

DRAWING NO.: B05 of B07

NOTES:

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LEGEND:

- - BRICK SPALLING
- - PAINT SPALLING
- - EFFLORESCENCE

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE	
DRAWING: NORTH-EAST CUT-SECTION	
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 18, 2018
SCALE: 1:150	DRAWING NO.: B06 of B07

NOTES:
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LEGEND:
- PAINT SPALLING

PROJECT: CONDITION ASSESSMENT - HEADER HOUSE		
DRAWING: SOUTH-WEST CUT-SECTION		
DRAWN BY: RALUCA DOBROTESCU	DATE: JANUARY 18, 2018	
SCALE: 1:150	DRAWING NO.: B07 of B07	

- NOTES:
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 - THE INFORMATION CONTAINED WITHIN THIS RECORD HAS NOT BEEN FIELD VERIFIED.
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LEGEND:	
	- CRACK
	- BRICK SPALLING
	- PAINT SPALLING
	- RUST

APPENDIX E – CONDITION SURVEY FORM

Existing Condition - Rating System: 1 Urgent Attention Required; 2 Poor - Considerable Repairs Required; 3 Fair - Average Condition - Some Repairs Required; 4 Good - Minor Repairs Required; 5 Excellent - No Repairs Needed													
F/I : Further Investigation is Required													
N/A : Not Applicable													
Area Investigated	Composition/ Material	Existing Condition					Heritage Value			F/I	N/A	Photo Figure #	Notes
		1	2	3	4	5	High	Medium	Low				
EXTERIOR													
1.0 Roof													
1.1 General					X			X					
1.2 Shingles	Cedar					X		X				97	
1.3 Flashing					X			X					
2.0 South-East Wall													
2.1 General				X			X					98	
2.2 Mortar Joints	Lime			X			X						
2.3 Masonry	Limestone			X			X						
2.4 Arched Lintel	Limestone					X	X						
2.5 Overhang						X		X					
2.6 Stairs	Limestone		X					X				98	
2.7 Railing	Iron			X				X					
3.0 South-West Wall													
3.1 General				X			X						
3.2 Mortar Joints	Lime			X			X						
3.3 Masonry	Limestone			X			X						
3.4 Arched Lintel	Limestone					X	X						
4.0 North-West Wall													
4.1 General						X	X						
4.2 Mortar Joints	Lime					X	X						
4.3 Masonry	Limestone					X	X					100	
4.4 Arched Lintel	Limestone					X	X						
4.5 Flat Lintel	Limestone					X		X		X			Partially obstructed by patio.

Area Investigated	Composition/ Material	Existing Condition					Heritage Value			F/I	N/A	Photo Figure #	Notes
		1	2	3	4	5	High	Medium	Low				
5.0 North-East Wall													
5.1 General					X		X						
5.2 Mortar Joints	Lime				X		X						
5.3 Masonry	Limestone				X		X						
5.4 Arched Lintel	Limestone					X	X						
5.5 Flat Lintel	Limestone							X					Obstructed by storage shed.
6.0 Windows & Doors - South-East Façade													
6.1 General					X			X					
6.2 Glass						X		X					
6.3 Caulking						X			X				
6.4 Paint					X				X				
6.5 Grates	Iron					X	X						
6.6 Window Frame	Wood					X	X						
6.7 Door Frame	Wood					X	X						
6.8 Door Hinges					X				X				
6.9 Garage Door						X			X			101	
7.0 Windows - South-West Façade													
7.1 General					X			X					
7.2 Glass						X		X					
7.3 Caulking						X			X				
7.4 Paint					X				X				
7.5 Grates	Iron					X	X						
7.6 Window Frame	Wood					X	X						
8.0 Windows - North-West Façade													
8.1 General					X			X					
8.2 Glass						X		X					

Area Investigated		Composition/ Material	Existing Condition					Heritage Value			F/I	N/A	Photo Figure #	Notes
			1	2	3	4	5	High	Medium	Low				
8.0 Windows - North-West Façade														
8.3	Caulking					X			X					
8.4	Paint				X				X					
8.5	Grates	Iron				X	X							
8.6	Window Frame	Wood					X	X						
9.0 Windows - North-East Façade														
9.1	General					X		X		X			Basement window is obstructed.	
9.2	Glass					X		X						
9.3	Caulking					X			X					
9.4	Paint					X			X					
9.5	Grates	Iron					X	X						
9.6	Window Frame	Wood					X	X						
INTERIOR														
10.0 Attic														
10.1	General					X			X			102		
10.2	Walls	Gypsum				X			X					
10.3	Floors	Vinyl					X		X					
11.0 Ground Floor														
11.1	General				X				X			103		
11.2	Walls	Clay Brick			X				X					
11.3	Floors	Vinyl					X		X					
11.4	Ceiling	Gypsum				X			X					
11.5	Stairs	Wood					X		X					
11.6	Doors	Wood					X		X			104		
11.7	Arch	Wood					X		X					

Area Investigated	Composition/ Material	Existing Condition					Heritage Value			F/I	N/A	Photo Figure #	Notes
		1	2	3	4	5	High	Medium	Low				
12.0 Basement													
12.1 General			X						X			105	
12.2 Walls	Clay Brick		X						X	X			Partially blocked by freezer.
12.3 Floors	Concrete			X					X			106	
12.4 Ceiling						X			X				
12.5 Stairs	Wood					X			X			107	
12.6 Doors	Wood					X			X				
12.7 Chimney Hatch	Iron			X				X					
12.8 Fixed Furniture	Freezer					X			X				
13.0 Garage													
13.1 General						X			X				
13.2 Walls	Gypsum					X			X				
13.3 Floors	Concrete					X			X				
13.4 Ceiling	Gypsum					X			X				
13.5 Stairs	Wood					X			X				
13.6 Fixed Furniture	Appliances					X			X				
BUILDING SERVICES													
14.0 Electrical													
14.1 General									X	X			Not investigated.
15.0 Drainage													
15.1 General									X	X			Not investigated.
16.0 Plumbing													
16.1 General									X	X			Not investigated.
17.0 HVAC													
17.1 General									X	X			Not investigated.

Area Investigated	Composition/ Material	Existing Condition					Heritage Value			F/I	N/A	Photo Figure #	Notes
		1	2	3	4	5	High	Medium	Low				
SITE AND ENVIRONMENT													
18.0 General Site													
18.1 General						X			X				
18.2 Pavement	Brick					X		X					
18.3 Pavement	Asphalt				X				X				
18.4 Patio	Wood					X			X			108	
18.5 Storage Shed	Wood					X			X			109

Figure 97: Cedar roof shingles.

Figure 98: South-east facade.

Figure 99: Entrance stairwell on south-east façade.

Figure 100: Limestone masonry wall on north-west façade.

Figure 101: Garage door on south-east façade.

Figure 102: Header House attic.

Figure 103: Header House ground floor.

Figure 104: Main entrance door on ground level.

Figure 105: Basement of Header House.

Figure 106: Basement concrete floor.

Figure 107: Basement stairwell.

Figure 108: Header House patio space.

Figure 109: Wooden storage shed adjacent to north-east façade,

APPENDIX F – RECORDED SEISMIC DATA

Table 17: Summary of values for the model cases of the Lower Bound Front Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.11%	3.62	108.54	1.29	0.05%	1.67
6a1_2	0.07%	2.44	104.53	1.12	0.00%	0.02
6a1_3	0.08%	2.79	103.79	0.92	0.01%	0.48
6a2_1	0.12%	4.09	107.13	1.21	0.06%	1.95
6a2_2	0.15%	5.00	107.52	1.09	0.10%	3.25
6a2_3	0.20%	6.76	109.12	0.92	0.15%	4.91
7a1_1	0.16%	5.35	109.84	1.03	0.12%	4.13
7a1_2	0.10%	3.42	110.15	0.94	0.05%	1.66
7a1_3	0.09%	3.13	104.52	0.81	0.03%	1.07
7a2_1	0.09%	2.93	102.99	0.71	0.02%	0.63
7a2_2	0.12%	3.99	105.00	0.81	0.07%	2.38
7a2_3	0.10%	3.49	107.92	0.62	0.04%	1.48
VDB	0.36%	12.16	113.34	1.31	0.32%	10.88
Mean	0.13%	4.55	107.26	0.98	0.08%	2.65
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.09%	2.96	108.86	1.32	0.02%	0.79
6a1_2	0.07%	2.20	95.63	1.17	0.01%	0.19
6a1_3	0.08%	2.61	106.03	0.95	0.02%	0.78
6a2_1	0.09%	3.14	107.13	1.29	0.02%	0.63
6a2_2	0.14%	4.75	107.90	1.12	0.09%	3.01
6a2_3	0.21%	7.00	109.30	0.96	0.12%	4.08
7a1_1	0.16%	5.35	109.57	0.96	0.10%	3.45
7a1_2	0.09%	3.10	108.32	0.99	0.00%	0.16
7a1_3	0.08%	2.63	102.12	0.84	0.01%	0.31
7a2_1	0.07%	2.52	99.86	0.79	0.00%	0.17
7a2_2	0.11%	3.76	104.94	0.77	0.04%	1.45
7a2_3	0.10%	3.49	107.02	0.66	0.04%	1.38
VDB	0.49%	16.60	114.00	1.34	0.49%	16.43
Mean	0.14%	4.62	106.21	1.47	0.07%	2.52

Table 18: Summary of values for the model cases of the Upper Bound Front Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.11%	3.59	106.99	1.29	0.05%	1.66
6a1_2	0.07%	2.41	102.69	1.13	0.00%	0.01
6a1_3	0.08%	2.77	101.47	0.93	0.01%	0.40
6a2_1	0.12%	4.05	105.62	1.22	0.06%	1.91
6a2_2	0.15%	4.99	105.86	1.09	0.10%	3.24
6a2_3	0.20%	6.77	107.54	0.92	0.15%	4.90
7a1_1	0.16%	5.33	108.15	1.03	0.12%	4.13
7a1_2	0.10%	3.34	108.45	0.94	0.05%	1.56
7a1_3	0.09%	3.14	102.92	0.81	0.03%	1.06
7a2_1	0.09%	2.93	100.93	0.71	0.02%	0.66
7a2_2	0.12%	4.00	103.52	0.81	0.07%	2.39
7a2_3	0.11%	3.59	106.45	0.62	0.03%	1.16
VDB	0.36%	12.19	111.77	1.31	0.32%	10.88
Mean	0.13%	4.55	105.57	0.98	0.08%	2.61
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.08%	2.84	107.24	1.32	0.02%	0.80
6a1_2	0.07%	2.28	96.34	1.17	0.01%	0.37
6a1_3	0.08%	2.54	103.47	0.97	0.02%	0.73
6a2_1	0.09%	3.08	106.22	1.28	0.01%	0.42
6a2_2	0.14%	4.57	106.25	1.12	0.06%	2.01
6a2_3	0.20%	6.92	107.78	0.96	0.12%	4.06
7a1_1	0.16%	5.33	107.88	1.14	0.10%	3.40
7a1_2	0.09%	3.03	106.73	1.00	0.00%	0.10
7a1_3	0.07%	2.45	95.58	0.83	0.01%	0.33
7a2_1	0.07%	2.50	99.42	0.79	0.02%	0.71
7a2_2	0.11%	3.67	103.65	0.77	0.04%	1.45
7a2_3	0.11%	3.58	105.62	0.68	0.04%	1.34
VDB	0.49%	16.57	112.28	1.34	0.49%	16.42
Mean	0.14%	4.57	104.50	1.03	0.07%	2.47

Table 19: Summary of values for the model cases of the Lower Bound Front Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.11%	3.62	196.44	1.29	0.05%	1.67
6a1_2	0.07%	2.44	189.67	1.12	0.00%	0.02
6a1_3	0.08%	2.79	188.30	0.92	0.01%	0.48
6a2_1	0.12%	4.09	193.62	1.21	0.06%	1.95
6a2_2	0.15%	5.00	194.30	1.09	0.10%	3.25
6a2_3	0.20%	6.76	197.21	0.92	0.15%	4.91
7a1_1	0.16%	5.35	198.53	1.03	0.12%	4.13
7a1_2	0.10%	3.42	199.95	0.94	0.05%	1.66
7a1_3	0.09%	3.13	188.83	0.81	0.03%	1.07
7a2_1	0.09%	2.93	185.82	0.71	0.02%	0.63
7a2_2	0.12%	3.99	190.06	0.81	0.07%	2.38
7a2_3	0.10%	3.49	194.98	0.62	0.04%	1.48
VDB	0.36%	12.16	204.86	1.31	0.32%	10.88
Mean	0.13%	4.55	194.04	0.98	0.08%	2.65
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.13	197.45	1.27	0.06%	1.89
6a1_2	0.08%	2.75	199.88	1.10	0.02%	0.62
6a1_3	0.10%	3.31	197.05	0.95	0.03%	1.01
6a2_1	0.13%	4.47	193.62	1.17	0.08%	2.64
6a2_2	0.18%	6.24	194.57	1.05	0.11%	3.58
6a2_3	0.21%	7.26	196.50	0.93	0.15%	5.07
7a1_1	0.17%	5.84	199.33	1.03	0.13%	4.38
7a1_2	0.12%	4.02	205.61	0.90	0.07%	2.35
7a1_3	0.13%	4.24	193.69	0.83	0.07%	2.51
7a2_1	0.08%	2.85	191.14	0.69	0.03%	0.93
7a2_2	0.12%	4.01	188.92	0.78	0.07%	2.41
7a2_3	0.14%	4.71	191.47	0.62	0.05%	1.76
VDB	0.36%	12.01	204.23	1.29	0.32%	10.67
Mean	0.15%	5.06	196.42	0.97	0.09%	3.06

Table 20: Summary of values for the model cases of the Upper Bound Front Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.11%	3.59	192.66	1.29	0.05%	1.66
6a1_2	0.07%	2.41	184.98	1.13	0.00%	0.01
6a1_3	0.08%	2.77	182.27	0.93	0.01%	0.40
6a2_1	0.12%	4.05	189.94	1.22	0.06%	1.91
6a2_2	0.15%	4.99	190.36	1.09	0.10%	3.24
6a2_3	0.20%	6.77	193.37	0.92	0.15%	4.90
7a1_1	0.16%	5.33	194.47	1.03	0.12%	4.13
7a1_2	0.10%	3.34	195.38	0.94	0.05%	1.56
7a1_3	0.09%	3.14	185.13	0.81	0.03%	1.06
7a2_1	0.09%	2.93	181.44	0.71	0.02%	0.66
7a2_2	0.12%	4.00	186.15	0.81	0.07%	2.39
7a2_3	0.11%	3.59	191.44	0.62	0.03%	1.16
VDB	0.36%	12.19	200.99	1.31	0.32%	10.88
Mean	0.13%	4.55	189.89	0.98	0.08%	2.61
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.02	192.93	1.27	0.06%	1.96
6a1_2	0.08%	2.73	195.89	1.10	0.02%	0.55
6a1_3	0.10%	3.29	193.14	0.95	0.03%	1.03
6a2_1	0.13%	4.47	188.99	1.18	0.08%	2.64
6a2_2	0.18%	6.20	190.55	1.06	0.10%	3.51
6a2_3	0.21%	7.24	192.60	0.93	0.15%	5.07
7a1_1	0.17%	5.82	195.30	1.03	0.13%	4.36
7a1_2	0.12%	4.00	201.52	0.90	0.07%	2.34
7a1_3	0.11%	3.77	190.01	0.83	0.06%	2.13
7a2_1	0.08%	2.83	187.26	0.69	0.03%	0.92
7a2_2	0.12%	4.03	185.73	0.78	0.07%	2.45
7a2_3	0.14%	4.66	187.84	0.63	0.05%	1.73
VDB	0.35%	11.99	200.04	1.29	0.32%	10.66
Mean	0.15%	5.00	192.45	0.97	0.09%	3.03

Table 21: Summary of values for the model cases of the Lower Bound Left Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.81	48.00	1.22	0.10%	3.31
6a1_2	0.14%	4.57	47.68	1.06	0.07%	2.23
6a1_3	0.16%	5.41	47.09	0.94	0.10%	3.44
6a2_1	0.15%	4.94	46.73	1.16	0.08%	2.70
6a2_2	0.20%	6.65	47.23	0.97	0.14%	4.77
6a2_3	0.27%	9.07	48.38	1.03	0.20%	6.85
7a1_1	0.19%	6.56	48.61	1.02	0.14%	4.65
7a1_2	0.12%	4.20	49.60	0.87	0.05%	1.55
7a1_3	0.12%	4.02	46.55	0.78	0.01%	0.31
7a2_1	0.17%	5.90	46.82	0.63	0.10%	3.46
7a2_2	0.16%	5.55	47.13	0.69	0.11%	3.88
7a2_3	0.18%	5.97	47.59	0.64	0.13%	4.30
VDB	0.20%	6.84	48.20	1.25	0.17%	5.64
Mean	0.17%	5.73	47.66	0.94	0.11%	3.62
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.87	47.89	1.24	0.10%	3.35
6a1_2	0.13%	4.38	47.51	1.03	0.07%	2.51
6a1_3	0.17%	5.62	47.11	0.95	0.11%	3.71
6a2_1	0.16%	5.54	46.73	1.17	0.09%	3.21
6a2_2	0.20%	6.71	47.13	0.98	0.14%	4.69
6a2_3	0.28%	9.55	48.69	1.03	0.21%	7.26
7a1_1	0.20%	6.66	48.64	1.03	0.15%	4.95
7a1_2	0.12%	4.08	49.60	0.88	0.04%	1.52
7a1_3	0.12%	4.19	45.72	0.78	0.07%	2.43
7a2_1	0.15%	4.97	46.82	0.64	0.10%	3.34
7a2_2	0.19%	6.32	45.76	0.71	0.12%	4.02
7a2_3	0.16%	5.38	47.70	0.66	0.11%	3.60
VDB	0.21%	7.07	47.79	1.25	0.17%	5.91
Mean	0.17%	5.80	47.47	0.95	0.11%	3.88

Table 22: Summary of values for the model cases of the Upper Bound Left Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.67	45.59	1.22	0.09%	2.89
6a1_2	0.14%	4.61	46.16	1.05	0.07%	2.30
6a1_3	0.16%	5.42	44.78	0.94	0.10%	3.47
6a2_1	0.15%	5.22	44.49	1.16	0.09%	3.06
6a2_2	0.19%	6.59	44.83	0.97	0.14%	4.64
6a2_3	0.27%	9.15	46.18	1.03	0.21%	6.95
7a1_1	0.19%	6.56	46.20	1.02	0.14%	4.70
7a1_2	0.12%	4.19	47.18	0.87	0.05%	1.60
7a1_3	0.12%	4.10	44.00	0.78	0.07%	2.31
7a2_1	0.17%	5.84	44.45	0.63	0.10%	3.44
7a2_2	0.16%	5.46	44.53	0.70	0.11%	3.75
7a2_3	0.17%	5.86	45.04	0.65	0.13%	4.23
VDB	0.20%	6.79	45.71	1.25	0.17%	5.60
Mean	0.17%	5.73	45.32	0.94	0.11%	3.76
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.71	45.58	1.24	0.09%	2.87
6a1_2	0.12%	4.05	45.29	1.03	0.07%	2.49
6a1_3	0.17%	5.58	44.82	0.95	0.11%	3.71
6a2_1	0.16%	5.50	44.78	1.17	0.10%	3.21
6a2_2	0.19%	6.59	44.90	0.98	0.14%	4.62
6a2_3	0.27%	9.04	46.12	1.02	0.20%	6.61
7a1_1	0.20%	6.74	46.31	1.03	0.15%	5.18
7a1_2	0.12%	4.04	47.22	0.88	0.05%	1.53
7a1_3	0.11%	3.81	43.45	0.79	0.06%	1.86
7a2_1	0.14%	4.77	44.51	0.64	0.09%	3.16
7a2_2	0.15%	5.13	43.41	0.73	0.10%	3.53
7a2_3	0.16%	5.25	45.31	0.66	0.11%	3.56
VDB	0.21%	7.10	45.27	1.25	0.18%	6.00
Mean	0.16%	5.56	45.15	0.95	0.11%	3.72

Table 23: Summary of values for the model cases of the Lower Bound Left Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.81	99.68	1.22	0.10%	3.31
6a1_2	0.14%	4.57	99.92	1.06	0.07%	2.23
6a1_3	0.16%	5.41	97.87	0.94	0.10%	3.44
6a2_1	0.15%	4.94	97.08	1.16	0.08%	2.70
6a2_2	0.20%	6.65	98.14	0.97	0.14%	4.77
6a2_3	0.27%	9.07	101.38	1.03	0.20%	6.85
7a1_1	0.19%	6.56	101.00	1.02	0.14%	4.65
7a1_2	0.12%	4.20	103.12	0.87	0.05%	1.55
7a1_3	0.12%	4.02	97.18	0.78	0.01%	0.31
7a2_1	0.17%	5.90	97.27	0.63	0.10%	3.46
7a2_2	0.16%	5.55	98.81	0.69	0.11%	3.88
7a2_3	0.18%	5.97	98.92	0.64	0.13%	4.30
VDB	0.20%	6.84	100.21	1.25	0.17%	5.64
Mean	0.17%	5.73	99.28	0.94	0.11%	3.62
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.79	99.92	1.22	0.10%	3.34
6a1_2	0.13%	4.42	101.11	1.07	0.06%	2.07
6a1_3	0.16%	5.39	97.83	0.93	0.10%	3.43
6a2_1	0.16%	5.28	97.08	1.13	0.10%	3.37
6a2_2	0.20%	6.67	98.29	0.97	0.14%	4.78
6a2_3	0.24%	8.16	101.33	1.02	0.18%	5.93
7a1_1	0.20%	6.72	100.94	1.01	0.14%	4.79
7a1_2	0.13%	4.26	103.06	0.87	0.03%	1.13
7a1_3	0.12%	4.17	97.32	0.76	0.02%	0.62
7a2_1	0.18%	6.17	96.89	0.63	0.11%	3.61
7a2_2	0.15%	5.05	99.08	0.68	0.10%	3.32
7a2_3	0.19%	6.35	98.83	0.64	0.14%	4.62
VDB	0.21%	7.16	100.49	1.25	0.18%	6.06
Mean	0.17%	5.74	99.40	0.94	0.11%	3.62

Table 24: Summary of values for the model cases of the Upper Bound Left Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.67	94.67	1.22	0.09%	2.89
6a1_2	0.14%	4.61	95.70	1.05	0.07%	2.30
6a1_3	0.16%	5.42	92.54	0.94	0.10%	3.47
6a2_1	0.15%	5.22	91.91	1.16	0.09%	3.06
6a2_2	0.19%	6.59	92.74	0.97	0.14%	4.64
6a2_3	0.27%	9.15	95.76	1.03	0.21%	6.95
7a1_1	0.19%	6.56	95.53	1.02	0.14%	4.70
7a1_2	0.12%	4.19	97.44	0.87	0.05%	1.60
7a1_3	0.12%	4.10	91.21	0.78	0.07%	2.31
7a2_1	0.17%	5.84	91.83	0.63	0.10%	3.44
7a2_2	0.16%	5.46	92.38	0.70	0.11%	3.75
7a2_3	0.17%	5.86	93.26	0.65	0.13%	4.23
VDB	0.20%	6.79	94.46	1.25	0.17%	5.60
Mean	0.17%	5.73	93.80	0.94	0.11%	3.76
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.82	94.63	1.22	0.10%	3.39
6a1_2	0.13%	4.47	95.23	1.07	0.06%	2.16
6a1_3	0.16%	5.39	92.43	0.93	0.10%	3.45
6a2_1	0.15%	5.18	91.86	1.14	0.10%	3.28
6a2_2	0.20%	6.64	92.84	0.97	0.14%	4.76
6a2_3	0.24%	8.27	95.84	1.02	0.18%	6.06
7a1_1	0.20%	6.72	95.38	1.01	0.14%	4.80
7a1_2	0.13%	4.25	97.33	0.87	0.04%	1.28
7a1_3	0.12%	4.09	91.93	0.77	0.01%	0.17
7a2_1	0.15%	5.13	91.87	0.63	0.10%	3.49
7a2_2	0.15%	5.10	93.95	0.69	0.10%	3.42
7a2_3	0.19%	6.28	93.20	0.64	0.14%	4.57
VDB	0.21%	7.12	94.60	1.25	0.18%	6.02
Mean	0.17%	5.65	93.93	0.94	0.11%	3.60

Table 25: Summary of values for the model cases of the Lower Bound Back Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.21	110.26	1.27	0.07%	2.27
6a1_2	0.09%	2.97	81.84	1.07	0.03%	0.92
6a1_3	0.11%	3.86	83.72	0.98	0.01%	0.32
6a2_1	0.13%	4.40	116.14	1.17	0.08%	2.55
6a2_2	0.17%	5.86	85.51	1.01	0.10%	3.45
6a2_3	0.22%	7.29	96.31	0.92	0.15%	5.14
7a1_1	0.17%	5.71	67.12	1.02	0.14%	4.71
7a1_2	0.12%	4.18	96.36	0.87	0.07%	2.51
7a1_3	0.11%	3.66	83.95	0.83	0.06%	1.99
7a2_1	0.11%	3.71	73.22	0.68	0.06%	1.98
7a2_2	0.12%	4.19	94.68	0.78	0.08%	2.61
7a2_3	0.17%	5.67	104.78	0.64	0.09%	3.07
VDB	0.31%	10.61	116.88	1.24	0.27%	9.09
Mean	0.15%	5.10	93.14	0.96	0.09%	3.12
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.10%	3.36	108.81	1.35	0.03%	1.11
6a1_2	0.07%	2.51	112.78	1.22	0.03%	0.92
6a1_3	0.07%	2.53	80.37	1.06	0.01%	0.46
6a2_1	0.10%	3.53	3.40	1.30	0.04%	1.32
6a2_2	0.11%	3.63	104.33	1.12	0.04%	1.47
6a2_3	0.13%	4.39	112.78	1.09	0.06%	2.17
7a1_1	0.19%	6.33	117.25	1.27	0.15%	4.91
7a1_2	0.11%	3.87	101.29	1.13	0.04%	1.49
7a1_3	0.09%	3.05	100.81	0.97	0.05%	1.64
7a2_1	0.13%	4.29	84.59	0.97	0.07%	2.36
7a2_2	0.15%	5.07	73.76	0.90	0.09%	3.03
7a2_3	0.11%	3.75	81.12	0.85	0.07%	2.33
VDB	0.41%	13.76	121.51	1.36	0.40%	13.49
Mean	0.14%	4.62	92.52	1.12	0.08%	2.82

Table 26: Summary of values for the model cases of the Upper Bound Back Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.01	107.44	1.27	0.05%	1.63
6a1_2	0.09%	2.91	77.38	1.08	0.02%	0.80
6a1_3	0.11%	3.72	38.87	0.97	0.01%	0.22
6a2_1	0.13%	4.39	73.23	1.17	0.08%	2.58
6a2_2	0.17%	5.89	116.69	1.02	0.10%	3.45
6a2_3	0.21%	7.25	62.93	0.93	0.15%	5.09
7a1_1	0.17%	5.68	97.84	1.02	0.14%	4.62
7a1_2	0.12%	4.14	117.71	0.87	0.07%	2.49
7a1_3	0.12%	4.02	100.96	0.82	0.07%	2.27
7a2_1	0.10%	3.45	120.36	0.68	0.05%	1.70
7a2_2	0.13%	4.27	34.22	0.78	0.08%	2.72
7a2_3	0.16%	5.47	42.97	0.63	0.09%	2.98
VDB	0.33%	11.10	34.37	1.28	0.28%	9.41
Mean	0.15%	5.10	78.84	0.96	0.09%	3.07
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.10%	3.44	107.46	1.35	0.04%	1.40
6a1_2	0.07%	2.50	115.44	1.23	0.03%	0.99
6a1_3	0.08%	2.61	80.81	1.07	0.01%	0.41
6a2_1	0.10%	3.38	108.29	1.32	0.03%	1.10
6a2_2	0.12%	3.99	98.52	1.20	0.06%	2.19
6a2_3	0.13%	4.24	106.40	1.10	0.06%	2.12
7a1_1	0.19%	6.46	117.57	1.27	0.15%	5.03
7a1_2	0.10%	3.51	95.32	1.13	0.04%	1.20
7a1_3	0.09%	3.07	93.32	0.95	0.05%	1.56
7a2_1	0.12%	4.10	82.02	0.99	0.06%	2.12
7a2_2	0.15%	5.06	74.78	0.90	0.09%	3.00
7a2_3	0.11%	3.68	76.55	0.88	0.07%	2.30
VDB	0.40%	13.48	117.09	1.37	0.38%	12.81
Mean	0.14%	4.58	97.97	1.14	0.08%	2.79

Table 27: Summary of values for the model cases of the Lower Bound Back Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.21	217.11	1.27	0.07%	2.27
6a1_2	0.09%	2.97	145.19	1.07	0.03%	0.92
6a1_3	0.11%	3.86	151.08	0.98	0.01%	0.32
6a2_1	0.13%	4.40	222.15	1.17	0.08%	2.55
6a2_2	0.17%	5.86	172.56	1.01	0.10%	3.45
6a2_3	0.22%	7.29	172.01	0.92	0.15%	5.14
7a1_1	0.17%	5.71	121.19	1.02	0.14%	4.71
7a1_2	0.12%	4.18	297.64	0.87	0.07%	2.51
7a1_3	0.11%	3.66	155.40	0.83	0.06%	1.99
7a2_1	0.11%	3.71	128.26	0.68	0.06%	1.98
7a2_2	0.12%	4.19	218.20	0.78	0.08%	2.61
7a2_3	0.17%	5.67	225.94	0.64	0.09%	3.07
VDB	0.31%	10.61	243.97	1.24	0.27%	9.09
Mean	0.15%	5.10	190.05	0.96	0.09%	3.12
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.80	197.21	1.15	0.09%	3.00
6a1_2	0.16%	5.42	207.97	1.05	0.12%	3.98
6a1_3	0.15%	4.91	141.52	0.95	0.10%	3.41
6a2_1	0.22%	7.56	17.49	1.08	0.17%	5.91
6a2_2	0.21%	7.19	172.37	0.97	0.16%	5.53
6a2_3	0.26%	8.70	166.92	1.04	0.21%	7.11
7a1_1	0.24%	8.08	172.20	0.99	0.23%	7.64
7a1_2	0.13%	4.37	142.60	0.88	0.06%	2.16
7a1_3	0.12%	3.93	160.90	0.76	0.07%	2.26
7a2_1	0.18%	6.21	148.00	0.65	0.15%	5.00
7a2_2	0.18%	6.11	142.50	0.72	0.13%	4.46
7a2_3	0.17%	5.65	134.24	0.66	0.12%	4.16
VDB	0.21%	7.13	184.92	1.26	0.18%	6.16
Mean	0.18%	6.16	152.99	0.94	0.14%	4.68

Table 28: Summary of values for the model cases of the Upper Bound Back Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.12%	4.01	199.19	1.27	0.05%	1.63
6a1_2	0.09%	2.91	147.19	1.08	0.02%	0.80
6a1_3	0.11%	3.72	78.43	0.97	0.01%	0.22
6a2_1	0.13%	4.39	133.01	1.17	0.08%	2.58
6a2_2	0.17%	5.89	235.42	1.02	0.10%	3.45
6a2_3	0.21%	7.25	123.75	0.93	0.15%	5.09
7a1_1	0.17%	5.68	194.15	1.02	0.14%	4.62
7a1_2	0.12%	4.14	281.76	0.87	0.07%	2.49
7a1_3	0.12%	4.02	206.10	0.82	0.07%	2.27
7a2_1	0.10%	3.45	273.36	0.68	0.05%	1.70
7a2_2	0.13%	4.27	57.77	0.78	0.08%	2.72
7a2_3	0.16%	5.47	93.76	0.63	0.09%	2.98
VDB	0.33%	11.10	141.13	1.28	0.28%	9.41
Mean	0.15%	5.10	166.54	0.96	0.09%	3.07
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.83	189.78	1.15	0.10%	3.25
6a1_2	0.16%	5.31	198.57	1.05	0.12%	3.89
6a1_3	0.14%	4.88	136.35	0.95	0.10%	3.34
6a2_1	0.22%	7.56	202.89	1.08	0.17%	5.91
6a2_2	0.22%	7.29	164.58	0.97	0.17%	5.80
6a2_3	0.26%	8.85	157.27	1.04	0.22%	7.27
7a1_1	0.24%	7.99	165.95	0.99	0.22%	7.57
7a1_2	0.13%	4.40	133.53	0.88	0.07%	2.22
7a1_3	0.12%	4.00	154.04	0.77	0.06%	2.19
7a2_1	0.16%	5.49	130.25	0.65	0.09%	2.95
7a2_2	0.18%	5.98	137.86	0.72	0.13%	4.32
7a2_3	0.17%	5.58	128.98	0.66	0.12%	4.11
VDB	0.21%	7.16	179.35	1.26	0.18%	6.21
Mean	0.18%	6.10	159.95	0.94	0.13%	4.54

Table 29: Summary of values for the model cases of the Lower Bound Right Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.86	47.21	1.28	0.09%	3.05
6a1_2	0.09%	3.15	45.66	1.05	0.04%	1.21
6a1_3	0.13%	4.45	47.34	0.98	0.07%	2.48
6a2_1	0.14%	4.84	47.85	1.19	0.09%	3.10
6a2_2	0.17%	5.75	48.37	0.99	0.10%	3.25
6a2_3	0.22%	7.48	48.84	0.92	0.16%	5.36
7a1_1	0.18%	6.07	47.69	1.03	0.15%	5.00
7a1_2	0.12%	4.22	49.10	0.88	0.07%	2.51
7a1_3	0.10%	3.46	46.20	0.84	0.05%	1.54
7a2_1	0.13%	4.53	43.51	0.72	0.06%	1.99
7a2_2	0.12%	4.18	44.97	0.77	0.08%	2.60
7a2_3	0.17%	5.88	45.12	0.66	0.10%	3.36
VDB	0.29%	9.94	61.55	1.22	0.25%	8.39
Mean	0.16%	5.29	47.96	0.96	0.10%	3.37
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.15%	5.06	48.04	1.27	0.12%	4.03
6a1_2	0.11%	3.82	50.84	1.12	0.04%	1.34
6a1_3	0.12%	4.15	34.17	0.99	0.07%	2.53
6a2_1	0.12%	4.10	0.58	1.18	0.05%	1.72
6a2_2	0.18%	5.99	44.30	1.05	0.11%	3.67
6a2_3	0.23%	7.69	49.20	0.96	0.20%	6.60
7a1_1	0.22%	7.45	44.88	1.10	0.18%	6.00
7a1_2	0.12%	4.13	37.84	0.96	0.09%	2.95
7a1_3	0.12%	4.10	35.36	0.89	0.07%	2.33
7a2_1	0.10%	3.44	30.81	0.73	0.05%	1.61
7a2_2	0.14%	4.63	29.88	0.82	0.10%	3.26
7a2_3	0.16%	5.33	28.83	0.68	0.07%	2.48
VDB	0.28%	9.37	40.95	1.30	0.20%	6.89
Mean	0.16%	5.33	36.59	1.00	0.10%	3.49

Table 30: Summary of values for the model cases of the Upper Bound Right Brick Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.72	46.44	1.28	0.09%	2.93
6a1_2	0.09%	3.09	42.64	1.06	0.03%	1.17
6a1_3	0.13%	4.31	42.61	0.98	0.05%	1.60
6a2_1	0.14%	4.59	41.85	1.19	0.08%	2.83
6a2_2	0.17%	5.85	45.00	1.00	0.11%	3.56
6a2_3	0.22%	7.46	45.75	0.92	0.16%	5.37
7a1_1	0.18%	5.95	48.14	1.03	0.15%	4.96
7a1_2	0.12%	4.22	47.98	0.87	0.08%	2.54
7a1_3	0.10%	3.49	40.89	0.84	0.05%	1.72
7a2_1	0.13%	4.43	41.27	0.72	0.08%	2.77
7a2_2	0.12%	4.15	42.74	0.77	0.08%	2.58
7a2_3	0.17%	5.78	42.04	0.65	0.10%	3.31
VDB	0.28%	9.39	83.08	1.22	0.20%	6.65
Mean	0.15%	5.19	46.96	0.96	0.10%	3.23
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.15%	4.98	45.88	1.27	0.12%	4.00
6a1_2	0.11%	3.83	47.66	1.12	0.06%	2.05
6a1_3	0.11%	3.89	31.48	0.98	0.07%	2.20
6a2_1	0.12%	4.05	38.56	1.16	0.05%	1.79
6a2_2	0.18%	5.93	43.12	1.06	0.09%	3.07
6a2_3	0.23%	7.67	43.16	0.96	0.14%	4.63
7a1_1	0.22%	7.49	44.93	1.11	0.18%	5.98
7a1_2	0.13%	4.25	38.20	0.96	0.09%	3.10
7a1_3	0.13%	4.23	32.52	0.89	0.05%	1.82
7a2_1	0.11%	3.72	29.58	0.72	0.06%	1.86
7a2_2	0.14%	4.89	27.69	0.82	0.10%	3.52
7a2_3	0.16%	5.48	28.53	0.69	0.08%	2.68
VDB	0.28%	9.52	42.05	1.31	0.21%	6.99
Mean	0.16%	5.38	37.95	1.00	0.10%	3.36

Table 31: Summary of values for the model cases of the Lower Bound Right Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.86	100.42	1.28	0.09%	3.05
6a1_2	0.09%	3.15	95.83	1.05	0.04%	1.21
6a1_3	0.13%	4.45	90.65	0.98	0.07%	2.48
6a2_1	0.14%	4.84	88.85	1.19	0.09%	3.10
6a2_2	0.17%	5.75	96.47	0.99	0.11%	3.59
6a2_3	0.22%	7.48	96.13	0.92	0.16%	5.36
7a1_1	0.18%	6.07	96.24	1.03	0.15%	5.00
7a1_2	0.12%	4.22	96.00	0.88	0.07%	2.51
7a1_3	0.10%	3.46	93.21	0.84	0.05%	1.54
7a2_1	0.13%	4.53	92.24	0.72	0.06%	1.99
7a2_2	0.12%	4.18	102.22	0.77	0.08%	2.60
7a2_3	0.17%	5.88	92.39	0.66	0.10%	3.36
VDB	0.29%	9.94	97.07	1.22	0.25%	8.39
Mean	0.16%	5.29	95.21	0.96	0.10%	3.40
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.15%	5.01	99.60	1.20	0.14%	4.67
6a1_2	0.14%	4.71	82.53	1.03	0.09%	3.05
6a1_3	0.14%	4.77	72.90	0.97	0.11%	3.58
6a2_1	0.21%	7.14	0.13	1.09	0.17%	5.91
6a2_2	0.20%	6.77	73.72	0.99	0.16%	5.34
6a2_3	0.32%	10.65	65.24	1.05	0.27%	9.04
7a1_1	0.22%	7.48	84.18	1.03	0.20%	6.81
7a1_2	0.13%	4.36	70.24	0.90	0.06%	2.20
7a1_3	0.13%	4.31	63.61	0.79	0.06%	2.07
7a2_1	0.13%	4.27	59.08	0.68	0.05%	1.81
7a2_2	0.15%	5.07	57.93	0.73	0.12%	3.89
7a2_3	0.15%	4.98	50.61	0.68	0.09%	2.93
VDB	0.25%	8.30	76.73	1.26	0.18%	6.03
Mean	0.18%	5.99	65.88	0.95	0.13%	4.41

Table 32: Summary of values for the model cases of the Upper Bound Right Stone Wall

Earth-quake ID	Rigid					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.14%	4.72	113.91	1.28	0.09%	2.93
6a1_2	0.09%	3.09	115.24	1.06	0.03%	1.17
6a1_3	0.13%	4.31	115.10	0.98	0.05%	1.60
6a2_1	0.14%	4.59	112.74	1.19	0.08%	2.83
6a2_2	0.17%	5.85	114.64	1.00	0.11%	3.56
6a2_3	0.22%	7.46	114.81	0.92	0.16%	5.37
7a1_1	0.18%	5.95	120.60	1.03	0.15%	4.96
7a1_2	0.12%	4.22	113.88	0.87	0.08%	2.54
7a1_3	0.10%	3.49	117.54	0.84	0.05%	1.72
7a2_1	0.13%	4.43	118.42	0.72	0.08%	2.77
7a2_2	0.12%	4.15	110.23	0.77	0.08%	2.58
7a2_3	0.17%	5.78	108.52	0.65	0.10%	3.31
VDB	0.28%	9.39	214.78	1.22	0.20%	6.65
Mean	0.15%	5.19	122.34	0.96	0.10%	3.23
Earth-quake ID	Flexible					
	Max Drift	Max Displacement (mm)	Max Base Shear (kN)	Max Acceleration (g)	Residual Drift	Residual Displacement (mm)
6a1_1	0.15%	5.06	95.73	1.20	0.15%	5.04
6a1_2	0.14%	4.69	78.72	1.03	0.09%	2.98
6a1_3	0.14%	4.77	69.63	0.97	0.10%	3.54
6a2_1	0.21%	7.00	75.72	1.09	0.17%	5.82
6a2_2	0.20%	6.81	70.47	0.99	0.16%	5.38
6a2_3	0.31%	10.59	62.99	1.04	0.26%	8.87
7a1_1	0.22%	7.47	80.96	1.03	0.20%	6.75
7a1_2	0.13%	4.36	67.48	0.90	0.07%	2.21
7a1_3	0.13%	4.29	60.12	0.79	0.06%	2.03
7a2_1	0.13%	4.25	55.26	0.68	0.02%	0.74
7a2_2	0.15%	5.08	55.44	0.73	0.12%	3.91
7a2_3	0.15%	4.97	47.84	0.68	0.09%	2.92
VDB	0.25%	8.32	72.22	1.26	0.18%	6.04
Mean	0.18%	5.97	68.66	0.95	0.13%	4.32