Material Characterization of Wood in Historic Structures

by

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Abstract

When the need arises to evaluate a historic wood building, structural evaluation is often based on current design codes. The material properties defined in current codes are based on modern fast-grown lumber, whereas historic buildings were often constructed from old-growth wood, which has a considerably higher quality and strength.

This thesis examines the relationship between resistance drill measurement and the mechanical properties of Eastern white pine. The goal of the research is to determine the viability of resistance drill testing for in-situ material characterization of historic wood structures.

The case study of a historic barn is presented in this research to establish a workflow for evaluating wood structures through on-site non-destructive testing, using the experimentally-determined relationships between material properties and resistance drill measurement. The workflow includes documentation, condition assessment, material characterization, and iterative evaluation of the building's structural performance.

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Acronyms

ALSC American Lumber Standard Committee

ASTM American Society for Testing Materials

ASTM D143 Standard Test Methods for Small Clear Specimens of Timber

ASTM D198 Standard Test Methods of Static Tests of Lumber in Structural Sizes

ASTM D1990 Standard Practice for Establishing Allowable Properties for Visually-Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens

ASTM D245 Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber

ASTM D2555 Standard Practice for Establishing Clear Wood Strength Values

ASTM D2915 Standard Practice for Sampling and Data-Analysis for Structural Wood and Wood-Based Products

ASTM D4442 Standard Test Methods for Direct Moisture Content Measurement of Wood and Wood-Based Materials

ASTM D4761 Standard Test Methods for Mechanical Properties of Lumber and Wood-Based Structural Materials

BIM Building Information Model

CESA Canadian Engineering Standard Association

CLSB Canadian Lumber Standards Bureau

CMHC Canadian Mortgage and Housing Corporation

CSA Canadian Standards Association

CSA O86 Engineering Design in Wood

CWC Canadian Wood Council

DCR Demand-Capacity Ratio

EMC Equilibrium Moisture Content

FEA Finite Element Analysis

FPL Forest Product Laboratory

FPS Forest Product Service

FSP Fibre Saturation Point

IATM International Association for Testing Materials

ICOMOS International Council of Monuments and Sites

ISO International Organization for Standardization

ISO 13822:2010 Bases for design of structures — Assessment of existing structures

LSD Limit States Design

MC Moisture Content

MoE Modulus of Elasticity

MoR Modulus of Rupture

NBC National Building Code

NDS National Design Specification

NDT Non-Destructive Testing

NLGA National Lumber Grading Authority

NRC National Research Council of Canada

RA Resistance Drill Amplitude

RD Ring Density

RM Resistance Drill Measurement

SCC Standards Council of Canada

ULS Ultimate Limit States

USDA United States Department of Agriculture

USFHA United States Federal Housing Authority

UTM Universal Testing Machine

WSD Working Stress Design

List of Symbols

f	Normally distributed property
f_b	Static bending strength
f_c	Compressive strength parallel to grain
f_{cp}	Compressive strength perpendicular to grain
h	Length of resistance drill graph
m	Mass
q	Measured property related to f
C	Capacity
E_b	Modulus of elasticity in bending
E_c	Modulus of elasticity in compression
E	Modulus of elasticity
J	Performance function
MC_P	Moisture content at fiber saturation point
MC	Moisture Content (%)
P_f	Probability of failure
P_{12}	Mechanical property at moisture content 12%
P_g	Mechanical property for green wood
P	Mechanical property at moisture content M
RA	Resistance drill graph amplitude
RM	Resistance drill measurement
G	Specific Gravity
V	Volume

D Demand

σ_f Standard deviation of f

σ_q Standard deviation of q

β Reliability Index

μ_f Mean of f

σ'_f Updated standard deviation of f

μ_q Mean of q

ρ Density

μ'_f Updated mean of f

ε Strain

σ Stress

1. Introduction

1.1 Background

Wood construction is a tradition seen throughout Canadian history that embodies tangible and intangible heritage values. The oldest surviving wood structures in Canada date as far back as the 17th century. As an abundant natural resource, wood was historically the preferred construction material for vernacular structures. In rural areas, when a new building was needed the wood was sourced from the site, often from old-growth forests. As a result, the wood used in these historic structures came from centuries-old trees and was of a very high quality. Prior to the development of building codes in the early twentieth century, structural design relied on empirical formulas rather than complex equations. A high degree of craftsmanship was necessary to construct structures of quality. Despite a lack of standard regulations, many wood structures constructed before the development of building codes still stand.

Well maintained wood structures can last for centuries (Lourenço, 2005; Weaver, 2003); although as a natural material, wood is susceptible to deterioration, decay, and weathering. Additionally, wood is an irregular material with natural imperfections and irregularities. Defects such as knots, slope of grain, checks, wane, and shake, can significantly impact the strength of wood. These natural defects can be especially critical at locations of high internal stresses within a structural member (R. W. Anthony et al., 2009). In addition to defects, wood is susceptible to deterioration due to fungus, insects or excessive moisture. Material degradation significantly limits the strength of structural wood; it is important to evaluate the extent and severity of any deterioration in a wood structure. When evaluating a historic wood building, one must consider both the material condition and the mechanical condition (Cruz et al., 2015).

In addition to the material characteristics, the load-carrying behavior of a historic structure must be evaluated. In order to preserve wooden built heritage, it is imperative to understand the structural performance. In new construction, engineers typically rely on standardized values from a recognized wood design code, such as *Engineering Design in Wood (CSA O86)* or *National Design Specification (NDS) for Wood Construction*, to estimate the load-carrying capacity of a structure (American National Standards Institute, 2018; Canadian Standards Association & Canadian Wood Council, 2015). Material properties specified in modern wood design codes are overly conservative when applied to historic wood in good condition. Modern codes are based on modern, rapid-grown, lumber which has significantly lower strength and quality than the old growth timber used in historic construction. Furthermore, old-growth wood often exceeds the highest visual National Lumber Grading Authority (NLGA) grading requirements, making it challenging to assign the correct grade (National Lumber Grading Association, 2017).

Furthermore, since wood is a natural material with highly variable properties, the material properties in North American wood design codes are based on statistical analyses. The values in these design codes are based on the fifth percentile exclusion values of small clear specimens and reliability analysis principles based on destructive testing of full-scale structural members (Canadian Wood Council, 2014). Based on this probabilistic approach, 95% of structural members are expected to meet or surpass the design values presented in the code. For new design, this limits the risk of failure due to unsound material. However, for existing wood buildings, the structural components have proven their adequacy throughout the building's lifespan. Members with strengths that fall below the fifth percentile would demonstrate structural insufficiency through mechanical damages, such as splitting or crushing. Hence, the values specified in current design codes can be conservative with respect to historic timber in good condition. Such conservative design values result in a calculated design capacity which underestimates the actual capacity of a structure.

In a typical structural evaluation, the capacity of the structure based on a recognized wood design code is compared to the applied loads on the structure. The applied loads are based on a recognized building code, such as the National Building Code (NBC), to determine whether the structure meets the current code requirements (American National Standards Institute, 2018; Canadian Commission on Building and Fire Codes, 2015a; Canadian Standards Association & Canadian Wood Council, 2015). The calculated structural capacity is based on modern wood design codes which tend to be overly conservative for historic wood; therefore many historic structures fail to meet code requirements. The NBC and most other building codes are intended for new construction, and are not always applicable for existing structures (Canadian Commission on Building and Fire Codes, 2015a). However, Structural Commentary L of the NBC 2015 does allow for a reduction in the factored load based on a building's past performance and the risk of occupant hazard (Canadian Commission on Building and Fire Codes, 2015b). In some cases, a historic wood structure may meet the code requirements, in which case there is no need for a more accurate analysis. However, when a structure that is deemed inadequate by the building code has stood for centuries without issue, there is a need to improve the accuracy of the analysis. When overly-conservative assumptions are made, structural analyses may find the existing structure inadequate and in need of reinforcement. This leads to a loss of the historic building fabric, additional cost for the client and excessive use of natural resources. By using more accurate wood properties in structural evaluations, unnecessary interventions may be avoided.

While wood can be qualitatively assessed through visual inspection in the field, it is difficult to accurately evaluate the material properties without destructive laboratory testing. Destructive testing of wood requires a statistical analysis and samples from an existing building. Numerous structural members would need to be destroyed in order to accurately determine the strength of the wood. This is a significant loss of historic building fabric, which is antithetical to heritage conservation. Such an extensive laboratory

testing campaign would also incur a significant cost, which cannot be justified in most conservation projects. Non-Destructive Testing (NDT) is a favorable method for assessing wood structures as it does not result in a loss of historic building fabric and can be done on-site at a much lower cost than destructive testing. However, there are limited standards on the evaluation of material properties through in-situ testing and the results are often less accurate than destructive testing.

One NDT tool often used to evaluate historic wood structures is the resistance drill. A resistance drill is an instrument used to locate deterioration in a wood section by indirectly measuring the relative strength of wood. The instrument has a drill bit roughly 1.5 mm to 3 mm in diameter and is generally 30-45 cm long (Rinn et al., 1996; Tannert et al., 2014). The resistance drill measures the amount of relative torque required to advance the drill bit through the wood member at a constant speed and rotation. The end of the drill bit is slightly flared to concentrate the forces at the tip to ensure that the machine measures the relative strength only at the tip of the drill bit. The resistance drill plots the relative resistance encountered on a graph as a function of drilling depth. Low areas on the graph indicate weaker material, generally due to decay, deterioration, or defects. Some resistance drills can even detect the growth rings and pith of a timber section. The resistance drill is predominantly used for qualitative assessment of historic wood, since the drill only plots the relative strength of the section. It is widely used in the field of timber conservation to locate and evaluate the extent of deterioration, such as decay or insect damage, in a wood member.

Resistance drills are used in the field to qualitatively assess structural timber. While the instrument can detect voids or areas of relative weakness within a cross section, it cannot directly determine the material's strength properties. However, research has shown that it is possible to correlate resistance drill measurements to wood density through laboratory evaluation. The material properties of wood are closely linked to density; these properties also depend on other variables, such as species, moisture content, grade

defects, and condition. European studies have set a precedent for comparing resistance drill measurements to wood density, and a few studies have extended this correlation to compressive strength and modulus of elasticity. However, to date there are no experimental campaigns of this type pertaining to Canadian species.

1.2 Problem Statement

This thesis aims to establish a correlation between resistance drill measurement and the mechanical properties of Eastern White Pine (*pinus strobus*). This study looks at specific gravity, compressive strength, bending strength and Modulus of Elasticity (MoE). While previous testing campaigns have explored the correlation between resistance drill measurement and density, there is a lack of information on Canadian softwood lumber. Furthermore, these studies do not extensively evaluate the influence of moisture content on resistance drill measurement or bending behavior. A case study and iterative structural analysis are included to demonstrate how these findings could be applied to assess and evaluate historic wood buildings.

The correlation developed between resistance drill measurement and the material properties of wood can be used in the field of heritage conservation to improve the accuracy of evaluations of historic wood structures. Increased accuracy in the evaluation of historic wood buildings may result in fewer interventions to the structure, allowing for a minimum intervention approach (Parks Canada, 2010).

The intent of this study is to improve upon current assessment methods for historic wood buildings. By understanding a historic building, it is possible to implement a conservation plan that minimizes the use of new construction, thereby preventing the loss of historic building fabric due to unnecessary structural interventions. In addition to preserving historic structures, accurate analysis of historic wood buildings can enable

a sustainable and economic approach to heritage conservation by minimizing embodied energy and construction costs.

1.3 Overview

Chapter 2 contains the review of the academic literature relevant to this topic. The literature review looks at the physical properties which affect wood strength; deterioration mechanisms in historic timber structures; the history and evolution of North American design codes and the basis of wood design values; destructive testing methods for experimentally determined wood properties; material characterization through Non-Destructive Testing (NDT) and previously established correlations between resistance drill measurements and mechanical properties.

Chapter 3 outlines the methodology for the experimental campaign carried out in the Carleton University Structures Lab. This section presents an overview of the materials, equipment, and testing procedures for the experimental research.

Chapter 4 discusses the results of the experimental campaign and analyses the correlation between the experimental and theoretical material properties. This chapter also discusses the implications of the findings.

Chapter 5 presents a case study of the methodology used to evaluate a historic wood framed barn. This methodology includes condition assessment and material characterization according to the relationships determined in the previous chapter.

Chapter 6 presents the final conclusions, addresses the problem statement and highlights areas of further research.

2. Literature Review

This research aims to determine whether the strength of historic wood can be accurately estimated using on site resistance drill measurement. This literature review outlines previous research on wood characteristics and their effect on strength; deterioration mechanisms and current practice for on site assessment are also summarized. An overview of the evolution of North American wood design codes and strength values have been researched. The destructive test methods which form the basis of wood strength values found in modern codes are summarized. Non-Destructive methods of material characterization for in-situ assessment are presented and previous studies on the correlation between non-destructive testing methods and wood strength have been included in this review. Reliability analysis principles have also been summarized in this review.

2.1 Physical Characteristics of Wood

Established relations exist between the physical characteristics and mechanical properties of wood. The physical characteristics with the most impact on material properties include species, grading defects, density, moisture content, and condition. The influence of each of these factors on material properties is described in the following sections.

2.1.1 Species

More than a hundred species of softwood trees are grown throughout Canada. Canadian climate, soil conditions, and weather patterns vary significantly across the country; unique forest regions are acclimated to different geographical regions. Each forest region is characterized by a distinct group of species which are acclimated to the regional environmental conditions, see fig. 2.1.

Figure 2.1: Canadian Tree Species Map (Canadian Forest Service, 2019).

Each species of tree has distinct anatomical features (Hoadley, 1990). This contributes to the material properties of the wood, such as axial strength, Modulus of Rupture (MoR), and Modulus of Elasticity (MoE).

Prior to the existence of design codes, carpenters were able to identify which species were suitable for applications such as framing, furniture making, or boat building. As early as 300 B.C.E the Greek biologist Theophrastus, Aristotle's successor in the Peripatetic school, noted on the different qualities of trees in the Mediterranean region. In volume five of his *Enquiry Into Plants*, titled *Of the Timber of Various Trees and its Uses*, he recognizes the effect of climate and geography on timber quality (Theophrastus, 1916).

Theophrastus recognizes the unique material properties of different species of wood and identifies which species are best suited for various practical uses.

While Theophrastus' remarks on the material properties of Mediterranean trees were qualitative, wood properties have since been quantitatively evaluated. Modern wood design codes group wood species with similar properties into species combinations. The CSA O86 wood species combinations are shown in table 2.1.

Table 2.1: CSA O86 Species Combinations

Species Combination	Species Included in Combination
Douglas Fir-Larch	Douglas Fir, Western Larch
Hem-Fir	Pacific Coast Hemlock, Amabilis Fir
Spruce-Pine-Fir	All Species of Spruce (except Coast Sitka Spruce), Jack Pine, Lodgepole Pine, Balsam Fir, Alpine Fir
Northern	Western Red Cedar, Red Pine, Ponderosa Pine, Eastern White Pine, Western White Pine, Trembling Aspen, Largetooth Aspen, Balsam Poplar

As a naturally grown material, wood material properties are highly variable, even for a single species. The design values presented for each species group in CSA O86 are based on the fifth percentile values for the weakest species in the combination (Canadian Wood Council, 2014).

ASTM D2555 “Standard Practice for Establishing Clear Wood Strength Values” presents material properties for commercially significant North American species of wood (ASTM International, 2017a). This standard provides average values and standard deviations for: MoR, MoE, compression parallel to grain, shear strength, compression perpendicular to grain and specific gravity. The strength values in ASTM D2555 are based on destructive testing of small defect-free unseasoned specimens.

While each species of wood has unique mechanical properties, these mechanical properties are generally proportional to the specific gravity (Berg et al., 1895). Some properties, such as modulus of elasticity and compressive strength are not always proportional to each

other. For instance, jack pine and eastern white pine have a similar flexural capacity, yet jack pine has a higher compressive strength (ASTM International, 2017a). Additionally, the species within each genus (e.g. spruce, pine, fir, cedar) of wood have unique properties, thus if species are to be grouped for structural purposes, they must be grouped based on their material properties and not based solely on the genus.

2.1.2 Grading Defects

Structural timber has variable mechanical properties due to random growth characteristics (Kretschmann, 2010a). These growth characteristics are responsible for the discrepancy in material properties between structural lumber and clear timber. Within one species, the strength of lumber can vary significantly due to growth characteristics, or grading defects (Wood, 1954). Lumber is grouped into grade categories based on observable grading defects (American National Standards Institute, 2018; Canadian Standards Association & Canadian Wood Council, 2015). This process, called visual grading, creates groups of lumber with similar strength properties, which allows for structural efficiency and economy in use. Typical grading defects specified by the National Lumber Grading Authority (NLGA) and their effect on wood strength properties are described in table 2.2 (National Lumber Grading Association, 2017; Southern Pine Inspection Bureau, 2015). The defects with the most influence on lumber strength are knots, slope of grain, and splits (R. W. Anthony et al., 2009; Cruz et al., 2015; Piazza & Riggio, 2008).

Knots have the most influence on wood strength (Berg et al., 1895). Tree branches grow perpendicular to the trunk, extending from the pith to the outer edge of the bark. When they fall off, the internal portion of the branch becomes encapsulated by new growth, see fig. 2.2. These defects have varying effect on material properties depending on their relative size and location, as well as the structural function of the member (Cruz et al., 2015). For instance, a knot in a compression member will have little impact on the

Table 2.2: Typical Wood Growth Characteristics and Effect on Strength

Characteristic	Description	Effect on Strength
Checks	Separation of wood across or through growth rings	Minor reduction of longitudinal shear strength
Knots	Encapsulated portion of branch, may be loose or tight	Reduction of tensile and bending strength
Pitch	Accumulation of resin in wood	Minor loss of strength due to cross-section reduction
Shake	Lengthwise fiber separation along the grain	Minor reduction of longitudinal shear strength
Slope of Grain	Angle between wood fibers and member neutral axis	Reduction of tensile and shear strength
Splits	Separation through a piece of wood to opposite or adjacent surface	Reduction of longitudinal shear strength
Stain	Variation from natural colour	No effect on strength
Unsound Wood	Deterioration due to rot	Minor loss of strength due to cross-section reduction
Wane	Missing wood at edges of lumber	Minor loss of strength due to cross-section reduction
Warp	Deviation from plane surface	May cause stress concentrations

compression resistance. Comparatively a knot in the tension side of a bending member reduces the net section thereby reducing the load carrying capacity.

The location of a knot in a wood member will also determine how it will affect the ultimate capacity of the member (R. W. Anthony et al., 2009; Piazza & Riggio, 2008). A knot is more likely to limit a member's load-carrying capacity if it is located in an area subjected to high stress. The size of a knot relative to the member's cross-section will also determine the effect on strength (Wood, 1954). Knots reduce wood strength by three mechanisms: localized grain distortion; reducing the effective cross section of a member; and creating stress concentrations due to material irregularity (R. W. Anthony et al., 2009). Closely-spaced knots, or knot clusters, also have the potential to cause further strength reductions (Cramer et al., 1996).

Figure 2.2: Localized Grain Distortion Due to Knot

Sloped grain occurs when logs are milled at an angle to the lumber grain, or if the grain is twisted due to spiral growth (Wood, 1954). This causes the grain to be angled from the member's axis, which reduces the strength of the piece (Kretschmann, 2010b). Uneven stresses may be induced due to grain slope, as these pieces are prone to shrinkage and warping with changing moisture content. If the fibers are sloped in a tension member, tension perpendicular to grain can be induced, which is the weakest failure mechanism for wood. Grain slope has the most prominent effect on tension and shear resistance with little effect on compression strength. This defect is most critical in tension members or near beam supports (R. W. Anthony et al., 2009; Cruz et al., 2015).

Checks are separations in the wood fibers along the grain which do not extend completely through a member, see fig. 2.3. On the other hand, splits are separations in the wood fibers that do extend completely through a member (R. W. Anthony et al., 2009). Splits may reduce the longitudinal shear strength if they measure more than four times the depth of a member (National Lumber Grading Association, 2017). Shakes are lengthwise separations through or along growth rings and behave similarly to checks, see fig. 2.4 (Kretschmann, 2010b). Splits are critical to the load-carrying capacity of beams at locations of high shear stress, generally near supports (Cruz et al., 2015). Checks,

shakes and splits do not reduce the cross-section of a member and have no effect on the compressive resistance of short columns not subject to buckling (Wood, 1954).

Figure 2.3: Seasoning Checks

Figure 2.4: Shakes in End Grain

The strengths of full-scale structural members and small clear specimens have been extensively studied over the course of the twentieth century (ASTM International, 2006). While strength values for clear wood are presented for North American species in ASTM D2555, this standard does not account for irregularities due to growth characteristics (ASTM International, 2017a). ASTM D245 “Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber” provides strength ratios to calculate the strength of a structural member based on the clear strength of the species and observable defects in the specimen. The strength ratios provided in ASTM D245 include the following:

- Knots in bending members
- Knots in compression members
- Slope of grain in bending and tension members
- Slope of grain for compression parallel to grain
- Shakes, checks and splits in bending members
- Specific gravity

In addition to these strength ratios, ASTM D245 relates the tensile strength of lumber to bending strength ratio based on experimental results. The modulus of elasticity is related to the bending strength ratio by a quality factor (ASTM International, 2006). The standard also allows for an increase in strength properties based on the grain density of a piece of lumber.

2.1.3 Density

Density is one of the key indicators of wood strength (Berg et al., 1895; Kretschmann, 2010a; Piazza & Riggio, 2008). Wood is composed of long hollow cells which transport sap and water throughout the trunk. These hollow cell cavities and pores reduce the density of a piece of wood. Variation in the size of openings and cell wall thickness determine the amount of wood substance in one piece of wood (Kretschmann, 2010a). High density wood has more wood substance per unit volume, and therefore more strength (Roth & Fernow, 1895). Mechanical properties within a species have been found to have a linear relation with density, or specific gravity, which is the ratio between the densities of wood and water (Kretschmann, 2010a). Density is calculated as the mass divided by the volume, as given by eq. (2.1).

$$\rho = \frac{m}{V} \quad (2.1)$$

Where:

ρ = specimen density

m = specimen mass

V = specimen volume

Ring Density (RD) determines how close the cells are to one another and therefore influences the density and mechanical strength of a piece of wood (Kretschmann, 2010a).

RD is based on growth rate; close growth rings indicate higher density and strength. According to ASTM D245 and the NLGA grading specifications, the growth rate of wood may be measured based on the number of annual rings per radial inch (ASTM International, 2006; National Lumber Grading Association, 2017). Increased strength ratios based on growth ring density are permitted for wood conforming to the specific gravity classification requirements as shown in table 2.3.

Table 2.3: Specific Gravity Classification and Strength Ratios

Specific Gravity	Specifications	Strength Ratio (%)
Dense Grain	Average of 6 or more growth rings per inch and at least one third latewood	117
Close Grain	Average of 6 or more growth rings per inch	107
Medium Grain	Average of 4 or more growth rings per inch	100

In addition to anatomical features, moisture content can affect the density of wood (Hunt, 2012). Wood is a hygroscopic material; changing moisture content affects both the weight and the volume of wood samples (Piazza & Riggio, 2008). Since density is defined as the mass per unit volume, the moisture content at which the density is measured must be specified. Specific gravity calculations are generally based on oven-dry mass, however the volume may be measured at any known MC. Density is based on the mass and volume at the same MC. Standard Moisture Contents for density measurement are oven-dry, green, or 12% (Glass & Zelinka, 2010).

2.1.4 Moisture Content

Moisture Content not only affects the specific gravity of wood; it also causes a well-established reduction in strength (Berg et al., 1895; Green & Evans, 2001; Kennedy, 1965; Roth & Fernow, 1895). In addition to causing wood to expand, thereby reducing the

wood fiber density, water softens wood cells (Kretschmann, 2010a; Roth & Fernow, 1895). The effect of shrinkage or expansion due to moisture can be accounted for in the specific gravity measurement, however the strength reduction due to the weakened cells is more difficult to quantify; fig. 2.5 illustrates the impact of moisture content on wood strength properties.

Figure 2.5: Effect of Moisture Content of Wood Properties (Kretschmann, 2010a)

Moisture content has been found to affect wood strength properties differently. Tension resistance parallel to grain reaches a peak between 10% and 15% moisture content while other properties see a decrease in strength with increasing moisture content. Furthermore, moisture content has little effect on tension and compression parallel to the grain (Kretschmann, 2010a). The intersection moisture content is the point at which material properties begin to change when wood is dried from green condition. This value is slightly lower than the Fibre Saturation Point (FSP) and varies with species; 25% to 30% may be assumed as a baseline intersection moisture content. The Wood Handbook describes the relationship between material properties by eq. (2.2) for clear wood (Davis, 2012; Kretschmann, 2010a). ASTM D2555 provides 12% MC to green ratios for the mechanical properties of clear wood.

$$P = P_{12} \left(\frac{P_{12}}{P_g} \right)^{\left(\frac{12-MC}{MC_P-12} \right)} \quad (2.2)$$

Where:

MC = the moisture content

P = the given material property at MC

P_{12} = the given material property at 12% MC

P_g = the given material property at the FSP

MC_P = the intersection moisture content from the graph shown in fig. 2.5

While strength and stiffness increase as wood dries, this effect may be partially offset by seasoning or shrinkage defects for timbers more than 90 mm thick (ASTM International, 2006). Per ASTM D245, strength increases are permitted for lumber less than 90 mm thick when its moisture content is below 19%. Additionally, the ratio of dry to green mechanical properties varies with species, as detailed in ASTM D2555 (ASTM International, 2017a).

2.1.5 Weathering and Condition

Weathering is caused by exposure to water, ultraviolet radiation, and staining micro-organisms (Feist, 1983). This causes chemical and physical reactions on the surface of the wood, resulting in a change of colour and degradation of the lignin (Davis, 2012). Weathering only penetrates about 2 mm into the wood, the material properties beneath the outer layer are not affected by weathering. Outdoor wood weathers faster due to the increased sunlight exposure and frequent changes in moisture content. Raised grain and surface checks due to weathering cause the surface to become rough.

Aging has not been found to affect the strength of wood (Wood, 1954). A 1975 study on historic wood aged between 900 and 4400 years found that while weathering affects

the surface of wood, age has negligible effect on the strength of wood in good condition (Borgin et al., 1975).

Other deterioration mechanisms can result in a loss of timber strength, as detailed in section 2.2. In addition to its natural characteristics, the condition of timber in existing structures influences the strength of the member. Condition must be considered in the preservation of existing timber buildings (Watt, 2010). Existing timber buildings must be evaluated through on site condition assessment to evaluate their structural behavior (Cruz et al., 2015).

2.2 Condition Assessment of Historic Timber

As an organic material, wood is susceptible to damages and decay which significantly reduce its structural capacity. An understanding of the pathological conditions present in a timber structure is a crucial basis for preservation decisions (Parks Canada, 2010). A condition assessment is a qualitative investigation of the structure to locate symptoms which may pose a threat to the structure (Watt, 2010). A condition assessment should make note of the varying levels of damage observed in a structure and determine the impact of the condition on the building's longevity. This allows for urgency-based planning of interventions.

On site condition assessment is a key component to understanding the behavior of historic timber structures. Observations must consider both the condition of the materials and the mechanical performance of the structure (Riggio et al., 2018). Historic timber structures are susceptible to decay, insect damage, biological growth, and moisture damage (Cruz et al., 2015; Mosoarca & Gioncu, 2013). These defects cause a reduction in the strength of timber; it is therefore crucial to observe and assess the extent of damage where it occurs. Mechanical defects in a timber structure are often signs of structural inadequacy. Mechanical conditions seen in timber buildings include large deformations,

crushing, splitting and connection separation. These are indicative of overloading in the structure, which can eventually lead to failure or collapse.

While the condition assessment makes note of the observed symptoms of deterioration, some consideration must be given to the causes so that they may be addressed through the design of interventions. In addition to recording the material and mechanical conditions, the structural geometry and character defining elements should be recorded. Accurate, clear and concise documentation creates a reference for future inspections and a record of the building at a particular moment in time. Building documentation creates a baseline for changing conditions over time and can be used as a basis for structural analysis (Parks Canada, 2010).

Documentation is a critical component of the early stages of a conservation project. Photographs, notes, sketches and drawings should be used during the condition assessment to record the location and extent of defects and decay (ICOMOS, 2003). In general, the change in conditions over time is a more significant representation of the pathological issues in a building than the static state. If an observed condition does not worsen over time, it may be stable; the underlying cause of the damage may have been previously addressed through prior intervention (Cruz et al., 2015).

2.2.1 Material Conditions

The material condition of timber can be assessed through visual inspection. Typical deterioration mechanisms in timber structures include decay, insect attack, biological growth, excess moisture and inappropriate prior interventions (Watt, 2010). These defects can severely reduce the strength of wood. Any material conditions observed on site must be considered in the preservation plan for the structure.

Material damage reduces the cross-section of a wood member, thereby reducing its load-carrying capacity. In order to preserve historic wood buildings, it is crucial to not only identify the symptoms of deterioration, but also to diagnose their underlying causes (Watt,

2010). For instance, damage such as moisture staining does not significantly affect the structural capacity of a member; but moisture stains indicate excessive water infiltration. This can lead to other forms of damage which weaken the structure. Therefore, it is more important to address the cause of the staining than the staining itself. Addressing the root cause, rather than the symptoms, ensures that the pathological issues seen in a historic structure do not reoccur.

Wood decay is caused primarily by three types of fungus: soft rot, brown rot and white rot (Downer & Perry, 2019). While all three break down the cellular structure of the wood, brown rot decay occurs at faster rate than decay by white rot or soft rot (Harris, 2001). Brown rot can also diffuse rapidly within a piece of wood, whereas white and soft rots cause local decay. Brown rot is distinct for the cubical pattern of decay, which causes wood to become dry and crumbly, see fig. 2.7 (Wikimedia Commons, 2005). White rot breaks down the lignin which holds the wood fibers together and causes wood to lose its colour and become moist and stringy, see fig. 2.6 (Porse, 2005). Soft rot breaks down cellulose and hemicellulose. Unlike white and brown rot, soft rot fungi grow mainly inside cell walls (Schmidt, 2006).

Figure 2.6: White Rot (Porse, 2005)

Figure 2.7: Brown Rot (Wikimedia Commons, 2005)

Dry rot is a subspecies of brown rot fungus. While it creates a similar deterioration patterns to typical brown rot, it appears dry to the touch (Harris, 2001). Dry rot fungi

do not appear to have a direct moisture source and can draw water from further away. It spreads much more rapidly than brown rot and requires urgent mitigation.

In addition to wood-decay fungi, surface fungi or mold can also grow on wood. These tend to cause staining, but do not attack the wood. While surface fungi can be damaging to human health, they do not require urgent intervention to preserve the structure. Fungi require favorable temperature, oxygen, moisture and a substrate in order to grow (Harris, 2001). Most types of fungus thrive at moisture contents above 20%. During a condition assessment, moisture content measurement can be used to determine if a wood structure is prone to fungal attack.

Insect infestation is another form of material damage seen in wood structures. In Canada, carpenter ants and termites are the most common wood-damaging insects. Both insects create voids within a piece of wood, however, they attack the wood differently. Carpenter ants create nests in wood, whereas termites consume wood. Both types of insect prefer moist wood. Carpenter ants create clean tunnels in the wood, and expel sawdust, called “frass” through holes in the wood. Termites however do not continue to live in the wood once it has been eaten. They often back-fill their tunnels with mud and create mud tunnels to their colonies.

2.2.2 Mechanical Conditions

While a structural analysis is generally required to fully assess the load-carrying capacity of a structure, many issues can be observed on site. Most historic structures were constructed based on empirical evidence, rather than engineering calculations or design codes. Although this differs from the modern approach to structural design, the quality of the materials and craftsmanship led to structures that were able to resist the applied loads at the time of construction. If the loading on a building has not changed and there have been no significant changes to the structure, it stands to reason that a building that has stood for a century will continue to perform adequately (Rinn, 2019). However,

Figure 2.8: Carpenter ant damage (Fritsche, 2005)

Figure 2.9: Termite damage (Pinnacle Solutions, 2019)

with increased snow and wind loads due to climate change, past building interventions, changes in use, and material degradation, current load paths may differ from the original construction. Even if the loading on a structure has not significantly changed, a historic structure should be visually assessed for signs of mechanical failure. Typical indicators of structural inadequacy are large deflections in timber members, signs of splitting, or crushing of wood fibres. Existing buildings may not have been subjected to seismic loading; structural assessment may be required to determine whether upgrades to the seismic reinforcing system are necessary.

Deformations observed on site usually indicate structural inadequacy. Large deflections are typically visible in timber frame structures, but these can be difficult to measure on site. Laser scanning technology may be used to obtain the geometry of a structure and measure deformations observed on site. It is important not only to observe and quantify deformations, but to understand their causes and effects (Milch et al., 2016). For instance, removal of structural elements, increasing the live load, or insulating the roof of a building could all result in excessive deformations. Despite not being constructed to any design code, it is unlikely that the original construction would intend for severe deflections.

Deformations are not always a structural concern; in some cases previous loading can cause plastic deformation. For instance, in the case of industrial or agricultural buildings with large live loads, structural members may have undergone plastic deformation due to creep. Even after the loading is removed due to a change in use case, the structure retains plastic deformation. While this does not indicate that the structure is currently overloaded, plastic deformation would likely cause damage to the member. An understanding of the building's use case history and past interventions can help interpret the mechanical damages observed on site. Observable deformations can be verified through structural analysis. If the results obtained through structural analysis are inconsistent with observations made on site, it is possible that the deformations are caused by previous loading. Defects, such as checks, cracks and warping can be observed in

overloaded members. These may indicate poor service conditions or structural inadequacy and require careful assessment to determine whether they need to be addressed.

Consistent structural health monitoring of the change in conditions over time is more representative of structural issues than a one-time inspection (Mosoarca & Gioncu, 2013). Long-term structural health monitoring can be used to determine if conditions are worsening over time. Changes in building condition are more indicative of issues than the conditions themselves. If the mechanical damage is stable, it is likely caused by a prior condition. However, if the mechanical conditions worsen over time, there is an ongoing structural concern that must be addressed.

2.3 History and Development of Wood Design Codes

2.3.1 Early Investigations of Wood Properties

Throughout history, structural design was governed by empirical standards based on the success or failure of previous buildings. Rules of thumb for estimating beam and column sizes based on various spans and heights were handed down through generations (Booth, 1964). As early as ancient Greece, Theophrastus (372-287 BCE), Aristotle's successor in the Peripatetic school, wrote on the properties of wood. Volume five of *Enquiry into Plants*, titled *Of the Timber of Various Trees and its Uses*, is the earliest documented description on the relative strengths of wood. In this volume, Theophrastus recognizes the different properties of different species of wood which make them suitable for applications such as carpentry, ship-building or house-building (Theophrastus, 1916). He describes the properties of wood in bending, bearing and compression, as well resistance to decay. Theophrastus also acknowledges the effect of knots, slope of grain, and density on the strength and quality of wood. In fact, the observations made by Theophrastus are reflected

in current wood design codes, which provide design values based on species, observed defects, and service conditions.

The first publications on the strength of materials appeared in the 17th century, thanks to the father of modern science, Galileo Galilei. Galileo developed a tensile test for wood, which is the first recorded effort to quantify this material property. Throughout the 17th and early 18th centuries, scientists such as Mariotte, Parent, Coulomb and Hooke built upon Galileo's work and further investigated elasticity, stress distribution, bending behavior, and support conditions for wood beams. Between 1729 and 1762, Dutch scientist Pieter van Musschenbroek developed a tensile test machine and investigated the strength properties of different parts of the tree. He also investigated the effect of species, moisture content and density on wood strength (Booth, 1964). Later, Gianluigi Buffon observed the effects of load duration and load-deflection behavior by testing large beams. Buffon found that the behavior of large wood specimens cannot be accurately predicted by testing small specimens due to the influence of defects on strength. A test machine for large bending and compression members was finally developed in 1798 by Pierre-Simon Girard. However, despite these early investigations into the properties of wood, structural design was still governed by craft experience up until the late nineteenth century (Green & Evans, 2001).

2.3.2 Standardization of Wood Strength Testing

Before the industrial revolution in the nineteenth century, builders were not able to quantify material properties of the products used in buildings; they simply relied on their own prior knowledge (ASTM International, 1998). With the industrial revolution came the development of new materials, such as steel, which was widely used in the railway and steamship industries. Unlike wood, builders could not rely on prior knowledge or traditional methods for steel construction. New materials required new expertise. Due to a lack of manufacturing standardization, many North American steel suppliers produced

inconsistent materials. In fact, some rail companies imported British steel, as it was much more reliable. This led to the formation of the International Association for Testing Materials (IATM) in Munich in 1895 (Green & Evans, 2001).

North American steel companies began performing tensile tests to ensure quality control and to stay competitive. In 1898 the American Chapter of IATM met in Philadelphia to form the first technical committee: “A-1 On Steel, Stainless Steel, and Related Alloys”. This group later became incorporated as the American Society for Testing Materials (ASTM) in 1902 (ASTM International, 1998). In 1904, Committee Q on “Standard Specifications for the Grading of Structural Timber,” which later became Committee D7 on Wood, was formed.

Prior to the establishment of the ASTM wood committee, lumber grading, testing, and derivation of material properties were based on proprietary procedures. These procedures were created by consulting engineers, wood scientists, and academics and were not standardized throughout the construction industry (Green et al., 2004). The first grading and material property assignment procedures were reported in 1895 by the American International Association of Railway Superintendents of Bridges and Buildings. This report outlined fifteen recommendations for timber bridge and trestle design, as well as allowable stresses for the main species used in construction (Berg et al., 1895). The report also provided conversion factors to establish design strengths based on laboratory testing, based on small and full scale studies dating from 1880 to 1895. In the same year, a bulletin was published by the United States Department of Agriculture (USDA) Forest Product Service (FPS) outlining the mechanical properties of wood (Roth & Fernow, 1895). This study presented the experimentally obtained average strength values for stiffness, bending, tension, compression, and shear for different species groups (Roth & Fernow, 1895). Additionally, this Forest Product Bulletin notes the influence of density and moisture content on the strength of timber.

Through the late nineteenth century and the turn of the twentieth century, several testing campaigns were established to determine the strength properties of the most common species of tree used in North American construction. This forestry program was developed under Bernard E. Fernow, chief of the USDA Division of Forestry from 1886 to 1898. Leading wood experts at that time were divided into two schools of thought. One group believed that full-scale structural members should be tested to develop realistic strength values to be used by engineers and architects. On the other hand, others saw this approach as impractical and expensive; they felt that the testing program should determine average strength values for various species by testing small specimens. As a result, the testing program incorporated both ideologies (Green & Evans, 2001).

Both types of timber testing continued throughout the first decade of the twentieth century. W. K. Hatt reported on the Bureau of Forestry's timber test program in 1904 (Hatt, 1904). This formed the basis for the establishment of ASTM Committee Q on "Standard Specifications for the grading of Structural Lumber." By 1910, significant data had been gathered from both small clear specimens and full scale timbers, however these findings were not consolidated until 1912 in FPS Bulletin No. 108 (R. W. Anthony et al., 2009). This article presented a summary of the full-scale testing program. Bending, compression and bearing strengths were evaluated based on their material characteristic; this became the basis for grading regulations. Visual grading of timber defects, including knots, splits, checks, slope of grain, shake, wane and warp, was established as a means to correlate small scale and large scale testing results and unite the two schools of thought.

The Central Committee on Lumber Standards, now the American Lumber Standard Committee (ALSC), was established in 1922 to develop lumber standards for grades, naming, forms, and molding. The committee worked closely with ASTM D7 who published Standard Test Methods for Small Clear Specimens of Timber (ASTM D143) which was ratified in 1927 (ASTM International, 2014). This standard outlined methods for sampling and mechanical testing based on species. At the same time, ASTM published Standard

Test Methods of Static Tests of Lumber in Structural Sizes (ASTM D198), which related to the testing of full-sized structural timbers (ASTM International, 2015). In 1926 Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber (ASTM D245) was adopted (ASTM International, 2006). Standard ASTM D245 addressed the longstanding need for allowable properties based on lumber species and quality and created coherence between the large scale and clear sample testing (Green et al., 2004). It was not until 1966 that ASTM published Standard Practice for Establishing Clear Wood Strength Values (ASTM D2555) which presented the use of a fifth percentile exclusion value (ASTM International, 2017a). ASTM D2555 was the first use of the fifth percentile exclusion value for determining allowable properties for timber design.

2.3.3 Establishment of Wood Design Codes in Canada

Under the constitution act in 1867, Canadian provinces had jurisdiction over their own building codes. Most wealthy land owners saw this as a government imposition and thus for decades there was no desire for the provinces to develop codes. Around the turn of the twentieth century, accidental fires and disease became more and more widespread; major Canadian cities, such as Halifax, Montreal, and Toronto, developed their own building codes to protect property and prevent human injury. By the 1930s, there were numerous contradictory municipal building codes in circulation. With the great depression came a rise in homelessness and the development of federal housing programs in 1937 (Archer, 2003). The lack of consistent codes resulted in structural failures. The National Research Council of Canada (NRC) saw the need for a unified code and brought on engineers, architects and building scientists to create the first National Building Code (NBC). Based on American Building Codes, the first edition of the NBC was published in 1941. By the 1960s, some provinces had developed their own codes based on the NBC. Objective-based codes and the coordinated national model code development system were created to

harmonize the Canadian building regulatory environment and maintain congruence over the long term.

The Canadian Engineering Standard Association (CESA) was established in 1919 during the First World War as a means to standardize resources between Allied Nations. While this committee was established upon request from the British government, the organization was independent and recognized Canadian weather conditions and the country's natural diversity and expansiveness. The CESA was created by volunteers who sought to establish Canada as an industrialized nation. Similar to ASTM, the first CESA standards were for steel railway bridges, which required manufacturing standards for the innovative new material (CSA Group, 2009).

The CESA continued to develop new standards for aircraft, radio and radar throughout the Second World War. Also in the 1940s, the CESA changed the organization's name to Canadian Standards Association (CSA) to reflect the wide range of industry standards being produced. The 1950s "Baby Boom" saw an increasing demand for housing, thereby creating a need for construction material standards. The Canadian Lumber Standards Bureau (CLSB) was established to create a national lumber grading system. This would ensure quality in houses constructed under both the Canadian Mortgage and Housing Corporation (CMHC) and the United States Federal Housing Authority (USFHA) (Canadian Lumber Standards Accreditation Board, n.d.).

The CSA published the first edition of Engineering Design in Wood (CSA O86) in 1959 to standardize wood construction across the country. This code was developed using Working Stress Design (WSD) methodology and was updated every five years (Canadian Wood Council, 2020). The first Limit States Design (LSD) version of the code was published in 1984; with the publication of CSA O86-94, the WSD version of the code was withdrawn.

The early design values presented in CSA O86 were obtained from small clear samples of wood based on ASTM D2555 (ASTM International, 2017a). ASTM D245 allowed for

these design values to be modified for full-size members and accounted defects observed through visual grading (ASTM International, 2006). In 1980 the CSA O86 adopted design values from in-grade testing for tension design. In-grade values were used for bending in 1984 and for compression parallel to grain in 1989. The in-grade Canadian Wood Council (CWC) testing program followed ASTM D4761 (Canadian Wood Council, 2014). Some properties such as compression perpendicular to grain, specific gravity, and horizontal shear are still based on small clear specimens, since these properties are not severely impacted by wood defects.

The 1984 Limit States Design (LSD) code was based on the CWC Full-Size Lumber Properties Program carried out in the late 1970s on behalf of the NLGA. Testing was carried out as per ASTM D1990 “Standard Practice for Establishing Allowable Properties for Visually-Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens” and ASTM D4761 “Standard Test Methods for Mechanical Properties of Lumber and Wood-Based Structural Materials” to determine the mechanical properties of wood in structural sizes (Canadian Wood Council, 2014; Jessome, 1977; Kennedy, 1965). CSA O86 provides design values for three classes of visually graded lumber: dimension lumber, beams and stringers, and post and timbers. The NLGA further breaks down dimension lumber into four categories: structural light framing, structural joists and planks, light framing, and studs.

The design values provided for dimensional lumber are based on in-grade testing of lumber in three sizes (38 mm x 89 mm, 38 mm x 184 mm, and 38 mm x 235 mm) from different growing regions across Canada. A representative sample was chosen for each grade category of the three major species groups: Spruce-Pine-Fir (S-P-F), Douglas Fir-Larch (D Fir-L) and Hemlock-Fir (Hem-Fir). The strengths for bending, tension, shear, compression parallel to grain, and MoE were derived based on reliability principles, using a two-parameter Weibull distribution to characterize material property data (Canadian

Wood Council, 2014). Design values for the Northern species category were based on the weakest species, to provide a basis for design using any Canadian species.

Design values for beams and stringers are still based primarily on the fifth percentile mechanical properties for small clear samples. The fifth percentile properties were modified according to ASTM D245 to approximate the strength of full-scale members with grading defects. However, limited testing of full-size beams has shown that values derived from small clear specimens are conservative (Canadian Wood Council, 2014). Thus the specified strengths were revised in 1989 based on full-scale testing campaigns. This allowed for an increase in the compression parallel to grain strength for beams and stringers and the introduction of size factors. Similarly, the CSA O86 design values for posts and timbers are based on the fifth percentile properties of small clear specimens and modified based on ASTM D245.

2.4 Destructive Material Characterization

Canadian design standards are largely based on ASTM standards for material testing. ASTM International is accredited by the Standards Council of Canada (SCC) and many ASTM standards have been adopted or referenced by federal and commercial agencies. CSA O86 references several ASTM standards for material properties. The primary testing procedures for destructive material characterization of wood used in Canada are:

- Standard Test Methods for Small Clear Specimens of Timber (ASTM D143) (ASTM International, 2014)
- Standard Test Methods of Static Tests of Lumber in Structural Sizes (ASTM D198) (ASTM International, 2015)
- Standard Test Methods for Mechanical Properties of Lumber and Wood-Based Structural Materials (ASTM D4761) (ASTM International, 2019)

Destructive testing is used to determine material characteristics. Since wood is a highly variable material, properties obtained through testing are determined using a statistical analysis. The material properties outlined in Engineering Design in Wood (CSA O86) are based on destructive testing campaigns from the 1970s (Canadian Wood Council, 2014). The strengths for bending, tension, shear, compression parallel to grain, and MoE were derived based on reliability principles, using a two-parameter Weibull distribution to characterize material property data (Canadian Wood Council, 2014). The design values presented in the code are based on a 5% exclusion limit. This exclusion limit predicts that 95% of lumber will meet or exceed the strength values presented in the code. Rather than using the conservative values from design codes, destructive testing can be used to determine the actual strength of a wood member.

In practical conservation projects, destructive testing of historic wood structures is not feasible. Destructive material characterization of wood requires access to laboratory testing equipment, which comes with a high cost, beyond the scope of most conservation projects. Additionally, testing procedures such as ASTM D143 and ASTM D198 would require removal and destruction of historic building components, which is antithetical to value-based heritage conservation (ASTM International, 2014, 2015). A statistical approach to material characterization would require several samples from a wood structure to validate the testing results. This may be practical for historic masonry structures, where a few stones can be removed from a wall and replaced, but for wood frame structures this would require the removal of significant portions of the structural system.

2.4.1 ASTM D143: Small Clear Specimens of Timber

Originally approved in 1922, the latest edition of Standard Test Methods for Small Clear Specimens of Timber (ASTM D143) was published in 2014 (ASTM International, 2014). This standard establishes the test methods to determine material properties from small, defect-free timber specimens (ASTM International, 2014). The test methods specified

in ASTM D143 are largely based on early tests conducted by the USDA Forest Service; recent editions reflect European testing campaigns. This standard outlines acceptable procedures to extensively determine timber properties. Standard methods for determining mechanical properties, such as bending, compression, shear, tensile, and bearing strength are provided. ASTM D143 also provides guidance for determining properties such as hardness, toughness, cleavage, shrinkage, specific gravity, and moisture content.

ASTM D143 allows for primary and secondary test methods for some strength tests. The specimens used in primary tests have a 50 mm by 50 mm (2in by 2in) cross-section while the secondary methods allow for a 25 mm by 25 mm (1 in by 1 in) section. The larger section is more representative of the sampled material and contains more growth rings. ASTM D143 allows for the smaller sections to be used for static bending and compression parallel to grain tests, only when larger clear sections of wood cannot be procured (ASTM International, 2014). Combined use of primary and secondary test methods should be avoided as the testing results are not directly comparable.

Moisture content and temperature have a significant influence on wood strength; these variables must be controlled during the testing. Additionally, the proportion of heartwood and sapwood in each specimen must be documented, as these tend to have slightly different strengths (ASTM International, 2014). This can be observed in the end grain of a specimen.

The strength properties obtained from ASTM D143 are based on small specimens without defects. This is not representative of structural timber, as defects present in large members reduce the ultimate capacity of the member. However this standard may be used in conjunction with ASTM D198 or ASTM D4761 to establish realistic allowable stress values for timbers in structural sizes. Alternatively, these values can be multiplied by the strength ratios provided in Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber (ASTM D245) to determine the strength of full-scale timber based on observable defects. ASTM D2555 summarizes the

average values and standard deviations obtained through ASTM D143 for commercially significant species in Canada and the United States.

2.4.2 ASTM D198: Static Tests of Lumber in Structural Sizes

ASTM D198 was published in 1924 as a response to ASTM D143 (Green & Evans, 2001) (ASTM International, 2015). The new standard addressed the discrepancy between mechanical properties obtained through small specimen testing and the structural performance of full sized timbers with defects (ASTM International, 2015). While not as extensive as ASTM D143, this standard provides allowable testing procedures to determine flexure, compression, tension, and torsion strengths as well as stiffness. With the development of engineered wood products such as glulam, I-joists and structural composite lumber, the standard has been adapted to include provisions for testing a larger range of wood products.

Unlike ASTM D143, ASTM D198 does not specify exact size requirements for specimens. Instead, the mechanical properties are given as a function the member geometry and applied load (ASTM International, 2015). The test method is highly accurate for determining mechanical properties of structural timber. However, the lack of standardization of specimen sizes limits compatibility between different studies.

Due to the broad allowances in the testing standard, proper documentation is crucial to allow the study to be reproduced. Standard ASTM D198 is intended to permit comparison of results from different studies (ASTM International, 2015). This is accomplished through uniformity of the procedures. Any deviations from the standardized method must be documented. Parameters such as the specimen dimensions, rate of loading, location of supports, deflection, and equipment type must be noted. Additionally, information such as wood species, origin, shape, wood condition, location of defects, and any other parameters that may affect strength must be reported. ASTM D198 is intended for studies where

accurate results are required, for instance the development of design values and scientific studies.

2.4.3 ASTM D4761: Mechanical Properties of Lumber and Wood-Based Structural Materials

While the standardized test methods provided in ASTM D198 allow for improved coherence between testing campaigns, Standard ASTM D4761 is more suited for on site testing (ASTM International, 2019). The standard test methods outlined in ASTM D4761 are based on ASTM D198, however they allow for testing outside the laboratory environment, which makes them more practical in the field. ASTM D4761 covers test methods for determining bending and axial strength of lumber and engineered wood products.

ASTM D4761 provides procedures that can be used in field conditions where environmental variables cannot be easily controlled. The procedures may be used to establish or confirm specified strength and stiffness values for a defined sample. They can also be used to evaluate the influence of environmental conditions and wood defects on strength and stiffness.

ASTM D4761 requires that all procedures be well documented as the standard allows for variability in the test methodology. As yet, the precision and bias of the test methods outlined in ASTM D4761 have not been established. These testing procedures are intended for field use, and therefore there is no great need for results to be easily comparable. If comprehensive data on a particular sample is needed, ASTM D198 or ASTM D143 should be used.

2.4.4 Summary

A summary of the applications and limitations of the the relevant ASTM standards for destructive testing is provided in table 2.4.

Table 2.4: Summary of ASTM Destructive Test Methods for Wood

Standard	Advantages	Limitations
ASTM D143	Easily comparable and accurate material properties	Not reflective of actual structural behavior
ASTM D198	Accurate material properties for structural timber	Not suitable for on site testing, not easily comparable between different studies
ASTM D4761	Material properties determined on site	Results not comparable, lowest accuracy

2.5 Non-Destructive Material Characterization

While material characterization is an important step in understanding a historic structure, destructive testing of an existing timber structure is not favorable in heritage conservation. While small samples may be removed from materials such as masonry or steel for destructive testing, this approach is not suitable for historic timber structures. Destructive testing of small samples is not reflective of the strength of full-scale timbers due to the effect of growth characteristics. Furthermore, timber is a variable material, therefore not all members will have the same strength, even if they belong to the same species or species category.

Due to the limitations of destructive testing with regards to existing structures, Non-Destructive Testing (NDT) methods are preferred in practice. NDT methods can be performed on site and are used to qualitatively or quantitatively determine the properties of wood without causing significant destruction. Typical NDT methods relevant in this study include visual grading, species identification, moisture content metering and resistance drill testing.

2.5.1 Visual Grading

Visual grading is one of the simplest methods of estimating the strength of timber in existing buildings (Piazza & Riggio, 2008; Riggio et al., 2014). The two factors considered

in visual strength grading are the species and the grading defects. Visual grading is used to provide preliminary strength data based on grade-limiting defects (Branco et al., 2017).

Wood species can be identified based on anatomical characteristics. Most of these identifiable features can be observed by the naked eye or with a hand lens, however there are limitations to these tools (Hoadley, 1990). Microscopic analysis may be used to confirm hand-lens results. Additionally, on-site species identification requires prior knowledge of the distinct anatomical features of wood and how they relate to species. Small samples may be removed from the structure for off-site laboratory analysis if the species cannot be identified on site (A. Feio & Machado, 2015). While the species group can often be estimated based on the geographic location of the structure, identifying the exact species used in the structure allows for a more accurate determination of its material properties.

The grading defects with the most influence on the strength and grade category are knots and slope of grain (R. W. Anthony et al., 2009; Cruz et al., 2015). These can be measured on site and compared to the National Lumber Grading Authority (NLGA) grading requirements to determine the grade category based on the thickness of the piece. Furthermore, the impact of a defect on strength depends on the member's structural function (Cruz et al., 2015). For new lumber, the end use is unknown so the most conservative grading requirements are applied based on member size. However, for existing structures the structural function and locations of high stress are known. The grading defects with the largest impact on strength for various structural functions are shown in table 2.5.

Once the grade and species categories for existing timber are known, design values from CSA O86 can be used in structural evaluation. The design values presented in the code are based on in-grade testing of full-size structural members (Canadian Wood

Table 2.5: Impact of Defects on Structural Function

Member Type	Grade-Limiting Defects
Bending Only	Knot clusters in areas with high moment Edge knots in tension side Slope of grain and splits in areas with high shear stresses
Compression Only	Large knots in small members without load-sharing Large knots in adjacent members in load-sharing system
Compression and Bending	Knots in areas with high moment Edge knots in tension side Tension side notches Cross-section reduction at connections
Tension Only	Checks and splits with slope of grain Loose knots Notches Cross-section reduction at connections
Tension and Bending	Checks and splits with slope of grain Knot clusters in areas with high moment Edge knots in tension side Slope of grain and splits in areas with high shear stresses Notches Cross-section reduction at connections

Council, 2014). These values are based on the weakest species in the combination and are founded on full-scale tests of modern rapid-grown lumber, which has a lower density than old growth timber particular to historic structures. This approach to structural evaluation is conservative; however, this is common practice for most conservation projects, especially for preliminary evaluation (R. W. Anthony et al., 2009).

While in-situ strength grading is a useful tool for predicting timber strength, there are limitations to this method. In some cases, the member may be partially concealed and therefore cannot be fully inspected on all sides. Additionally, visual grading guidelines are intended for new timber and do not take into account the conversion methods or the size effect for historic timber (Piazza & Riggio, 2008). Most methods of visual grading

are still based on fifth percentile values which underestimate the actual strength of timber (Cruz et al., 2015).

2.5.2 Species-Specific Design Values

Visual grading guidelines such as, the NLGA 2017 Standard Grading Rules For Canadian Lumber, group wood into species categories to facilitate grading (National Lumber Grading Association, 2017). However, if the species of the wood is known, the following ASTM standards may be used to obtain more accurate material properties for a particular species:

- Standard Practice for Establishing Clear Wood Strength Values (ASTM D2555) (ASTM International, 2017a)
- Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber (ASTM D245) (ASTM International, 2006)

ASTM D2555 provides material properties for commercially important North American species based on testing of small defect-free samples (ASTM International, 2017a). If the exact species is known, material properties from ASTM D2555 can be used. This standard presents the mean values and standard deviations from which the fifth percentile design values can be determined for compression and tension parallel to grain, shear, and bending strengths; average values are used for Modulus of Elasticity and compression perpendicular to grain. However, even when the fifth percentile strengths are used, these values still represent the clear strength of wood and must be modified according to the strength ratios in ASTM D245.

The strength ratios in ASTM D245 adjust the design strengths given in ASTM D2555 to represent the actual strength of a full-scale timber (ASTM International, 2006). The strength values must be modified based on observable defects, size factors, Moisture Content, and grain density. Adjustment factors for safety based on the use case are also included in ASTM D245.

Once the fifth percentile strengths from ASTM D2555 have been determined, these must be adjusted by dividing by the adjustment factors presented in Table 8 of ASTM D245 (ASTM International, 2006). These adjustment factors differ for softwoods and hardwoods and modify the design values based on a long term load duration, account for size effects, and include safety factors (R. Anthony & Nehil, 2018; ASTM International, 2006).

Tables 1 through 4 of ASTM D245 provide strength ratios for knots and slope of grain in timbers (ASTM International, 2006). These strength ratios are determined based on the defects observed in the member; the lowest strength ratio governs and is used to further adjust the design value. Since ASTM D2555 provides properties for unseasoned wood, the design values must be further adjusted for a lower Moisture Content (MC). Table 10 of ASTM D245 provides modification factors for wood with a maximum MC of 19% and 15% (ASTM International, 2006).

Additional factors may be applied to further modify the design values. For instance, as discussed previously, table 2.3 presents the strength factors which allow for an increase based on growth ring density as shown in Table 6 of ASTM D245 (ASTM International, 2006). Additional size adjustment factors are included for lumber more than 51 mm in depth. Factors for treated wood and small bearing areas may also be used to adjust the design values as necessary.

2.5.3 Moisture Content Metering

Moisture content measurement is used in the field to evaluate the level of moisture present in a wood member (Kloiber et al., 2016). The moisture content influences the material properties of the wood, such as shrinkage, density and strength. The presence of moisture in a timber element also increases the risk of decay and insect attack. Moisture content is expressed as a percentage and represents the amount of water in a piece of wood relative

to the dry weight of the wood. Moisture content in wood can be measured by direct or indirect methods.

The most accurate method of measuring moisture content is by direct measurement, as is detailed in Standard Test Methods for Direct Moisture Content Measurement of Wood and Wood-Based Materials (ASTM D4442) (ASTM International, 2020). The weight of the water in the specimen is measured and divided by the dry weight of the specimen to obtain the moisture content percentage. The weight of the water is measured by subtracting the oven-dry mass from the original mass of the specimen, as shown in eq. (2.3).

$$MC = \frac{M_{original} - M_{ovendry}}{M_{ovendry}} \times 100\% \quad (2.3)$$

Where:

MC = Moisture content

$M_{original}$ = Original mass of the specimen

$M_{ovendry}$ = Oven-dry mass of the specimen

Indirect moisture content measurement using a moisture meter is common practice in the field. Moisture meters must be calibrated based on the species, temperature and thickness of the wood to obtain accurate results (Cruz et al., 2015; Piazza & Riggio, 2008). Moisture meters are a quantitative tool used to determine the amount of moisture in wood, relative to the fibre saturation point. This in situ testing method is helpful for predicting the likelihood of decay occurring in a timber. In addition to reducing timber strength, decay can occur in wood with a moisture content above the Fibre Saturation Point (FSP), typically 25% to 30%. At this moisture content, wood is also prone to insect damage, such as carpenter ants or termites.

2.5.4 On-Site Resistance Drilling

Due to the limitations of on-site visual grading of historic timber, Non-Destructive Testing (NDT) methods should be used to supplement visual grading (Acuña et al., 2011). Developed in the 1980s, the resistance drill was originally intended for inspecting live trees. Since then, it has become useful in the field of heritage conservation for inspecting existing timber structures. Resistance drill analysis is a form of NDT used to assess the quality of timber in existing structures (Brashaw, 2015).

The test is performed using a resistance drill, an instrument which advances a small drill bit through the thickness of a structural member at a constant rate. The flared shape of the drill bit ensures that the force is concentrated at the tip (Rinn et al., 1996). The torque required to advance the drill bit at uniform speed is related to the relative resistance of the sample; this relative resistance is plotted on a graph for evaluation, see fig. 2.10 (Piazza & Riggio, 2008; Stepinac et al., 2017). Resistance drilling is considered non-destructive because the drill measures only 1-2 mm in diameter. The length of the drill bit varies between 30 and 45 cm and can be used to drill through members as thick as the needle's length (Rinn et al., 1996).

Figure 2.10: Resistance Drill Graph Showing Internal Decay

Local minima on the resistance drill graph represent areas of low density or weakness in the member, typically due to decay or insect damage (Brashaw, 2015). The instrument can also detect growth rings and pith. The needle feed (the speed at which the drill penetrates the member) is constant but may be adjusted based on the approximate density of the wood; higher feed speeds provide more accurate measurements for dense wood (Rinn et al., 1996). Several case studies have successfully assessed historic timber structures using a resistance drill to locate discontinuities within the timber members (Branco et al., 2017; Imposa et al., 2014).

2.6 Material Characterization by Resistance Drill Testing

Resistance drill testing is typically used for qualitative on site assessment. However, the resistance drill measurement is closely based on relative density; this relationship has been studied in order to establish a correlation between resistance drill measurement and strength properties. Visual grading and in-situ resistance drill testing are excellent methods for assessing existing timber structures. Yet, a correlation between resistance drill testing and strength properties may be used to determine the properties of historic timber which are not reflected in wood design codes.

In most cases, Resistance Drill Measurement (RM) is determined as the integral of the Resistance Drill Amplitude (RA) divided by the length of the graph (Calderoni et al., 2010; Ceraldi et al., 2001; Henriques et al., 2011). This provides an average value, since the RA varies with earlywood and latewood due the difference in density. RM is calculated based on RA as shown in eq. (2.4).

$$RM = \frac{\int_0^h RA dh}{h} \quad (2.4)$$

where:

RM = resistance drill measurement

RA = area under resistance drill curve

h = length of resistance drill graph

A 1996 study by Rinn and Schweingruber found high correlation between resistance drill profiles and x-ray density charts (Rinn et al., 1996). This study looked at anatomical features such as the width of growth rings, earlywood, and latewood. High correlative correspondence and synchronicity were seen for all of these key features for six species of coniferous and deciduous woods. Furthermore, the average levels of the resistance drill charts showed close correspondence with the gross density of the wood. Conifers, which have a relatively higher proportion of latewood, showed slightly higher local densities than the x-ray density charts. This indicates that resistance drill measurement does not depend only on density, but other wood properties as well. This study also found that the resistance drill sensitivity depends primarily on the feed rate, with higher feed rates improving the accuracy of the results.

2.6.1 Previous Testing Campaigns

Studies have shown that while resistance drill measurement is highly dependent on density, the species has been found to influence the correlation (Acuña et al., 2011; Ceraldi et al., 2001). Studies dating between 1996 and 2017 on several species of wood have established correlations between RM and density ranging between 50% and 90%. A 2001 study found moderate correlation between density and RM ($R^2 = 0.66$) for a sample consisting of eight different species of coniferous and deciduous woods (Ceraldi et al., 2001). Further studies have shown stronger correlations (up to $R^2 = 0.87$) for a variety of European species such as *pinus sylvestris* and *pinus pinaster* (Scotch pine and maritime pine) (Henriques et al., 2011; Kloiber et al., 2014). A 2011 study on six different species of wood

grown in the Mediterranean region found a strong correlation of ($R^2 = 0.82$) between RM and density for all species (Acuña et al., 2011). The coefficient of determination increased by about 10% when the species was factored into the generalized linear model (Acuña et al., 2011). Previous studies have found that the direction of the resistance drilling will affect the accuracy of the results. The correlation between Resistance Drill Measurement and density is significantly higher in the radial direction than in the tangential direction, due to friction (İçel & Güler, 2016).

Since strength properties are closely related to density, the correlation between Resistance Drill Measurement and density has been extended to estimate the mechanical properties of wood. Several studies have explored the correlation between compressive strength and RM in the past two decades (Chen & Guo, 2017; Faggiano & Marzo, 2015; A. O. Feio et al., 2007; Henriques et al., 2011). These studies have looked at new and historic timber for Non-Destructive Testing of small clear specimens. The experimentally-determined coefficients of determination for this relationship range from 50% to 75%, slightly lower than the RM-density correlation. An extensive 2010 study on historic chestnut wood from existing structures found a moderate correlation between RM and compression strength ($R^2 = 0.58$) (Calderoni et al., 2010). A similar study in 2010 conducted by Henriques *et al* found a strong correlation between RM and compressive strength ($R^2 = 0.70$) for Scotch pine and maritime pine (Henriques et al., 2011).

Few studies have attempted to relate bending behavior to Resistance Drill Measurement; the RM-MoE correlation has been found to be much weaker than that of compressive strength. In his 2005 thesis, Feio found no significant correlation between elastic modulus and RM; a significant difference in the coefficient of determination was seen for new and old chestnut wood, $R^2 = 0.61$ and $R^2 = 0.39$, respectively (A. O. Feio, 2005). A 2017 study by Chen and Guo combined stress wave tests with resistance drilling to predict the density, compressive strength, MoE, and MoR (Chen & Guo, 2017). In this study, the dynamic MoE was calculated based on the stress wave velocity. The static Modulus of

Elasticity was defined as a linear function of the dynamic MoE. The relationship between Resistance Drill Measurement and bending behavior has proven difficult to define; further research and expansion of the current knowledge database is necessary to establish a stronger correlation.

Moisture content has a significant effect on strength properties and density. For this reason, the moisture content of the testing specimens should be controlled to 12% and measured prior to testing (Calderoni et al., 2010; A. O. Feio et al., 2007; Kloiber et al., 2014). The effect of moisture content on strength may be quantitatively explained; however, its effect on resistance drill measurement has not been studied as extensively. A 2010 study found that moisture content and air temperature can affect the relation between Resistance Drill Measurement and density (Ukrainetz & O'Neill, 2010). However, this study was conducted on standing trees at significantly higher MC (19% 25% and 168%) than what is observed in structural wood. The study found a large discrepancy between the density indices for wood at 168% MC and 19% MC. The results obtained for wood at 25% MC were similar to, but slightly higher than the values for wood at 19%. The study proposes that resistance drilling of live and dead trees be considered separately. In this study, resistance drill values were found to decrease with decreasing MC (Lin et al., 2002).

Many studies have been conducted on timber from existing historic structures (Calderoni et al., 2006; Faggiano & Marzo, 2015; A. O. Feio et al., 2007; Zhang et al., 2015). It has been found that aging does not significantly affect the strength of the correlation between RM and mechanical properties for timber in good condition (Feist, 1983). However, deterioration such as insect damage or fungal decay affect the global density of the timber and weaken the correlation between RM and mechanical properties. Henriques et al., 2011 found that the relationship between RM and density was significantly weaker for decayed timber. A mass loss between 3% and 12% led to losses in strength ranging from 20% to 45% and greater dispersion of the results.

The majority of previous studies on material characterization through resistance drill testing have been conducted on small clear specimens. While this does not reflect the actual strength of structural timber, this approach enables easy comparability between studies. A study by Zhang *et al* used Finite Element Analysis (FEA) to predict the behavior of full scale timber with grading defects (Zhang et al., 2015). The analytical model used material properties determined through correlation with RM and accounted for defects such as knots and cracks to predict the behavior of the specimen.

While significant research has already been conducted to establish material properties through resistance drill testing, these studies primarily focus on European and Asian species, see 2.6. There is currently a lack of knowledge on the resistance drill correlation for Canadian species. The research reviewed in this thesis has highlighted the need for further experimental research on North American species (Calderoni et al., 2010; Ceraldi et al., 2001). Furthermore, the wide range of correlation strengths determined experimentally has led to conflicting recommendations for the use of in-situ material characterization by resistance drilling. In general, it is recommended that resistance drilling be combined with other NDT methods, such as Pylodin testing or ultrasonic tomography, to obtain accurate results (A. O. Feio et al., 2007; Henriques et al., 2011). While some studies claim that resistance drilling may be used to determine the density of wood on site, other state that it is not sufficiently accurate. No study examined in the current literature review has found the correlation between Resistance Drill Measurement and mechanical properties to be strong enough to allow for in situ determination of compressive strength or bending behavior without the use of additional NDT methods.

Table 2.6: Summary of Previous Findings

Study	Wood Species	Relationship	R^2
Ceraldi et al., 2001	Beech, <i>Fagus sylvatica</i>	RM - ρ	66%
A. O. Feio, 2005	Chestnut, <i>Castanea sativa</i>	RM - ρ	81%
		RM - f_c	48%
		RM - f_{cp}	65%
		RM - E	50%
Calderoni et al., 2010	Chestnut, <i>Castanea sativa</i>	RM - f_c	58%
Acuña et al., 2011	Scots pine, <i>Pinus sylvestris</i>	RM - ρ	82%
	Black pine, <i>Pinus nigra</i>		
	Maritime pine, <i>Pinus pinaster</i>		
	Chestnut, <i>Castanea sativa</i>		
	Walnut, <i>Juglans regia</i>		
	Oak, <i>Quercus robur</i>		
Henriques et al., 2011	Scots pine <i>Pinus sylvestris</i>	RM - ρ	87%
	Maritime pine, <i>Pinus pinaster</i>	RM - f_c	70%
Kloiber et al., 2014	Norway spruce, <i>Picea abies</i>	RM - ρ	73%
	Silver fir, <i>Abies alba</i>	RM - f_c	49%
	Scots pine, <i>Pinus sylvestris</i>		
Faggiano and Marzo, 2015	Chestnut, <i>Castanea sativa</i>	RM - ρ	64%
Zhang et al., 2015	Douglas fir, <i>Pseudotsuga menziesii</i>	RM - ρ	69%
		RM - f_c	72%
Chen and Guo, 2017	Faber's fir, <i>Abies fabri</i>	RM - ρ	56%

2.7 Reliability Analysis

2.7.1 Reliability-Based Design Principles

Design variables tend to be random in nature; reliability-based design takes uncertainties into account using a probabilistic approach (Foschi et al., 1993). A structural design problem may be characterized by the limit states function shown in eq. (2.5).

$$J = C - D \quad (2.5)$$

Where:

C = the capacity of the system or member

D = the demand on the system or member

J = the performance function, $J(X_1, X_2, \dots, X_N)$, based on N random variables X_i

From eq. (2.5), when demand on the system exceeds its capacity, structural failure is indicated by $J < 0$. Conversely, $J > 0$ implies structural survival; $J = 0$ corresponds to the limit state. The probability of failure, P_f , is defined as $P_f = P(J < 0)$. The reliability index β , represents the shortest distance to the failure region, based on eq. (2.6) (Bensoussan, 2005).

$$\beta = \frac{\mu}{\sigma} \quad (2.6)$$

Where:

β = the reliability index of the system or member

μ = the mean of the performance function J

σ = the standard deviation of the performance function J

Assuming C and D are normally distributed, they can be combined to determine the reliability index, β , of the structural system or member according to eq. (2.7) (Bensoussan, 2005).

$$\beta = \frac{\mu_C - \mu_D}{\sqrt{\sigma_C^2 + \sigma_D^2}} \quad (2.7)$$

Where:

μ_C = the mean of the capacity C

μ_D = the mean of the demand D

σ_C = the standard deviation of the capacity C

σ_D = the standard deviation of the demand D

Then the probability of failure, P_f , for a structural system with both normally distributed load D and capacity C can be described by eq. (2.8) (International Organization for Standardization, 2015).

$$P_f = N(-\beta) = 1 - N(\beta) \quad (2.8)$$

Where:

N = the standard normal probability distribution

β = the reliability index of the system or member

Figure 2.11: Failure Probability From Reliability Index

The area under the standard normal distribution curve corresponds to the failure probability of a structural system or member, as shown in fig. 2.11.

The International Organization for Standardization (ISO) standard 13822:2010 provides target reliability indices for Ultimate Limit States ranging from 2.3 to 4.3, for very low to high consequences of failure over a standard period of time (International Organization for Standardization, 2010). The $\phi = 0.9$ resistance factor used in CSA O86 is based on an

average reliability index of $\beta = 2.6$ with a range of 2.4 to 2.9 (Canadian Wood Council, 2014). This reliability index is for design loads based on a 1-in-30 year return period, and provides a similar reliability to steel structures.

The factored loads specified in the National Building Code were developed based on reliability principles from International Organization for Standardization (ISO) standard 2394 so as to achieve a low probability of exceedance (International Organization for Standardization, 2015). Since the 2005 edition of the NBC, climactic loads have been based on a 50-year return period, which corresponds to a 2% probability of exceedance (Bartlett et al., 2003).

The design values provided for wind and snow loads correspond to the 98th percentile of the climactic data (Canadian Commission on Building and Fire Codes, 2015b). Due to a lack of statistical information, the specified values for dead and live loads are taken as the nominal values. Bartlett et al., 2003 provides the coefficients of variation for live and dead loads adopted in National Building Code 2005 as 0.206 and 0.100 respectively.

2.7.2 Analysis of Existing Wood Structures

The first stage in analysing a historic building is understanding it; this can be done through documentation, condition assessment, Non-Destructive Testing (NDT), structural analysis, or a combination of all of these methods (Parks Canada, 2010). Condition assessments are typically used to determine the level of damage, deterioration, and decay in an existing building. Qualitative and quantitative inspection methods may be used, but for both methodologies the accuracy and limitations should be kept in mind.

According to ISO 13822:2010, the properties and reliability estimates of a structure may be updated based on information gathered during inspection (International Organization for Standardization, 2010). This can be achieved by either direct updating of the structural failure probability, or by updating the probability distribution of the random variables.

The latter method, which is used in this study, allows for design values to be updated based their correlation with measured properties.

Typically, design values are taken as the fifth percentile values for a given property f , as shown in eq. (2.9).

$$f_k = \mu_f - 1.645\sigma_f \quad (2.9)$$

Where:

f_k = the design value for f

μ_f = the average value of f

σ_f = the standard deviation of f

However, if another property, q , is directly measured to be above average and there is a correlation between f and q , then the mean and standard deviation can be modified based on the correlation (International Organization for Standardization, 2010). The standard deviation may be updated based on the strength of the correlation, as shown in eq. (2.10). The mean value may also be updated for the considered member according to eq. (2.11), based on the measured value of q for that member. The updated fifth percentile design strength of the member, f'_k may then be determined based on the updated values of μ'_q and σ'_q as shown in eq. (2.12).

$$\sigma'_f = \sigma_f \sqrt{1 - R_{f,q}^2} \quad (2.10)$$

$$\mu'_f = \mu_f + R_{f,q}\sigma_f \left(\frac{q_m - \mu_q}{\sigma_q} \right) \quad (2.11)$$

Where:

$R_{f,q}$ = the Pearson correlation coefficient for f and q

q_m = the measured value of q for a given member

μ_q = the average value of q

σ_q = the standard deviation of q

$$f'_k = \mu'_f - 1.645\sigma'_f \quad (2.12)$$

Figure 2.12: Updated Design Values

This updating process is illustrated in fig. 2.12. Note that the process of updating the mean and standard deviation of the member should only be used to determine design values. The updated mean and standard deviation do not supersede the actual values.

3. Testing Methodology

The goal of the experimental testing campaign was to evaluate the relationships between the properties determined through destructive and NDT methods. The material properties of Eastern white pine examined in this research are specific gravity (G), Resistance Drill Measurement (RM), growth ring density, compressive strength (f_c), Modulus of Elasticity (E) and bending strength (f_b). The three sample groups included: oven-dry compression (55 samples), air-dry compression (54 samples), and oven-dry bending (28 samples). The following sections outline the material selection process and equipment and procedures used to perform the testing.

3.1 Materials

3.1.1 Selected Wood Species

Eastern white pine (*pinus strobus*) was selected for this research as this species has been historically significant in construction in the Ottawa valley. The species is prevalent throughout Eastern Canada, see fig. 3.1 and is still used in wood frame construction in the 21st century in Eastern Canada and the Northeastern United States.

The availability of a sufficient sample size of the selected species was an important consideration, as the procurement of the research materials took place during November and December of 2020. At this time, the global COVID-19 pandemic had caused a decrease in production and supply chain disruptions, while demand for lumber increased due to a swell in home construction; resulting in a lumber shortage (Fleming, 2021).

The wood used in this research was procured from two Canadian sources, one located in Fredericton, New Brunswick and the other in Cumberland, Ontario. The wood sample comprised a variety of different growth rates and ring densities. The sample included

Figure 3.1: Eastern White Pine Distribution Map (Natural Resources Canada, 2015)

wood cut from different parts of the trunk; varying proportions of heartwood and sapwood were present in the specimens.

3.1.2 Test Specimens

Per ASTM Standard Test Methods for Small Clear Specimens of Timber (ASTM D143), the recommended cross section for the compression parallel to grain and static bending specimens measures 50 mm by 50 mm (2 in. by 2 in.). The compression specimens used in this study had a length of 200 mm (8 in.), while the bending specimens were 760 mm (30 in.) long.

These specimens were all free of knots and other irregularities as per ASTM D143, see fig. 3.2. Typical wood defects are proportionally large compared to the cross-section of the test specimens and can have a significant influence on strength and failure mechanisms. For small-scale testing, any wood defects will cause significant variability in the results. In order to establish a baseline correlation between strength and resistance drill measurement, the small specimens were uniform in quality.

Figure 3.2: Compression Specimens Prior to Testing

3.1.3 Sample Size

The sample size was selected based on ASTM Standard Practice for Sampling and Data-Analysis for Structural Wood and Wood-Based Products (ASTM D2915) (ASTM International, 2017b). Based on this standard, at least fifty-two compression specimens and seventy-eight bending specimens would be required to obtain a 95% confidence interval. Due to the difficulty of obtaining a large number of 760 mm (30 in.) long defect-free specimens, the sample size for the bending specimens was based on a 75% confidence interval. Per ASTM D2915 twenty-seven bending specimens would be necessary to obtain a confidence interval of 75%. Ultimately, twenty-eight bending specimens and one hundred and nine compression specimens were prepared for testing. Due to the surplus of compression samples, other parameters were varied, such as the moisture content and the number of drill measurements.

Table 3.1: Test Specimens

Destructive Test Type	Moisture Content (%)	Sample Size
Bending	0	28
Compression	0	55
Compression	8.5	54

3.2 Equipment and Procedures

3.2.1 Moisture Content and Density Measurement

The oven-dry mass of each sample was obtained by drying the samples in a 93 ± 2 °C oven for over twenty-four hours. Once no change in mass was observed, the oven-dry mass and volume of each sample were recorded. The specific gravity measurement was based on the oven dry mass and oven-dry volume of the samples using eq. (3.1) (Glass & Zelinka, 2010).

$$G_0 = \frac{m_0}{V_0} \times \frac{1}{\rho_{water}} \quad (3.1)$$

Where:

G_0 = the oven-dry specific gravity

m_0 = the oven-dry mass

V_0 = the oven-dry volume

ρ_{water} = density of water

The compression specimens were tested at two different Moisture Contents: oven-dry (0%) and air-dry (8.5%) in order to evaluate the effect of Moisture Content on the Resistance Drill Measurement. The air-dry samples were conditioned in a sealed chamber with a temperature of 20 ± 2 °C and a relative humidity of $45 \pm 5\%$. This resulted in an Equilibrium Moisture Content of approximately 8.5% which was attained after two

weeks. In order to prevent end rolling, the air dry compression samples were stacked and dried in a 93 °C (200 °F) oven for 10 to 15 minutes prior to testing to slightly dry the ends in accordance with ASTM D143. Drying the ends of the specimens prevents end rolling failures and ensures that failure develops in the body of the specimen (ASTM International, 2014).

A Tramex[®] MRH3 pin-probe moisture and humidity meter was used to verify the internal and surface MC of the air-dry sample prior to testing. The average MC of each specimen was calculated directly based on the oven-dry mass according to eq. (3.2).

$$MC_{test} = \frac{m_{test} - m_0}{m_0} \quad (3.2)$$

Where:

m_{test} = the mass at testing

m_0 = the oven-dry mass

The bending specimens were only tested in the oven-dry state to prevent significant variability in the relatively small sample. Similar studies have not shown strong correlations with bending behavior and non-destructive testing, maintaining uniformity within the bending sample was expected to yield a stronger correlation.

In addition to Moisture Content and specific gravity, the average number of growth rings per radial inch and proportion of earlywood to latewood were recorded for each specimen to determine the growth ring density (ASTM International, 2006; National Lumber Grading Association, 2017). The primary observable feature that differentiates old growth from modern rapid cultivated timber is the growth ring density. Since this contributes to the specific gravity and strength, by determining the influence of growth ring density on strength, these findings may be applied to historic wooden structures.

Figure 3.3: From Top to Bottom: Slow Growth, Medium Growth, and Fast Growth Rates

3.2.2 Resistance Drill Measurement

An IML-RESI Power Drill 300[®] resistance drill was used for the testing, see fig. 3.4. The drill bit was 300 mm long and 3 mm in diameter at the tip, with a shaft diameter of 1.5 mm. A feed speed of 127 cm/min and a needle speed of 5000 rpm were used.

Figure 3.4: IML-RESI PD300

Figure 3.5: Drilling Setup

The specimens were supported on two wooden blocks and drilled from the top down. The tangential surface farthest from the pith was selected as the drilling surface, to allow for drilling perpendicular to the growth rings see fig. 3.5. Prior research has shown that Resistance Drill Measurements in the radial direction have a higher correlation with density than measurements taken in the tangential direction, due to friction (İçel & Güler, 2016; Nowak et al., 2015). All drilling measurements were made in the central third of the specimen width so as to maximize the angle between the drill bit and the growth rings.

The compression test specimens were drilled at either one or two locations at regular intervals along the length of the member. The purpose of this approach was to determine the minimum number of Resistance Drill Measurements required to accurately determine the material properties of the specimen. The bending specimens were drilled at either three or five locations at regular intervals along the member length.

Figure 3.6: Typical Resistance Drill Graph

A resistance drill measures the torsional moment of the motor as the bit advances at a constant rate through the thickness of the wood specimen. Torsional moment is induced by the friction along the needle shaft and wood chipping at the tip of the needle. The needle must drill through to the opposite side of the specimen to determine the torsional moment in air. The amplitude of the resistance drill graph is the relative increase in torsional moment expressed as a percentage, see fig. 3.6. The Resistance Drill Measurement (RM) was calculated as the average measurement for the sample, which can also be taken as the ratio of the area below the averaged curve to the drilling depth, see eq. (3.3).

$$RM = \frac{\int_0^d A}{h} \quad (3.3)$$

Where:

A = is the area below the resistance drill curve

h = is the length of the resistance drill graph

For specimens with multiple resistance drill measurements, the average RM value was used in calculations.

3.2.3 Compression Testing

An Instron® 5582 Universal Testing Machine (UTM) was used for the destructive compression testing, see fig. 3.7 for the setup. This UTM has a capacity of ± 100 kN and measures load with an accuracy of 0.5% and deflection with an accuracy of 0.25 μm .

The rate of loading was displacement controlled, with a constant rate of 0.6 mm/min applied using flat platens on the top and bottom of the specimen. Axial deflection was measured by the UTM.

Figure 3.7: Instron 5582 Setup

Figure 3.8: Tinius Olsen Setup

The predicted fail load of the specimens was estimated as 85 kN, based on material properties determined from Standard Practice for Establishing Clear Wood Strength Values (ASTM D2555). However, upon testing, some specimens did not fail under the 100 kN load applied by the Instron 5582 UTM. These samples were then tested using a Tinius Olsen hydraulic UTM with a capacity of 1800 kN (400 kip) , see fig. 3.8.

The compression specimens were tested until failure; the failure type was noted. The ultimate load was recorded and used to calculate the specimen's compressive strength f_c , as shown in eq. (3.4).

$$f_c = \frac{P_{ult}}{A} \quad (3.4)$$

Where:

P_{ult} = the ultimate compressive load at failure

A = the cross-sectional area of the specimen

Figure 3.9: Typical Stress-Strain Curve for Compression Test

The compressive Modulus of Elasticity E_c was calculated as the slope of the stress-strain curve over the elastic portion for each sample, see fig. 3.9. The slope was estimated

based on the stress and strain at 40% and 10% of the ultimate compressive load, see eq. (3.5).

$$E_c = \frac{\sigma_{40} - \sigma_{10}}{\varepsilon_{40} - \varepsilon_{10}} \quad (3.5)$$

Where:

σ_{40} = 40% of ultimate stress supported by the specimen

σ_{10} = 10% of ultimate stress supported by the specimen

ε_{40} = the strain at σ_{40}

ε_{10} = the strain at σ_{10}

The use of a 10% to 40% range ensures that the modulus of elasticity is calculated over the linear portion of the stress-strain curve, below the proportional limit. In practice, curve fitting should cover a minimum range of 20% of the stress-strain curve and should exclude the non-linear portion at typically seen the beginning of the curve and near the proportional limit (ASTM International, 2015).

3.2.4 Static Bending Testing

The bending specimens were tested in the same Instron[®] 5582 Universal Testing Machine using a three-point bending test fixture to support the specimen horizontally. The specimens were supported on rollers spaced 710 mm (28 in.) apart, allowing for a 25 mm (1 in.) overhang on each end, as shown in fig. 3.10. A load rate of 2.5 mm/min was applied at mid-span using a steel bearing block with a radius of 75 mm (3 in.) as described in Standard Test Methods for Small Clear Specimens of Timber (ASTM D143) (ASTM International, 2014). Deflection was measured at mid-span by the UTM with an accuracy of 0.25 μ m.

Figure 3.10: Bending Test Setup

The specimens were oriented such that the load was applied to the tangential surface of the growth rings nearest to the pith, as specified in ASTM D143. The bending specimens were tested until failure and the failure type was noted. The ultimate load was recorded and used to calculate the specimen's bending strength, f_b , as shown in eq. (3.6) (ASTM International, 2015).

$$f_b = \frac{P_{ult}l}{2S} \quad (3.6)$$

Where:

P_{ult} = the ultimate load supported by the specimen

l = the span of the bending specimen

S = the section modulus, $\frac{bd^2}{6}$

The load and mid-span deflection were recorded during testing, fig. 3.11 shows the results of a typical bending test.

Figure 3.11: Typical Load-Deflection Curve for Bending Test

The modulus of elasticity, E_b , was calculated based on the load and the deflection at mid-span for a simply-supported beam eq. (3.7) (ASTM International, 2015). The slope of the tangent between 10% and 40% of the ultimate load was used to calculate the MoE from bending.

$$E_b = \frac{l^3}{48I} \times \frac{P_{40} - P_{10}}{\Delta_{40} - \Delta_{10}} \quad (3.7)$$

Where:

P_{40} = 40% of ultimate load supported by the specimen

P_{10} = 10% of ultimate load supported by the specimen

Δ_{40} = the deflection at P_{40}

Δ_{10} = the deflection at P_{10}

l = the span of the bending specimen

I = the moment of inertia, $\frac{bd^3}{12}$

The use of a 10% to 40% range ensures that the modulus of elasticity is calculated over the linear portion of the load-deflection curve, below the proportional limit. In practice, curve fitting should cover a minimum range of 20% of the load-deflection curve and should exclude the non-linear portion at typically seen the beginning of the curve and near the proportional limit (ASTM International, 2015).

4. Results

The material properties of Eastern white pine examined in this research are specific gravity (G), compressive strength (f_c), Modulus of Elasticity (E) and bending strength (f_b). The following sections outline the results of the experimental tests with regards to characterization of these material properties.

4.1 Specific Gravity Determination

The oven dry specific gravity was calculated using the 0% Moisture Content (MC) basis for mass and volume measurements. The average oven-dry specific gravity for all samples was found to be 0.35, with a standard deviation of 0.03, see fig. 4.1. ASTM D2555 “Standard Practice for Establishing Clear Wood Strength Values” gives the basic specific gravity of Eastern white pine (*pinus strobus*) as 0.36.

Figure 4.1: Oven-Dry Specific Gravity Distribution

4.1.1 Growth Ring Density

The sample included a variety of growth ring densities ranging from 3 to 12 growth rings per radial inch, with an average of 6.2 growth rings, see fig. 4.2. For the purposes of this study, the specimens were classified into low (2-5 rings/inch), medium (5-7 rings/inch), and high (7-10 rings/inch) grain density categories.

Figure 4.2: Ring Density Distribution

The correlation between specific gravity and growth ring density was low, with a regression coefficient of 39%, see fig. 4.3. This correlation does not account for the proportion of earlywood to latewood. Based on these results, growth ring density alone is not an accurate predictor of material properties. Furthermore, this characteristic can be difficult to measure on-site as end-grain is not always visible in existing structures.

Figure 4.3: Growth Ring Density vs. Specific Gravity

4.1.2 Resistance Drill Measurement

For specimens with multiple drill locations, the average Resistance Drill Measurement (RM) was used. A very strong linear correlation was found between the RM and the oven-dry specific gravity for all specimens, see fig. 4.4. The coefficient of determination was found to be 80%, with a standard error of 0.013. The correlation was similar between the three sample groups (oven-dry compression, air-dry compression, and oven-dry bending). The 200 mm (8 in.) long compression samples were drilled at one or two locations, while the 720 mm long (30 in.) bending specimens were drilled at three or five locations along the member.

Figure 4.4: Resistance Drill Measurement vs. Specific Gravity

The specimens with the highest number of drill measurements per meter length had a slightly stronger correlation. The compression specimens that were drilled at two locations along the member length had a coefficient of determination (R^2) of 84%, compared to 81% for the specimens drilled at only one location. The bending specimens show similar results, though the sample sizes were smaller, see table 4.1.

Table 4.1: Drill Measurements per Meter Length

Sample Type	Drill Locations	Specimens Tested	R^2
Compression	2	54	0.84
Compression	1	55	0.81
Bending	5	14	0.79
Bending	3	14	0.74

Based on the results of this study, the average Resistance Drill Measurement can be used to estimate the oven-dry specific gravity G_0 according to eq. (4.1) within a reasonable degree of accuracy.

$$G_0 = 0.107(RM)^{0.47} \quad (4.1)$$

Figure 4.5: Percent Error Specific Gravity Calculated From eq. (4.1)

The predicted specific gravity of the specimens based on eq. (4.1) ranged from 12% below to 16% above the actual specific gravity of the sample, see fig. 4.5. By applying a 10% margin of error to eq. (4.1), the equation can be modified to eq. (4.2) to obtain conservative values for specific gravity.

$$G_0 = 0.097(RM)^{0.47} \quad (4.2)$$

The minimum number of resistance drill measurements examined in this study was four per meter length; this implies that in the field, a structural timber must be drilled at

least every 250 mm along its length to estimate the clear specific gravity according to eq. (4.2). The relationships between RM and specific gravity in this study were determined by using the IML-RESI Power Drill 300[®] resistance drill with a feed speed of 127 cm/min and a needle speed of 5000 rpm. These relationships can be expected to vary for different drill speeds, needle speeds and drill bit sizes and profiles.

4.2 Compression Parallel to Grain Behavior

The material and mechanical properties that were studied for the compression sample were: compressive strength (f_c), compressive Modulus of Elasticity (E_c), Resistance Drill Measurement (RM), and oven-dry specific gravity (G_0). The relationships between these four properties for the two Moisture Contents are outlined in the following sections. The raw experimental data may be found in chapter A of the Appendix.

4.2.1 Compressive Strength

The effect of Moisture Content on the mechanical properties of wood is well documented in existing wood design guidelines (ASTM International, 2006). Increasing MC has been proven to decrease strength properties up to the Fibre Saturation Point (FSP); this effect has a particularly strong influence on compressive strength (Glass & Zelinka, 2010). This was seen in the testing results, as the oven-dry (0% MC) specimens were roughly 13 MPa stronger than their air-dry (8% MC) counterparts, see table 4.2.

Table 4.2: Compressive Strength Values (f_c) in MPa

	Oven-Dry	Air-Dry	All
Average	42.7	30.0	36.3
Minimum	32.5	18.4	18.4
Maximum	53.7	41.7	53.7
CV (%)	11	18	23

A two-tailed pooled t-test was used to determine the significance of the difference between the compression strengths of the two different moisture contents at a 99% confidence level. The results of the t-test indicate a statistical difference between the compression results at oven-dry and air-dry moisture content.

While species and grade play an important role in determining the strength of wood, compressive strength is generally understood to be related linearly to density (A. O. Feio, 2005). This is shown in fig. 4.6, which illustrates the relationship between compressive strength and specific gravity for the two different Moisture Contents.

Figure 4.6: Compression Strength vs. Specific Gravity

The oven-dry and air-dry samples were related separately to specific gravity, as the change in MC created a divergence in the compression strengths. Both sample groups show a moderate correlation between compression strength and specific gravity; 53% and 50% for the oven-dry and air-dry samples, respectively. The near-linear relationships

between compressive strength and specific gravity are described by eq. (4.3) for the oven-dry sample and eq. (4.4) for the air-dry sample.

$$f_{c,OD} = 128G^{1.04} \quad (4.3)$$

$$f_{c,AD} = 157G^{1.56} \quad (4.4)$$

As seen in fig. 4.6 and the regression equations, the slopes of the regression lines for both sample groups appear to be similar for the range of specific gravity seen in the sample group.

CSA O86 classifies Eastern white pine (*pinus strobus*) under the “Northern” species category for wood. Due to the straight grain and lack of defects, the small clear specimens used in this campaign would be visually graded as “Select Structural” according to NLGA guidelines. The CSA O86-14 design compressive strength of Select Structural grade Northern wood is 13 MPa. This design strength is based on wood at 19% MC and does not take variations in density into account (Canadian Wood Council, 2014). ASTM D2555 provides the compressive strength for Eastern white pine at the Fibre Saturation Point and at 12% MC (ASTM International, 2017a) as 18 MPa and 35 MPa, respectively. Furthermore, the Forest Product Laboratory’s Wood Handbook describes a near-linear relationship between specific gravity and compressive strength, according to the function in eq. (4.5) (Kretschmann, 2010a).

$$f_c = 93.7G^{0.97} \quad (4.5)$$

The relationship shown in fig. 4.7 compares the values determined experimentally in this study to the design values given by CSA O86, ASTM D2555 and the Forest Product Laboratory (FPL) Wood Handbook. All experimentally determined compressive strengths

exceed the CSA O86 design values, which are based on the fifth percentile for wood at 19% moisture content. The oven dry compression strengths were found to be higher than the dry values from both the Wood Handbook and ASTM D2555. The air dry compression strengths exceeded the green values from both standards.

Figure 4.7: Compression Strength Comparison with Design Standards

The relationship between compressive strength and resistance drill measurement was similar to the relationship with specific gravity. A moderate correlation was observed, 61% for the oven-dry sample and 60% for the air-dry sample. The correlation with compressive strength was slightly higher than the relationship with specific gravity, possibly because the resistance drill detects local variations in the material along the drilling depth.

As seen in fig. 4.8, the linear regression equations have similar slopes, according to the equations eq. (4.6) for the oven-dry and eq. (4.7) for the air-dry specimens.

$$f_{c,OD} = 10.4RM^{0.56} \quad (4.6)$$

Figure 4.8: Compression Strength vs. Resistance Drill Measurement

$$f_{c,AD} = 3.87RM^{0.82} \quad (4.7)$$

Two different failure types were prevalent depending on the moisture content. Splitting failure was observed in 73% of the oven-dry specimens, compared to only 22% of the air-dry specimens. The majority of the air-dry samples failed in crushing, rather than splitting.

4.2.2 Compressive Modulus of Elasticity

The modulus of elasticity was calculated from the stress and strain at 40% and 10% of the ultimate compressive strength, see eq. (4.8). Moisture Content was not found to significantly affect the Modulus of Elasticity, see table 4.3.

Figure 4.9: Typical Splitting Failure Figure 4.10: Typical Crushing Failure

$$E_c = \frac{\sigma_{40\%} - \sigma_{10\%}}{\varepsilon_{40\%} - \varepsilon_{10\%}} \quad (4.8)$$

Where:

σ_{40} = 40% of ultimate stress supported by the specimen

σ_{10} = 10% of ultimate stress supported by the specimen

ε_{40} = the strain at σ_{40}

ε_{10} = the strain at σ_{10}

Table 4.3: Compression Modulus of Elasticity Values (E_c) in MPa

	Oven-Dry	Air-Dry	All
Average	4380	4010	4210
Minimum	2740	2740	2740
Maximum	5530	5090	5530
CV (%)	14	12	16

The MoE was related separately to the compressive strength for the oven-dry and air-dry samples. As shown in fig. 4.11, the oven-dry specimens have a compressive strength

roughly 10 MPa higher than the air-dry specimens with the same MoE. The linear relationships between compressive strength and Modulus of Elasticity were strong, with correlation coefficients of 62% and 72% for the oven-dry and air-dry samples, respectively.

Figure 4.11: Modulus of Elasticity vs Compressive Strength

The oven-dry and air-dry samples were evaluated separately to qualify the relationship with specific gravity and Modulus of Elasticity, however MoE does not vary significantly with moisture content (ASTM International, 2006; Kretschmann, 2010a). The overall correlation between the Modulus of Elasticity and specific gravity was moderate with a correlation coefficient of 38%. When examined separately, the correlation between specific gravity and MoE was slightly stronger for each sample, see fig. 4.12.

Figure 4.12: Modulus of Elasticity vs Specific Gravity

The relationship between Modulus of Elasticity (MoE) and specific gravity established through this experiment may be described for the entire compression sample by the function shown in eq. (4.9).

$$E_c = 15699G_0^{1.25} \quad (4.9)$$

CSA O86 provides the modulus of elasticity for Eastern white pine as 7500 MPa. Unlike other material properties, this is an average value based on destructive bending testing of in-grade specimens, and not the fifth percentile value from normal distribution (Canadian Standards Association & Canadian Wood Council, 2015). The values provided in tables 6.3.1A-D in CSA O86-14 reflect standard term loading and 19% MC (Canadian Standards Association & Canadian Wood Council, 2015). ASTM D2555 provides the MoE for dry wood as 8100 MPa and 6540 MPa for green wood (ASTM International,

2017a). These values do not account for variations based on specific gravity. However, the FPL Wood Handbook describes the near-linear relationship between specific gravity and Modulus of Elasticity for dry wood according to the function in eq. (4.10)

$$E = 20500G^{0.84} \quad (4.10)$$

The relationships shown in fig. 4.13 compare the values determined experimentally in this study to the design values proposed by CSA O86-14 and ASTM D2555 and the Forest Product Laboratory Wood Handbook. The MoE specified in these design codes is based on bending testing, where the primary failure mechanism is tensile failure of the wood fibres. The MoE determined through compressive testing was significantly lower than bending testing. Kim et al., 2011 found through testing several softwood species in axial tension and compression and bending that the compressive MoE is roughly half of the tensile MoE. A. O. Feio, 2005 clarifies that the MoE in tension and bending are roughly equivalent.

Figure 4.13: Modulus of Elasticity Comparison with Design Standards

The correlation between the compression Modulus of Elasticity and RM was slightly weak ($R^2 = 0.45$), see fig. 4.14. Similarly to the compressive strength results, the resistance drill found a slightly higher correlation with MoE than specific gravity. Again, the correlation increased slightly when the samples were grouped based on Moisture Content, but this had a minor effect on the correlation.

Figure 4.14: Modulus of Elasticity vs Resistance Drill Measurement

The relationship between Modulus of Elasticity (MoE) and Resistance Drill Measurement established through this experiment for the entire compression sample may be described by the function shown in eq. (4.11).

$$E_c = 809RM^{0.66} \quad (4.11)$$

The MoE from axial compression testing cannot be determined as accurately as compression strength with respect to specific gravity and Resistance Drill Measurement.

However, the compressive MoE, E_c is not as influenced by Moisture Content as f_c (ASTM International, 2017a). This makes in-situ material characterization of MoE in buildings with uncontrolled MC more feasible if the MC is unknown.

4.3 Static Bending Behavior

The material and mechanical properties that were studied for the bending specimens were: bending strength (f_b), Modulus of Elasticity (E_b), Resistance Drill Measurement (RM), and oven-dry specific gravity (G_O). The bending sample was tested at 0% Moisture Content and categorized based on the failure modes defined in ASTM D143. The relationships between the four properties for the two primary failure modes are outlined in the following sections. The raw experimental data may be found in chapter B of the Appendix.

4.3.1 Bending Strength

Bending strength, or Modulus of Rupture (MoR), of wood is based on the ratio between the tensile and compressive strengths (Buchanan, 1990). Tension failure in wood is exhibited by a brittle rupture of the fibres, whereas compressive failure tends to be a more gradual ductile failure caused by micro-buckling, or crushing, of the fibres. In a typical bending failure governed by tension, as the extreme fibers begin to fail in compression, the neutral axis shifts towards the tension face, which increases the tensile stresses until failure occurs (Zakic, 1983).

Comparatively, shear failure is governed by perpendicular to grain strengths and is exhibited by longitudinal splitting along the length of the specimen (Soltis & Gerdhart, 1988). “Standard Test Methods for Small Clear Specimens of Timber” (ASTM D143) recommends documenting the failure mechanism of bending specimens. ASTM D143 categorizes static bending failures into the following six types, shown in fig. 4.15:

Figure 4.15: ASTM D143 Bending Failure Modes, (Bodig & Jayne, 1993)

- (a) **Simple Tension:** Tension and compression failures occur, with splitting along the neutral axis
- (b) **Cross-Grain Tension:** Splitting failure caused by excessive grain slope, inadmissible failure type
- (c) **Splintering Tension:** Tension governs, fibrous tensile fracture showing splinters
- (d) **Brash Tension:** Tension governs, abrupt tensile fracture
- (e) **Compression:** Compression strength governs, compressive yielding occurs without tensile failure
- (f) **Horizontal Shear:** Shear governs, horizontal splitting occurs at ends

Reducing Moisture Content below the Fibre Saturation Point increases compression strength, whereas tension parallel to grain strength peaks between 10%-15% MC (Kretschmann, 2010a). This decreases the ratio of tension to compression strength; when tested in bending, the failure mode of wood with a MC lower than 10% is more likely to be governed by tension than compression (Buchanan, 1990). The tension and compression strengths perpendicular to grain are largely unaffected by variations in Moisture Content (Kretschmann, 2010a). Furthermore, dry wood is more prone to checking and splitting due to shrinkage, thus shear failure is also more prevalent in specimens with a lower MC.

The primary failure modes seen in this experiment were a) Simple Tension, c) Splintering Tension, and f) Horizontal Shear. Few specimens failed in cross-grain tension, however these results were culled as dictated by ASTM D143. The results shown in table 4.4 illustrate the range of MoR observed in the bending sample in this study for each failure mode.

Table 4.4: Bending Strength Values (f_b) in MPa

	Tensile Failure	Shear Failure	All
Average	35.3	69.1	55.2
Minimum	24.5	52.0	24.5
Maximum	47.0	91.3	91.3
CV (%)	47	19	36

The variety of brittle failure modes in flexure resulted in a high variability in the bending strengths. Tension failures were evidenced by rupture in the tension zone with splitting along the neutral axis, see fig. 4.16. Shear failures, were identified by longitudinal splitting near the support, as seen in fig. 4.17. As seen in table 4.4, the specimens which failed in shear had much higher bending strengths and less dispersion compared to the tension-governed specimens.

While compression strength has been shown to relate linearly to specific gravity, bending strength is governed by brittle failure mechanisms which makes it difficult to predict strength based only on specific gravity (Bohannon, 1966). The overall correlation

Figure 4.16: Typical Tensile Failure

Figure 4.17: Typical Shear Failure

between bending strength and specific gravity was very weak ($R^2 = 0.08$), as seen in fig. 4.18.

Figure 4.18: Bending Strength vs. Specific Gravity

The CSA O86 design value for bending strength, f_b , for Select Structural grade wood in the Northern species category is 10.6 MPa, for an MC of 19%, regardless of specific gravity or species. ASTM D2555 provides the clear specimen bending strength for Eastern white pine (*pinus strobus*) at the FSP and 12% MC as 35.4 MPa and 61.6 MPa, respectively.

The FPL Wood Handbook describes a near-linear relationship between bending strength and specific gravity according to the function shown in eq. (4.12).

$$f_b = 170.7 * G_0^{1.01} \quad (4.12)$$

The relationships shown in fig. 4.19 compare the experimentally-determined bending strength values to the design values provided in CSA O86, ASTM D2555, and the FPL Wood Handbook. All experimentally-determined values exceed the CSA O86 design values, which are based on the fifth percentile exclusion limit. Roughly half of the bending strengths exceed the Wood Handbook bending strengths for dry wood, the other half are centered around the values for wood in the green condition. Note that specimens which failed in shear generally had higher bending strengths than those which failed in tension.

Figure 4.19: Bending Strength Comparison with Design Standards

The relationship between bending strength and Resistance Drill Measurement was stronger than the relationship with specific gravity, however the overall correlation for the entire sample was weak ($R^2 = 0.29$). The resistance drill accounts for slight variations

in the member as it drills through a specimen, which may be why there was a higher correlation. Resistance Drill Measurement has not shown to be a particularly strong predictor of bending strength in prior studies. The presence of natural defects will most likely determine the governing bending failure mode for structural timbers. Due to the low correlation, Resistance Drill Measurement is not well-suited for in-situ material characterization to determine bending strength.

Figure 4.20: Bending strength vs. Resistance Drill Measurement

4.3.2 Bending Modulus of Elasticity

The Modulus of Elasticity (MoE) was calculated based on the slope of the load-deflection curve at 40% and 10% of the maximum load and the deflection at mid-span. While the specimens were still categorized based on failure mode, this did not appear to have a

significant influence on the MoE, however the specimens which failed in shear generally had a higher Modulus of Elasticity than those which failed in tension, as seen in table 4.5.

Table 4.5: Modulus of Elasticity Values (E_b) in MPa

	All
Average	7230
Minimum	5040
Maximum	9770
CV (%)	17

A strong correlation of 74% was established between the MoE and the bending strength, see fig. 4.21.

Figure 4.21: Modulus of Elasticity vs. Bending Strength

The correlation between specific gravity and Modulus of Elasticity for the entire sample was weak, with a coefficient of determination of 38%, see fig. 4.22.

Figure 4.22: Modulus of Elasticity vs. Specific Gravity

Design values presented for MoE are typically based on bending tests rather than the compressive MoE from axial testing since the compressive MoE are much lower (Glass & Zelinka, 2010; Kim et al., 2011). CSA O86 provides the Modulus of Elasticity for Eastern white pine (*pinus strobus*) as 7500 MPa. Unlike other material properties, this is an average value based on experimental testing of in-grade specimens, and not the fifth percentile value from normal distribution (Canadian Standards Association & Canadian Wood Council, 2015). ASTM D2555 provides the MoE for dry wood as 10050 MPa and 8110 MPa for green wood (ASTM International, 2017a). These values do not account for variations based on specific gravity. However, the FPL Wood Handbook describes the near-linear relationship between specific gravity and Modulus of Elasticity for dry wood according to the function previously shown previously in eq. (4.10) (Glass & Zelinka, 2010).

Although the MoE values determined by destructive testing of bending specimens were much higher than those from axial compression testing, these values were still slightly lower than the design values presented in the recognized design standards, see fig. 4.23. The adjustment factor in ASTM D245 results in a 6% increase of the MoE as this factor adjusts the span to depth ratio from 14 to 21 and adjusts for uniformly distributed loading (ASTM International, 2006). Since design values represent an average value that is increased by the size adjustment factor, it follows that the average value determined experimentally was slightly lower.

Figure 4.23: MoE Comparison with Design Standards

The MoE is not dependant on failure mode, and is therefore less variable than bending strength. The overall correlation between MoE and RM was strong, with a coefficient of determination of 64%, see fig. 4.24.

Based on these results, resistance drilling may provide an indication of the Modulus of Elasticity (MoE) with a higher degree of accuracy than specific gravity. The correlation established in the present research is not sufficiently strong for in-situ material charac-

Figure 4.24: Modulus of Elasticity vs. Resistance Drill Measurement

terization of the MoE of existing structural members without the use of supplementary Non-Destructive Testing methods.

4.4 Discussion

4.4.1 Relationships Between Material Properties

The coefficients of correlation determined in this study between the various mechanical properties for Eastern white pine are shown in table 4.6.

The MoE values calculated from the bending specimens were significantly larger than those determined through axial compression testing. The MoE determined through destructive bending testing is largely the same as the experimentally determined tensile MoE (A. O. Feio, 2005). Prior studies have shown that tensile MoE of wood is approx-

Table 4.6: Strength of Relationships Between Material Properties

R^2 (%)	ρ	RM	f_c	f_b	E_c	E_b
RM	80					
f_c	51	60				
f_b	9	29	NA			
E_c	45	38	67	NA		
E_b	38	64	NA	74	NA	

imately two times larger than the compressive MoE (Kim et al., 2011). Wood loaded in axial compression experiences larger deformations due to separation of the fibres and micro-buckling, as a result, the compression MoE is typically lower than the tensile MoE.

The compressive MoE found in the present study was closely correlated to compressive strength. Similarly, the MoE determined through bending testing was closely related to bending strength, despite the high variability seen in the bending strength results. Strength and stiffness are generally proportional within a species of wood (Kretschmann, 2010a).

The correlations between growth ring density and the mechanical properties were weak. However, for all material properties, specimens with higher growth ring densities had higher strength and stiffness as seen in table 4.7. Growth ring density is one of the predominant features which distinguishes old growth wood from new, rapid-cultivated lumber.

Table 4.7: Growth Ring Density Effect

Ring Density	$\bar{f}_c$ (MPa)	$\bar{E}_c$ (MPa)	$\bar{f}_b$ (MPa)	$\bar{E}_b$ (MPa)
Low	30.7	3594	46.1	5378
Medium	36.8	4319	52.6	6938
High	39.4	4603	55.2	7431

4.4.2 Comparison with Published Material Properties

A comparison between the mechanical properties studied in this research with the values provided by recognized design standards is shown in table 4.8. In keeping with CSA O86, the fifth percentile values have been used for compression and bending strength, while the average values were taken as the design values for MoE.

Figure 4.25: Experimental Compressive Strength Distribution

Figure 4.26: Experimental Bending Strength Distribution

Since the specimens were defect-free and straight-grained, only the general adjustment factors for clear wood properties from Table 8 of ASTM D245 were used to convert the clear wood properties. The design values were divided by the ASTM D245 adjustment

factors for clear wood, which are 1.9, 2.1 and 0.94 for compression strength, bending strength and Modulus of Elasticity respectively (ASTM International, 2006). These factors adjust the clear specimen properties for normal load duration and account for a factor of safety. The adjustment factor for Modulus of Elasticity adjusts the design value obtained from small specimens with a span-depth ratio of 14 to 21 and assumed uniformly distributed loading, rather than a point load. Note that CSA O86 Select Structural Grade wood contains knots, as per NLGA grading guidelines, whereas the design values from ASTM D2555 and the present study are for defect-free wood (Canadian Standards Association & Canadian Wood Council, 2015; National Lumber Grading Association, 2017). For comparison with CSA O86, all design values have been adjusted to 19% MC using eq. (4.13) from the Forest Product Laboratory Wood Handbook (Glass & Zelinka, 2010).

$$P = P_{12} \left(\frac{P_{12}}{P_g} \right)^{\left(\frac{12-MC}{MC_P-12} \right)} \quad (4.13)$$

Where:

MC = the moisture content

P = the given material property at MC

P_{12} = the given material property at 12% MC

P_g = the given material property at the FSP

MC_P = the intersection moisture content

Table 4.8: Design Values, Adjusted to 19% MC

Property	CSA O86	ASTM D2555	Current Study
f_b (MPa)	10.6	15.5	10.1
f_c (MPa)	13.0	9.9	14.8
E (MPa)	7500	9833	6746
G (MPa)	0.35	0.36	0.35

The fifth percentile experimentally-determined compression strength was roughly 14% higher than the CSA O86 design value and 49% higher than the design strength determined by adjusting the ASTM D2555 material properties for Eastern white pine according to ASTM D245. The experimental bending strengths had a high dispersion, which has resulted in a low fifth percentile strength. The design value for bending strength determined in this study was about 5% less than the CSA O86 design value and 35% lower than the value based on the ASTM D2555 material properties. The experimental MoE determined from the bending specimens was about 10% lower than the design value from CSA O86 and 32% below the average value from ASTM D2555.

The sample studied in this research included a wide range of growth ring densities with varying proportions of earlywood and latewood. In historic wooden structures built from old growth wood, the Ring Density (RD) of the wood is likely higher than the average values seen in this study (6.2 rings per inch). As shown previously in table 4.7, the specimens with higher RD were generally stronger, with design strengths 5% to 10% higher than the average design values for the entire sample. In historic wooden structures built from dense old growth wood, the design values provided in CSA O86 are more likely to be conservative.

4.4.3 Feasibility and Limitations of In-Situ Resistance Drilling

Resistance Drill Measurement (RM) is a strong indicator of wood density but there are limitations to in-situ material characterization, as this technique was not originally developed for NDT material characterization. Compression strength and stiffness are global properties which are related to density. Resistance drilling may be used to obtain an indication of the compressive strength and Modulus of Elasticity, however these relationships are not strong enough to permit in-situ material characterization. Further testing campaigns may improve the correlation between RM, compressive strength, and MoE.

Bending and tensile strengths are local material properties; these failure modes are controlled by small random material defects in the wood (A. O. Feio, 2005; Piazza & Riggio, 2008). For this reason, bending failure of wood is often brittle and sudden; it would be nearly impossible to drill a bending member at the precise location of the failure-controlling defect. Furthermore, while moderate correlations were determined for the two primary failure modes seen in this test campaign, it is not possible to easily determine the failure mode of in-situ wood members. Therefore, resistance drill testing is not a suitable method for in-situ determination of bending strength.

Moisture Content did not appear to affect the resistance drill's ability to determine the basic specific gravity of a specimen. Therefore, this instrument could be used to estimate the clear density of structural members where moisture content is not controlled. Moisture content can be measured in-situ by using a hand held moisture meter. The location and number of defects present in large structural members will also influence the overall density and strength of full-scale structural timber. ASTM D245 can be used to account for visible defects in existing wood members.

Another factor that influences the Resistance Drill Measurement on site is the growth ring orientation. Resistance Drill Measurement is most accurate in the radial direction. Resistance drills were originally developed for the evaluation of standing trees; for this application, the drilling measurement is always approximately in the radial direction, perpendicular to growth rings (Rinn et al., 1996). Depending on the cutting method used to process trees into lumber, the growth rings in a member may be oriented diagonally from the surface of the wood. In some historic timber structures, it may not be possible to drill a piece of timber in the radial direction, which will reduce the accuracy of the Resistance Drill Measurement.

5. Case Study: Assessment of an Historic Wooden Barn

The following case study of a historic wooden barn applies the relationships determined in chapter 4 to characterize the material properties of an existing building. The iterative structural analysis presented in this chapter demonstrates the utility of Non-Destructive Testing such as Resistance Drill Measurement and visual inspection in evaluating existing structures. The case study of a historic barn is presented to establish a workflow for the in-depth evaluation of historic wood structures through on-site Non-Destructive Testing, using the experimentally-determined relationships between material properties and Resistance Drill Measurement from this study. The workflow used in this case study includes the following five phases:

1. Background Research
2. Building Documentation
3. Condition Assessment
4. Material Characterization
5. Iterative Structural Analysis

5.1 Background

Building 95, also known as Meadowview Barn, is a historic barn constructed around 1912 on the Central Experimental Farm in Ottawa, Canada, see fig. 5.1. Building 95 is associated with the development of hygienic milk production and science-driven livestock advances in Canada, which occurred throughout the early 1900s. The building was first

used by the Animal Husbandry Division as a dairy barn, until the 1970s when the dairy cows were housed elsewhere. In the 1980s the Agricultural Engineering Institute used the building to perform stress tests on the roof trusses and in 1988 the building was re-purposed as a sheep showcase (Federal Heritage Buildings Review Office, 1989). The barn is now included in the Canada Agriculture and Food Museum, established in 1983, which is part of Ingenium: Canada's Museums of Science and Innovation.

Figure 5.1: Exterior View of Building 95

The two-story wood-framed building sits on a concrete foundation. The building is iconic for its gambrel roof shape, red board-and-batten siding, and large windows (Canadian Register of Historic Places, 2008). The ground floor is used for event rentals and summer camps; whereas the second story, formerly the hayloft, is now only accessible to staff. The focus of this study was the wooden roof structure, accessible from the second floor.

5.2 Documentation

The documentation process of Building 95 aimed to create a Building Information Model (BIM) of the structure from which plans, section drawings and elevations were created. Documentation is an important step in the conservation-decision making process as it creates a record of the building at a particular moment in time. The documentation of Building 95 included a measured survey of the structure, as well as building digitization through photogrammetry and laser scanning.

Figure 5.2: Building 95 BIM

Figure 5.3: Photogrammetric Model

Elevation drawings of Building 95 were prepared from the terrestrial photogrammetry model, shown in fig. 5.3. These drawings were used to note damages on the building exterior and identify patterns of decay or deterioration. The 3D BIM shown in fig. 5.2 was built from existing laser scanning data. Hand measurements of the structural members and the BIM were used to create a structural model of Building 95.

5.3 Condition Assessment

A preliminary assessment of the overall building was performed to evaluate the condition of the structural elements based on on-site observations (Parks Canada, 2010). The condition levels were defined as follows:

- **Good:** The element shows no observable signs of deterioration and is at minor risk of deterioration under normal service conditions.
- **Fair:** The element shows some signs of deterioration and poses a minor risk of short-term deterioration under normal service conditions.
- **Poor:** The element displays signs of severe deterioration and is at serious risk of deterioration or loss under normal service condition.

The trussed roof structure, shown in fig. 5.4, was examined on foot from the second floor. Scaffolding below the two south-most trusses allowed the entirety of these trusses to be examined; the remaining four trusses were evaluated from the ground and assumed to be in a similar level of condition.

Overall, the roof structure was in good condition, with localized elements in fair condition. Many of the roof boards were stained due to moisture, see fig. 5.5. The current metal roof was installed around 1997 and it is likely that some of the damaged roof boards were replaced during this intervention. Since the new boards did not show visual signs of moisture staining, it appears that moisture infiltration issues at the roof were resolved during the 1997 intervention.

The trusses were found to be in good condition overall, with some minor moisture stains, but no visual signs of rot or insect damage. Qualitative resistance drill testing was used to detect voids and weakness within the members, but the members were found to be in good condition. No signs of overloading, such as excessive deflection, splitting, or crushing were seen in the structural members.

Figure 5.4: Interior View of Building 95 Roof Structure

Other minor defects, such as notches and seasoning checks were seen in the structure. The few notches were relatively small and away from areas of high stress in the truss. Seasoning checks occur naturally as wood dries to Equilibrium Moisture Content (EMC). They typically reach their maximum size within a few years of service as the wood dries and stabilize, unless subjected to significant changes in relative humidity (R. Anthony & DeStefano, 2018). Since checks do not reduce the cross-sectional area of a timber, they have little impact on axial strength. They do impact shear strength, however this effect is accounted for by the adjustment factors used in ASTM D245 to develop design values.

Figure 5.5: Replaced Roofing Boards and Previous Staining

5.4 Material Characterization

5.4.1 Species Identification

The wood species in Building 95 was identified through macroscopic observation, without the use of a hand lens. A utilitarian farm building constructed in the early twentieth century would have been constructed from locally available wood (Bridge, 2012). The wood used in Building 95 was identified as softwood due to the lack of pores in the endgrain; additionally, the wood was easily scratched when a light pressure was applied with a fingernail (Hoadley, 1990).

Within the conifer, or *pinophyta*, division, several macroscopic features can be used to identify wood species (Wiemann, 2010). The features used to identify the wood species in Building 95 are shown in table 5.1.

Table 5.1: Features Used for Softwood Identification

Feature	Description	Identification
Resin Canals	Small secretions of pitch or resin between cells	Size, spacing, and number are used to distinguish softwoods
Tracheids	Long thin cells in the longitudinal direction of the trunk	Contribute to coarseness of wood texture
Growth Ring Transition	Abrupt or smooth transition between the earlywood and latewood	Rate of transition creates distinct grain patterns
Grain Contrast	Colour contrast between earlywood and latewood	Creates distinct colour patterns

Figure 5.6: Exposed Wood

Figure 5.7: End Grain

The wood appeared to be reddish brown in colour. However exposed grain and notches revealed that the wood was stained; the natural colour was a light brown with a yellow tint, see fig. 5.6. The end grain had been stained throughout the structure, making it difficult to inspect the growth features. In a few areas the wood had been cut, revealing the end grain and resin canals, see fig. 5.7. Resin canals are seen in most softwood species; however, the distinctly large, numerous, and evenly distributed resin canals seen in Building 95 are unique to the pine family (Hoadley, 1990).

The species of pine native to the Southeastern Ontario are:

- Eastern white pine (*pinus strobus*)
- Jack pine (*pinus banksiana*)
- Red pine (*pinus resinosa*)
- Pitch pine (*pinus rigida*)

Figure 5.8: Eastern White Pine End Grain

Pitch pine is not native to the Ottawa valley; it has a very small growth region along the St Lawrence river and prefers sandy soils and harsh conditions (Ontario Ministry of Northern Development, Mines, Natural Resources and Forestry, 2021). Red pine has a white-red colour, which does not match the species seen in Building 95 (Wiedenhoeft, 2010). Both Jack pine and Eastern white pine correspond to the yellow-brown colour of the wood observed on site. Jack pine has medium resin canals, an abrupt earlywood to

latewood transition and a moderate to high colour contrast (Wiedenhoeft, 2010). Eastern white pine has larger resin canals a gradual earlywood to latewood transition and low colour contrast, see fig. 5.8 (Wiedenhoeft, 2010). The low contrast of the growth rings, shown in fig. 5.7, suggests that the wood used in Building 95 is Eastern white pine. Furthermore, while Jack pine may presently be included with lower grades of white and red pine for structural applications, it has historically been more significant to the pulp and paper industry rather than construction (Rudolph, 1985). Based on the species identification, the clear specimen design strengths were taken from ASTM D2555 for Eastern white pine.

Note that while every effort was made to accurately identify the wood species, the wood was only identified by the unaided eye; microscopic evaluation by certified wood scientists did not take place. Furthermore, only select members were accessible for close visual inspection. For the purposes of this research, it is assumed that the entire structure was constructed from Eastern white pine; however, it is possible that multiple species were used. The goal of this case study is to showcase a methodology for the assessment and analysis of historic timber structures. Should an in-depth species investigation of Building 95 be required, it is recommended that samples of the wood be sent to a research laboratory for microscopic evaluation by certified wood scientists.

5.4.2 Visual Grading

The lumber was visually assessed to determine the grade based on the 2017 National Lumber Grading Authority (NLGA) Standard Grading Rules for Canadian Lumber (National Lumber Grading Association, 2017). As discussed in chapter 2, the main defects that affect wood strength and grade are knots and grain slope (R. W. Anthony et al., 2009; Cruz et al., 2015). These were measured using a clear plastic sheet with a 13 mm (1/2") grid.

Figure 5.9: Knot Measurement Using Plastic Grid

The structural members were 50 mm by 150 mm (2" by 6") structural light framing. The maximum allowable sizes for edge knots, face knots and unsound knots or holes are 29 mm, 48 mm, and 25 mm ($1\frac{1}{8}$ ", $1\frac{7}{8}$ " and 1"), respectively for Select Structural wood. Based on the knot and slope of grain measurements, the wood structure in Building 95 exceeded the requirements for Select Structural lumber. Splits, wane, and shakes are also grade-limiting defects, but these defects were not seen in the trusses.

The sizes of the knots and slope of grain for each member were recorded. This allowed the clear specimen properties to be adjusted according to ASTM D245 for full-size members with defects. Although it requires a further investigation, this standard is more accurate and usually less conservative than the CSA O86 material properties that would typically be used in a structural evaluation.

The wood roof structure was neither insulated nor sealed, as evidenced by the bird nests in the upper portion of the structure. Only the ground floor of Building 95 is heated in the winter, which keeps the structure slightly warmer than the outside air temperature. Based on average climactic data for Ottawa, the Equilibrium Moisture Content (EMC) of the structure was estimated to be around 12%.

5.5 Structural Evaluation

An analytical model of a representative truss from Building 95 was created using Dlubal RFEM 5 software (Dlubal Software, Inc, 2021); the analysis method was linear and did not account for second order effects. The structural geometry was obtained from the BIM of the building and site-verified hand measurements, see fig. 5.10. The truss was evaluated under the combined effects of gravity loading, which included dead, live and snow loads, calculated as per NBC 2015. The loads were applied as line loads to the top chord of the truss based on the tributary width of the truss. Pinned connections were assumed at the ends of members; continuous members were modelled with pin connections at the ends and hinges at intersections with other members.

Structural analysis of historic buildings is typically an iterative process. Generally, the first iteration is the most conservative, and therefore the least accurate. Based on the results of the evaluation, the assumptions can be modified for the proceeding iterations, with increasing accuracy. The following sections detail the iterative analysis procedure used to determine the structural performance of Building 95. The first iteration uses material properties from CSA O86 with NBC 2015 loading and follows a Limit States Design approach (Canadian Commission on Building and Fire Codes, 2015a; Canadian Standards Association, 2014). The second iteration employs the same approach, but with material properties specific to Eastern white pine from ASTM D2555, adjusted by ASTM D245 (ASTM International, 2006, 2017a). The third iteration modifies the fifth percentile

Figure 5.10: Analytical Model of Building 95

design values based on inspection information, as per ISO 13822:2010 (International Organization for Standardization, 2010).

The analytical model was used to determine the internal forces in each member under factored loading, the member resistances were calculated in each iteration using equations from chapter 6 of CSA O86-14 and compared to the internal forces to obtain a Demand-Capacity Ratio (DCR) (Canadian Standards Association & Canadian Wood Council, 2015). The full analysis procedure and results for each iteration may be found in chapter C.

5.5.1 Iteration 1: CSA 086-14 Material Properties

The truss from Building 95 was first evaluated using Limit States Design (LSD), where the factored demand on each member was compared to the factored capacity to obtain a Demand-Capacity Ratio (DCR). For the first iteration, the material properties for Select

Structural grade Northern lumber were used to calculate the factored resistance of each member, as per CSA O86-14, Table 6.3.1A, see table 5.2 (Canadian Standards Association, 2014).

Table 5.2: Specified Strengths from CSA O86-14, Table 6.3.1A, in MPa

f_b	f_v	f_c	f_t	E	E_05
10.6	1.3	13.0	6.2	7500	5500

The resistances of each member were calculated by using the design equations provided in CSA O86-14 (Canadian Standards Association, 2014).

When the sixteen truss members were evaluated using material properties from CSA O86, three members failed in bending and two failed in compression, see table 5.3. The bending members were overloaded by up to 71%, whereas the demand on the compression members exceeded the capacity by less than 5%. None of the members were found to be overloaded in tension or shear. The members were also evaluated under combined bending and axial load, which found that four members failed.

Figure 5.11: Failed Members Using CSA O86-14 Design Values

Table 5.3: Demand-Capacity Ratios for First Iteration

Member	Compression DCR (%)	Bending DCR (%)	Combined (%)
1.1	15	68	70
1.2	117	19	166
1.3	8	0	1
1.4	17	0	3
1.5	0	0	0
1.6	40	0	16
1.7	0	0	0
1.8	2	0	0
2.1	57	171	208
2.2	115	63	203
2.3	0	0	0
2.4	0	0	0
3.1	0	92	92
3.2	57	171	205
3.3	11	117	118
3.4	38	1	16

5.5.2 Iteration 2: ASTM D2555 and ASTM D245 Material Properties

The second iteration of the structural analysis also followed a Limit States Design approach, but with material properties specific to the species of wood used in Building 95. The clear wood strength values for Eastern white pine from ASTM D2555 were used to determine the design value. The fifth percentile values were used for compression and bending strengths while the average Modulus of Elasticity was taken as the design value.

These clear wood design values were adjusted based on the maximum knots, slope of grain, Moisture Content, and Ring Density of the structural members, according to the strength ratios and adjustment factors provided in ASTM D245. These factors were used to calculate allowable design values for bending strength, compression parallel to grain strength, and Modulus of Elasticity as shown in table 5.4. The resistances of each member were calculated by using the ASTM strengths and the design equations provided in CSA O86-14 (Canadian Standards Association, 2014).

Table 5.4: Allowable Design Values for Building 95

Property	Clear Wood Design Value (MPa)	Size Factor	Defect Strength Ratio	MC Factor	Grain Density Factor	Design Value (MPa)
f_{c05}	12.6	0.526	0.81	2.01	1.17	12.6
f_{b05}	26.1	0.421	0.74	1.84	1.17	17.5
E	8108	1.064	1.00	1.16	1.05	10506
E_{05}	5170	1.064	1.00	1.16	1.05	6699

The clear wood design values were adjusted based on a maximum knot size of 32 mm and a slope of grain of 1:14, as measured on site. The design values shown in table 5.4 are the product of the five preceding columns. The size factor for bending adjusts for a member depth of six inches, which was the depth of the structural members measured on site.

When the Limit States Design (LSD) evaluation from section 5.5.1 was reiterated using fifth percentile ASTM design strengths adjusted based on table 5.4, only two members failed in bending with DCRs of 103% due to the increased design value. The ASTM strength for compression was slightly lower than the CSA O86 value, however, the increased value of E_{05} resulted in higher compression resistances; only one member was overloaded in compression. The same four members failed under combined loading, see fig. 5.12 and table 5.5.

Figure 5.12: Failed Members Using ASTM Design Strengths

Table 5.5: Demand-Capacity Ratios for Second Iteration

Member	Compression DCR (%)	Bending DCR (%)	Combined (%)
1.1	15	41	43
1.2	103	12	122
1.3	7	0	1
1.4	17	0	3
1.5	0	0	0
1.6	36	0	13
1.7	0	0	0
1.8	2	0	0
2.1	55	103	136
2.2	98	38	138
2.3	0	0	0
2.4	0	0	0
3.1	0	55	55
3.2	55	103	135
3.3	10	71	72
3.4	34	1	13

5.5.3 Iteration 3: Updated Material Property Distribution

ISO 13822:2010 outlines a procedure for updating measured quantities based on inspection results (International Organization for Standardization, 2010). The ASTM material properties from section 5.5.2 were updated according to ISO 13822:2010 based on the RM measured on site. In this iteration, the Limit States Design evaluation was repeated using the updated fifth percentile material properties. The resistances of each member were

calculated by using the design equations provided in CSA O86-14 (Canadian Standards Association, 2014).

The mean and standard deviation for compressive strength, bending strength and Modulus of Elasticity from ASTM D2555 were adjusted based on the positive correlation with Resistance Drill Measurement. By using the ISO 13822:2010 principles, the standard deviation of the compression strength distribution was adjusted from 3.21 MPa to 2.58 MPa, as seen in eq. (5.1).

$$\begin{aligned}\sigma'_c &= \sigma_c \sqrt{1 - R_{fc-RM}^2} \\ \sigma'_c &= (3.21) \sqrt{1 - (0.60)} \\ \sigma'_c &= 2.03\end{aligned}\tag{5.1}$$

Where:

σ_c = the standard deviation of the compressive strength, given by ASTM D2555

R_{fc-RM}^2 = the coefficient of determination between fc and RM found experimentally in chapter 4

The standard deviation of the bending strength distribution was adjusted from 5.7 MPa to 4.8 MPa, as seen in eq. (5.2) (International Organization for Standardization, 2010).

$$\begin{aligned}\sigma'_b &= \sigma_b \sqrt{1 - R_{fb-RM}^2} \\ \sigma'_b &= (5.7) \sqrt{1 - (0.29)} \\ \sigma'_b &= 4.8\end{aligned}\tag{5.2}$$

Where:

σ_b = the standard deviation of the bending strength, given by ASTM D2555

R_{fb-RM}^2 = the coefficient of determination between f_b and RM found experimentally in chapter 4

The standard deviation of the MoE distribution was adjusted from 1786 MPa to 1072 MPa, as seen in eq. (5.3) (International Organization for Standardization, 2010).

$$\begin{aligned}\sigma'_E &= \sigma_E \sqrt{1 - R_{E-RM}^2} \\ \sigma'_E &= (1786) \sqrt{1 - (0.64)} \\ \sigma'_E &= 1072\end{aligned}\tag{5.3}$$

Where:

σ_E = the standard deviation of the MoE, given by ASTM D2555

R_{E-RM}^2 = the coefficient of determination between E and RM found experimentally in chapter 4

For the four members which failed under the previous iteration, the Resistance Drill Measurements were equal to or higher than the population average of 12.5% determined experimentally for Eastern white pine. The mean compressive strength, MoR and Modulus of Elasticity were adjusted as shown in eq. (5.4) and eq. (5.6), based on the average in-situ Resistance Drill Measurement (International Organization for Standardization, 2010).

$$\begin{aligned}\mu'_c &= \mu_c + R_{fc-RM} \sigma_c \left(\frac{RM - \mu_{RM}}{\sigma_{RM}} \right) \\ \mu'_c &= (17.9) + (\sqrt{0.6})(3.21) \left(\frac{RM - (12.5)}{(2.12)} \right)\end{aligned}\tag{5.4}$$

$$\begin{aligned}\mu'_b &= \mu_b + R_{fb-RM}\sigma_b \left(\frac{RM - \mu_{RM}}{\sigma_{RM}} \right) \\ \mu'_b &= (35.4) + (\sqrt{0.29})(5.7) \left(\frac{RM - (12.5)}{(2.12)} \right)\end{aligned}\tag{5.5}$$

$$\begin{aligned}\mu'_E &= \mu_E + R_{E-RM}\sigma_E \left(\frac{RM - \mu_{RM}}{\sigma_{RM}} \right) \\ \mu'_E &= (8108) + (\sqrt{0.64})(1786) \left(\frac{RM - (12.5)}{(2.12)} \right)\end{aligned}\tag{5.6}$$

Where:

μ_c = the average compressive strength, given by ASTM D2555

μ_b = the average bending strength, given by ASTM D2555

μ_E = the average MoE, given by ASTM D2555

R_{fc-RM} = the Pearson correlation coefficient between f_c and RM, taken as $\sqrt{R_{fc-RM}^2}$

R_{fb-RM} = the Pearson correlation coefficient between f_b and RM, taken as $\sqrt{R_{fb-RM}^2}$

R_{E-RM} = the Pearson correlation coefficient between E and RM, taken as $\sqrt{R_{E-RM}^2}$

σ_c = the standard deviation of the compressive strength, given by ASTM D2555

σ_b = the standard deviation of the bending strength, given by ASTM D2555

σ_E = the standard deviation of the MoE, given by ASTM D2555

RM = the average Resistance Drill Measurement value for the member

μ_{RM} = the average RM for Eastern white pine, determined in chapter 4

σ_{RM} = the standard deviation of RM for Eastern white pine, determined in chapter 4

The updated mean and standard deviation for compression strength, bending strength, and MoE of the failed members were modified according to ASTM D245, using the same factors from section 5.5.2. The material property distributions based on the in-situ Resistance Drill Measurement and fifth percentile values are shown in table 5.6. Note

that the fifth percentile design values, f'_{c05} , f'_{b05} and E'_{05} , shown in the table have been modified by the ASTM D245 factors used in section 5.5.2.

Table 5.6: Updated Properties Based on RM, in MPa

Member	RM	μ'_c	σ'_c	f'_{c05}	μ'_b	σ'_b	f'_{b05}	μ'_E	σ'_E	E'_{05}
1.2	13.6	19.2	2.03	15.9	37.0	4.8	19.6	8854	1072	9189
2.1	13.0	18.5	2.03	15.2	36.3	4.8	19.1	8498	1072	8727
2.2	12.6	18.0	2.03	14.8	35.7	4.8	18.7	8224	1072	8373
3.2	12.5	17.9	2.03	14.6	35.4	4.8	18.5	8108	1072	8222

The Demand-Capacity Ratios for the four members which failed in the second iteration were re-calculated based on the updated material properties, see table 5.7. The combined bending and axial load DCRs for members 1.2, 2.1 and 2.2 still exceeded the updated resistance by 17%, 2%, and 20% respectively, which indicates that these members require structural intervention. The DCRs in compression and bending all fell below 100% using the updated properties. The DCRs for the third iteration are shown in fig. 5.13.

Table 5.7: Demand-Capacity Ratios for Third Iteration

Member	Compression DCR (%)	Bending DCR (%)	Combined (%)
1.2	77	11	72
2.1	45	95	117
2.2	79	36	102
3.2	47	98	121

Figure 5.13: Failed Members Using Updated Characteristic Values

5.6 Structural Intervention

Although the roof trusses did not exhibit any visual signs of overloading, the structural analysis found that the bottom chord members 1.2 and 2.2 are overloaded in combined axial and bending. In order to ensure an adequate level of safety, these two members require structural intervention to support the applied loads. By adopting an iterative

approach to structural evaluation, it was possible to refine the analysis assumptions based on inspection information.

Given the heritage nature of the building, any intervention to the structure must comply with the *Standards and Guidelines for the Conservation of Historic Places in Canada* (Parks Canada, 2010). Interventions must be compatible with, yet distinct from the original structure. In this case, the installation of dimensional lumber bracing between the overloaded members would reduce their span and provide lateral support against buckling. Such an intervention would be compatible as it uses the same material as the trusses, and distinct, as the new wood would be clearly distinguishable from the existing wood which has changed colour over time.

5.7 Assessment Conclusions

The condition assessment of Building 95 found that the wood structure was in good condition overall. Since wood strength does not decrease with age, it may be assumed that the roof trusses have the same strength properties as when they were first constructed. The structural analysis assumed the wood members to be in good condition, based on the results of the condition assessment. Should the condition worsen, the structure must be re-evaluated to account for any loss of strength due to material degradation. Furthermore, the building's condition should be monitored regularly to ensure that the material remains in good condition.

The iterative approach to structural analysis enabled the material properties to be refined in each iteration based on inspection information. In existing buildings, past performance provides an indication of the structure's capacity and resilience. This must be taken into consideration in order to reduce the loss of heritage fabric due to inappropriate interventions. While the CSA O86 material properties provide a good baseline for structural analysis, the properties given by ASTM D2555 are more accurate

since they are specific to each species (ASTM International, 2017a). By measuring wood defects on site, ASTM D245 can be used to determine appropriate design values of wood that exceeds the grading requirements for Select Structural, which is often the case in historic construction ASTM International, 2006.

While resistance drill testing cannot be used for direct material characterization, according to ISO 13822:2010 principles, the correlation between resistance drill measurement and strength values can be used to update the normal distribution of mechanical properties. This results in higher design strengths for wood that has a higher than average resistance drill measurement.

To preserve the heritage character of the building, any interventions to the structure must be compatible with the original materials, yet visually distinct upon close inspection (Parks Canada, 2010). By conducting an iterative analysis, it was possible in this case to follow a minimum intervention approach, as recommended by the *Standards and Guidelines for the Conservation of Historic Places in Canada* (Parks Canada, 2010) and the ICOMOS *Principles for the Conservation of Wooden Built Heritage* (ICOMOS, 2017).

6. Conclusions

6.1 Summary of Contributions

In Canada, wood was historically used in vernacular construction; conserving these structures provides a connection to the people who built them. The variability of wood provides a unique challenge to conservation, as this makes it difficult to determine the material properties without the use of destructive testing methods. In recent years, Non-Destructive Testing (NDT) methods have become instrumental in the conservation of historic wood buildings as these can be used to determine some properties of wood without causing significant damage to the historic building fabric.

The correlation between Resistance Drill Measurement (RM) and specific gravity established in chapter 4 was found to be roughly 80%, therefore this method may be used in-situ to indicate the density of Eastern white pine within a reasonable degree of accuracy. Resistance Drill Measurement was moderately correlated with compressive strength and Modulus of Elasticity, with correlation coefficients of 60% and 64%, respectively. However, while RM may provide some indication of the compressive strength and stiffness of a piece of wood, the variability present in full scale timbers render this method unsuitable for direct material characterization. Variability due to growth defects in full size members is expected to reduce the strength of the correlation. Additionally, this experimental campaign made every effort to drill the specimens in the radial direction, which has shown to provide the most accurate Resistance Drill Measurements. In some existing structures, it may not be possible to drill a piece of timber in the radial direction if the growth rings are oriented diagonally from the surface of the wood; this will reduce the accuracy of the Resistance Drill Measurement.

The combination of NDT methods, such as Pylodin testing, proof loading, or ultrasonic tomography, with RM may result in more accurate material characterization. The results of the bending tests show that Resistance Drill Measurement is not suitable for determination of the bending strength of wood. This is due to the brittle failure modes of wood in bending, which are often governed by small local defects.

The workflow outlined in chapter 5 highlights the importance of condition assessments and visual inspection of historic wood structures. The condition of wood will have a larger impact on the strength than most observable defects (Cruz et al., 2015). Non-Destructive Testing of Building 95 has demonstrated the utility of resistance drill testing for qualitative assessment of historic timber structures. Species identification allows for material properties from ASTM D2555 to be used, which are less conservative than the design values provided in CSA O86. Since the values provided are for small clear specimens, they must be adjusted based on visual grading according to ASTM D245 to represent the mechanical properties of full scale timbers.

Although Resistance Drill Measurement may not be used for direct in-situ material characterization, this tool may be used to modify material properties used in a probabilistic structural analysis, according to ISO 13822:2010 (International Organization for Standardization, 2010). Iterative structural analysis allows for assumptions to be refined and less conservative design values to be used, based on inspection information.

In many cases, spending more time and resources on condition assessment and structural evaluation can be beneficial as it may result in savings on construction costs related to the implementation of structural interventions. Furthermore, historic buildings can be more sustainable than new construction, due to their lower embodied energy of the course of their life-cycle (Larsen & Marstein, 2000). Numerous recognized conservation guidelines, such as the *Standards and Guidelines for the Conservation of Historic Places in Canada* (Parks Canada, 2010), the Venice Charter (ICOMOS, 1964) and other doctrinal texts developed by the ICOMOS Wood Council (ICOMOS, 1998, 2003) recognize the

importance of understanding a historic place prior to intervening. The work presented in this thesis demonstrates the utility and limitations of Non-Destructive Testing and how Resistance Drill Measurement may be used to improve current understanding of historic wood buildings.

6.2 Further Work

While this study focused solely on Eastern white pine, there is a lack of similar research on other common North American species of wood. Future work may include applying the methodology described in chapter 3 to other North American softwood species to determine whether the strong relationship between RM and specific gravity is species-dependent. The establishment of standardized population values for Resistance Drill Measurement of North American species by large-scale testing campaigns would allow for the fifth percentile material properties for a known species to be modified according to ISO 13822:2010, based on in-situ NDT; this would facilitate probabilistic analysis of existing wood buildings.

Additionally, full-scale verification was not included in the scope of this study. Although the effect of load duration, size, and wood defects may be accounted for by ASTM D245 when determining appropriate design strengths, correlation between RM and experimentally determined strengths should be carried out on structural-size timbers. Furthermore, the effect of Moisture Content above 8% has not been established in the present study. This effect should be studied in the future, as it pertains to historic structures which have variable moisture contents.

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Appendices

A. Experimental Compression Data

The laboratory data obtained for the compression specimens is shown in table A.1 and table A.2 on the following pages. The stress-strain relationships shown in fig. A.1 and fig. A.2 illustrate the range of behavior seen in the compression sample at the two different moisture contents.

Figure A.1: Stress Strain Curves for Air-Dry Sample

Figure A.2: Stress Strain Curves for Oven-Dry Sample

Table A.1: Air Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-001	45.25	50.85	200.00	0.33	4.86	12.4	59739	25.96	0.411	0.808	3920
EWP-C-002	46.99	50.34	199.00	0.35	6.22	12.1	70158	29.66	0.446	0.901	3890
EWP-C-003	41.45	47.95	200.00	0.33	6.64	13.7	60200	30.29	0.266	0.667	4535
EWP-C-004	45.04	51.24	201.00	0.34	5.08	12.6	68567	29.71	0.503	0.939	4108
EWP-C-005	45.80	48.32	201.00	0.33	5.47	17.3	52533	23.74	0.410	0.864	3153
EWP-C-006	45.89	47.80	201.00	0.33	6.54	11.9	44511	20.29	0.363	0.761	3075
EWP-C-007	47.00	51.39	202.00	0.34	5.40	12.7	65121	26.96	0.490	0.960	3478
EWP-C-008	45.26	51.40	201.00	0.34	4.86	9.9	70741	30.41	0.403	0.855	4050
EWP-C-009	45.11	51.00	201.00	0.31	4.48	13.3	55025	23.92	0.388	0.752	3965
EWP-C-010	46.55	51.01	201.00	0.34	5.18	12.6	54555	22.98	0.398	0.837	3159
EWP-C-011	45.70	50.40	202.00	0.36	9.07	11.3	68822	29.88	0.499	0.956	3955
EWP-C-012	45.54	51.22	202.00	0.33	5.50	12.9	72074	30.90	0.547	1.014	4006
EWP-C-013	45.70	51.84	199.00	0.32	5.60	12.4	65778	27.76	0.469	0.968	3322
EWP-C-014	49.89	50.51	200.00	0.36	5.69	12.8	73192	29.05	0.589	1.074	3590
EWP-C-015	47.16	50.40	201.50	0.33	5.90	12.3	73956	31.12	0.552	1.034	3900
EWP-C-016	45.30	50.22	200.00	0.39	6.03	9.2	82890	36.44	0.627	1.210	3748

Table A.1: Air Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-017	45.80	49.69	200.50	0.36	5.52	10.2	79006	34.72	0.489	0.952	4512
EWP-C-018	44.21	50.51	200.00	0.32	5.70	16.9	57897	25.93	0.421	0.822	3872
EWP-C-019	46.11	50.99	200.00	0.30	4.45	12.5	62060	26.40	0.506	0.986	3298
EWP-C-020	53.09	50.11	204.00	0.40	7.96	10.9	105000	39.47	0.290	0.833	4448
EWP-C-021	51.29	50.51	203.00	0.36	5.54	14.7	75861	29.28	0.464	0.914	3960
EWP-C-022	53.70	50.36	203.00	0.41	9.11	14.7	103080	38.12	0.505	0.979	4891
EWP-C-023	53.71	50.74	203.00	0.41	9.30	16.9	105001	38.53	0.571	1.134	4168
EWP-C-024	53.65	51.10	203.50	0.42	9.47	10.5	106400	38.81	0.538	1.019	4930
EWP-C-025	53.95	50.65	203.00	0.42	10.46	15.7	103297	37.80	0.601	1.086	4748
EWP-C-026	51.91	51.24	203.00	0.36	7.85	13.8	74886	28.15	0.755	1.272	3316
EWP-C-027	51.86	50.42	203.00	0.35	8.77	11.4	75447	28.85	0.610	1.089	3673
EWP-C-028	47.10	50.75	201.00	0.34	5.82	11.6	76299	31.92	0.567	1.055	3944
EWP-C-029	45.59	50.84	199.00	0.31	4.99	12.1	70606	30.46	0.443	0.890	4065
EWP-C-030	45.55	50.90	199.00	0.31	4.99	13.4	67049	28.92	0.548	1.002	3801
EWP-C-031	45.91	50.99	202.00	0.33	7.34	12.3	69466	29.67	0.657	1.158	3590
EWP-C-032	41.30	50.24	200.00	0.33	6.47	11.5	79561	38.34	0.474	0.940	4931

Table A.1: Air Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-033	45.55	50.16	198.00	0.38	5.77	12.0	88059	38.54	0.593	1.090	4599
EWP-C-034	50.21	50.71	200.50	0.34	6.60	11.1	73673	28.94	0.522	0.998	3659
EWP-C-035	46.00	50.10	201.00	0.35	6.88	12.6	76431	33.16	0.517	0.974	4382
EWP-C-036	45.70	51.05	201.00	0.33	4.48	11.4	70812	30.35	0.408	0.867	3989
EWP-C-037	45.96	50.39	201.00	0.33	5.08	9.6	84311	36.40	0.571	1.061	4478
EWP-C-038	41.09	50.90	198.00	0.38	5.60	11.3	87166	41.68	0.520	1.008	5078
EWP-C-039	45.76	50.21	201.00	0.34	6.74	12.6	81208	35.34	0.503	0.979	4473
EWP-C-040	46.67	50.41	202.00	0.33	6.21	11.5	83539	35.51	0.588	1.072	4447
EWP-C-041	46.41	50.32	200.00	0.34	5.82	10.0	67895	29.07	0.540	1.030	3558
EWP-C-042	45.88	50.26	200.00	0.32	4.57	14.0	51694	22.42	0.428	0.831	3337
EWP-C-043	46.54	51.01	199.00	0.30	5.52	9.4	43644	18.38	0.332	0.709	2914
EWP-C-044	45.10	51.55	202.50	0.30	6.89	15.6	52244	22.47	0.469	0.855	3530
EWP-C-045	46.28	50.84	200.50	0.37	5.64	14.4	59992	25.50	0.402	0.775	4119
EWP-C-046	46.92	51.10	202.00	0.35	5.69	10.4	57259	23.88	0.501	0.976	3044
EWP-C-047	45.65	48.06	201.00	0.33	6.45	10.3	57609	26.26	0.535	0.987	3502
EWP-C-048	46.95	50.94	202.00	0.30	3.12	17.5	50859	21.27	0.397	0.872	2712

Table A.1: Air Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-049	46.23	47.52	200.00	0.33	6.64	13.0	63779	29.03	0.476	0.895	4155
EWP-C-050	46.00	50.94	201.50	0.34	4.93	16.0	70117	29.92	0.487	0.916	4221
EWP-C-051	45.67	51.00	201.00	0.34	5.00	15.9	70475	30.26	0.663	1.168	3612
EWP-C-052	46.47	50.55	203.00	0.29	2.89	16.2	48087	20.47	0.511	0.956	2802
EWP-C-053	45.95	50.14	202.00	0.36	6.21	16.5	66099	28.69	0.619	1.023	4297
EWP-C-054	45.86	51.39	202.00	0.36	8.00	11.8	72058	30.58	0.437	1.000	3290

Table A.2: Oven Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-055	45.52	50.60	201.00	0.35	7.11	11.8	79282	34.42	0.494	0.924	4828
EWP-C-056	45.51	49.85	201.00	0.33	5.82	12.4	89743	39.56	0.450	1.026	4141
EWP-C-057	44.30	49.50	201.00	0.36	6.74	10.1	101009	46.06	0.535	1.121	4741
EWP-C-058	44.76	49.48	199.00	0.38	5.86	10.3	105001	47.41	0.578	1.173	4756
EWP-C-059	41.55	50.52	200.00	0.32	5.24	11.4	83745	39.90	0.677	1.382	3397
EWP-C-060	46.43	47.39	200.50	0.36	5.29	11.5	105001	47.72	0.434	1.001	5058
EWP-C-061	46.05	50.46	201.00	0.33	4.52	15.7	89772	38.63	0.462	1.102	3640

Table A.2: Oven Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-062	45.31	48.90	200.00	0.35	5.77	11.2	99897	45.09	0.692	1.284	4564
EWP-C-063	45.40	50.85	199.50	0.30	3.81	12.5	75046	32.51	0.618	1.322	2767
EWP-C-064	46.25	50.69	201.00	0.34	6.47	11.0	97812	41.72	0.552	1.225	3737
EWP-C-065	45.25	50.80	202.00	0.30	4.37	11.9	84768	36.88	0.456	1.003	4083
EWP-C-066	46.05	48.06	198.50	0.37	5.70	15.5	105001	47.44	0.578	1.173	4748
EWP-C-067	44.55	50.39	201.00	0.36	8.64	12.4	105000	46.77	0.490	1.037	5155
EWP-C-068	45.70	49.56	199.00	0.35	5.64	11.4	105000	46.36	0.367	0.890	5296
EWP-C-069	45.88	50.23	202.00	0.31	3.11	11.8	75864	32.92	0.584	1.292	2819
EWP-C-070	44.43	49.89	200.00	0.33	5.56	9.0	99056	44.69	0.456	1.009	4854
EWP-C-071	44.32	50.30	202.00	0.38	7.26	9.2	116591	52.30	0.613	1.167	5722
EWP-C-072	46.74	50.34	201.00	0.33	6.68	8.8	89531	38.05	0.562	1.191	3644
EWP-C-073	46.52	49.41	200.00	0.36	5.52	12.3	107260	46.66	0.571	1.134	4974
EWP-C-074	45.60	50.00	193.00	0.33	5.39	11.0	96378	42.27	0.429	0.979	4444
EWP-C-075	44.40	45.50	200.00	0.37	7.11	10.1	105000	51.98	0.415	0.957	5749
EWP-C-076	44.47	50.00	201.00	0.33	5.41	9.0	99191	44.61	0.380	0.990	4410
EWP-C-077	44.99	49.99	198.00	0.38	6.15	10.8	105001	46.69	0.434	1.001	4886

Table A.2: Oven Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-078	44.90	50.45	199.00	0.40	5.29	11.5	105000	46.35	0.415	0.957	5101
EWP-C-079	45.02	51.60	202.00	0.34	5.24	11.0	103703	44.64	0.619	1.255	4249
EWP-C-080	46.11	49.94	202.00	0.36	5.70	8.5	82684	35.91	0.325	0.809	4488
EWP-C-081	44.98	50.05	202.00	0.33	5.93	12.6	90266	40.10	0.403	0.928	4625
EWP-C-082	44.99	50.02	200.00	0.36	8.77	11.8	105000	46.66	0.415	0.957	5161
EWP-C-083	45.56	49.97	200.00	0.34	5.82	12.2	91373	40.14	0.422	1.028	3970
EWP-C-084	45.02	50.47	200.00	0.38	6.64	10.2	91370	40.21	0.555	1.301	3235
EWP-C-085	45.00	50.05	202.00	0.33	5.54	15.1	94173	41.81	0.475	1.033	4540
EWP-C-086	45.59	49.90	200.00	0.36	5.47	14.7	90012	39.57	0.616	1.300	3471
EWP-C-087	45.01	50.99	200.00	0.30	3.39	9.7	85117	37.09	0.585	1.128	4093
EWP-C-088	44.99	48.31	198.00	0.38	6.25	13.4	105001	48.31	0.432	0.978	5248
EWP-C-089	44.45	50.01	200.00	0.34	5.60	12.9	97713	43.96	0.421	0.942	5060
EWP-C-090	44.95	51.10	198.00	0.40	5.08	11.8	105001	45.71	0.358	0.887	5129
EWP-C-091	44.92	50.05	201.00	0.37	8.77	8.0	80749	35.92	0.434	0.872	4942
EWP-C-092	45.02	50.96	200.00	0.34	5.60	11.3	100680	43.88	0.385	0.922	4902
EWP-C-093	46.13	50.58	199.50	0.30	3.95	9.3	76709	32.88	0.447	1.040	3318

Table A.2: Oven Dry Compression Data

Specimen	b (mm)	d (mm)	L (mm)	SG	RD (/in)	RM	P_{max} (N)	f_c (MPa)	Δ_{10}	Δ_{40}	E (MPa)
EWP-C-094	46.12	51.11	202.00	0.32	5.69	14.0	93399	39.62	0.610	1.231	3868
EWP-C-095	45.36	50.41	201.50	0.32	4.99	12.4	93244	40.78	0.530	1.264	3360
EWP-C-096	46.20	49.62	200.00	0.35	4.23	13.1	93377	40.73	0.441	0.987	4472
EWP-C-097	46.39	48.95	199.00	0.35	6.00	9.8	105001	46.24	0.397	0.944	5044
EWP-C-098	46.69	48.80	198.00	0.35	5.59	11.7	102981	45.20	0.436	1.000	4762
EWP-C-099	43.94	50.80	201.00	0.31	6.00	15.4	92241	41.32	0.435	0.992	4468
EWP-C-100	45.49	46.50	200.50	0.34	7.01	10.6	83322	39.39	0.448	1.080	3753
EWP-C-101	44.12	49.43	199.00	0.40	6.60	14.9	105000	48.15	0.290	0.833	5293
EWP-C-102	45.68	50.00	201.00	0.29	3.39	12.7	80517	35.25	0.322	0.863	3928
EWP-C-103	43.99	50.66	201.50	0.36	6.97	15.2	105000	47.12	0.367	0.890	5450
EWP-C-104	46.55	48.87	200.50	0.35	5.29	12.6	105000	46.16	0.415	0.957	5118
EWP-C-105	50.65	49.10	203.00	0.36	8.79	15.5	101196	40.69	0.435	1.069	3906
EWP-C-106	52.50	49.79	204.00	0.40	8.47	15.9	140468	53.74	0.368	0.962	5540
EWP-C-107	50.45	49.31	203.00	0.36	8.79	13.1	105833	42.54	0.356	0.963	4266
EWP-C-108	51.59	49.91	203.50	0.35	8.79	11.8	105001	40.78	0.397	0.944	4549
EWP-C-109	50.60	49.55	204.00	0.35	7.82	10.8	109970	43.86	0.411	0.808	6755

B. Experimental Bending Data

The laboratory data obtained for the bending specimens is shown in the following table. The graph shown in fig. B.1 illustrates the range of load-deflection data seen in the sample group for bending.

Figure B.1: Load Deflection Curves for Bending Sample

Table B.1: Bending Test Data

Specimen	RM	RD (/in)	G_0	P_{ult} (N)	Δ (mm)	$I(mm^4)$	$S(mm^3)$	f_b (MPa)	E (MPa)	Fail mode
EWP-B-001	12.4	8.47	0.36	3926.389	8.658177	556767	21804	31.92	5993	cross-grain
EWP-B-002	12.1	6.47	0.31	6767.036	16.84163	536058	21209	56.61	6571	simple tension
EWP-B-003	13.7	5.82	0.36	7318.972	20.8999	395623	17145	75.73	8585	simple tension
EWP-B-004	12.6	9.96	0.35	4291.261	8.975002	557607	21752	34.98	6474	splinter tension
EWP-B-005	17.3	11.98	0.41	12476.55	21.96684	653915	24246	91.32	9767	shear
EWP-B-006	11.9	5.08	0.33	5830.511	19.05019	372025	16631	62.21	7083	shear
EWP-B-007	12.7	9.77	0.35	4200.743	8.991564	564068	21927	33.97	6271	splinter tension
EWP-B-008	9.9	3.46	0.30	3548.668	14.65829	523441	20896	30.11	3423	cross-grain
EWP-B-009	13.3	7.47	0.36	7222.053	22.79164	359547	16343	78.39	8940	shear
EWP-B-010	12.6	9.24	0.35	5750.223	12.4334	556292	21713	46.99	6582	splinter tension
EWP-B-011	11.3	5.64	0.32	6818.24	20.44592	484517	18934	63.88	7060	cross-grain
EWP-B-012	12.9	7.47	0.35	4789.612	10.22508	551497	21627	39.28	6280	splinter tension
EWP-B-013	12.4	5.60	0.30	7491.658	23.32742	436571	17768	74.80	8300	shear
EWP-B-014	12.8	5.18	0.39	4058.551	10.94978	418081	17928	40.14	6896	simple tension
EWP-B-015	12.3	7.82	0.34	3752.947	8.633065	565726	21974	30.28	5606	splinter tension
EWP-B-016	9.2	5.59	0.30	2740.037	8.025066	515563	19829	24.50	5040	shear
EWP-B-017	10.2	4.82	0.30	5521.085	19.57634	403190	17465	56.10	6541	shear
EWP-B-018	16.9	9.70	0.41	11151.14	22.75158	619070	23291	84.92	9256	shear
EWP-B-019	12.5	5.48	0.34	7667.474	23.42906	488041	19321	70.43	6953	shear
EWP-B-020	10.9	5.40	0.33	3736.946	10.29993	497448	19565	33.85	5571	cross-grain
EWP-B-021	14.7	5.69	0.37	3613.892	7.908477	442979	18055	35.47	7522	brash tension
EWP-B-022	14.7	5.39	0.38	3442.606	8.891537	459659	18438	33.11	6366	splinter tension
EWP-B-023	16.9	8.94	0.41	11319.36	21.96661	607805	23023	87.24	9472	shear
EWP-B-024	10.5	3.94	0.31	5586.353	17.8715	477437	19059	52.01	6169	simple tension
EWP-B-025	15.7	5.69	0.37	5625.883	13.51667	403330	17574	56.79	8086	splinter tension
EWP-B-026	13.8	8.96	0.35	4178.149	9.0834	477005	19084	38.83	7369	simple tension
EWP-B-027	11.4	10.62	0.29	6797.28	21.23329	476944	18889	63.85	7163	shear
EWP-B-028	11.6	5.48	0.30	6354.745	18.9999	511101	20024	56.30	6157	shear

C. Building 95 Design Calculations

The internal forces of the members in Building 95 were calculated using Dlubal RFEM 5 software (Dlubal Software, Inc, 2021). NBC 2015 factored gravity loads were applied to a representative truss, based on a measured tributary width of 2.75 m.

C.1 Compressive Resistance

The factored compression resistances parallel to grain were calculated for each member according to CSA O86-14 clause 6.5.6. Only the specified strengths in compression and fifth percentile Modulus of Elasticity, f_c and E_{05} , were updated for each iteration. Standard term loading, dry service condition, and untreated lumber were assumed based on site observations.

$$P_r = \phi F_c A K_{zc} K_c \quad (\text{C.1})$$

Where:

$$\phi = 0.8$$

$$F_c = f_c (K_D K_H K_{SC} K_T)$$

f_c = Specified strength in compression

$K_D = K_H = K_{SC} = K_T = 1$ for dry, untreated lumber under standard load duration

$$K_{zc} = (dL)^{-0.13} \leq 1.3$$

K_c = Slenderness factor, calculated as per CSA O86-14 clause 6.5.6.2.4

Table C.1: Compression Analysis - First Iteration

Member	b (mm)	d (mm)	f_c (Mpa)	E_{05} (MPa)	K_{Sc}	K_{SE}	L (mm)	K_{zc}	C_c	K_c	P_r (kN)	P_f (kN)	DCR
1.1	80	150	13	5500	1.0	1.0	600	1.3	7.5	0.95	201.22	29.82	15%
1.2	80	140	13	5500	1.0	1.0	2300	1.3	28.8	0.27	53.03	62.07	117%
1.3	40	140	13	5500	1.0	1.0	850	1.3	21.3	0.48	46.98	3.65	8%
1.4	38	89	13	5500	1.0	1.0	381	1.3	10.0	0.90	53.31	8.87	17%
1.5	40	140	13	5500	1.0	1.0	640	1.3	16.0	0.68	67.07	0.00	0%
1.6	38	89	13	5500	1.0	1.0	926	1.3	24.4	0.38	22.42	8.87	40%
1.7	40	140	13	5500	1.0	1.0	864	1.3	21.6	0.47	45.78	0.00	0%
1.8	40	160	13	5500	1.0	1.0	864	1.3	21.6	0.47	52.32	0.90	2%
2.1	40	140	13	5500	1.0	1.0	600	1.3	15.0	0.72	71.06	40.49	57%
2.2	50	140	13	5500	1.0	1.0	1800	1.3	36.0	0.16	19.45	22.32	115%
2.3	40	160	13	5500	1.0	1.0	706	1.3	17.7	0.61	69.11	0.00	0%
2.4	25	300	13	5500	1.0	1.0	928	1.3	37.1	0.15	19.28	0.00	0%
3.1	40	140	13	5500	1.0	1.0	2100	1.3	52.5	0.06	5.62	0.00	0%
3.2	40	140	13	5500	1.0	1.0	600	1.3	15.0	0.72	71.06	40.49	57%
3.3	25	250	13	5500	1.0	1.0	454	1.3	18.2	0.59	65.25	6.91	11%
3.4	40	140	13	5500	1.0	1.0	1005	1.3	25.1	0.36	35.02	13.45	38%

Table C.2: Compression Analysis - Second Iteration

Member	b (mm)	d (mm)	f_c (Mpa)	E_{05} (MPa)	K_{Sc}	K_{SE}	L (mm)	K_{zc}	C_c	K_c	P_r (kN)	P_f (kN)	DCR
1.1	80	150	12.61	6699	1.0	1.0	600	1.3	7.5	0.96	197.1	29.82	15%
1.2	80	140	12.61	6699	1.0	1.0	2300	1.3	28.8	0.32	60.4	62.07	103%
1.3	40	140	12.61	6699	1.0	1.0	850	1.3	21.3	0.53	51.0	3.65	7%
1.4	38	89	12.61	6699	1.0	1.0	381	1.3	10.0	0.92	52.8	8.87	17%
1.5	40	140	12.61	6699	1.0	1.0	640	1.3	16.0	0.73	69.6	0.00	0%
1.6	38	89	12.61	6699	1.0	1.0	926	1.3	24.4	0.43	24.9	8.87	36%
1.7	40	140	12.61	6699	1.0	1.0	864	1.3	21.6	0.52	49.8	0.00	0%
1.8	40	160	12.61	6699	1.0	1.0	864	1.3	21.6	0.52	557.0	0.90	2%
2.1	40	140	12.61	6699	1.0	1.0	600	1.3	15.0	0.77	73.1	40.49	55%
2.2	50	140	12.61	6699	1.0	1.0	1800	1.3	36.0	0.19	22.8	22.32	98%
2.3	40	160	12.61	6699	1.0	1.0	706	1.3	17.7	0.67	72.8	0.00	0%
2.4	25	300	12.61	6699	1.0	1.0	928	1.3	37.1	0.18	22.6	0.00	0%
3.1	40	140	12.61	6699	1.0	1.0	2100	1.3	52.5	0.07	6.8	0.00	0%
3.2	40	140	12.61	6699	1.0	1.0	600	1.3	15.0	0.77	73.1	40.49	55%
3.3	25	250	12.61	6699	1.0	1.0	454	1.3	18.2	0.65	69.0	6.91	10%
3.4	40	140	12.61	6699	1.0	1.0	1005	1.3	25.1	0.41	39.1	13.45	34%

Table C.3: Compression Analysis - Third Iteration

Member	b (mm)	d (mm)	f_c (Mpa)	E_{05} (MPa)	K_{Sc}	K_{SE}	L (mm)	K_{zc}	C_c	K_c	P_r (kN)	P_f (kN)	DCR
1.2	80	140	15.86	9189	1.0	1.0	2300	1.3	28.8	0.34	80.6	62.07	77%
2.1	40	140	15.24	8727	1.0	1.0	600	1.3	15.0	0.78	89.8	40.49	45%
2.2	50	140	14.76	8373	1.0	1.0	1800	1.3	36.0	0.20	28.1	22.32	79%
3.2	40	140	14.56	8222	1.0	1.0	600	1.3	15.0	0.78	85.5	40.49	47%

C.2 Bending Moment Resistance

The factored bending resistances were calculated for each member according to CSA O86-14 clause 6.5.4. Only the specified strength in bending and fifth percentile Modulus of Elasticity, f_b , was updated for each iteration. Standard term loading, dry service condition, and untreated lumber were assumed based on site observations.

$$M_r = \phi F_b S K_{zb} K_L \quad (C.2)$$

Where:

$$\phi = 0.9$$

$$F_b = f_b (K_D K_H K_{St} K_T)$$

f_b = Specified strength in bending

$K_D = K_H = K_{St} = K_T = 1$ for dry, untreated lumber under standard load duration

K_{zb} = Size factor in bending, from clause 6.4.5

K_L = Lateral stability factor, from clause 6.5.4.2

Table C.4: Bending Analysis - First Iteration

Member	b (mm)	d (mm)	f_b (Mpa)	K_{sb}	K_{zb}	M_r (kN)	M_f (kN)	DCR
1.1	80	150	10.6	1.0	1.4	4.01	2.7	68%
1.2	80	140	10.6	1.0	1.4	3.49	0.7	19%
1.3	40	140	10.6	1.0	1.4	1.75	0.0	0%
1.4	38	89	10.6	1.0	1.7	0.81	0.0	0%
1.5	40	140	10.6	1.0	1.4	1.75	0.0	0%
1.6	38	89	10.6	1.0	1.7	0.81	0.0	0%
1.7	40	140	10.6	1.0	1.4	1.75	0.0	0%
1.8	40	160	10.6	1.0	1.4	2.28	0.0	0%
2.1	40	140	10.6	1.0	1.4	1.75	3.0	171%
2.2	50	140	10.6	1.0	1.4	2.18	1.4	63%
2.3	40	160	10.6	1.0	1.4	2.28	0.0	0%
2.4	25	300	10.6	1.0	1.0	3.58	0.0	0%
3.1	40	140	10.6	1.0	1.4	1.75	1.6	92%
3.2	40	140	10.6	1.0	1.4	1.75	3.0	171%
3.3	25	250	10.6	1.0	1.1	2.73	3.2	117%
3.4	40	140	10.6	1.0	1.4	1.75	0.0	1%

Table C.5: Bending Analysis - Second Iteration

Member	b (mm)	d (mm)	f_b (Mpa)	K_{sb}	K_{zb}	M_r (kN)	M_f (kN)	DCR
1.1	80	150	17.53	1.0	1.4	6.63	2.7	41%
1.2	80	140	17.53	1.0	1.4	5.77	0.7	12%
1.3	40	140	17.53	1.0	1.4	2.89	0.0	0%
1.4	38	89	17.53	1.0	1.7	1.35	0.0	0%
1.5	40	140	17.53	1.0	1.4	2.89	0.0	0%
1.6	38	89	17.53	1.0	1.7	1.35	0.0	0%
1.7	40	140	17.53	1.0	1.4	2.89	0.0	0%
1.8	40	160	17.53	1.0	1.4	3.77	0.0	0%
2.1	40	140	17.53	1.0	1.4	2.89	3.0	103%
2.2	50	140	17.53	1.0	1.4	3.61	1.4	38%
2.3	40	160	17.53	1.0	1.4	3.77	0.0	0%
2.4	25	300	17.53	1.0	1.0	5.92	0.0	0%
3.1	40	140	17.53	1.0	1.4	2.89	1.6	55%
3.2	40	140	17.53	1.0	1.4	2.89	3.0	103%
3.3	25	250	17.53	1.0	1.1	4.52	3.2	71%
3.4	40	140	17.53	1.0	1.4	2.89	0.0	1%

Table C.6: Bending Analysis - Third Iteration

Member	b (mm)	d (mm)	f_b (Mpa)	K_{sb}	K_{zb}	M_r (kN)	M_f (kN)	DCR
1.2	80	140	19.59	1.0	1.4	6.45	0.7	11%
2.1	40	140	19.08	1.0	1.4	3.14	3.0	95%
2.2	50	140	18.69	1.0	1.4	3.85	1.4	36%
3.2	40	140	18.52	1.0	1.4	3.05	3.0	98%

C.3 Combined Bending and Axial Load Resistance

The factored resistances to combined axial and bending were calculated for each member according to CSA O86-14 clause 6.5.10. The specified strengths in compression, f_c , bending, f_b , and the fifth percentile Modulus of Elasticity, E_{05} , were updated for each iteration. Standard term loading, dry service condition, and untreated lumber were assumed based on site observations.

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_f}{M_r} \left(1 - \frac{P_f}{P_E}\right)^{-1} \leq 1 \quad (\text{C.3})$$

$$P_E = \frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2} \quad (\text{C.4})$$

Where:

P_E = the Euler buckling load in the plane of the applied moment

L_e = effective length in the plane of the applied moment

Table C.7: Combined Loading - First Iteration

Member	Le (mm)	E_{05} (MPa)	PE (kN)	Pr (kN)	Pf (kN)	M_r (kN)	M_f (kN)	DCR
1.1	600	5500	3392.7	201.22	29.82	4.01	2.7	70%
1.2	2300	5500	187.7	53.03	62.07	3.49	0.7	166%
1.3	850	5500	687.2	46.98	3.65	1.75	0.0	1%
1.4	381	5500	834.8	53.31	8.87	0.81	0.0	3%
1.5	640	5500	1212.2	67.07	0.00	1.75	0.0	0%
1.6	926	5500	141.3	22.42	8.87	0.81	0.0	16%
1.7	864	5500	665.1	45.78	0.00	1.75	0.0	0%
1.8	864	5500	992.8	52.32	0.90	2.28	0.0	0%
2.1	600	5500	1379.2	71.06	40.49	1.75	3.0	208%
2.2	1800	5500	191.6	19.45	22.32	2.18	1.4	203%
2.3	706	5500	1486.9	69.11	0.00	2.28	0.0	0%
2.4	928	5500	1986.0	53.07	0.00	3.58	0.0	0%
3.1	2100	5500	112.6	5.62	0.00	1.75	1.6	92%
3.2	600	5500	4908.4	71.06	40.49	1.75	3.0	205%
3.3	454	5500	2408.9	65.25	6.91	2.73	3.2	118%
3.4	1005	5500	491.6	35.02	13.45	1.75	0.0	16%

Table C.8: Combined Loading - Second Iteration

Member	Le (mm)	E_{05} (MPa)	PE (kN)	Pr (kN)	Pf (kN)	M_r (kN)	M_f (kN)	DCR
1.1	600	5500	3392.7	201.22	29.82	4.01	2.7	70%
1.2	2300	5500	187.7	53.03	62.07	3.49	0.7	166%
1.3	850	5500	687.2	46.98	3.65	1.75	0.0	1%
1.4	381	5500	834.8	53.31	8.87	0.81	0.0	3%
1.5	640	5500	1212.2	67.07	0.00	1.75	0.0	0%
1.6	926	5500	141.3	22.42	8.87	0.81	0.0	16%
1.7	864	5500	665.1	45.78	0.00	1.75	0.0	0%
1.8	864	5500	992.8	52.32	0.90	2.28	0.0	0%
2.1	600	5500	1379.2	71.06	40.49	1.75	3.0	208%
2.2	1800	5500	191.6	19.45	22.32	2.18	1.4	203%
2.3	706	5500	1486.9	69.11	0.00	2.28	0.0	0%
2.4	928	5500	1986.0	53.07	0.00	3.58	0.0	0%
3.1	2100	5500	112.6	5.62	0.00	1.75	1.6	92%
3.2	600	5500	4908.4	71.06	40.49	1.75	3.0	205%
3.3	454	5500	2408.9	65.25	6.91	2.73	3.2	118%
3.4	1005	5500	491.6	35.02	13.45	1.75	0.0	16%

Table C.9: Combined Loading - Third Iteration

Member	L_e (mm)	E_{05} (MPa)	P_E (kN)	P_r (kN)	P_f (kN)	M_r (kN)	M_f (kN)	DCR
1.2	2300	9189	313.6	80.58	62.07	6.45	0.68	72%
2.1	600	8727	2188.4	89.82	40.49	3.14	2.98	117%
2.2	1800	8373	291.6	28.10	22.32	3.85	1.38	102%
3.2	600	8222	7337.6	85.54	40.49	3.05	2.98	121%

C.4 Tension Resistance

The factored tensile resistances parallel to grain were calculated for each member according to CSA O86-14 clause 6.5.9. Under the first iteration, no members were found to be overloaded, thus tension resistance was not re-evaluated in the following iterations. Standard term loading, dry service condition, and untreated lumber were assumed based on site observations.

$$T_r = \phi F_t A_n K_{zt} \quad (C.5)$$

Where:

$$\phi = 0.9$$

$$F_t = f_t (K_D K_H K_{St} K_T)$$

f_t = Specified strength in tension

$K_D = K_H = K_{St} = K_T = 1$ for dry, untreated lumber under standard load duration

K_{zt} = Size factor in tension, from clause 6.4.5

Table C.10: Tension Analysis

Member	b (mm)	d (mm)	f_t (Mpa)	K_{st}	K_{zt}	T_r (kN)	T_f (kN)	DCR
1.1	80	150	10.6	1.0	1.3	193.471	10.65	6%
1.2	80	140	10.6	1.0	1.3	180.573	0.00	0%
1.3	40	140	10.6	1.0	1.3	90.287	0.00	0%
1.4	38	89	10.6	1.0	1.5	72.595	28.15	39%
1.5	40	140	10.6	1.0	1.3	90.287	0.00	0%
1.6	38	89	10.6	1.0	1.5	72.595	28.15	39%
1.7	40	140	10.6	1.0	1.3	90.287	0.00	0%
1.8	40	160	10.6	1.0	1.3	103.185	0.00	0%
2.1	40	140	10.6	1.0	1.3	90.287	22.57	25%
2.2	50	140	10.6	1.0	1.3	112.858	0.00	0%
2.3	40	160	10.6	1.0	1.3	103.185	7.97	8%
2.4	25	300	10.6	1.0	1.0	71.550	5.72	8%
3.1	40	140	10.6	1.0	1.3	90.287	8.00	9%
3.2	40	140	10.6	1.0	1.3	90.287	22.57	25%
3.3	25	250	10.6	1.0	1.1	72.146	2.76	4%
3.4	40	140	10.6	1.0	1.3	90.287	15.11	17%

C.5 Shear Resistance

The factored shear resistances were calculated for each member according to CSA O86-14 clause 6.5.5. Under the first iteration, no members were found to be overloaded, thus shear resistance was not re-evaluated in the following iterations. Standard term loading, dry service condition, and untreated lumber were assumed based on site observations.

$$V_r = \phi F_v \frac{2A_n}{3} K_{zv} \quad (\text{C.6})$$

Where:

$$\phi = 0.9$$

$$F_v = f_v (K_D K_H K_{St} K_T)$$

f_v = Specified strength in shear

$K_D = K_H = K_{St} = K_T = 1$ for dry, untreated lumber under standard load duration

K_{zv} = Size factor in shear, from clause 6.4.5

Table C.11: Shear Analysis

Member	b (mm)	d (mm)	f_v (Mpa)	K_{sv}	K_{zv}	V_r (kN)	V_f (kN)	DCR
1.1	80	150	1.3	1.0	1.4	13.1	10.65	56%
1.2	80	140	1.3	1.0	1.4	12.2	0.00	8%
1.3	40	140	1.3	1.0	1.4	6.1	0.00	0%
1.4	38	89	1.3	1.0	1.7	4.5	28.15	0%
1.5	40	140	1.3	1.0	1.4	6.1	0.00	0%
1.6	38	89	1.3	1.0	1.7	4.5	28.15	0%
1.7	40	140	1.3	1.0	1.4	6.1	0.00	0%
1.8	40	160	1.3	1.0	1.4	7.0	0.00	0%
2.1	40	140	1.3	1.0	1.4	6.1	22.57	92%
2.2	50	140	1.3	1.0	1.4	7.6	0.00	32%
2.3	40	160	1.3	1.0	1.4	7.0	7.97	0%
2.4	25	300	1.3	1.0	1.0	5.9	5.72	0%
3.1	40	140	1.3	1.0	1.4	6.1	8.00	0%
3.2	40	140	1.3	1.0	1.4	6.1	22.57	92%
3.3	25	250	1.3	1.0	1.1	5.4	2.76	66%
3.4	40	140	1.3	1.0	1.4	6.1	15.11	36%