

**Advancing Structural Applications of Glulam Beams: Strengthening Techniques through
Experiment and Finite Element Modelling**

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Abstract

Advancing Structural Applications of Glulam Beams: Strengthening Techniques through Experiment and Finite Element Modelling

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The growing demand for sustainable structural materials has led to the use of Glued Laminated Timber (GLT) in contemporary construction, including bridges and small-to-large-scale infrastructure projects. Bridges are subject to repeated vehicular loads, making fatigue and cyclic performance essential considerations in their design. This study examines the flexural performance of GLT beams for structural applications through experimental testing, theoretical analysis, and Finite Element (FE) modelling. Full-scale four-point bending tests under monotonic and cyclic loading conditions revealed that beams tested under monotonic loads exhibited higher strength, displacement, and energy dissipation. On the flip side, cyclic loading provided critical insights into the durability and serviceability performance of GLT beams.

FEM were validated using experimental results. These models effectively captured the non-linear response of GLT beams, allowing parametric studies that highlighted the significant influence of beam geometry on stiffness and energy absorption. Hybrid reinforcement strategies were assessed to improve the structural performance of GLT beams, utilizing steel in tension and Fibre Reinforced Polymers (FRPs) in compression. Experimental benchmarks informed FEM simulations of reinforced beams with varying bar spacings and reinforcement ratios. CFRP-steel hybrid configurations produced the most substantial improvements in both strength and stiffness, achieving more than twice the capacity of the reference (unreinforced) beams. While GFRP and BFRP hybrid reinforcement configurations yielded moderate enhancements, reaching approximately one-half of the capacity increase observed in reference GLT beams. The combination of experimental data with validated FE modelling advances the structural application of GLT beams in engineered timber bridge design. The results address gaps in current standards (CSA O86, CSA S6:19) and demonstrate that hybrid reinforcement substantially improves flexural strength, stiffness, and ductility. These findings advance sustainable, high-performance timber structures for modern infrastructure.

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I deeply appreciate my parents for their unwavering support and belief in me. I am also grateful to my siblings, whose encouragement has motivated me throughout this journey.

Contribution of Authors

This thesis, prepared in manuscript-based format, presents research by Hassan Malik under the supervision of Dr. Ghazanfarah Hafeez. The study consisted of two stages. The first stage included experimental testing and finite element modelling of GLT beams under monotonic and cyclic loading. The author designed and conducted full-scale four-point bending tests, analyzed force displacement and energy dissipation data, and developed finite element models in Abaqus. These models were validated using experimental results and employed in a parametric study to evaluate the impact of beam geometry on strength, stiffness, and energy dissipation. The second stage involved developing a finite element model for unreinforced GLT beams. The author then extended this model to simulate hybrid reinforcement configurations, using steel in tension and FRPs in compression. This approach enabled the evaluation of reinforcement spacing, ratios, and material combinations, demonstrating that CFRP steel hybrids outperformed the unreinforced beams.

In addition to these technical contributions, the author conducted comprehensive literature reviews, prepared figures and technical images, curated abbreviation lists, and drafted and refined all thesis chapters. The thesis is presented with clarity, logical flow, and academic rigor, in full compliance with Concordia University's requirements.

Thesis Published Papers

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Table of Contents

Abstract	iii
Contribution of Authors	v
List of Figures	viii
List of Tables.....	x
List of Abbreviations.....	xi
1. Chapter 1	1
Introduction.....	1
1.1. Research Needs and Motivation	1
1.2. Research Objectives.....	2
1.3. Research Methodology	3
1.4. Thesis Organization	3
1.5. Novel contribution of the research.....	4
References.....	5
2. Chapter 2.....	6
Background and Literature Review	6
2.1 General.....	6
2.2 Wood as a construction material	7
2.2.1. Moisture content and density of wood.....	8
2.3 Structural Anisotropy of Wood	9
2.4 Flexural Performance of Wood	10
2.4.1 Typical Failure Modes in Wood.....	12
2.5 Mass timber.....	13
2.5.1. Glue-Laminated Timber (GLT).....	15
2.5.2 Flexural Reinforcement of Glue Laminated Timber (GLT).....	16
2.6 Review of the Strengthening of Timber Beams	20
2.7 Review of the Finite Element Modelling (FEM) in Wood	24
References.....	27
3. Chapter 3	33
3.1 Abstract	33
3.2 Introduction.....	34

3.3	Methodology	35
3.3.1.	Glulam Timber	36
3.3.2.	Non-destructive Testing (NDT)	37
3.3.3.	Destructive Four-Point Bending Tests	37
3.3.3.1.	Specimen Preparation and Instrumentation	37
3.3.3.2.	Test Set-up and Loading Protocols	38
3.3.3.3.	Derived Quantities	39
3.4.	Results And Discussion.....	39
3.4.1.	Monotonic Tests	41
3.4.2.	Cyclic Tests	43
3.5	Parametric Assessments	48
3.6.	Conclusions	52
	References	55
4.	Chapter 4	59
4.1.	Abstract	59
4.2.	Introduction	59
4.3.	Methodology	62
4.3.1.	Materials and Test Protocol.....	62
4.3.2.	Experimental Results and Discussion	64
4.4.	Finite Element Model Development and Validation	66
4.2.1.	Effect of Reinforcement Spacing on Bending Performance of GLT Beams	70
4.2.2.	Effect of Reinforcement Ratio (ρ) on Strength and Stiffness of GLT Beams	72
4.3.	Comparison of FEM Analysis and Theoretical Predictions of GLT Beams	74
4.4.	Conclusion	79
	Data Accessibility	80
	Acknowledgments:	80
	Conflicts of Interest:	80
	References	81
	Chapter 5	84
5.1.	Conclusions	84
5.2.	Limitations and Future Work Recommendations	85

Appendix.....	86
Appendix A:.....	86
B Appendix B: Flexural Performance of Hybrid Glulam Beams: Influence of Reinforcement Material and Arrangement	96

List of Figures

Figure 1.1 Research Methodology.....	3
Figure 2.1 Microstructure of the Wood [10]	8
Figure 2.2 Wood Fibre Orientation [13]	10
Figure 2.3 Mechanical Stresses in Wood [17]	11
Figure 2.4 Ideal Stress-Strain Curve for Wood.....	12
Figure 2.5 Failure Modes in Timber [21].....	13
Figure 2.6 Common Engineered Wood Members in Mass Timber Construction [22]	13
Figure 2.7 Trends in Mass Timber Material Use (2007–2019) [22]	14
Figure 2.8 Types of GLT [25]	15
Figure 2.9 Mistissini Bridge in Quebec [28]	16
Figure 2.10 Composition of Fibre Reinforced Polymer (FRP)[31].....	17
Figure 2.11 Composition of Steel	17
Figure 2.12 Various Types of EBR & NSM [36].....	18
Figure 2.13 Equilibrium analysis of the reinforced beam cross-section for two configurations [40]	19
Figure 2.14 Stress-Strain Curve for Reinforced Material [55]	23
Figure 3.1 Non Destructive Measurements with Stress-Wave Transit Time (schematic).	37
Figure 3.2 Non-Destructive measurements with stress-wave transit time (schematic).	39
Figure 3.3 Force-displacement profiles (Monotonic tests).	41
Figure 3.4 Representative failure modes observed in the glulam beams: (a)–(c) monotonic tests; (d) cyclic test.	42
Figure 3.5 Force-displacement profiles (Cyclic tests).	43
Figure 3.6 Finite Element Model.	44
Figure 3.7 Analysis results of B1.....	45
Figure 3.8 Validation of FE model with experimental work (Cyclic).	46
Figure 3.9 Validation of the FE model under half-cyclic loading: numerical vs.\ experimental hysteresis responses (specimens C1 and C2).	47
Figure 3.10 Parametric assessment under a monotonic and cyclic loading protocol.	49
Figure 3.11 Energy curve for beam B3.....	52
Figure 4.1 Experimental Setup	64

Figure 4.2 Failures of G1 and G2 Beams	64
Figure 4.3 Experimental Force Vs Displacement Curve	65
Figure 4.4 FEM Assembly of Unreinforced GLT Beam	67
Figure 4.5 FEM Validation Against Experimental Results	68
Figure 4.6 FEM Assembly of GLT Beam with Reinforcement Bars	69
Figure 4.7 Force - Displacement Curve with GFRP and Steel Bars	72
Figure 4.8 Force - Displacement Curve with BFRP and Steel Bars	73
Figure 4.9 Force - Displacement Curve with CFRP and Steel Bars	73
Figure 4.10 FEM Analysis	74
Figure 4.11 Idealized Stress - Strain Curve of Wood	75
Figure 4.12 Idealized Stress - Strain Curve of (a) FRP and (b) Steel	75
Figure 4.13 Cross – Section Equilibrium Analysis of GFRP Reinforced GLT Beam	76
Figure 4.14 Different Reinforcement Configurations	78

List of Tables

Table 3.1 Mechanical properties of glulam beams.	36
Table 3.2 Geometric information of glulam beams.	36
Table 3.3 Experimental results of the glulam beams.	40
Table 3.4 Orthotropic elastic properties adopted for the glulam numerical model.	45
Table 4.1 GLT Beam Material Properties	63
Table 4.2 Experimental Results	65
Table 4.3 FEM Material Properties.....	67
Table 4.4 Results of Various Horizontal Bar Spacing.....	71
Table 4.5 Comparison of FEM Vs Theoretical Results	78

List of Abbreviations

GLT	Glue-Laminated Timber
CLT	Cross-Laminated Timber
FEM	Finite Element Modelling
FRP	Fiber-Reinforced Polymer
CFRP	Carbon Fiber-Reinforced Polymer
GFRP	Glass Fiber-Reinforced Polymer
NSM	Near-Surface Mounted
EBR	Externally Bonded Reinforcement
MOE	Modulus of Elasticity
MOR	Modulus of Rupture
ASTM	American Society for Testing and Materials
CSA	Canadian Standards Association
LVDT	Linear Variable Differential Transformer
NDT	Non-Destructive Test
Aft FRP Cross-Sectional Areas at Tensile Zone	

Chapter 1

Introduction

1.1. Research Needs and Motivation

Current progress in the timber construction sector, along with its environmental benefits compared to other materials such as concrete and steel, has led to an increase in the use of wood in infrastructure projects [1]. Currently, no specific guidelines or standards exist to enhance the bending capacity of Glue Laminated Timber (GLT) beams with reinforcing materials. When structures like bridges or buildings decline in serviceability, they require scheduled, consistent maintenance to continue functioning and reach their expected service life. Therefore, it is necessary to establish an effective retrofitting technique for structures to extend their service life and prevent premature, costly demolition.

In Canadian highway bridges [2], timber beams are widely used not only because of their favorable strength-to-weight ratio, but also due to their ease of construction. However, the ageing and presence of knots in timber beams can reduce flexural strength, resulting in early structural failure [3]. To strengthen existing timber beams, Fibre-Reinforced Polymer (FRP) materials have emerged as an effective means of increasing beam stiffness [4]. An effective method to enhance the stiffness of beams is to use a retrofitting technique known as Near-Surface-Mounted (NSM) reinforcement. A study by Plevris et al. (1992) demonstrated that reinforcing timber beams with a thin layer of FRP (1% of the cross-sectional area) in the tensile region can significantly improve the beam's stiffness and flexural capacity [5]. In another study, Mathuros et al. (2024) investigated the flexural behaviour of timber beams reinforced with GFRP under monotonic and cyclic loading. The authors reported a significant increase in load capacity of approximately 50-70% and an increase in initial stiffness of 30-50% [6]. Although enhancing these beams appears to have benefits, Canadian standards have not fully adopted the use of FRP or steel for Glued-Laminated Timber (GLT) structures. CSA O86 [7] does not cover design guidelines for reinforced wood members with FRP; however, CSA S6:19 does cover guidelines for solid timber beams in bridges but lacks guidance on the strengthening of GLT beams with FRP or steel. Furthermore, there is very little published data available on the behaviour of GLT under monotonic and cyclic loading, which is critical for establishing the framework.

The bending performance of timber, specifically beams, can be enhanced by reinforcing with FRP, allowing the wood on the compression side to reach its peak and crash. Similarly, placing FRP in the tensile zone will withstand most of the tensile forces from external loads, resulting in a reduction of stress on the wood member [8]. This balance enables the beam to develop greater flexural capacity, allowing the material to be utilized to its full strength before failure.

Over the last two decades, many reinforcing materials have been deployed to overcome their limitations for providing structural capacity; however, FRP has gained considerable attention due to its superior mechanical properties, chemical resistance, and advantages over other reinforcing materials. FRP can resist corrosion and has a low density, making it a unique composite material [9]. Although FRP appears to offer numerous benefits, it may raise some concerns. Arora et al. (2022) noted that FRP experiences fatigue and creep over time [10]. All types of FRP, such as CFRP, GFRP, and BFRP, fail in a brittle manner without any warning. Additionally, compared to steel, it lacks ductility and has low stiffness. To achieve both, a hybrid approach combining the two materials by inserting steel bars in the tension region and FRP in the top compression region of beams would be beneficial, thereby improving structural stability.

1.2. Research Objectives

This proposed research aims to investigate the flexural behavior of GLT under monotonic and cyclic loading, along with various reinforcement layouts. The detailed goals are:

1. To investigate and evaluate the flexural response of a GLT beam under monotonic and cyclic loading experimentally, focusing on initial stiffness, ultimate capacity, and energy dissipation.
2. To develop and validate a finite element model capable of simulating the behavior of the GLT beam under monotonic and cycling loading.
3. To assess the effect of aspect ratio, on the flexural performance of GLT beams.
4. To extend the FEM to propose hybrid reinforcement configurations for improved ductility, and to conduct a parametric study of the critical parameters influencing the strength and stiffness of GLT beams.

1.3. Research Methodology

This research combines both experimental testing and numerical modelling. Destructive and non-destructive tests were conducted on several GLT beams, followed by validation of an FEM, to achieve the objectives of the current study.

The schematic of the adopted methodology is illustrated in Figure 1.1, accompanied by a brief explanation that follows.

1. Performing non-destructive tests to evaluate the mechanical properties of GLT beams.
2. Conducting 4-point bending tests on the unreinforced GLT beams under monotonic and half-cyclic loadings.
3. Develop and validate FEM for unreinforced GLT beams using experimental data. Then, use these validated models and analytical checks to conduct parametric assessments of hybrid reinforcement configurations, evaluating their combined load-bearing capacity.

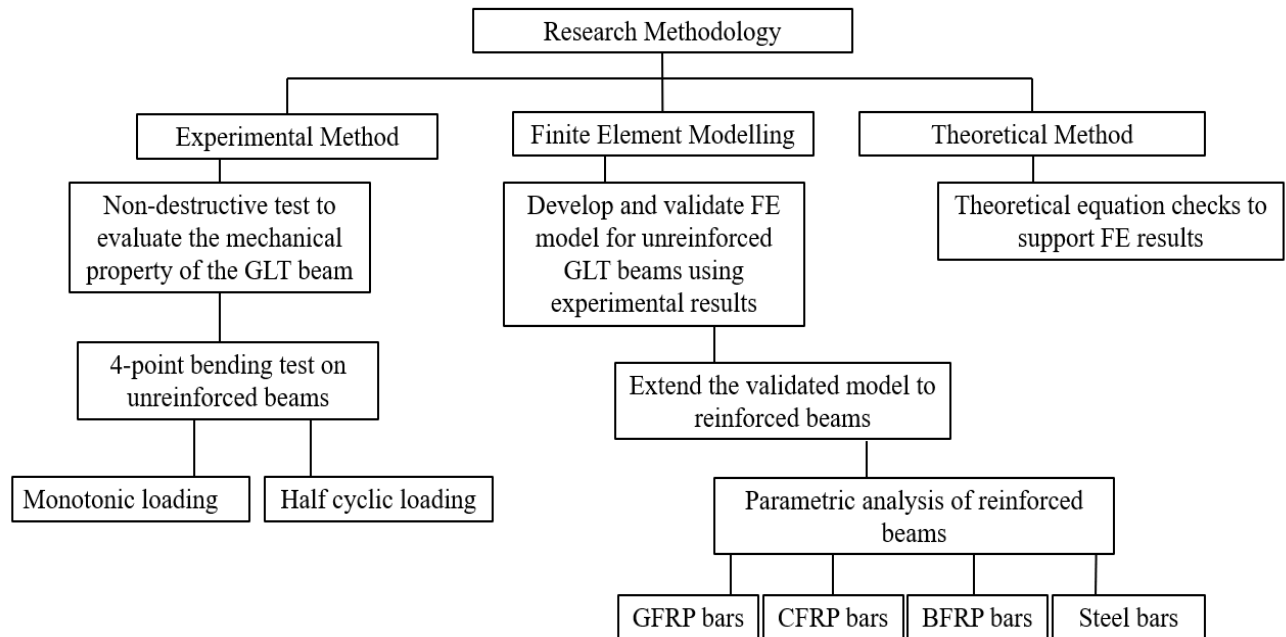


Figure 1.1 Research Methodology

1.4. Thesis Organization

Chapter 1 provides an overview of the research need. It highlights the research gap in design. The chapter concludes by outlining the research objective, methodology, and the approaches used to achieve it.

Chapter 2 presents a detailed background of wood, along with published work relevant to this current research, in which a GLT beam is subjected to Cyclic and Monotonic loading. Additionally, it examines research on GLT reinforced with GFRP, BFRP, CFRP, and other reinforcing materials. Limitations and key findings from previous studies are discussed to provide context for the present work.

Chapter 3: Reports on the experimental and numerical investigation of GLT beams subjected to monotonic and cyclic loadings. It further describes the test method, material properties, and the development of the finite element model used to interpret the results.

Chapter 4 explores various reinforcement layouts and hybrid material combinations that affect the flexural performance of GLT beams. Using the validated finite element modelling, it evaluates the influence of reinforcement arrangement on stiffness, strength, and ductility.

Chapter 5 concludes this research by presenting the main findings and recommending future work.

1.5. Novel contribution of the research

The thesis focuses explicitly on understanding Glued-Laminated Timber through the implementation of experimental and numerical research methods. A comprehensive experimental study of GLT beams under both monotonic and cyclic loading, yielding new data on stiffness and energy dissipation, addresses a gap in existing literature. Additionally, an FEM was developed and validated to capture the orthotropic nature of wood, providing a practical tool for predicting beam behavior under bending.

Furthermore, the research introduces and evaluates a novel reinforcement strategy for these beams by exploring a hybrid model that includes reinforcing materials, using FEM to provide crucial information. This desired information, especially for reinforced GLT beams, is currently absent from the Canadian standards (CSA O86 and CSA S6:19). The validated model plays a crucial role in laying the foundation for updating design codes and practical guidelines, while showcasing the advanced application of GLT beams in modern structures.

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Chapter 2

Background and Literature Review

2.1 General

This chapter investigates the flexural performance of Glue Laminated Timber (GLT) beams under monotonic and cyclic loading in mass timber structures. The chapter begins with an overview of wood as construction material and its key properties. Moisture content, density, structural anisotropy, and mechanical behaviour all affect how beams respond to loads. These include tension rupture on the beam's tension side, compression crushing on the compression side, and shear failure, which typically occurs along the neutral axis or within the web region. Other less frequent modes, such as delamination, rolling shear, or local buckling, may also occur depending on the beam's geometry, material quality, and loading conditions.

Understanding these material properties and their effects on mechanical characteristics is critical for evaluating the flexural response of GLT beams. It also helps assess the effectiveness of reinforcement strategies in mitigating failure modes. Recent research investigates various reinforcement methods to improve flexural capacity, stiffness, and serviceability. Steel and fibre-reinforced polymers (FRPs) are frequently studied for their potential to enhance structural performance. FRPs are especially recognized for their high strength-to-weight ratio, corrosion resistance, and ease of installation. The Near-Surface Mounted (NSM) method stands out for its method of embedding reinforcement in shallow grooves. This improves bond performance and fire resistance.

The chapter also examines numerical modelling methods for simulating timber behaviour under flexural loading. Finite element techniques are commonly used to assess stress distribution, predict failure mechanisms, and evaluate reinforcement strategies. These models enable parametric studies of geometric factors and hybrid reinforcement configurations, such as steel and FRP.

The following sections address the mechanical properties of wood as structural material and its typical failure modes. The sections also provide information on reinforcement techniques using steel and FRP, as well as review of studies on numerical modelling of GLT beams under various loading conditions.

2.2 Wood as a construction material

Wood is one of the oldest construction materials, known and valued for its lightweight characteristic, strength, and availability. Structural applications of timber have been flourishing during the Middle Ages, especially in churches and temples across Europe and Asia, where builders appreciated its strength-to-weight efficiency and ease of shaping [1]. During the 19th and 20th centuries, steel and concrete became the primary construction materials, resulting in a decrease in the use of wood(2) . The evolution of Canadian timber construction has been defined by structural and fire safety constraints, historically limiting buildings to four to six stories[3]. In the last few decades, the introduction of mass timber, including Glued Laminated Timber (GLT) and Cross-Laminated Timber (CLT), has shown exceptional load-bearing capacity compared to traditional lumber [4]This advancement in the construction industry has led to changes in building regulations, with Ontario and Québec implementing new provisions, Ontario Regulation 163/24 [5] and Québec's Order 2025-001 [6], that allow for Encapsulated Mass Timber Construction (EMTC) in buildings up to 18 stories high. These regulatory advancements have significantly increased the possibilities for designing taller and more sustainable timber structures across Canada and increasingly recognized wood as a practical and sustainable material for constructing mid-rise and high-rise buildings, as well as bridges globally. Further, when considering sustainability, the building and construction sector contributes approximately 39% of energy-related CO₂ emissions[7]. As a natural carbon sink, wood can store about 50% of its dry mass as carbon from the atmosphere, resulting in the sequestering of CO₂ throughout its service life[8].

At the microscopic level as shown in Figure 2.1, wood is a natural composite made primarily from cellulose, lignin, hemicellulose, and extractives. Cellulose, a strong crystalline polymer, forms microfibrils that give wood its tensile strength. Hemicellulose and lignin act as binding agents, and hemicellulose is partly crystalline, while lignin is amorphous and adds rigidity and decay resistance[9]. Wood is composed of long axial cells and thinner radial and tangential cells. These cells contain layers of microfibrils, which are bundles of cellulose chains surrounded by hemicellulose and lignin. The microfibrils are layered in different directions, which helps make wood strong in various ways[9].

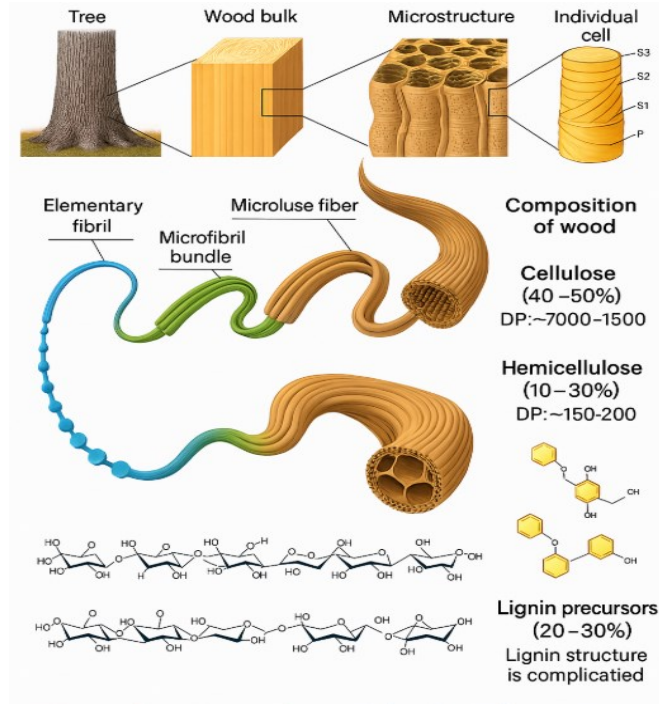


Figure 2.1 Microstructure of the Wood [10]

2.2.1. Moisture content and density of wood

Wood is a hygroscopic material, meaning it naturally absorbs and releases moisture to match its surrounding environment. This moisture exchange is influenced by ambient humidity and temperature, both of which affect the mechanical and physical properties of wood[11]. One of the most critical parameters in wood is moisture content (MC), typically expressed as a percentage. It is calculated using the equation:

$$MC = \frac{m_{\text{water}}}{m_{\text{wood}}} \times 100\% \quad \text{Eqn 2.1}$$

where m_{water} is the mass of water in the wood, and m_{wood} is the mass of oven-dry wood. Moisture content is calculated by comparing the mass of a wood specimen in its current state to its oven-dry mass:

$$MC = \frac{m_{\text{wet}} - m_{\text{dry}}}{m_{\text{dry}}} \times 100\% \quad \text{Eqn 2.2}$$

where m_{wet} is the mass at the current moisture condition, and m_{dry} is the mass after oven-drying.

Understanding the moisture content of wood is essential before evaluating its density, as wood's hygroscopic nature causes its mass and volume to fluctuate with changes in moisture conditions.

Since density is defined as mass per unit volume, any change in moisture content directly influences its value. Therefore, accurately determining moisture content is crucial for density calculations, especially when assessing wood for structural applications. Wood density is typically expressed in kilograms per cubic meter (kg/m³) and is often moisture-specific[12]. One method for calculating wood density at a given moisture content involves the use of basic specific gravity equations:

$$S_x = S_0 \left(1 - \frac{x}{MC_{fs}} \right) \quad \text{Eqn 2.3}$$

$$G_x = \frac{G_b}{1 - \frac{S_x}{100}} \quad \text{Eqn 2.4}$$

$$\rho_x = \rho_w G_x \left(1 + \frac{x}{100} \right) \quad \text{Eqn 2.5}$$

Where:

- S_x = Shrinkage (%) at moisture content x
- S_0 = Total shrinkage from green to oven-dry
- MC_{fs} = Fibre saturation point moisture content
- x = Moisture content (%)
- G_x = Specific gravity of wood at moisture content x
- ρ_x = Density of wood at moisture content x
- ρ_w = Density of water

2.3 Structural Anisotropy of Wood

Wood is known as an orthotropic material, indicating its strength and stiffness vary in different directions. The longitudinal (L) runs along the fibre directions and is the strongest. Radial (R) is perpendicular to the fibre grains, and Tangential (T) is also perpendicular to the grain but follows the growth rings[13] as illustrated in Figure 2.2. The size and strength of timber are largely determined by the natural conditions where trees grow. Trees in different environments receive varying amounts of water and nutrients and face unique environmental challenges [14]. Therefore, the wood of different species exhibits distinct physical and mechanical properties.

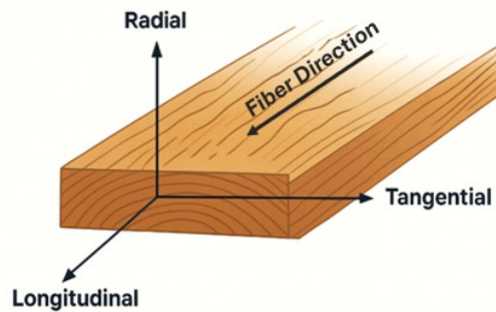


Figure 2.2 Wood Fibre Orientation [13]

Additionally, understanding these principal directions is essential as they play a crucial role in developing the numerical model to simulate structural behaviour. To achieve accuracy in FEM, the model utilizes three different elasticities: E_1 for the longitudinal direction, E_2 for the radial direction, and E_3 for the tangential direction. Similarly, for shear modulus G_{12} longitudinal, G_{13} as radial, and G_{23} tangential, which describe how much the material resists shear in these directions, along with three Poisson ratios ν_{12} , ν_{13} , and ν_{23} . As a required step in FEM modelling, wood must be defined as orthotropic to obtain accurate results. Therefore, nine independent material properties are needed to determine the material properties in the model, which helps understand how the strain along the fibre affects the strain perpendicular to the fibre under flexural conditions. When wood beams are tested under lower loading conditions, they remain in the elastic region and can return to their original shape. However, at higher loads, they enter a state of plastic deformation, cannot return to their original shape, and ultimately fail.

2.4 Flexural Performance of Wood

Mechanical performance of timber changes depending on the direction of the applied force, as illustrated in Figure 2.3. Wood is much stronger and stiffer along the grain, where the cellulose fibres line up, helping it supports heavy loads such as snow, branches, and the weight of the tree itself[15]. However, wood demonstrates weaker structural performance, including strength and stiffness, when force is applied across the grain due to the non-continuity of fibers in that direction[15]. When wood is compressed along the grain, its strength is greater than when it is under tension parallel to the grain. This difference arises from the inherent material behaviour of

cellulose fibres and the interlocking arrangement of wood cells. When wood is compressed, its internal fibres buckle and compact, pushing back against the pressure to prevent further deformation [16]. In tension, wood's strength comes from tightly bonded fibres running lengthwise. These microscopic fibres act like cables, allowing the wood to resist splitting and providing toughness against stretching [16].

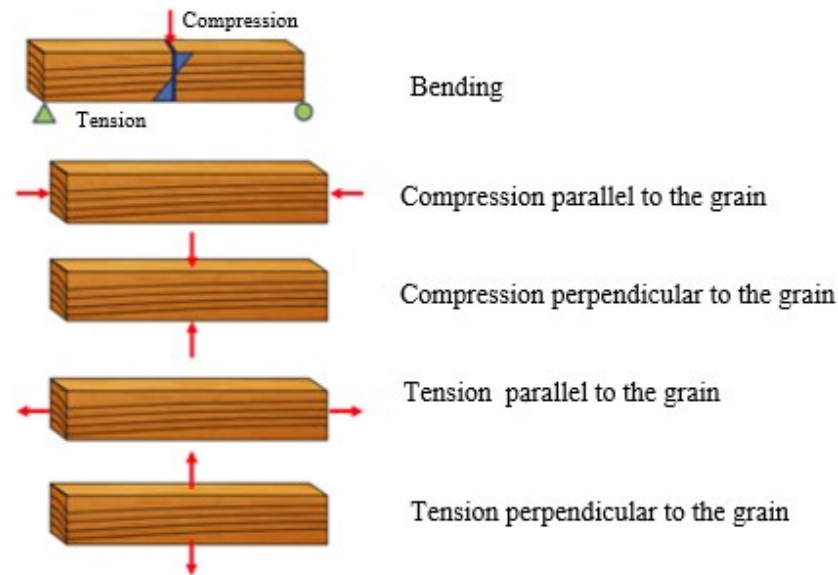


Figure 2.3 Mechanical Stresses in Wood [17]

Comprehension of the flexural performance of wood members in structural engineering is essential for designing structures that bear substantial loads. Under flexural loads, beam behaviour is governed by the moment-curvature relationship. The relationship can be understood through the inherent material properties of wood, particularly its ability to withstand both compression and tension. This behaviour, as proposed by Bazan et al. (1980), suggested a bilinear stress-strain model for predicting the ultimate bending strength of timber beams, based on testing over 250 beams [18]. The model assumes that, in tension, it remains linearly elastic until brittle failure at a specific stress, σ_{gt} and at strain, ε_{gt} . In compression, it follows a bilinear trend, with an initial elastic region at the compressive yield stress, σ_{gcy} and yield at strain, ε_{gcy} defined by modulus

mE_{gc} . Then, the material yield drops, and the stress follows a straight line until it reaches its ultimate compressive stress, σ_{gc} and strain reaches, ϵ_{gc} , as shown in Figure 2.4

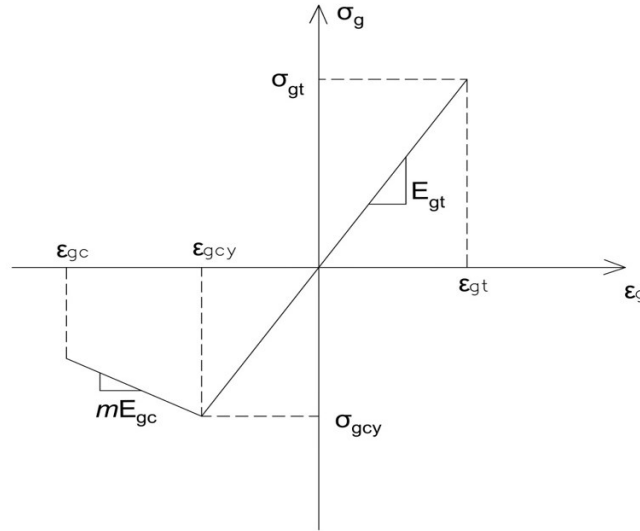


Figure 2.4 Ideal Stress-Strain Curve for Wood

The bilinear stress-strain model effectively describes traditional timber behaviour, particularly during the transition from elastic tension to plastic compression. Timber often lacks the strength and stiffness required for high-load conditions in structures such as bridges and buildings, underscoring the need for advanced structural engineering wood material [19].

2.4.1 Typical Failure Modes in Wood

When a beam made of wood is subjected to flexural loads, the response is dependent on the direction of the wood fibres. The wood's strength and stiffness are higher when it is loaded parallel to the grain rather than perpendicular to the grain. The study on failure modes by Franke et al. (2015) reveals that the most common failure feature in timber beams typically occurs along the grain, with cracks developing due to changes in moisture or stress concentration. Additionally, authors reported that the beam under bending showed that the compressive side deforms while the tensile side ruptures in brittle mode, making tensile failure the critical and dominant failure mode [20]. They also observed that in short spans, shear stress will dominate and produce splits parallel to the grain. Some of the standard failure modes are shown in Figure 2.5.

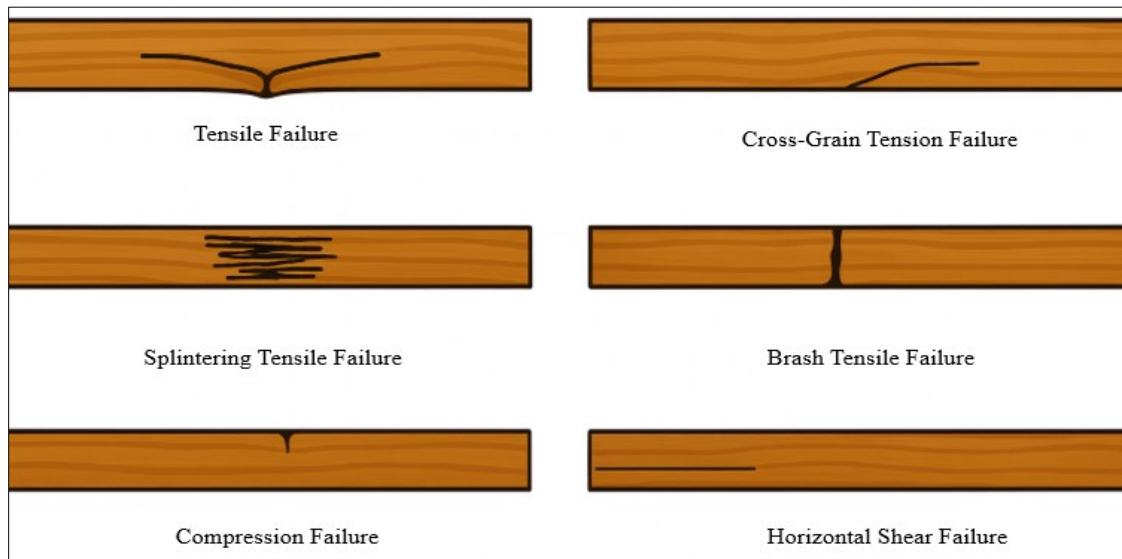


Figure 2.5 Failure Modes in Timber [21]

2.5 Mass timber

In the construction industry, recent advancements in wood have paved the way for innovative building development in Canada. Mass timber is a common term for panels with a thick layer, manufactured by mechanically fastening or bonding smaller wood components[6]. These components include dimensional lumber, veneers, strands, or fibres. The process produces significant prefabricated elements, including beams, columns, arches, and walls with large volume and cross-sectional dimensions to sustain massive loads. As a result, it helps with fire resistance, acoustic performance, structural integrity, and construction efficiency[22]. These products are also used for all manufactured engineered wood products, such as CLT, GLT, Laminated Veneer Lumber (LVL), and Nailed Laminated Lumber (NLL) [6]as shown in Figure 2.6.

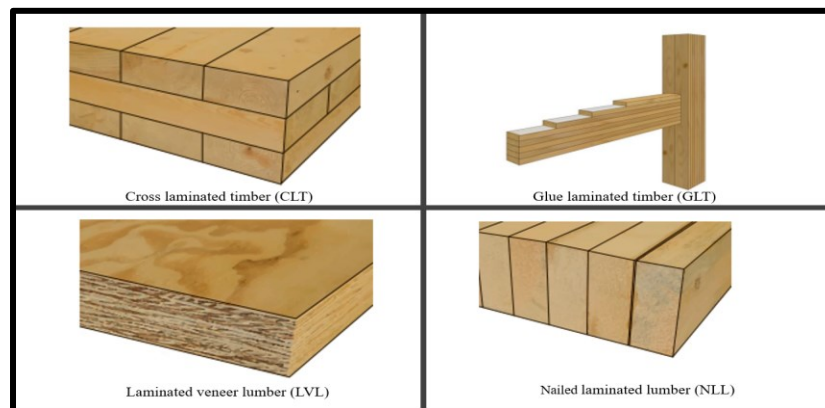


Figure 2.6 Common Engineered Wood Members in Mass Timber Construction [22]

Mass timber has gained popularity in recent years due to its lightweight construction, ease of availability in North America, and environmental benefits, such as reducing carbon emissions. These factors make it a viable alternative to concrete and steel. Several heavy timber structures up to nine stories were constructed in the early 1900s. Despite this progress, building codes still imposed restrictions on the development of tall wood buildings in Canada until the early 21st century. In 2009, British Columbia amended its building code to allow the construction of wood structures up to six stories[22]. This policy change led to increased adoption of timber mid- and high-rise projects. It was later incorporated into the National Building Code of Canada (NBCC) and other provincial codes in 2015, initiating a new phase in nationwide timber construction. The adoption of mid-rise mass timber progressed gradually. The significant change came in 2018, when more than 50 low-rise mass timber projects were completed across Canada, emphasizing their importance, while only 7 were classified as mid-rise [22]. Recent developments in building codes have driven the rapid adoption of mass timber projects. From 2007 to 2011, the number of completed projects more than tripled, with steady annual growth thereafter, as shown in Figure 2.7.

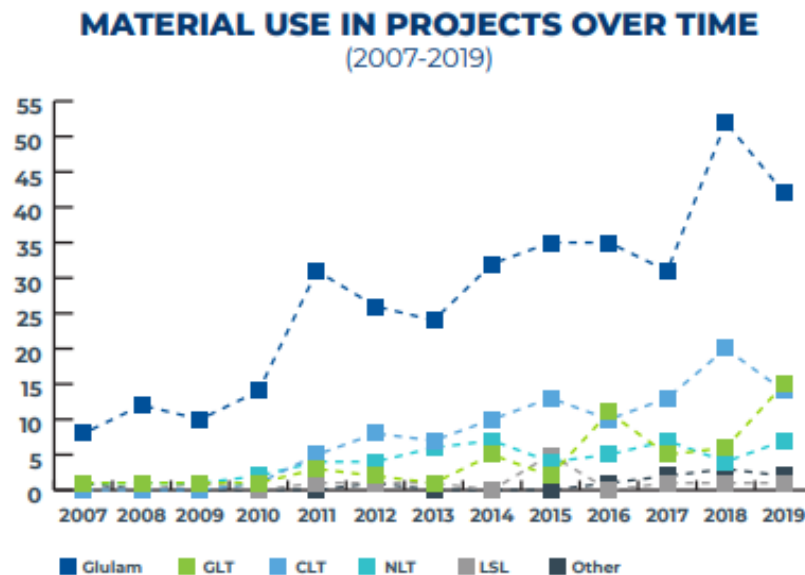


Figure 2.7 Trends in Mass Timber Material Use (2007–2019) [22]

2.5.1. Glue-Laminated Timber (GLT)

Glue-laminated timber is typically manufactured by bonding multiple layers of wood under controlled conditions with an adhesive to ensure a high-quality bond. Each layer of lumber is known as a lamination, and its grains are oriented in the same direction[23]. In heavy mass timber applications, GLT offers strong structural capacity due to its aligned fibres and multiple laminations, allowing it to withstand higher loads. Furthermore, it provides fire resistance comparable to traditional lumber [23]. According to the design specification, the maximum allowable lamination for GLT is 50 mm, though 25 mm and 50 mm laminations are commonly available. In North America, Douglas-Fir, Larch, Southern Pine, Hemlock, and Spruce-Pine-Fir (SPF) are used to produce GLT [24]. GLT beam can be categorized into two types. One of them is non-homogeneous, in which the top and bottom fibres, or the timber lamination, are made of higher strength grade lumber, and the middle region is usually made of lower grade lumber. However, it can be made from the same high-strength lumber throughout, known as homogeneous GLT[25], as shown in Figure 2.8.

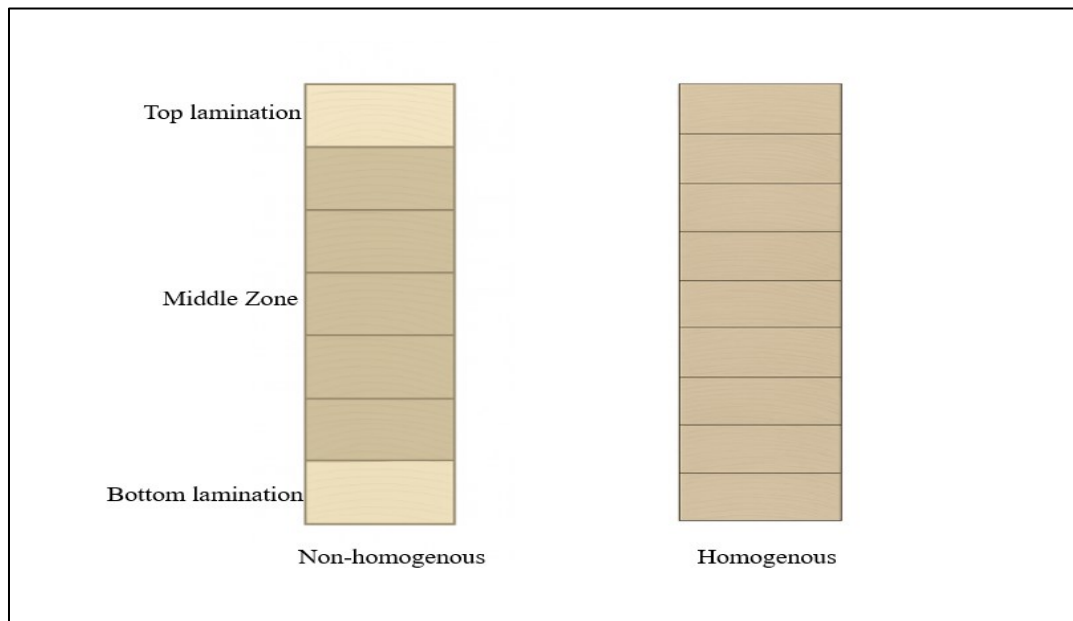


Figure 2.8 Types of GLT [25]

GLT is primarily used in wooden bridges, beam structures, and mass timber buildings. It is widely used as curved beams for aesthetic purposes in structures [26]. Figure 2.9 shows an early example of GLT beams used in the Mistissini Bridge in Quebec, demonstrating the use of this

environmentally friendly material in infrastructure. GLT was chosen due to its higher weight to strength ratio compared to other traditional construction materials, such as steel and concrete, allowing for longer spans with less material [27].



Figure 2.9 Mistissini Bridge in Quebec [28]

2.5.2 Flexural Reinforcement of Glue Laminated Timber (GLT)

GLT offers superior bending strength and other advantages over conventional timber products. However, its structural integrity can weaken due to ageing, prolonged heavy loads, material degradation, and changes in moisture[29]. The mechanical properties of engineered wood products (EWP) are similar to those of traditional timber, such that these products behave in a brittle manner under bending loads. Repeated loading, or fatigue, can lead to the formation of microcracks, which may ultimately result in structural failure, indicating a key weakness of GLT, as it is susceptible to recurring stress. This vulnerability poses significant challenges for long-lasting structures, such as timber bridges. Maintaining these structures is complex and expensive, often resulting in high maintenance costs that necessitate demolition. To address this weakness, engineers retrofit timber beams using reinforcement techniques, including steel and various forms of fibre-reinforced polymers (FRP)[30]. These methods directly target the durability issue by improving strength, extending service life, and reducing maintenance, thereby delaying costly demolitions and re-buildings.

Fibre-reinforced polymer (FRP)

FRP consists of two constituents: high-strength fibres and a polymer resin matrix, as shown in Figure 2.10. The fibres provide strength and stiffness, while the matrix binds them together and

transfers stresses, enabling the FRP to function as a single, strong composite material[31]. This composite material is utilized in construction, primarily to strengthen existing structural elements. As the primary load-bearing components in composite materials, reinforcing fibres are essential. Due to their small size and filamentary structure, they are limited in use as independent materials. FRP composites first appeared in civil engineering after their use in aerospace and defence industries in the 1940s. These materials mix strong fibres, such as glass or carbon, with plastic resins to make lightweight parts that resist corrosion and support heavy loads[31]. One common way to make FRP parts is pultrusion, which pulls a fibre-and-resin mixture through heated molds to create solid shapes for bridges and buildings. Engineers initially used FRP for non-structural elements such as ladder rails, but by the 1990s, they were building entire structures, including pedestrian bridges.

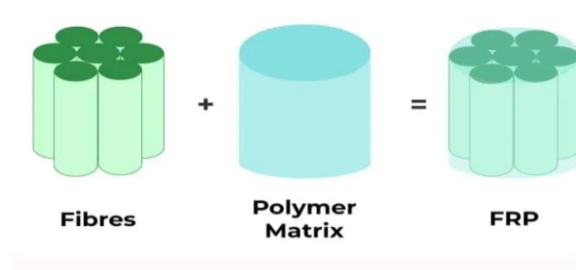


Figure 2.10 Composition of Fibre Reinforced Polymer (FRP)[31]

Steel

Carbon steel is primarily composed of iron and carbon, with only small amounts of other elements[32], as shown in Figure 2.11. This composition provides high strength and resistance to deformation, but limited corrosion resistance. Steel is widely used in concrete reinforcement for adding tensile strength and ductility[33]. Concrete, while strong in compression, needs reinforcement to avoid brittle failure. Similarly, the inherent anisotropic nature of wood makes it weaker in tension perpendicular to the grain, requiring reinforcement to improve its strength.



Figure 2.11 Composition of Steel

Externally bonded reinforcement (EBR) VS Near surface mounted (NSM)

EBR and NSM reinforcement are the most widely studied methods for strengthening GLT beams with FRP and steel. In EBR, FRP sheets or plates are bonded to the timber surface, usually on the tension side. This method is easy to use and reliably improves flexural strength and stiffness. However, its performance is often limited by early debonding, particularly under high strain or long-term loading. Plevris and Triantafillou (1992) demonstrated that EBR with CFRP sheets can significantly enhance moment capacity and ductility; yet, bond failure remains a permanent issue [34]. On the other hand, in the NSM method, FRP bars or strips are placed into shallow grooves in the timber, which not only enhance bond but also protect the reinforcement and facilitate more effective stress transfer. Recent reviews by Jian et al. (2022) [35] and Salčin (2022)[36] indicate that, with the application of anchorage or prestress, NSM delivers superior bond reliability and strain mobilization compared to EBR. Similarly, other researchers have also studied EBR and NSM techniques for strengthening timber beams[37] [38] [39]. Their work supports existing trends and provides experimental data to inform design and detail guidelines. Combining timber with FRP alters the beam's stress-strain response, such that, timber delivers a bilinear compression response, while FRP contributes stable elastic tensile resistance. Together, these entities limit tensile failures, enable timber to absorb compressive forces and delay brittle behavior thereby enhancing strength.[37]. EBR and NSM types are illustrated in Figure 2.12

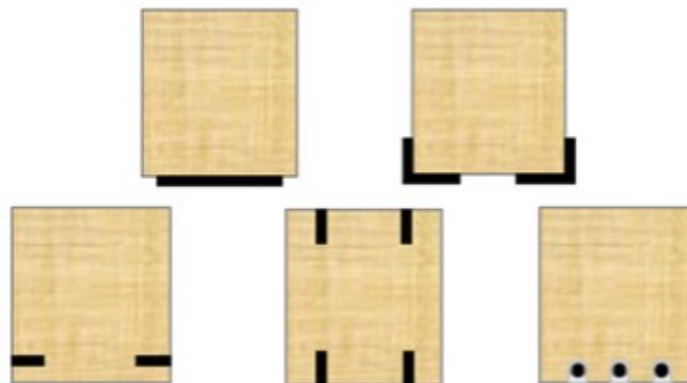


Figure 2.12 Various Types of EBR & NSM [36]

Yeboah and Gkantou (2021) applied a theoretical model to predict the flexural capacity of timber beams reinforced with NSM-FRP. In this model, they assumed pure bending theory and considered the FRP and timber as a composite unit [40]. This unified approach relies on two primary assumptions: linear strain distribution and equilibrium of internal forces across the beam's cross-section. The equilibrium of the reinforced beam cross-section depends on how internal forces are distributed across the beam's tension and compression areas. Each force, represented by $\mathbf{F}$, denotes an internal force specific to a material and location; the subscripts identify the particular material, such as timber or reinforcement and its exact position within the section, as shown in Figure 2.13. The depth of the compression zone is shown as x_c , and for the tension zone as x_t . Within the compression zone, x_{c1} and x_{c2} represent the plastic and elastic portions of timber beams. The ultimate theoretical moment capacity $M_{u, theo}$ is determined by combining the effects of timber and FRP in both zones.

Figure 2.13 Equilibrium analysis of the reinforced beam cross-section for two configurations [40]

$$M_{u,theor} = F_{wc1} \left(x_c - \frac{1}{2} x_{c1} \right) + \frac{2}{3} F_{wc2} x_{c2} + F_{fc} (x_c - z_{fc}) + \frac{2}{3} F_{wt} x_t + F_{ft} (x_t - z_{ft}) =$$

$$bf_{wcy} x_{c1} \left(x_c - \frac{1}{2} x_{c1} \right) + \frac{1}{3} bf_{wcy} x_{c2}^2 + f_{wcy} \frac{E_f}{E_w} A_{fc} (x_c - z_{fc}) + \frac{1}{3} bf_{wtu} x_t^2 + f_{wtu} \frac{E_f}{E_w} A_{ft} (x_t - z_{ft})$$

Eqn 2.6

A_{ft} Is the area of the FRP bars in the tension zone of the beam, and A_{fc} is the area of the FRP bars in the compression zone. If the bars are only in the tension zone, A_{fc} is zero. Equations (2.7) and (2.8) are derived from geometric principles and serve as the foundation for the analytical computations in Equations (2.10) and (2.11)

$$x_c + x_t = h \quad \text{Eqn 2.7}$$

$$x_{c1} + x_{c2} = x_c \quad \text{Eqn 2.8}$$

$$\frac{x_t}{x_{c2}} = \frac{\varepsilon_{wtu}}{\varepsilon_{wcy}} \quad \text{Eqn 2.9}$$

$$F_{wc1} + F_{wc2} + F_{fc} = F_{ft} + F_{wt} \quad \text{Eqn 2.10}$$

$$bf_{wcy} x_{c1} + \frac{1}{2} bf_{wcy} x_{c2} + f_{wcy} \frac{E_f}{E_w} A_{fc} = \frac{1}{2} bf_{wtu} x_t + f_{wtu} \frac{E_f}{E_w} A_{ft} \quad \text{Eqn 2.11}$$

The theoretical moment capacity and internal forces were estimated using known material properties, including the modulus of elasticity of timber, BFRP, and GFRP, as well as tensile strain values from the beam. All beams had $x_{c1}=0$, indicating that the wood remained within its elastic range under compression. In each test, failure started when the tensile strain reached its ultimate limit. On average, the experimental moment capacity was 9% higher than the theoretical value ($M_{u,exp}/M_{u,theo}=1.09$), indicating that the model closely matches the actual results. This thesis employs theoretical formulation to validate the FE results. This theoretical method provides a benchmark for comparing stiffness and ultimate moment capacities of composite wood beams. It ensures that the FEM accurately reflects the composite action between the GLT and the hybrid FRP/steel model.

2.6 Review of the Strengthening of Timber Beams

Timber is a brittle material, and its strength and stiffness depend on moisture and the direction of the wood grain. Although GLT makes timber more uniform, unreinforced beams can still fail

suddenly at weak points or joints under bending. To overcome these issues, early research sought ways to reinforce timber to enhance its stiffness, ductility, and load capacity. Johns and Lacroix (2000) found that adding CFRP and GFRP strips to sawn timber beams could double the bending strength in weaker beams. These composites bridge defects like knots and grain slope, helping limit crack growth and altering how the beams fail. Instead of breaking suddenly under tension, the beams fail more ductility under compression [41]. Gentile et al. (2002) improved on this by placing FRP bars near the surface of Douglas-fir beams, thereby increasing strength by 18–46% and shifting the failure mode from tension rupture to compression crushing [42]. These early studies demonstrated that FRP reinforcement not only enhances the strength of timber but also reduces its variability. Fiorelli and Dias (2003) reinforced pine beams with CFRP and GFRP strips, achieving stiffness gains of 15 to 30% at low reinforcement ratios (0.4 to 1%) and up to 60% with 3% GFRP. Notably, the timber failed rather than the glue, showing that FRP bonding is reliable [43]. Buell et al. (2005) found that externally bonded FRP sheets increased stiffness by 17% to 27% and strength by 40% to 53%. The failure mode also changed from a sudden break in tension to a gradual crushing in compression. CFRP also helped confine the beam, reducing the impact of knots and splits [38].

When FRP reinforcement began to be used in bridges, durability and fatigue became significant concerns. To address these, Davids et al. (2008) tested full-size GLT bridge girders with fibreglass composites by exposing them to rapid changes in temperature and humidity, as well as up to two million load cycles, simulating 50 years of use. Beams with reinforcement along their entire length kept their strength and stiffness, even after these tests; however, beams with only partial reinforcement and unconfined ends did not perform as well. Thus, the study showed that continuous reinforcement and a strong bond are crucial, since environmental exposure can lead to peeling and shear stresses at the wood FRP interface [44].

As FRP was being developed, researchers also looked at steel reinforcement. For example, Alam et al. (2009) repaired broken spruce beams by bonding steel and FRP rods to both sides, finding that reinforcing both the tension and compression sides greatly improved strength; this showed that reinforcing both faces is effective [45]. Similarly, Borri and Corradi (2011) inserted high-strength steel cords into timber beams, which increased the strength by up to 70% and changed the failure mode from sudden rupture to compression failure [46]. In another study, Luca et al. (2012)

investigated different GLT beams and observed that reinforcement increased stiffness by 25.9%, ultimate load by 48.1%, and ductility by 43.8%. Reinforcing and prestressing the beams produced even greater gains: 37.9% more stiffness, 40.2% more ultimate load, and 79.1% more ductility. They also noted a 50% reduction in strain in reinforced and prestressed beams compared to unreinforced beams, indicating that the timber is less likely to crack or deform over time (47).

Building on previous reinforcement strategies, Lu et al. (2015) tested 24 GLT beams reinforced with near-surface-mounted CFRP laminates. They found that ultimate strength increased by 34% to 52% and stiffness increased by 8% to 29%. The beams also exhibited more ductile behaviour and a lower neutral axis, which enhanced their compression performance [48]. Extending this work, Yang et al. (2016) [49][50] tested GLT beams with CFRP bars. In one study, both bonded and end-anchored CFRP bars were used in four-point bending tests. Prestressing the CFRP bars raised flexural capacity by up to 93% and stiffness by 42%. Combining prestressing and reinforcement led to gains of 131%. In another study, they compared FRP and steel reinforcement and found average improvements of 56% in flexural capacity and 27% in stiffness. These results suggest that using both FRP and steel can help strike a balance between ductility, strength, and cost. Jemaa et al. (2019) glued BFRP rods into GLT beams, which increased stiffness by 20-30% and load capacity by almost 50% compared to the controls. In these tests, the wood failed rather than the adhesive, demonstrating that glued in reinforcement is reliable [51].

Over the last five years, research has shifted toward testing a broader range of reinforcement types and geometries. Shekarchi et al. (2020) strengthened beech beams with pultruded GFRP profiles in flat, U-shaped, and L-shaped forms. They observed increases of up to 61% in modulus of rupture, 59% in stiffness, and 79% in ductility. Using shaped profiles also lowered the risk of delamination, a common issue with flat bonded plates [52]. Further expanding on varied configurations, Lacroix et al. (2020) studied GLT with multi-directional fabric under blast loading and found that GFRP wraps improved strength, stiffness, and ductility compared to unreinforced GLT. The improvements were about 30% in stiffness and 20% to 70% in strength [53]. Other studies with different FRPs have also shown promising results. Wdowiak et al. (2023) used NSM under monotonic loading to reinforce GLT beams with BFRP bars in both tension and compression zones. They found that load-bearing capacity increased by about 36% for beams reinforced only in the tension zone and by 47% for beams reinforced on both sides. Failures in the reinforced

beams were primarily due to knots, while the BFRP beam stayed intact and limited crack growth [54]. Figure 2.14 shows the stress–strain relationship for different FRPs and steel. Steel exhibits a yield point and strain hardening, which makes it highly ductile. In comparison, GFRP, BFRP, and CFRP exhibit linear elastic behaviour and then break suddenly, which is a brittle failure. CFRP is the strongest FRP, which can increase strength, but does not add ductility.

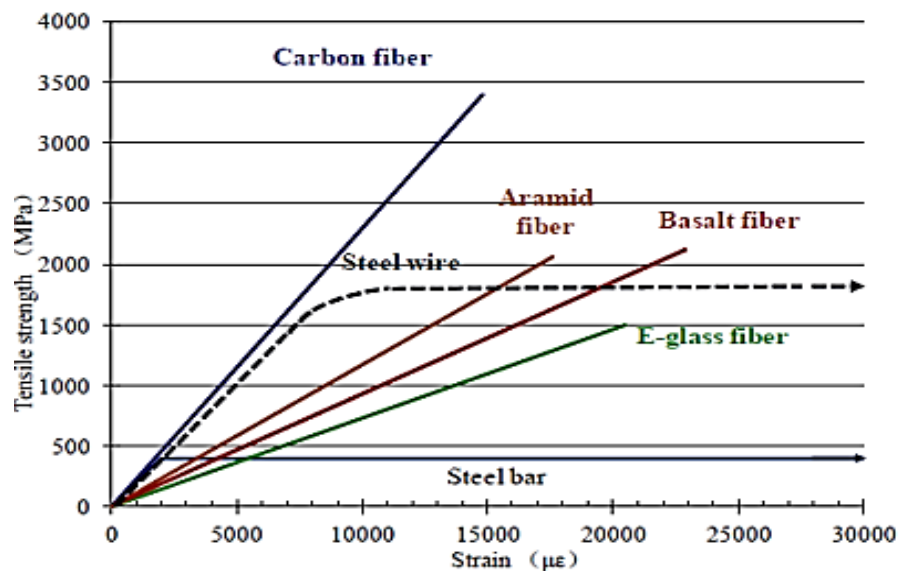


Figure 2.14 Stress-Strain Curve for Reinforced Material [55]

Adding developments in FRP and steel, other research has shown that adding steel reinforcement to GLT beams, especially when prestressed, can significantly increase their strength. More recently, studies have examined combinations of reinforcement methods to enhance the performance of GLT beams under bending. Hybrid models utilize two materials, such as steel and FRP, in conjunction with GLT. For instance, Alkhudery et al. (2023) used this approach by placing NSM steel bars in both the tension and compression zones and wrapping the shear areas with CFRP sheets. This setup increased the ultimate load capacity by 16% to 49%, depending on the reinforcement arrangement, and showed significant improvements in beams reinforced in both flexural zones and wrapped with CFRP. Using only CFRP wrapping resulted in an additional 2–7% improvement over steel-only reinforcement, demonstrating that CFRP is effective in controlling shear and reducing cracks. The hybrid system also altered the way the beams failed, making them less likely to break suddenly in tension and more likely to fail in a ductile manner under compression and shear [56].

Building on these hybrid strategies, recent studies have examined how hybrid-reinforced timber beams perform under repeated loading. Mathuros et al. (2024) tested 16 beams with NSM GFRP bars and steel bars under both steady and repeated bending conditions. The reinforcement ratios ranged from 0.59% to 1.32%. For steady loading, beams with GFRP rods had 20–61% higher load capacity, 58–71% more stiffness, and up to 96% more energy absorption. Under repeated loading, load capacity improved by 22–75%, stiffness by 58–90%, and energy absorption by 19–37%. This was one of the first studies to measure the energy absorption capacity of FRP-reinforced timber beams under repeated loading [57]. Many studies have explored reinforcement materials and configurations, but few have examined bar spacing in GLT beams. Jemaa et al. (2024) needed clear design rules for bar spacing, noting its effect on failure modes and hybrid layouts. This shows that bar spacing is a relatively understudied factor for improving flexural performance and ductility [51].

2.7 Review of the Finite Element Modelling (FEM) in Wood

FEM has become an essential tool in computational mechanics. It helps us simulate how structures respond to different loads and conditions. Over the last twenty years, research has focused on modeling timber beams, particularly GLT and solid timber, under Monotonic loading. These studies have advanced finite element methods to assess the effectiveness of various reinforcement methods, such as FRPs, prestressing, and hybrid reinforcements, in enhancing the mechanical response and predicting accuracy of timber behaviour.

As timber becomes increasingly popular for its sustainability and design flexibility, it is essential to understand how it bends and withstands stress. This is particularly important for reinforced timber beams, such as those using FRPs, prestressing, or hybrid approaches, which directly influence their mechanical performance. Researchers utilize experimental tests, computer simulations, and novel reinforcement designs to enhance the reliability of numerical predictions and improve the strength and efficiency of timber structures.

Early models often used orthotropic elastic definitions, which could show stiffness but not nonlinear responses or failure. Glišović et al. (2017) improved this by adding anisotropic plasticity in compression zones to GLT beams with CFRP plates, demonstrating that these models closely reflect experimental results and underscore the value of directional plasticity. Their model, while simplifying adhesive behaviour, matched experimental results and showed the value of directional

plasticity [58]. Later, Kawecki et al. (2019) employed Hashin's damage criteria and cohesive-zone modelling for adhesives, enabling the realistic simulation of fibre rupture, matrix cracking, and delamination. They emphasized the need for traction-separation laws to model adhesive failure accurately [59]. With these modelling techniques in place, researchers soon shifted their attention to reinforcement approaches. Navaratnam et al. (2022) found that reinforcement with pultruded GFRP bars in GLT beams mitigates strength reductions caused by fibre misalignment, even when considering grain deviation and adhesive interaction [60]. Christoforo et al. (2022) identified key parameters for effective reinforcement of GLT beams with steel bars and developed regression models for design applications using Hill's criterion and isotropic hardening [61]. Luo et al. (2022) demonstrated that their analytical equations accurately predict stiffness and shear stress in the elastic-plastic stage, as validated by finite element analysis and experiments, offering computational efficiency without losing accuracy [62]. To further inform the selection of modelling approaches, Szczecina (2023) compared different model complexities, examining elastic and elastic-plastic timber definitions, as well as whole-section versus laminated cross-section models. He found that elastic models often overestimate stress, whereas elastic-plastic laminated models more closely match experimental results, especially for serviceability limits [63]. This clarified that increasing model sophistication improves the reliability of predictions for design limits.

Parallel to developments in modeling and reinforcement, prestressing has become a promising method for enhancing the performance of GLT beams. Mei et al. (2021) found that maintaining prestress between 18% and 30% of the ultimate load significantly increases stiffness and load capacity. Their finite element tests revealed that prestressing changes how beams fail and distribute stress differently [64]. Kulman et al. (2019) developed a method using pre-curved lamellae glued together, and SolidWorks simulations showed these beams withstand over three times more load and deflect much less than non-prestressed beams [65]. Both studies highlight that prestressing can substantially improve beam performance and reduce material use. In addition to improvements in mechanical modeling and design strategies, Harrach et al. (2022) introduced a probabilistic approach by employing reliability-based finite element analysis. Their results showed that accounting for timber property variability using Monte Carlo methods revealed the importance of reliability indices in structural design. This supports Hill's criterion under uncertainty and improves safety-critical timber engineering [66].

Expanding on these reinforcement and modelling techniques, recent research highlights key performance differences between solid and GLT beams. Kili arslan et al. (2023) tested CFRP-reinforced beams and found that GLT beams outperformed solid timber in all bending tests, achieving up to 77% more load capacity and a 43% higher modulus of rupture key advantages. ANSYS simulations closely matched these experimental results, confirming the reliability of finite element modelling for reinforced timber [67]. Ferrier et al. (2012) introduced a hybrid GLT beam using ultra-high-performance concrete and fibre reinforcement. Their models demonstrated that the hybrids had significantly higher ultimate load capacity despite reduced bending stiffness, suggesting critical new directions for composite timber structures [68].

Adding to these findings, Rajczyk and Jo czyk (2019) investigated BFRP bar reinforcement and found that bar placement and diameter have a significant impact on the load-carrying capacity. Although elastic models could not capture post-yield behaviour, the results provided guidance for optimizing reinforcement layouts to maximize effectiveness [69]. Kandler and F ssl (2020) contributed stochastic FEM frameworks that account for random stiffness variations and material uncertainties. By realistically predicting failure probabilities, these models highlight the importance of reliability-based approaches in robust timber engineering [70].

By concluding, these studies show that modeling and design strategies for GLT beams have significantly advanced. The field initially focused on simple elastic models, but it has now moved towards more sophisticated ones that consider anisotropic plasticity and damage. Further, reinforcement options now include CFRP, GFRP, BFRP, steel bars, and UHPC composites. In addition, improvements in prestressing methods have led to better beam performance, while probabilistic modelling has made designs more reliable. However, many models continue to rely on idealized material properties or simplify how adhesives behave. As a result, key challenges remain, such as accounting for timber variability, defect interactions, and applying research findings to regulations. There is still a major gap between detailed nonlinear modelling and practical reinforcement methods, as well as between these and current codes.

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Chapter 3

Experimental and Numerical Investigation of Glulam Beams under Cyclic and Monotonic Flexural Loading

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3.1 Abstract

The growing use of engineered wood products has increased the application of glued laminated timber (glulam) in modern construction. This study investigates the flexural performance of glulam beams through a combination of experimental testing and finite element (FE) modelling. Full-scale four-point bending tests were performed under monotonic and cyclic loading protocols to evaluate force–displacement responses and failure modes. The experimental results showed that beams under monotonic loading achieved 4.7% higher ultimate load capacity, 13.9% greater midspan displacement, and 24.7% more energy dissipation compared with beams subjected to cyclic loading. The FE models, developed in Abaqus, were validated against the experimental results and provided accurate predictions of beam behaviour under both loading protocols. A subsequent parametric assessment demonstrated the influence of beam geometry on strength, stiffness, and energy dissipation, confirming that larger cross-sections improved structural performance. The combined experimental and numerical approach highlights the capacity of FE modelling to capture the non-linear behaviour of glulam beams and provides a reliable tool for assessing structural performance under varying load conditions. These findings contribute to a better understanding of glulam beam behaviour and support the development of design practices for engineered timber structures.

Keywords: Glued Laminated Timber, Glulam, Cyclic Loading, Monotonic Test, Finite Element Analysis, Structural Timber, Energy Dissipation, Parametric Study, Non-linear Behaviour.

3.2 Introduction

Engineered wood products (EWPs) have gained momentum in modern construction due to favourable mechanical performance, sustainability attributes, and growing code acceptance. Historically abundant in North America, wood has long served residential and civil infrastructure needs [1]. From the early use of fallen trees as makeshift buildings and bridges to contemporary pre-engineered timber systems, wood has consistently demonstrated availability and suitability for structural applications [2]. The construction sector accounts for nearly 39% of global energy-related carbon emissions and has increasingly adopted timber to mitigate environmental impacts [3]. This shift has catalyzed advances in materials and informed the evolution of design guidelines. Driven by technological developments in engineered timber systems—including the context-dependent use of fibre-reinforced polymer (FRP) [4]—the latest editions of the National Building Code of Canada (NBCC) and the International Building Code (IBC) permit encapsulated mass-timber buildings up to 12 and 18 stories, respectively [5] .

Beyond residential construction, untreated timber has demonstrated notable durability in covered bridges, with many structures achieving lifespans exceeding a century. Among EWPs, glued laminated timber (glulam) combines sawn laminations bonded with structural adhesives, enabling large cross-sections, curvature, and prechamber for long-span applications [6, 7]. In highway applications, glulam is frequently used in stringer bridges with transverse glulam decking the predominant form of glulam bridge superstructures [8]. Glulam slab bridges have also been shown to be viable and cost-effective alternatives to precast prestressed double-tee systems for short spans [9]. Fatigue investigations indicate that grain slope markedly reduces glulam fatigue resistance and that flexural fatigue can govern failure, even in cambered, curved, or reinforced members [10–17]. FRP reinforcement consistently improves glulam performance in static bending: approximately 35% stiffness gains have been reported for aged Douglas-fir across reinforcement layouts [18]; with near-surface-mounted (NSM) BFRP, load capacity increases of approximately 47% but stiffness gains of only around 6% have been observed, with failure governed by timber rather than the fabric [19]; and with CFRP strips on the tension face, increases of approximately 35% in capacity, about 43% in stiffness, and nearly 60% in energy dissipation have been reported, while anchorage provides further increases of roughly 20% in stiffness and 26% in energy dissipation with ductility retained [20].

In parallel with these experimental advances, prior studies have shown that appropriately calibrated finite element (FE) models can reproduce the flexural response of glulam beams—with and without FRP reinforcement—under four-point bending [21, 22]. FE analysis thereby offers a cost-effective and scalable means to explore parameter spaces (e.g., section geometry, loading/support conditions, and interface assumptions) that are impractical to cover experimentally [23]. Principal modelling challenges include representing wood orthotropy and tension–compression asymmetry (compression plasticity versus brittle tension), shear/rolling effects, lamination/adhesive interfaces, and contact at supports and load points. With suitable constitutive laws and contact/interface formulations, these effects can be captured at moderate computational cost; recent work demonstrates good fidelity to experimental force–displacement behaviour and failure patterns [23, 24].

Most prior investigations have concentrated on static loading. Given the expanding use of mass timber, there remains a need to assess glulam performance under both monotonic and cyclic (pre-fatigue) protocols. Accordingly, this paper (i) presents full-scale four-point bending tests on glulam beams subjected to monotonic and cyclic loading, (ii) develops and validates a three-dimensional FE model against the experimental response, and (iii) conducts a parametric assessment of the influence of beam geometry on strength, stiffness, and energy dissipation. The combined experimental–numerical approach clarifies performance differences between loading protocols and provides modelling guidance for glulam flexural behaviour.

3.3 Methodology

This section first describes the experimental programme devised to assess full-scale glulam beams in four-point bending under two protocols: monotonic loading to failure and amplitude-controlled cyclic loading. Specimen characteristics (species/grade, lay-up, cross-section, span, and moisture content), boundary conditions, instrumentation, and loading rates are reported. From the measured load–displacement data, initial stiffness, ultimate load, midspan deflection, and energy dissipation (area enclosed by the load–displacement loops) are obtained. Subsequently, a three-dimensional finite element (FE) model is developed in Abaqus to replicate the test configuration. The model includes orthotropic timber behaviour with compression plasticity, appropriate element types and mesh density, and surface-to-surface contact at supports and load points. Model predictions are validated against the experimental response using stiffness, ultimate load, and displacement

metrics. Finally, a parametric study varies section geometry while other variables are held constant, quantifying their influence on strength, stiffness, and energy dissipation for both loading protocols.

3.3.1. Glulam Timber

Wood is commonly classified into two botanical categories: hardwood and softwood. In this study, the glued laminated timber (glulam) beams were manufactured from Douglas-fir (softwood), with laminations oriented parallel to the member's longitudinal axis and bonded with a structural adhesive. The supplier-rated stress grade was 24f-ES/NPG. Manufacturer-reported mechanical properties and the moisture/density range measured at the time of testing are summarized in Table 3.1; nominal specimen geometries are listed in Table 3.2. Moisture content (MC) fell within 7–12% during testing, consistent with service-class indoor conditions. Throughout, f_b denotes bending strength, f_c compression parallel to the grain, and f_t tension parallel to the grain.

Table 3.1 Mechanical properties of glulam beams.

Property	Value
Stress grade	24f-ES/NPG
Bending strength (f_b)	30.7 MPa
Compression parallel to grain (f_c)	33.0 MPa
Tension perpendicular to grain (f_t)	20.4 MPa
Modulus of Elasticity (MOE)	11 770 MPa
Density	560 kg/m ³
Moisture content (MC)	7–12%

Table 3.2 Geometric information of glulam beams.

Beam ID	Width (mm)	Depth (mm)	Length (mm)
B1	184	292	3000
B2	184	292	3000
C1	184	292	3000
C2	184	292	3000
B3	184	343	3000
B4	184	343	3000

Specimen identifiers distinguish nominally identical replicates within each size and loading protocol (see Section *Destructive four-point bending tests* for protocol allocation and

instrumentation). All beams were stored and tested in a controlled indoor environment, and no preservative treatment was applied.

3.3.2. Non-destructive Testing (NDT)

Recent advances in non-destructive testing (NDT) for timber include colour measurement [25], ultrasonic/CT tomography [26], Resistograph drilling resistance [27], acoustic wave velocity [28], and active infrared thermography [29]. In this study, NDT was used to estimate the modulus of elasticity (MOE) of the glulam beams prior to bending tests. Longitudinal stress-wave transit time was measured using a Fakopp Microsecond Timer [30]. Two spike sensors (Start and End) were driven along the fibre direction on each beam, ensuring a clear path free of knots or visible defects between them. As illustrated in Figure 3.1, a steel hammer was used to excite the Start sensor repeatedly, and the travel time to the End sensor was recorded with microsecond resolution.

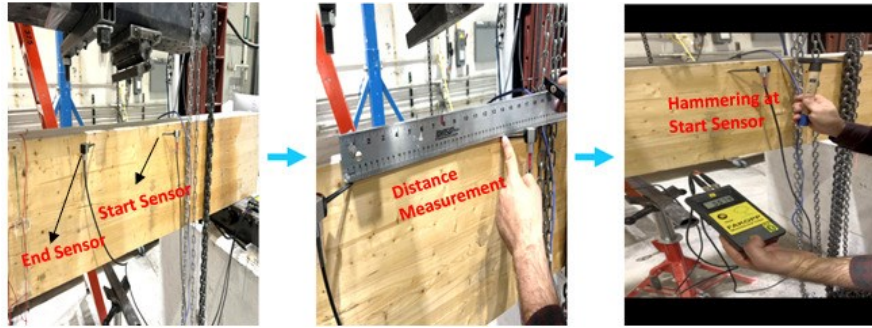


Figure 3.1 Non Destructive Measurements with Stress-Wave Transit Time (schematic).

The wave speed was obtained from the measured distance L and travel time Δt as $v = L/\Delta t$, and the modulus of elasticity (MOE) was estimated as shown in Equation 3.1.

$$E \approx \rho v^2 \quad (3.1)$$

where ρ is the timber density and v is the measured stress-wave velocity between the two points. Moisture content (MC) was assessed with a pin-type meter and fell within 7–12%, consistent with indoor service conditions discussed in Table 3.1. These NDT measurements informed the baseline elastic properties used in the subsequent finite element (FE) modelling.

3.3.3. Destructive Four-Point Bending Tests

3.3.3.1. Specimen Preparation and Instrumentation

Six Douglas-fir glulam beams were tested: four in monotonic bending (B1, B2, B3, B4) and two in cyclic bending (C1, C2). Nominal mechanical properties and geometries are reported in Tables

3.1 and 3.2. Three 5 mm foil strain gauges (vinyl-coated flat leads) were bonded with instant adhesive at midspan—one in the compression zone, one in the tension zone, and one near the neutral axis—to monitor strain distribution during loading. A linear potentiometer (LVDT) at midspan recorded vertical displacement.

3.3.3.2. Test Set-up and Loading Protocols

A four-point bending configuration was adopted in accordance with ASTM D198 [31]. Loads were applied via a hydraulic actuator and measured by an in-line load cell. A steel spreader beam created two equal line loads at one-third point (i.e., the load points were spaced at $L/3$). Supports provided a simply supported boundary condition: a roller (allowing in-plane translation and rotation) at one end and a pin (restraining translation but allowing rotation) at the other. The arrangement is shown schematically in Figure 3.2. Forces and corresponding displacements were acquired continuously to construct force–displacement curves.

Monotonic protocol.

Displacement control at a constant rate of 0.01 mm/s was applied until failure. Key response metrics included initial stiffness, ultimate load, and midspan displacement at peak load; failure modes were recorded visually.

Cyclic protocol.

Displacement-controlled cycles followed an amplitude-escalation scheme: 0.01 mm/s to ± 4 mm; 0.02 mm/s to ± 8 mm; 0.03 mm/s to ± 16 mm; 0.04 mm/s to ± 24 mm; and 0.05 mm/s to ± 32 mm, continuing until failure, consistent with practices referenced in [31, 32]. Each cycle ramped from zero to the target displacement, returned to zero, and repeated symmetrically. Energy dissipation was computed as the area enclosed by the load–displacement loops.

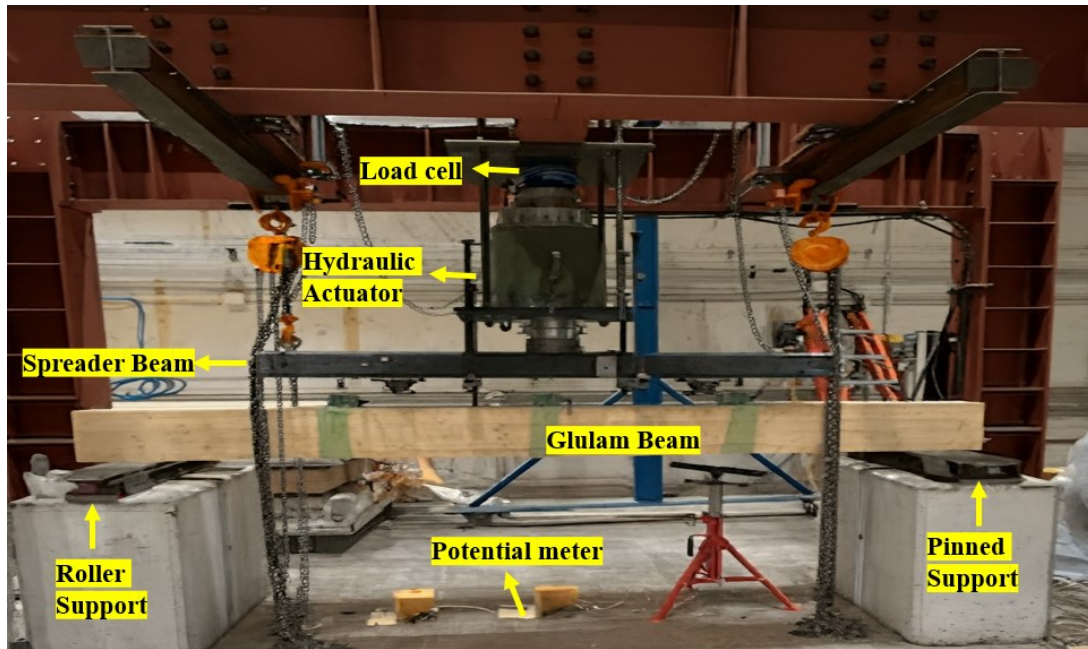


Figure 3.2 Non-Destructive measurements with stress-wave transit time (schematic).

3.3.3.3. Derived Quantities

From the recorded signals, the following quantities were obtained: (i) initial stiffness (initial tangent of the force–displacement curve), (ii) ultimate load and corresponding midspan displacement, (iii) for cyclic tests, cumulative and per-cycle energy dissipation (loop area), and (iv) qualitative failure patterns. These metrics were later used to calibrate and validate the finite element (FE) model and to support the subsequent parametric assessment.

3.4. Results And Discussion

The experimental results are discussed in terms of ultimate load, midspan displacement, energy dissipation (for cyclic tests), and observed failure modes. In summary, all beams exhibited brittle behaviour governed by along the grain cracking that initiated near local weaknesses (e.g., finger joints or defects) and propagated rapidly. Table 3.3 compiles the key response metrics and failure observations, while Figures 3.3 illustrate the monotonic and cyclic force–displacement responses and representative failure patterns.

Table 3.3 Experimental results of the glulam beams.

Beam ID	Test type	Force F (N)	Midspan disp. (mm)	Initial stiffness K (N/mm)	Energy to ultimate E (kN·mm)	Observed failure modes
B1	Monotonic	200,930	28.64	7,887.9	2,780	Brittle tension failure. At finger joints.
B2	Monotonic	200,478	26.54	7,711.8	2,640	Brittle failure initiation. Along-grain cracking observed.
C1	Cyclic	196,008	24.98	7,845.3	2,180	Brittle failure initiation. Along-grain cracking observed.
C2	Cyclic	186,561	22.53	8,146.1	1,900	Brittle tension failure. Finger-joint separation at bottom.
B3	Monotonic	295,180	24.47	12,065	3,790	Knot at bottom initiated failure. Along-grain cracks observed.
B4	Monotonic	280,970	24.01	12,180.3	3,660	Brittle tension failure. Localised in the tension zone.

Pairwise differences (%) were as follows: between specimens B1 and B2, $F = 0.22$, $\delta = 7.33$, $K = 2.23$, $E = 5.03$; between C1 and C2, $F = 4.8$, $\delta = 9.8$, $K = 3.83$, $E = 12.8$; and between B3 and B4, $F = 4.8$, $\delta = 1.87$, $K = 0.95$, $E = 3.43$.

3.4.1. Monotonic Tests

Table 3.3 indicates a 7% difference in maximum displacement between beams B1 and B2 and about a 2% difference between B3 and B4. These variations are consistent with the inherent heterogeneity of timber, such as grain slope, defect distribution, and microcracking. Figure 3.3 presents the corresponding force–displacement curves for beams B1–B4.

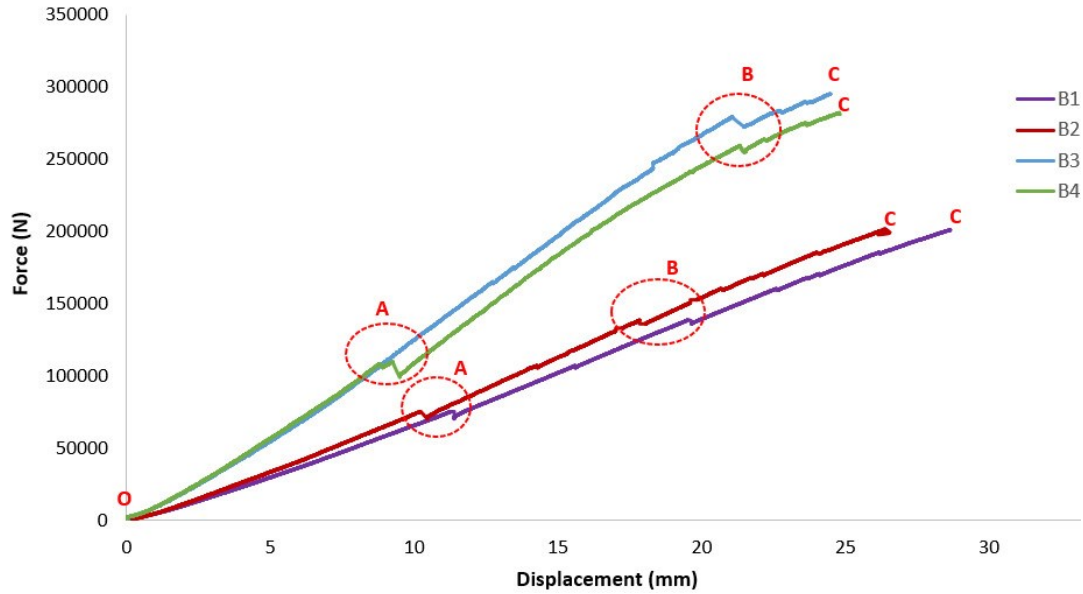
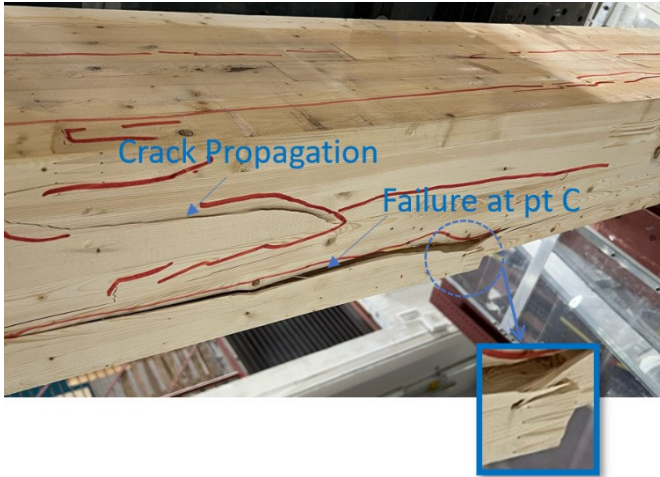


Figure 3.3 Force-displacement profiles (Monotonic tests).

An initial quasi-linear regime was followed by a nonlinear response associated with compression-side inelasticity. The first post-peak drop (region A) was attributable to localised loss of capacity in weaker laminations or finger joints. Stress then redistributed to the remaining intact laminations (region A–B), producing a partial load recovery before accelerated crack propagation caused a second drop (region B) and ultimate failure at point C. The bending strength of glulam is known to be sensitive to lamella tensile strength and finger-joint performance [33].

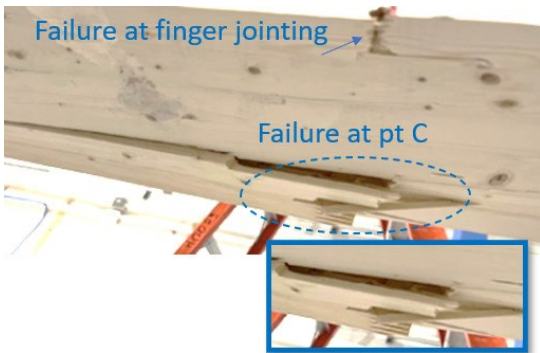
Overall, the monotonic tests confirmed brittle, tension-governed failure characterised by horizontal cracks propagating along the grain. Figures 3.4 illustrate typical failure patterns observed in beams B1, B2, and B4. In B1 and B4, finger joints located at midspan within the tension zone coincided with abrupt collapse, while beam B2 displayed brash behaviour (local splintering) prior to tensile rupture.



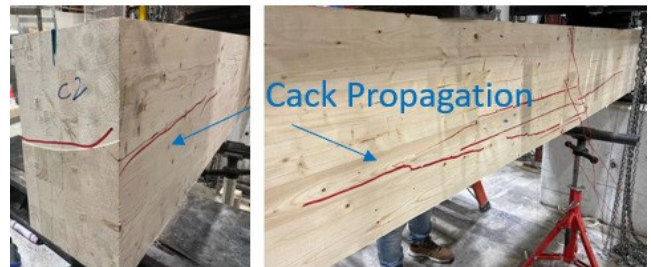
(a) B1 — tension failure with crack propagation.



(b) B2 — beam middle failure.



(c) B4 — beam bottom view.



(d) C2 — shear/along-grain cracks under cyclic loading.

Figure 3.4 Representative failure modes observed in the glulam beams: (a)–(c) monotonic tests; (d) cyclic test.

3.4.2. Cyclic Tests

For the cyclic specimens, Table 3.3 reports a 9.8% difference in maximum displacement between C1 and C2 and approximately a 12% difference in energy dissipation, reflecting both material variability and pre-existing microdefects. Figure 3.5 shows the half-cyclic, displacement-controlled responses.

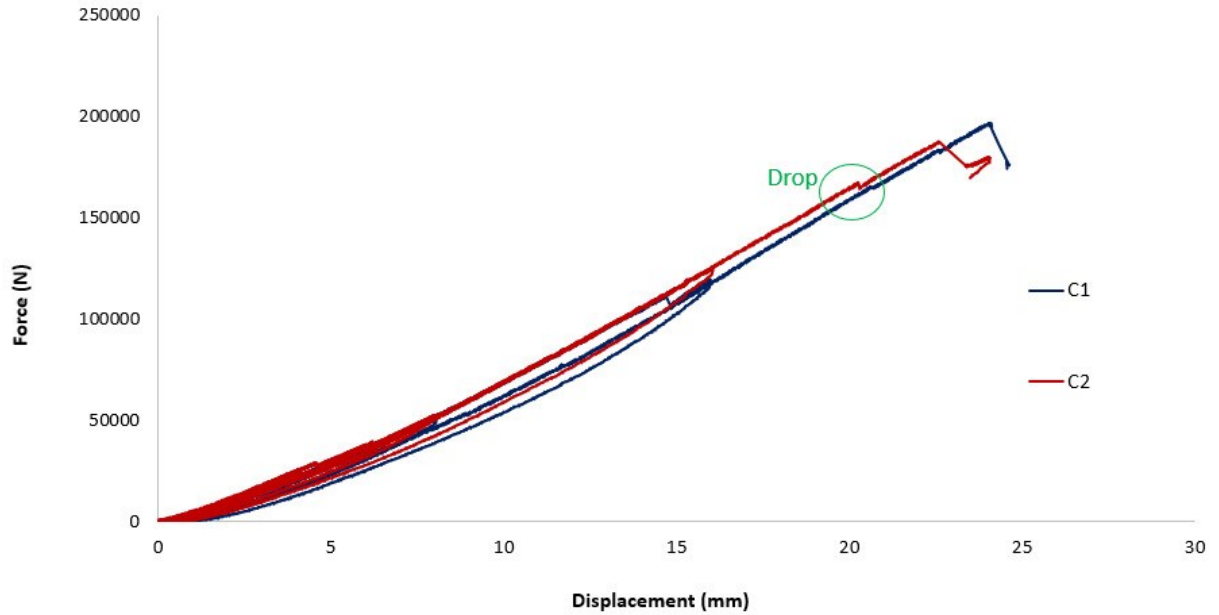


Figure 3.5 Force-displacement profiles (Cyclic tests).

An initial elastic trend, up to roughly 10 mm, transitioned to widening hysteresis loops as local damage accumulated, indicating progressive stiffness degradation and increasing energy dissipation. Beam C2 experienced a sudden post-peak load drop near 21 mm, consistent with a localised tensile rupture in the bottom lamella or a bond-related weakness near the tension zone, whereas beam C1 sustained loading more steadily beyond peak, exhibiting a comparatively more ductile response. The larger hysteresis loop areas for C1 indicated higher energy dissipation than C2. Figure 3.4d shows the shear cracks observed in C2 under cyclic loading, underscoring the sensitivity of cyclic performance to small differences in defects and grain patterns.

Monotonic Loading Protocol

Finite element (FE) models were developed in Abaqus 2025 [34] for two beam sizes, $184 \times 292 \times 3000$ mm and $184 \times 343 \times 3000$ mm, and validated against the monotonic four-point bending tests. Glulam was assumed homogeneous, with all laminations of the same strength grade, permitting each beam to be modelled as a single solid part without explicitly meshing individual laminations. The solid was discretized using reduced-integration eight-node brick elements (C3D8R), suitable for three-dimensional bending-dominated response. Simply supported boundary conditions were applied, with a roller at one end and a pin at the other; in-plane translation was free at the roller, while vertical displacement was restrained at both supports. Load and support blocks were modelled as deformable steel parts. Reference points were defined at the two load-application locations and kinematically coupled to the top faces of the loading blocks to ensure uniform load transfer.

A partition through the beam depth delineated tension and compression zones for property assignment only, without introducing any physical interface. Timber–steel interaction was modelled using surface-to-surface contact with hard pressure-overclosure in the normal direction and a tangential penalty law with a friction coefficient of $\mu = 0.3$, following practices consistent with [23]. Figure 3.6 shows the finite element setup for specimen B1.

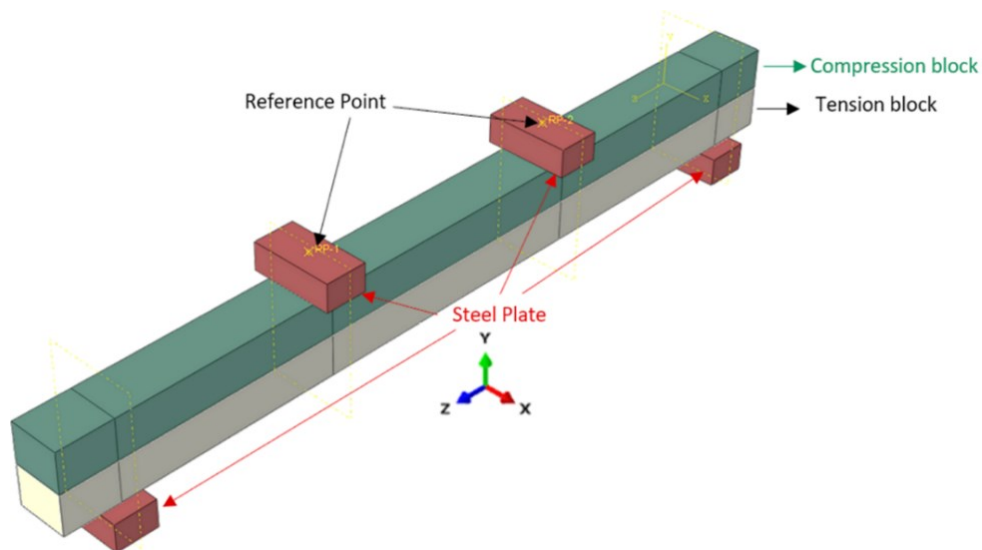


Figure 3.6 Finite Element Model.

Orthotropic elastic properties were assigned via engineering constants to capture the anisotropy of timber. The longitudinal modulus E_1 was estimated from the pre-test stress-wave (NDT) measurements, whereas the remaining elastic moduli, Poisson's ratios, and shear moduli were adopted from the literature [1]; Table 3.4 summarizes the adopted values. Consistent with typical timber behaviour, tension was treated as brittle and linearly elastic, while compression incorporated plasticity.

Table 3.4 Orthotropic elastic properties adopted for the glulam numerical model.

Property	Value	Description
E1	11,770 MPa	Longitudinal modulus.
E2	800.36 MPa	Radial modulus.
E3	588.5 MPa	Tangential modulus.
ν_{12}	0.449	Poisson's ratio in the longitudinal–radial plane.
ν_{13}	0.39	Poisson's ratio in the longitudinal–tangential plane.
ν_{23}	0.374	Poisson's ratio in the radial–tangential plane.
G12	918.06 MPa	Shear modulus in the longitudinal–radial plane.
G13	753.28 MPa	Shear modulus in the longitudinal–tangential plane.
G23	82.39 MPa	Shear modulus in the radial–tangential plane.

E and G are reported in MPa; Poisson's ratios ν are dimensionless

Identical orthotropic elastic properties were used for both zones, with compression plasticity defined only in the compression region using stress–strain data for Douglas-fir glulam [35]. Displacement-controlled loading was applied vertically at the two load points to reproduce the experimental four-point bending configuration. The resulting model outputs (force–displacement curves) were directly compared with the experimental results, as presented in Figure 3.7.

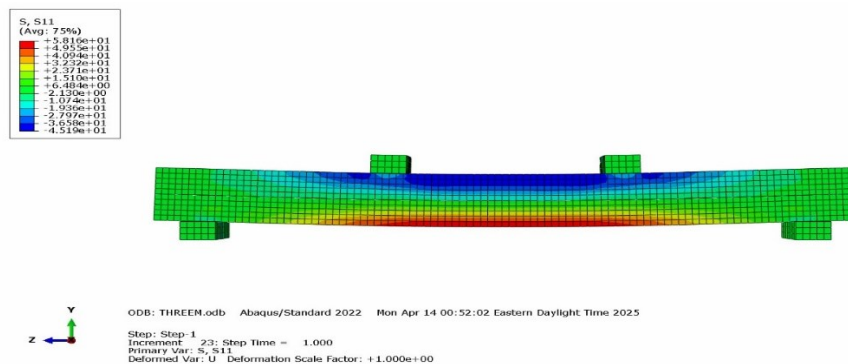


Figure 3.7 Analysis results of B1.

Figure 3.8 compares the experimental and numerical results for both beam sizes. For the shallower beams (B1 and B2), the FE curves differed from the experiments by an average of approximately 9% at peak load, while for the deeper beams (B3 and B4), the average difference was around 6%. These discrepancies are attributed primarily to idealized boundaries and contact assumptions, as well as uncertainties in local material properties. Overall, the agreement between experimental and numerical results was considered satisfactory, confirming the accuracy and reliability of the monotonic FE model for subsequent parametric analyses.

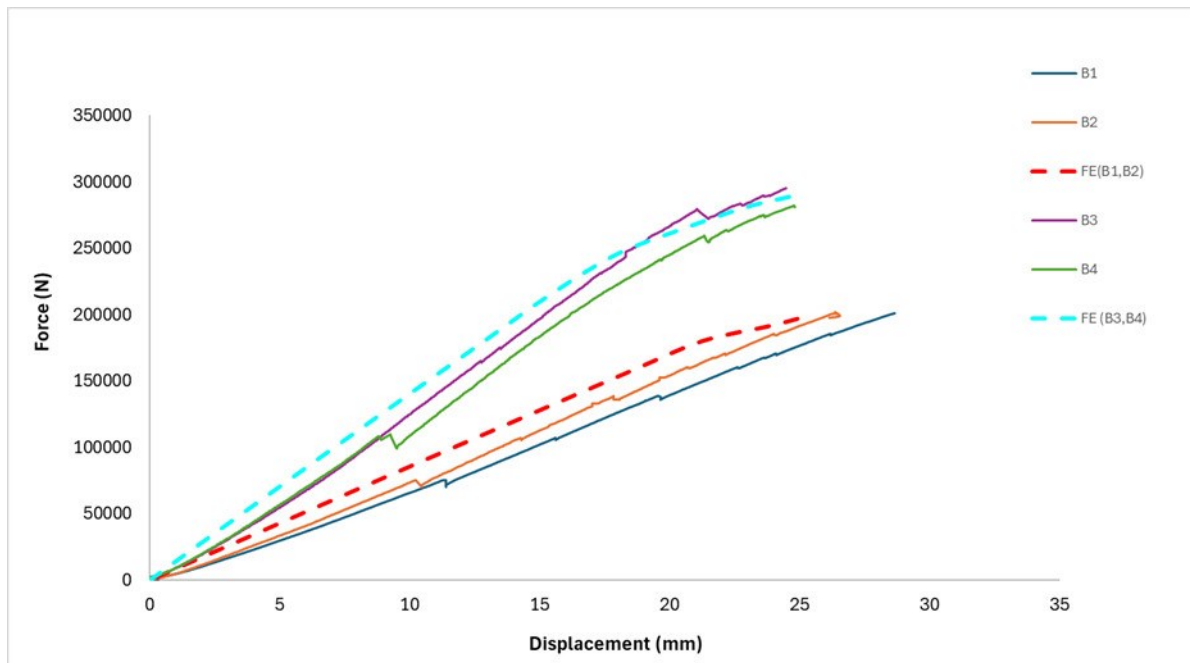


Figure 3.8 Validation of FE model with experimental work (Cyclic).

Cyclic Loading Protocol

An FE model was developed in Abaqus [34] to simulate the behaviour of glulam beams under the half-cyclic (compression-dominant) displacement-controlled protocol used in the laboratory and validated against the corresponding tests. The modelling workflow—geometry, mesh with C3D8R elements, tension/compression zoning by partition, boundary conditions, contact, and orthotropic properties with compression plasticity—followed that of the monotonic model to ensure consistency. The cyclic loading history was imposed via a tabular amplitude definition in Abaqus, reproducing the one-sided, amplitude-escalating displacement sequence adopted experimentally.

The simulated force–displacement hysteresis curves were compared with the corresponding tests, as shown in Figure 3.9.

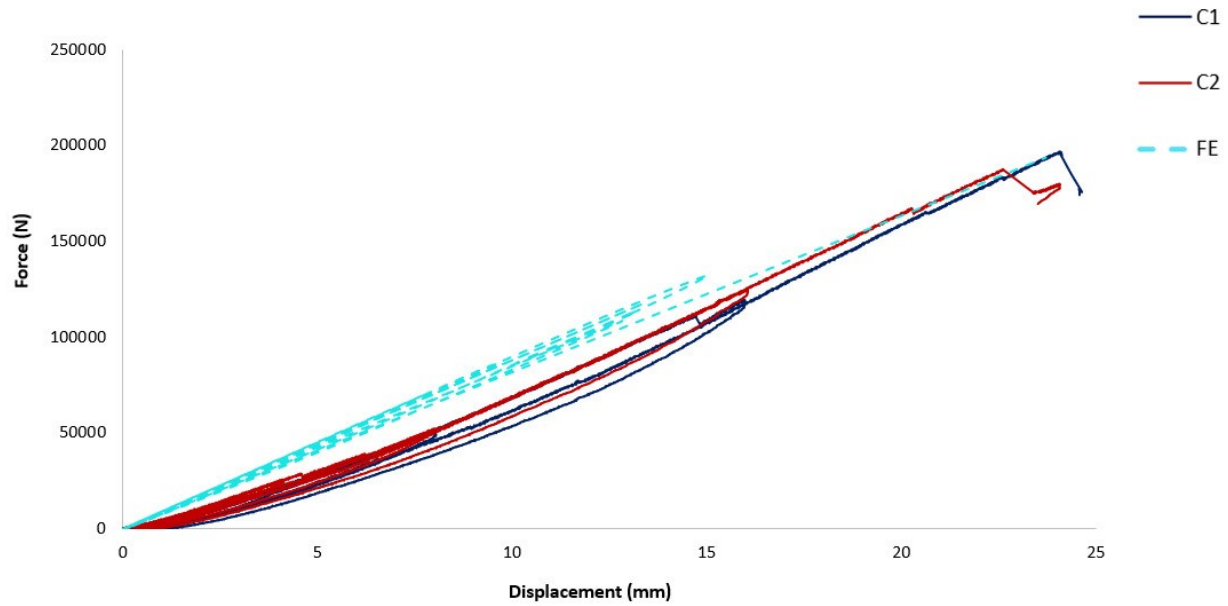


Figure 3.9 Validation of the FE model under half-cyclic loading: numerical vs. experimental hysteresis responses (specimens C1 and C2).

Table 3.5 presents the comparison results, indicating average absolute differences of approximately 2% in peak force and 5% in midspan displacement, confirming the fidelity of the cyclic FE model (FEcyclic) for subsequent analyses.

Table 3.5 Comparison of numerical and experimental results for monotonic and cyclic protocols.

Beam size (mm)	Test type	$Force_{num.}$ (N)	$Disp_{num.}$ (mm)	$Force_{exp.}$ (N)	$Disp_{exp.}$ (mm)	ΔF (%)	$\Delta \delta$ (%)
184×292×3000	Monotonic	194862.4	25.3	200704	27.59	3	9
184×292×3000	Cyclic	193684.1	22.7	191284.5	23.75	2	5
184×343×3000	Monotonic	281927	22.94	288074.9	24.24	2	6

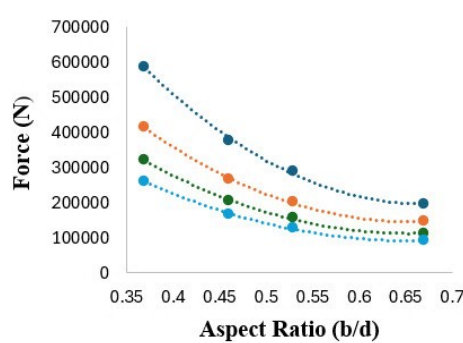
Notes: The percentage differences were computed as $\Delta F = |F_{num} - F_{exp}| / F_{exp} \times 100 \%$ and $\Delta \delta = |\delta_{num} - \delta_{exp}| / \delta_{exp} \times 100 \%$. Numerical values are reported with the precision provided by the simulations, while experimental values represent the average of the corresponding tests.

3.5 Parametric Assessments

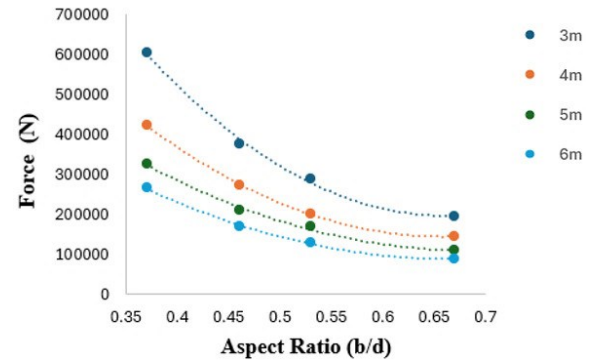
Effect of Aspect Ratio on Strength and Stiffness under Monotonic and Cyclic Loading

Protocols

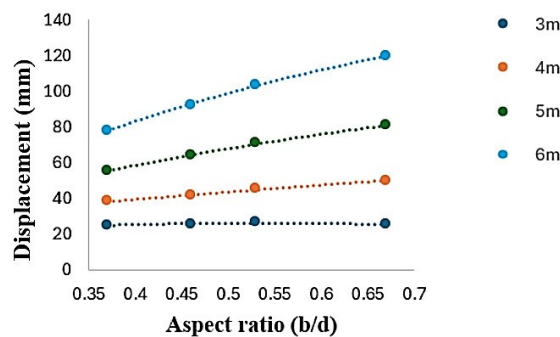
Validated FE models (FEmono and FEcyclic) were used to quantify the influence of cross-sectional aspect ratio—defined as the width-to-depth ratio b/d —and span length on flexural response. The beam width was kept constant at $b = 184$ mm, while the depth d was varied as 292, 343, 394, and 492 mm (i.e., $b/d \in \{0.63, 0.53, 0.46, 0.37\}$). For each depth, spans of $L = 3000, 4000, 5000$, and 6000 mm were analyzed under both monotonic and half-cyclic displacement-controlled four-point bending. This factorial variation isolated geometric effects on load capacity, deformability, stiffness, and energy dissipation. Trends are illustrated in Figure 3.10, and results are summarized in Tables 3.6 and 3.7.



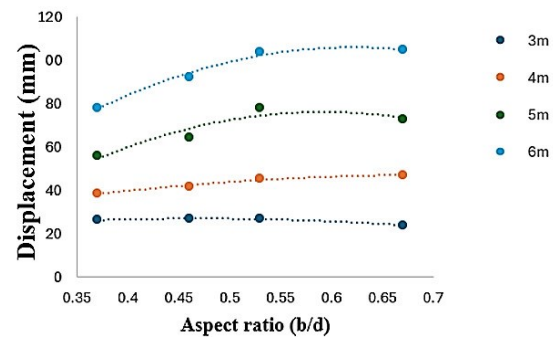
(a) Force versus aspect ratio (Monotonic).



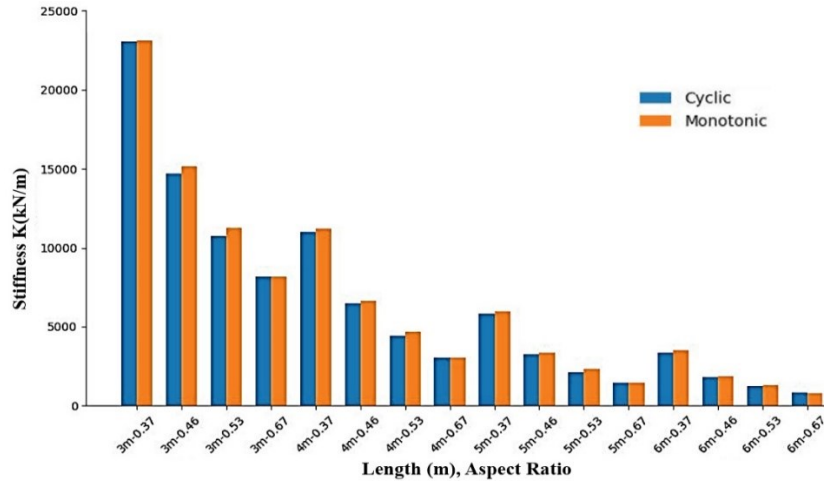
(b) Force versus aspect ratio (Cyclic).



(c) Displacement versus aspect ratio (Monotonic).



(d) Displacement versus aspect ratio (Cyclic).



(e) Stiffness versus aspect ratio (Cyclic and Monotonic).

Figure 3.10 Parametric assessment under a monotonic and cyclic loading protocol.

Table 3.6 Parametric assessment under monotonic loading (FEmono).

Beam dimension (mm)	Aspect ratio b/d	Initial stiffness K (N/mm)	Energy to $F_{ult.}$ E (kN·mm)	Max force F (N)	Max disp. δ (mm)
184×292×3000	0.63	7860.18	2690	194862.4	25.3
184×292×4000	0.63	3035.4	3963	145285	50.13
184×292×5000	0.63	1450.3	4942	112314	80.83
184×292×6000	0.63	810.88	6017	91473.4	119.55
184×343×3000	0.53	11248	6125	288000	26.74
184×343×4000	0.53	4693.4	9741	201053	45.18
184×343×5000	0.53	2314.2	6016	154910	70.81
184×343×6000	0.53	1291.4	7192	126217	103.65
184×394×3000	0.46	15184	5158	375502	25.55
184×394×4000	0.46	6656.6	6003	265138	41.67
184×394×5000	0.46	3363	7176	204912	63.96
184×394×6000	0.46	1888.6	8473	166890	92.42
184×492×3000	0.37	23100	8189	584308	25.17
184×492×4000	0.37	11226	8562	412574	38.34
184×492×5000	0.37	5956.6	9712	318859	55.76
184×492×6000	0.37	3514.7	11150	259693	78.09

Table 3.7 Parametric assessment under monotonic loading (FEcyclic).

Beam dimension (mm)	Aspect ratio b/d	Initial	Energy to $F_{ult.}$ E (kN·mm)	Max force F (N)	Max disp. δ (mm)
		stiffness K (N/mm)			
184×292×3000	0.63	8172	2435	193684.1	23.7
184×292×4000	0.63	3045.18	3608	142819	46.9
184×292×5000	0.63	1477.39	4196	107406.4	72.7
184×292×6000	0.63	821.16	4611	85967.71	104.69
184×343×3000	0.53	10764.95	4165	287747	26.73
184×343×4000	0.53	4453.321	4930	200844.8	45.1
184×343×5000	0.53	2138.229	6278	167124	78.16
184×343×6000	0.53	1239.429	7359	128566	103.73
184×394×3000	0.46	14686.1	5148	375083	26.54
184×394×4000	0.46	6509.973	6178	271140.4	41.65
184×394×5000	0.46	3259.812	7323	208465	63.95
184×394×6000	0.46	1806.547	8482	166943	92.41
184×492×3000	0.37	23058.28	8430	602974	26.15
184×492×4000	0.37	11003.76	8774	421334.1	38.29
184×492×5000	0.37	5820.223	9881	324477.5	55.75
184×492×6000	0.37	3377.033	13100	263442.4	78.01

Maximum force vs. aspect ratio:

Across spans and loading protocols, lower b/d ratios (i.e., deeper sections) produced higher peak strengths. The best performance was observed for b/d = 0.37 (d = 492 mm), reaching $F_{max} \approx 5.84 \times 10^5$ N under monotonic loading and 6.03×10^5 N under cyclic loading. In contrast, the most slender sections (b/d = 0.63) exhibited the lowest capacities, with $F_{max} \approx 9.15 \times 10^4$ N (monotonic) and 8.60×10^4 N (cyclic) for L = 6000 mm. Figures 3.10(a)–(b) show that strength decreases as b/d increases, with sensitivity most pronounced for longer spans when $0.35 \lesssim b/d \lesssim 0.55$.

Midspan displacement vs. aspect ratio:

At lower b/d ratios (i.e., greater depths), midspan displacements were smaller, while increasing b/d led to higher deflections due to reduced stiffness. Under monotonic loading, displacement exhibited a nearly linear relationship with aspect ratio, showing mild concavity at $L = 6000$ mm [Figure 3.10(c)]. Under cyclic loading, however, a more pronounced concave trend was observed [Figure 3.10(e)], attributable to additional hysteretic energy dissipation caused by internal friction, microcracking, and localized plasticity. This effect was strongest for deeper sections and shorter spans, where bending moments were highest.

Initial Stiffness vs. Aspect Ratio:

The initial stiffness was defined according to Equation (3.2) [35–38]:

$$K = \frac{P}{\delta} \quad (3.2)$$

where K is the initial stiffness, P is the applied load, and δ is the corresponding midspan displacement. The value of K was evaluated from the load–displacement curve within the 10–40% interval of the peak load, consistent with prior studies [31, 33, 36–38]. As summarised in Tables 3.6 and 3.7 and illustrated in Figure 3.10(e), the largest stiffness occurred for $b/d = 0.37$ (e.g., $K \approx 23,100$ N/mm under monotonic loading and 23,058 N/mm under cyclic loading for $L = 3000$ mm), while the smallest stiffness values corresponded to $b/d = 0.63$ (e.g., $K \approx 811$ – 821 N/mm for $L = 6000$ mm). Differences between monotonic and cyclic protocols in K were small for matched geometries.

Energy to Ultimate Load

The energy to ultimate load (E) was computed as the area under the envelope (or “backbone”) of the load–displacement response up to the maximum displacement $\delta_{\max}$, as expressed in Equation (3.3):

$$E = \int_0^{\delta_{\max}} F(\delta) d\delta \quad (3.3)$$

where $F(\delta)$ represents the applied force as a function of displacement. Figure 3.11 provides an example based on beam B3, where $F_{\max} \approx 292 \text{ kN}$ at $\delta \approx 25 \text{ mm}$.

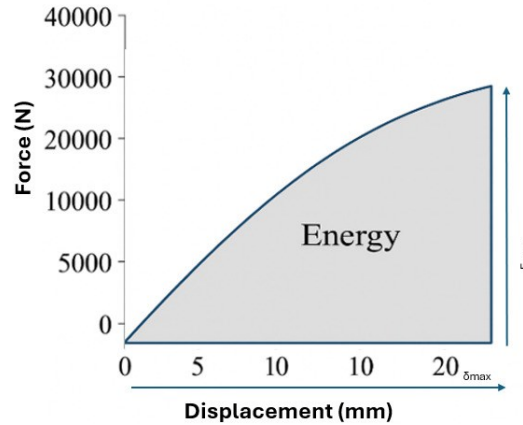


Figure 3.11 Energy curve for beam B3.

Consistent with the strength and stiffness trends, higher aspect ratios ($b/d = 0.63$) exhibited greater E values than lower aspect ratios ($b/d = 0.37$), reflecting increased deformability. Across matched geometries, cyclic loading dissipated on average approximately 30.6% more energy to ultimate than monotonic loading.

3.6. Conclusions

This study examined the monotonic and cyclic flexural performance of glued-laminated timber (glulam) beams. Three-dimensional finite element (FE) models were developed and validated against experimental results, and a parametric assessment quantified the influence of aspect ratio (b/d) and span on strength, stiffness, displacement, and energy-related response metrics. The principal findings are as follows:

- Destructive tests performed on two deeper beams ($184 \times 343 \text{ mm}$) and two shallower beams ($184 \times 292 \text{ mm}$) under monotonic loading revealed an 8% difference in maximum displacement between specimens B1 and B2, and approximately 2% between B3 and B4, reflecting natural material variability. All beams failed in a brittle, tension-governed mode. Two additional $184 \times 292 \text{ mm}$ beams tested under cyclic loading exhibited an 11% difference in maximum displacement (C1 versus C2), likewise attributable to the inherent variability of wood.

- Under monotonic loading, the deeper beams (B3 and B4) outperformed the shallower beams (B1 and B2), exhibiting 43.5% higher load capacity, 55.4% greater initial stiffness, and 37.5% higher energy dissipation. These findings align with the aspect-ratio trends observed in the parametric analysis. On average, monotonic tests achieved 4.7% higher ultimate load, 13.9% greater midspan displacement, and 24.7% more energy dissipation compared with cyclic tests, indicating superior load-bearing capacity under monotonic conditions. Conversely, cyclic tests exhibited slightly higher initial stiffness (approximately +2.5%), likely because cyclic loading induces earlier microdamage and reduces subsequent energy absorption.
- The validated finite element models (FEmono and FEcyclic) reproduced the experimental peak forces within approximately 5.3% and 4.42% accuracy, respectively. Residual discrepancies were primarily attributed to idealized boundaries and contact assumptions, as well as local variations in material properties. The parametric study confirmed that smaller aspect ratios (e.g., $b/d = 0.37$) and shorter spans ($L = 3$ m) maximize stiffness and strength, whereas larger aspect ratios ($b/d = 0.63$) and longer spans lead to reduced structural performance under both loading protocols.

Building on these conclusions, a natural extension of this research is to characterize cyclic degradation more comprehensively by adopting fully reversed loading protocols and/or explicit fatigue criteria (both low- and high-cycle), thereby enabling a clearer quantification of stiffness reduction and damage accumulation. Additional research priorities include: (i) assessing the sensitivity of flexural response to variations in moisture content, temperature, and lamination grade (homogeneous versus non-homogeneous layups); (ii) refining the numerical representation to incorporate explicit lamination and adhesive interfaces, as well as finger joints, in order to capture failure initiation sites more accurately; and (iii) extending model calibration to larger datasets and alternative constitutive formulations (e.g., orthotropic damage–plasticity), including probabilistic calibration for reliability-based assessment. Collectively, these developments would enhance the applicability of the validated framework and support performance-based design of glulam members under both service and extreme loading conditions.

Credit Authorship Contribution Statement

Hassan Malik: Formal analysis, validation, finite element modelling, experimental setup and testing, writing—original draft preparation and editing. Laith Alhelalet: Experimental test setup. Ghazanfarah Hafeez: Conceptualization, methodology, investigation, formal analysis, writing—original draft, review and editing, supervision, and data analysis.

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Declaration of Conflicting Interests

The authors declare no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

Data Availability Statement

The data supporting the findings of this study are available from the corresponding author upon reasonable request.

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Chapter 4

Flexural Performance of Hybrid Glulam Beams: Influence of Reinforcement Material and Arrangement

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(Submitted to Wood Material Science & Engineering) (Submitted)

4.1. Abstract

Glued Laminated Timber (GLT) beams reinforced with steel and Fibre-Reinforced Polymers (FRPs) can significantly enhance flexural strength and stiffness in timber structures. However, current design standards, such as CSA O86 and CSA S6:19, provide no guidance on how to arrange reinforcement or on the use of hybrid material reinforcement configurations in GLT beams. This research investigates the effect of horizontal bar spacing and various configurations of hybrid reinforcement material on the flexural response of GLT beams. The methodology includes experimental testing, theoretical approaches, and Finite Element Modelling (FEM) to achieve the objectives of this study effectively. The GLT beams underwent monotonic four-point bending tests, reaching an average peak load of approximately 200 kN and an initial stiffness of 7.8 kN/mm. This experimental data supported the development of a FEM in Abaqus. The model simulated reinforced beams with bar spacings ranging from 15 to 120 mm and reinforcement ratios of 1% to 3%. Steel was used in tension, while GFRP, BFRP, or CFRP were used in compression. The CFRP steel hybrid performed best, reaching a peak load of 432.4 kN at a 3% reinforcement ratio. This was a 116.38% increase over the reference beams. GFRP and BFRP hybrids led to lower strength and stiffness gains due to their lower modulus of elasticity compared to CFRP. However, all reinforced beams benefited from the ductile behaviour of steel in the tension zone. These results show that hybrid reinforcement can significantly improve GLT beam design.

Keywords: Monotonic Test, Glulam, GLT, Beams, Finite Element Model, Theoretical Analysis, Hybrid Model, GFRP, CFRP, BFRP, ABAQUS

4.2. Introduction

Timber has been a fundamental material in structural engineering due to its favourable strength-to-weight ratio and ease of handling [1]. However, solid-sawn lumber is limited by natural defects

and size constraints, prompting the development of Engineered Wood Products (EWP). Among these EWPs, Glued Laminated Timber (GLT) has emerged as a primary solution. GLT is produced by bonding multiple layers of lumber with adhesives, with grains aligned parallel to the beam length. This manufacturing process enables longer spans, custom cross-sections, and greater reliability by reducing the effect of defects.[2]. GLT is widely used in structural applications such as beams, trusses, and bridges, offering significant sustainability advantages as a renewable material capable of sequestering carbon [3]. Despite these benefits, GLT beams are prone to brittle flexural failure, especially when subjected to repeated or high intensity loading conditions [4].

To address these challenges, reinforcement techniques have been developed to enhance the strength of GLT beams. One commonly adopted method for integrating these reinforcements is the Near Surface Mounted (NSM). This method strengthens GLT beams by inserting reinforcement bars into shallow grooves cut along the beam's surface. These reinforcement bars are bonded to the surrounding wood using epoxy adhesives, forming a bond that allows effective stress transfer between the two materials [5]. creating composite material with enhanced mechanical performance. Reinforcing steel or Fibre Reinforced Polymers (FRPs) are highly effective and have become a topic of interest today, as they improve stiffness and load capacity. Steel provides ductility and yield strength, while FRPs, such as carbon (CFRP), glass (GFRP), and basalt (BFRP), offer high tensile strength and corrosion resistance [6]. Gentile et al. (2002) demonstrated that using FRP reinforcement in timber beams can increase the strength of Douglas fir members with NSM GFRP bars. They reported flexural strength gains of 18–46%, depending on the reinforcement ratio, while failure continued to occur in tension [7]. Fiorelli and Dias (2003) strengthened pine beams using CFRP. The authors found that strength increased by 20-40% and stiffness improved by 15-30%, with better ductility at failure [8]. Buell et al. (2005) demonstrated that externally bonded FRP sheets significantly enhanced the flexural response of timber beams. The stiffness increased by 17–27%, and the strength improved by 40–53%. The reinforcement also altered the failure mode so that instead of sudden tensile rupture, the beams experienced a more gradual compressive crushing. Additionally, the use of CFRP enhanced beam confinement and mitigated the negative impact of inherent defects, such as knots and splits [9]. De Luca et al. (2012) shifted attention to steel reinforcement, embedding non-prestressed and prestressed steel bars in GLT beams, resulting in stiffness increases of 25.9–40.2% and improved ductility compared to beams without reinforcement, indicating that steel can facilitate stress redistribution before brittle

failure [10]. Raftery and Whelan (2013) demonstrated that bonded-in GFRP rods can significantly enhance the flexural performance of low-grade GLT beams, with moment capacity gains of up to 68%. The authors further observed that positioning the rods farther from the neutral axis improved strength utilization, highlighting the importance of reinforcement layout [11]. Yang et al. (2016) compared reinforcement methods in GLT beams. One study found GLT beams reinforced with GFRP bars, CFRP plates, and steel bars increased flexural capacity by about 56%, stiffness by 28%, and tensile strain at failure by 49%. Analytical model predictions were within 10% [12]. Another study showed prestressed CFRP bars increased capacity by up to 131% and stiffness by 42%. Adequate anchorage stopped failures on the compression side and prevented brittle tension rupture [13].

Rajczyk and Jończyk (2019) reinforced GLT beams with basalt FRP rods, reporting capacity gains of up to 68% and stiffness improvements of 15% [14]. Although CSA O86-24 [15] provides general provisions for timber design; it does not address the reinforcement of timber or GLT beams with steel or FRP bars. Similarly, CSA S6-19 includes specific provisions for timber elements in bridge applications but lacks explicit guidance for GLT reinforcement [16]. This knowledge gap highlights the necessity for research-based recommendations. Corradi et al. (2021) investigated the effect of adding CFRP sheets over knots and defects in beams, finding that this approach can partially restore bending capacity and stiffness by bridging fibre gaps and limiting local breaks [17]. Similarly, Yeboah and Gkantou (2021) tested NSM basalt and glass FRP bars in spruce beams, observing that while the ultimate load increased by 33–69% and stiffness by 22–33%, brittle tensile failure remained the primary issue [18].

Recently, research has progressed beyond single-material reinforcement and adopted hybrid strategies that combine the stiffness and strength of FRP with the ductility of steel. This transition was driven by the ongoing issue of brittle tensile rupture in FRP reinforced beams and the necessity to achieve more balanced mechanical performance. In these hybrid approaches, FRP specifically serves in the tension zones to provide high strength and stiffness where cracking occurs, while steel is strategically placed in either the compression or tension zones to contribute ductility. Alkhudery et al. (2023) demonstrated that combining NSM steel bars with CFRP shear wraps increased the flexural capacity by 16 to 49%. Furthermore, the authors observed ductile compression-side failure when the steel was in the tension zone [19]. Mathuros et al. (2024) continued this research by placing steel in compression and FRP rods in tension during both

monotonic and cyclic loading. As a result, they found that the bending capacity increased by 20 to 61%, stiffness increased by 58 to 71%, and energy absorption reached up to 96% under monotonic loading [20].

Previous studies show that FRP reinforcement enhances flexural strength and stiffness but often fails in a brittle manner, whereas steel reinforcement offers ductility. When considering hybrid reinforcement strategies to balance these characteristics, research primarily focuses on reinforcement ratio and bar configurations, while keeping the horizontal bar spacing constant. The potential contribution of FRP in compression zones, where it could improve stiffness and reduce localized stress concentrations, remains insufficiently explored in timber beams. To address these gaps, the current study examines the flexural performance of GLT beams using experimental testing, FEM, and theoretical model verification. Initially, unreinforced beams were tested under four-point bending to establish baseline behaviour. The FE model was subsequently developed and validated using these experimental results and then extended to simulate hybrid reinforcement configurations and the effect of bar spacing. The FE modelling predictions were then verified through sectional analysis calculations.

4.3. Methodology

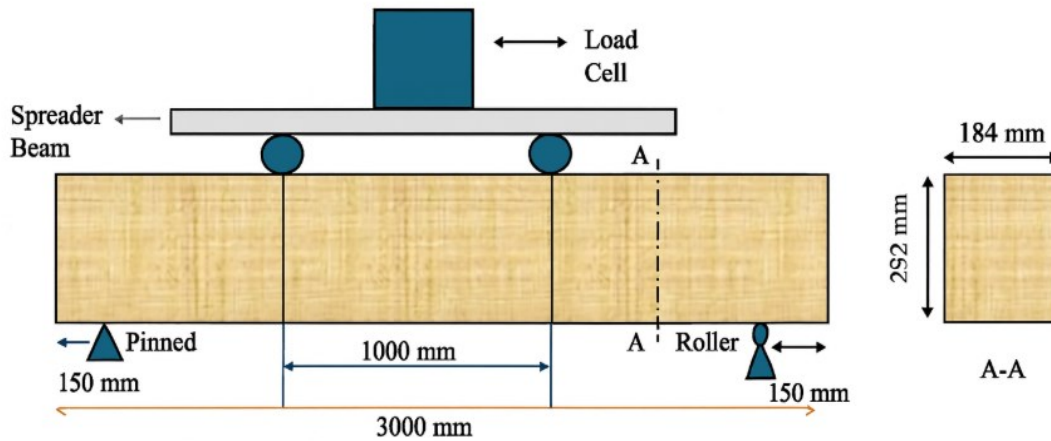
4.3.1. Materials and Test Protocol

The beams in this experimental program were made of GLT, which is made from softwood. Table 4.1 presents their material properties. Two specimens, designated G1 and G2, were tested. Each had cross-sectional dimensions of 0.184×0.292 m and a span length of 3.0 m. To measure the strain, three 5 mm foil strain gauges with vinyl-coated flat leads were bonded at midspan using instant adhesive. One gauge was placed in the compression zone, one in the tension zone, and one near the neutral axis. Vertical displacement at midspan was recorded using a linear variable displacement transducer (LVDT), as illustrated in Figure 4.1 (a) and (b).

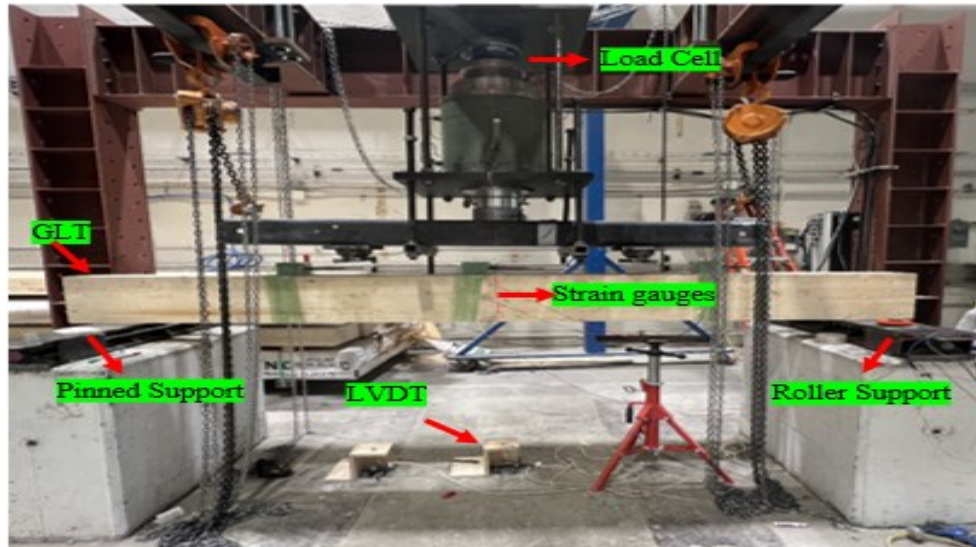
Table 4.1 GLT Beam Material Properties

Properties	Values
Stress grade	24f-ES/NPG
Bending moment (f_b)	30.7 Mpa
Compression parallel to grain (f_c)	33.0 Mpa
Tension perpendicular to grain (f_t)	20.4 Mpa
Modulus of Elasticity (MOE)	11770 Mpa
Density	560 Kg/m ³
Moisture Content (MC)	7% to 12%

The four-point bending tests, as shown in Figure 4.1, were conducted in accordance with ASTM D19 [21], using a hydraulic actuator to apply the load, which was monitored by a load cell. A steel spreader beam distributed the load evenly as two equal forces at the one-third span points ($L/3$). The setup created a simply supported boundary condition with a roller support at one end and a pin support at the other. Force and displacement were continuously monitored to generate the corresponding curves, which serve as the primary basis for assessing beam stiffness and ultimate strength.



(a)



(b)

Figure 4.1 Experimental Setup

4.3.2. Experimental Results and Discussion

The GLT beams in this investigation primarily exhibited brittle tensile failure. Specifically, specimen G1 failed at the finger joint due to concentrated tensile stresses that initiated fracture. In comparison, specimen G2 developed a horizontal crack along the fibre direction, which subsequently propagated and led to brittle failure. The corresponding failure patterns are shown in Figures 4.2(a) and 2(b), respectively. Taken together, these reference results provided essential validation data for the finite element model and, importantly, highlighted the necessity for reinforcement strategies to improve the ductility of GLT beams.

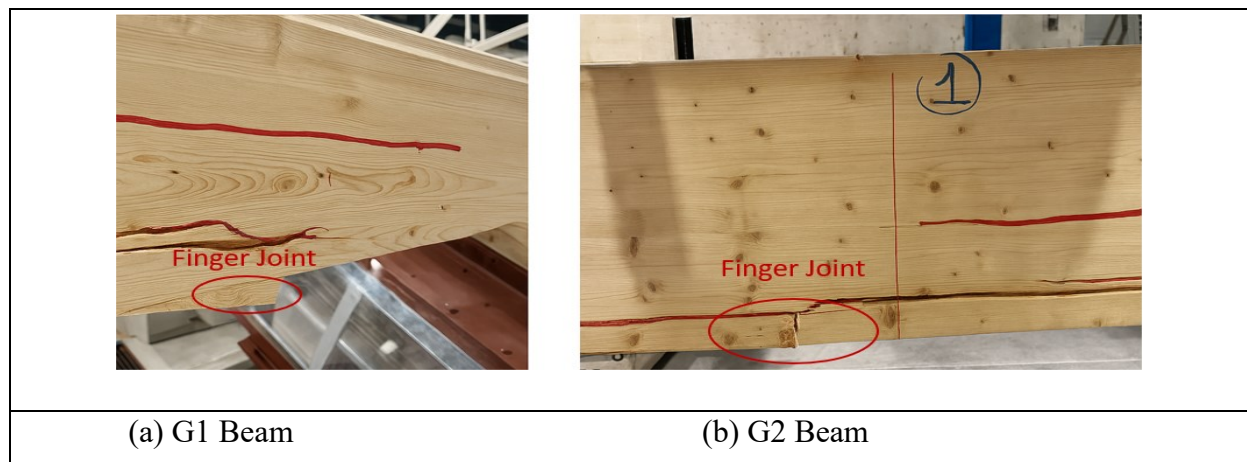


Figure 4.2 Failures of G1 and G2 Beams

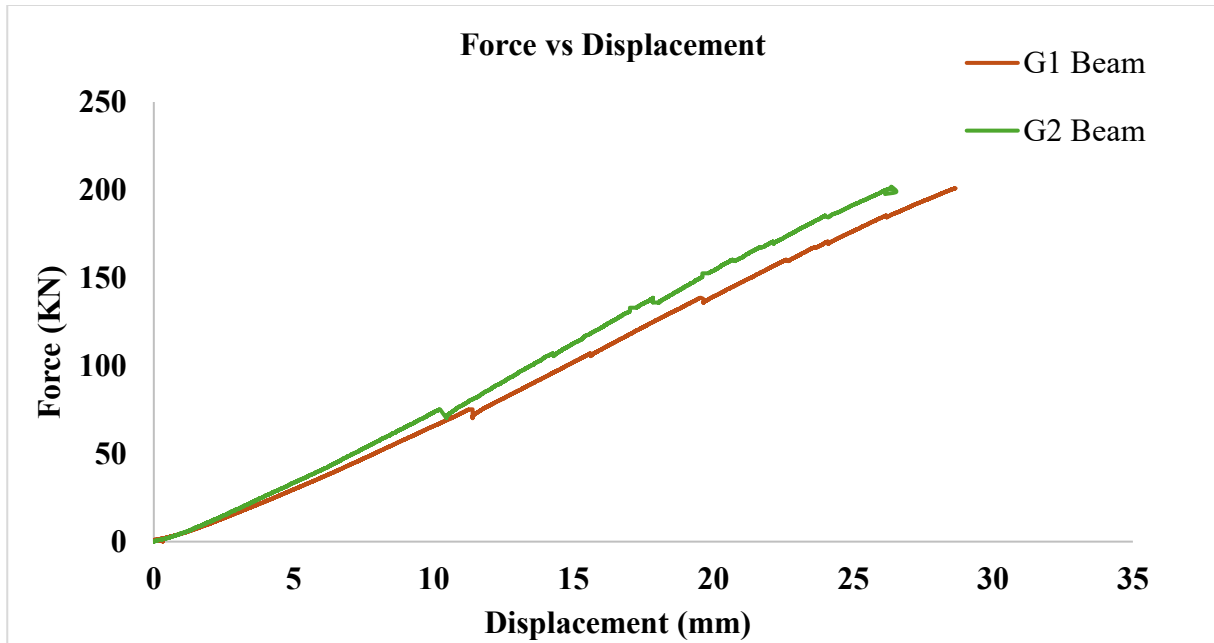


Figure 4.3 Experimental Force Vs Displacement Curve

Table 4.2 Experimental Results

Beam ID	Cross section(m)	Test Type	Force (kN)	Mid Span Displacement (mm)	Moment (kNm)	Initial stiffness (N/mm)	Failure Modes
G1	0.184 x 0.292	Monotonic	200.9	28.64	80.36	7887.9	Brittle tension failure at the finger joint
G2	0.184 x 0.292	Monotonic	198.8	26.54	79.52	7711.8	Brittle failure mode of timber initiated by horizontal cracks along the fibre direction.

The experimental results summarized in Table 4.2 reveal only a marginal difference of approximately 2% in the load-carrying capacity between beams G1 and G2. Conversely, a more substantial variation of nearly 8% in displacement was recorded, attributable to inherent variability in the wood material properties. Figure 4.3 shows a sudden displacement drop between 10 mm and 13 mm, likely due to localized microcracking at the finger joint, delamination, and possibly the

compression in the top fibres. This caused a temporary loss of stiffness, followed by stress redistribution to the remaining laminations, restoring the force–displacement response.

Additionally, initial stiffness was determined following the methodology adopted by Lacroix, (2017) using a range of 10%-40% of the peak load, eliminating early nonlinear behaviour and low-load effects while accurately capturing the linear elastic component of the force-displacement response [22].

The experimental program established the baseline behaviour of unreinforced GLT beams, specifically their stiffness, load-carrying capacity, and displacement response under monotonic bending. To extend the investigation, a finite element model was developed and validated against the experimental data. This validation demonstrated that the FEM framework can reliably reproduce the behaviour of GLT beams, thereby providing a sound basis for its subsequent use in parametric studies of reinforcement configurations and bar spacing studies.

4.4. Finite Element Model Development and Validation

A Finite element model was developed to simulate the four-point bending tests of unreinforced GLT beams. The beam geometry was defined as 0.184×0.292 meters in cross-section with a span length of 3.0 meters. To accurately represent timber behaviour, orthotropic engineering constants were specified. The longitudinal modulus of elasticity E_1 was measured using non-destructive stress-wave measurements. Values for E_2 , E_3 , shear modulus G_{12} , G_{13} , G_{23} , and Poisson's ratios ν_{12} , ν_{13} , and ν_{23} were found from literature [23], as summarized in Table 4.3. To model the material behaviour, tension was considered linear elastic up to rupture, reflecting the brittle nature of wood fibres. At the same time, compression included both yielding and post-yield softening to capture the nonlinear response. For the assembly illustrated in Figure 4.4, a solid section was assigned with correctly oriented orthotropic axes, replicating the experimental setup, including roller and pin supports, and load application points at one-third spans. Monotonic displacement-controlled loading was then applied using a static general step, consistent with the experimental rate of 0.01 mm/s. To ensure the accuracy of the results, mesh refinement was performed near midspan to capture strain gradients, and convergence checks were conducted to confirm mesh independence.

Table 4.3 FEM Material Properties

Property	Value (MPa)	Description
E1	11770	Longitudinal modulus
E2	800.36	Radial modulus
E3	588.5	Tangential modulus
ν_{12}	0.449	Poisson's ratio in the longitudinal-radial plane
ν_{13}	0.390	Poisson's ratio longitudinal-tangential
ν_{23}	0.374	Poisson's ratio in the radial-tangential plane
G12	918.06	Shear modulus in the longitudinal-radial plane
G13	753.28	Shear modulus in the longitudinal-tangential plane
G23	82.39	Shear modulus in radial-tangential plane

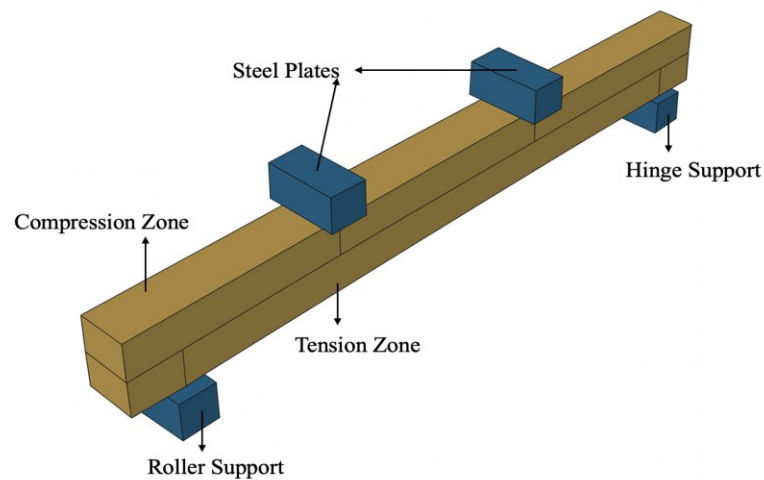


Figure 4.4 FEM Assembly of Unreinforced GLT Beam

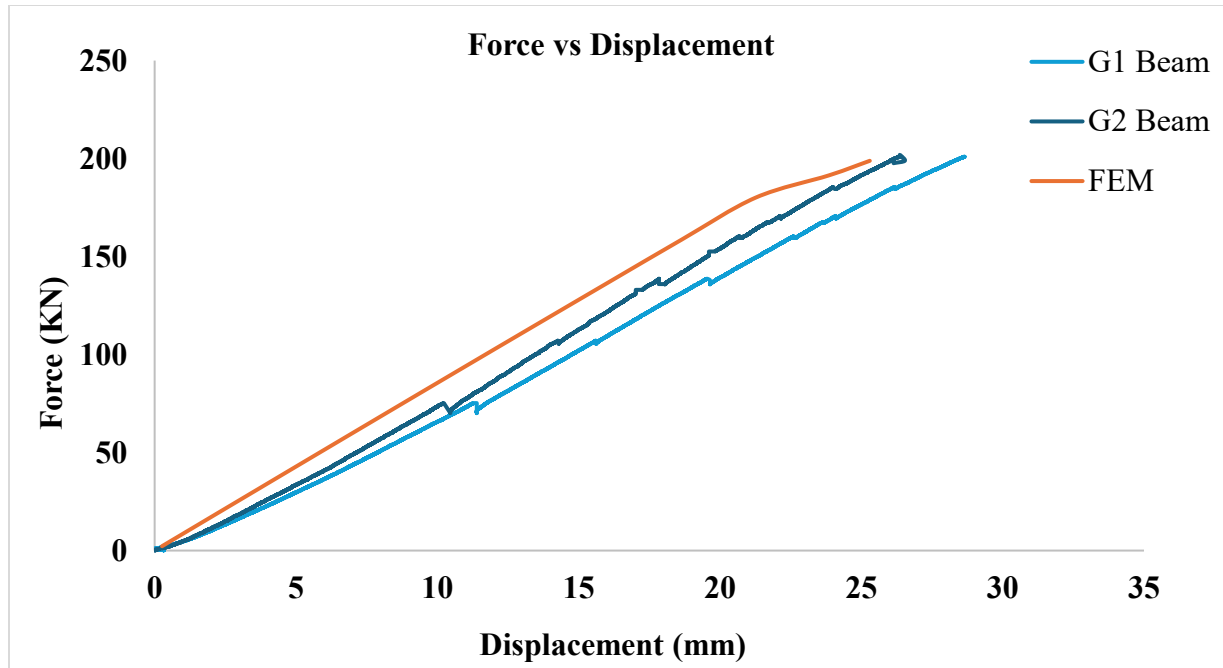


Figure 4.5 FEM Validation Against Experimental Results

FEM analysis accurately reproduced the flexural response of unreinforced GLT beams as demonstrated in Figure 4.5. The numerical model predicted a final load of 198.86 kN and a displacement of 25.3 mm, which closely matched the experimental mean values of 199.85 kN and 27.59 mm, respectively. Deviations between FEM and experiment were less than 10% for both values, indicating the model reliably represents beam stiffness and load capacity under bending, validating the modelling approach for predicting the flexural performance of unreinforced GLT beams.

Following the validation of the unreinforced model, the finite element framework was expanded to simulate reinforced GLT beams using hybrid reinforcement strategies. The reinforced model was built using embedded region technique. In this approach, reinforcement bars were defined as truss elements and embedded within the GLT beam to achieve perfect bond behaviour. Wdowiak-Postulak and Kawecki (2025) reported the material properties of FRPs and steel reinforcement, which were adopted in this study for use in FEM [24]. The hybrid reinforcement scheme of the GLT beam is illustrated in Figure 4.6. Reinforcement ratios ranging from 1% to 3% were chosen to balance structural efficiency with practical considerations [25][26][27]. Plevris and Triantafillou (1992) showed through experiments and analysis that ratios above 3% yield diminishing returns in flexural performance[25]. Dagher et al. (1996) confirmed this experimentally, demonstrating

that efficiency reached a limit once reinforcement ratios exceeded approximately 3%. Together with earlier findings, these results establish a clear assessable threshold for reinforcement efficiency in GLT beams[26]. Bulleit et al. (1989) noted that reinforcement behaviour strongly depends on the method of bar embedding within the tension lamination. Failures consistently start at the bar level. This is due to bond stresses and splitting along the steel wood interface [27]. The evidence suggests that reinforcement effectiveness depends on both the quantity of steel and the detailing and placement. In a beam with multiple bars, spacing within the tension zone is critical. Closely spaced bars create interacting stress fields. These amended stress fields can change bond behaviour and reduce the tensile capacity of the surrounding wood. Therefore, the present study investigated horizontal bar spacing by varying the bar spacing from 15 mm to 120 mm. This established an efficient framework for evaluating both reinforcement ratio and layout effects on the mechanical behaviour of GLT beams.

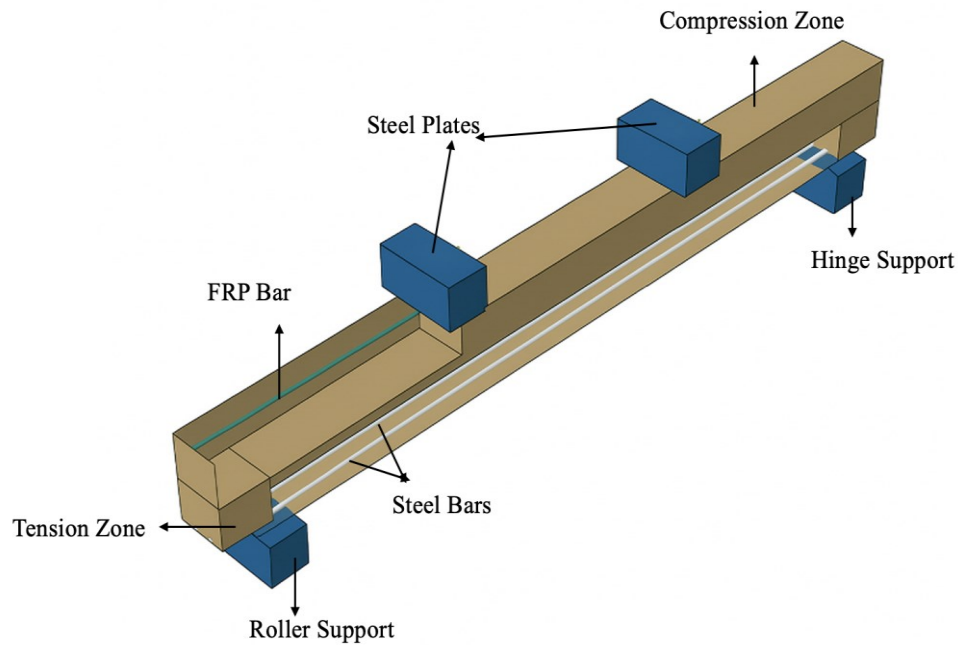
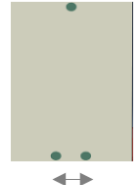
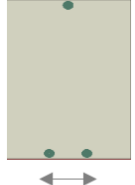
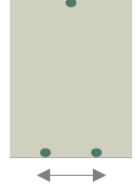
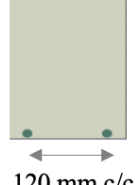


Figure 4.6 FEM Assembly of GLT Beam with Reinforcement Bars

4.2.1. Effect of Reinforcement Spacing on Bending Performance of GLT Beams

Table 4.4 provides the results of various configurations of horizontal bar spacings from (S1-S7) and the outcome parameters, including maximum bending capacity (force) and associated displacement. The configurations include 16 mm diameter reinforcement bars, which were used with a center to center (c/c) spacing of the bars. At a spacing of 15 mm (S1) Configuration, the maximum load capacity reached 246,197 N with a displacement of 33 mm. Increasing the spacing to 30 mm (S2) and 40 mm (S3) resulted in incremental gains, with a peak value of 247,722 N observed at a spacing of 40 mm between reinforced bars. Although this improvement of (S3) is less than 1% compared to the other spacing configurations (S1), it is a mechanically essential parameter because it shows that reinforcement spacing influences both the strength and stiffness of the beam. Despite the slight numerical increase, the consistent trend indicates that spacing contributes to optimizing reinforcement efficiency. For spacings beyond 40 mm (S3), including 60 mm (S4) and larger, no further increase in load capacity was observed. When reinforced bars are placed very close together, multiple grooves must be cut into the timber to accommodate them. This reduces the amount of intact wood in the section and disrupts the natural continuity of the fibres, potentially weakening the beam rather than strengthening it. This (S1) configuration also leads to overlapping bond stress fields, thereby limiting the efficiency of stress transfer. At a moderate spacing of approximately 40 mm (S3), these adverse effects are minimized. Each bar can then mobilize its bond strength independently and contribute fully to load resistance. When spacing exceeds this threshold, the reinforcement ratio becomes the primary factor governing the response and further increases in separation do not affect performance. These findings suggest that 40 mm (S3) spacing is the optimal configuration for maximizing stiffness and load capacity while minimizing adverse interactions. Therefore, this spacing was adopted as the baseline parameter for subsequent assessments of reinforcement configurations in GLT beams.

Table 4.4 Results of Various Horizontal Bar Spacing

Series	Max force (N)	Max Displacement (mm)	Configuration
S1	246197	33	 15 mm c/c
S2	246374	33	 30 mm c/c
S3	247722	33	 40 mm c/c
S4	246343	33	 60 mm c/c
S5	246448	33	 80 mm c/c
S6	246417	33	 100 mm c/c
S7	246313	33	 120 mm c/c

4.2.2. Effect of Reinforcement Ratio (ρ) on Strength and Stiffness of GLT Beams

The parametric study primarily examined both reinforcement ratio and type, assessing their effects on the flexural performance of GLT beams. Notably, the unreinforced beam demonstrated an average load-carrying capacity of 199.85 kN, serving as the baseline for comparison. When FRPs were used in compression and steel in tension zones, significant improvements were observed for all reinforcement ratios ($\rho=1\%$, $\rho=2\%$ and $\rho=3\%$). These results demonstrate the effect of increasing the reinforcement ratio on flexural performance. For beams reinforced with GFRP and steel, the load-carrying capacity increased to 298 kN at a 1% reinforcement ratio, representing a 49.11% improvement or approximately 1.5 times the capacity of the unreinforced beam. At a 2% reinforcement ratio, the capacity increased to 354.3 kN, representing a 77.13% improvement, or approximately 1.8 times that of the unreinforced beam. A 3% reinforcement ratio resulted in a capacity of 419.44 kN, corresponding to a 109.87% increase or approximately 2.1 times higher than the unreinforced beam. The force-displacement curves for the hybrid reinforcement configuration with GFRP, BFRP and CFRP bars in the compression zone and steel bars in the tension zone, including the unreinforced behaviour, are represented in Figure 4.7, Figure 4.8 and Figure 4.9, Figure 4.10, respectively.

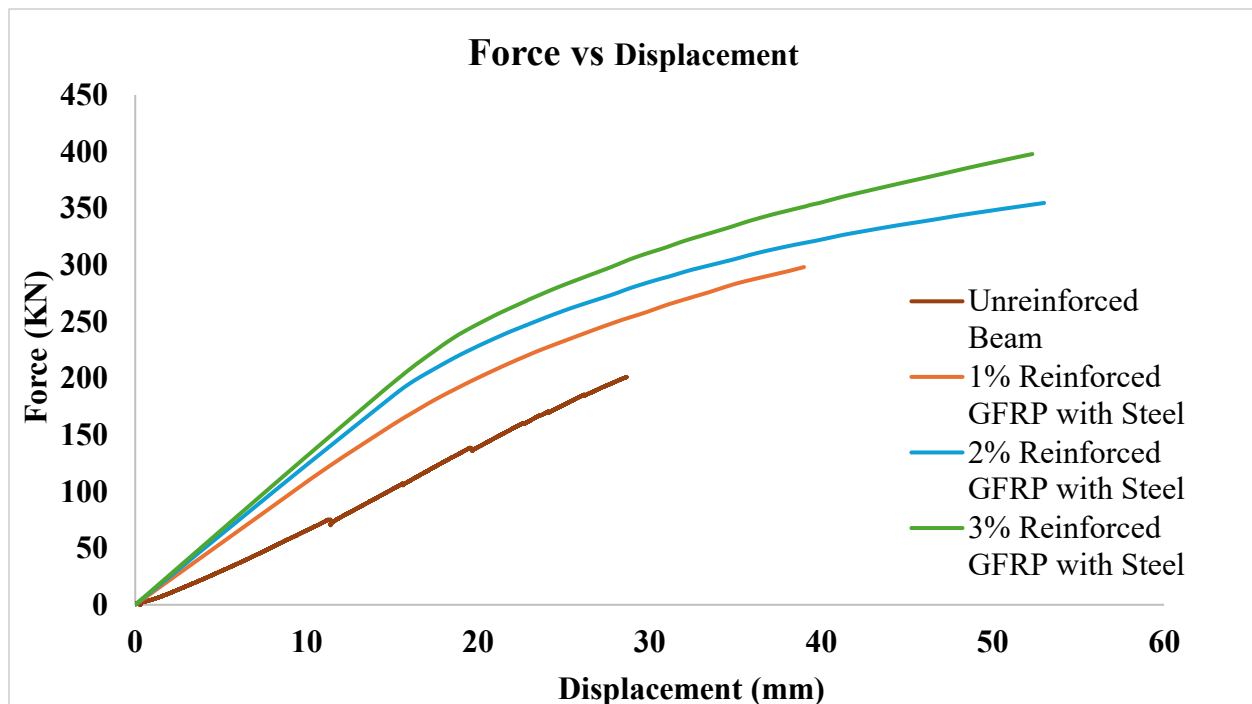


Figure 4.7 Force - Displacement Curve with GFRP and Steel Bars

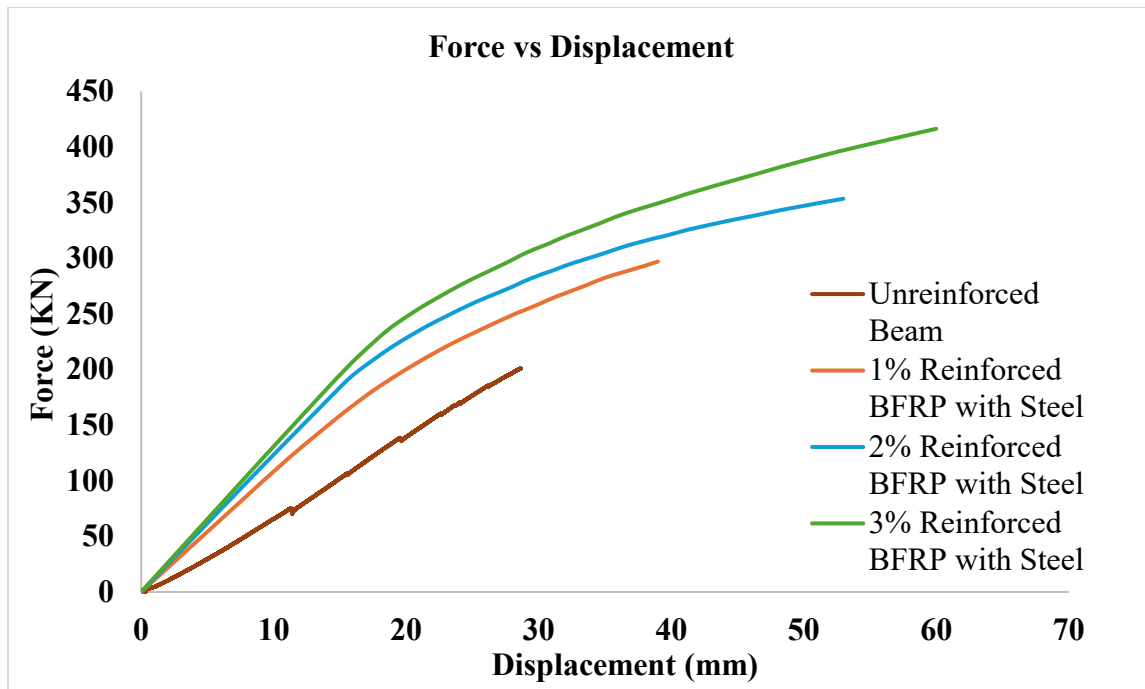


Figure 4.8 Force - Displacement Curve with BFRP and Steel Bars

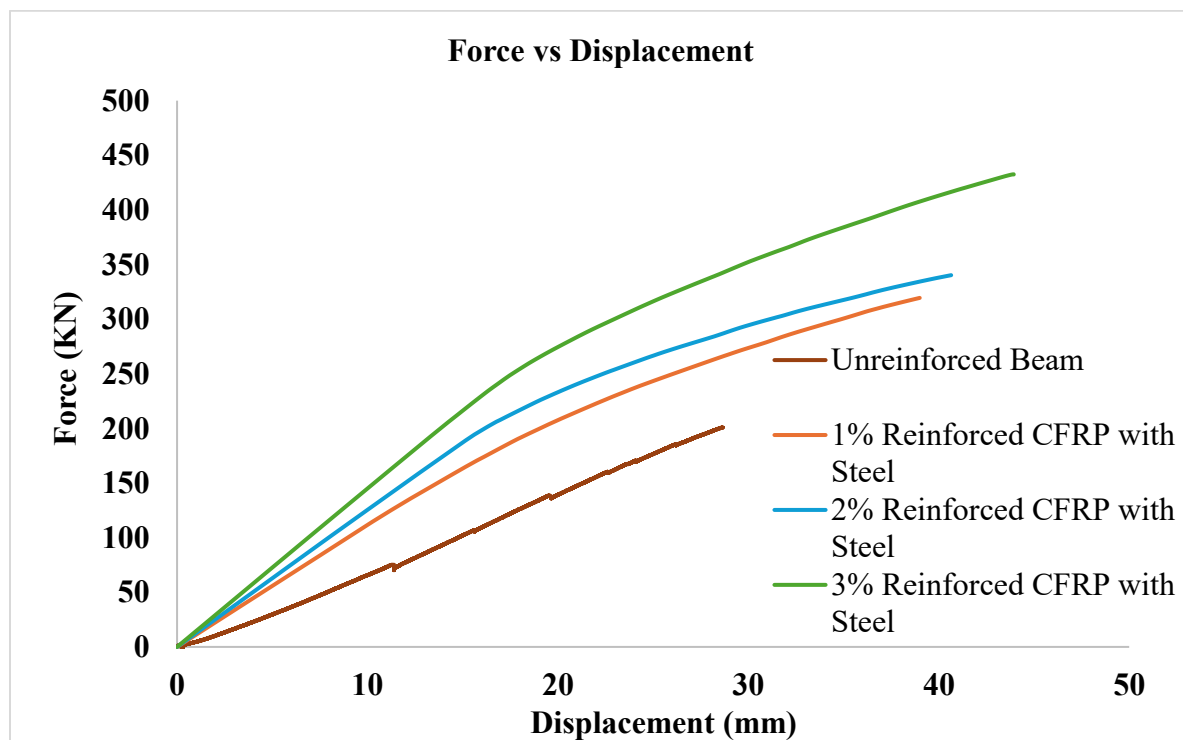


Figure 4.9 Force - Displacement Curve with CFRP and Steel Bars

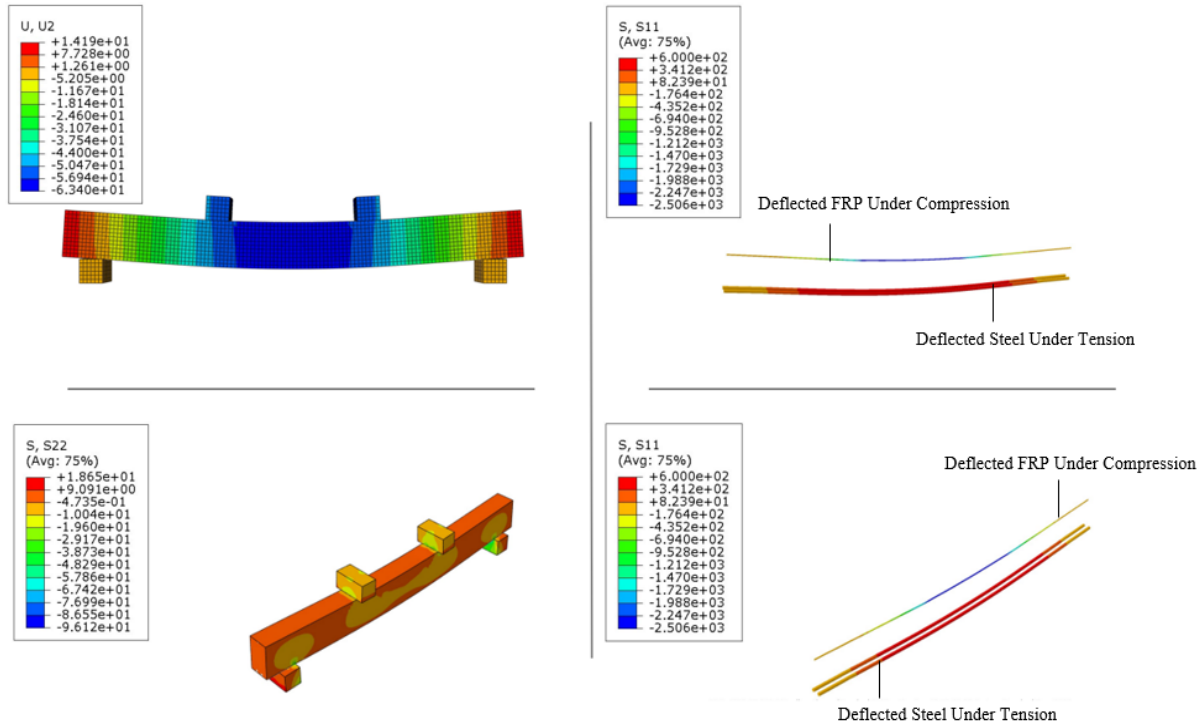


Figure 4.10 FEM Analysis

4.3. Comparison of FEM Analysis and Theoretical Predictions of GLT Beams

The FEM assessment provided reliable predictions of the flexural response of GLT beams, effectively capturing the effects of reinforcement ratio and spacing between bars through simulations. To further verify these results, theoretical sectional analysis was conducted using established stress–strain relationships for both timber and reinforcement materials. These findings confirm that the FEM framework is reliable and theoretically robust, establishing it as a highly dependable tool for parametric studies of hybrid reinforcement configurations as shown in Table 4.5.

The idealized stress-strain relationship for clear wood was initially proposed by Bazan (1980) and subsequently modified by Buchanan (1990) to better represent the behaviour of in-grade lumber. In uniaxial tension, wood exhibits linear elastic behaviour up to failure, characterized by the ultimate tensile stress σ_{gt} and corresponding tensile strain ε_{gt} , as illustrated in **Figure 4.11** [28][29]. Under compression, the material response is idealized as linear elastic up to the compressive yield stress σ_{gcy} and its corresponding strain ε_{gcy} followed by a descending linear

softening branch with a slope defined as mE_{gc} , where E_{gc} denotes the longitudinal modulus of elasticity.

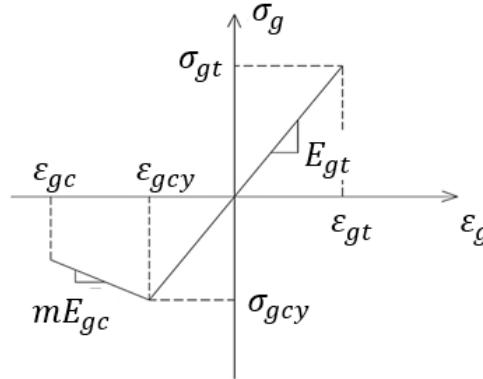


Figure 4.11 Idealized Stress - Strain Curve of Wood

The typical stress-strain behaviour of reinforcing materials, such as steel bar and FRP bar, is also depicted in **Figure 4.12 (a) and (b)** for comparative purposes.

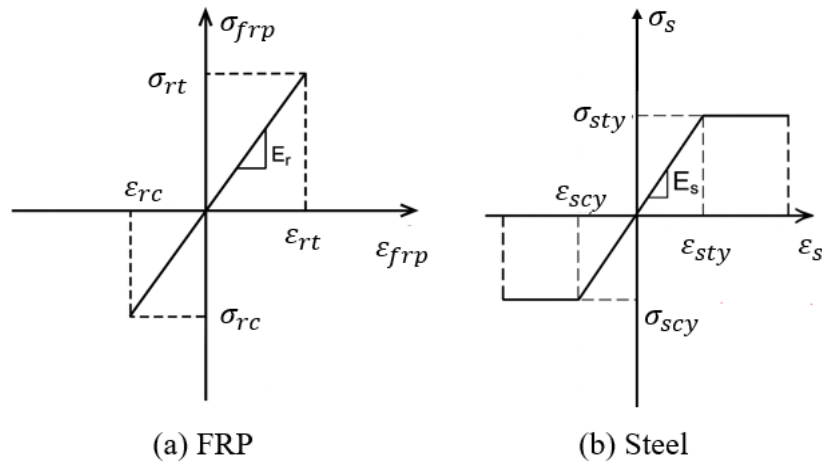


Figure 4.12 Idealized Stress - Strain Curve of (a) FRP and (b) Steel

These idealizations formed the basis for the sectional analysis. To simplify the Analysis, the following assumptions were adopted: (a) a perfect bond between reinforcement and timber, with no slip; (b) shear deformations neglected, restricting the analysis to flexural behaviour; and (c) plane sections assumed to remain plane under bending. A theoretical sectional analysis was

conducted for steel and FRP reinforced GLT beams. The cross-sectional configuration used in the analysis is presented in **Figure 4.13**.

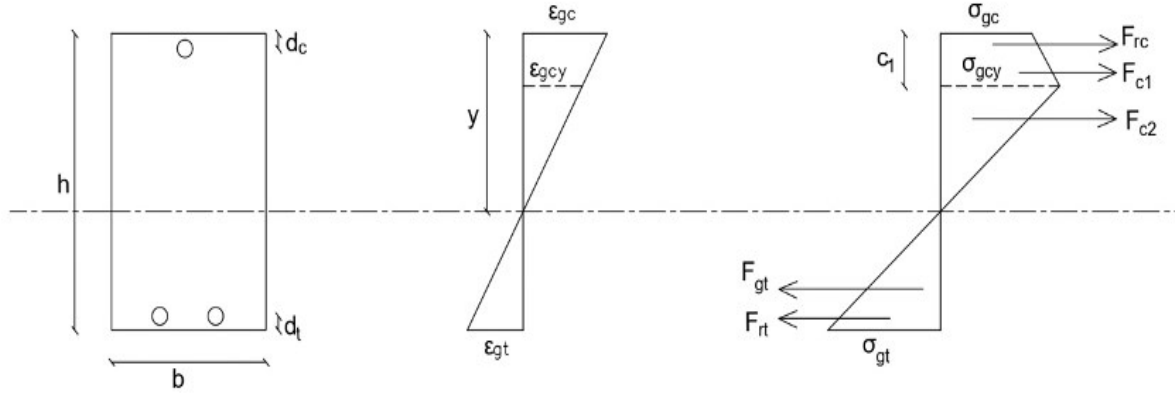


Figure 4.13 Cross – Section Equilibrium Analysis of GFRP Reinforced GLT Beam

From the condition of internal force equilibrium, the governing equations for sectional analysis were derived, as presented in Equations (4.1–4.5).

$$F_{c1} = \left(1 - \frac{\varepsilon_{gcy}}{\varepsilon_{gc}} \right) \left(\frac{1}{m} - \frac{\varepsilon_{gc}}{2 \varepsilon_{gcy}} + \frac{1}{2} \right) m \sigma_{gc} y b \quad 4.1$$

$$F_{c2} = 0.5 E_{gc} (y - c_1) b \quad 4.2$$

$$F_{rc} = \frac{E_{rc}}{E_{gc}} E_{gc} \varepsilon_{gc} A_{rc} \quad 4.3$$

$$F_t = 0.5 E_{gt} (h - y) b \quad 4.4$$

$$F_{rt} = \frac{E_{rt}}{E_{gt}} E_{gt} \varepsilon_{gt} A_{rt} \quad 4.5$$

In the section analysis of the reinforced GLT beam, the total depth and width of the section are denoted by h and b , respectively. The parameter y refers to the distance from the extreme compression fibre to the neutral axis, while c_1 represents the depth of the plastic zone within the compression zone. The distances from the centroids of the reinforcement in tension and compression to their respective extreme fibres are given as d_t and d_c . The material properties involved in the analysis include the modulus of elasticity of GLT in compression and tension,

denoted by E_{gc} and E_{gt} , respectively and the modulus of elasticity of the reinforcement under compression and tension, denoted by E_{rc} and E_{rt} . The associated strain parameters are the ultimate compressive strain of GLT ε_{gc} , the yield compressive strain ε_{gcy} , and the ultimate tensile strain ε_{gt} . The corresponding stress parameters are the ultimate compressive stress, σ_{gc} , the yield compressive stress, σ_{gcy} , and the ultimate tensile stress, σ_{gt} . The cross-sectional areas of the reinforcement in compression and tension are represented by A_{rc} and A_{rt} , respectively. Internal force components include the compressive force in the plastic region of the GLT section F_{c1} , the compressive force in the elastic region F_{c2} , the tensile force in the GLT, F_t and the internal forces in the reinforcement under compression and tension, denoted by F_{rc} and F_{rt} respectively. The position of the neutral axis is determined by applying axial force equilibrium across the cross section, expressed as in equation 4.6,

$$0.5 E_{gc} \varepsilon_{gc} y b + \frac{E_{rc}}{E_{gc}} E_{gc} \varepsilon_{gc} A_{rc} = 0.5 E_{gt} \varepsilon_{gt} (h - y) b + \frac{E_{rt}}{E_{gt}} E_{gt} \varepsilon_{gt} A_{rt} \quad 4.6$$

Strain compatibility between the plastic and elastic compression zones was used to determine the plastification depth c_1 , as expressed in Equation (4.7)

$$\frac{y - c_1}{\varepsilon_{gcy}} = \frac{y}{\varepsilon_{gc}} \quad 4.7$$

With the neutral axis and plastification depth established, the ultimate moment capacity M_u was calculated by summing the internal force contributions about the neutral axis, as shown in Equation (4.8).

$$M_u = F_{c1} \left((y - c_1) + \left(\frac{(\sigma_{gcy} + 2 \sigma_{gc}) c_1}{3 (\sigma_{gcy} + 2 \sigma_{gc})} \right) \right) + F_{c2} \left(\frac{2}{3} (y - c_1) \right) + F_{rc} (y - d_c) + F_t \left(\frac{2}{3} (h - y) \right) + F_{rt} (h - y - d_t) \quad 4.8$$

In this study, the theoretical sectional analysis was conducted following the approach established by Yeboah et al (2021) [18], with several reinforcement configurations examined as shown in Figure 4.14 (a-d). Among these, the arrangement with two reinforcement bars in tension and one in compression (Figure 4.14 d) was identified as the most effective, providing superior ultimate strength and stiffness. This outcome is consistent with previous analytical investigations, which

reported similar asymmetrical layouts to enhance tensile performance while efficiently utilizing minimal reinforcement in compression. Parallel studies by other authors [19][20] Have likewise confirmed the effectiveness of this configuration, noting that it strengthens timber where it is weakest in tension and improves overall stiffness, while relying on the inherent compressive capacity and ductility of wood.

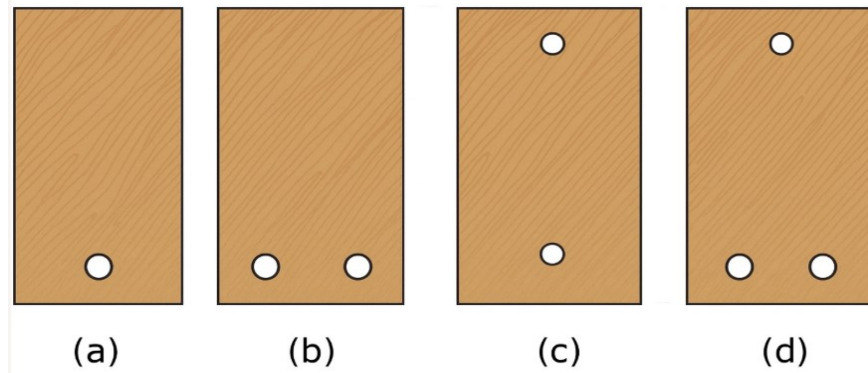


Figure 4.14 Different Reinforcement Configurations

(a) Reinforcement on Tension Zone (b) Two Bars on Tension Zone (c) Symmetrical Reinforcement on Tension and Compression Zone (d) Two Bars on Tension Zone and One Bar on Compression Zone

Table 4.5 Comparison of FEM Vs Theoretical Results

Configuration (Steel on Tension zone)	Reinforcement Ratio	$M_{theoretical}$ (kNm)	M_{FEM} (kNm)	$M_{FEM} /$ $M_{theoretical}$	Initial Stiffness (kN/m)
GFRP in the compression zone	1%	120.69	119.23	0.99	7.64
	2%	156.87	141.87	0.90	6.69
	3%	181.05	167.77	0.93	6.99
BFRP in the compression zone	1%	120.50	118.86	0.99	7.61
	2%	156.74	141.41	0.90	6.67
	3%	180.65	166.56	0.92	6.94
CFRP in the compression zone	1%	124.66	127.78	1.03	8.19
	2%	163.34	172.97	1.06	7.12
	3%	188.82	172.8	1.09	9.84

The comparison between theoretical and FE modelling results shows that beams reinforced with GFRP and BFRP exhibit close agreement at lower reinforcement ratios ($M_n/M_{th} = 0.99$) but show slight reductions at higher ratios due to nonlinear compression effects and limited bond efficiency. In comparison, CFRP reinforced beams demonstrate higher performance, with FEM moments exceeding theoretical predictions at 1% and 2% reinforcement, confirming the positive influence of their higher stiffness and compressive strength. However, at 3%, the capacity drops notably ($M_n/M_{th} = 1.09$), indicating possible over-reinforcement. The initial stiffness results follow a similar trend, with CFRP showing the highest values, followed by GFRP and BFRP, emphasizing that the FRP's modulus of elasticity primarily governs stiffness enhancement. In comprehension, CFRP provides the most significant improvement in flexural capacity and stiffness, while GFRP and BFRP offer stable performance at moderate reinforcement ratios.

4.4. Conclusion

This study investigates the structural performance of GLT beams with various reinforcement configurations and materials. FEM was developed and validated against experimental baseline data obtained from bending tests on unreinforced GLT beams. Following validation, the FEM was extended to incorporate hybrid reinforcement systems using CFRP, BFRP and GFRP, enabling a comprehensive evaluation of their effectiveness under flexural loading.

The main conclusions from these investigations are summarized below.

- Parametric assessment was performed to determine the optimum spacing between two reinforcement bars at tension zone of the GLT beam. The analysis revealed negligible differences (approximately 1%) in ultimate capacities across various spacing configurations (S1–S7). Among these, configuration (S3) with 40 mm bar spacing exhibited the highest capacity of 247.7 kN, indicating it as the optimal arrangement.
- For spacings below 40 mm, the load-carrying capacity decreased slightly due to overlapping stress zones between adjacent bars, which reduced reinforcement efficiency. Conversely, spacings greater than 40 mm resulted in a gradual decline in capacity (approximately 0.5–1%) as stress transfer between timber and reinforcement became less effective.

- The FE model demonstrates that using CFRP in the compression zone and steel in the tension zone yields the highest load capacity and stiffness, achieving 432.44 kN and 9.84 kN/mm at a 3% reinforcement ratio. This hybrid configuration enhances the GLT beam's strength by 116% and stiffness by 27% compared to the unreinforced beam. Among all reinforcement types, CFRP provides the most significant improvement in flexural capacity and stiffness, while GFRP and BFRP deliver stable performance at moderate reinforcement ratios, making them suitable for cost-effective applications.

Data Accessibility

All data generated or analyzed in this study, including the experimental measurements and FEM simulation files, are available from the corresponding author upon reasonable request.

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Conflicts of Interest:

The authors declare that they have no conflicting interests concerning the publication of this article.

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Chapter 5

5.1. Conclusions

This thesis examined the flexural performance of Glue-Laminated Timber (GLT) beams through experimental testing, finite element (FE) modelling, and parametric analysis. Initially, it studied the monotonic and cyclic behaviour of unreinforced beams, determining how aspect ratio and span affect strength, stiffness, displacement, and energy dissipation. The research was then extended to reinforced beams with two steel rebars in the tension zone, spaced 40 mm apart. The compression zone was reinforced with various fibre-reinforced polymers (FRPs). Comparative analyses were conducted using steel + GFRP, steel + CFRP, and steel + BFRP configurations. The reinforcement ratio ranged from 1% to 3% by adjusting bar diameter, while spacing varied from 15 mm to 120 mm. These studies demonstrate that reinforcement significantly improves the mechanical performance of GLT beams.

The following conclusions are drawn from this thesis.

- Tensile failure was identified as the main factor affecting the bending performance of unreinforced beams under monotonic and cyclic conditions loadings.
- Cyclic loading initiated microdamage sooner than monotonic loading, leading to a decrease in energy absorption and ultimate capacity. Interestingly, it also produced a slight increase in initial stiffness of 2.5%, attributed to the stiffening effect of microcracks.
- The validated FE models successfully aligned with the experimental peak forces, exhibiting an error margin of just 5% suggesting that the models can be trusted to reliably predict force-displacement trends of GLT beams.
- Parametric analysis in finite element (FE) studies indicated that the beams with smaller aspect ratios, specifically b/d of 0.37, and shorter spans of 3 meters, yield the highest strength and stiffness.
- When the bars were spaced 40 mm apart, the beam reached its maximum capacity of 247.7 kN. However, the parametric study indicated that decreasing the distance between reinforcement capacity stress fields overlap, reducing efficiency and ultimately the capacity.

- Notable improvements in bending capacities were observed when the beams used CFRP as the compression material and steel for tension in hybrid reinforcement. Such that, considering 3% reinforcement ratio, the load capacity reached 432.44 kN, and the stiffness was 9.84 kN/mm, representing a 116% increase in strength and a 27% rise in stiffness compared to unreinforced GLT beams tested experimentally. These findings are consistent with earlier research on hybrid models with various materials and configurations [19] [20].
- The hybrid reinforcement setup in various configurations showed significant improvement in the flexural performance of GLT beams. CFRP's high modulus helps sustain the stability of the compression zone. Conversely, Steel's ductility in tension delays brittle failure, resulting in a more balanced stress distribution.
- Increasing the reinforcement ratio from 1% to 3% significantly improved both moment capacity and stiffness. CFRP outperformed GFRP and BFRP, especially at lower ratios, thanks to its superior modulus of elasticity (MOE).

5.2. Limitations and Future Work Recommendations

This study has significantly contributed to understanding the mechanical behaviour of the GLT beam through experimental testing and numerical modelling. The experimental work was limited to unreinforced beams.

The thesis recommends expanding the experimental program to overcome its limitations by including full-scale testing of hybrid GLT beams. This enhancement is essential for accurately verifying numerical predictions under both monotonic and cyclic loading conditions. Additionally, in this study, the finite element model was based on a simplified assumption that ignored local effects, such as adhesive failures, finger joints, and bond slip. Therefore, the thesis suggests that future FE models should include damage criteria to better monitor failure mechanisms and bond-slip behaviour reliability.

Appendix

Appendix A: Experimental and Numerical Investigation of Glulam Beams under Cyclic and Monotonic Flexural Loading

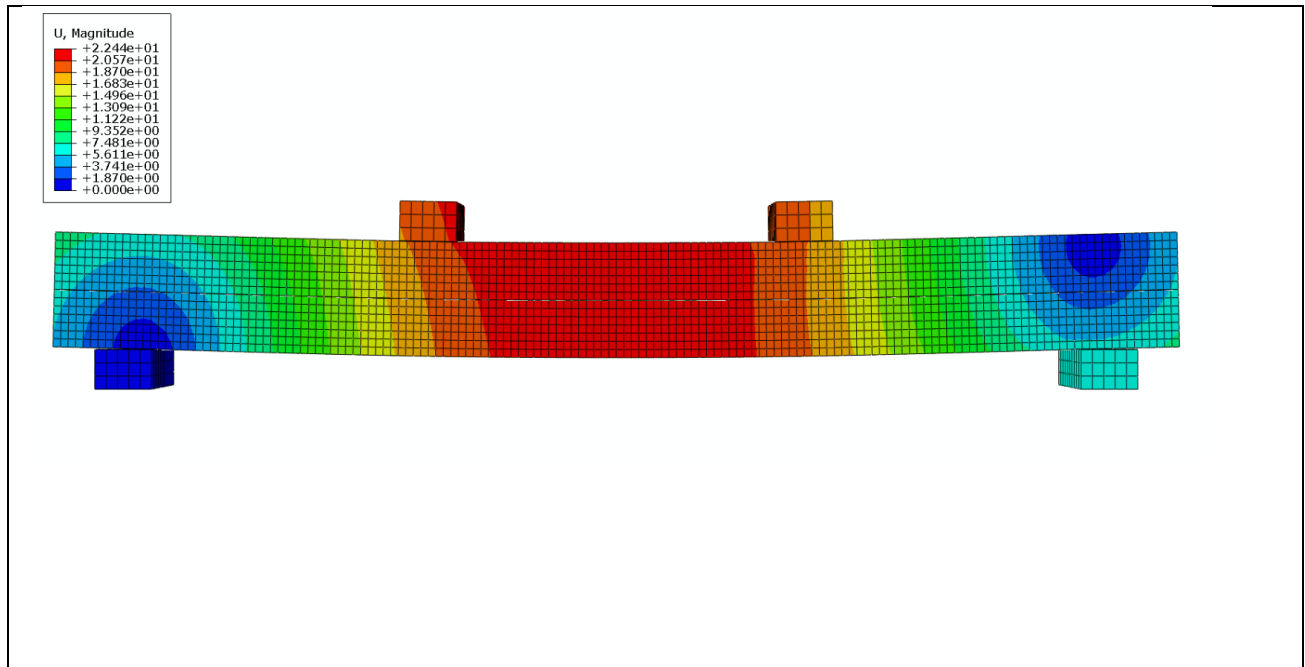


Figure A-1 B(184x292)3000 (Aspect Ratio 0.63)

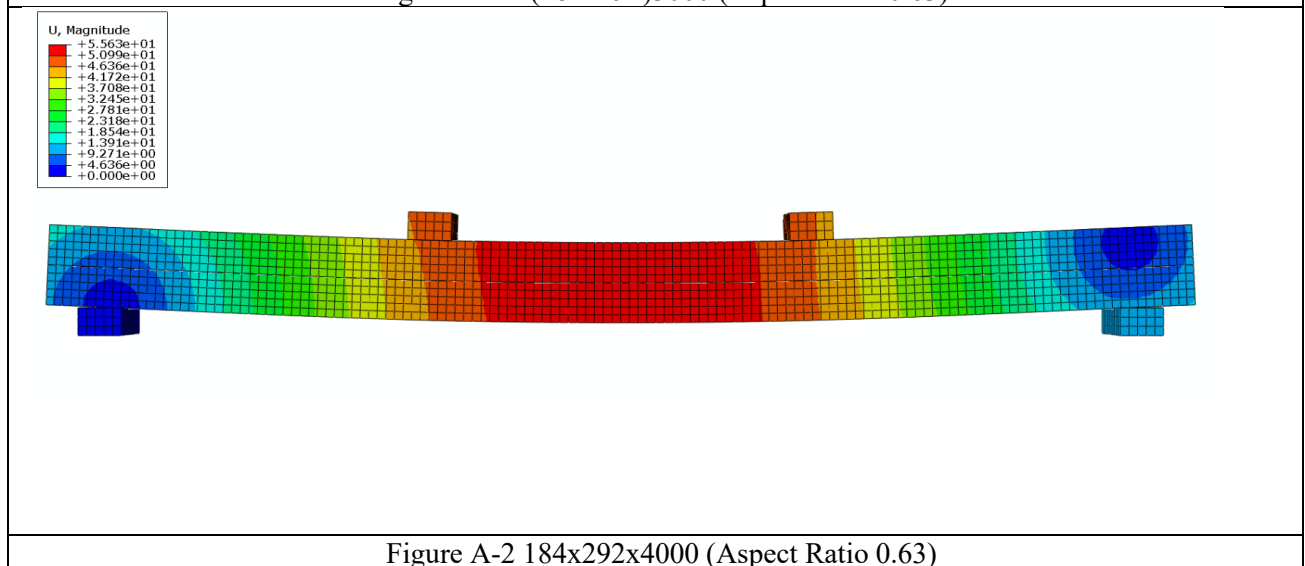


Figure A-2 184x292x4000 (Aspect Ratio 0.63)

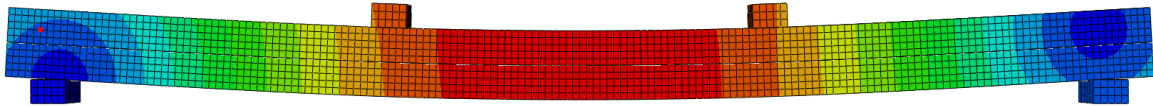
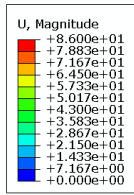


Figure A-3 184x292x5000 (Aspect Ratio 0.63)

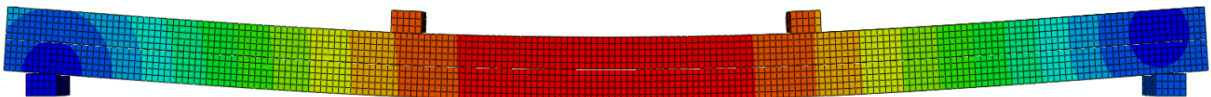
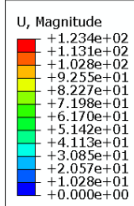


Figure A-4 184x292x6000 (Aspect Ratio 0.63)

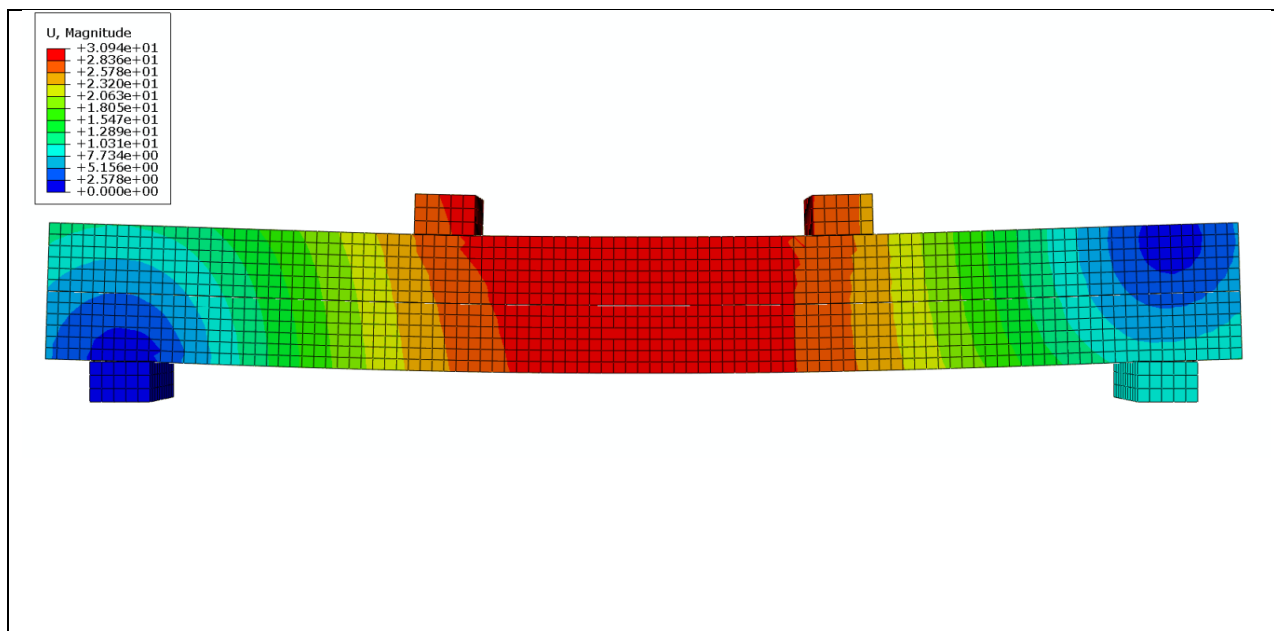


Figure A-5 184x343x3000 (Aspect Ratio 0.53)

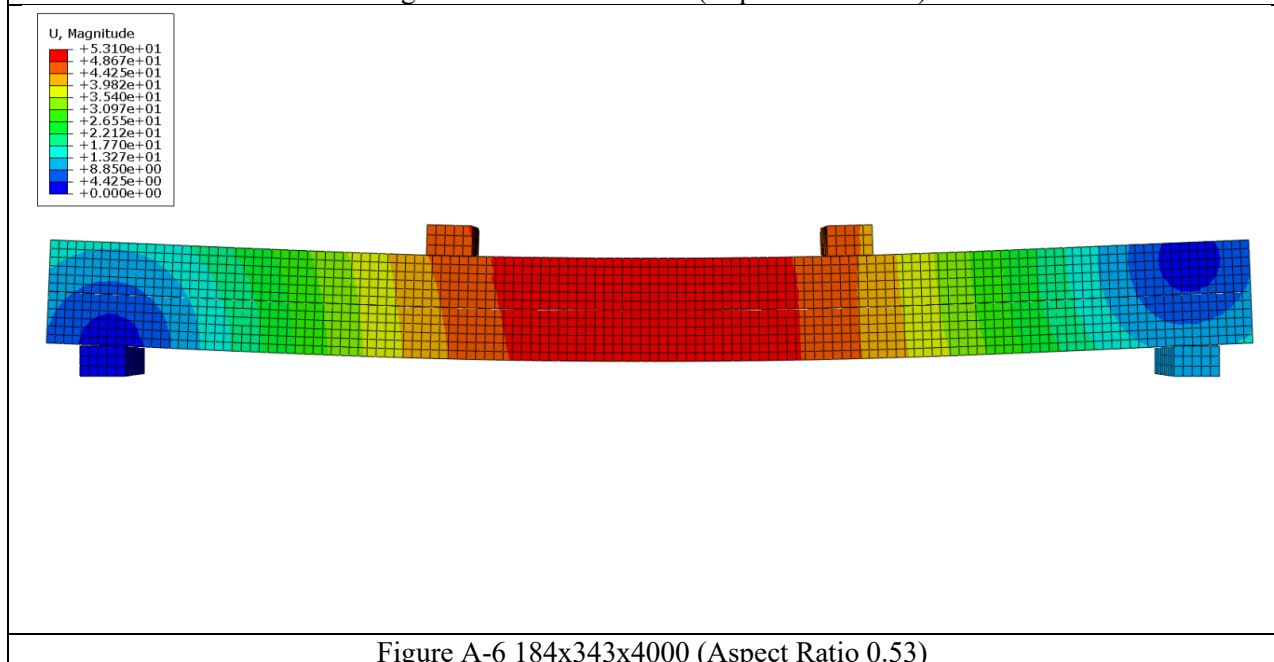


Figure A-6 184x343x4000 (Aspect Ratio 0.53)

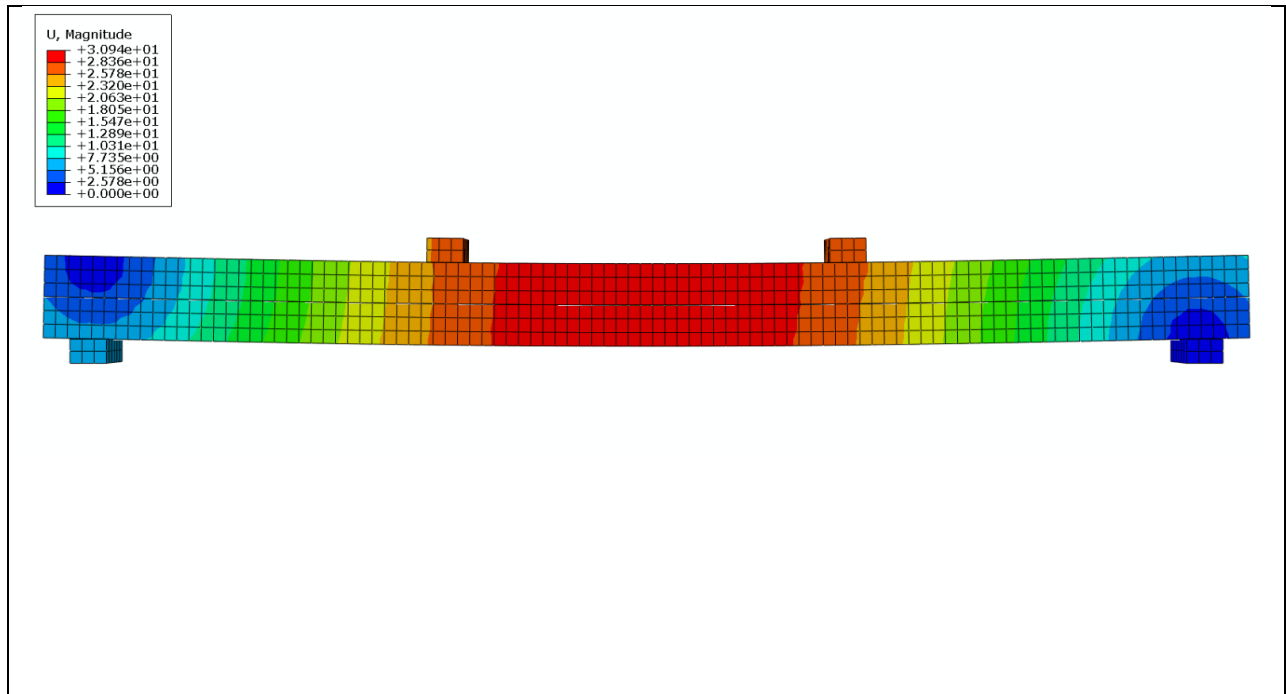


Figure A-7 184x343x5000 (Aspect Ratio 0.53)

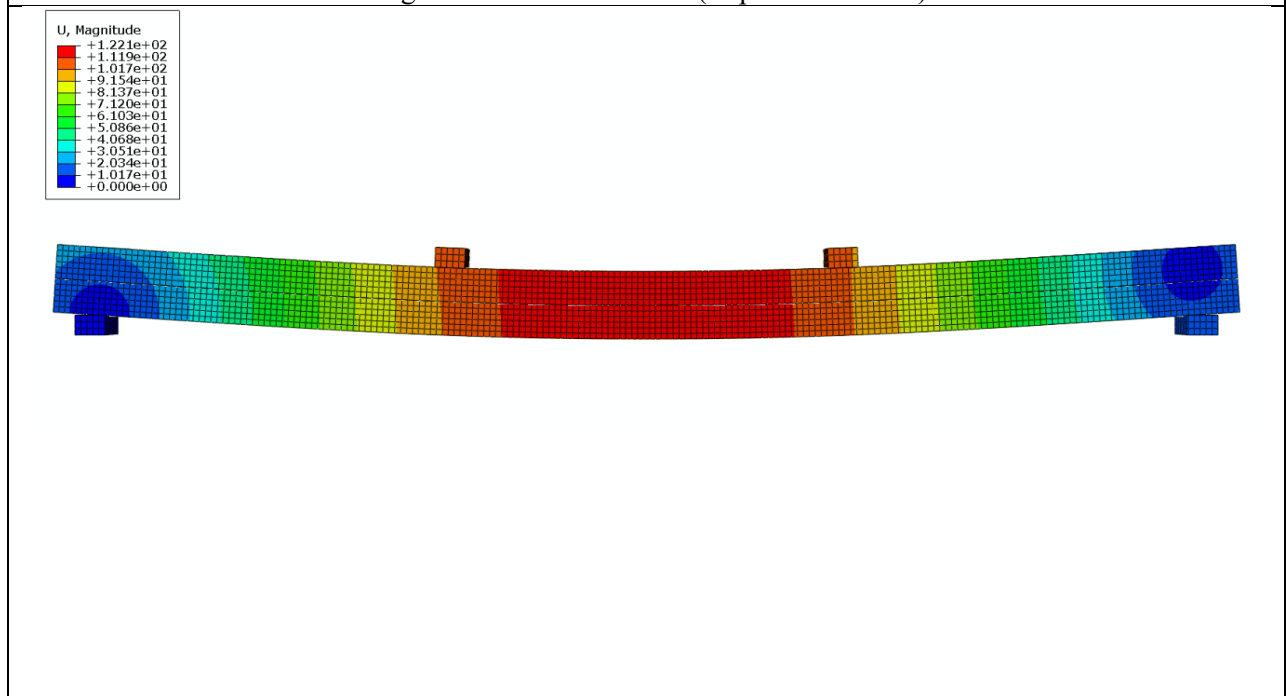


Figure A-8 184x343x6000 (Aspect Ratio 0.53)

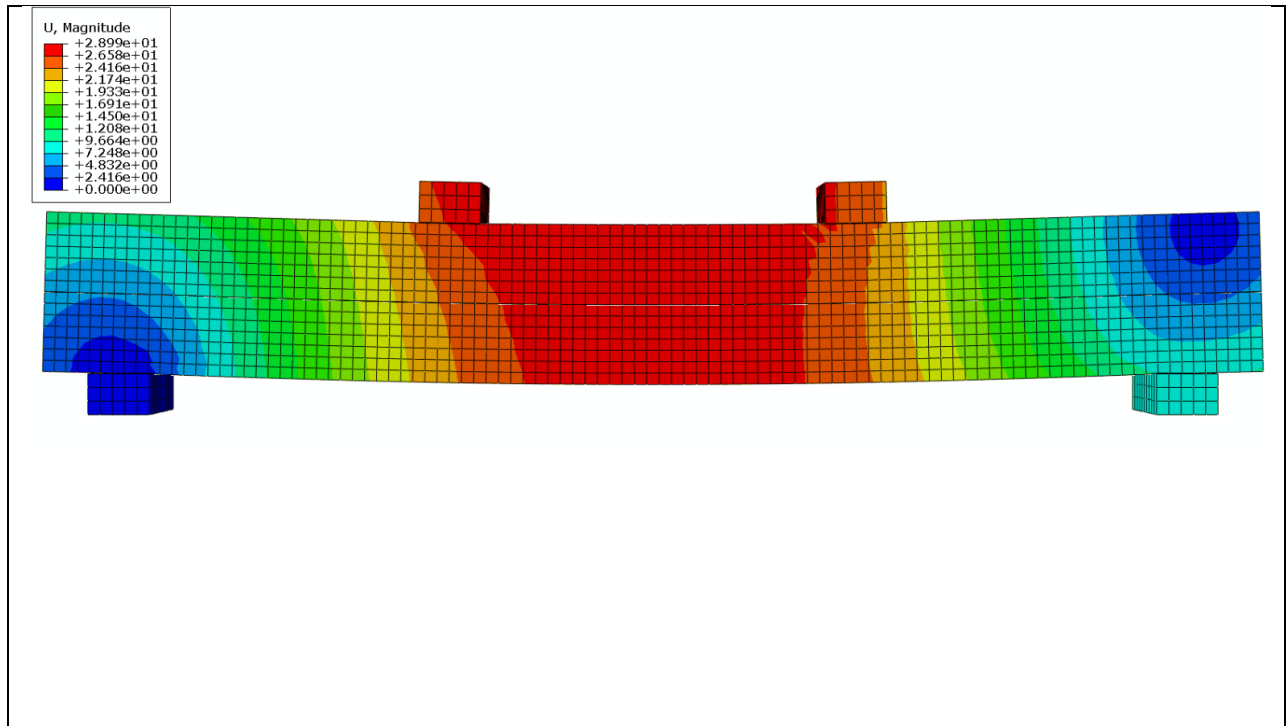


Figure A-9 184x394x3000 (Aspect Ratio 0.46)

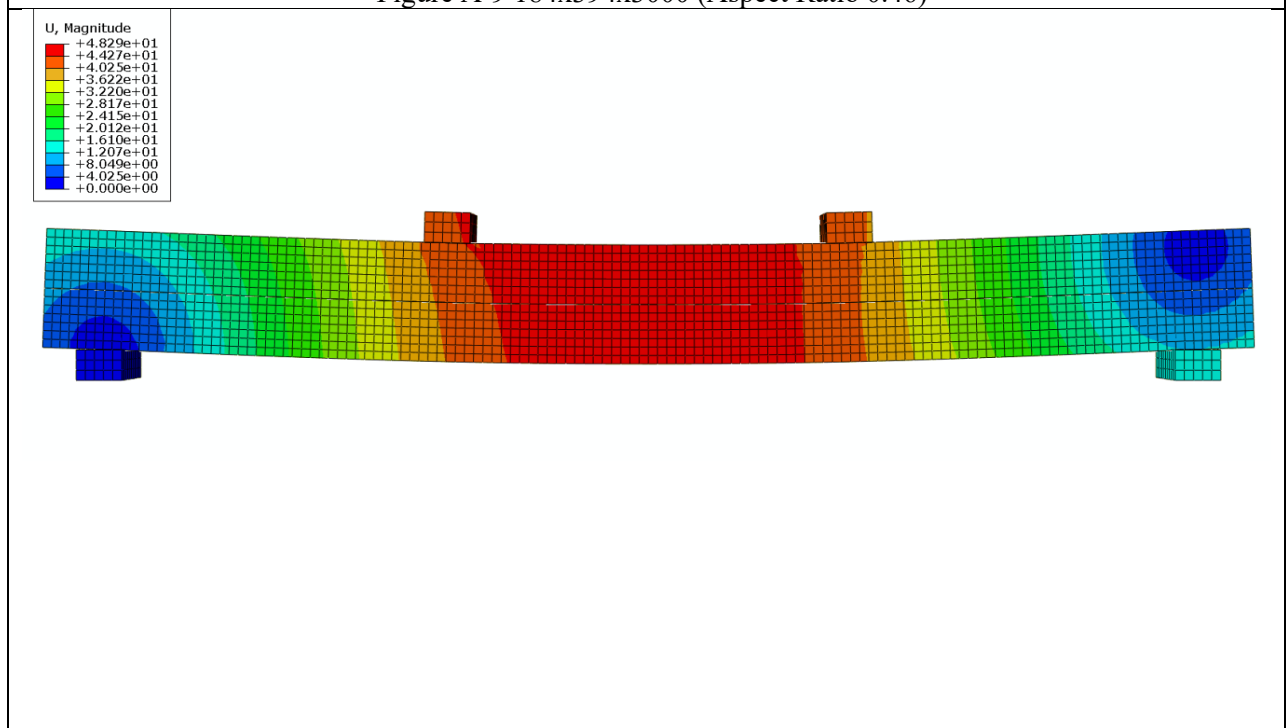


Figure A-10 184x394x4000 (Aspect Ratio 0.46)

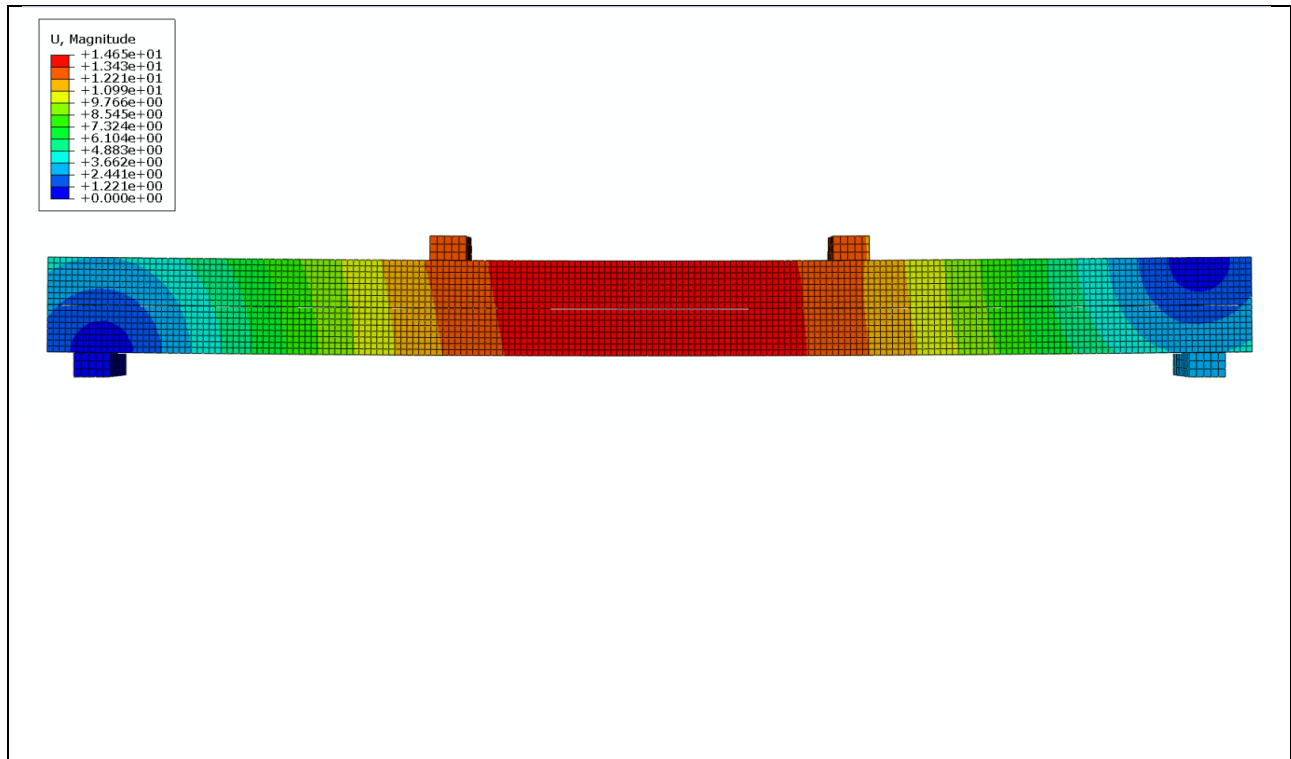


Figure A-11 184x394x5000 (Aspect Ratio 0.46)

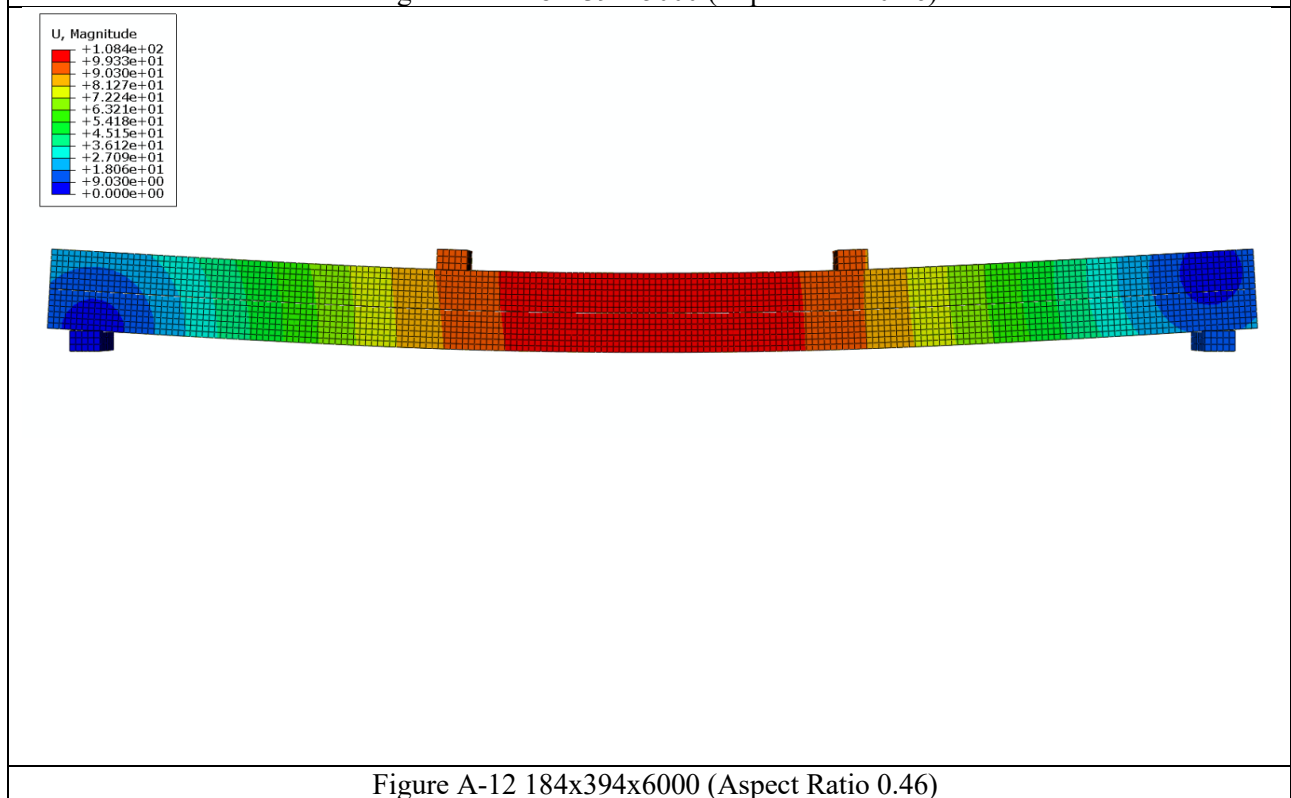


Figure A-12 184x394x6000 (Aspect Ratio 0.46)

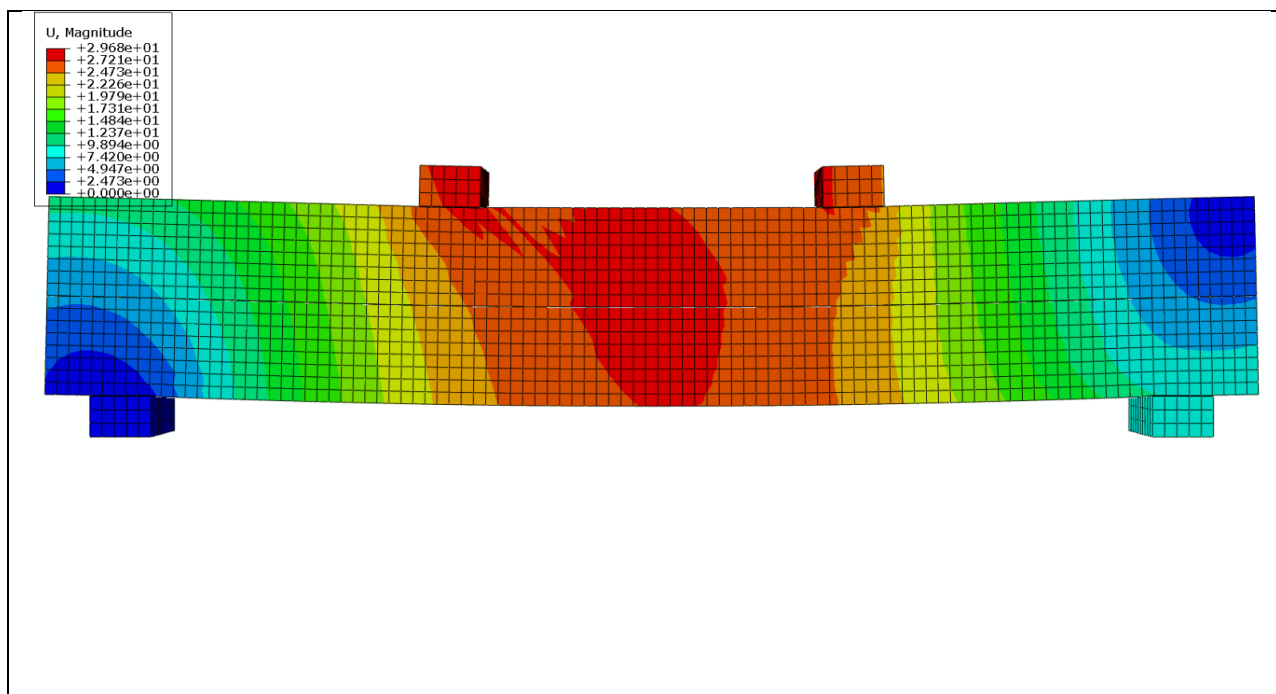


Figure A-13 184x492x3000 (Aspect Ratio 0.37)

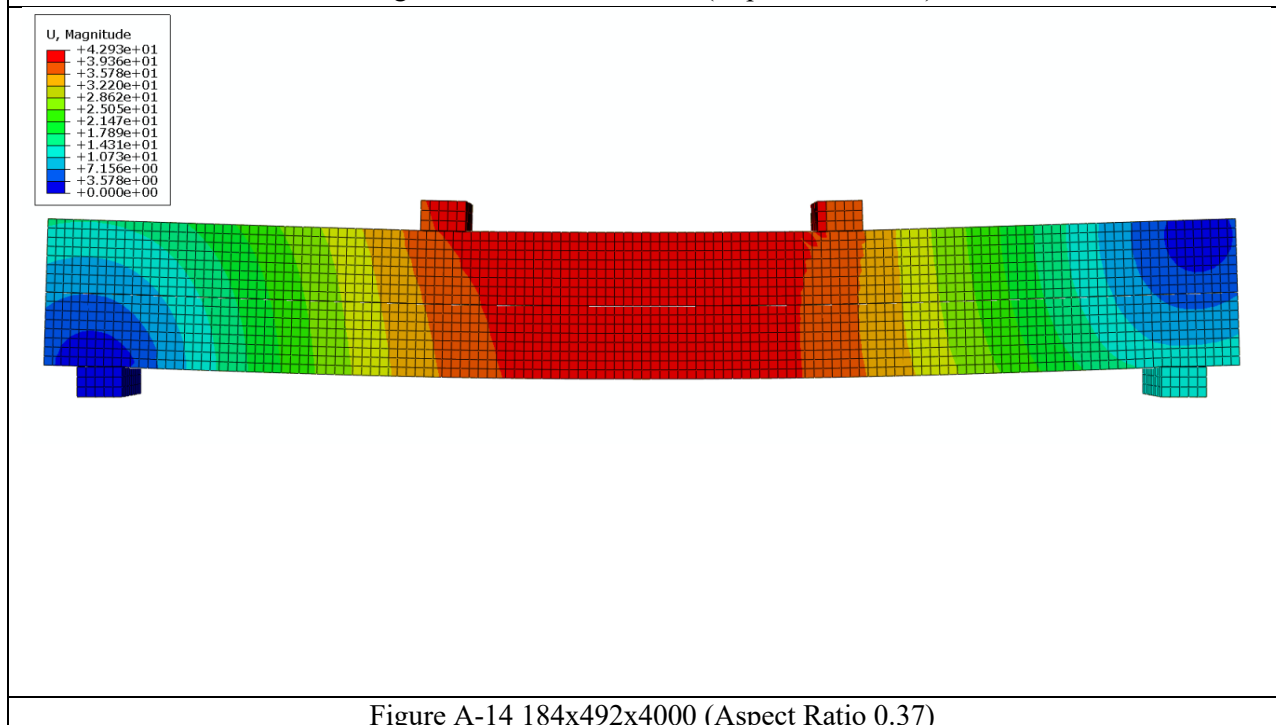


Figure A-14 184x492x4000 (Aspect Ratio 0.37)

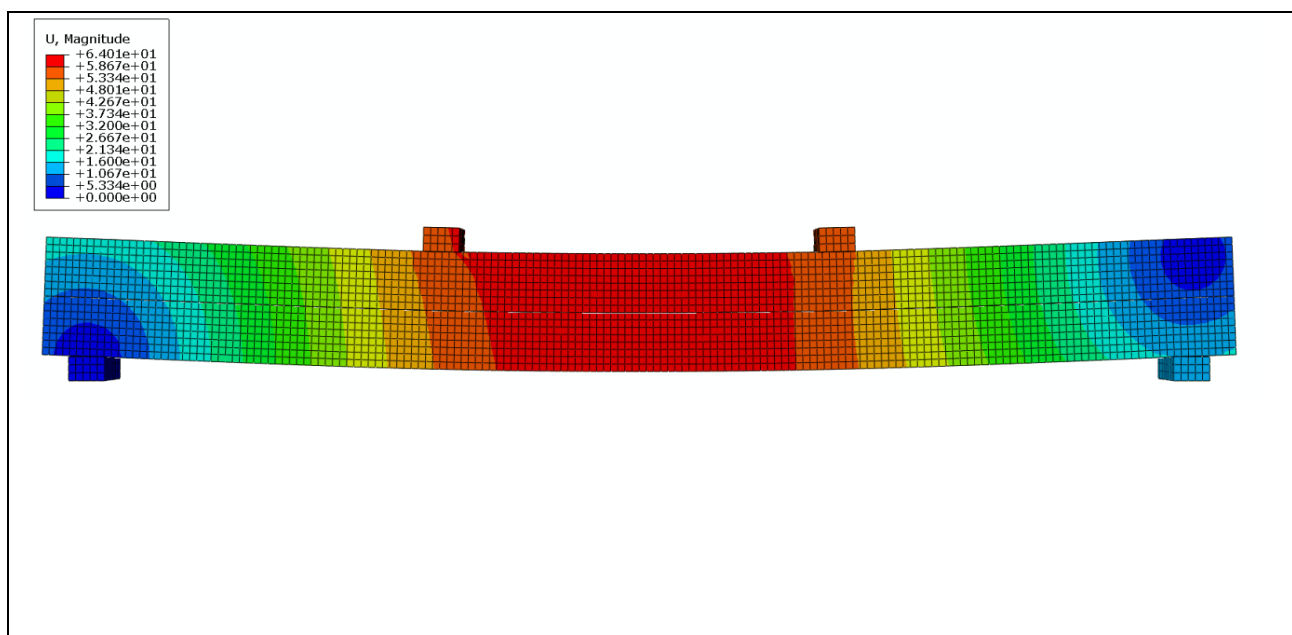


Figure A-15 184x492x5000 (Aspect Ratio 0.37)

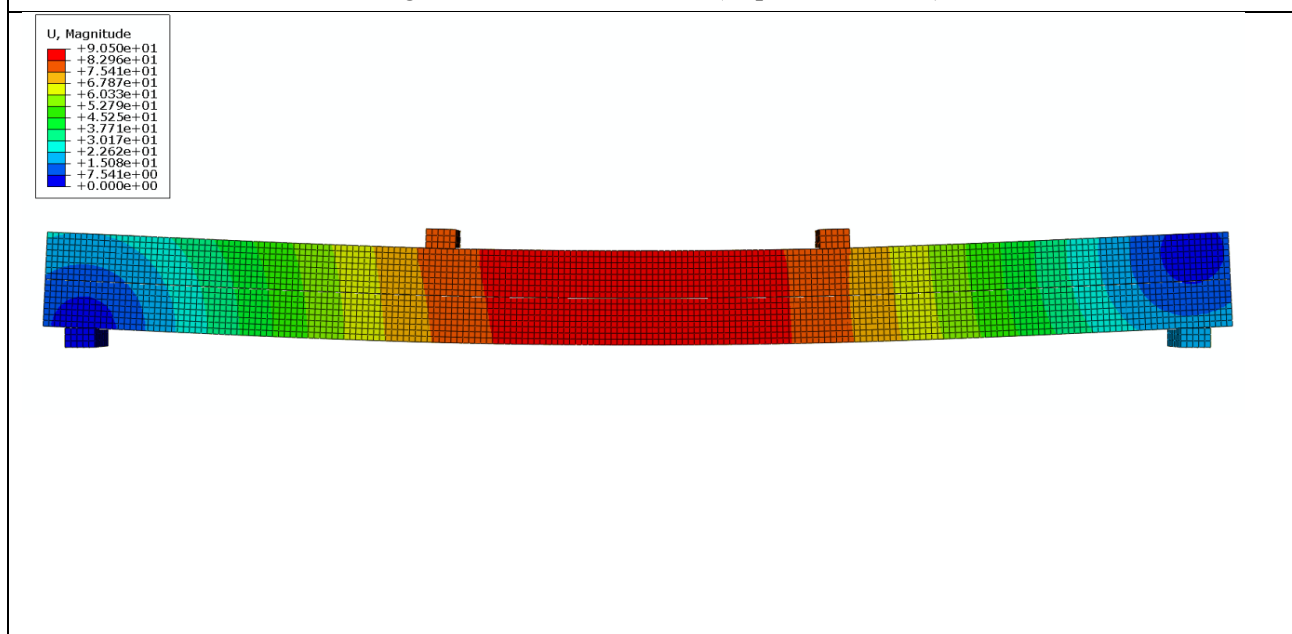


Figure A-16 184x492x6000 (Aspect Ratio 0.37)

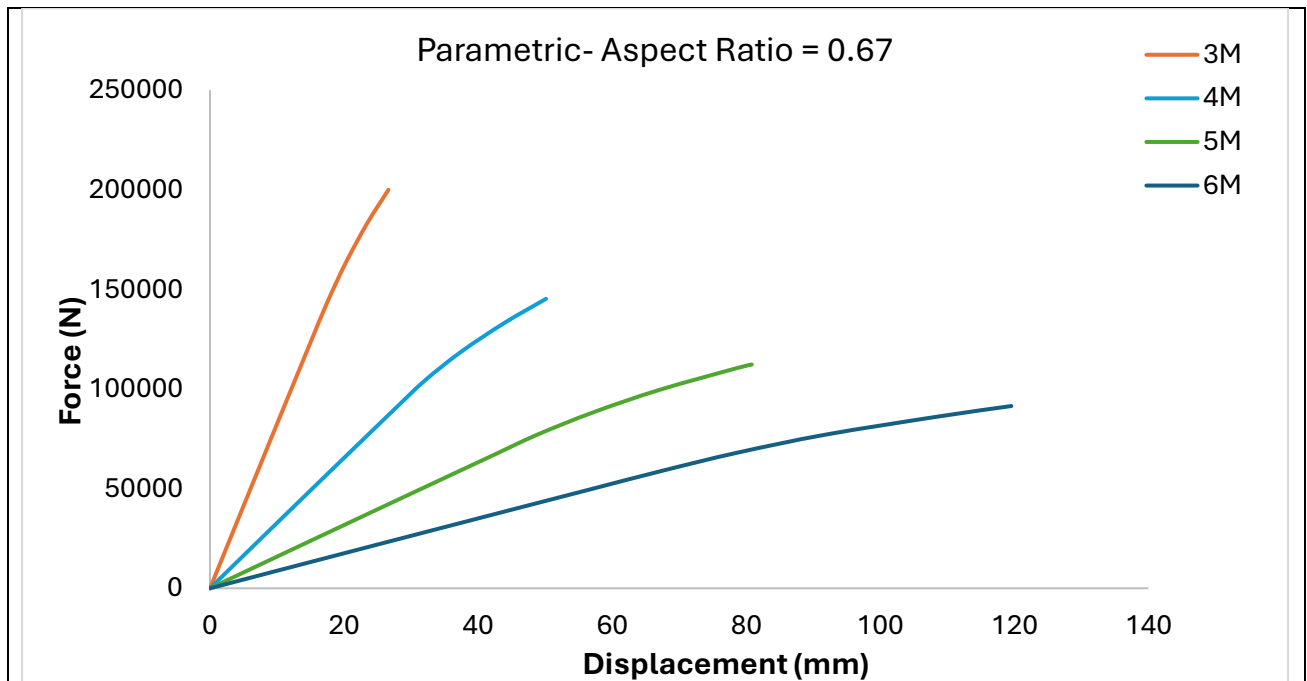


Figure A-17 Force vs Displacement (Aspect Ratio 0.67)

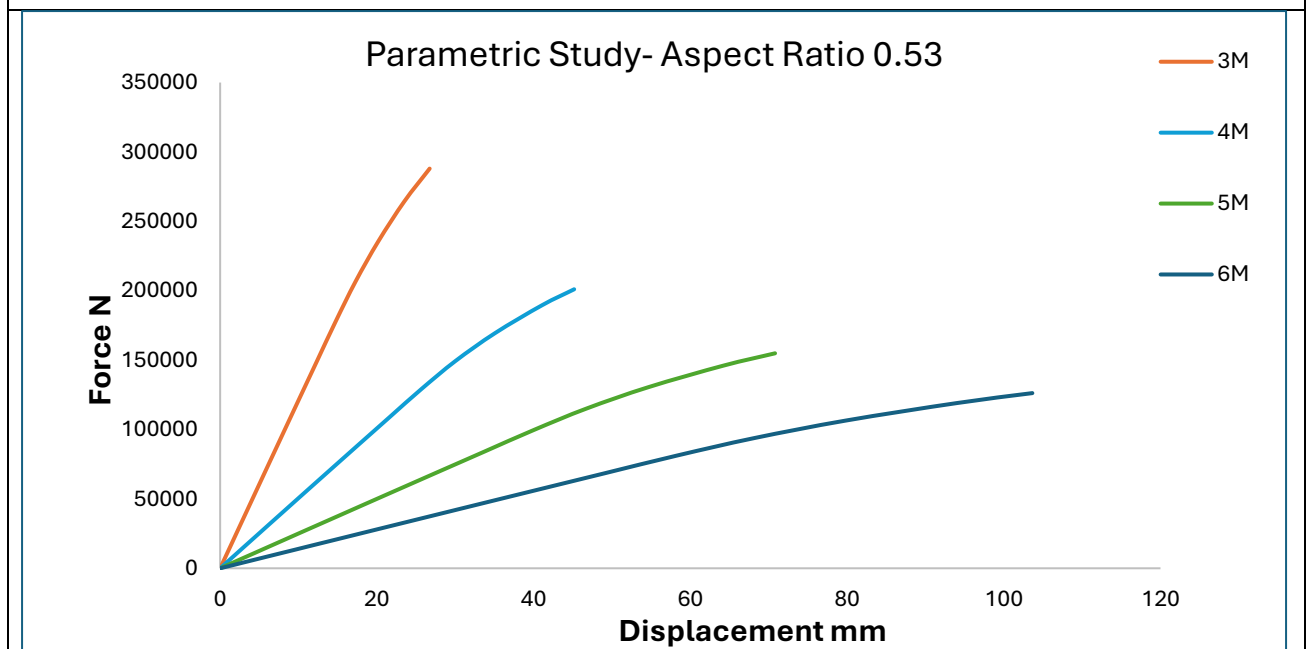


Figure A-18 Force vs Displacement (Aspect Ratio 0.53)

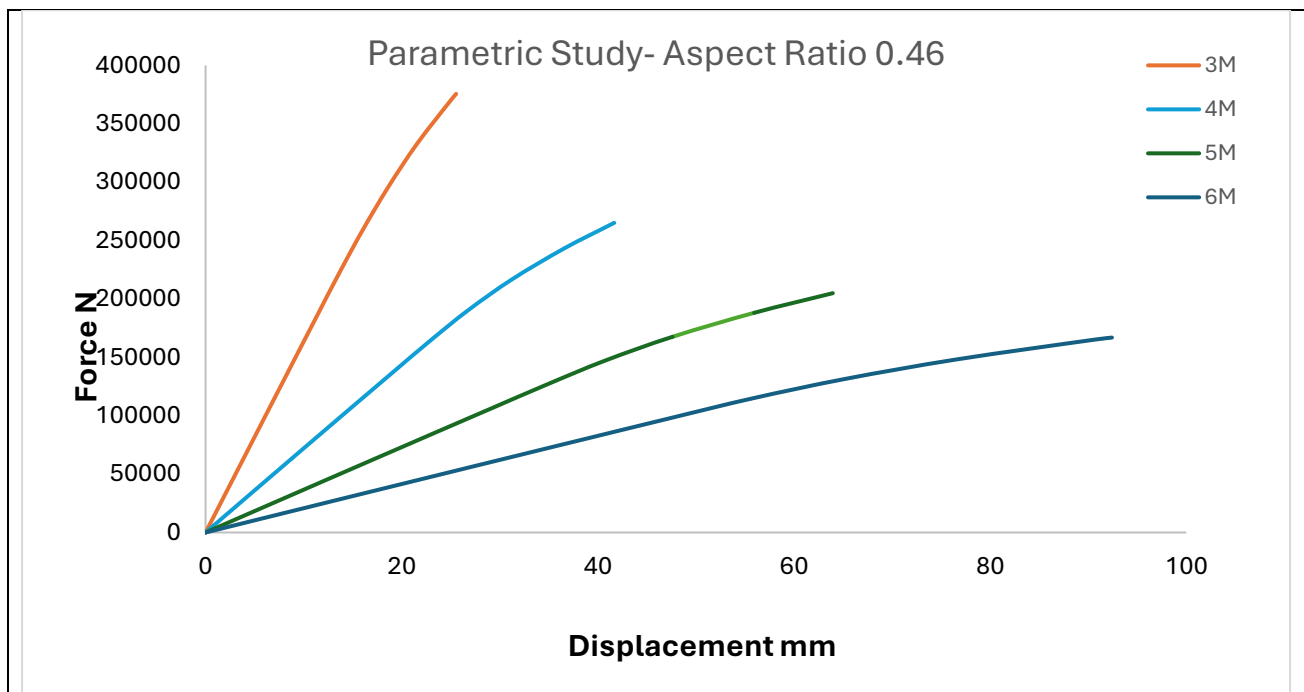


Figure A-19 Force vs Displacement (Aspect Ratio 0.46)

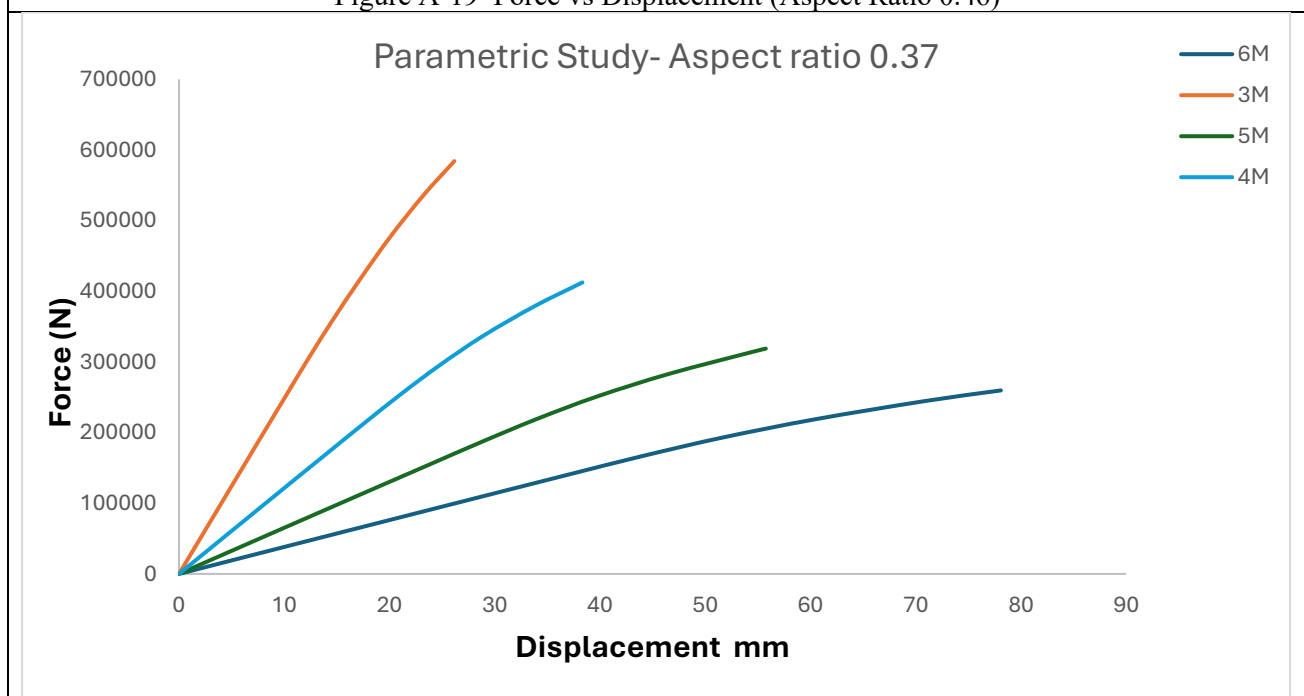


Figure A-20 Force vs Displacement (Aspect Ratio 0.37)

B Appendix B: Flexural Performance of Hybrid Glulam Beams: Influence of Reinforcement Material and Arrangement

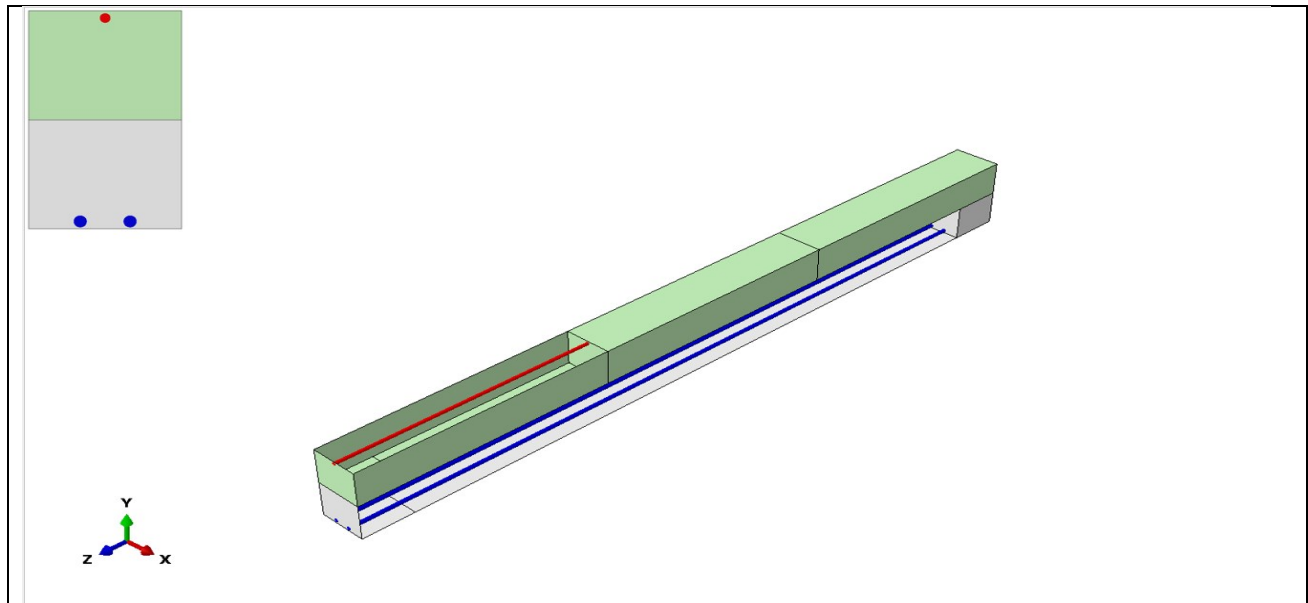


Figure B-1 Assembly BFRP with Steel (Reinforcement Ratio 1%)

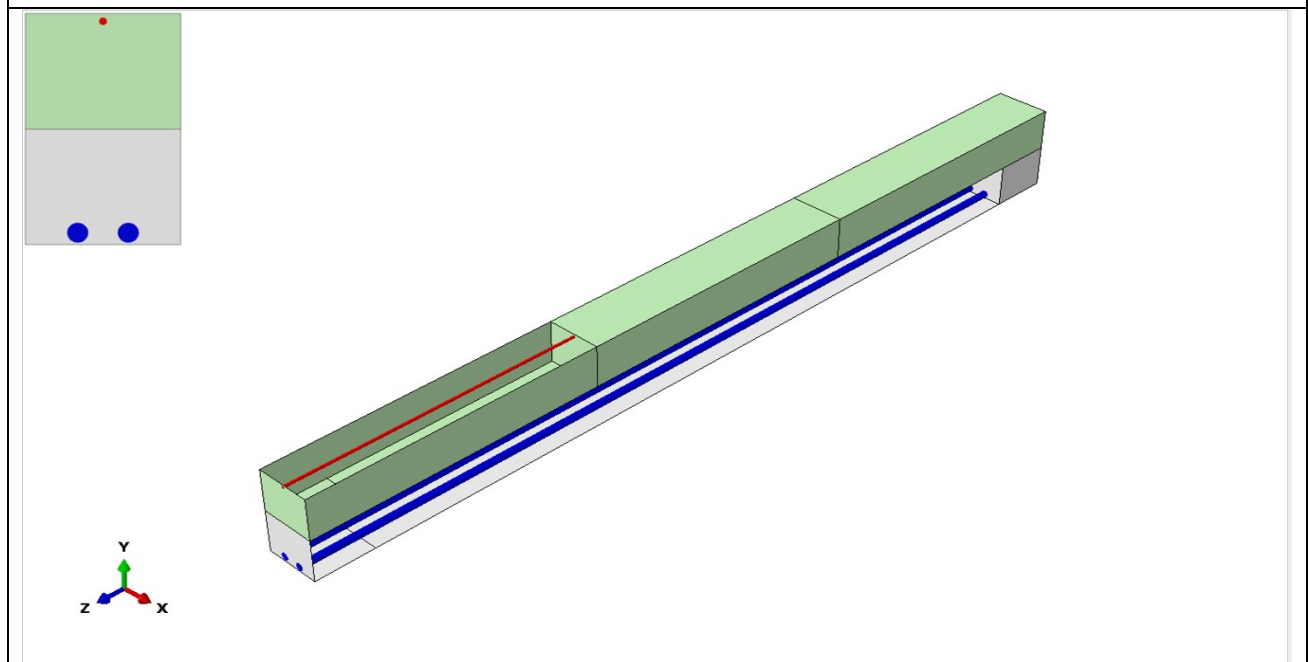
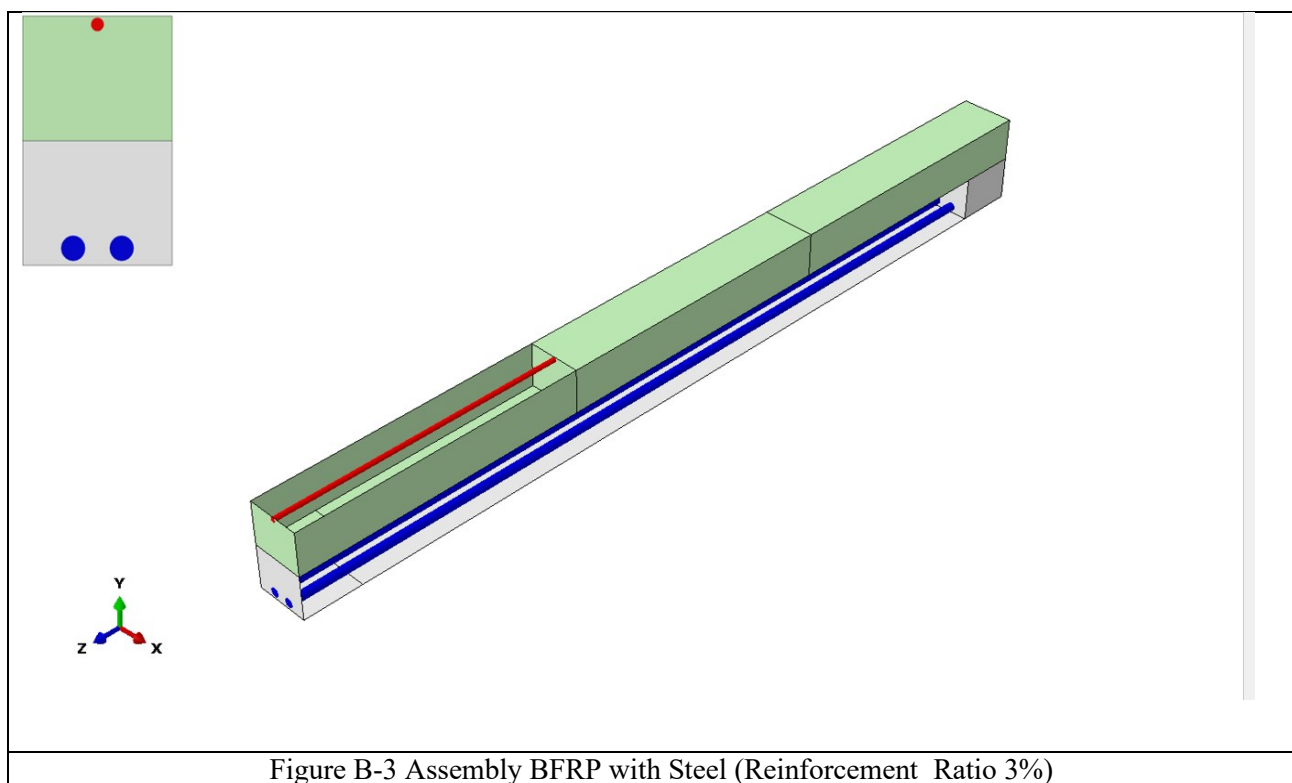


Figure B-2 Assembly BFRP with Steel (Reinforcement Ratio 2%)



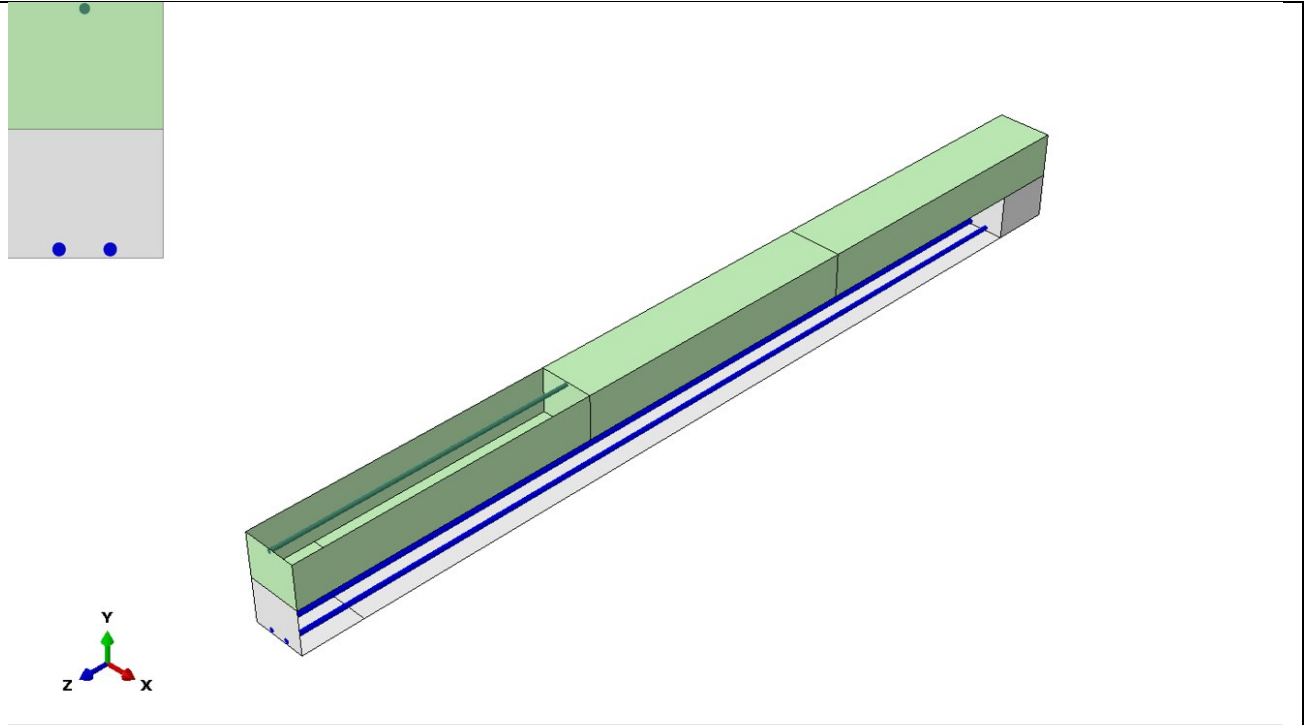


Figure B-4 Assembly CFRP with Steel (Reinforcement Ratio 1%)

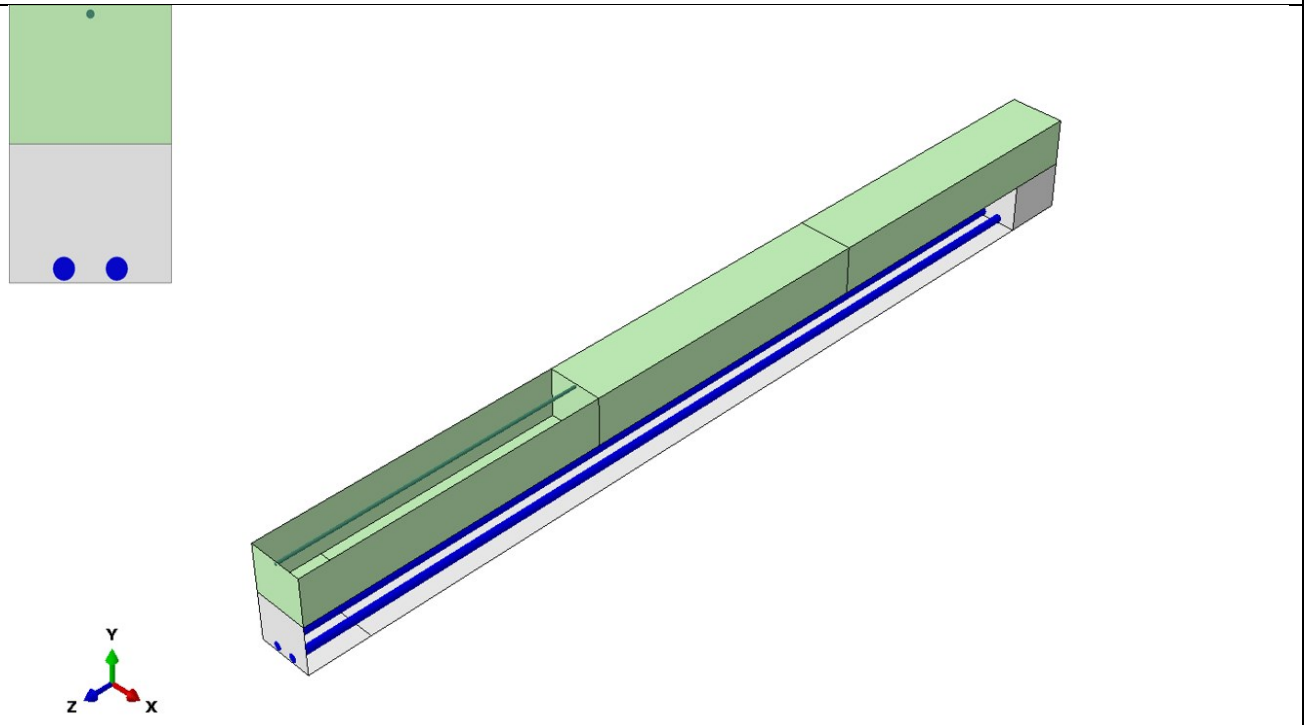
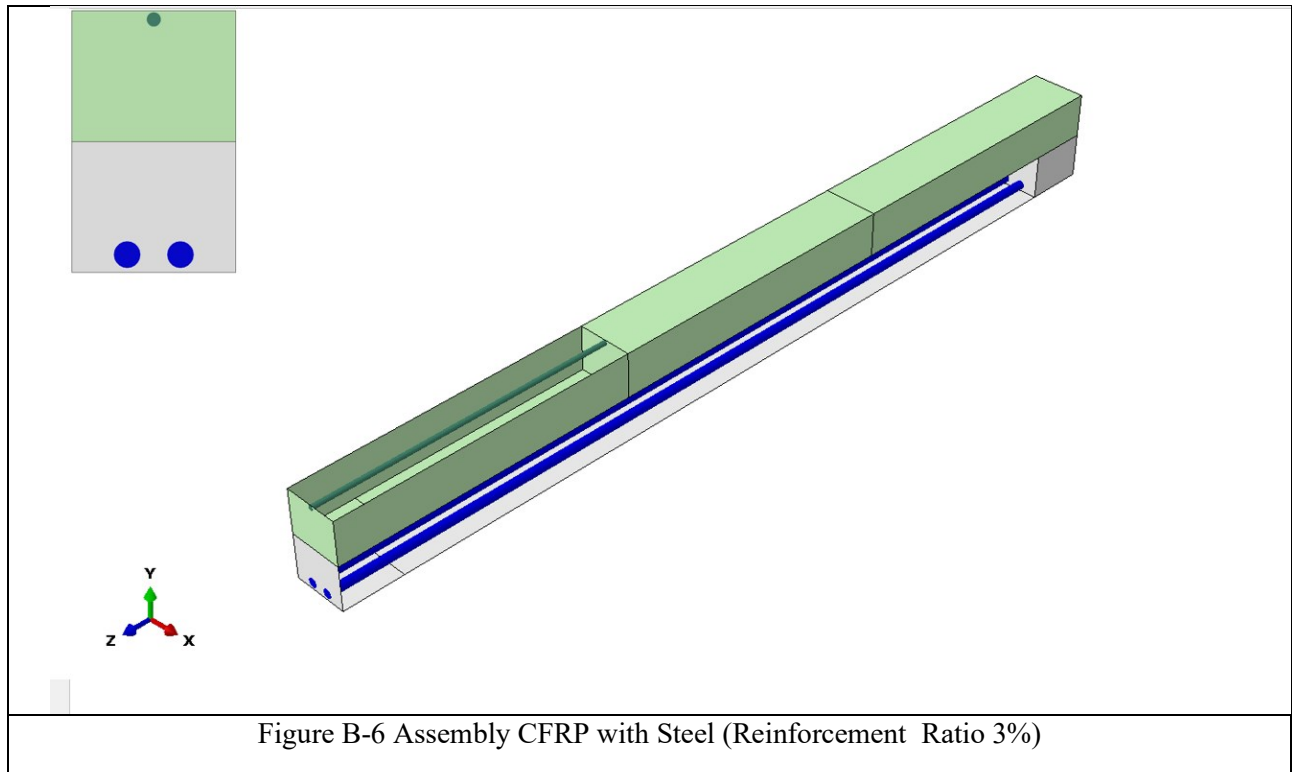


Figure B-5 Assembly CFRP with Steel (Reinforcement Ratio 2%)



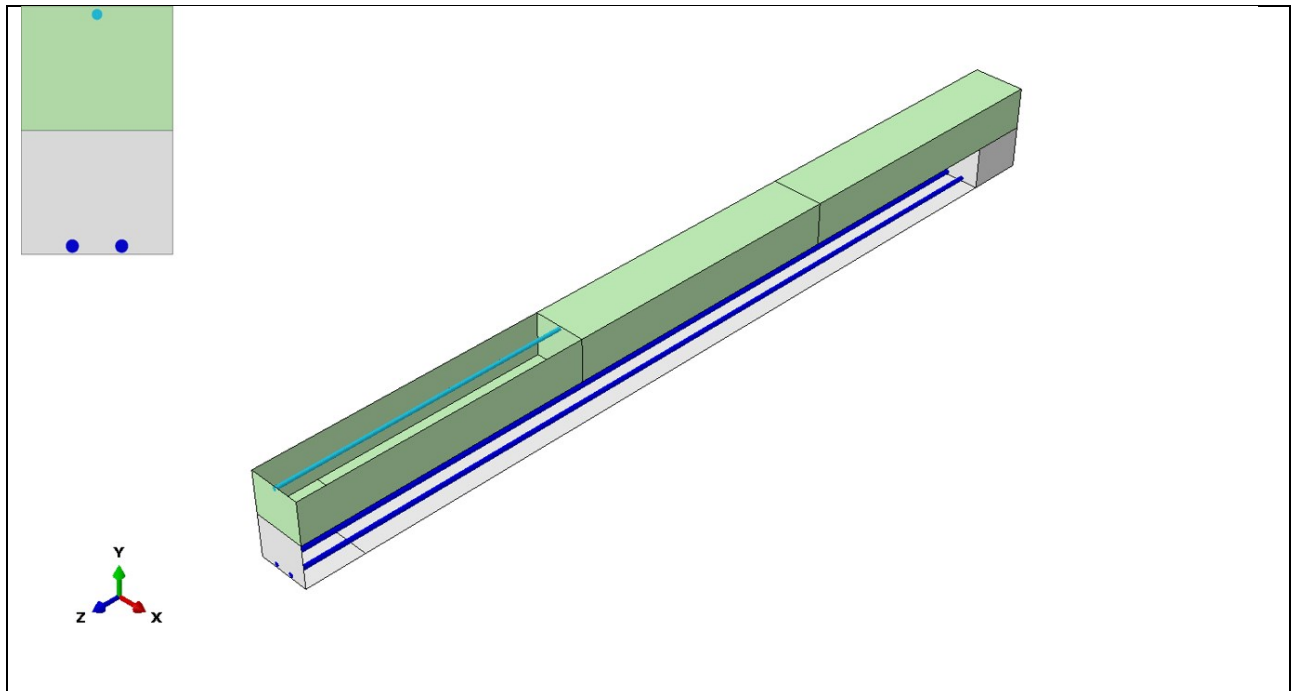


Figure B-7 Assembly GFRP with Steel (Reinforcement Ratio 1%)

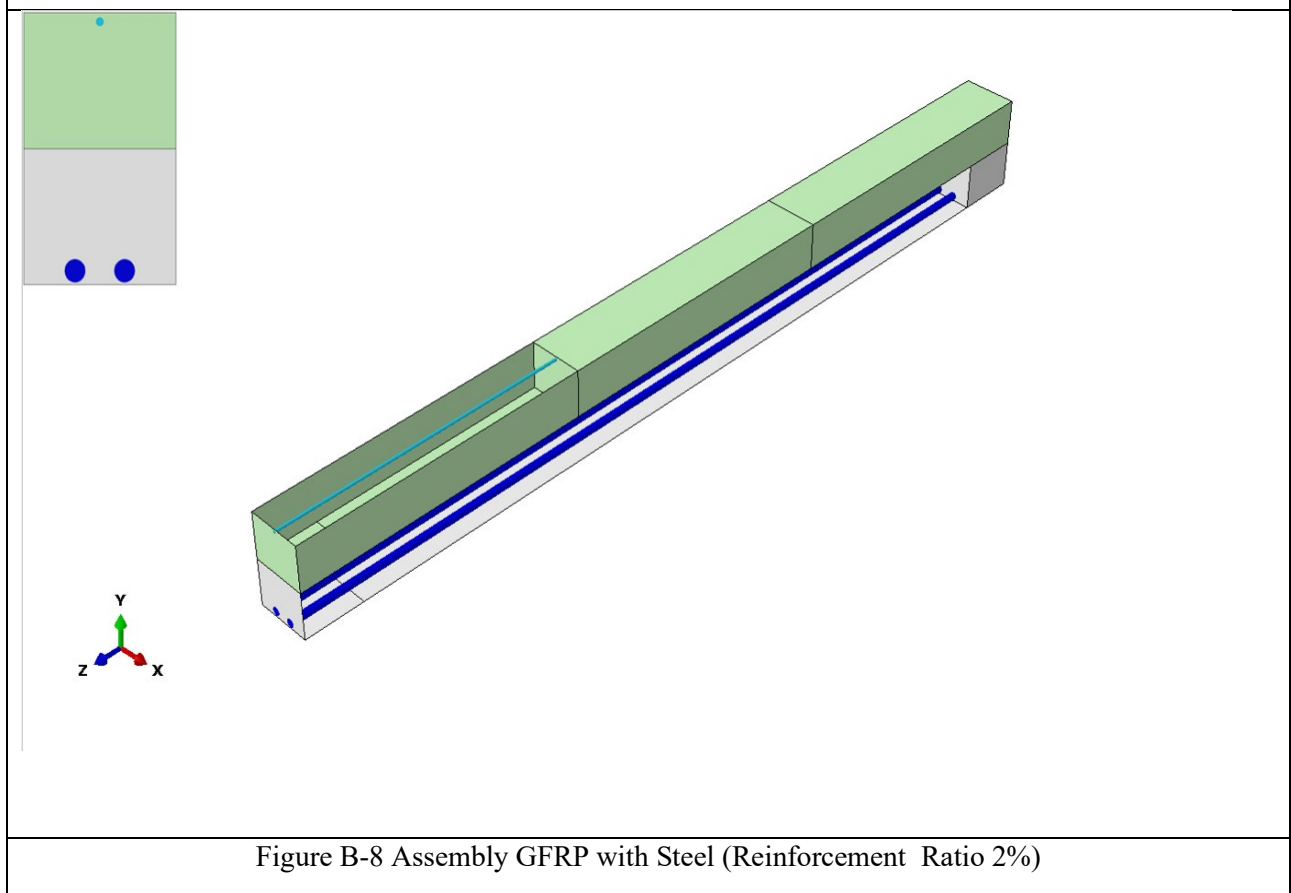


Figure B-8 Assembly GFRP with Steel (Reinforcement Ratio 2%)

