

Investigating the Performance
of Straight and Bent GFRP Bars as
Flexural Reinforcement for
Glulam Beams

By

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

The heightened interest in using wood as a sustainable building material contributed to an increased demand for glued-laminated timber (glulam). Despite this, fundamental research is required on how to rehabilitate and retrofit deficient structural wooden members to extend the service life of the structure.

The research focuses on the effects of reinforcement configurations consisting of glass-fibre reinforced polymer (GFRP) bars on the flexural behaviour of glulam beams. Of particular interest are the effects of reinforcement length, adhesive type, and knurling on the failure modes of the reinforced members when compared to unreinforced glulam. A total of eighteen pullout specimens were tested to investigate the effects of adhesive and knurling patterns on bond strength, and fourteen full-scale glulam beams were tested to failure under four-point static bending, including four unreinforced and ten GFRP-reinforced.

The pullout test results showed that the texture and density of an adhesive had a critical role on the overall behaviour with improved behaviour in specimens using a fluid density in comparison to those with dense. The addition of GFRP reinforcement to the glulam beams contributed to an increase in strength, failure displacement, and stiffness by factors ranging between 1.16 – 1.30, 1.04 – 1.24, and 1.13 – 1.18, respectively, in comparison to unreinforced glulam irrespective of the failure mode obtained. The effects of reinforcement length and termination point showed that the change from short to long bars resulted in improvements in maximum resistance and stiffness by factors of 1.10 and 1.19, respectively, for the bent bar reinforced specimens, and insignificant improvements for the straight bar reinforced specimens. Additionally, the change from straight to bent bars resulted in improvements in maximum resistance and stiffness by factors of 1.06 and 1.03, respectively, for the specimens with longer lengths of bars, and insignificant improvements for the specimens with short lengths of bars. The addition of knurling in the full-scale GFRP-reinforced beams resulted in increases of 1.07 and 1.03 for the maximum resistance and stiffness, in comparison to beams without knurling. Additionally, a change in failure mode from shear to flexure was observed with the addition of knurling.

A material model was developed to predict the flexural behaviour of unreinforced and GFRP-reinforced glulam beams, and the two proposed approaches were shown to generally captured the overall behaviour with a tendency to overpredict displacements at initial failure. Finally, the improvement in tensile failure strains in flexure due to the reinforcement was not observed to be present due to the mixed failure modes of shear and flexure. Strains from the digital image correlation system were observed to be lower than those measured by localized strain gauges, suggesting that measuring strains over a large area is critical.

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CHAPTER 1 | INTRODUCTION

1.1 General

An increased awareness of the impacts that various industries have on climate change and global emissions have encouraged change within the Canadian construction sector (Natural Resources Canada, 2021). As of 2023, the buildings and construction sector accounted for 37% of global emissions, as the manufacturing of products such as cement, steel and aluminum have a large carbon footprint (United Nations Environment Programme, 2023). Wood is a plentiful renewable resource in Canada that is well suited for various construction practices. In particular, wood products offer two effects towards lowering carbon emissions: 1. during growth wood absorbs atmospheric carbon and stores it within its structure of carbon, and 2. wood products require fewer fossil fuels to manufacture than non-wood products (Bergman et al., 2014). By incorporating the use of more wood products into building and construction practices, strides towards reduced global carbon emissions can be made. To that end, the Canadian government has supported initiatives aimed at growing the mass timber industry, including programs such as Green Construction through Wood (Natural Resources Canada, 2023b), and the Forest Innovation Program (Natural Resources Canada, 2023a).

The heightened interest in mass timber construction, combined with advancements in manufacturing technologies and research efforts on the performance of mass timber products and structural systems has led to major updates in the newest editions of the National Building Code of Canada (NBCC) (CCBFC, 2020) and International Building Code (IBC) (International Code Council, 2021). One of the most significant changes is that encapsulated mass timber can now be employed in the construction of 12- and 18-storey buildings in the NBCC (2020) and IBC (2021), respectively.

Mass timber is a family of engineered wood products (EWPs) that consist of smaller wood elements (e.g., dimensional lumber, veneers, strands) joined together with adhesives, dowels, nails, or screws to form larger structural products that can be used in buildings and bridges that were typically reserved for other materials such as concrete and steel. Examples of mass timber products include glued-laminated timber (glulam), cross-laminated timber (CLT), nail-laminated timber (NLT), and dowel-laminated timber (DLT). Mass timber products exhibit superior properties and improved behaviour in comparison to dimensional lumber and traditional heavy timber (Forestry Innovation Investment, 2023). Unlike for dimensional lumber and heavy timber, the cross-sectional dimensions and lengths of mass timber products do not

depend on the tree trunk size, thus making them suitable for mid- to high-rise buildings (e.g., Origine, QC, Apex Plaza, VA, T3 Bayside, ON), long-span facilities (e.g., Richmond Oval, BC, Haendel Park, QC, Lenexa Aquatic Center, KS), and bridges (e.g., Mistissini Bridge, QC, Maicasagi Bridge, QC, Montmorency Forest Bridge, QC).

1.2 Research Needs

Despite the increased adoption of mass timber in the buildings and construction industry, fundamental research is required on how to rehabilitate and retrofit deficient structural wooden members to extend the service life of the structure in order to avoid the need for decommissioning or replacing entirely. For instance, the lack of education on the rehabilitation and repair techniques for timber bridges has contributed to a 4.9% decrease of bridges with a timber superstructure in the North American bridge inventory between 1992 and 2019 due to many of them being decommissioned and replaced with new bridges employing other materials (Legg & Tingley, 2020).

Design provisions regarding the rehabilitation and repair of glulam beams using fibre reinforced polymers (FRPs) are currently lacking in the CSA O86 *“Engineering design in wood”* (CSA, 2024) and the CSA S6 *“Canadian Highway Bridge Design Code”* (CHBDC) (CSA, 2019). Although there are some design guidelines with regards to rehabilitating heavy sawn timber beams in the CHBDC, they are prescriptive and offer limited guidance such as “the reinforcement length termination shall be as close to the supports as possible” (CSA, 2019). Additional research is required to provide guidance on the reinforcement, rehabilitation, and repair of glulam beams using FRPs that can be implemented into codes and standards. Fundamental knowledge is required on the effects and implications of varying parameters on the overall response of FRP-reinforced glulam members, both in terms of performance and failure modes, prior to developing general design guidelines.

A common method for strengthening structurally deficient wooden members has been through the use of FRPs, such as glass (GFRP) and carbon (CFRP), in the form of fabric sheets or bars. FRP fabrics have successfully been employed in the past for the retrofitting and strengthening of old bridges (e.g., Meadow Lake Bridge 61.4, SK) as well as in new bridge construction (e.g., Lighthouse Bridge, WA). Furthermore, the use of hybrid FRP-glulam girders for a 32m span in a Western Washington University gymnasium contributed to reductions of 25% and 17% in width and depth of the hybrid FRP fabric-glulam member, respectively, compared to a conventional member leading to savings of approximately \$17,000 for the beams alone (Gilham & Williamson, 2007). FRPs have also been observed to reduce the variability in

strength by delaying crack opening and bridging the natural defects found in wood (Gentile et al., 2002; Johns & Lacroix, 2000; Lacroix et al., 2021; Lacroix & Doudak, 2018; Yang, Liu, et al., 2016). As a result, an enhancement in wood tensile failure strain in reinforced specimens compared to the unreinforced specimens is observed (Johns & Lacroix, 2000; Lacroix & Doudak, 2018, 2020; Vetter, 2022; Yang, Liu, et al., 2016).

Despite increases in resistance and stiffness up to 50% and 20%, respectively (e.g., Buell & Saadatmanesh, 2005; Dorey & Cheng, 1996b; Hernandez et al., 1997; Johns & Lacroix, 2000; Plevris & Triantafillou, 1992; Shrimpton et al., 2023; Vetter, 2022; Yang, Liu, et al., 2016), one of the main drawbacks of using FRP fabrics as simple tension reinforcement is the premature debonding of the FRP, caused by the wood on the tensile face failing in flexure and pushing outwards on the FRP reinforcement (Dorey & Cheng, 1996b; Hernandez et al., 1997; Johns & Lacroix, 2000; Lacroix & Doudak, 2018). Research has also shown that the FRP termination point when using fabrics can shift the failure from an initial flexural failure of the wood in the maximum moment region to one that is dominated by stress concentration at the FRP termination point with no debonding of the FRP (Che, 2023; Shrimpton et al., 2023; Vetter, 2022). It was shown that a change in the FRP termination point from 82.5 mm (Lacroix & Doudak, 2018) to 143 mm (Shrimpton et al., 2023) for beams with similar shear span-to-depth ratios was sufficient to alter the primary failure mode.

While FRP sheet reinforcement offers a means of external reinforcement, FRP bars are better suited for internal reinforcement applications. FRP bars offer a wide range of applications as they can be manufactured in various configurations including bent and straight and have the possibility of use in hybrid FRP-wood products. A common issue with FRP sheets is debonding, whereas studies investigating the behaviour of FRP bars for reinforcing wood members (e.g., Gentile et al., 2002; Yang, Liu, et al., 2016) have not reported any instances of debonding failures. Despite the results reported on observed failure modes, the studies have not focused on investigating parameters that could alter the failure mode.

Systematic research investigating the effects of key parameters affecting the behaviour of glulam beams reinforced with GFRP bars is required to address the lack of existing guidelines in the CSA S6 (CSA, 2019). The observed failure modes in the literature warrants further research in the performance of glulam reinforced with GFRP bars in terms of understanding adequate reinforcement lengths, quantity of reinforcement, and the development of a material model to predict the strength of these reinforced members.

1.3 Research Objectives

The overarching aim of the research is to develop fundamental knowledge of the behaviour of glulam beams when reinforced with GFRP bars experimentally and analytically. The research program focuses on the flexural behaviour under static loading and the effects of bent versus straight GFRP bars used as simple tension reinforcement. Of particular interest to the current study are the effects of reinforcement length, adhesive type, and knurling on the failure modes of the reinforced members when compared to unreinforced glulam. The specific research objectives are to:

1. Evaluate the performance of different adhesives and effects of knurling in pullout tests.
2. Determine the influence of reinforcement length and termination point on the flexural behaviour of glulam beams reinforced with GFRP bars as simple tension reinforcement.
3. Investigate the effects of knurling and profile of bar on the flexural behaviour of GFRP-reinforced glulam beams.
4. Develop a material model to predict the flexural behaviour of unreinforced and GFRP bar reinforced glulam beams.

1.4 Scope

The research objectives above are accomplished through:

- A thorough literature review on the behaviour of wood as a construction material, FRP as a reinforcing agent and FRP-reinforced wood members subjected to loading in flexure;
- Testing of wood coupons to establish the properties to be used as inputs in the analytical model;
- Testing a total of eighteen pullout tests to investigate the effects of knurling and adhesive types;
- Testing of four unreinforced glulam beams to establish baseline resistance curves;
- Testing of ten glulam beams with varying reinforcement configurations under static bending;
- Developing an analytical model to generate resistance curves to compare against experimental test results;
- A discussion of the results and proposal of recommendations for future work.

1.5 Thesis Organization

Provided below is a brief outline of the seven chapters included in this thesis:

Chapter 1 introduces the subject and states the research needs, objectives, and scope of the research program.

Chapter 2 presents a detailed literature review covering the topics of wood as a material, EWPs, FRPs, and the compressive and flexural behaviour of reinforced wood members.

Chapter 3 describes the experimental methodology used in this research program, including the specimen descriptions and test setups used for both the small- and full-scale tests.

Chapter 4 presents data and observations from both the small- and full-scale experimental tests.

Chapter 5 introduces the material properties and equations used to develop the moment-curvature and resistance curve models.

Chapter 6 provides a discussion on the experimental and analytical results.

Chapter 7 summarizes the key findings of the research and proposes potential future work.

CHAPTER 2 | LITERATURE REVIEW

2.1 General

This chapter is purposed to introduce important background knowledge pertaining to wood as a building material and provide a review of the work conducted over the years on wooden members and fibre reinforced polymers (FRPs). The foundation for understanding the material properties and behaviour of wood and FRP-reinforced wood members under applied loading is provided herein. Finally, a brief introduction to the methods of analysis for unreinforced and FRP-reinforced wood members is presented.

2.2 Overview of Wood as a Construction Material

2.2.1 Mass timber products

Mass timber is a family of engineered wood products (EWPs) that consist of smaller wood elements (e.g., dimensional lumber, veneers, strands) that are joined together with adhesives, dowels, nails, or screws to form larger structural products. Figure 2.1 shows examples of EWPs, including glued-laminated timber (glulam), cross-laminated timber (CLT), parallel strand lumber (PSL), laminated veneer lumber (LVL), and laminated strand lumber (LSL).

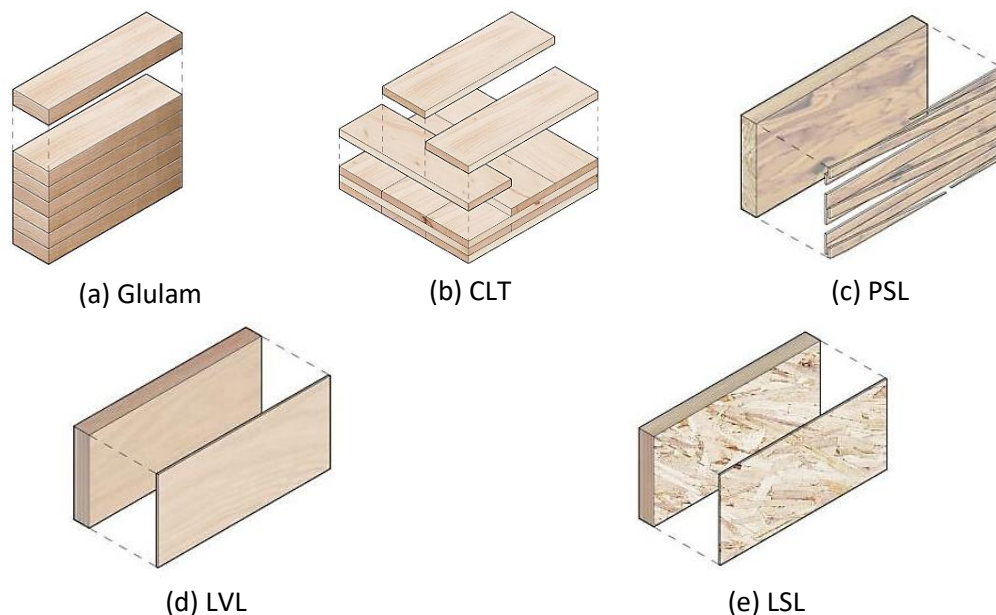


Figure 2.1 Mass timber products

Reproduced from Mass Timber Institute (n.d.)

Mass timber is rising in popularity due to its allowance for taller timber buildings, ease of assembly, strength, eco-friendly nature, and its aesthetically pleasing appearance (Natural Resources Canada, 2021). Additional advantages of mass timber products include superior strength than the individual boards or strands used, lower variability compared to dimensional lumber and traditional heavy timber, and larger sizes not limited by the size of the tree trunk.

Although considered as part of the mass timber family, each product has different properties and ideal applications given how they are manufactured. It should be noted that both glulam and CLT are made of dimensional boards whereas PSL, LVL, and LSL are considered structural composite lumber (SCL) and fabricated using veneers and strands of wood. Despite both using dimensional lumber, glulam and CLT are fundamentally different in their manufacturing and application. For example, CLT consists of layers of dimensional lumber that are oriented at right angles with respect to one another and glued together to form a structural panel. CLT can be used both for walls and floors due to their strength and dimensional stability. On the other hand, the dimensional lumber boards used in the manufacturing of glulam are aligned in the same direction and generally intended for beam and column applications. The following section will provide an overview of the manufacturing process of glulam and their properties. For additional information on other mass timber products, the reader is referred to the report on the state of mass timber in Canada (Natural Resources Canada, 2021).

2.2.2 Manufacturing and properties of glulam

Glulam is a popular product in the mass timber construction industry due to its versatility and broad variety of uses (e.g., bridges, buildings, canopies), including long-spanning and curved shapes, additionally, it has a high strength-to-weight ratio (Yang, Ju, et al., 2016). Glulam is an EWP comprised of pieces of standard dimension lumber glued together in horizontal laminations in accordance with CSA O122 (CSA, 2021). A generalized manufacturing process is shown in Figure 2.2 where it can be seen that the dimensional lumber boards undergo several steps prior to being glued together. Since the individual pieces of lumber are visually stress-rated or machine-rated, it allows for a tighter control on the individual laminations' properties to achieve configuration with more predictable characteristics than those found in sawn timber (Wood Solutions, 2024). To achieve longer lengths, the individual pieces are connected using finger-joints. These finger-joints are distributed throughout the glulam member and must meet stringent spacing and distribution requirements (CSA, 2021). The laminations are then pressed together using a mechanical or hydraulic press and are bonded together using adhesives.

Glulam members are graded based on their strength and appearance, and various grades are available for use dependent on the members intended use (e.g., flexure, tension, compression). In Canada, glulam is commonly made from a variety of species such as Douglas-Fir-Larch, Spruce-Pine-Fir, or a combination of Douglas-Fir-Larch and Hem Fir (CSA, 2024).

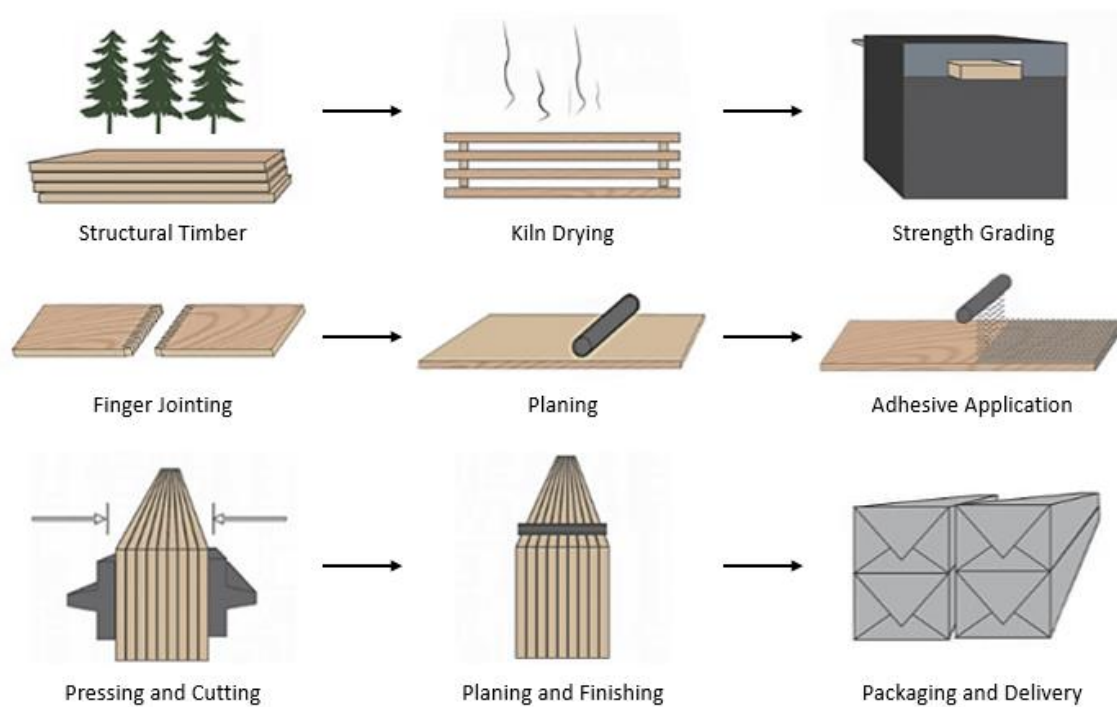


Figure 2.2 Glulam production process

Reproduced from RSB (n.d.)

2.2.3 Physical and mechanical properties of wood

Wood is considered an orthotropic material, meaning it has unique and independent properties in the three principal axes. Figure 2.3 shows a cross-section representative of a dimensional lumber board that would be obtained from the log conversion process where three mutually perpendicular axes are seen relative to the growth rings, namely, longitudinal, tangential, and radial (Forest Product Laboratory, 2021). For design purposes the tangential and radial directions are grouped as one, since they exhibit similar behaviour and because it is difficult in practice to know the precise cut of a tree that will be used during construction (Forest Product Laboratory, 2021). The mechanical properties are thus referred to relative to their directions respective to the grain in engineering applications: parallel-to-grain and perpendicular-to-grain.

Wood is a composite material composed of cellulose, hemicellulose, and lignin, with long and hollow cell structure. The purpose of the cells is to store and transfer water and nutrients. Water can be stored in either the cell cavities (free water) or the cell walls (bound water). As wood dries, the cell cavities are first drained of their free water, and the fibre saturation point (FSP) occurs once only the bound water is left. Drying beyond the FSP is when the bound water leaves and therefore changes in wood properties occur, most notably shrinkage (Legg & Tingley, 2020).

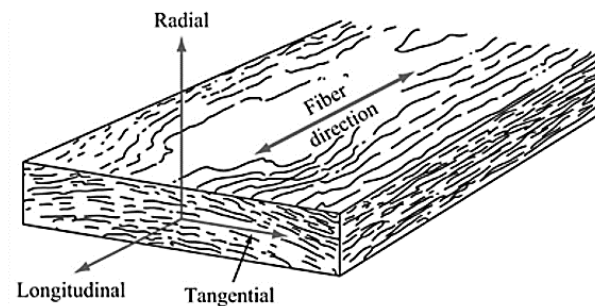


Figure 2.3 Wood grain orientations

Reproduced from Azeufack et al. (2019)

The physical properties of wood, such as density, shrinkage, and strength, vary with the species of tree and their growth process (You et al., 2021). Wood is also a viscoelastic material, meaning its mechanical properties are time-dependent and thereby affected by the rate of loading, load duration, creep, and fatigue (Forest Product Laboratory, 2021). The various species of trees can be divided into two categories: hardwoods and softwoods. Hardwood species are deciduous trees, with broad leaves that shed every year. Softwood species are coniferous with needle-like leaves and are generally evergreen; they tend to have tall, straight stems making them ideal for use as structural members (Harte, 2009). Species used for structural applications in Canada are divided into 4 categories as provided in the CSA O86 design guidelines: Douglas Fir-Larch (D.Fir-L), Hemlock-Fir (Hem-Fir), Spruce-Pine-Fir (SPF) and Northern species (NOR) (CSA, 2024).

Since wood is a natural material, it is subject to defects. Defects can emerge naturally or due to seasoning. Defects such as knots, compression zones or non-linear grain direction, occur naturally during the growth of the tree, whereas defects such as checks, splits, warping or bowing, are a result of seasoning (Harte, 2009).

2.2.4 Structural behaviour of wood under applied loading

The following section describes the behaviour of wood in tension, compression, and flexure. The description is not meant to be an exhaustive explanation, but rather to provide a brief description to provide context of the overall behaviour of the specimens tested in the current research program.

In the parallel-to-grain direction, wood behaves in a linear-elastically fashion with strengths of clear softwoods ranging between 70 to 140 MPa for a moisture content of 12% (Harte, 2009). The perpendicular-to-grain tensile strength can be as little as 5-8% and 3-5% of the parallel-to-grain strength for the radial and tangential directions, respectively (Harte, 2009). The perpendicular-to-grain tensile capacity of wood is usually considered to be negligible according to the design guidelines (CSA, 2024). The failure mode for both parallel- and perpendicular-to-grain in tension is brittle.

The compressive strength of clear wood parallel-to-grain ranges from 30 to 60 MPa for softwoods with a 12% moisture content (Harte, 2009). The failure modes of clear wood under compression parallel-to-grain, which are shown in Figure 2.4, can be classified into one of six categories: crushing, wedge split, shearing, splitting, brooming/end rolling, and compression and shearing parallel-to-grain (ASTM, 2022a). In the perpendicular-to-grain direction, the compressive strengths are approximately 10 to 20% of the parallel-to-grain values (Harte, 2009).

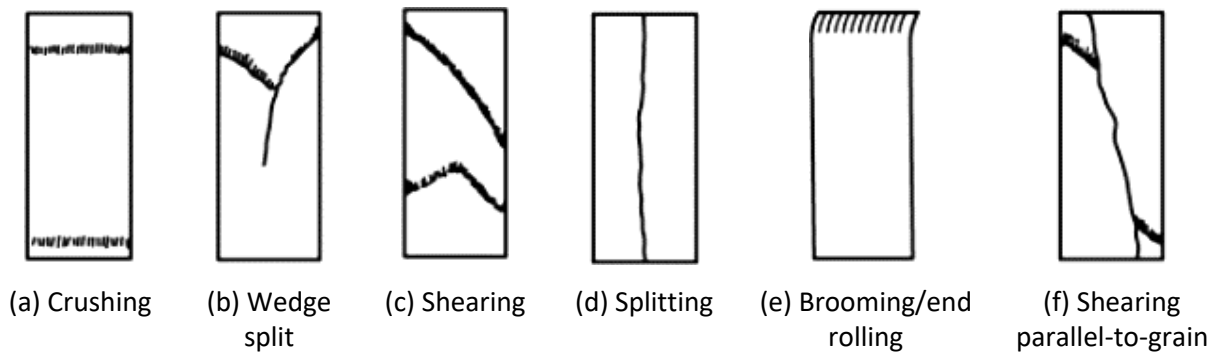


Figure 2.4 Compressive failure modes of wood under parallel-to-grain compression

Reproduced from ASTM (2022a)

The stress-strain model used to predict the behaviour of wood in compression parallel-to-grain is based on the model proposed by Bazan (1980) for clear wood and modified by Buchanan (1990a) to account for in-grade lumber. Over the years, variations of the idealized stress-strain relationship of wood subjected to compression parallel-to-grain loading were proposed by various researchers as shown in Figure 2.5. Initially, Neely (1898) proposed a simple elastoplastic model (Fig. 2.5a); however, the model did not

accurately describe the post-peak behaviour of wood under compression. Glos (1978) made changes to the model to quantify the falling branch (Fig. 2.5b), and although this model provided a better representation of the actual behaviour of full-scale wood members in compression parallel-to-grain, the parameters required to predict the actual behaviour needs calibration specific to the set of materials. To arrive at one of the most commonly accepted models used today, Bazan (1980) and Buchanan (1990a) simplified Glos' model to be a bilinear relationship with the falling branch being a ratio of the modulus of elasticity (Fig. 2.5c)

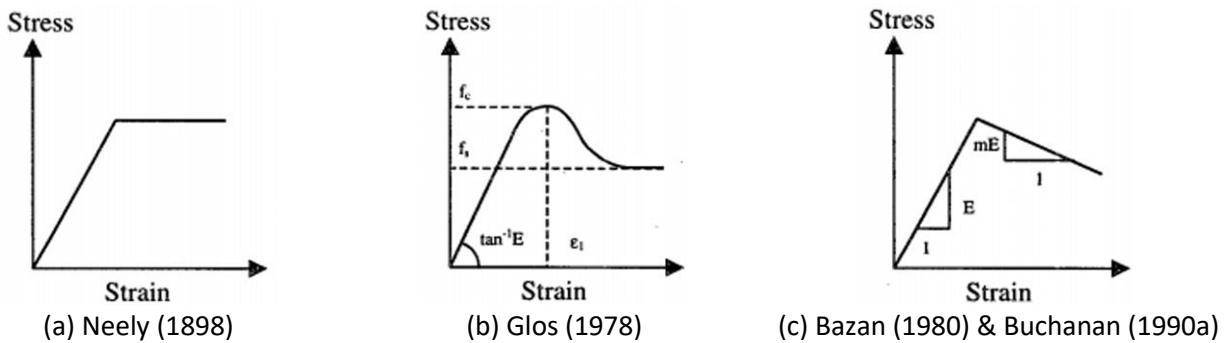


Figure 2.5 Stress-strain models for wood under compression loading

For wooden beams in flexure, the longitudinal grain is typically parallel to the span, thus engaging both tension and compression parallel-to-grain. The stress-strain model, shown in Figure 2.6a, is described with the tension behaviour assumed to be linear elastic, having failure occurring at a stress f_t , and the compression behaviour having a bilinear relationship, such that beyond maximum compression stress (f_c) and corresponding strain (ϵ_y), the slope of the failing branch follows a ratio (m) of the elastic modulus (E). The resulting strain and stress distributions for a planar rectangular cross-section are shown in Figure 2.6b.

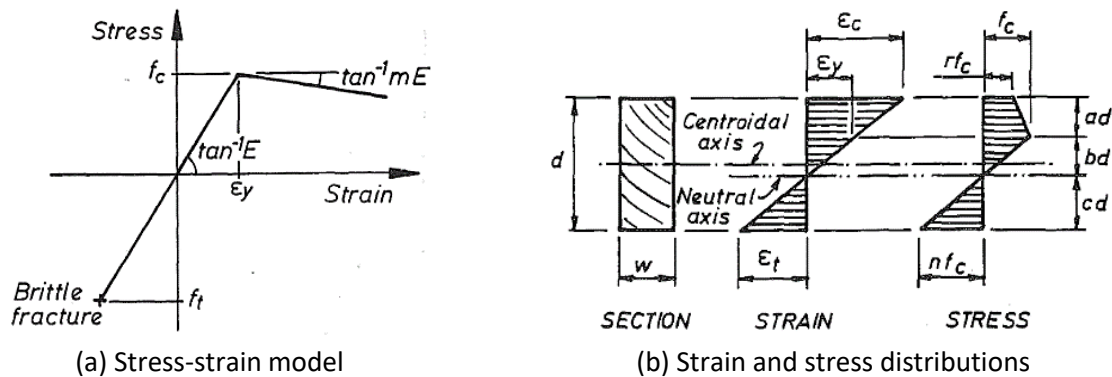


Figure 2.6 Strength models of wood

Reproduced from Buchanan (1990a)

The flexural failure modes of wood differ depending on whether or not the member is clear (i.e., free of defects). For clear wood members, flexural failure is governed by the wrinkling of the fibres on compression side of the member. Tension failure is typical of lumber with defects in flexure (Fig. 2.7a-d); however, other failures such as compression (Fig. 2.7e) and shear (Fig. 2.7f) can also occur in wooden members.

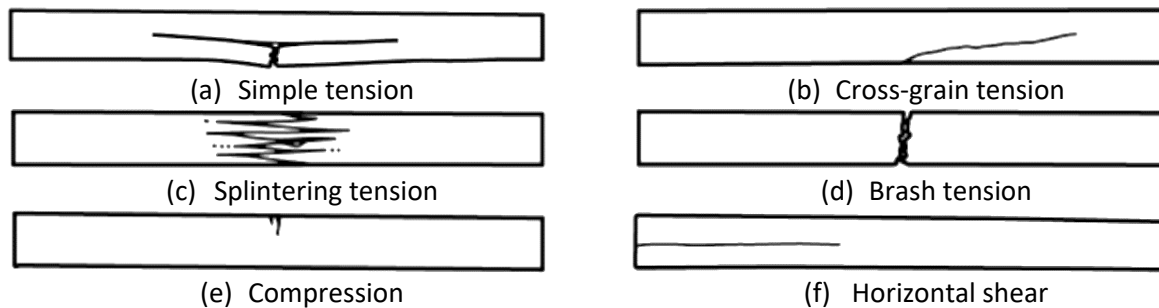


Figure 2.7 Failure modes of wood under static bending

Reproduced from ASTM (2022a)

2.3 Fibre Reinforced Polymers as a Strengthening Material for Wood

2.3.1 Overview of FRP materials

Fibre reinforced polymers (FRPs) can be used as a reinforcing agent for timber elements. FRPs are composite materials consisting of fibres embedded in a resin matrix. Their use for retrofitting steel and concrete structures dates back to the 1980's and was first used for wood structures in 1990 (Hoseinpour et al., 2018). FRPs are high performance materials, with many advantages, including being lightweight, non-conductive, having high mechanical strength and stiffness, and having corrosion resistance (Jian et al., 2022). The effectiveness of FRP as a reinforcing material is dependent upon the type and volume of the FRP, as well as the type of wood it is being applied to.

For the purpose of reinforcing wood, FRPs are commonly used in the form of sheets (Fig. 2.8a) or bars (Fig. 2.8b). Sheet reinforcement offers means of external reinforcement, while bars can be used for internal reinforcement applications. For their use in rehabilitation and repair, FRPs are best used in the form of sheets due to their ease of application in comparison to bars (Corradi et al., 2017). FRP sheets can be used in one or more layers and are applied by first saturating the sheet with a resin, then adhering the sheet on the surface of the element to cure. FRP bars, on the other hand, are placed in grooved out sections of the element and bonded to said element through the use of chemical adhesives. FRP bars can

be manufactured in various configurations, including straight and bent bars, allowing for their use in a wide array of reinforcement schemes.

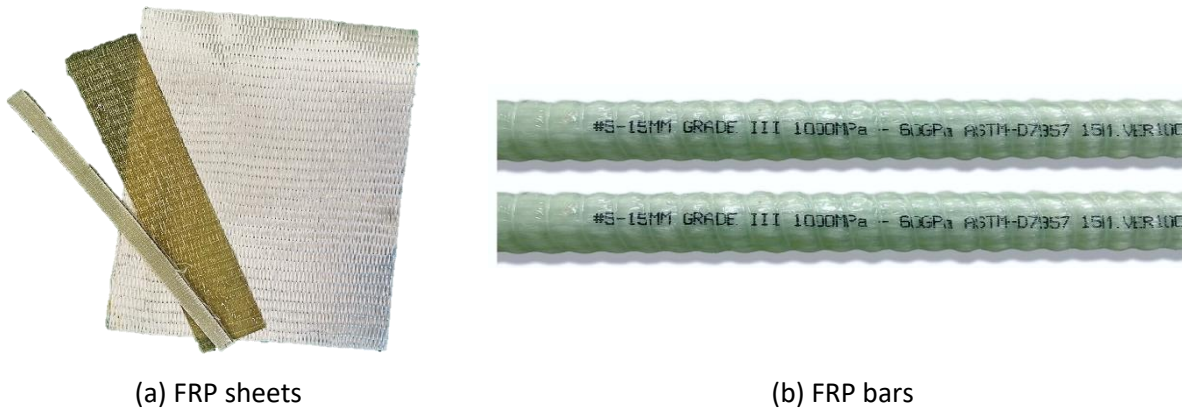


Figure 2.8 Forms of FRP reinforcement

Reproduced from MST Rebar Inc. (n.d.)

There are multiple types of FRPs available for structural retrofitting, most notably; carbon fibre reinforced polymers (CFRPs) and glass fibre reinforced polymers (GFRPs). This experimental program will focus on the use and behaviour of GFRP bars. GFRPs are used in the construction industry because of their high tensile strength and strong insulating properties while remaining at a low cost (Ascione et al., 2024). They are composed of glass-fibre filaments and a resin matrix. The load-bearing components are the filaments, possessing a high tensile strength and a high modulus of elasticity. The matrix is purposed to hold the fibres together, preventing shearing between them, as well as maintaining the dimensional stability of the GFRP (Benmokrane et al., 1995).

The difference in the modulus of elasticity between timber and FRPs makes it ideal for strengthening timber elements, as the FRP contributes to the stiffness and strength of the member (Kliger et al., 2016). Although FRP reinforcement can improve the reliability and strength of timber elements, recent studies have shown that under freeze-thaw cycles FRP-reinforced elements exhibit brittle behaviour compared to those at room temperature (Neale, 2000). It is important to understand the different bonding methods of FRP to timber as it can influence the mechanical properties of these structural elements. The bonding methods of FRP to wood structural elements include bottom anchoring, externally bonded reinforcement (EBR), vertical near surface mounting (VNSM), horizontal near surface mounting (HNSM) and glued-in rods (GiR). The bond strength also varies with shape, mechanical properties, boundary conditions and arrangement of the wood specimen (Jian et al., 2022).

2.3.2 Bonding of reinforcement to wood using adhesives

When working with FRPs, it is necessary to use an adhesive to bond the FRP to the wood. Commonly used adhesives for glued-in rods are epoxies, polyurethanes (PUR), and phenol-resorcinol-formaldehyde (PRF) based adhesives. Epoxy adhesives are both brittle and strong, thus fillers can be added to decrease cost, brittleness, and shrinkage, as well as to increase viscosity. PUR adhesives are characterized as ductile, plastic materials when hardened (Kemmsies, 1999). While PRF adhesives offer similar properties, regulations and exposure limits regarding the use of them have been implemented due to the presence of formaldehyde (APA, 2018).

Previous studies on the performance of various adhesives, given all other parameters are the same (bar diameter and glue-line thickness), show that glued-in rods with epoxies exhibited the highest axial capacity, followed by PUR (Broughton & Hutchinson, 2001; Lavischi et al., 2003). Although PUR adhesives often exhibit lower of stiffness and strength than epoxies (Vallée et al., 2017), the gap-filling ability of PUR adhesives is an important characteristic (Gonzalez et al., 2016). The choice of adhesive depends on the application and desired properties. For example, for a bar placed in an oversized hole, an adhesive with strong gap-filling properties should be used. Other factors affecting the choice of adhesive are the desired failure type and location of said failure, bearing capacity, geometric properties, and ease of application (Tlustochowicz et al., 2011).

The bond between the wood, adhesive, and FRP bar can be investigated through tensile pullout tests, where the bar is pulled upwards out of the specimen until failure occurs. Possible failure modes are shear, tensile, tear-out, splitting and yielding (Fig. 2.9). A shear failure can occur along the bar due to the shear forces in the connection in a single member (Fig. 2.9a) or in multiple members (Fig. 2.9c). It is possible for a failure to occur behind the top face of the specimen due to tensile stresses (Fig. 2.9b). A splitting failure (Fig. 2.9d) can also occur, which is usually caused by tension perpendicular to grain. Serrano (2001) states that splitting is the dominant failure mode in specimens where the edge distance is 1.5-2.25 times the diameter of the bar. The last possible failure mode that can happen is when the bar yields (Fig. 2.9e), which results in more deformation (i.e., ductility) compared to other failure modes. The latter depends on the type of bar used, for example, if a steel bar is used, then significant ductility can be used. However, if GFRP bars are used, the ultimate failure will be sudden and brittle despite the increased deformations due to the GFRP behaving in a linear-elastic fashion.

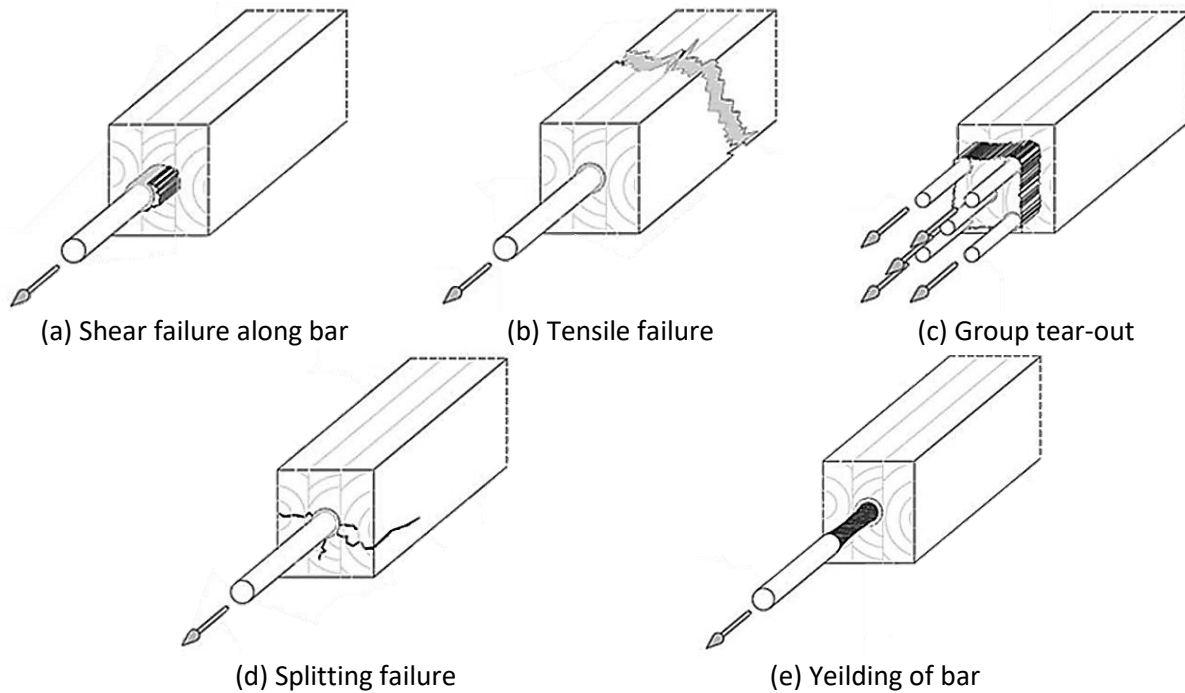


Figure 2.9 Pull-out test failures

Reproduced from Tlustochowicz et al. (2011).

2.3.3 Knurling of wood and development lengths

Knurling is the process of creating a textured pattern on a surface to improve grip through friction and is most commonly done to metals. A previous study by Albers and White (1996) investigates the effects of surface treatments on the shear bond strength between epoxy and aluminum. Rod pull-out tests were conducted to determine the interfacial shear strengths. The results have shown that surface knurling caused a 72% increase in the interfacial shear strength when compared to the untreated specimens. Although the technique of knurling is commonly used for metals, it can be done to wood as well (John, 2023). Lyons and Ahmed (2005) hypothesized that knurling could be beneficial for increasing the bond strength between the FRP rods and wood surfaces. The effects of the application of composites on a rough surface versus a smooth surface showed scattered and inconclusive results, warranting further research to be done on the topic.

The bond between the reinforcement and structural members plays an important role in the expected performance. When determining bond strength, one parameter to consider is the development length. The development length refers to the length of the rebar embedded into the member required to achieve the desired bond strength between any two types of materials. The bond strength between FRP and timber can be measured through pullout testing. Pullout testing procedures are specified in ASTM D7913

“Standard Test Method for Bond Strength of Fiber-Reinforced Polymer Matrix Composite Bars to Concrete by Pullout Testing” (ASTM, 2021a) for FRP and concrete, and similar practices have proven successful for FRP to timber (Kemmsies, 1999; F. Wang & Xiong, 2019). Liu (2025) performed a series of pullout tests on FRP bar reinforced timber specimens to investigate the effects of development length on bond strength. The results showed a change in failure mode from FRP bar pullout to either shear failure of the lumber around the bonded surface or a combination of the bar pullout and vertical splitting occurred due to increased development length (Liu et al., 2025). Also, the strain developments remained concentrated around the loaded end for specimens with longer development lengths (Liu et al., 2025).

2.4 Behaviour of FRP-Reinforced Wood

2.4.1 Overview

Previous research on reinforcing wood products has included an array of reinforcement materials and configurations. The use of steel to reinforce wood members has been studied, including applications of steel plates (Dar et al., 2021; Waseem et al., 2022), bars (Bulleit et al., 1989; Gharaibeh & Doudak, 2023; Soriano et al., 2016) and pre- and post-tensioned tendons (Al-Hayek & Svecova, 2014; McConnell et al., 2014; Zhang et al., 2020), but have seen limited commercial success. Since, the use of FRPs have become a viable and popular option. The use of FRP as a strengthening material for concrete has been studied vastly (e.g., Ben Oueddou et al., 2009; Benmokrane et al., 1995; Pellegrino et al., 2008; S. T. Smith, 2011; Teng et al., 2001; Yao et al., 2005), and many of the theories of the wood-to-FRP relationship stem from that of the concrete-to-FRP.

The following sections will provide a review of the research conducted on the reinforcement of wood elements in compression and in flexure using FRP fabrics and bars. Different models to predict the flexural strength of FRP-reinforced wood is also reviewed.

2.4.2 Application of FRPs to reinforce the compression strength

Compression members such as timber columns, posts, and bridge piles often times use FRP confinement as a method of strengthening. Various studies have been conducted to investigate and better understand the behaviour of FRP-reinforced wood members under compressive loads. Overall, the effects of compression reinforcement are improved post-peak behaviour and load-carrying capacity while the effects on ductility vary.

Dong et al. (2015) investigated the behaviour of square timbers strengthened with externally bonded FRP sheets under axial compression. The load-carrying capacity of the reinforced members showed improvements ranging from 68.9 to 100.2%; however, this was accompanied by a decrease in the ultimate displacement between 46.1 to 56.6%. No proportional relationship was seen between the load-carrying capacity and the reinforcement ratio. Other improvements were seen in the post-peak behaviour and stiffness. O’Callaghan (2021) studied the effects of GFRP wraps using unidirectional and bidirectional fabrics on timber columns on the stress-strain relationship and failure modes. A total of 36 columns were tested to failure with results showing that with an adequate thickness GFRP reinforcement can prevent splitting and shearing failure modes, negate the effects of defects on crushing, and improve the post-peak behaviour. In comparison to the unreinforced specimens, the GFRP reinforced specimens averaged peak stresses and modulus of elasticity of 1.01-1.39 and 1.14-1.26 times greater, respectively. Figure 2.10 shows a representative failure for an unreinforced (Fig. 2.10a) and a GFRP reinforced (Fig. 2.10b) beam, it can be seen that the reinforcement causes the damage to be localized, whereas in the unreinforced specimen the damage has propagated throughout the specimen.



(a) Unreinforced member failure



(b) GFRP-reinforced member failure

Figure 2.10 Effects of GFRP confinement on timber columns

Reproduced from O’Callaghan et al. (2021)

The effects of FRP reinforcement on the ductility of wood members show contrasting results. Some studies showed improvements in ductility (Bhat, 2021; Najm et al., 2007; H. Wang et al., 2023; Xiong & Su, 2015), while other research observed negative impacts on the ductility (De La Rosa Garcia et al., 2021; Dong et al., 2015; Li et al., 2013). Bhat (2021) studied the behaviour of CLT columns with FRP sheets and

found that for both long and short columns the maximum ductility occurred when the specimens were wrapped with double-cross helix FRP. Najm et al. (2007) investigated the behaviour of circular timber columns with CFRP reinforcement, results showed that full FRP confinement increased the ductility. Wang et al. (2023) saw improvements in the ductility of long timber columns externally wrapped with hybrid-FRP. Li et al. (2013) conducted a study on the behaviour of short columns with CFRP sheet reinforcement and found that the improvements in the carrying capacity and stiffness were at the cost of the ductility. Similarly, Rosa Garcia et al. (2021) found that CFRP sheets could improve the carrying capacity by 96 to 133%; however, it was accompanied by a decrease in ductility ranging from 16% to 25% lower. Kim and Andrawes (2016) studied the use of FRP fabrics to strengthen timber piles under to uniaxial compression after being subjected to accelerated aging. Results showed an increase in capacity for the control and aged specimens of up to 80% and 35%, respectively. Increases in ductility of up to 90% were seen in comparison to the unreinforced specimens, and the accelerated aging was observed to have no effect on the ductility.

2.4.3 Application of FRPs to reinforce flexural behaviour

Most of the literature on the topic of FRP and wood is directed to study the flexural behaviour of the FRP-reinforced members. The primary objective of flexural reinforcement is often to increase the strength and stiffness of the member, and in some applications (e.g., impact, blast) to increase the overall ductility.

The use of FRP sheets or plates purposed for simple tension reinforcement of wood members has been studied extensively. Overall, the effects of simple tension FRP reinforcement on the behaviour of wood are increased strength, stiffness, and ductility, as well as decreased variability in said behaviour (Johns & Lacroix, 2000; Plevris & Triantafillou, 1992; Raftery & Harte, 2011). The increase in strength is attributed to the local bridging of defects and confining action the FRP provides (Johns & Lacroix, 2000; Raftery & Harte, 2011). Bridging the effects are captured when modelling the behaviour of FRP-reinforced members by using an α_m factor which accounts for increased wood tensile strain at failure with FRP. The model and α_m factor are discussed further later in this section. However, results can be greatly influenced by the quantity and properties of the FRP used as well as the wood product. Plevris & Triantafillou (1992) conducted research on the FRP area fraction and its significant effects on the behaviour of a structural member. The FRP area fraction is dependent on three parameters: the ratio of tensile to compressive failure of the wood, the ratio of FRP ultimate strain to wood compressive strain, and the ratio of the elastic moduli of FRP and wood. The effects of the FRP area fraction show that with a small increase of 1%

substantial increases in moment capacity and stiffness are observed. However, a critical FRP area fraction is observed for the moment capacity, beyond which the bending moment capacity plateaus.

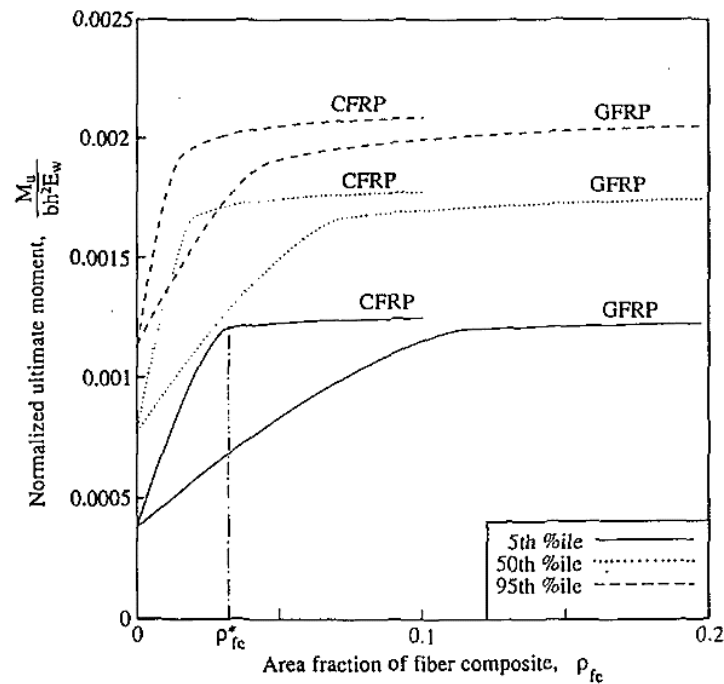


Figure 2.11 Dependence of ultimate bending capacity on FRP area fraction

Reproduced from Plevris & Triantafillou (1992)

A drawback of using simple tension reinforcement in flexure is the partial-length or full-length debonding of the reinforcement that occurs when the outer wood tension layers fail (Dorey & Cheng, 1996a, 1996b; Hernandez et al., 1997; Lacroix & Doudak, 2018). Figure 2.12a shows the debonding that occurred in the experimental program conducted by Lacroix and Doudak (2018). A different failure mode was observed by Vetter (2022) where instead of premature partial- or full-length debonding of the reinforcement due to the wood failing and pushing outwards on the FRP, the failure was observed to occur at the termination of the reinforcement due to stress concentrations (Fig. 2.12b-c). Although no debonding occurred, this failure mode is considered undesirable and highlights the importance of reinforcement length on the behaviour of the reinforced specimens (Shrimpton et al., 2023, 2024; Vetter, 2022).



(a) Debonding of reinforcement



(b) Failure at FRP termination point



(c) Failure at FRP termination point

Figure 2.12 Failure modes of simple tension reinforced sawn timbers

Reproduced from Lacroix & Doudak (2018) and Vetter (2022)

Another method of using FRP as reinforcement is in the form of partial- or full-length confinement. Typically, confinement reinforcement is provided in the form of unidirectional and/or bidirectional FRP fabrics. The effects of confinement reinforcement on the behaviour of wood generally causes an increase in strength and stiffness (Buell & Saadatmanesh, 2005; De La Rosa García et al., 2013; Lacroix & Doudak, 2020; Vetter, 2022).

Buell et al. (2005) studied the behaviour of strengthening timber bridge beams with bidirectional carbon fabrics. A total of 10 beams were tested under four-point loading. Results showed increases in stiffness from 5 to 27%, ultimate bearing capacity from 40 to 53% and horizontal shear strength from 28 to 68%. It was also observed that the FRP provided support and confinement to the wood, reducing the effects of defects which significantly reduce the strength and stiffness of the beams. Rosa Garcia et al. (2013)

conducted a study on the behaviour of timbers reinforced with different grammages of basalt fibre reinforced polymers (BFRP) and different configurations for CFRP. A total of 26 beams were tested to failure under three-point bending where the reinforced beams showed higher fracture loads and increased rigidity; however, no increases in ultimate displacement were observed. Conclusions were made that the use of CFRP is optimal compared to the BFRP, due to its higher elasticity modulus. Lacroix et al. (2021) studied the effects of GFRP on wood studs in terms of flexural performance. A total of 48 unreinforced and GFRP-reinforced members were tested to failure under four-point bending. A variety of reinforcement schemes were investigated. It was observed that specimens with U-shaped reinforcement saw increases in strength ranging between 1.2-1.9 and increases in stiffness of up to 1.3. Ductility ratios of 3.0-3.7 were observed. Compared to the unreinforced specimens, the post-peak strength was sustained for much larger displacements for the U-shaped reinforced specimens. The addition of a fully wrapped layer on top of the U-shaped layer showed increases in strength and stiffness of 1.7-2.4 and 1.3-1.8, respectively, compared to the unreinforced members. Ductility ratios of 2.8 to 3.9 were reported and can be attributed to the combined effect of extending the FRP to the side faces of the stud and the confinement, which prevented delamination of the FRP at the initial wood fracture. In general, the addition of FRP localized the damage, as shown in Figure 2.13



(a) U-shaped failure (localized damage)



(b) Full confinement failure (localized damage)

Figure 2.13 Representative failure modes of FRP-reinforced wood studs

Reproduced from Lacroix et al. (2021)

Lacroix and Doudak (2020) conducted research on the effects of full confinement FRP reinforcement on the behaviour of glulam beams subjected to blast loading. Increases in maximum resistance and stiffness by factors ranging from 1.2-1.7 and 1.0-1.3, respectively, were observed in the reinforced members relative to the unreinforced beams. Representative failures of the beams are shown in Figure 2.14. The reinforcement was shown to confine the failure to the midspan of the specimens (Fig. 2.14a). Unzipping failures (Fig. 2.14b) were present due to the high-strain rate loading; however, the confinement provided sufficient reinforcement to minimize the wood damage.



(a) Localized damage



(b) Unzipping failure

Figure 2.14 Full confinement failure modes

Reproduced from Lacroix & Doudak (2020)

The use of FRPs in the form of bars as a practice for reinforcing wood members has also been studied. In general, the effects of FRP bars on the behaviour of the beams are increased strength and stiffness (A. Yusof & A. L. Saleh, 2010; Gentile et al., 2002; Yang, Liu, et al., 2016).

Gentile (2002) studied the flexural behaviour of timber beams strengthened with near-surface mounted GFRP bars. Half- and full-scale tests were conducted, and results showed an increase in ultimate load and modulus of rupture, and increased stiffness, respectively. It was observed that the addition of GFRP bars caused a change in failure mode from brittle tension to ductile compression. Similar to FRP fabrics, the bars were able to overcome the effects of defects. Increases in flexural strength were seen between 18 to 46%. A modification factor α_m of 1.3 was introduced to account for this strength enhanced in the model. Yang et al. (2016) incorporated this modification factor into their model and suggested that further research be done on this modification factor to account for timber grades. Further research into the use of modification factor α_m was conducted by Lacroix and Doudak (2018) where the wood tensile failure strains of unreinforced and GFRP sheet reinforced glulam beams were measured and used to determine an α_m of 1.17. Similarly, Shrimpton et al. (2024) conducted flexural tests on unreinforced and FRP sheet reinforced glulam beams, and the results were modelled with $\alpha_m = 1.17$ and 1. In scenarios where non-flexural failures occurred, the modelled results with α_m factors larger than 1 produced inaccurate results. The study highlighted the importance of accounting for diverse failure modes and behaviours of

unreinforced and FRP-reinforced glulam beams. A study conducted by Yusof & Saleh (2010) found similar results to that of Gentile's. The program included the testing of 6 GFRP bar reinforced timber beams to failure under four-point bending. The addition of GFRP rods increased the ultimate load by 20 to 30% and the stiffness by 24 to 60%. The same change in failure mode, from brittle to ductile, was seen when the percentage of GFRP was greater than 0.35%.

Che (2023) conducted research on both sawn timbers and glulam beams reinforced with GFRP bars. Reinforcement ratios of 2.16% and 1.81% were used for the sawn timber and glulam specimens, respectively. Failure for both the sawn timber and glulam originated in the wood cover in the unreinforced tension fibres but the depth of crack propagation arrested at the GFRP bars as shown in Figures 2.15 and 2.16, respectively.

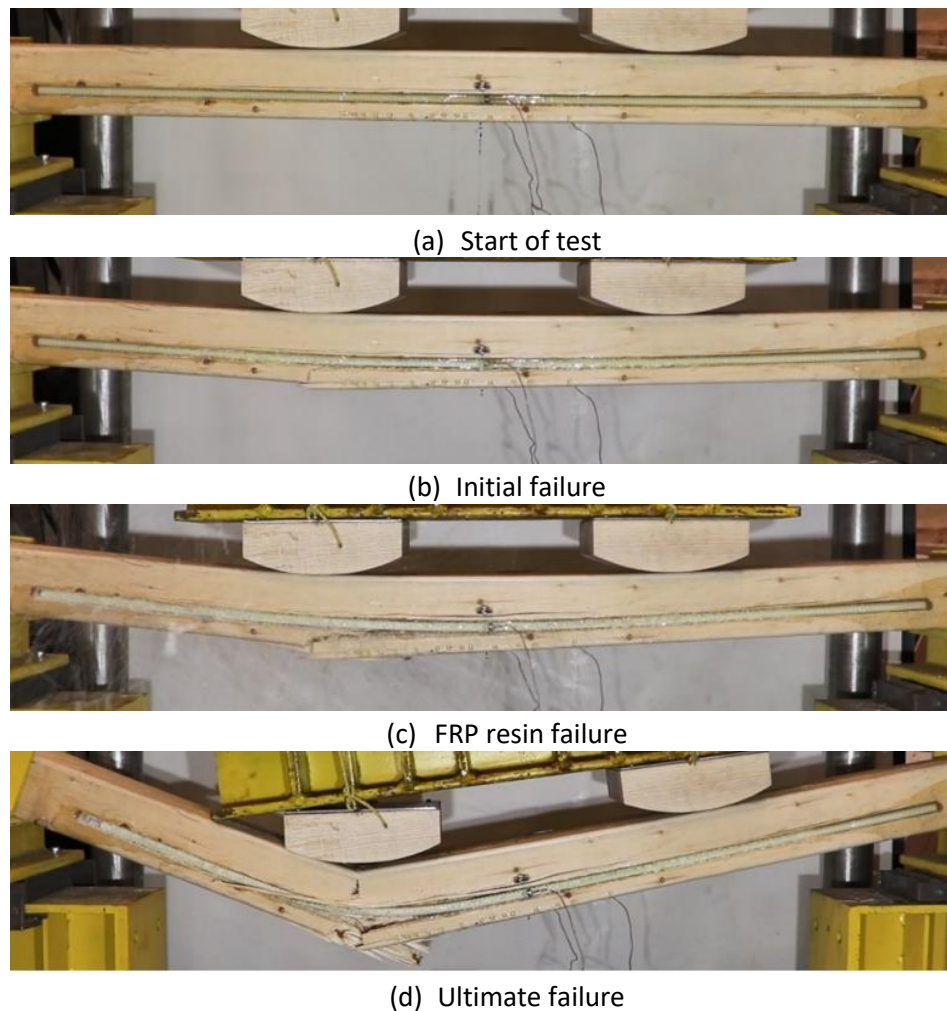


Figure 2.15 Representative failure mode of a sawn timber member reinforced with GFRP bars

Reproduced from Che (2023)

The next component to fail was the epoxy resin encasing and bonding the GFRP bars to the wood, the failure observed was a sudden shattering of the epoxy with a partial bond remaining, allowing for the specimens to continue developing post-peak strength as a part of the semi-composite beam. The behaviour of the sawn timber and glulam specimens began to differ upon further loading. The sawn timber specimens showed the cracks continuing to propagate across the depth where the initial failure occurred, creating a localized failure (Fig. 2.15). For the glulam specimens, the cracks were observed to propagate throughout the specimens, following the path of least resistance (Fig. 2.16).

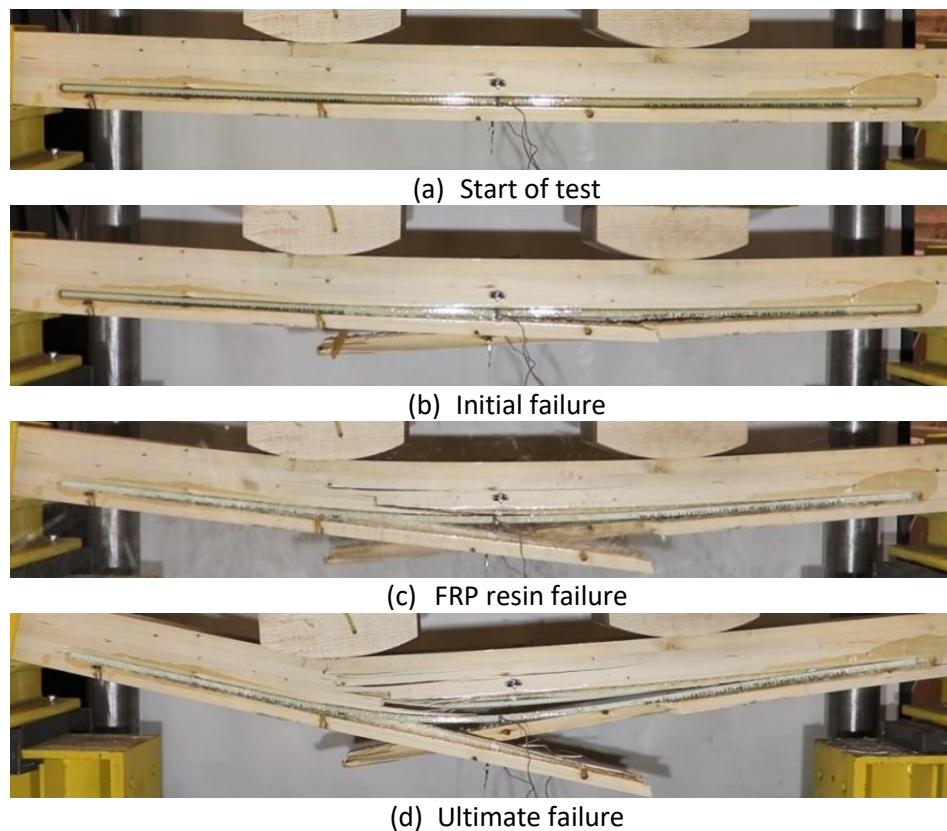


Figure 2.16 Representative failure mode of a glulam beam reinforced with GFRP bars

Reproduced from Che (2023)

Since the GFRP bars were side mounted rather than near-surface mounted, no debonding of the GFRP was observed in neither the sawn nor glulam members, unlike in simple tension reinforcement with fabrics. However, due to the unreinforced wood cover on the tension side, the reinforced specimens observed little difference in terms of peak strength and stiffness compared to the control specimens. The reinforcement did contribute to a significant improvement to the post-peak performance and ductility. Overall, the beams were then able to redevelop load resistance up to 60% and 95% of the peak load in the sawn timber and glulam beams, respectively (Che, 2023).

2.4.4 Predicting the flexural strength of FRP-reinforced wood beams

There exists a need for material models to accurately predict the behaviour of reinforced wood members. Methods of analysis include the transformed-area, force-equilibrium, and moment-curvature methods. In this thesis the moment-curvature method will be used as it is capable of including the idealized bilinear behaviour of wood in compression.

As discussed, a material model was originally developed based on the behaviour of wood being linear elastic in tension up to a failure stress of f_y and bilinear in compression such that beyond maximum compression stress (f_c) and corresponding strain (e_y) the slope of the failing branch follows a ratio (m) of the elastic modulus (E) (Buchanan, 1990a). Later a modification factor α_m was introduced to the model by Gentile et al. (2002), purposed to account for the higher wood tensile failure strains in FRP-reinforced wood.

Other researchers have adequately modelled their results using the non-linear stress-stress relationship and modification factor α_m . Lacroix and Doudak (2018) applied a moment curvature approach in their investigation to obtain the bending resistance for the timber beams reinforced with FRP in a semi-confined application. The program was able to derive resistance curves from the moment-curvature and stress-strain relationship. An α_m factor of 1.17 was used in this study to account for the enhancement of the bending strength. The model was able to accurately predict the maximum resistance and displacement at maximum resistance, shown in Figure. 2.17.

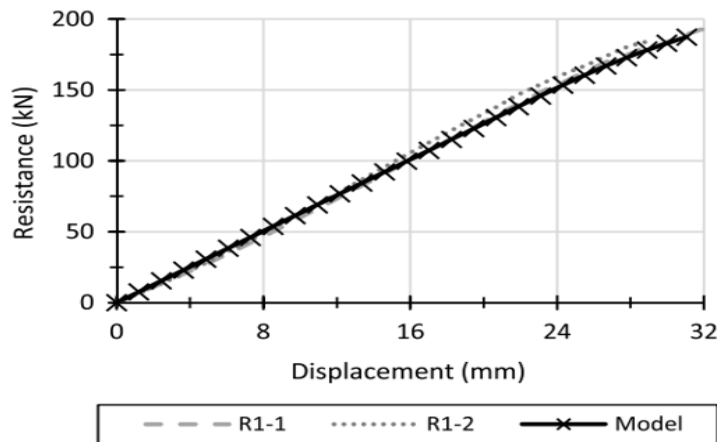


Figure 2.17 Experimental versus modelled results

Reproduced from Lacroix & Doudak (2018)

A numerical study was conducted by Lacroix (2017) to investigate different wood tension and compression modulus of elasticity ratios and axial loads and their effect on the out-of-plane flexural response. The results, shown in Figure 2.18, are consistent with that of Buchanan's (1990a) such that an increase in axial load results in a decrease in moment capacity.

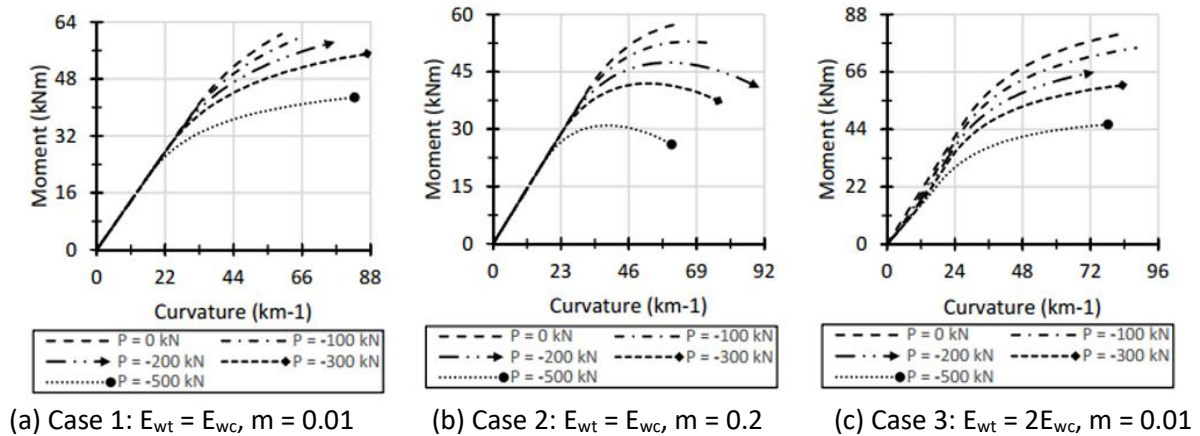


Figure 2.18 Moment-curvature analysis

Reproduced from Lacroix (2017)

2.5 Summary

The recent uptake in wood structures has led to the need for a more thorough understanding of the effects reinforcement can have on wood members. Research is needed to further develop design guidelines and to contribute to code advancements. The use of FRP composites have proven to be a promising means of reinforcing wood members. In terms of the flexural behaviour of glulam beams, the strength provided by the FRP, whether that be in the form of sheets or bars, has been shown to cause increases in strength and stiffness of the member, as well as decrease the effects of defects. The behaviour of FRP-reinforced compression members shows improved post-peak behaviour and load-carrying capacity.

Despite the significant progress made towards understanding the effects of FRP as a means for reinforcement, there is little research directed at the effects of reinforcement configuration, specifically in terms of reinforcement lengths, adhesives, straight versus bent bars, and knurling. As such, further research is needed into the effects of key parameters aforementioned in order to develop design guidelines for FRP-reinforced glulam beams with GFRP bars.

CHAPTER 3 | EXPERIMENTAL PROGRAM

3.1 General

This chapter describes the experimental program investigating the flexural behaviour of both the unreinforced and FRP-reinforced glulam beams under static loading. The experimental program can be divided into three phases, namely the testing of wood coupons to obtain the material properties to be used as inputs to the material model (i.e., Chapter 5), pullout tests investigating the effects of different adhesives and knurling on the bond strength, and full-scale tests of unreinforced and reinforced beams subjected to static four-point bending. A total of sixteen wood coupon tests, eighteen pullout tests and fourteen full-scale beam tests were completed. The following sections provide descriptions of the test specimens, experimental test setups, and procedures.

3.2 Materials

3.2.1 Glulam

The glulam used in this study was obtained from a single supplier and consisted of four 137mm x 241mm x 16,459mm 24F-ES glulam beams. The glulam was prepared to different sizes, as shown in Figure 3.1 for the flexural tests and Figure 3.2 for the wood tension and compression coupons as well as pull-out tests.

For the purpose of the flexural tests, the specimen lengths were determined in accordance with ASTM D198 “*Standard Test Methods of Static Tests of Lumber in Structural Sizes*” (ASTM, 2022b). According to the standard, a shear span to depth (a/d) ratio between 4 to 6 allows for the evaluation of flexural properties of uniform rectangular cross-section. Furthermore, specimens with a/d ratio near 2.5 would produce a high percentage of shear failures. It should be noted that these ratios are based on the commonly accepted assumption that the flexural stress (i.e., MOR) is 10 times greater than the shear stress (τ_m); however, this depends on the lumber grade and type of product amongst other variables (ASTM, 2022b).

The use of various a/d ratios have been seen in numerous studies where flexural failures occurred in unreinforced and reinforced glulam beams (Gharaibeh & Doudak, 2023; Lacroix & Doudak, 2018; Vetter, 2022). The dimensions, shear spans and a/d ratios from past research are summarized in Table 3.1.

Table 3.1: Summary of a/d Ratios

Research	Length (mm)	Clear Span (mm)	Shear Span (mm)	Depth (mm)	a/d
Gharaibeh & Doudak (2023)	2,500	2,235	745	191	3.9
(Lacroix & Doudak, 2018)	2,500	2,235	745	222	3.4
Vetter (2022)	1,355	1,200	400	100	4.0

Therefore, the specimens intended for full-scale flexural tests were cut in lengths of 2,800mm with a clear span of 2,500mm. This resulted in an a/d ratio of 3.46 which is in-line with the previous literature albeit being smaller than 4 but still was above the ratio of 2.5 that would result in a high percentage of shear failures. This decision was done to maximize the material. A total of four unreinforced 137mm x 241mm x 2,800mm 24F-ES glulam beams were tested to failure.



Figure 3.1 Glulam beams

Wood coupons to be tested in tension and compression were fabricated in accordance to ASTM D143, “Standard Test Methods for Small Clear Specimens of Timber” (ASTM, 2022a). A total of eight compression coupons were cut to size using a table saw, with final dimensions of 45mm x 54mm x 200mm as it can be seen in Figure 3.2a. Eight wood tension coupons, shown in Figure 3.2b, were prepared using a bandsaw, with dimensions of 45mm x 54mm x 460mm and a reduced cross-section of 5mm x 10mm. To evaluate the bond of the GFRP bar to wood, a total of eighteen glulam specimens with dimensions of 137mm x 241mm x 400mm were prepared (Fig. 3.2c).



(a) Compression



(b) Tension



(c) Pullout

Figure 3.2 Wood component specimens

Prior to testing, all glulam specimens were stored in a humidity chamber, allowing for the specimens to remain at an average moisture content of 11% with a coefficient of variation of 0.12. The average density of the beams was determined to be 551 kg/m^3 with a coefficient of variation of 0.02.

3.2.2 FRP

The GFRP bars in this study were obtained from a single supplier. The bars used were all No. 6 bars, having a diameter of 20mm. Both straight and bent bars were used (Fig. 3.3) in lengths of 2,046.8mm and 2,246.8mm for the full-scale tests and straight bars of length 1,295mm for the pullout tests. The geometric and material properties of the GFRP bars are given in Table 3.2 and 3.3, respectively, as provided by the manufacturer.



Figure 3.3 GFRP bars

Table 3.2: GFRP Bar Geometric Properties

Bar Size	Bar Diameter (mm)	Cross-sectional Area (mm^2)	Weight (kg/m)
#6	20	285	0.7

Table 3.3: GFRP Bar Material Properties

Property	Value
Tensile Strength (MPa)	> 1000
Modulus of Elasticity (GPa)	> 60
Ultimate Strain	> 1.7%
Shear Strength (MPa)	> 220

3.2.3 Adhesives

Five different adhesives (Adh) were investigated in the pullout tests to determine the best adhesive to use based on application and performance for the full-scale flexural tests. There is a lack of knowledge in regard to the properties and workability of various adhesives pertaining to their use in wood applications, therefore testing was done to evaluate their behaviours. The material properties obtained from each manufacturer are shown in Table 3.4. Adh-3 is used in combination with Adh-2, where Adh-2 is first used as a primer, and then Adh-3 is used at a paste to fill the groove. Adh-5 is divided into two columns as they are the same adhesive but in two different forms, fluid (Adh-5F) and dense (Adh-5D).

Table 3.4: Adhesive Material Properties

Property	Adhesive Type					
	Adh-1	Adh-2	Adh-3	Adh-4	Adh-5F	Adh-5D
Type of Adhesive	Two-part epoxy	Two-part epoxy (saturant)	Two-part epoxy (paste)	Two-part epoxy	Two-part epoxy	Two-part epoxy
Density (kg/L)	2.12	1.15	1.44	1.07	1.45	2.00
Tensile Strength (MPa)	-	36	21	30	30	28
Tensile Modulus (MPa)	-	2,200	4,720	2,000	4,600	6,600
Flexural Strength (MPa)	15	63	51	-	38	46
Flexural Modulus (MPa)	1,400	3,660	6,340	-	2,600	5,400
Compressive Strength (MPa)	100	109	106	-	85	84
Compressive Modulus (MPa)	-	2,990	4,750	-	3,937	3,824

3.3 Description of Test Specimens

3.3.1 Pullout tests

The structural performance of GFRP bar-to-wood bond was evaluated by conducting pullout tests. The glulam specimens had dimensions of 137mm x 241mm x 400mm with an embedment depth of 300mm, which has been shown to be adequate for proper bond length development (Kemmsies, 1999; F. Wang & Xiong, 2019). All the pullout tests were done following a modified protocol based in accordance with the

ASTM D7913/7913M “Standard Test Method for Bond Strength of Fiber-Reinforced Polymer Matrix Composite Bars to Concrete by Pullout Testing” (2020). The major difference between the standard procedure and the modified one is that the bar is located on the surface as opposed to the centre of the specimen. This was done to reflect the conditions when implemented in the full-scale flexural tests.

Three different groove patterns were investigated as shown in Figure 3.4. A standard groove of depth 25.4mm (Fig. 3.4a) and compared to grooves with alternating notches of 38.1mm and 25.4mm (Fig. 3.4b) as well as 44.45mm and 25.4mm (Fig. 3.4c). The alternating groove pattern was investigated as a means to improve the GFRP bar to wood bonds based on the principles of knurling.

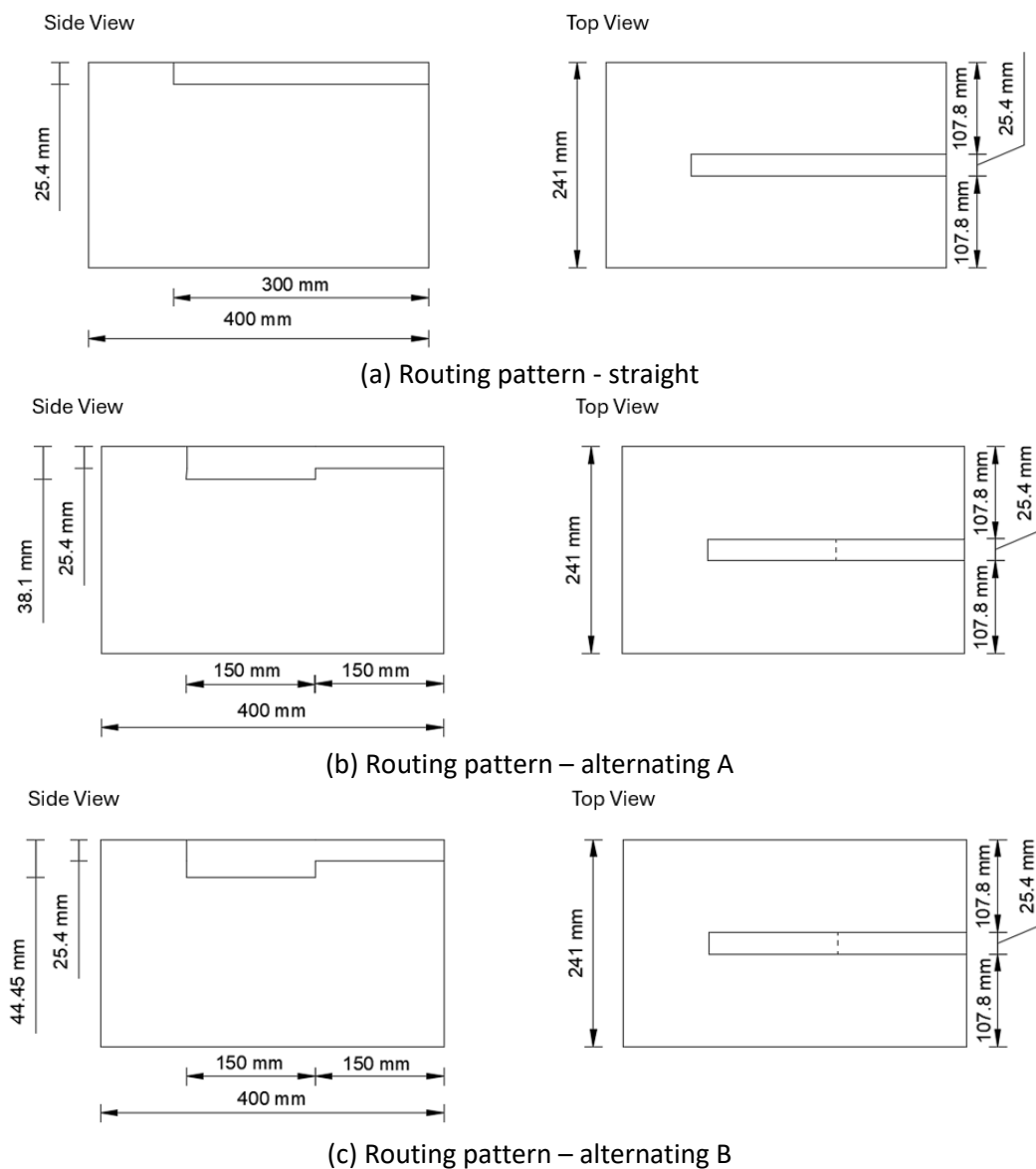


Figure 3.4 Routing patterns

Due to original embedment length of 300 mm, a single alternating groove length was chosen to replicate the groove length in the full-scale flexural tests. Initially, all of the eighteen specimens were routed with a 25.4mm x 25.4mm x 300mm straight groove, then for the specimens with the alternating knurling pattern, the additional depths were routed.

DOM tubes were installed to the end of the FRP bar to prevent crushing of the bar from occurring during the test. The size of the DOM tubes was chosen in accordance to ASTM D7205 “*Standard Test Method for Tensile Properties of Fiber Reinforced Polymer Matrix Composite Bars*” (2021a), and installed onto the FRP bars with an expansive grout.

The epoxies were prepared as per the manufacturer’s instructions and poured into the routed grooves. As only a small amount of adhesive was needed in this application, Adh-1, 4, 5F and 5D were applied through a cartridge (Fig. 3.5b), Adh-1 being a single cartridge and Adh-4, 5F and 5D being two-part cartridges. Adh-2 and Adh-3 were mixed as per the manufacturer’s instructions. To prepare the Adh-2 its components part A and B were mixed at a 2:1 ratio by volume using a mixing drill for 5 minutes. Adh-3 was prepared by first applying a thin layer of Adh-2 to the grooved wood section and the Adh-3 was prepared by combining part A and B at a 2:1 ratio by volume and mixing for 5 minutes with a drill and mixing attachment. For all specimens, the FRP bars were placed in the grooves and left to cure for 48 hours prior to testing (Fig. 3.5c). During the first hours of curing the specimens were checked regularly to ensure proper curing and no air bubbles were present.

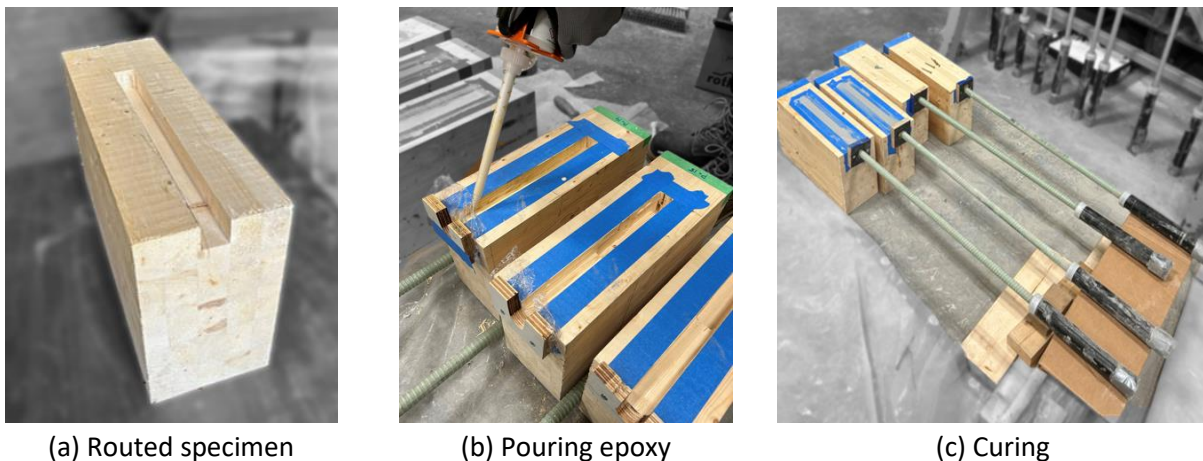


Figure 3.5 Specimen preparation

Table 3.5 presents the test matrix. The adhesives were first tested with a straight groove, and then, based on performance, workability and availability, select adhesives were used to test the effects of an

alternating groove pattern. Specimens P-11 through P-14, first had a thin layer of Adh-2 applied as a primer and then were filled with the Adh-3 paste.

Table 3.5: Pullout Test Matrix

Specimen	Adhesive Type	Routing Pattern
P-01, P-02	Adh-1	Straight
P-03, P-04	Adh-2	Straight
P-05, P-06	Adh-4	Straight
P-07, P-08	Adh-5D	Straight
P-09, P-10	Adh-5F	Straight
P-11, P-12	Adh-2 & Adh-3	Straight
P-13, P-14	Adh-2 & Adh-3	Alternating A
P-15	Adh-5D	Alternating A
P-16	Adh-5F	Alternating A
P-17, P-18	Adh-5F	Alternating B

3.3.2 Flexure tests

A total of ten 137mm x 241mm x 2,800mm glulam beams were reinforced with two GFRP bars along the tension face, resulting in a reinforcement ratio of approximately 1.73%. Spacing requirements are provided in CSA S6 (2019) for heavy timber, as such, the groove depths and widths were chosen to meet the requirements while the clear spacing of the bars was chosen to be 29mm based spacing observed in past research (A. Yusof & A. L. Saleh, 2010; Gentile et al., 2002; Gharaibeh & Doudak, 2023), as the standard suggests that should additional reinforcement be required experimental evidence shall be used to determine the amount (CSA, 2019). The reinforced beam configurations can be divided into two categories. The first included the testing of beams reinforced with straight rebar, including three different reinforcement schemes, them being straight grooves of two different lengths and an alternating groove pattern, and the second included the testing of beams reinforced with bent bars, including two lengths of reinforcement. Figure 3.6 shows the side and cross-sectional views of the beams tested.

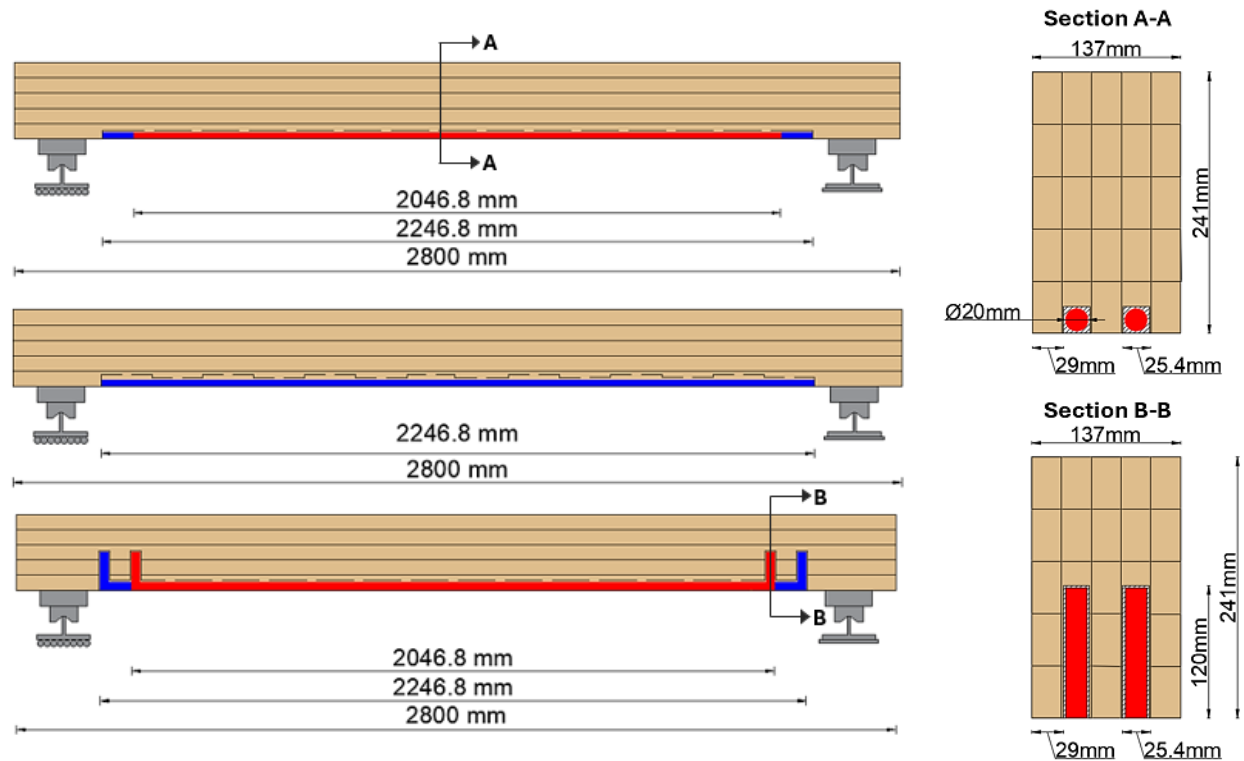


Figure 3.6 Reinforcement schemes for flexural tests of GFRP-reinforced glulam beams

To prepare the beams for the application of the FRP bars, grooves were routed into the beams. Depend on the reinforcement scheme of the specimen, grooves were routed to the required length and specified knurling pattern along the tension surface of the beam. For the beams with bent bars, grooves were drilled vertically at the location of the reinforcement end to accommodate for the 120mm bent portions. The adhesives were mixed and poured according to the manufacturer's instructions.

For the specimens bonded with Adh-3, a two-part process was required. First the wood surface of the groove for the bar to be bonded to was coated in Adh-2 primer and saturant. To prepare the saturant its components part A and B were mixed at a 2:1 ratio by volume using a mixing drill for 5 minutes, as per the manufacturer instructions. After the saturant was applied to the grooved section, the epoxy, Adh-3 epoxy paste and filler, was applied. Adh-3 was prepared by combining part A and B at a 2:1 ratio by volume and mixing for 5 minutes with a drill. A similar process was done for the Adh-5F specimens. After The epoxy was prepared by mixing its parts A and B at a 100:50 ratio by volume and mixing with a drill until homogenous in colour. The routed grooves were filled with the epoxies, then the bars were placed in and covered with more epoxy (Fig. 3.7a). The specimens were then left to cure for 48 hours and checked frequently in the first hours to ensure the specimens were curing properly and no air bubbles were forming (Fig. 3.7b).

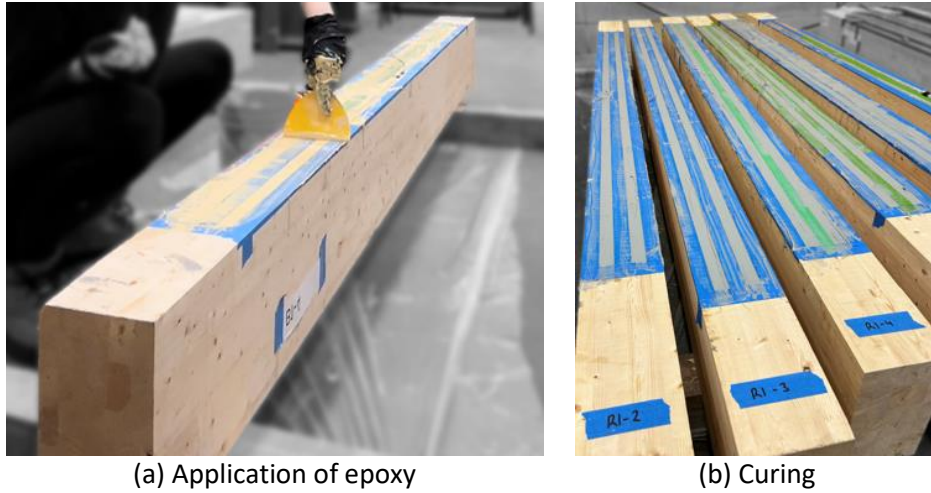
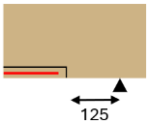
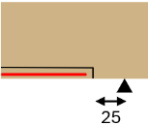
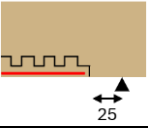
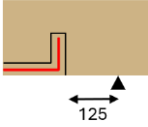
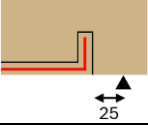


Figure 3.7 Application of epoxy for full-scale GFRP-reinforced glulam beams

Table 3.6 provides a summary of the specimens tested in flexure. The naming convention is **YX-#**; where “Y” refers to the type of specimen (**U** for unreinforced and **R** for reinforced), “X” is the reinforcement scheme, and “#” is the specimen number. The end distance is defined as the distance from the end of reinforcement to the start of the plate at the supports.

Table 3.6: Test Matrix for Full-Scale Flexural Tests of GFRP-Reinforced Glulam Beams

Reinforcement Type	Specimen Nomenclature	Adhesive Type	Groove Pattern	Reinforcement Length (mm)	End Distance (mm)	Diagram
Unreinforced	U-01-U-04	-	-	-	-	-
Straight Bars	R1-1, R1-2	Adh-2/3	Straight	2,046.8	125	
	R1-3, R1-4	Adh-2/3	Straight	2,246.8	25	
	R1-5, R1-6	Adh-2/3, Adh-5F	Alternating A	2,246.8	25	
Bent Bars	R2-1, R2-2	Adh-5F	Straight	2,046.8	125	
	R2-3, R2-4	Ad-5F	Straight	2,246.8	25	

3.4 Test Setup and Instrumentation

3.4.1 Wood components

The uniaxial stress-strain relationships of the wood and the GFRP are required as inputs in the moment-curvature analysis. Sixteen coupon tests were done to determine the wood tension and compression properties. The compression coupons were loaded in compression in a 500 kN hydraulic test frame and the applied force was recorded. The displacements were recorded using a linear variable displacement transducer (LVDT), as shown in Figure 3.8a. The tension coupons were loaded in a 10 kN electromechanical frame and the displacements were determined using an extensometer placed at midspan (Fig. 3.8b).



(a) Compression



(b) Tension

Figure 3.8 Coupon test setups

3.4.2 Pullout specimens

The structural performance of GFRP bar-to-wood bond was evaluated by conducting pullout tests. A 500 kN hydraulic test frame was used to load the specimens under displacement control at 1 mm/min. The frame measured the applied load, and the displacements were measured using a string pot (Fig. 3.9).

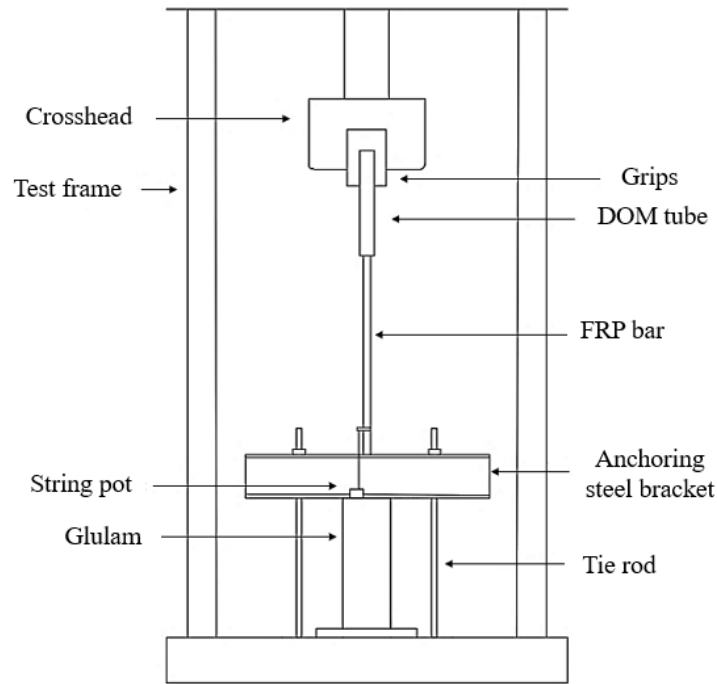


Figure 3.9 Pullout test setup

3.4.3 Flexural tests

The flexural tests were conducted in accordance with ASTM D198 “*Standard Test Methods of Static Test of Lumber in Structural Sizes*” (ASTM, 2022b) where both the unreinforced and reinforced glulam beams were tested to failure under four-point bending with simply supported boundary conditions, as shown in Figure 3.10. Prior to testing, the weight and moisture content of each beam was recorded, and any potentially critical visual imperfections (e.g., knots, finger joints in areas of high stresses, indentations) were noted.

A 500 kN hydraulic load frame with load cell was used to load all the beams for the flexural tests. The beams were loaded under displacement control with loading rates of 5 mm/min, to ensure that failure occurred within the first five to ten minutes (ASTM, 2022b). The applied load was measured using the load cell and a string pot was used to measure the deflections. Additionally, strain gauges were used to measure the strains for the wood and the FRP. For the unreinforced specimens, a total of two strain gauges were used to measure the compressive and tensile strains at mid-span, whereas the reinforced specimens were instrumented with five strain gauges. The strain gauges were strategically located to measure the strains at points of interest such as the midspan of the beam and at the end and of the reinforcement. This allows for a better understanding of the failure mechanism in the reinforced beams.

To determine the effect of the reinforcement configurations on the stiffness, all fourteen glulam beams were tested non-destructively prior to reinforcing where the midspan deflection was measured for comparison of the true versus apparent modulus of elasticity (MOE).

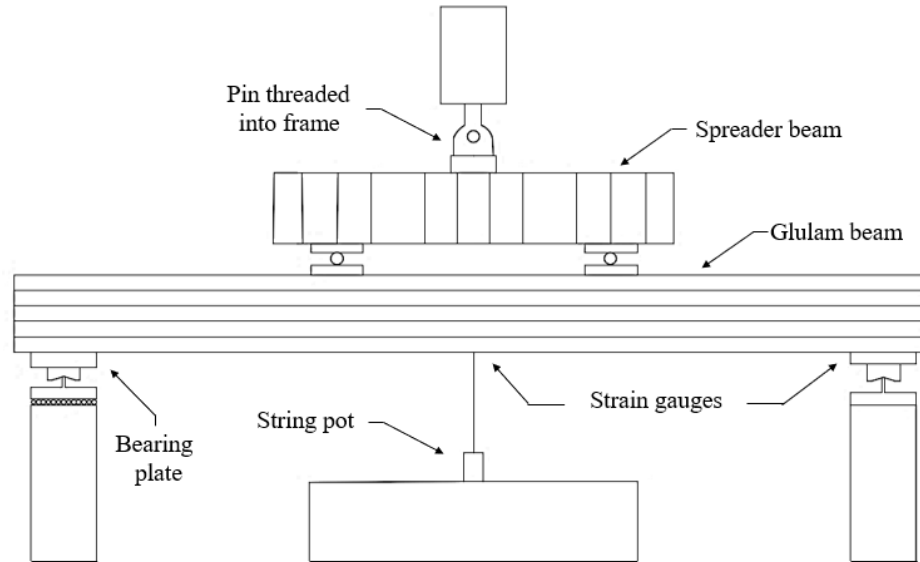


Figure 3.10 Flexural test setup

3.4.4 Digital image correlation

Digital image correlation (DIC) is the process of tracking movement through the analysis of displacements of a pattern along the specimen's surface. DIC can be implemented in civil engineering applications to measure displacements and deformations of structural elements, where previously equipment such as strain gauges, extensometers, and string pots were used. A DIC system can be used in a large array of applications as it is a non-intrusive method of capturing data and can be applied to specimens ranging in size from 10^{-9} meters to 10^2 metres (Correlated Solutions, 2025).

The DIC technique is done by creating a high contrast speckled pattern on an area of interest on the specimen and taking a series of high-speed images during the experimental test. From each image X, Y and Z coordinates are captured and compared against a reference image to measure three-dimensional displacements and then used to calculate strains, surface velocities and curvatures along the entire field. DIC systems have been used for research purposes in structural timber design, including CLT panels (Kleinhenz et al., 2023; Navaratnam et al., 2020) and glulam beams (Timbolmas et al., 2024; Vafadar et al., 2024), to investigate crack initiation and growth, strain distribution, and failure modes.

A DIC system was used to capture deformations along the specimen's surface for both the pullout and flexural tests. To prepare the specimens, a speckled pattern was applied to the area of interest selected for each type of test, that being the entire front face of the specimens for the pullout tests, as the area could be adequately captured in the cameras frame (Fig. 3.11a) and for the flexural tests the area was defined from the midspan to reinforcement end (Fig. 3.11b). Two 5 MP cameras with 8 mm lens were used to capture images of the speckled area during the test at approximately 1.4 m from the glulam and at an angle of 18° between the cameras. The images and data were later analyzed to determine the strains and compared against that of the strain gauges.



(a) Pullout tests



(b) Flexural tests

Figure 3.11 Specimen preparation for digital image correlation

CHAPTER 4 | EXPERIMENTAL RESULTS

4.1 General

This chapter presents the results for the coupon, pullout, and full-scale tests. A total of eight tension and eight compression coupon tests, eighteen pull-out tests, and fourteen full-scale bending tests were completed. Included in this chapter are the results from each test conducted in addition to the description of their failure mechanisms and overall behaviour.

4.2 Component Tests

4.2.1 Wood compression coupons

A summary of results is shown in Table 4.1 for the wood compression coupons. Presented in the table are the compressive yield strength (f_{cy}) and associated strain (ϵ_{cy}), the ultimate compressive strength (f_{cu}) and associated strain (ϵ_{cu}), and the modulus of elasticity (MOE), taken as the slope of the stress-strain curve from 10-40% of the maximum stress, for each specimen.

Table 4.1: Compression Test Results

Specimen Number	f_{cy}^a (MPa)	$\epsilon_{cy}^b \times 10^{-4}$ (mm/mm)	f_{cu}^c (MPa)	$\epsilon_{cu}^d \times 10^{-4}$ (mm/mm)	MOE ^e (MPa)
C-01	42.5	83.3	29.8	405	7,081
C-02	39.4	86.3	20.9	383	7,094
C-03	38.4	71.5	24.4	353	6,758
C-04	40.4	67.4	28.3	388	7,253
C-05	38.4	68.7	26.8	382	6,591
C-06	41.7	69.5	26.5	387	7,320
C-07	34.2	102	21.6	369	6,693
C-08	41.3	75.4	27.6	374	7,376
Average	39.6	78.0	25.7	380	7,021
Std. Dev.	2.5	11.1	3.0	14.4	282.8
COV	0.06	0.14	0.12	0.04	0.04

^a Compression yield strength ^b Compression yield strain ^c Compression ultimate strength

^d Compression ultimate strain ^e Modulus of elasticity

A representative stress-strain curve for the wood compression coupons is shown in Figure 4.1. The stress-strain behaviour is initially linear with some softening occurring prior to attaining the maximum compressive stress (Fig. 4.1), at which point little to no damage is visible. After the maximum load is attained, the wood begins to fail, and the damage is increasingly visible at greater strains as evidenced

with the loss of strength post-peak. The compression coupons were tested under displacement control at a rate of 0.6 mm/min until the stress reached 50% of the maximum stress, the strain exceeded 0.04 mm/mm, or if the specimen could no longer be loaded safely due to torsional forces being applied on the actuator, at which point the test was terminated. The strains for the coupons were obtained by dividing the linear variable displacement transducer (LVDT) displacements by the initial specimen lengths.

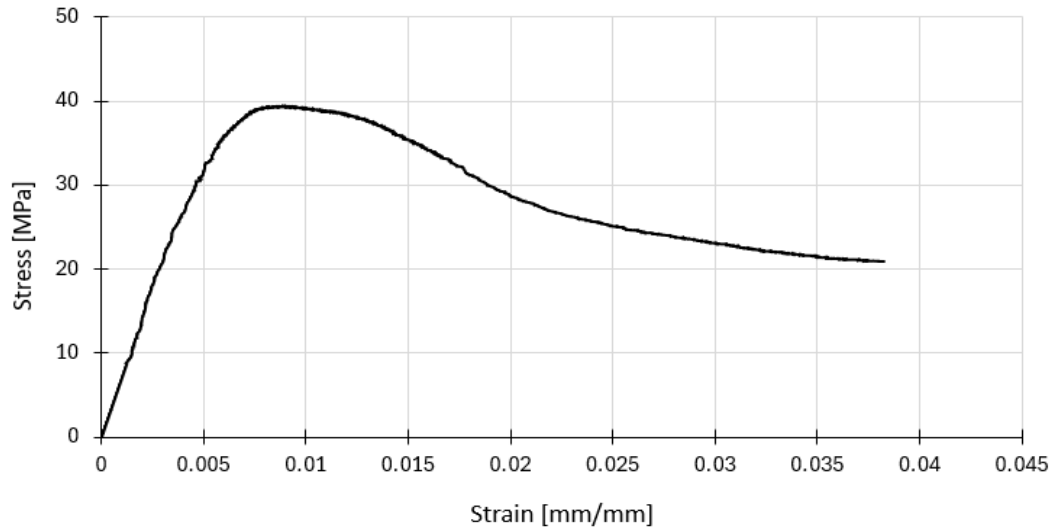


Figure 4.1 Representative stress-strain behaviour

Representative failure modes of the wood compression coupons are shown in Figure 4.2, including crushing, splitting, and shearing. Detailed failure modes and stress-strain curves for all of the compression coupons can be found in Appendix A.



(a) Crushing



(b) Splitting



(c) Shearing

Figure 4.2 Representative compression failures of glulam coupons

4.2.2 Wood tension coupons

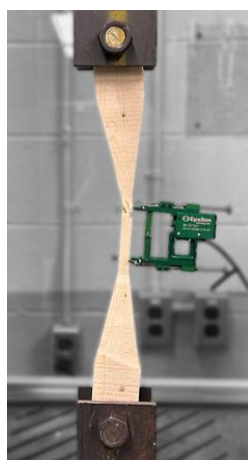
Table 4.2 presents a summary of the eight wood tension coupons tested to failure. Presented in the table are the modulus of elasticity (MOE), taken as the slope of the stress-strain curve from 10-40% of the maximum stress, and the ultimate tensile strength (f_{tu}) and corresponding strain (ϵ_{tu}) for each specimen along with the average of the group.

Table 4.2: Tension Test Results

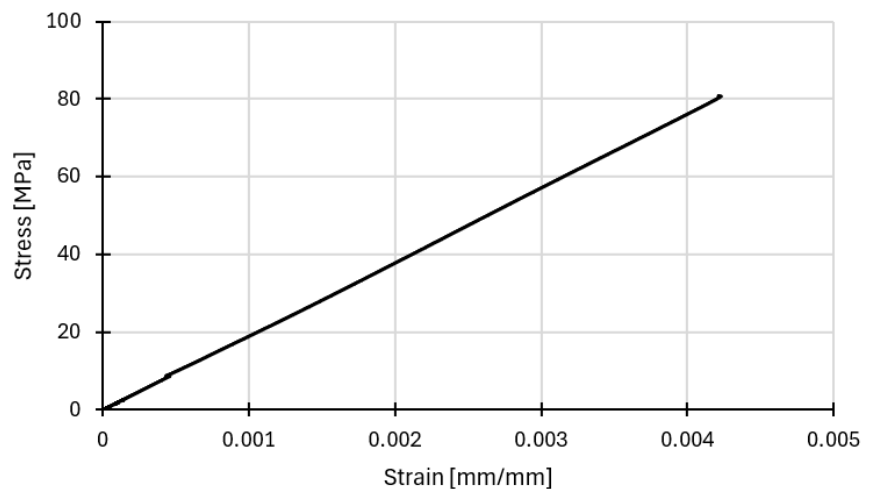
Specimen Number	f_{tu}^a (MPa)	$\epsilon_{tu}^b \times 10^{-4}$ (mm/mm)	MOE ^{c} (MPa)
T-01	75.0	49.3	16,682
T-02	75.4	77.4	10,220
T-03	75.1	75.4	10,067
T-04	81.0	86.5	11,391
T-05	87.2	61.9	13,695
T-06	119.5	82.0	15,158
T-07	119.9	75.3	14,283
T-08	76.7	67.0	14,080
Average	88.7	71.8	13,197
Std. Dev.	18.3	112	2239
COV	0.21	0.16	0.17

^a Tensile ultimate strength ^b Tensile ultimate strain ^c Tensile modulus of elasticity

A representative failure mode and corresponding stress-strain diagram for the wood tension coupon tests are shown in Figure 4.3. Failure of the tension coupons occurred within the reduced midspan region (Fig. 4.3a), and the wood was observed to behave linearly until failure was attained (Fig. 4.3b). The failures and stress-strain curves for all the tension coupons can be found in Appendix B.



(a) Failure



(b) Stress-strain behaviour

Figure 4.3 Representative tension coupon

4.2.3 Pullout tests

A summary of the results for the eighteen pullout tests is presented in Table 4.3 along with representative force-displacement curves in Figure 4.4. Presented in the table is the failure load (P_{max}), the displacement at failure (Δ_{max}), the tensile stiffnesses (K_1 and K_2), where K_1 was calculated as the slope of load-displacement curve from 10-40% of the maximum load, and K_2 was defined as P_{max} divided by Δ_{max} to provide consistent results between all the curves as some specimens showed high initial stiffness.

Table 4.3: Pullout Test Results

Specimen	Adhesive Type	Routing Pattern	P_{max}^a (kN)	Δ_{max}^b (mm)	K_1^c (kN/mm)	K_2^d (kN/mm)
P-01	Adh-1	Straight	21.9	0.6	49.1	36.8
P-02			13.4	0.3	46.1	52.2
P-03	Adh-2	Straight	62.7	1.7	51.6	36.3
P-04			72.1	2.0	52.6	35.8
P-05	Adh-4	Straight	85.6	2.2	41.0	39.1
P-06			87.3	2.3	45.8	37.3
P-07	Adh-5D	Straight	62.1	1.5	58.7	41.6
P-08			57.1	1.4	59.53	38.4
P-09	Adh-5F	Straight	81.3	2.5	39.9	32.1
P-10			88.3	2.4	48.5	37.0
P-11	Adh-2, Adh-3	Straight	54.1	1.1	38.2	48.5
P-12			56.3	1.9	49.7	30.2
P-13	Adh-2, Adh-3	Alternating A	57.8	1.7	59.2	33.4
P-14			63.3	1.7	49.5	34.9
P-15	Adh-5D	Alternating A	53.9	1.1	48.4	37.1
P-16	Adh-5F		96.9	2.9	48.9	33.1
P-17	Adh-5F	Alternating B	78.1	2.6	47.8	29.6
P-18			81.2	3.3	30.6	23.9

^a Maximum Load

^b Deflection at maximum load

^c Tensile stiffness (10-40%)

^d Tensile stiffness (P_{max}/Δ_{max})

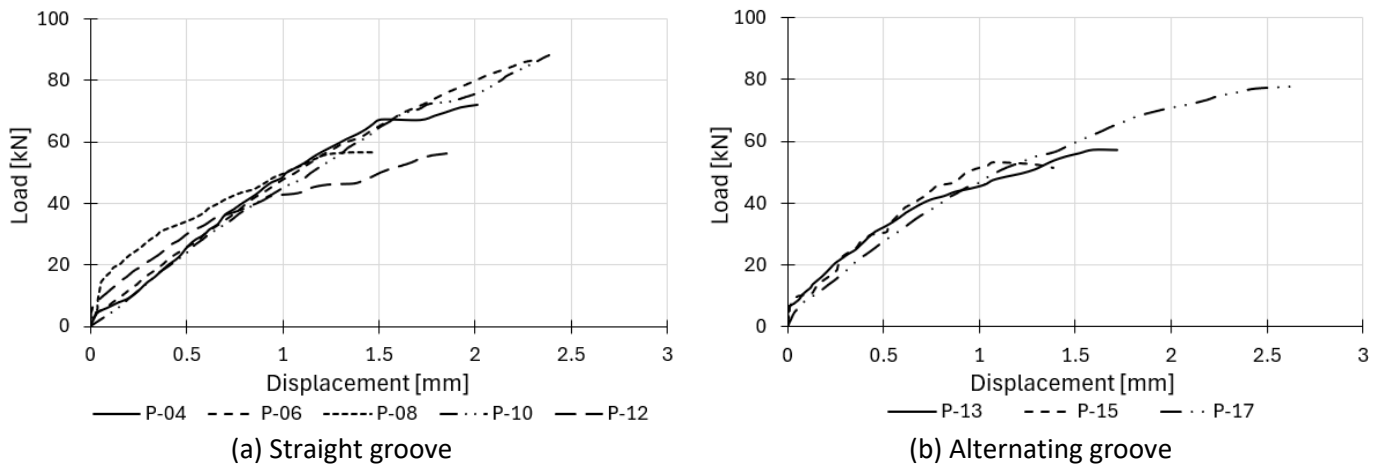


Figure 4.4 Representative force-displacement curves

Figure 4.5 shows representative failure modes observed during the testing. Most of the failures observed were either shear along the bar (Fig. 4.5a) and/or splitting of the wood (Fig. 4.5b). In one case, a failure in the bond between the wood and adhesive was seen (Fig. 4.5c).

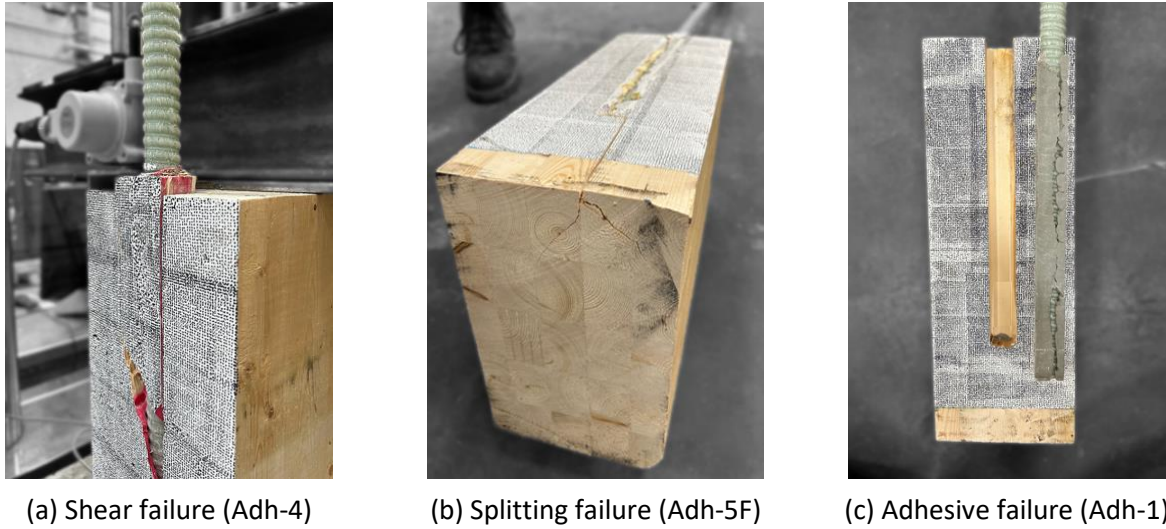


Figure 4.5 Representative failure modes of pullout specimens

Figure 4.6 shows the longitudinal strain distribution captured by the digital image correlation (DIC) system of the specimens shown in Figure 4.5 moments prior to failure. Depictions of failure mode, force-displacement curve, and strain distribution prior to failure for each specimen can be found in Appendix C.

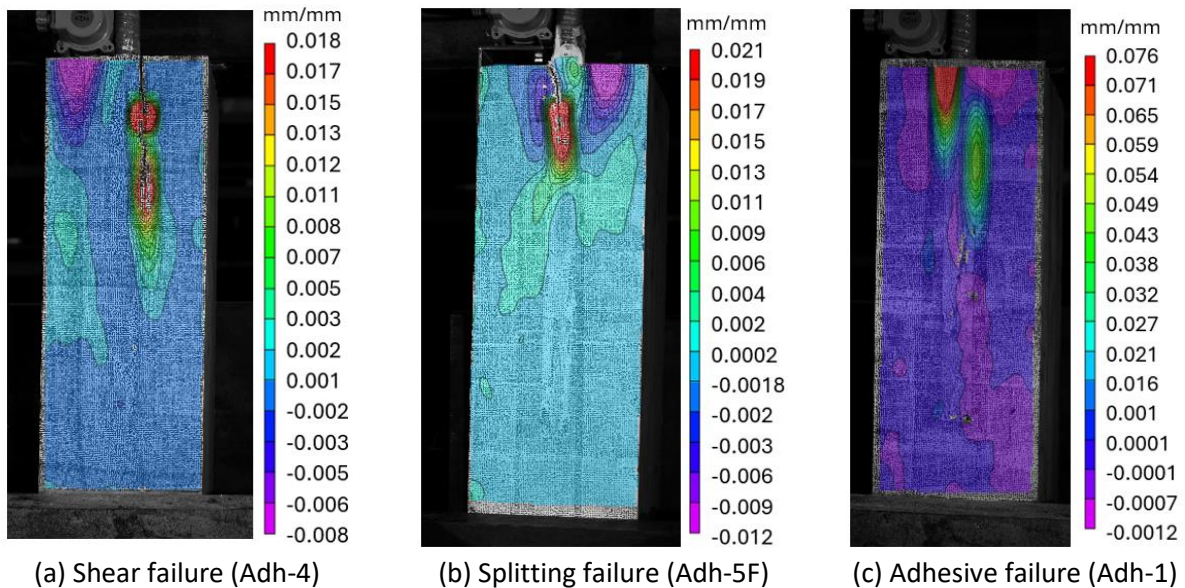


Figure 4.6 Representative strain distribution of pullout specimens captured with the DIC

4.3 Bending Test Results

4.3.1 Overview and general terminology

The following sections describe the observed behaviour and failure modes of the full-scale unreinforced and reinforced glulam beams. For both the unreinforced and reinforced beams, a summary of key properties describing the global behaviour along with resistance curves (i.e., load-displacement) while representative failure modes, load- and strain-displacement relationships, and strain contours obtained with the DIC are presented. The failure progression for all the unreinforced and reinforced beams are included in Appendix D while force- and strain-displacement curves for the beams are included in Appendix E. Finally, compressive and tensile strain-time histories extracted from the DIC are included in Appendix F.

An idealized resistance curve (Fig. 4.7) shows the bending member behaving elastically up to a maximum load ($P_{\max}$) and corresponding displacement ($\Delta_{\max}$), referred to as the peak or maximum resistance. Once the peak resistance is attained the initial failure occurs, and an increasing or plateauing post-peak behaviour is observed.

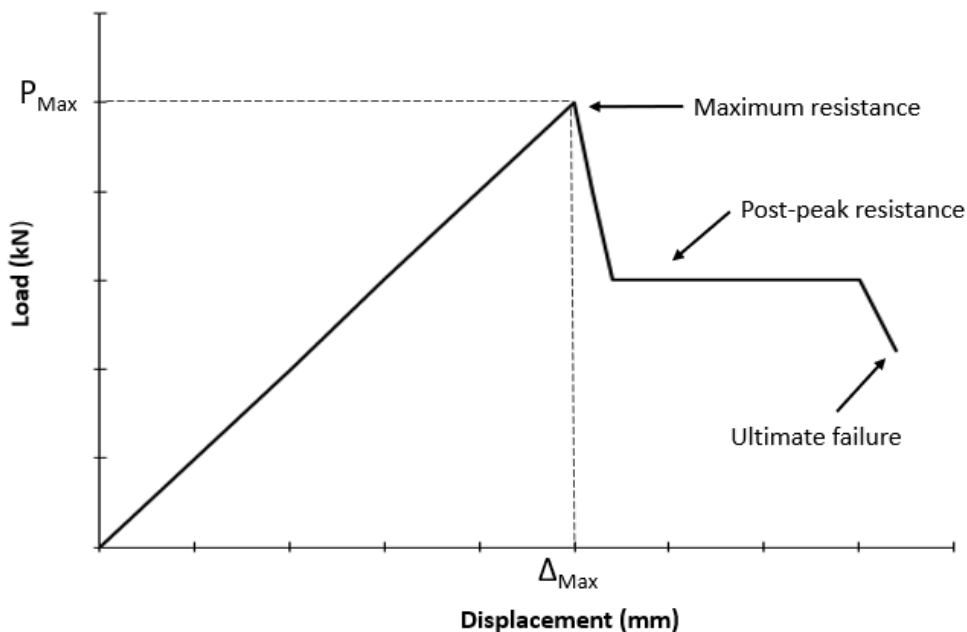


Figure 4.7 Idealized resistance curve

4.3.2 Unreinforced specimens

The summary of the flexural results for the four unreinforced glulam specimens tested destructively is presented in Table 4.4. Key information is presented in the table including, P_{max} the maximum load, Δ_{max} the corresponding midspan deflection as measured by the string pot, $\epsilon_{t, max}$ the largest recorded mid-span tensile strain (i.e., gauge), $\epsilon_{c, max}$ the largest recorded mid-span compressive strain (i.e., gauge), $\epsilon_{t, max-DIC}$ and $\epsilon_{c, max-DIC}$ the largest recorded tensile and compressive strains as measured using the DIC system, modulus of rupture (MOR), bending stiffness (K) defined as the slope of load-displacement curve from 10-40% of the maximum load, and modulus of elasticity (MOE).

Table 4.4: Summary of Flexural Test Results for Unreinforced Beams

Specimen Number	P_{max}^a (kN)	Δ_{max}^b (mm)	$\epsilon_{t, max}^c \times 10^{-3}$ (mm/mm)	$\epsilon_{c, max}^d \times 10^{-3}$ (mm/mm)	$\epsilon_{t, max-DIC}^e \times 10^{-3}$ (mm/mm)	$\epsilon_{c, max-DIC}^f \times 10^{-3}$ (mm/mm)	MOR ^g (MPa)	K ^h (N/mm)	MOE ⁱ (MPa)
U-01	189.2	30.3	3.1	-5.2	4.7	-4.6	59.4	6,791	11,783
U-02	167.3	28.9	1.7	-3.7	4.9	-4.2	52.6	6,213	10,781
U-03	144.0	26.2	3.5	-5.2	4.4	-5.1	45.3	5,731	9,945
U-04	162.4	30.0	5.5	-9.6	5.5	-6.6	51.0	5,973	10,365
Average	165.7	28.9	3.5	-5.9	4.9	-5.1	52.1	6,177	10,718
Std. Dev.	16.1	1.6	1.4	2.2	0.4	0.9	5.1	393	628
COV	0.1	0.06	0.4	-0.38	0.08	0.17	0.1	0.06	0.06

^a Maximum Load

^b Deflection at maximum load

^c Maximum tensile strain

^d Maximum compressive strain

^e Maximum tensile strain (DIC)

^f Maximum compressive strain (DIC)

^g Modulus of rupture

^h Bending stiffness

ⁱ Modulus of elasticity

The MOR and MOE presented in Table 4.4 are “apparent material properties” meaning that they are based on the global response of the beams and were obtained using Equations 4.1 and 4.2, respectively, which are representative of the beams’ loading scenario.

$$MOR = \frac{P_{max} * L}{6 * S} \quad [4.1]$$

$$MOE = \frac{23 * K * L^3}{1296 * I} \quad [4.2]$$

Where P_{max} is the maximum applied load, L is the clear span, S is the section modulus, K is the bending stiffness defined as the slope of load-displacement curve from 10-40% of the maximum load, and I is the moment of inertia.

The resistance curves for the four unreinforced beams are shown in Figure 4.8 where the unreinforced specimens are observed to behave linearly up to the maximum load followed by a drop in resistance that is associated with the initial failure. Generally, unreinforced glulam specimens exhibit little to no post-peak resistance as is generally evidenced in Figure 4.8. The drop in resistance post-peak (i.e., after the initial failure) corresponds to losses of strength in the order of 25-76% of the maximum load applied for specimens U-01, U-02, and U-04.

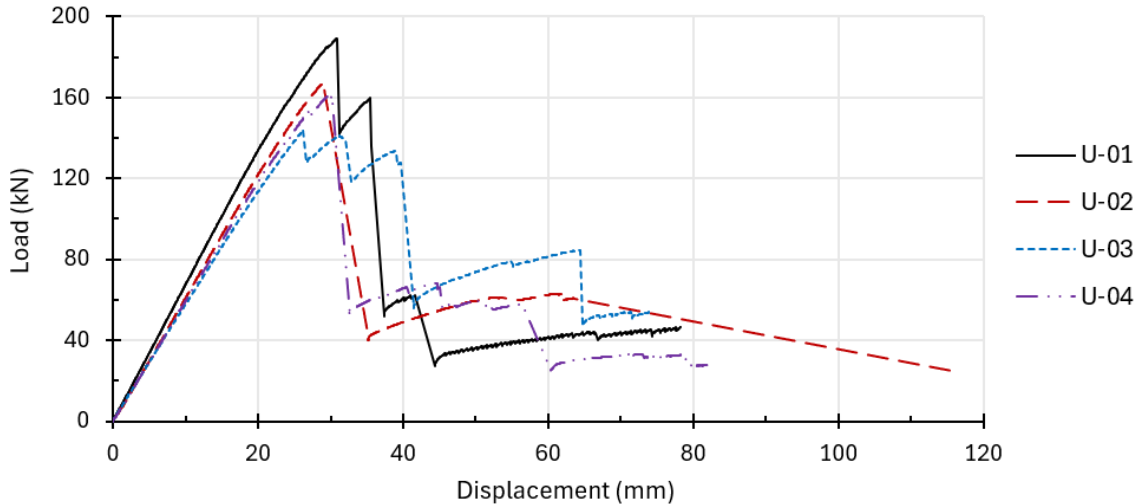


Figure 4.8 Resistance curves for unreinforced beams

The representative failure modes observed for the four unreinforced beams included splintering, and/or shear. Figure 4.9 shows splintering tension failure for specimens U-02 and U-04 where the failures initiated in the maximum moment region and continued to propagate via the path of least resistance in the specimen.

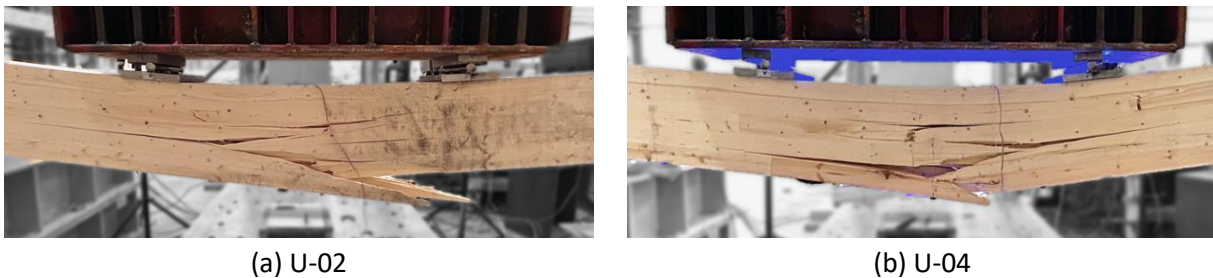


Figure 4.9 Splintering tension failures of unreinforced beams

Figure 4.10 shows the failure modes for specimens U-01 and U-03 for which splintering and shear failures were observed. For specimen U-01, a combination of splintering tension and shear failure was observed as shown in Figures 4.10a and 4.10b. As soon as the splintering occurred at a knot (Fig. 4.10a), a longitudinal shear failure initiated at the splintering location and propagated along the length of the beam

(Fig. 4.10b). In the case of specimen U-03, the initial failure mode observed was shear along the mid-height of the specimen (Fig. 4.10d) with splintering occurring upon further loading (Fig. 4.10c). For both beams, the failure was observed to propagate through the path of least resistance along the length of the specimen while moving upwards in the cross-section depth upon further loading.

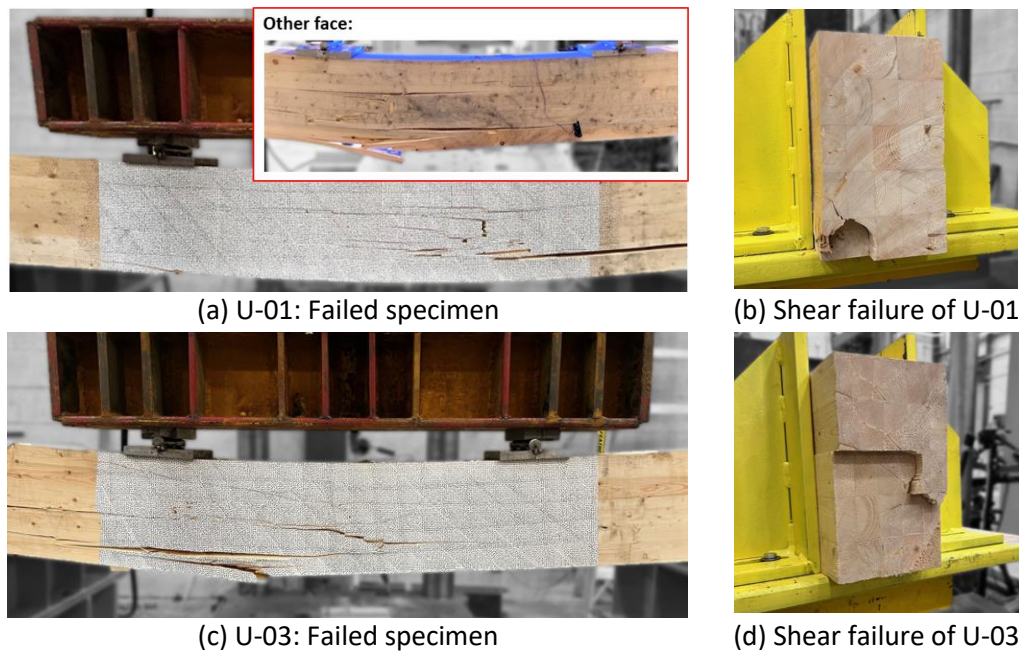


Figure 4.10 Unreinforced specimens with shear failures

Representative load- and strain-displacement relationships for two unreinforced specimens are shown in Figure 4.11. While unreinforced glulam in flexure generally exhibits little post-peak resistance (i.e., Fig. 4.11a), some level of post-peak resistance prior to the major loss of strength is observed for the specimen shown in Figure 4.11b. In Figure 4.11a, the maximum tensile strain coincided with the initial splintering tension failure, after which the strain gauge failed due to being close to the failure location (i.e., Fig. 4.9b). It can also be seen that upon further loading that the compression strain continued to increase with jumps coinciding with subsequent layers of wood failing with no significant post-peak resistance. In Figure 4.11b, the maximum tensile strain coincided with the maximum load; however, due to the location of the initial failure for U-03 being halfway between the midspan and loading point (i.e., Fig. 4.10c), the tensile strain gauge continued to record reading. Similarly, the compression strain continued to increase with jumps coinciding with subsequent layers of wood failing. The unusual levels of post-peak resistance displayed for U-03 (Figs. 4.8 and 4.11b) is attributed to the secondary failure being shear which then resulted in one composite section to behave as two sections sliding relative to another (Fig. 4.10d) prior to failing both sections (i.e., larger drop in resistance).

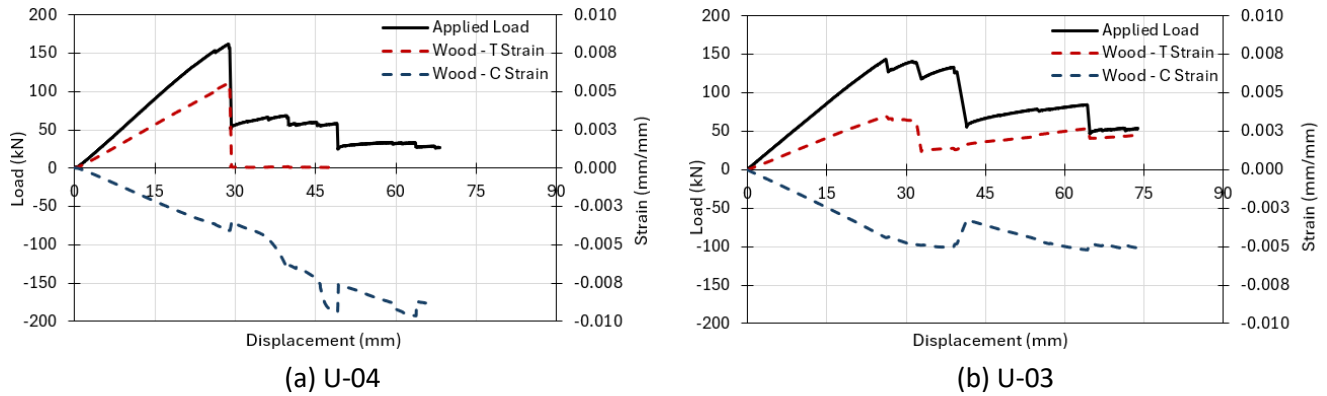
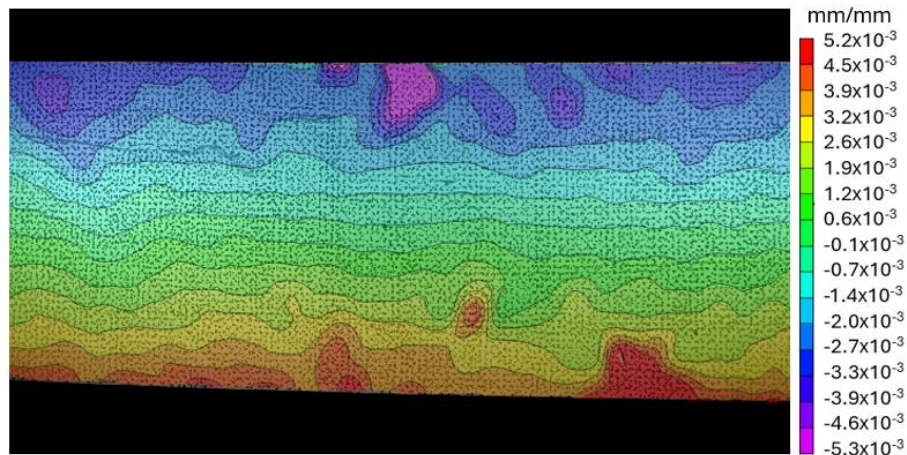
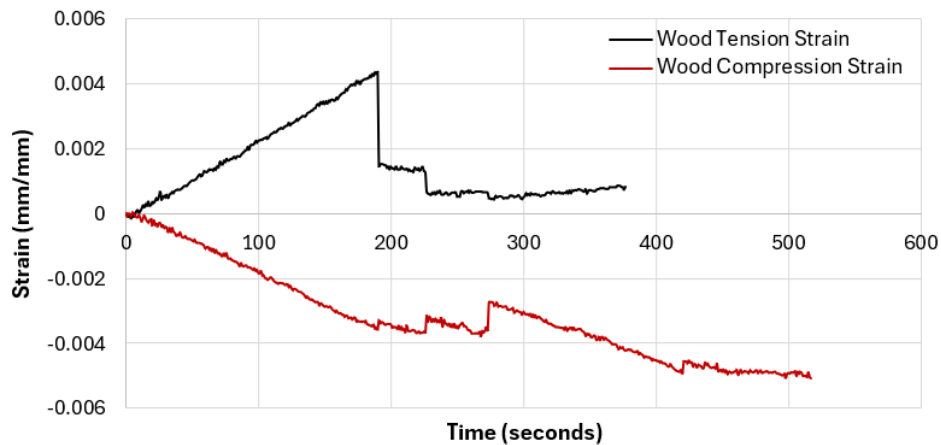


Figure 4.11 Representative stress-strain curves for unreinforced beams

While the strain gauges provide accurate strain readings, they capture the strain at a very small and discrete location. As stated in Chapter 3, a digital image correlation (DIC) system was used to capture and understand the global behaviour of the specimens. Figure 4.12a shows specimen U-03 at the maximum tensile longitudinal strain.



(a) Strain contours prior to failure



(b) Strain-time history

Figure 4.12 Strain data extracted from DIC for specimen U-03

4.3.3 Reinforced specimens with straight bars

A summary of results from the flexural tests for the straight bar reinforced beams are presented in Table 4.5. Key parameters describing the global behaviour are presented in the table, namely, $P_{\max}$ as the maximum load, $\Delta_{\max}$ as the corresponding midspan deflection measured by the string pot, $\epsilon_{t, \max}$ the largest recorded mid-span tensile strain (i.e., gauge), $\epsilon_{c, \max}$ as the largest recorded mid-span compressive strain (i.e., gauge), $\epsilon_{t, \max-DIC}$ and $\epsilon_{c, \max-DIC}$ as the largest recorded tensile and compressive strains using the DIC, and ϵ_{FRP} as the maximum tensile strain of the FRP. The bending stiffnesses (K and K_u) were calculated as the slope of load-displacement curve from 10-40% of the maximum load for the reinforced specimen, where K is the bending stiffness for the reinforced specimens and K_u is the bending stiffness of the specimen prior to reinforcement, determined during stress testing.

Table 4.5: Summary of Flexural Test Results for Straight Bar Reinforced Beams

Specimen Number	$P_{\max}^a$ (kN)	$\Delta_{\max}^b$ (mm)	$\epsilon_{t, \max}^c$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c, \max}^d$ $\times 10^{-3}$ (mm/mm)	ϵ_{FRP}^e $\times 10^{-3}$ (mm/mm)	$\epsilon_{t, \max-DIC}^f$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c, \max-DIC}^g$ $\times 10^{-3}$ (mm/mm)	K^h (N/mm)	K_u^i (N/mm)
R1-1	218.3	34.9	5.36	-3.81	7.02	4.98	-5.08	7,482	6,210
R1-2	203.4	28.2	5.71	-7.13	5.45	3.80	-5.84	7,145	6,057
Average	210.8	31.6	5.54	-5.47	6.24	4.39	-5.46	7,314	6,134
R1-3	173.1	26.2	2.84	-5.41	6.77	3.89	-3.97	7,076	5,877
R1-4	229.2	43.6	4.42	-4.73	5.99	3.32	-8.21	6,987	5,763
Average	201.2	34.9	3.63	-5.07	6.38	3.61	-6.09	7,032	5,820
R1-5	205.9	34.3	3.94	-4.27	6.83	3.09	-6.71	6,976	5,468
R1-6	225.3	37.2	5.05	-14.8	7.12	5.09	-10.8	7,232	5,989
Average	215.6	35.8	4.49	-9.54	6.98	4.09	-8.76	7,104	5,729

^a Maximum Load

^d Maximum compressive strain of wood

^g Maximum compressive strain of wood (DIC)

^b Deflection at maximum load

^e Maximum tensile strain of FRP

^h Bending stiffness

^c Maximum tensile strain of wood

^f Maximum tensile strain of wood (DIC)

ⁱ Bending stiffness of unreinforced beam

The resistance curves corresponding to the beams reinforced with straight bar are shown in Figure 4.13. It can be seen that the beams behave linearly up to initial failure, followed by a significant drop in resistance, which is observed for all specimens except for R1-3. The initial failure modes observed for reinforcement configurations R1-1 through R1-6 consisted of splintering (i.e., R1-2, R1-6) and simple (i.e., R1-5) tension as well as longitudinal shear (i.e., R1-1, R1-3, R1-4).

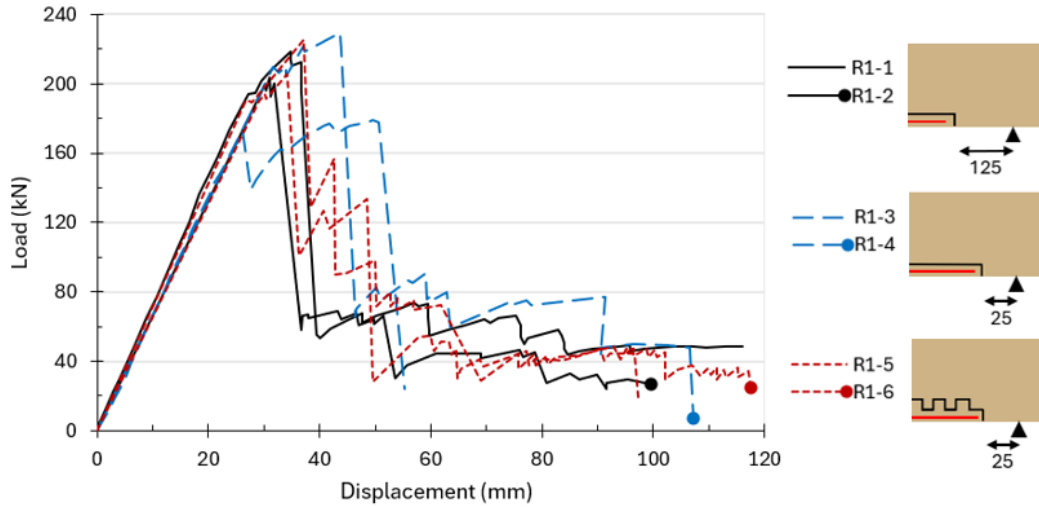


Figure 4.13 Load-displacement curves for straight bar reinforced beams

Figure 4.14 depicts the splintering and tension failure modes observed in specimens reinforced with straight bars. For R1-2 (Figs. 4.14a-b) and R1-6 (Figs. 4.14c-d), both failures initiated from a defect in an area of high bending stress and continued to propagate through the path of least resistance with further loading (displacement control), causing the surface tension laminates to be pushed outwards. For specimen R1-5 the failure was one governed by simple tension in the maximum moment region (Figs. 4.14e-f). The GFRP bars were not observed to detach from the beams.

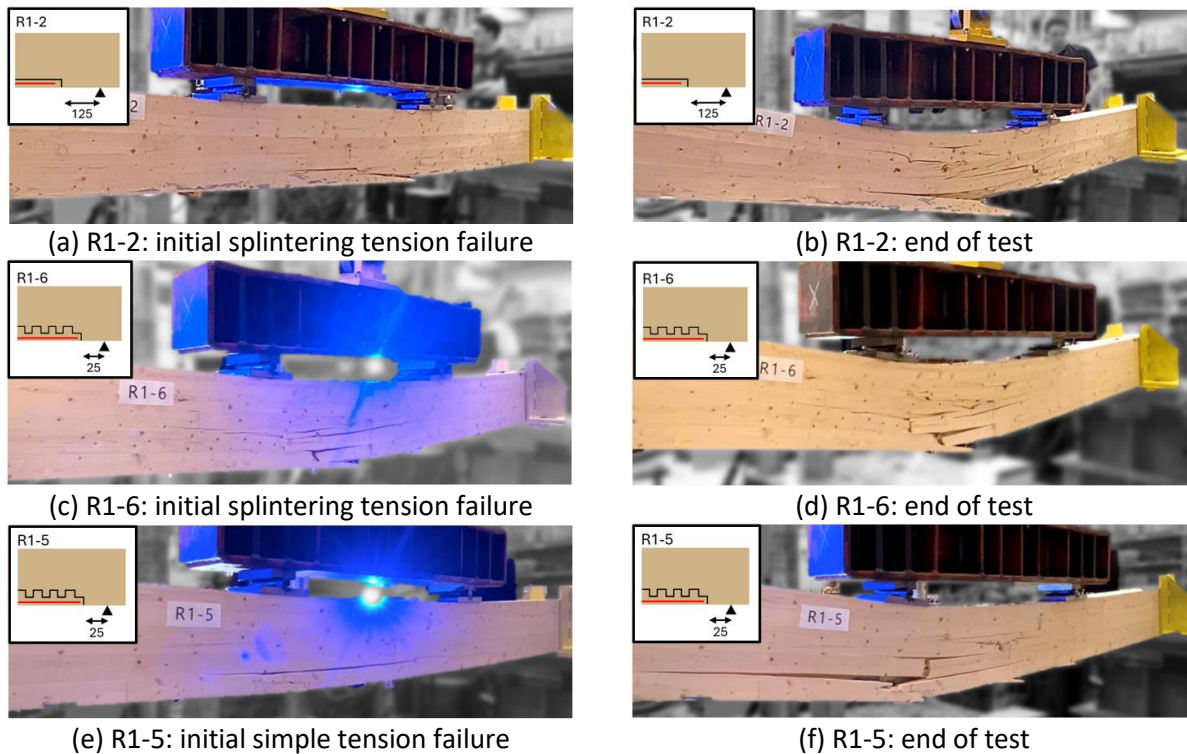


Figure 4.14 Flexural tension failures for beams with straight bars

The addition of GFRP bars contributed to a shift from initial failure modes of splintering and simple tension in unreinforced beams to one dominated by longitudinal shear as shown in Figure 4.15 for specimens R1-1, R1-3, and R1-4. Specimen R1-1 had some initial cracking (i.e., initiation of splintering failure) prior to reaching its maximum resistance (Fig. 4.15a) after which a longitudinal shear failure occurred (Fig. 4.15b). Further loading led to an amplification of the initial splintering failure along with the failure continuing to progress through the specimen. Specimen R1-3 had an initial shear failure (Fig. 4.15c) causing the resistance to decrease by 20%, the test continued reaching a secondary peak resistance prior to a simple tension failure initiating from a defect in an area of high bending stress (Fig. 4.15d). Specimen R1-4 experienced initial cracking and then a longitudinal shear failure (Fig. 4.15e). As the test progressed, the cracks continued to propagate (i.e., compression wrinkling of wood fibres in Fig. 4.15f) leading to one of the FRP bars to fall out of the beam at one end that is accompanied with a simple tension failure on the opposing face (Fig. 4.15f). The bar did not fall out due to bond failure, but rather a wood failure, as the FRP bar came out encased in a wood block.

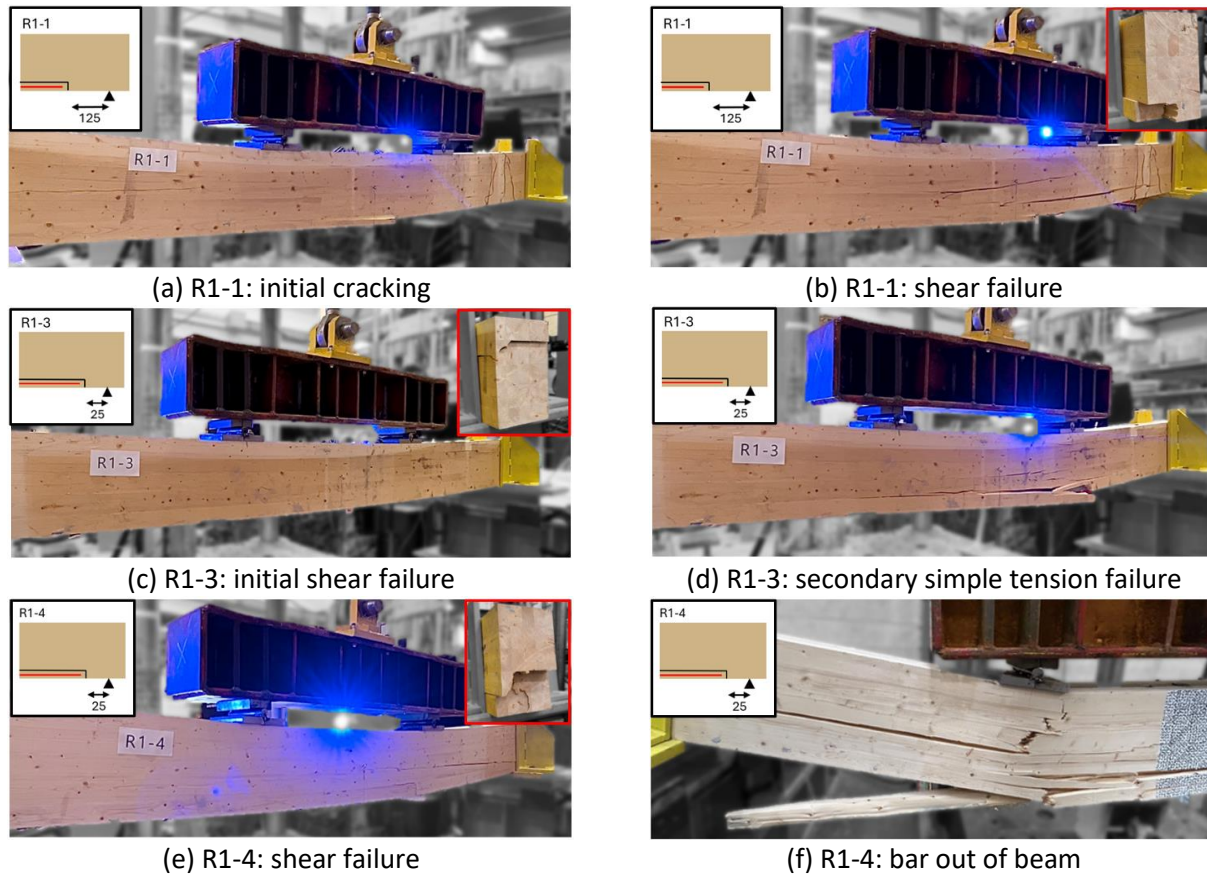


Figure 4.15 Shear failures for beams with straight bars

Representative load- and strain-displacement relationships for the straight bar reinforced specimens are shown in Figure 4.16. In Figure 4.16a, the maximum wood tensile strain coincided with the initial splintering tension failure, after which the strain gauge failed due to being close to the failure location (i.e., Fig. 4.14a). The tensile strain of the FRP bar followed similar behaviour with the maximum strain corresponding with the maximum load. It can also be seen that upon further loading that the compression strain continued to increase with jumps coinciding with subsequent layers of wood failing with no significant post-peak resistance. In Figure 4.16b it is observed that the maximum wood tensile strain occurs prior to maximum resistance. After initial failure the strain gauges fail due to their location in proximity to the failure (Fig. 14e).

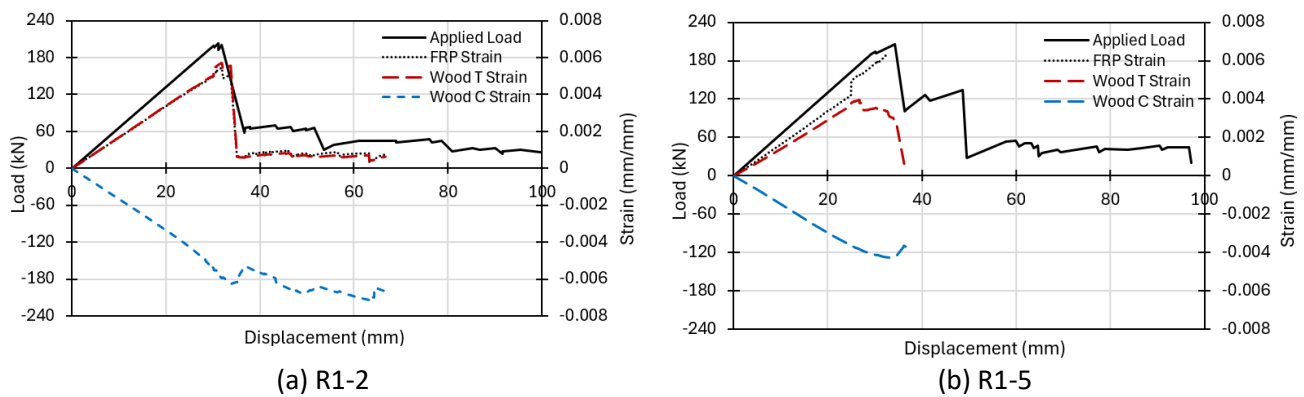
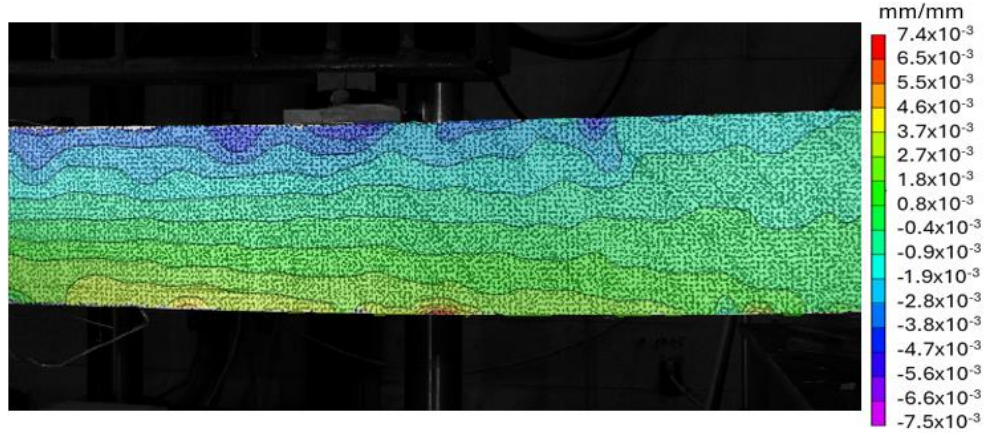
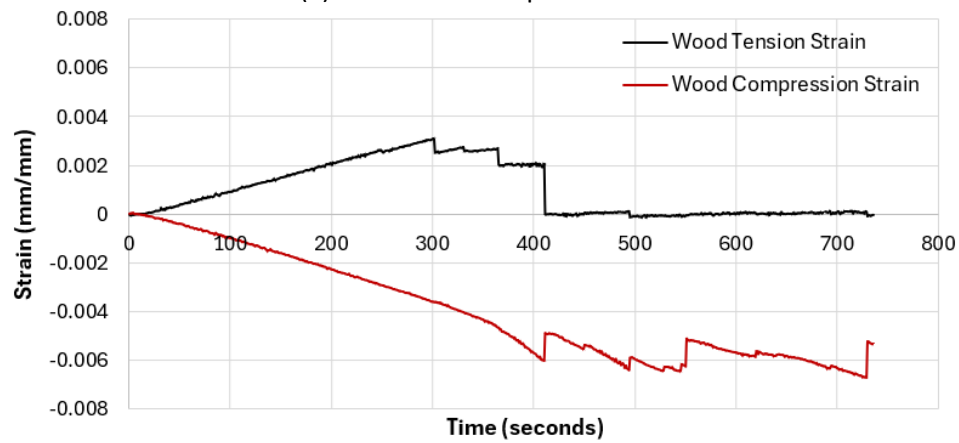


Figure 4.16 Representative stress-strain curves for straight bar reinforced beams

The longitudinal strain behaviour as captured by the DIC system is presented in Figure 4.17. Figure 4.17a shows specimen R1-5 at the maximum tensile longitudinal strain, occurring prior to failure.



(a) Strain contours prior to failure



(b) Strain-time history

Figure 4.17 Longitudinal strain data extracted from DIC for specimen R1-5

4.3.4 Reinforced specimens with bent bars

A summary of results from the flexural tests for the bent bar reinforced beams is presented in Table 4.6. Included in the table is, $P_{\max}$ the maximum load, $\Delta_{\max}$ the corresponding midspan deflection as measured by the string pot, $\epsilon_{t, \max}$ the largest recorded mid-span tensile strain (i.e., gauge), $\epsilon_{c, \max}$ the largest recorded mid-span compressive strain (i.e., gauge), $\epsilon_{t, \max-DIC}$ and $\epsilon_{c, \max-DIC}$ the largest recorded tensile and compressive strains using the DIC, and ϵ_{FRP} the maximum tensile strain of the FRP. The bending stiffnesses (K and K_u) were calculated as the slope of load-displacement curve from 10-40% of the maximum load for the reinforced specimen, where K is the bending stiffness for the reinforced specimens and K_u is the bending stiffness of the specimen prior to reinforcement, determined during stress testing.

Table 4.6: Summary of Flexural Test Results for Bent Bar Reinforced Beams

Specimen Number	P_{max}^a (kN)	Δ_{max}^b (mm)	$\epsilon_{t,max}^c$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c,max}^d$ $\times 10^{-3}$ (mm/mm)	ϵ_{FRP}^e $\times 10^{-3}$ (mm/mm)	$\epsilon_{t,max-DIC}^f$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c,max-DIC}^g$ $\times 10^{-3}$ (mm/mm)	K^h (N/mm)	K_u^i (N/mm)
R2-1	194.0	34.6	5.47	-4.04	5.65	3.99	-6.79	6,816	5,673
R2-2	191.0	25.7	8.87	-5.27	-	7.27	-5.74	7,324	6,348
Average	192.5	30.2	7.17	-4.66	5.65	5.63	-6.27	7,070	6,011
R2-3	207.6	35.7	5.73	-3.25	11.8	4.68	-9.55	6,832	5,900
R2-4	216.9	36.0	3.73	-12.7	11.3	4.25	-14.0	7,178	6,067
Average	212.3	35.9	4.73	-7.98	11.6	4.47	-11.8	7,005	5,984

^a Maximum Load

^d Maximum compressive strain of wood

^g Maximum compressive strain of wood (DIC)

^b Deflection at maximum load

^e Maximum tensile strain of FRP

^h Bending stiffness

^c Maximum tensile strain of wood

^f Maximum tensile strain of wood (DIC)

ⁱ Bending stiffness of unreinforced beam

The resistance curves corresponding to the bent bar beams are shown in Figure 4.18. It can be seen that the beams behave linearly up to initial failure, followed by a significant drop in resistance. The difference in behaviour of the R2-2 curve can be attributed to the beam having an initial failure mode of shear. An improvement in post-peak behaviour is seen in comparison to the straight bar reinforced beams. The initial failure modes observed for reinforcement configurations R2-1 through R2-4 consisted of splintering (i.e., R2-1, R2-3) and longitudinal shear (i.e., R2-2) as well as cross-grain tension (i.e., R2-4).

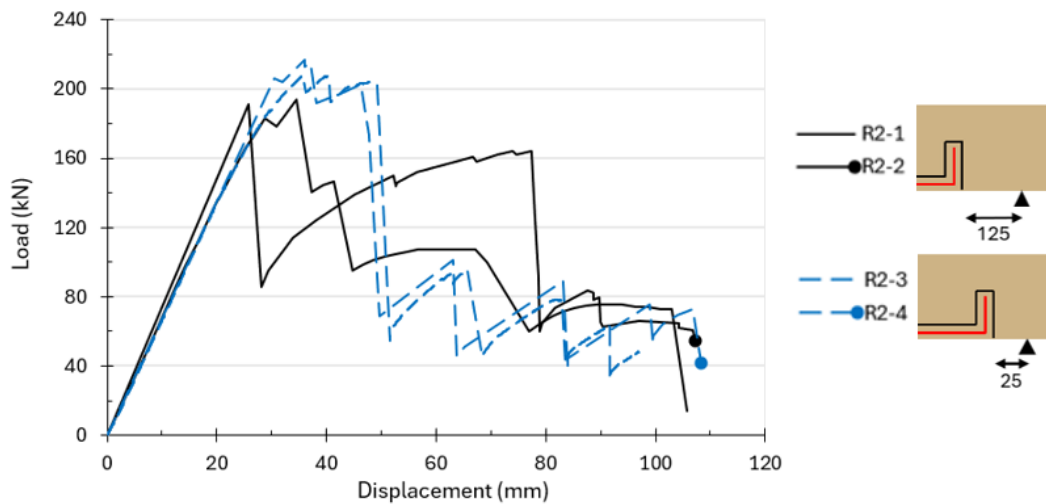


Figure 4.18 Load-displacement curves for bent bar reinforced beams

When tested under four-point bending the majority of the bent bar reinforced beams failed tension, with one specimen experiencing a failure governed by horizontal shear (i.e., R2-2). Specimens R2-1 and R2-3 experienced splintering tension failures (Fig. 4.19). The failures initiated from finger joints on the tension faces of the beams. After further loading and propagating of failures, the bars in specimen R2-1 came out of the beams, as shown in Figure 4.19c. Both bars fell out at one end of the beam, the bars detached encased in a wood cover, suggesting the failure was not due to the bond strength. Specimen R2-4 was the

only beam to experience a brittle cross grain tension failure, (Fig. 4.19e). A large knot was found on the tension face of beam and is responsible for the sudden failure mode of this specimen. The bars remained in the beam for the duration of the test, however experienced some visible deflection.

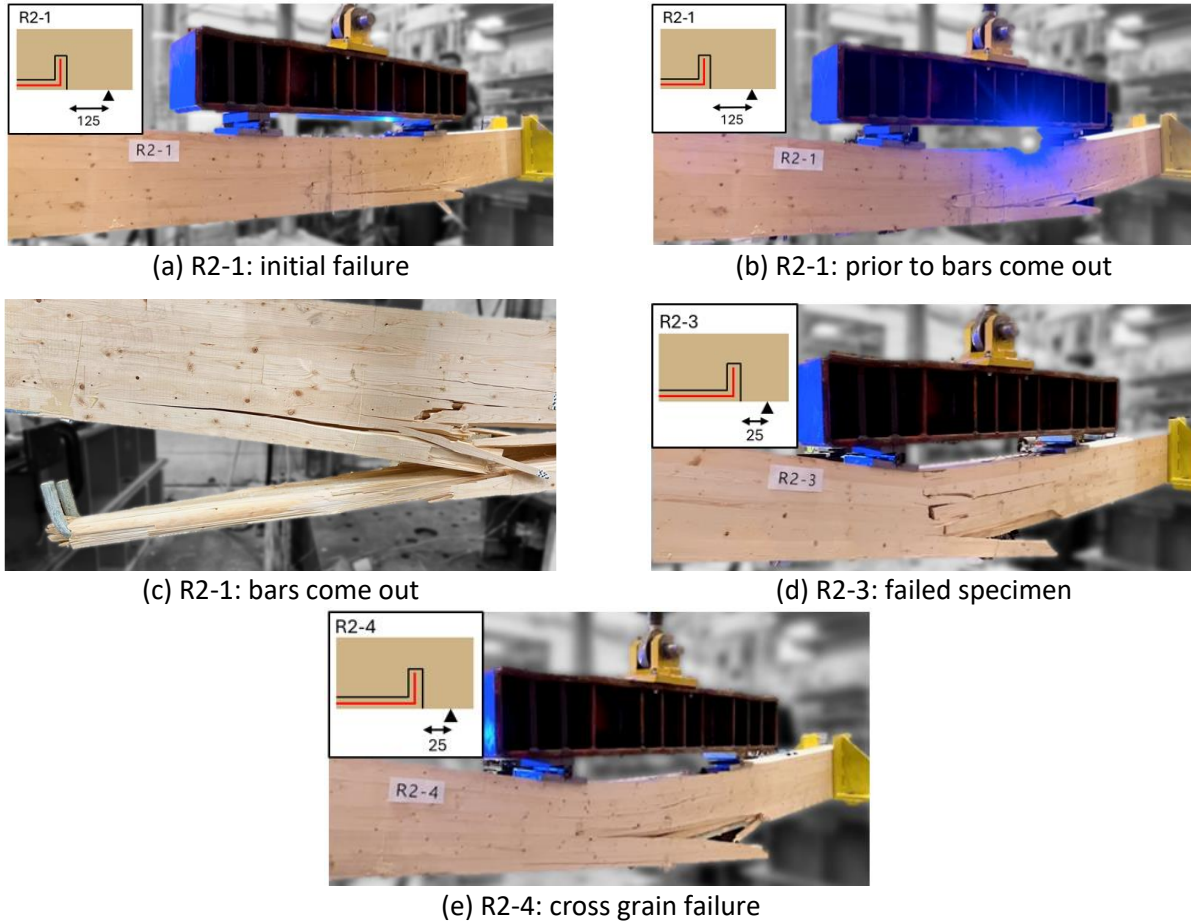


Figure 4.19 Bent bars reinforced beams with tension failures

A longitudinal shear failure was observed in specimen R2-2 (Fig. 4.20). The shear failure occurred at the height of the bent portion of the rebar across the entire width of the beam. Afterwards a secondary failure of simple tension failure occurred and continued to propagate for the duration of the test.



Figure 4.20 Initial shear failure followed by simple tension of beam R2-2

Representative load- and strain-displacement relationships for two bent bar reinforced specimens are shown in Figure 4.21. In Figure 4.21a, the maximum tensile strain coincided with the initial splintering tension failure, after which the strain gauge failed due to being close to the failure location (i.e., Fig. 4.19a). It can also be seen that upon further loading that the compression strain continued to increase with jumps coinciding with subsequent layers of wood failing. The tensile strain of the FRP bar followed similar behaviour with the maximum strain corresponding with the maximum load. In Figure 4.21b, the maximum tensile strain coincided with the maximum load, due to the location of the failure (i.e., Fig. 4.19e), the tensile strain gauge was interrupted and readings stopped after failure. The wood compression and FRP tensile strains follow the same behaviour, reaching the maximum strain reading coinciding with peak resistance then failing due to the location of the strain gauge relative to the failure.

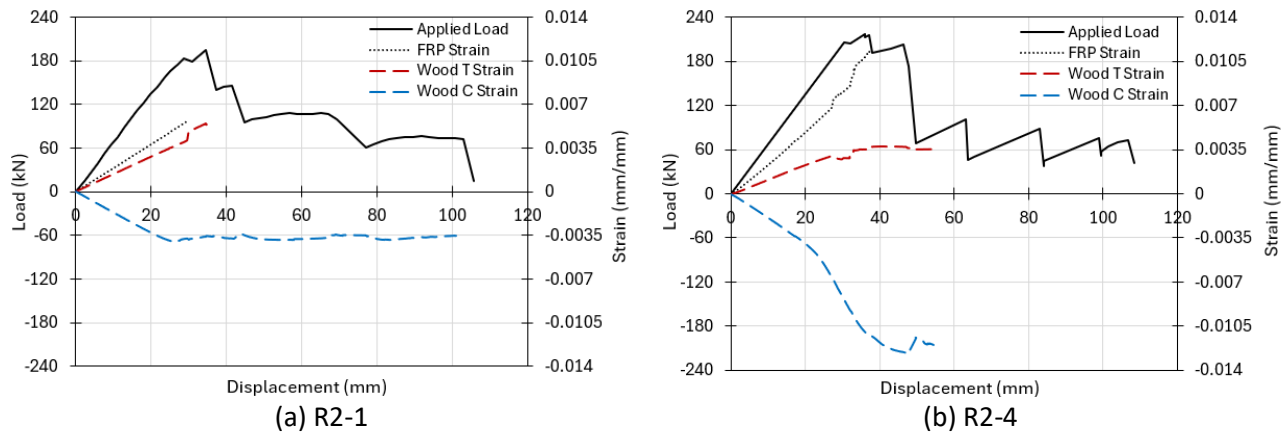
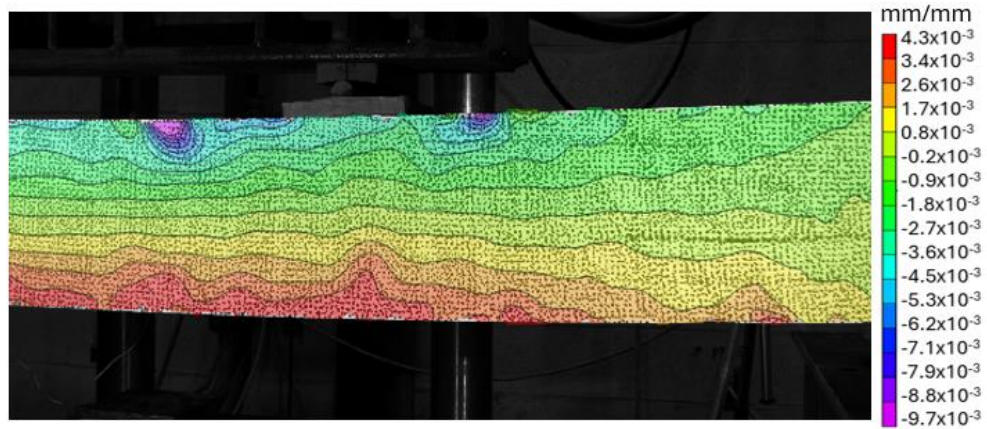
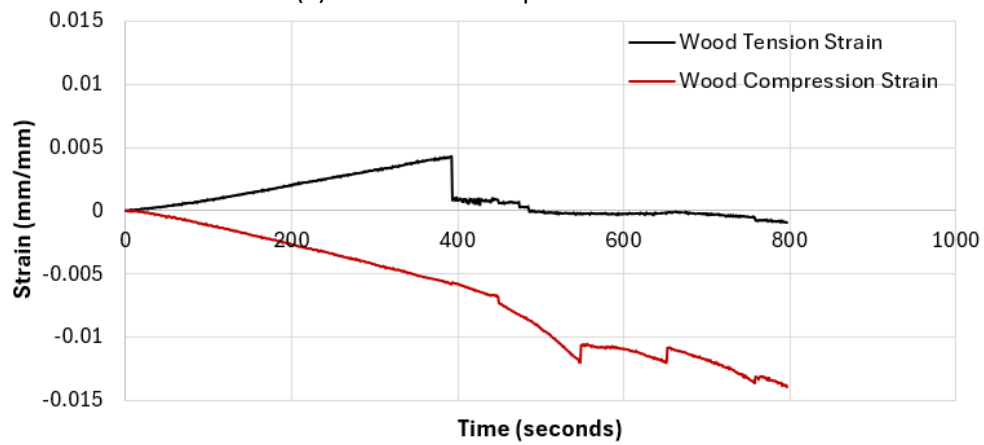


Figure 4.21 Representative stress-strain curves for straight bar reinforced beams

The longitudinal strain behaviour as captured by the DIC system is presented in Figure 4.22. Figure 4.22a shows specimen R2-4 at the maximum tensile longitudinal strain, occurring prior to failure.



(a) Strain contours prior to failure



(b) Strain-time history

Figure 4.22 Strain data extracted from DIC for specimen R2-4

CHAPTER 5 | ANALYTICAL MODEL

5.1 General

This chapter outlines the theory and methodology behind the development of the material model that predicts the flexural behaviour of the unreinforced and reinforced glulam beams. Two different approaches are used to define the idealized wood stress-strain curves to be employed in the moment-curvature analysis of the unreinforced and GFRP-reinforced beams. Both approaches are based on the material properties obtained from tension and compression coupons that are adjusted for size-effects; however, there are key differences in how the wood constitutive relationship is built. For the GFRP, material properties specified in the manufacturers' compliance report are used (MST Bar, 2015). Finally, the details of the methods used to determine the moment-curvature and resistance curves for the unreinforced and reinforced beams are discussed and verified.

5.2 Constitutive Stress-Strain Relationship

5.2.1 Overview of approaches used for the development of wood stress-strain relationships

The original stress-strain relationship for clear wood was developed by Bazan (1980) and later modified to account for defects by Buchanan (1990b) (i.e., Fig. 2.6). In Buchanan's simplified approach, the idealized tensile behaviour of wood can be defined as linear elastic up to an ultimate tensile stress, f_{tu} , and corresponding strain, ε_{tu} , while the compressive behaviour is assumed to be bilinear, with a linear response up to the compression yielding stress f_{cy} , and corresponding strain, ε_{cy} , and then followed by a linear falling branch until an ultimate compressive stress of f_{cu} and corresponding ε_{cu} . The slope of the branch is defined by mE , where E is the modulus of elasticity in the longitudinal direction and m is the ratio of the slopes in the stress-strain relationship (Buchanan, 1990b).

The values used to define the stress-strain relationship of the wood material are size and specie dependent and can either be extracted from the literature (Barrett & Lau, 1994) or experimental data from coupon testing that is adjusted for size effects (Buchanan, 1990b; Gentile et al., 2002; D. N. Lacroix, 2017; Madsen & Buchanan, 1986).

The first approach relies on parameters obtained from the coupon tests to establish the wood tensile and compressive stress-strain relationship, including both the MOEs for tension and compression. The second approach replaces the tension and compression MOEs with that of the one reflective of the stiffness on

the unreinforced beams, as determined from the stress testing. The purpose of investigating two different approaches is to evaluate the effects of the MOE on the overall flexural response as previous studies have highlighted the importance of the tension to compression MOE ratio in predicting the response (Vetter et al., 2021)

5.2.2 Adjustment of coupon strength for size-effects

Size effect is a concept in which materials with failure-causing defects have a larger probability of having a critical defect to be present in larger members than smaller members (Madsen & Buchanan, 1986). The tension and compression strength values obtained experimentally were adjusted for size effects using Equations [5.1] and [5.2], respectively (Barrett & Lau, 1994; Buchanan, 1990b).

$$f_t = f_{t_exp} \left(\frac{L_{t_exp}}{L_e} \right)^{\frac{1}{k_1}} \left(\frac{d_{t_exp}}{d} \right)^{\frac{1}{k_2}} \quad [5.1]$$

$$f_c = f_{c_exp} \left(\frac{L_{c_exp}}{L_e} \right)^{\frac{1}{k_1}} \left(\frac{d_{c_exp}}{d} \right)^{\frac{1}{k_2}} \quad [5.2]$$

Where f_t and f_c are the modified tensile and compressive strengths to be used in the material model, f_{t_exp} and f_{c_exp} are the average experimental tensile and compressive strengths obtained from coupon testing, L_{t_exp} and L_{c_exp} are the average length of the coupon specimen in tension and compression, respectively, d_{t_exp} and d_{c_exp} are the average depths of the tensile and compressive coupon specimens, L_e is the equivalent stressed length, d is the average depth of the laminates of the glulam, k_1 is the size effect parameter for length, and k_2 is the size effect parameter for depth.

Since load configuration can influence the beam strength (Buchanan, 1990b), the length of the beam under its tested loading condition is adjusted for using Equation [5.3]. For a simply supported beam with two symmetrically placed point loads, L_e is determined as follows.

$$L_e = L \left(\frac{1 + \frac{a_1 k_1}{L}}{k_1 + 1} \right) \quad [5.3]$$

Where L_e is the equivalent stressed length, L is the span of the beam, a_1 is the distance between point loads, and K_1 is the length effect parameter.

Table 5.1 presents the parameters used to determine the primary input model strength values f_t and f_c . The values for length effect parameter, K_1 , were obtained from Barrett and Lau (1994), 10.0 for compression and 5.88 for tension. The compressive yield strength was not modified for depth effects as the coupons cross-section consisted of the entire laminate, and therefore not reduced in size. The value for depth effect parameter, K_2 , was 5.60 for tension, as defined in Madsen and Buchanan (1986) based on the coefficient of variation for the tension specimens.

Table 5.1: Model Input Strengths

Properties	Compression	Tension
Exp. Strength: f_{c_exp}, f_{t_exp} (MPa)	39.6	90.8
Coefficient of variation	0.06	0.2
Exp length: L_{c_exp}, L_{t_exp} (mm)	200	460
Length effect parameter: K_1	10	5.88
Depth effect parameter: K_2	-	3.7
Equivalent stressed length: L_e (mm)	1,103.0	1,119.2
Average lamination depth: d_{c_exp}, d_{t_exp} (mm)	-	9.8
Input model strength: f_c, f_t (MPa)	33.4	51.4

5.2.3 Stress-strain relationship using experimental coupon test results

The previous section determined that the tension (f_t) and compression (f_c) adjusted for size effects is less than the average obtained for the coupon tests reported in Chapter 4. As a result, it is necessary to adjust the constitutive stress-strain relationships accordingly for use in the moment-curvature analysis. Figure 5.1 shows the scaled tension stress-strain relationship to reflect the incorporation of the size-effects relative to the experimental average and individual coupon tests.

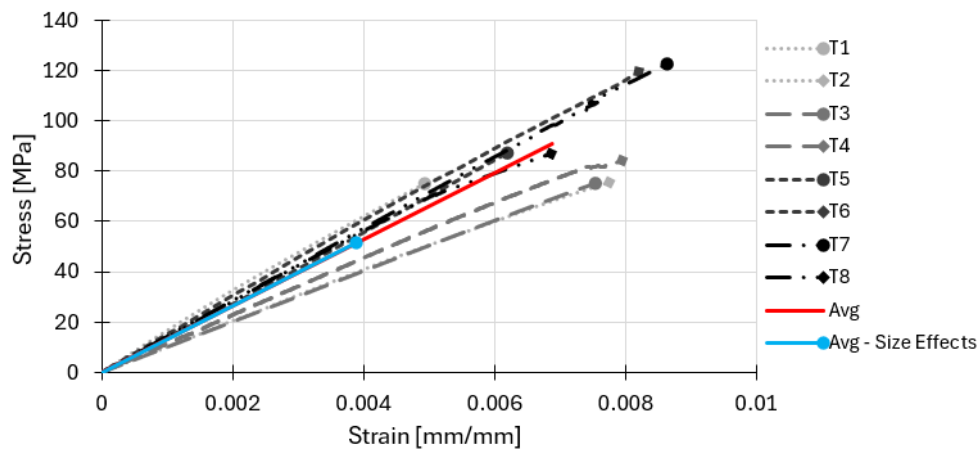


Figure 5.1 Stress-strain relationship of tension coupons

While the scaling was simple for the tension coupons, additional considerations are required for the scaled compression stress-strain relationship given the non-linear behaviour reported in Chapter 4. Figure 5.2a shows the stress-strain relationship for all six compression coupons along with an approximation of an experimental average curve depicting that the overall behaviour observed is closer to that described by Glos (1978). A closer look at the average ultimate strains achieved in the compression coupon tests (i.e., $\epsilon_{cu} = -38 \times 10^{-3}$ mm/mm) show that they are significantly larger than the maximum compression strains recorded in the flexural tests, both unreinforced ($\epsilon_{c,max} = -9.6 \times 10^{-3}$ mm/mm) and reinforced ($\epsilon_{c,max} = -14.8 \times 10^{-3}$ mm/mm). Therefore, the portion of the coupons' compression behaviour is limited to include the initial linear portion pre-peak and the post-peak response up to strains of 0.0215 mm/mm as shown in Figure 5.2b. The average curve was defined by using the average yield strength (f_{cy}), the slope of descending branch (m), and the stiffness (MOE) from the experimental results presented in Chapter 4. The curve was then scaled to account for size-effects as shown in Figure 5.2b.

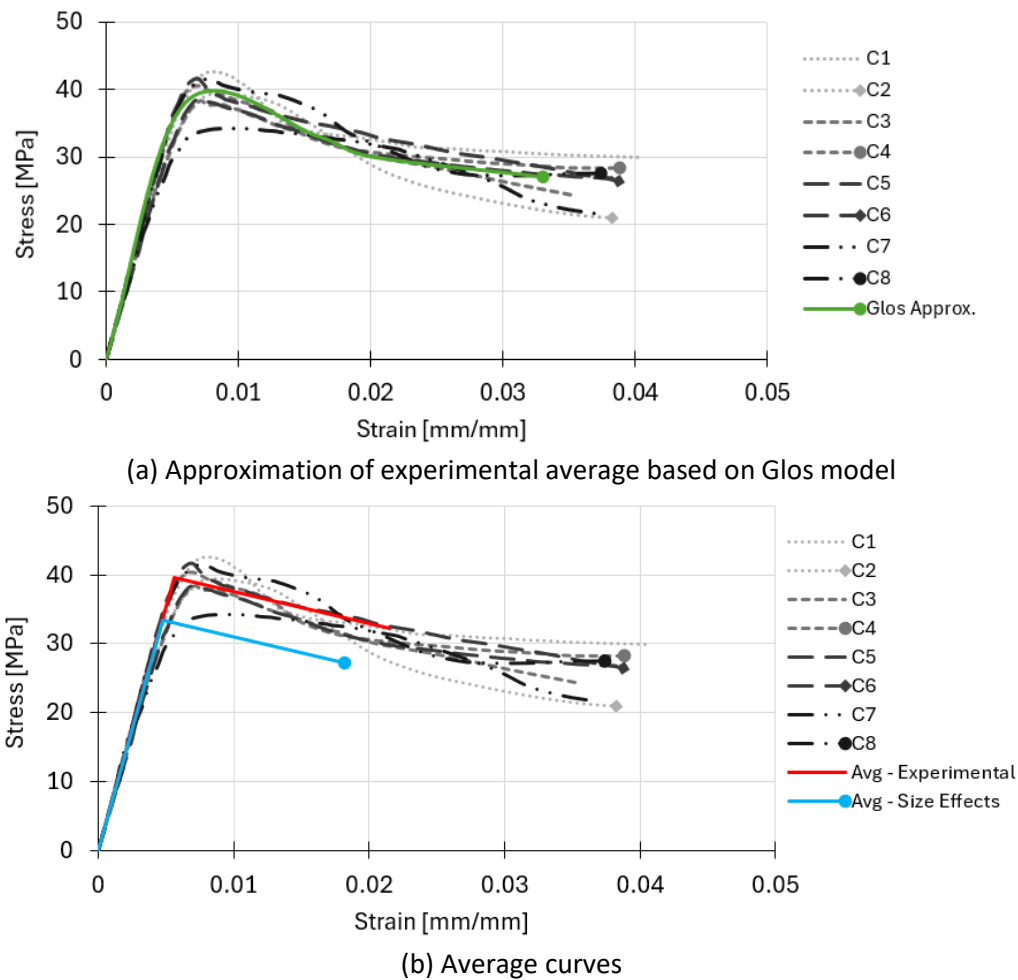


Figure 5.2 Stress-strain relationship of compression coupons

5.2.4 Stress-strain relationship using full-scale test MOE

An alternative stress-strain relationship was investigated using the average flexural MOE of 10,718 MPa as determined from the full-scale tests on the four unreinforced beams tested to failure in addition to the ten reinforced beams that were prestressed. The second approach retains the shape of the tension and compression stress-strain curves; however, they are adjusted by determining new strain values based on the flexural MOE relative to the tension and compression MOEs from the coupon tests as shown in Figures 5.3a and 5.3b, respectively. For the tension stress-strain curve, this leads to an increased tensile failure strain since the flexural MOE is lower than the one recorded for the coupon tests. On the other hand, the strains for the compressive stress-strain curve were reduced in accordance to the ratio of the coupon to flexural MOE.

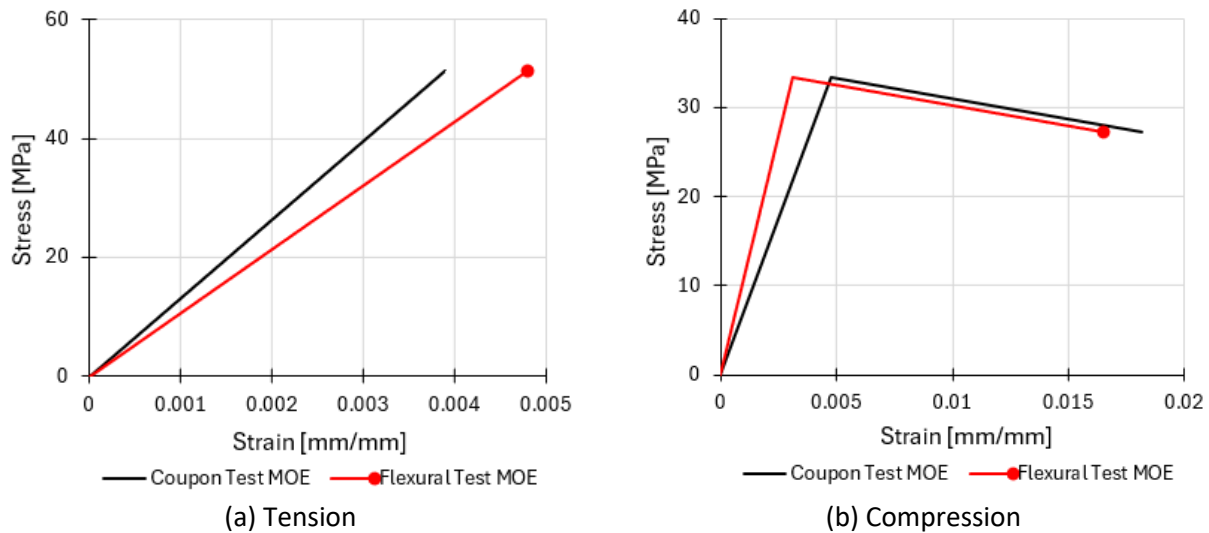


Figure 5.3 Stress-strain curves with flexural MOE

5.3 Moment-Curvature Analysis

5.3.1 Analysis of unreinforced beams

The strain and stress profiles, and the resulting forces of a rectangular cross-section of an unreinforced member prior to exceeding the compression yield strain ($\epsilon_{top} \leq \epsilon_{cy}$) are shown in Figure 5.4. A linear distribution of stresses and strains is observed with the linear-elastic behaviour of the material.

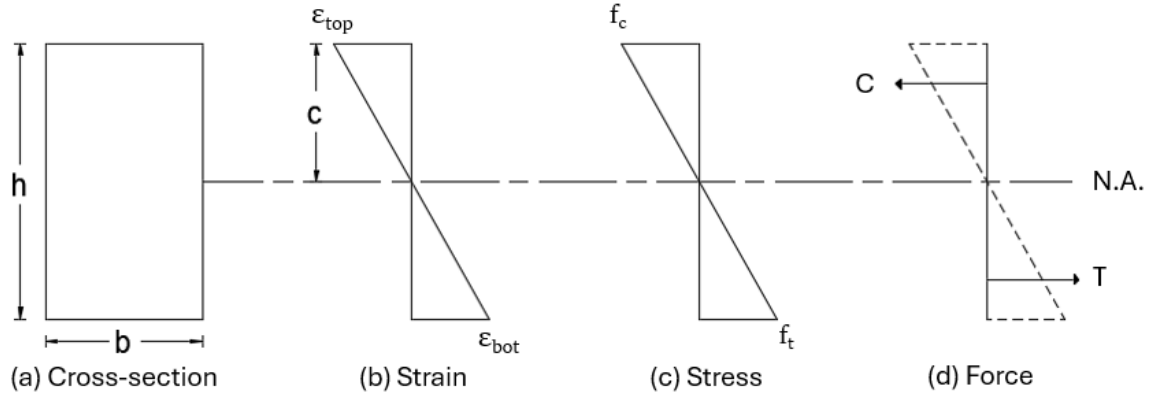


Figure 5.4 Unreinforced glulam: strain, stress, and force diagrams prior to compression yielding

For the unreinforced beams prior to exceeding compression strain, the moment about the mid-height can be calculated using Equations [5.4]-[5.6].

$$C = 0.5E_c\epsilon_{top}cb \quad [5.4]$$

$$T = 0.5E_t\epsilon_{bot}(h - c)b \quad [5.5]$$

$$M_{h/2} = C_1\left(\frac{h}{2} - \frac{c}{3}\right) + T\left(\frac{h}{2} - \frac{h - c}{3}\right) \quad [5.6]$$

Where ϵ_{top} and ϵ_{bot} are the top and bottom strains, representing the compression and tension strains, E_c and E_t are the compression and tension moduli of elasticity, c is the depth to the neutral axis, b is the width of the member and h is the height of the member.

The strain and stress profiles, and the resulting forces of a rectangular cross-section of an unreinforced member having exceeded the compression yield strain (ϵ_{cy}) are shown in Figure 5.5.

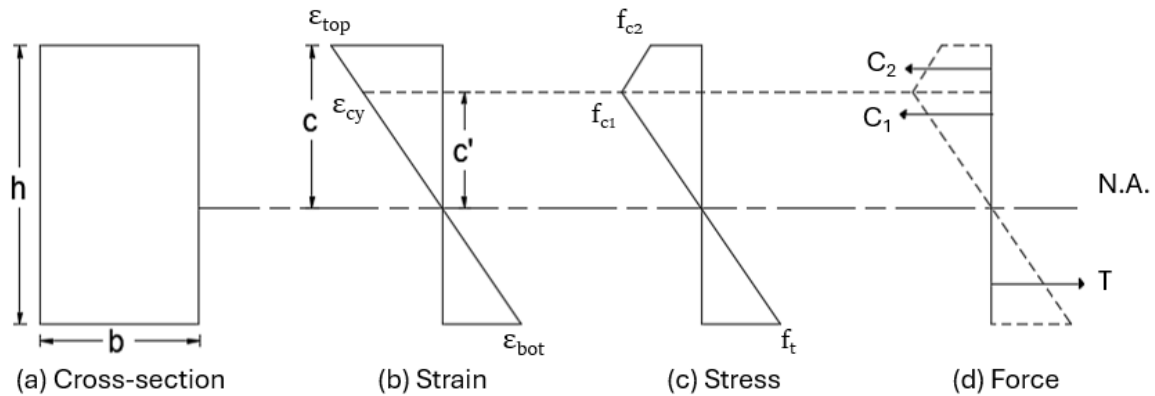


Figure 5.5 Unreinforced glulam: strain, stress, and force diagrams post compression yielding

For the unreinforced beams that have exceeded the compressive yielding strain, the moment about the mid-height can be calculated using Equations [5.7]-[5.12].

$$C_1 = 0.5f_{c1}cb \left(\frac{\varepsilon_{cy}}{\varepsilon_{top}} \right) \quad [5.7]$$

$$f_{c1} = E_c \varepsilon_{cy} \quad [5.8]$$

$$C_2 = 0.5cb(f_{c1} + f_{c2}) \left(1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}} \right) \quad [5.9]$$

$$f_{c2} = f_{c1} - mE_c(\varepsilon_{top} - \varepsilon_{cy}) \quad [5.10]$$

$$T = 0.5E_t \varepsilon_{bot} b(h - c) \quad [5.11]$$

$$M_{h/2} = C_1 \left(\frac{h}{2} - c + \frac{2c'}{3} \right) + C_2 \left(\frac{h}{2} - \frac{c - c'}{2} \right) + T \left(\frac{h}{2} - \frac{h - c}{3} \right) \quad [5.12]$$

Where c' is the depth of linear-elastic portion of the compression zone measured from the neutral axis, f_{c1} and f_{c2} are the compressive stresses, C_1 , C_2 are the compressive forces, and T is the tensile force acting on the cross-section.

The stress distribution factor explains the phenomenon that strength depends on the distribution of stresses at a given cross-section and needs to be accounted for when calculating the bending capacity of wood members using experimental tensile results (Madsen & Buchanan, 1986). It is accounted for using Equation [5.13].

$$f_m = f_t \left(\frac{d(1 + k_3)}{d - c} \right)^{\frac{1}{k_3}} \quad [5.13]$$

Where f_m is the extreme tensile bending stress, f_t is the uniaxial tensile stress, d is the depth of the member, k_3 is the stress distribution factor, and c is the depth to the neutral axis.

The moment-curvature modelling was done through a VBA program which was developed to generate moment resistance curves based on cross-sections with predefined dimension and material stress-strain behaviours. The moment-curvature relationship is generated until failure by increasing the compressions strain and recording the moment resistance. The program detects failure as the point at which the material strains of the FRP and wood in tension and compression are greater that of the experimentally determined material limits. There are two main types of failure modes that can be determined through the model. The first being tensile failure, which occurs when the tensile strain of the wood exceeds the allowable bending tensile strain, and the second occurring when the compressive strain of the wood exceeds the ultimate compressive strain. Figure 5.6 shows representative moment-curvature diagrams for the defined cross-section of an unreinforced beam.

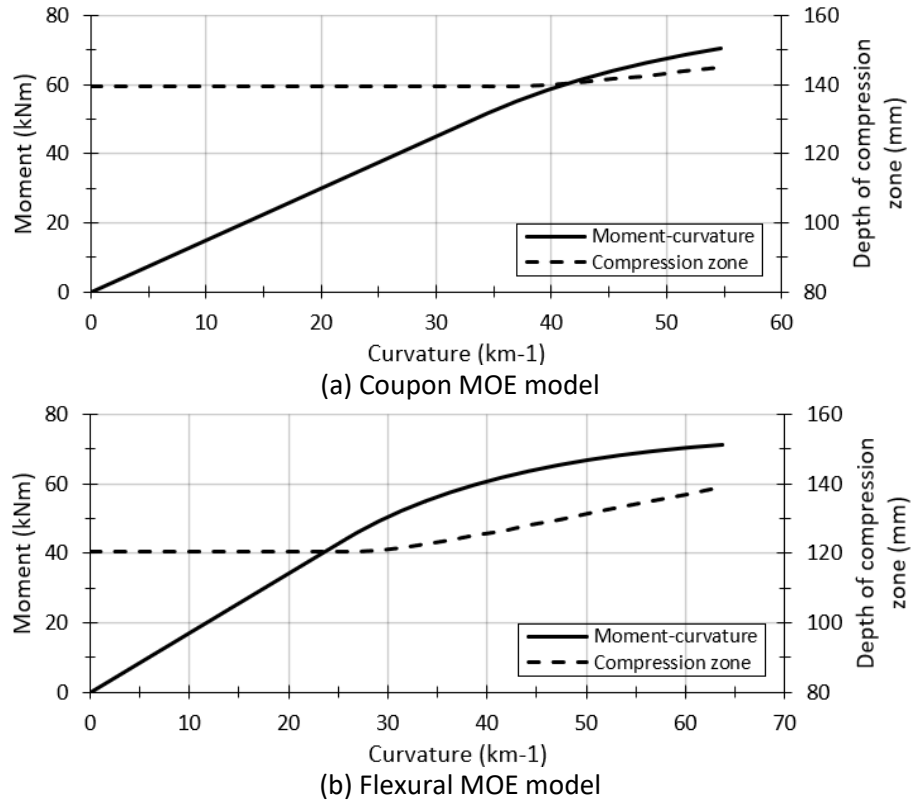


Figure 5.6 Representative moment-curvature analysis for unreinforced beams

5.3.2 Analysis of reinforced beams

The strain and stress profiles, and the resulting forces for a reinforced member prior to exceeding the compression yield strain ($\epsilon_{top} \leq \epsilon_{cy}$) are shown in Figure 5.7.

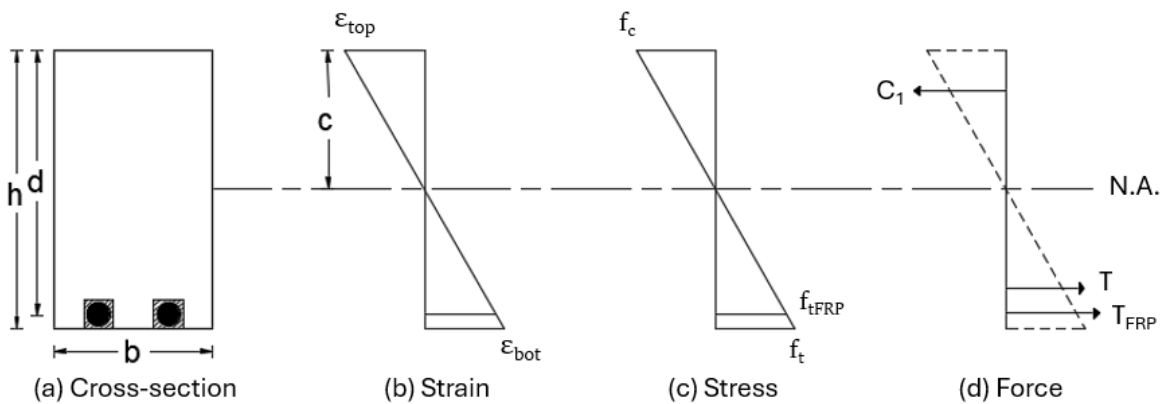


Figure 5.7 Reinforced glulam: strain, stress, and force diagrams prior to compression yielding

For the reinforced beams that have not yet reached the compressive yield strain, the moment resistance about the mid-height is calculated using Equations [5.14]-[5.17].

$$C_1 = 0.5E_c\varepsilon_{top}bc \quad [5.14]$$

$$T_1 = 0.5E_t\varepsilon_{bot}b(h - c) \quad [5.15]$$

$$T_{FRP} = E_{FRP}\varepsilon_{bot}A_{FRP} \quad [5.16]$$

$$M_{h/2} = C_1\left(\frac{h}{2} - \frac{c}{3}\right) + T_1\left(\frac{h}{2} - \frac{h}{3}\right) + T_{FRP}\left(d - \frac{h}{2}\right) \quad [5.17]$$

Where ε_{top} and ε_{bot} are the top and bottom strains, representing the compression and tension strains, E_c and E_t are the compression and tension moduli of elasticity, E_{FRP} is the FRP modulus of elasticity, c is the depth to the neutral axis, b is the width of the member, h is the height of the member, d is the distance to the reinforcement measured from the top, T_{FRP} is the tension force from the FRP, and A_{FRP} is the area of FRP reinforcement.

The strain and stress profiles, and the resulting forces of a rectangular cross-section of a reinforced member having exceeded the compression yield strain (ε_{cy}) are shown in Figure 5.8.

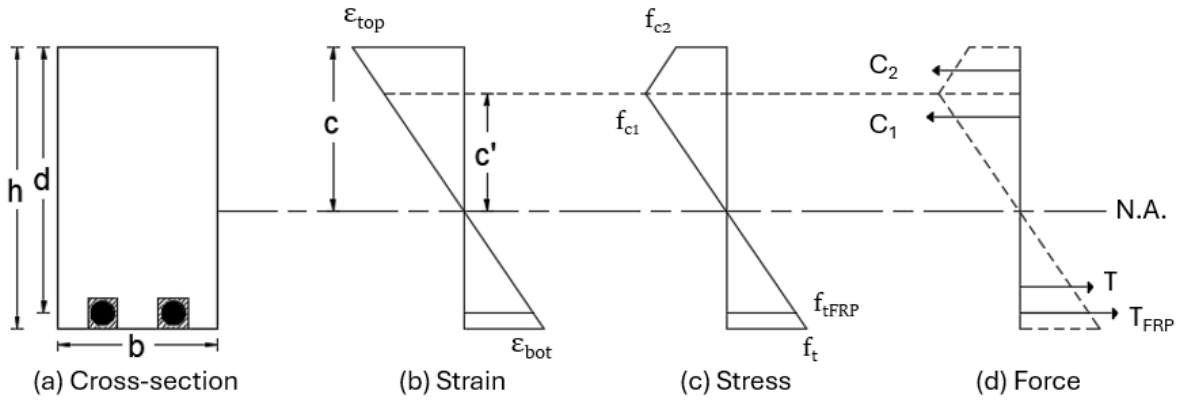


Figure 5.8 Reinforced glulam: strain, stress, and force diagrams post compression yielding

For the reinforced beams that have exceeded the compressive yield strain, the moment resistance about the mid-height is calculated using Equations [5.18]-[5.22].

$$C_1 = 0.5f_{c1}bc\left(\frac{\varepsilon_{cy}}{\varepsilon_{top}}\right) \quad [5.18]$$

$$C_2 = 0.5cb(f_{c1} + f_{c2})\left(1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}}\right) \quad [5.19]$$

$$T_1 = 0.5E_t\varepsilon_{bot}b(h - c) \quad [5.20]$$

$$T_{FRP} = E_{FRP}\varepsilon_{bot}A_{FRP} \quad [5.21]$$

$$M_{h/2} = C_1\left(\frac{h}{2} - c + \frac{2c'}{3}\right) + C_2\left(\frac{h}{2} - \frac{c - c'}{2}\right) + T_1\left(\frac{h}{2} - \frac{h}{3}\right) + T_{FRP}\left(d - \frac{h}{2}\right) \quad [5.22]$$

Where c' is the depth of linear-elastic portion of the compression zone measured from the neutral axis, f_{c1} and f_{c2} are the compressive stresses, C_1 and C_2 are the compressive forces, and T_1 and T_{FRP} are the tensile force acting on the cross-section.

Much like the unreinforced specimens, the stress distribution factor in the reinforced specimens must also be accounted for. It is accounted for using Equation [5.23]. Gentile (2002) made modifications to the equation, purposed to account for the added benefits that the addition of FRP reinforcement have on the tensile behaviour. Higher tensile failure strains are observed in FRP reinforced wood members, this can be attributed to the FRP minimizing the effects defects in wood members (Gentile et al., 2002; Johns & Lacroix, 2000; Raftery & Harte, 2013).

$$f_{m,r} = f_t \alpha_m \left(\frac{d(1 + k_3)}{d - c} \right)^{\frac{1}{k_3}} \quad [5.23]$$

Where $f_{m,r}$ is the extreme tensile bending stress, f_t is the uniaxial tensile stress, α_m is the modification factor introduced by Gentile et al. (2000), d is the depth of the member, k_3 is the stress distribution factor, and c is the depth to the neutral axis.

The VBA model runs the same for the reinforced beams as it does the unreinforced beams with the addition of a possible failure mode. Previously, the failure modes were governed by either the tensile strain or compressive strain of the wood exceeding the allowable and ultimate strain, respectively. With the addition of FRP reinforcement and new failure mode the introduced in the model and occurs when the FRP strain exceeds its limit causing the material to rupture. Figure 5.9 shows a representative moment-curvature diagram for the defined cross-section of a reinforced beam.

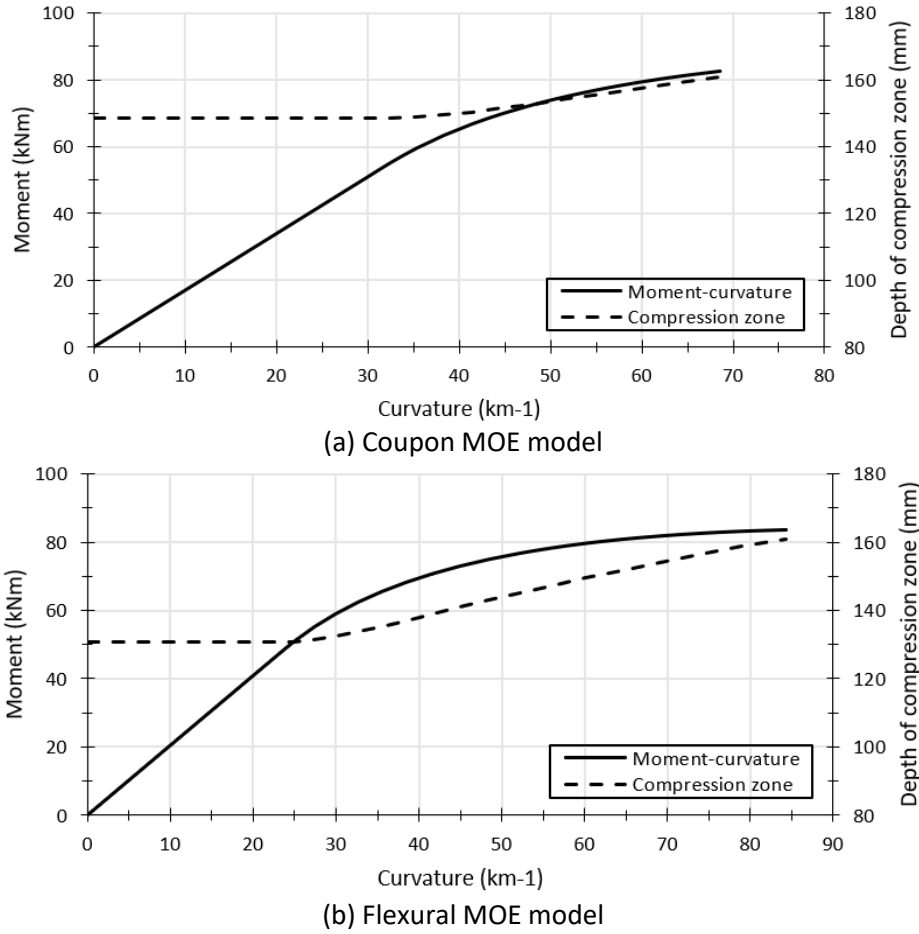


Figure 5.9 Representative moment-curvature analysis for reinforced beams

5.4 Development of Resistance Curves

5.4.1 Derivation of displacement equations for linear elastic materials

The model was developed to present the flexural behaviour of the beams in the form of resistance curves, such that the results of the modelled and experimental analyses can be compared easily. For a simply supported beam subjected to four-point bending of equal loading at thirds the displacement equations are derived from first principals (Fig. 5.10). By solving the differential equation for the elastic curve, the displacements for the linear elastic material are calculated using Equation [5.24].

$$\Delta_{max} = \frac{23\alpha^2\varphi}{24} \quad [5.24]$$

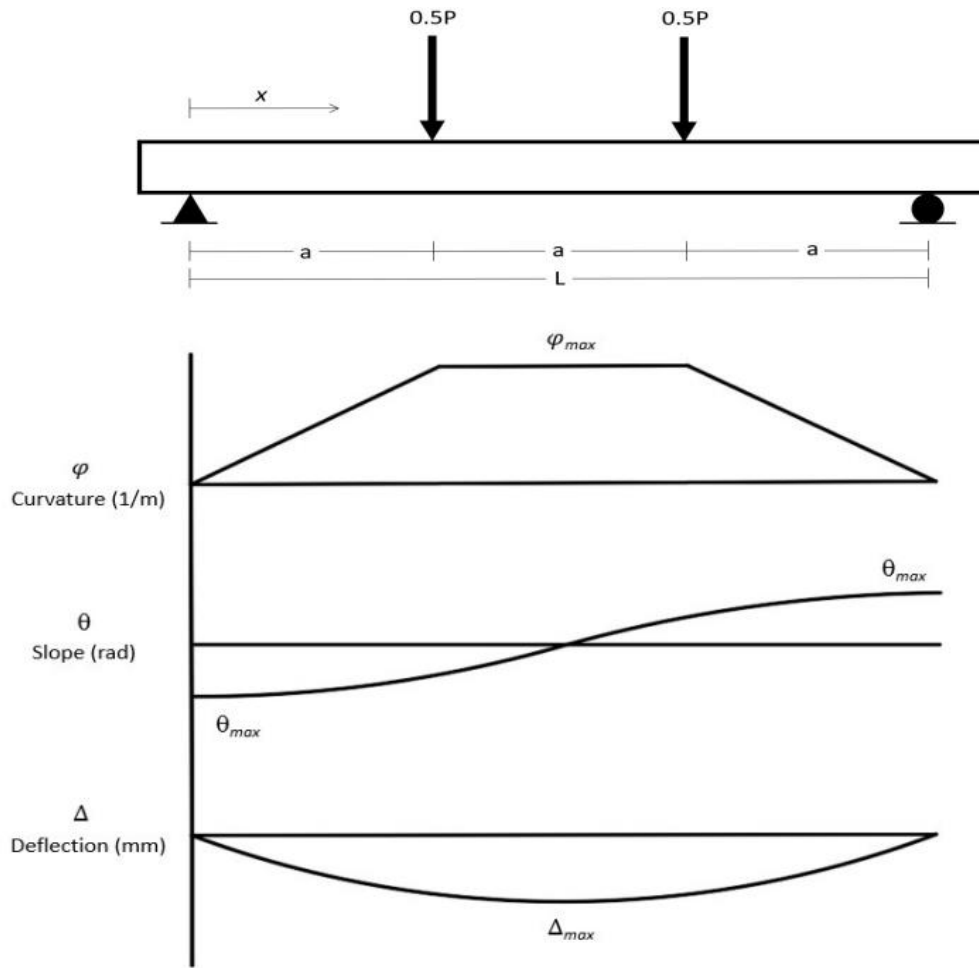


Figure 5.10 Beam diagrams for the derivation of the linear material displacements

5.4.2 Derivation of displacement equations for nonlinear materials

As discussed in Section 5.2 the compressive behaviour of glulam is assumed to be bilinear, having a linear response up to the compression yielding stress and corresponding strain, followed by a linear falling branch until an ultimate compressive stress. Such that once the compressive yield strain is exceeded and the beam continues to deflect causing nonlinearity in the member. The midspan displacements of the yielded beams were determined using a VBA program by graphing the double integral of the moment-curvature diagram. The program was developed to generate the curvatures along the length of the beam based on the assigned midspan curvature and predetermined moment-curvature. The program integrates following the trapezoidal rule to produce the slope and displacement diagrams (Fig. 5.11).

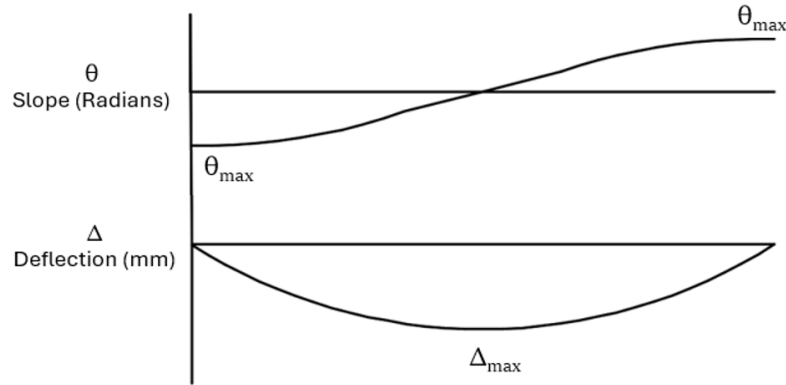


Figure 5.11 Beam diagrams for the derivation of the nonlinear material displacements

5.4.3 Generation of resistance curves

To generate the resistance curves the program follows a procedure that converts data points on the moment-curvature plot into equivalent force-displacement points. The force is calculated by determining the load required to produce the target moment, the corresponding displacement is then calculated by converting the curvature into a displacement. Figure 5.12a shows a moment-curvature plot and Figure 5.12b shows the converted load-displacement curve for representative reinforced glulam beam as predicted with the model using the coupon determined MOE, as Figures 5.12c and Figure 5.12d for the model with the flexural MOE.

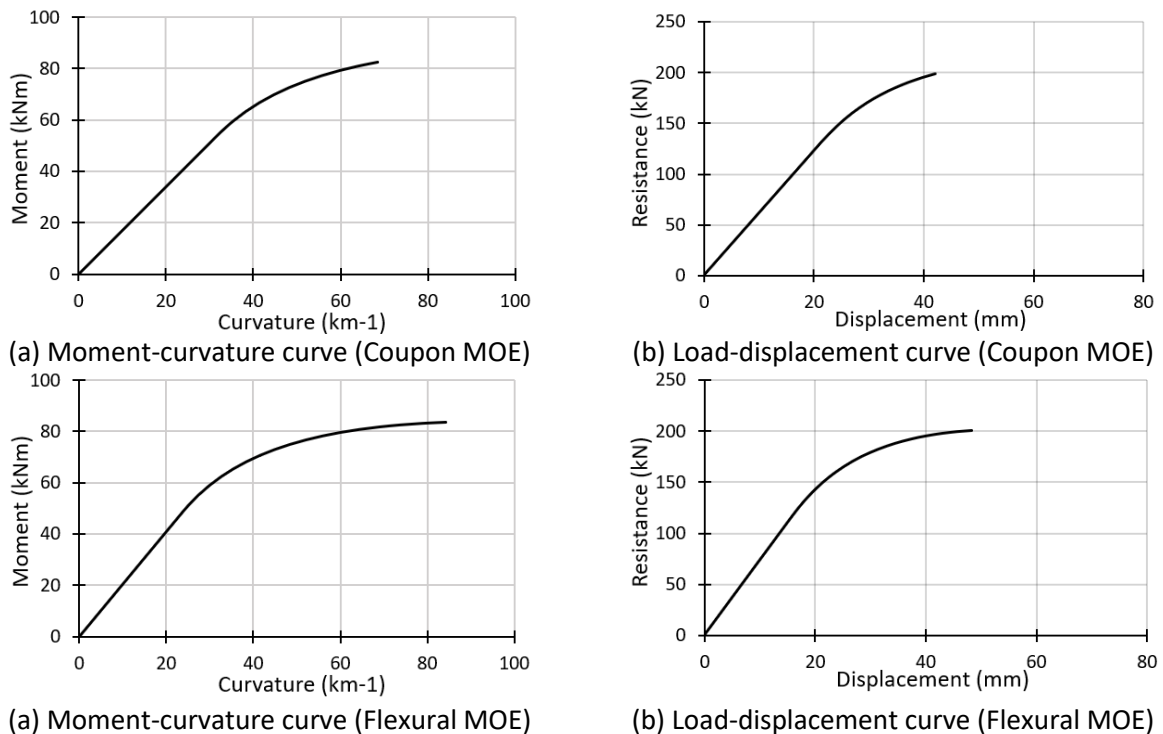


Figure 5.12 Representative moment-curvature and load-displacement curves for a reinforced beam

CHAPTER 6 | DISCUSSION

The following sections discuss the findings of the experimental program presented in Chapter 4, specifically regarding the effects of adhesive type and knurling pattern on the bond strength of the pullout specimens, and of the reinforcement length and reinforcement configuration on the behaviour and failure modes of GFRP-reinforced glulam beams. Additionally, the predictions from the material model are presented and compared against those of the experimental results. The implications of the findings on the practical application in design are also discussed.

6.1 Effects of Adhesive and Knurling Pattern on Bond Strength

During the small-scale testing phase, eighteen pullout tests were conducted to investigate the bond strength of the specimens based on the adhesive used and the knurling pattern. Initially, the purpose was to find an adequate adhesive and as such, a total of five different adhesives were investigated followed by three different routing patterns with the specific adhesives to be used in the full-scale tests.

Table 6.1 shows a summary of the results in terms of averages for the maximum applied load, displacement at maximum load, and stiffness. In general, all adhesives performed reasonably well, except for Adh-1. Additionally, the texture and density of the adhesive was observed to be a dominant factor in affecting the overall performance of the pullout tests compared to the addition of knurling. The following sections discuss in greater details the global trends that were observed.

Table 6.1: Summary of Pullout Test Results

Routing Pattern	Specimen	Adhesive Type	P_{max}^a (kN)	Δ_{max}^b (mm)	K_1^c (kN/mm)	K_2^d (kN/mm)
Straight	P-01; P-02	Adh-1	17.7	0.45	47.6	44.5
	P-03; P-04	Adh-2	67.4	1.85	52.1	36.1
	P-05; P-06	Adh-4	86.5	2.25	43.4	38.2
	P-07; P-08	Adh-5D	59.6	1.45	59.1	40.0
	P-09; P-10	Adh-5F	84.8	2.45	44.2	34.6
	P-11; P-12	Adh-2, Adh-3	55.2	1.50	44.0	39.4
Alternating A	P-13; P-14	Adh-2, Adh-3	60.6	1.70	54.4	34.2
	P-15	Adh-5D	53.9	1.10	48.4	37.1
	P-16	Adh-5F	96.9	2.90	48.9	33.1
Alternating B	P-17; P-18	Adh-5F	79.7	2.95	39.2	26.8

^a Maximum load

^b Deflection at maximum load

^c Bending stiffness (10-40%)

^d Bending stiffness (P_{max}/Δ_{max})

Adh-1 was deemed to be inadequate for structural applications, failing on average at loads 4.03 times less than that of the other adhesives. Despite achieving comparable stiffness to the other adhesives, this two-part epoxy marketed for anchors and reinforcement bars with masonry and concrete is clearly not compatible in this case. While not specifically marketed for wood applications, the destined use of the marketed product was similar to the needs of the study, and on paper, its technical data sheet warranted its evaluation for an off-the-shelf product.

The strongest adhesive was identified as Adh-4 with an average failure load of 86.5 kN, which is only 1.02 times larger than the next strongest adhesive (i.e., Adh-5F). It also performed well in terms of stiffness; however, due to the limited availability of Adh-4 in large quantities (i.e., not in tubes), further tests could not be conducted. Therefore, it is suggested further testing be done to determine Adh-4's capabilities when used to bond GFRP bars to wood in bending applications.

The results of the different adhesives have also shown that the density and texture of the adhesive affect the bond strength. Fluid epoxies (e.g., Adh-4, Adh-5F) performed better than epoxy pastes (e.g., Adh-2/3, Adh-5D). This increase in strength can be attributed to the fluid epoxies' ability to fill all voids and thus create a better bond between reinforcement and the wood. While both Adh-5 epoxies (i.e., fluid and dense) have the similar material properties on paper in terms of tensile, flexural and compressive strengths, they performed differently. Figure 6.1a shows the differences between the dense and fluid adhesives for straight grooves for which the average strength using the fluid epoxy is 1.42 times that of the dense epoxy. Similarly, increases in resistance ranging between 1.4 – 1.8 were observed for the specimens with alternating grooves A and B using fluid epoxy, respectively, in comparison to the dense one with alternating groove A (Fig. 6.1a).

It should be noted that while the maximum applied load for specimen P-15 using Adh-5D with alternating grooves ($P_{max} = 53.9$ kN) is significantly lower than P-16 using Adh-5F with alternating grooves ($P_{max} = 96.9$ kN), that the value is in the range of the average for the specimens using Adh-5D with straight grooves ($P_{max} = 59.6$ kN). A closer look at the specimens using Adh-5F with alternating routing pattern A (i.e., P-16) and B (i.e., P-17, P-18) relative to the straight groove specimens using the same adhesive, ratios of 1.14 and 0.94 are obtained, respectively. Similarly, the ratio of the maximum load of the specimens using alternating routing pattern A (i.e., P-13, P-14) to the straight groove (i.e., P-11, P-12) for the combination of Adh-2 and Adh-3 is 1.10. Therefore, the depth of the alternating notch does not appear to play a critical role on the maximum load as much as the adhesive itself. On average, the alternating routing patterns

contributed to displacement at failure of 1.13 for the combination of Adh-2 and Adh-3, 1.18 for Adh-5F with alternating routing pattern A, and 1.20 for Adh-5F with alternating routing pattern B. Figure 6.1b shows a comparison between specimens with groove pattern B compared to straight groove.

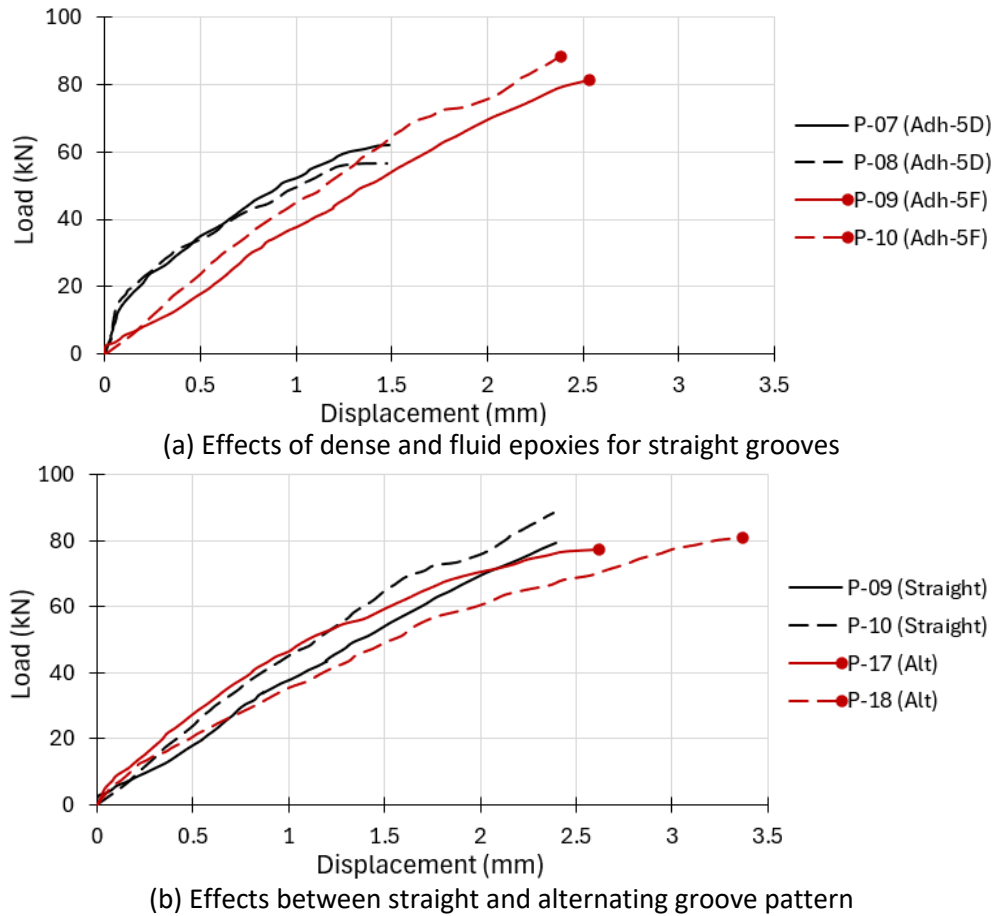


Figure 6.1 Pullout test load-displacement curves

Due to the limited number of replicas per configurations, the reported averages in Table 6.1 are shown to highlight the observed trends, and additional testing shall be conducted to confirm the statistical validity of observations on a larger sample. Furthermore, while two stiffness values are provided, K_2 is consistently lower than K_1 ; however, when comparing two adhesives, having K_1 higher for one adhesive does not necessarily guarantee that K_2 will also be higher (e.g., Adh-2 vs Adh-4). This is primarily attributed to some adhesives exhibiting an unusually high stiffness compared to the failure displacement.

6.2 Behaviour of Unreinforced Beams

Four unreinforced glulam beams were tested to failure under four-point bending. A summary of the results is presented in Table 6.2 for the maximum load, displacement at maximum load, stiffness, modulus

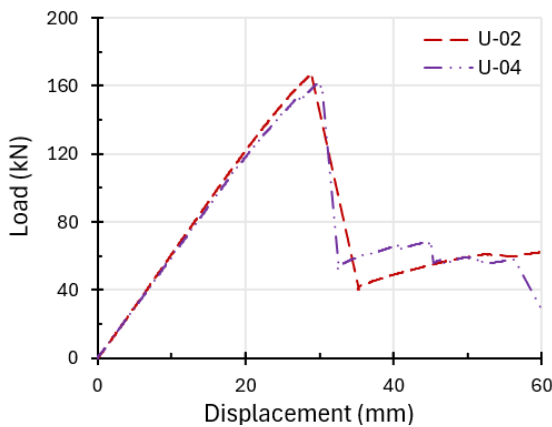
of rupture, modulus of elasticity, and initial failure mode type. The main failure modes observed consisted of splintering, shear, or a combination thereof as shown in Figure 6.2. Despite the different failure modes observed, the maximum coefficient of variation (CoV) is 10% or less for all properties reported.

Table 6.2: Summary of Unreinforced Test Results

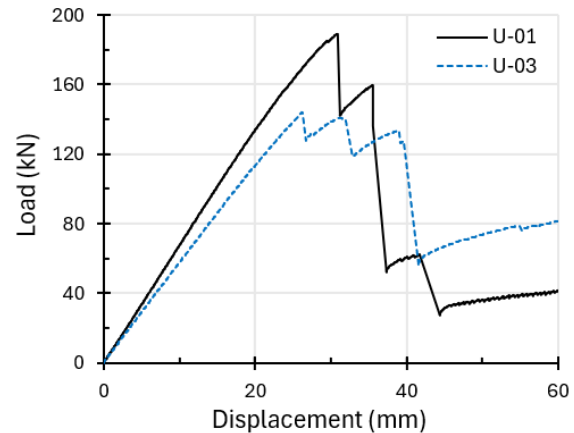
Specimen Number	$P_{\max}^a$ (kN)	$\Delta_{\max}^b$ (mm)	K^c (N/mm)	MOR ^d (MPa)	MOE ^e (MPa)	Initial Failure Mode
U-01	189.2	30.3	6,791	59.4	11,783	Splintering, Shear
U-02	167.3	28.9	6,213	52.6	10,781	Splintering
U-03	144.0	26.2	5,731	45.3	9,945	Shear, Splintering
U-04	162.4	30.0	5,973	51.0	10,365	Splintering
Average	165.7	28.9	6,177	52.1	10,718	
CoV	0.1	0.06	0.06	0.1	0.06	

^a Maximum Load ^b Deflection at maximum load ^c Bending stiffness ^d Modulus of rupture ^e Modulus of elasticity

In general, the resistance curves show that the unreinforced glulam beams behave linearly up to the maximum load followed by a drop in resistance that is associated with the initial failure. For splintering tension failures (i.e., U-02, U-04), the beams showed little to no post-peak resistance with the failure propagating through the path of least resistance (Figs. 6.2a and 6.2c). However, for specimens U-01 and U-03, the initial drop in resistance (Fig. 6.2b) is less significant due to the mixed failure modes observed involving shear (Fig. 6.2d).



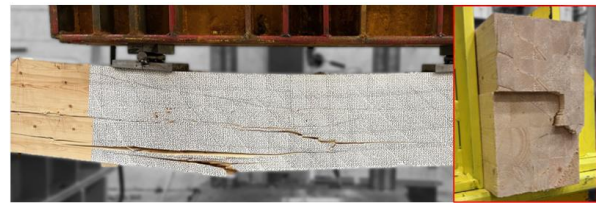
(a) Resistance curves for U-02 and U-04



(b) Resistance curves for U-01 and U-03



(c) Representative splintering failure (U-04)



(d) Representative shear failure (U-03)

Figure 6.2 Behaviour of unreinforced beams

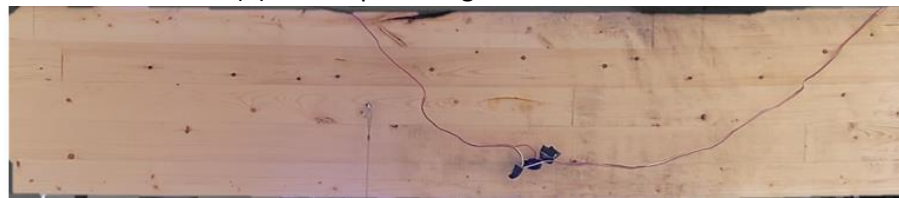
Figure 6.3 shows specimens U-01 and U-03 before and after the shear failure. In the case of U-01, a small splintering failure at a knot (Fig. 6.3a) was immediately followed by a longitudinal shear failure in the lower half of the beam (Fig. 6.3b). For specimen U-03, Figure 6.3c shows the specimen before the longitudinal shear failure while Figure 6.3d highlights the longitudinal shear failure in the upper half of the specimen. These longitudinal shear failures, although they contributed to the higher post-peak resistance prior to attaining the next major break, are not a desirable failure mode and should be avoided.



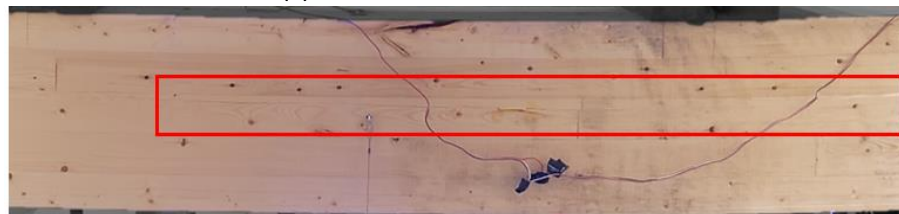
(a) U-01: Initial splintering failure



(b) U-01: Splintering and shear failure



(c) U-03: Prior to shear failure



(d) U-03: Following initial shear failure

Figure 6.3 Shear failures of unreinforced beams

The shear failures observed in specimens U-01 and U-03 can partly be attributed to the a/d ratio of the specimens being 3.5. The standard recommends a shear span-to-depth ratio of at least 4 when evaluating the flexural behaviour of beams and states that specimens with an a/d ratio of 2.5 or less are likely to produce more shear failures (ASTM, 2022b). In the a/d ratios suggested by the standard (ASTM, 2021b), it is assumed that the modulus of rupture (MOR) is ten times greater than the shear strength (τ_m).

Therefore, if the a/d ratio is equal to that of $MOR/4\tau_m$, then the specimen is likely to fail in either shear or in flexure.

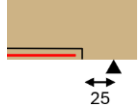
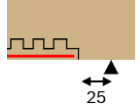
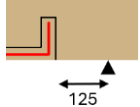
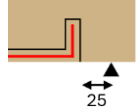
However, previous research conducted by Gharaibeh and Doudak (2023) and Lacroix and Doudak (2018) on similar cross-section sizes assuming that the MOR is ten times greater than τ_m and had a/d ratios of 3.9 (i.e., 137mm x 191mm, clear span 2,235mm) and 3.4 (i.e., 137mm x 222mm, clear span of 2,235mm), respectively, where no shear failures were observed in both unreinforced and reinforced glulam beams. On the other hand, the specimens investigated by Vetter (2022) had a/d ratios of 4.0 (i.e., 69mm x 100mm, clear span 1,200mm) with the assumption that the MOR is ten times greater than τ_m and while no shear failures were reported for the unreinforced specimens, the specimens reinforced with GFRP fabrics resulted in stress concentration failure at the FRP termination point.

In the current study, the unreinforced specimen that initially failed in shear (i.e., U-03) corresponded to a shear stress of 3.27 MPa based on the load corresponding to the failure (P_{max}). Similarly, specimen U-01 initially experienced a small splintering failure that was immediately followed by a shear failure corresponding to a shear stress of 4.29 MPa. Based on the average MOR of specimens U-02 and U-04 (i.e., 51.8 MPa), which had no shear failure, this results in ratios of $MOR/4\tau_m$ ranging 3.02 – 3.96. Therefore, the mixed failure modes can in part be attributed to the lower MOR-to-shear strength ratio of the glulam specimens.

6.3 Effects of GFRP Reinforcement

Ten beams were tested to failure under four-point bending to investigate the behaviour of reinforced glulam beams with varying reinforcement schemes involving the length of reinforcement, type of bar (i.e., straight vs bent), and knurling profile. Table 6.3 provides a comparison of the average peak loads (P_{max}), corresponding displacements (Δ_{max}), and stiffnesses (K) between the unreinforced and reinforced beams. Irrespective of the reinforcement configurations investigated and failure modes observed, increases in maximum load, displacement at maximum load, and stiffness ranging 1.16 – 1.30, 1.04 – 1.24, and 1.13 – 1.18 were obtained, respectively, relative to the unreinforced specimens. The failure modes observed for the reinforced specimens included simple tension, splintering, cross-grain tension, and shear, or a combination thereof.

Table 6.3: Effects of Reinforcement on Glulam Beams

Specimen Number	Diagram	P_{max}^a (kN)	Δ_{max}^b (mm)	K^c (N/mm)	P_R/P_{UR}	Δ_R/Δ_{UR}	K_R/K_{UR}	Initial Failure Mode
U-01	-	165.7	28.9	6,177	-	-	-	Splintering, Shear
U-02								Splintering
U-03								Shear, Splintering
U-04								Splintering
R1-1		210.8	31.6	7,314	1.27	1.09	1.18	Shear, Splintering
R1-2								Splintering
R1-3		201.2	34.9	7,032	1.21	1.21	1.14	Shear, Simple Tension
R1-4								Shear, Simple Tension
R1-5		215.6	35.8	7,104	1.30	1.24	1.15	Simple Tension
R1-6								Splintering
R2-1		192.5	30.2	7,070	1.16	1.04	1.14	Splintering
R2-2								Shear, Simple Tension
R2-3		212.3	35.9	7,005	1.28	1.24	1.13	Splintering
R2-4								Cross-Grain Tension

^a Maximum Load ^b Deflection at maximum load ^c Bending stiffness

The addition of straight bars, which corresponded to 1.73% of GFRP reinforcement-to-wood area, contributed to shifting the initial failure to one that is dominated by shear (i.e., R1-1, R1-2, and R1-4) followed by secondary failure modes upon further loading.

The introduction of knurling has contributed to higher maximum resistances and associated displacements compared to the other specimens with straight bars and no knurling while also eliminating the shear failure. Considering this observation, the knurling contributed to an enhancement of flexural performance of the GFRP-reinforced glulam beams. The disparity between the benefits in the pullout and flexural tests can partly be attributed to the pullout specimens having a single groove that had the same length as the one for the flexural specimens. As such, scaling of the groove size or larger pullout specimens should be further investigated to comprehend the full effects of knurling on the global performance of the GFRP reinforcement-to-wood bond.

The introduction of bent GFRP bars was investigated as an alternative to knurling for improving the performance of GFRP-reinforced glulam specimens with respect to premature debonding and stress

concentration failures observed in simple tension reinforcement using GFRP fabrics (Shrimpton et al., 2023; Vetter, 2022). From Table 6.3, the overall performance of the glulam beams reinforced with GFRP bent bars appear to have been influenced by the reinforcement length. Specifically, the shorter bar length (i.e., R2-1, R2-2) had on average a lower resistance and displacement at maximum resistance than the specimens reinforced with straight bars (i.e., R1-1 – R1-6) with one of the specimens having a failure dominated by a shear failure. The longer bent bars performed better than the shorter bars with the average resistance and stiffness being in line with those of the specimens reinforced with straight bars and knurling. No shear failure was observed for the specimens with longer bent GFRP bars.

Figures 6.4a and 6.4b present the resistance curves for the specimens reinforced with straight and bent GFRP bars, respectively, relative to the average resistance of the unreinforced beams. Due to the limited post-peak resistance of the unreinforced glulam, only the portion up to peak resistance is included. Irrespective of the reinforcement, it is clearly seen that the addition of reinforcement improved the global response albeit in some cases introducing undesirable failure modes. The following sections will elaborate on the effects of the various reinforcing schemes in more detail.

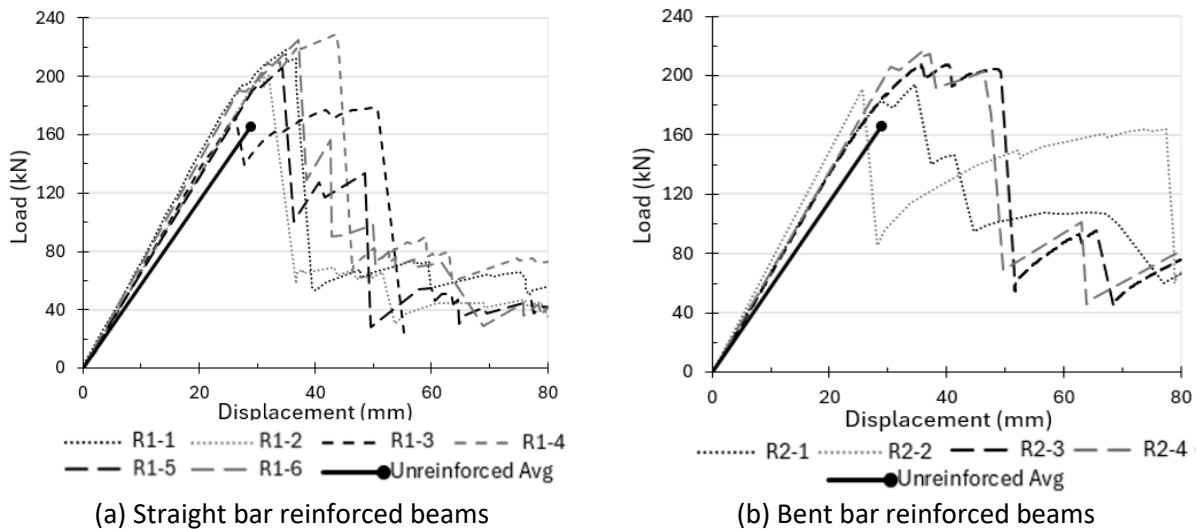


Figure 6.4 Resistance curves relative to average unreinforced beams

The flexural failures in the reinforced beams often stemmed from defects along the beam's edges (Fig. 6.5), possibly more common due to the spacing of the bars on the tension face of the beam being less than the standard requires. The flexural failures observed were as expected, occurring in the maximum moment region and then propagating through the rest of the beam, as described in Chapter 4.



Figure 6.5 Tension failure stemming from edge (R2-3)

6.3.1 Effects of reinforcement length

A key part of this research was to investigate the effects of the reinforcement length on the flexural behaviour of GFRP-reinforced beams. Table 6.4 facilitates the comparison of the effects of reinforcement length on the flexural performance while Figure 6.6 shows the flexural resistance curves. Presented in Table 6.4 is a comparison of the average maximum loads (P_{max}), corresponding displacements (Δ_{max}), and stiffnesses (K) between the specimens with shorter and longer reinforcement lengths for both the straight and bent bar reinforced beams.

Table 6.4: Comparison of Short Versus Long Bar Reinforcement

Specimen Number	Diagram	P_{max}^a (kN)	Δ_{max}^b (mm)	K^c (N/mm)	P_{Long}/P_{Short}	$\Delta_{Long}/\Delta_{Short}$	K_{Long}/K_{Short}	Initial Failure Mode
R1-1		210.8	31.6	7,314	0.95	1.10	0.96	Shear
R1-2								Splintering
R1-3		201.2	34.9	7,032				Shear
R1-4								Shear
R2-1		192.5	30.2	7,070	1.10	1.19	0.99	Splintering
R2-2								Shear
R2-3		212.3	35.9	7,005				Splintering
R2-4								Cross-Grain Tension

^a Maximum Load ^b Deflection at maximum load ^c Bending stiffness

For the beams reinforced with straight bars, the change in length of reinforcement did not contribute to significant improvements in terms of resistance and failure mode with both specimens' failure being dominated by shear. Despite the average maximum resistance and associated displacement being 0.95 and 1.10 times that of the average for beams reinforced with shorter bars, respectively, Figure 6.6a clearly shows distinct behaviour for R1-3 and R1-4, thus affecting the overall average. In the case where the specimen does not fail early (i.e., R1-4), the longer bars appear to allow for the development of a larger resistance and displacement at maximum, although ultimately failing in shear.

The effect of providing longer reinforcement length with the bent GFRP bars is evident with increases of 1.10 and 1.19 in the maximum resistance and failure displacement, respectively, in comparison to the specimens with shorter bars. Figure 6.6b shows the improved behaviour of the specimens reinforced with longer bent bars. As such, three out of the four specimens reinforced with longer bars (i.e., shorter distance from end of reinforcement to support) showed overall improved behaviour relative to the specimens with shorter bar lengths. This observation is in line with the CHBDC (CSA, 2019) that the reinforcement termination point should be as close as possible to the supports.

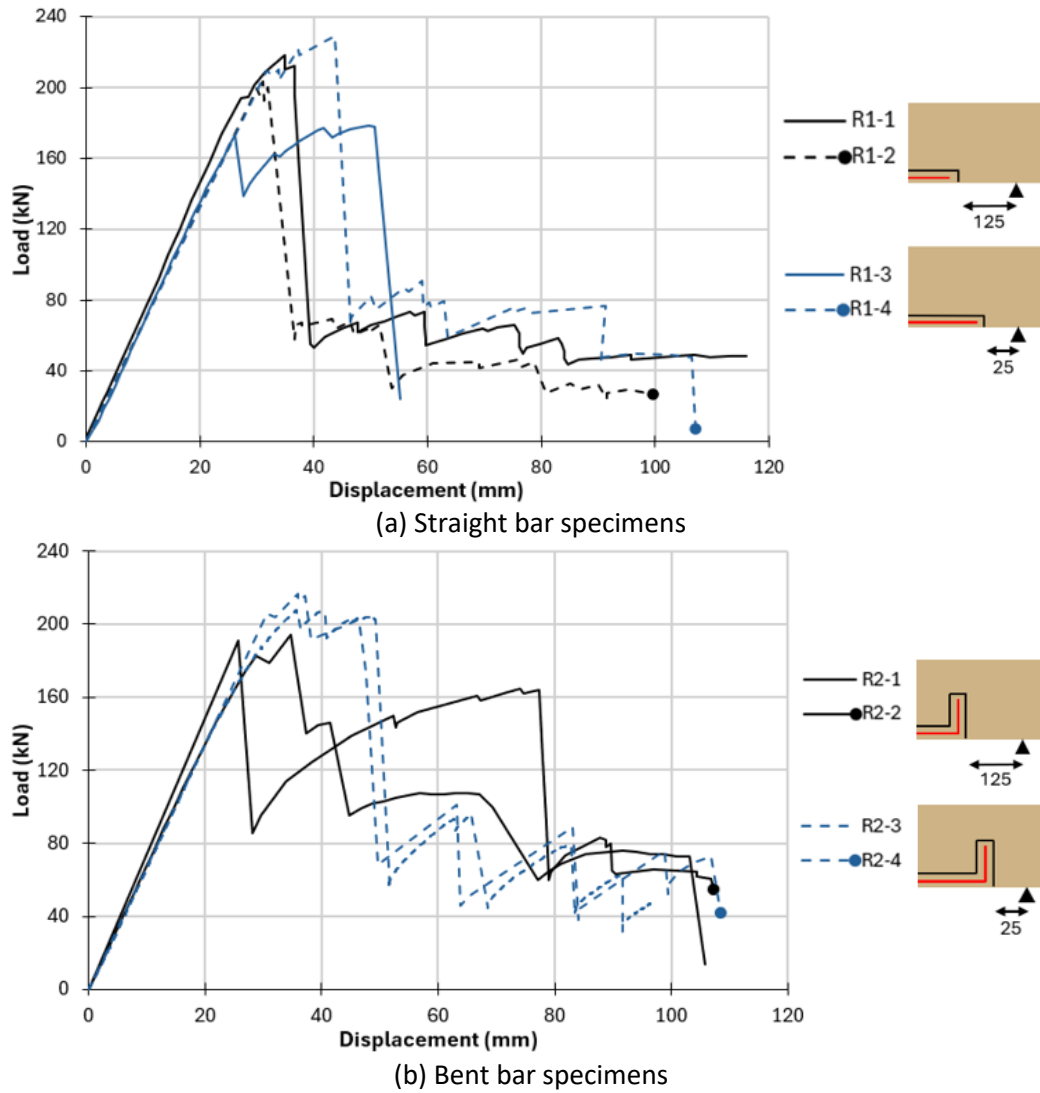


Figure 6.6 Comparison of short versus long bar specimens

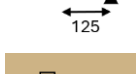
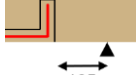
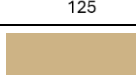
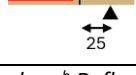
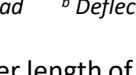
Past research (e.g., Shrimpton et al., 2023) on glulam beams reinforced with GFRP fabrics as simple tension reinforcement has shown that insufficient development length of the reinforcement can lead to stress concentration failures occurring between the support and the end of the FRP reinforcement. The failures occurring at the FRP termination points were not observed by Lacroix and Doudak (2018) who reported debonding of the FRP due to the wood failing in flexure and pushing outwards on the reinforcement. For comparison, the distance between the support and the end of the FRP reinforcement was 105.5mm for Shrimpton et al whereas it was 12.5mm for the other study (Lacroix and Doudak 2018). The current study investigates termination points 125mm and 25mm from the supports, and no initial failures due to stress concentration at the FRP termination point was observed. Several factors can be attributed to the observed behaviour, including the reinforcement type (i.e., bars vs fabrics),

reinforcement installation (i.e., surface mounted for fabrics vs grooves), and finally the aspect ratio of the beams having lower shear strength in the current study.

6.3.2 Effects of straight versus bent bars

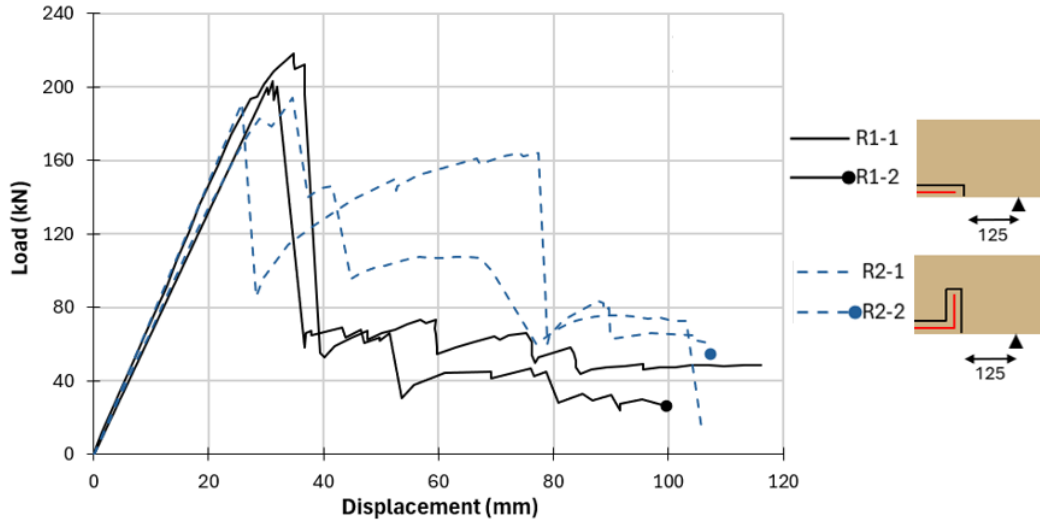
One of the study's hypothesis was that larger distances between the reinforcement termination point and support could result in a failure caused by stress concentrations at the end of the reinforcement akin to those observed in Shrimpton et al. (2023). As such, the effect of bent bars was investigated as a means to provide increased anchoring at the reinforcement termination point. Table 6.5 facilitates the comparison of the effects of reinforcement type on the flexural performance while Figure 6.7 shows the flexural resistance curves. Table 6.5 provides a comparison of the average maximum load (P_{max}), corresponding displacements (Δ_{max}), and stiffnesses (K) between the specimens with straight and bent reinforcement for lengths of reinforcement investigated.

Table 6.5: Comparison of Straight Versus Bent Bar Reinforcement

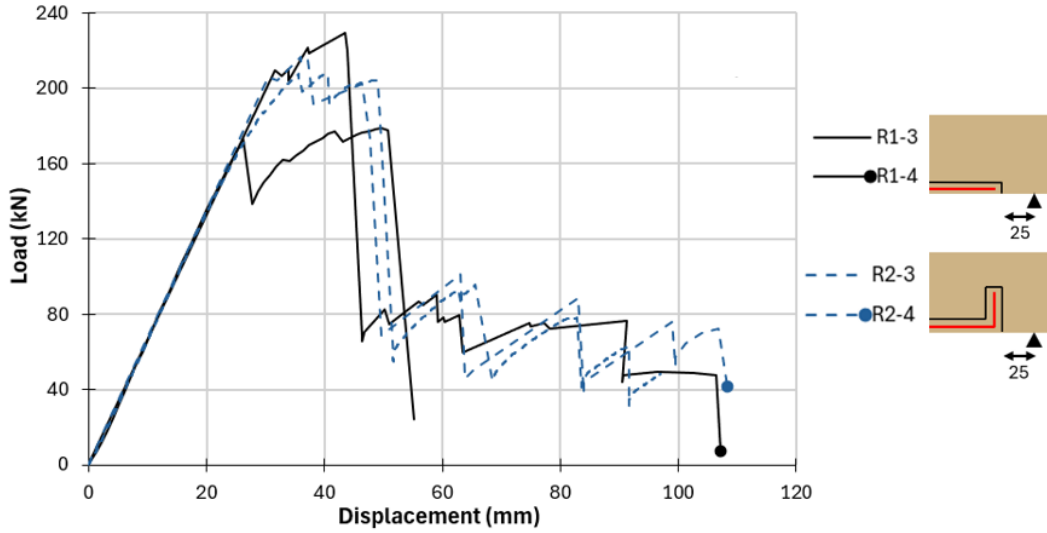
Specimen Number	Diagram	P_{max}^a (kN)	Δ_{max}^b (mm)	K^c (N/mm)	P_{Bent}/P_{Str}	$\Delta_{Bent}/\Delta_{Str}$	K_{Bent}/K_{Str}	Initial Failure Mode
R1-1		210.8	31.6	7,314	0.91	0.96	0.97	Shear
R1-2								Splintering
R2-1		192.5	30.2	7,070				Splintering
R2-2					1.06	1.03	0.99	Shear
R1-3		201.2	34.9	7,032				Shear
R1-4								Shear
R2-3		212.3	35.9	7,005				Splintering
R2-4								Cross-Grain Tension

^a Maximum Load ^b Deflection at maximum load ^c Bending stiffness

For the shorter length of bars, the bent bars did not contribute to an improved resistance achieving lower maximum resistances corresponding to 0.91 times that of the straight bars. However, the longer bent bars provided slight improvement by factors of 1.06 and 1.03 with respect to the maximum load and failure displacement, respectively. Additionally, the change from straight to bent bars resulted in a shift in failure mode from shear to flexure for the longer length of reinforcement.



(a) Short bar specimens



(b) Long bar specimens

Figure 6.7 Comparison of straight versus bent bar specimens

In general, it can be seen that following the initial failure, that the specimens with bent bars show higher levels of post-peak resistance (Fig. 6.7). This is particularly true for the specimens with the longer bent bars (i.e., R2-3 and R2-4) where the initial drops in resistance are less than 10% of the maximum value. Also, it should be noted that the post-peak behaviour for R2-2 is attributed to the fact that it failed in shear, which is not a desirable failure mode. The increase in levels of post-peak behaviour can be seen when comparing the behaviour of the reinforcement ends as shown in Figure 6.7. For the straight bars, upon initial failure and further loading, it can be seen that the straight bars tend to slip (Fig. 6.8a) creating a gap in the groove where they used to sit. On the other hand, the bent bars are observed to yield (Fig.

6.8b) due to the additional anchoring provided. The additional force provided with the U-shaped bent bars can be seen in the failure of the wood, which is similar to group-tear-out in bolted connections (CSA, 2024). Therefore, while having reinforcement as close as possible to the members' end, additional consideration is required in terms of secondary failures modes.

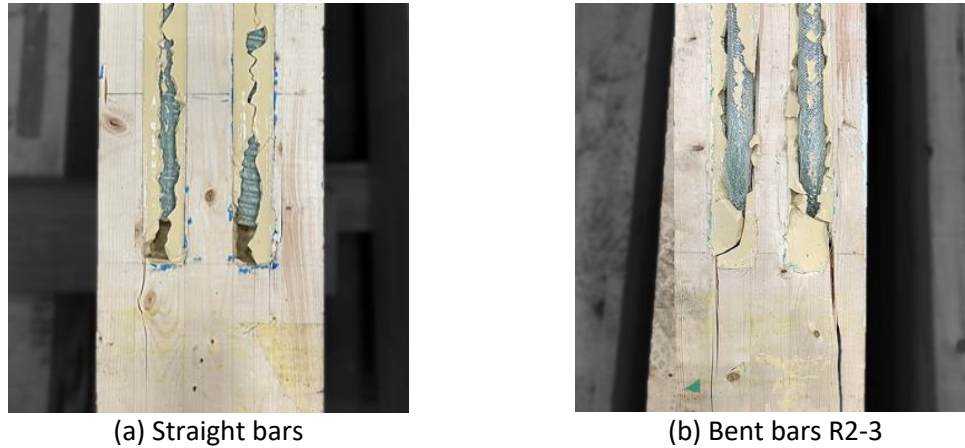


Figure 6.8 Reinforcement end behaviour

6.3.3 Effects of groove pattern on FRP reinforced glulam beams

Knurling was investigated as a potential alternative to improve the flexural behaviour of glulam beams reinforced with GFRP bars with straight grooves only. Table 6.6 facilitates the comparison of the effects of knurling on the flexural performance while Figure 6.9 shows the flexural resistance curves. Table 6.6 provides a comparison of the average peak loads (P_{max}), corresponding displacements (Δ_{max}), and stiffnesses (K) between the straight bar reinforced beams with and without knurling.

The specimens with an alternating groove pattern (i.e., R1-5, R1-6) showed increases in peak resistance and failure displacement of 1.07 and 1.03, respectively, in comparison to those with the same reinforcement type but no knurling (i.e., R1-3, R1-4). In terms of failure modes observed, both alternating groove specimens failed in flexure, suggesting that the alternating groove pattern caused a change in failure mode from shear to flexure. Although no significant improvements were seen with the varied groove pattern when tested in tension, the behaviour in flexure was uncertain and therefore investigated. The difference in the observed behaviour between the pullout and flexural tests can partly be attributed to the pullout specimens having a single groove of additional depth of the same length as the groove for the flexural specimens. As such, scaling of the groove size or larger pullout specimens should be further investigated to better understand the full effects of knurling.

Table 6.6: Comparison of Straight Versus Alternating Groove Pattern

Specimen Number	Diagram	P_{max}^a (kN)	Δ_{max}^b (mm)	K^c (N/mm)	P_{Alt}/P_{Str}	$\Delta_{Alt}/\Delta_{Str}$	K_{Alt}/K_{Str}	Initial Failure Mode
R1-3		201.2	34.9	7,032	1.07	1.03	1.01	Shear
R1-4								Shear
R1-5		215.6	35.8	7,104				Simple Tension
R1-6								Splintering

^a Maximum Load ^b Deflection at maximum load ^c Bending stiffness

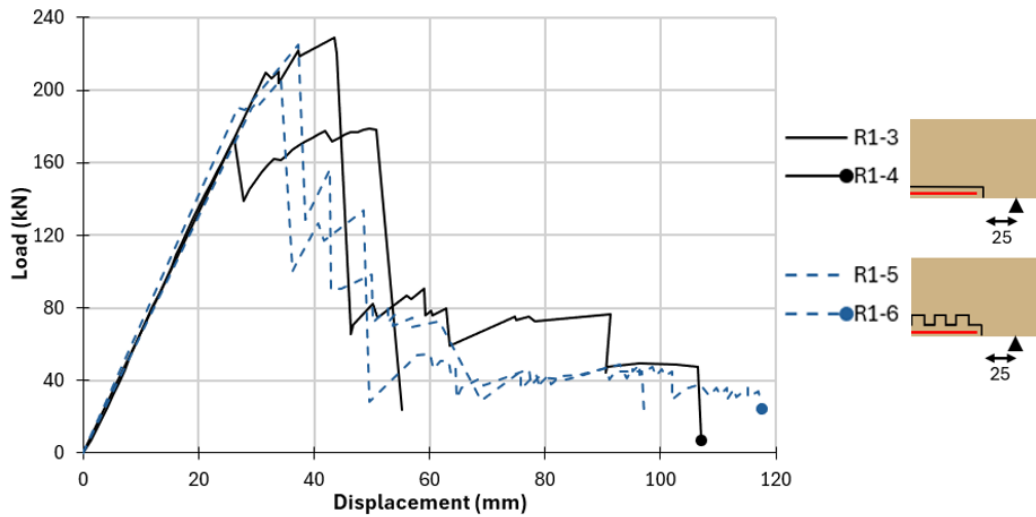


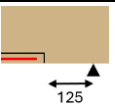
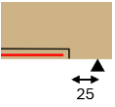
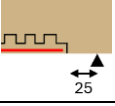
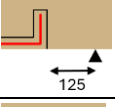
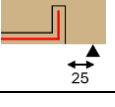
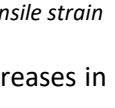
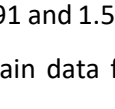
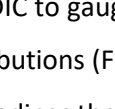
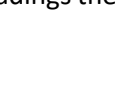
Figure 6.9 Comparison of specimens with varied groove pattern

6.3.4 Effects of reinforcement on strain distribution

Through the course of the research on the full-scale specimens, a combination of strain gauges and digital image correlation (DIC) analysis were used to study the effects of strain distribution. For the DIC analysis, all unreinforced and reinforced glulam beams were speckled from one side of the support up to mid-span. In terms of strain gauges, a total of two and three gauges were used on the unreinforced and reinforced beams, respectively. The strain at mid-span of the wood tension and compression face were instrumented along with one bar for the reinforced specimens. While strain gauges offer a localized measurement, the maximum strains using the DIC were obtained by taking the average along the tension and compression faces from mid-span to the loading point, where the moment is constant. In theory, this should provide average strains.

Table 6.7 presents a summary of the wood tensile and compressive strains of the unreinforced and GFRP-reinforced beams, including the largest recorded mid-span tensile and compressive strains (i.e., gauge), and the largest recorded tensile and compressive strains determined through the DIC analysis.

Table 6.7: Summary of Strains

Specimen Number	Diagram	$\epsilon_{t,max}^a$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c,max}^b$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{t,max-DIC}^c$ $\times 10^{-3}$ (mm/mm)	$\epsilon_{c,max-DIC}^d$ $\times 10^{-3}$ (mm/mm)	R/UR Tension (Gauge)	R/UR Comp (Gauge)	R/UR Tension (DIC)	R/UR Comp (DIC)
U-01	-	3.50	-5.90	4.90	-5.10	-	-	-	-
U-02									
U-03									
U-04									
R1-1		5.36	-3.81	4.98	-5.08	1.53	0.66	1.02	1.00
R1-2		5.71	-7.13	3.80	-5.84	1.63	1.21	0.78	1.15
R1-3		2.84	-5.41	3.89	-3.97	0.81	0.92	0.79	0.78
R1-4		4.42	-4.73	3.32	-8.21	1.26	0.80	0.68	1.61
R1-5		3.94	-4.27	3.09	-6.71	0.99	0.72	0.63	1.31
R1-6		5.05	-14.8	5.09	-10.8	1.44	2.51	1.04	2.11
R2-1		5.47	-4.04	3.99	-6.79	1.56	0.68	0.81	1.33
R2-2		8.87	-5.27	7.27	-5.74	2.53	0.89	1.48	1.13
R2-3		5.73	-3.25	4.68	-9.55	1.63	0.55	0.96	1.87
R2-4		3.73	-12.7	4.25	-14.0	1.06	2.15	0.87	2.75
Average						1.44	1.11	0.91	1.50

^a Maximum tensile strain ^b Maximum compressive strain ^c Maximum tensile strain (DIC) ^d Maximum compressive strain (DIC)

Average increases in maximum tensile and compressive strain by factors of 1.44 and 1.11 for the gauge data and 0.91 and 1.5 for the DIC data, respectively. Figure 6.10 presents a comparison of the strain gauge and DIC strain data for an unreinforced and GFRP-reinforced specimen. On average, the ratio of the maximum DIC to gauge for the tension and compression wood strains is 0.90 and 1.37, respectively. The strain distributions (Figs 6.10a-b) show that difference between gauge and DIC strains is likely due to the localized readings the strain gauges provide versus the global behaviour the DIC system is able to capture.

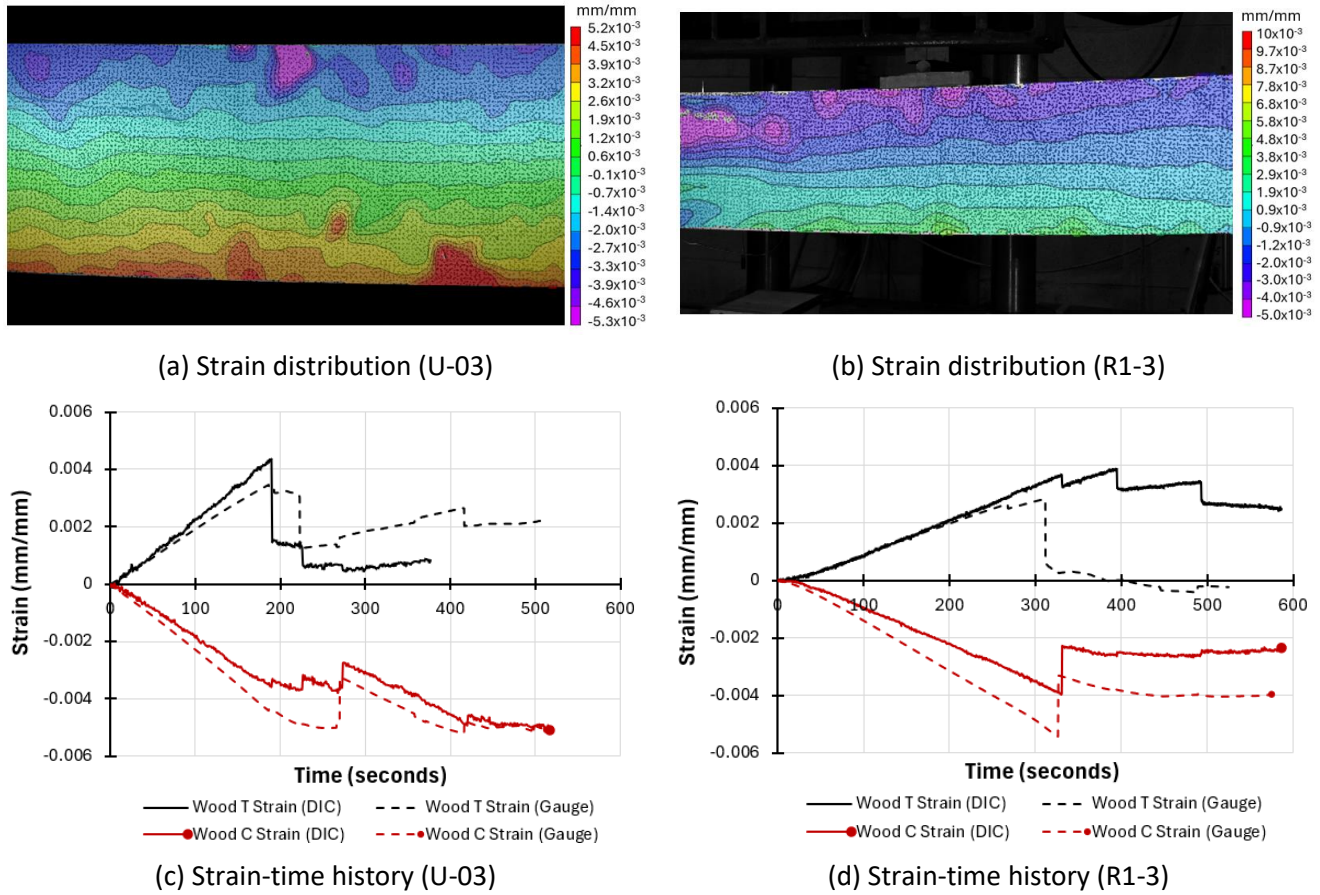


Figure 6.10 Effects of reinforcement on strain distribution

6.4 Predicting the Flexural Behaviour of Glulam Beams

The material model presented in Chapter 5 was used to create predictive moment-curvature and resistance curves for both the unreinforced and reinforced glulam beams. The following sections present comparisons between the predictions and experimental results.

6.4.1 Determining effects of reinforcement on strain distribution

The use of FRP as means for reinforcing wooden members has been shown to increase the tensile failure strains in comparison to unreinforced members. As discussed in Chapter 2, Gentile (2002) introduced the use of a modification factor α_m purposed to account for the higher tensile failure strains observed in FRP reinforced wood members, and since further research has been done (D. Lacroix & Doudak, 2018; Shrimpton et al., 2023; Yang, Liu, et al., 2016a).

Strain gauges provide readings limited to a specific point, whereas through conducting a DIC analysis, the strains across a larger area are available. The average tensile failure strains from the strain gauges and DIC analysis are presented in Table 6.8. Only beams that failed in flexure were considered, and beams failing in shear were excluded as they did not benefit from the enhanced tensile strain provided by the FRP. By comparing the ratio of reinforced to unreinforced tensile failures strains, an average increase of 1.5 is obtained using the strain gauges while a decrease of 0.8 is observed using the DIC strains. The discrepancy between the results using strain gauges and DIC warrants further research. It should be noted that strain gauges have successfully been used in the past to define α_m values (e.g., Lacroix and Doudak 2018, 2020, Vetter 2022) in the order of 1.17 – 1.21 for glulam reinforced with GFRP fabrics. For heavy timber beams reinforced with GFRP bars, Gentile et al. (2002) calibrated half-scale beams to experimental results and obtained an α_m factor of 1.3. However, given the discrepancy and the mixed failure modes involving shear as opposed to clear flexural failures in cited studies, an α_m factor of unity will be used.

Table 6.8: Wood Tensile Failure Strains

Specimens	Average Tensile Failure Strain (Strain Gauge) $\times 10^{-3}$ (mm/mm)	Reinforced / Unreinforced	Average Tensile Failure Strain (DIC) $\times 10^{-3}$ (mm/mm)	Reinforced / Unreinforced
U-01, U-02, U-04	3.4	-	5.0	-
R1-2	5.7	1.7	3.8	0.8
R1-5, R1-6	4.5	1.3	4.1	0.8
R2-1	5.5	1.6	4.0	0.8
R2-3, R2-4	4.7	1.4	4.5	0.9
Average		1.5		0.8

6.4.2 Predicting the behaviour of unreinforced glulam beams

The flexural behaviour of the unreinforced beams was predicted using the methodology described in Chapter 5. A moment-curvature analysis was initially conducted and then used to determine the resulting resistances for both models proposed in Chapter 5. The results are presented in Table 6.9 and Table 6.10 for the models with the MOE determined through coupon (i.e., Approach 1) and flexural tests (i.e., Approach 2), respectively, while the resistance curves are shown in Figure 6.11.

Table 6.9: Comparison of Results for Unreinforced Beams (Coupon Test MOE)

Specimen	Experimental		Model		Model/Experimental	
	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)
U-01	189.2	30.3	157.4	30.9	0.83	1.02
U-02	167.3	28.9			0.94	1.07
U-03	144.0	26.2			1.09	1.18
U-04	162.4	30.0			0.97	1.03
Average					0.96	1.08
St. Dev.					0.09	0.06
COV					0.09	0.05

Table 6.10: Comparison of Results for Unreinforced Beams (Flexural Test MOE)

Specimen	Experimental		Model		Model/Experimental	
	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)
U-01	189.2	30.3	165.5	34.2	0.87	1.13
U-02	167.3	28.9			0.99	1.18
U-03	144.0	26.2			1.15	1.31
U-04	162.4	30.0			1.02	1.14
Average					1.01	1.19
St. Dev.					0.10	0.07
COV					0.10	0.06

For the maximum resistances and corresponding displacements, average ratios of 0.96 and 1.08 were determined for the model using the adjusted coupon test results, and 1.01 and 1.19 for the model using the flexural test MOE. The model with inputs determined from the coupons test captures the average peak load well, and the model with inputs determined from the flexural tests captures the stiffness accurately (Fig. 6.11). The discrepancies between the experimental and modelled results for displacement can be attributed to the variability in the wood properties used as material model inputs and the adjustments for size-effects.

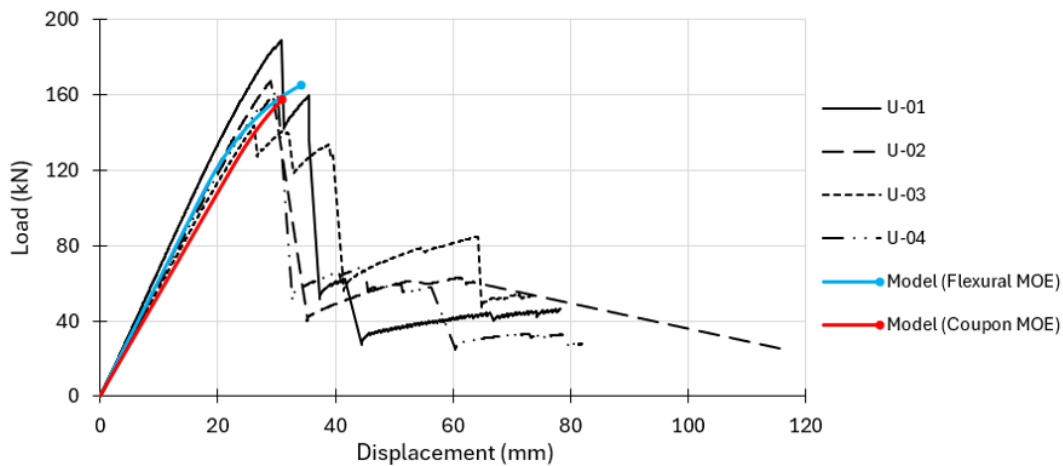


Figure 6.11 Modelled and experimental resistance curves for unreinforced beams

6.4.3 Predicting the behaviour of FRP-reinforced glulam beams

The resistance curves for the FRP-reinforced glulam beams were predicted as per the procedure described in Chapter 5. The results are presented in Table 6.11 and Table 6.12 for the models with the MOE determined through coupon and flexural tests, respectively, and the resistance curves are shown in Figure 6.12 and Figure 6.13.

Table 6.11: Comparison of Results for Reinforced Beams (Coupon Test MOE)

Specimen	Experimental		Model		Model/Experimental	
	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)
R1-1	218.8	34.9	198.3	42.1	0.91	1.21
R1-2	203.4	28.2			0.97	1.49
R1-3	173.1	26.2			1.15	1.61
R1-4	229.2	43.6			0.87	0.96
R1-5	205.3	34.3			0.97	1.23
R1-6	225.3	37.2			0.88	1.13
R2-1	194.0	34.6	200.4	48.2	1.02	1.22
R2-2	191.0	25.7			1.04	1.64
R2-3	207.6	35.7			0.96	1.18
R2-4	216.9	36.0			0.91	1.17
Average					0.97	1.30
St. Dev.					0.08	0.21
COV					0.16	0.16

Table 6.12: Comparison of Results for Reinforced Beams (Flexural Test MOE)

Specimen	Experimental		Model		Model/Experimental	
	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)	P _{max} (kN)	Δ _{max} (mm)
R1-1	218.8	34.9	200.4	48.2	0.92	1.38
R1-2	203.4	28.2			0.99	1.71
R1-3	173.1	26.2			1.16	1.84
R1-4	229.2	43.6			0.81	1.11
R1-5	205.3	34.3			0.98	1.41
R1-6	225.3	37.2			0.89	1.30
R2-1	194.0	34.6			1.03	1.39
R2-2	191.0	25.7			1.05	1.88
R2-3	207.6	35.7			0.97	1.35
R2-4	216.9	36.0			0.92	1.34
Average					0.98	1.48
St. Dev.					0.08	0.24
COV					0.08	0.16

For the maximum resistances and corresponding displacements average ratios of 0.97 and 1.30 were determined for the model using the adjusted coupon test results, and 0.98 and 1.48 for the model using the flexural test MOE. The model with inputs determined from coupon testing (Figure 6.12) capture the

average failure load well; however, underpredict the stiffness. The model with inputs determined from the flexural tests (Figure 6.13) represents the stiffness better while still capturing the average failure load well; however, it is overpredicting the failure displacement. The discrepancies between the experimental and modelled results can be attributed to the variability in the wood properties used as material model inputs and size-effects, as well as the mixed failure modes observed. In general, despite the predicted-to-experimental displacement ratios ranging 1.3 – 1.48, the two approaches generally capture the behaviour of the stronger specimens.

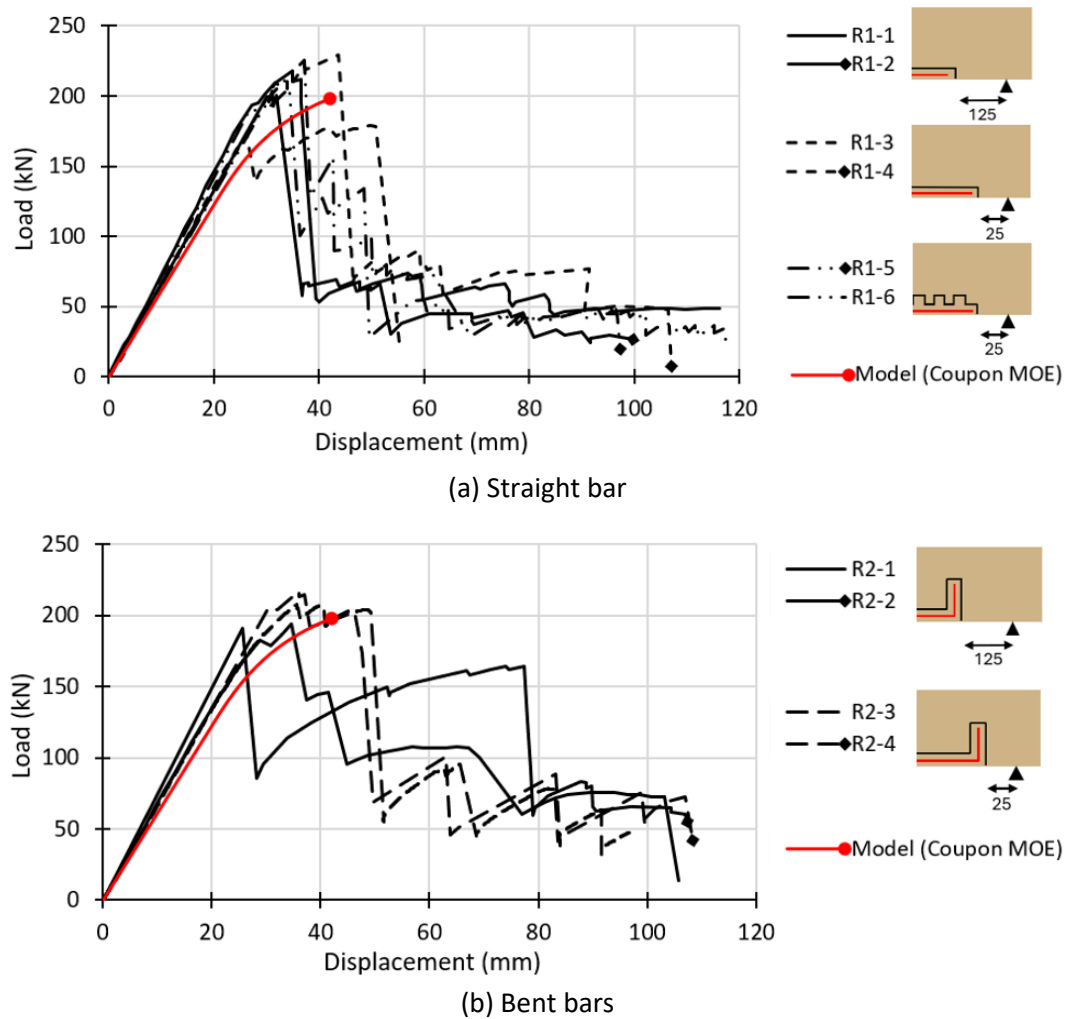


Figure 6.12 Modelled and experimental resistance curves for reinforced specimens (coupon MOE)

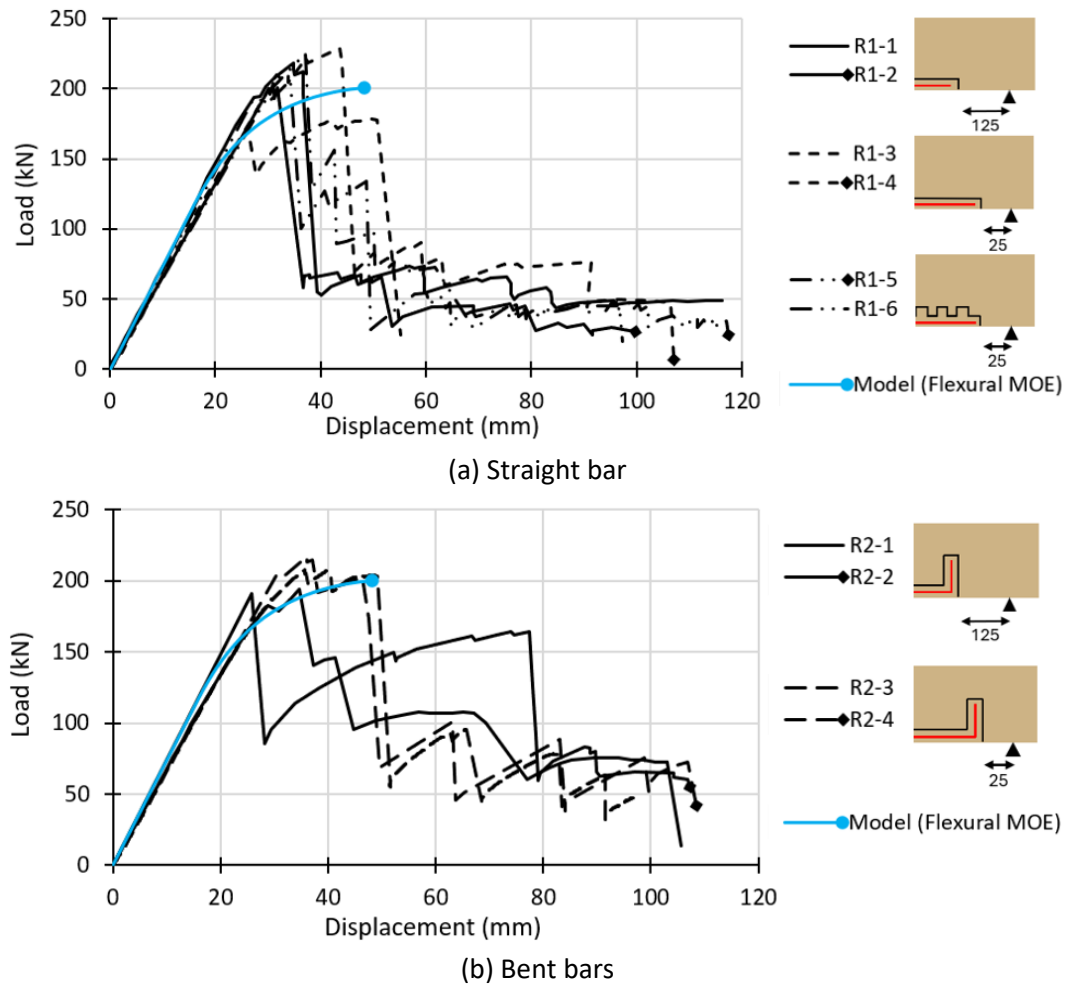


Figure 6.13 Modelled and experimental resistance curves for reinforced specimens (flexural MOE)

6.5 Practical Applications of GFRP Reinforced Glulam Beams

The reinforcement configurations investigated all required that the beams be upside down during the fabrication (i.e., routing, installation of FRP, curing of adhesive) while the intended use consists of glulam beam subjected to positive bending moment with GFRP reinforcement on the tension side. While this may not be practical in an in-situ rehabilitation, the reinforcement configurations investigated, and findings are applicable to hybrid FRP-reinforced glulam manufactured in the factory. This is particularly relevant to cases where optimization of girder size is required (e.g., Western Washington University Gymnasium) (Gilham & Williamson, 2007). It should be noted that if the reinforcement configurations studied herein were to be employed in practice, a wooden bumper layer would be provided to protect the reinforcement and provide adequate fire protection.

Furthermore, the current study has highlighted the importance of detailing and influence of key parameters such as adhesive type, reinforcement length, reinforcement profile, and knurling can have on the failure modes. Currently, the design guidelines provided in the current Canadian codes (CSA, 2019, 2024) are prescriptive and are not based on mechanics nor materials models. The mixed failure modes observed herein, which consisted of shear, flexure, or a combination thereof, clearly highlight the need for fundamental research aimed at establishing the overall interaction between the different failure modes and what is desired in design. For example, it is critical to understand the effects of the distance between the reinforcement termination point and the support (e.g., potential premature failure, group tear-out with bent bars in Fig. 6.8). In the case of group tear-out (Fig. 6.8), simply saying that the reinforcement needs to be as close as possible to the supports, by introducing bent bars, secondary failure mechanisms are introduced. Therefore, fundamental research to establish the baseline performance and relationships between the different materials need to be accounted for.

For the case of in-situ rehabilitation, future research should develop reinforcement strategies leveraging the potential of GFRP bars by developing techniques where smaller sections or laminates can be manufactured in-house and installed on-site. In sum, the research has highlighted the potential of GFRP bars as flexural reinforcement and further research is required for the development of design guidelines informed by principles of mechanics and material predictive models.

CHAPTER 7 | CONCLUSIONS

7.1 Conclusions

The current research program included small-scale and full-scale testing, aimed at investigating the effects of reinforcement configuration on FRP-reinforced glulam beams. The first of three experimental phases of testing was purposed to determine the material properties, in which wood tensile and compressive tests were completed. In the second phase, a series of eighteen pullout tests were conducted to investigate the effects of different adhesives and knurling on bond strength. Fourteen glulam beams were tested to failure under four-point static bending, including four unreinforced beams and ten beams with varying configurations of GFRP bar reinforcement focused on key parameters such as reinforcement length, straight versus bent bars, and straight versus alternating grooves. A predictive material model was developed and compared to the analytical results. The analytical and experimental results, including the mechanical properties, failure modes and predicted resistance curves were presented and discussed.

The following conclusions can be drawn from the current study:

- The texture and density of an adhesive used to bond FRP and wood affects the overall behaviour of the pullout tests, where the observed failure strength was 1.42 times larger for the fluid adhesives in comparison to the dense specimens.
- Knurling offers the potential for improvements in bond strength; however, the alternating pattern does not appear to play a critical role on the maximum load as much as the adhesive itself.
- Unreinforced glulam beams generally failed in flexure, although they were close to the critical ratio of producing a combination of shear on flexural failures.
- Irrespective of reinforcement configuration, the addition of 1.73% GFRP reinforcement provided increases in maximum load, displacement at maximum load, and stiffness ranging between 1.16 – 1.30, 1.04 – 1.24, and 1.13 – 1.18 relative to the unreinforced specimens.
- The effects of reinforcement length and termination point on the behaviour of beams reinforced with straight bars did not show significant improvements in resistance or stiffness and did not prove to have an effect on failure mode; however, for the beams reinforced bent bars improvements in maximum resistance and stiffness of 1.10 and 1.19, respectively, were observed with longer reinforcement lengths in comparison to the specimens with shorter bars.

- The effects of bar profile on the behaviour of GFRP-reinforced glulam beams did not contribute to improvements in resistance for the shorter length of bars, although for the longer length of bars the change from straight to bent reinforcement provided slight improvement by 1.06 and 1.03 with respect to the maximum load and failure displacement, respectively. Additionally, for the longer length of reinforcement the change from straight to bent bars resulted in a shift in failure mode from shear to flexure.
- The addition of knurling showed increases in peak resistance and failure displacement of 1.07 and 1.03, respectively, for the GFRP-reinforced glulam beams in comparison to those without knurling. The beams with alternating groove patterns failed in flexure, suggesting that the addition of knurling caused a change in failure mode from shear to flexure.
- The effects of FRP reinforcement on the tensile failure strain of the glulam beams were observed to be minimal through the analysis done with digital image correlation in comparison to the strain gauge results. This can be attributed to the strain gauges providing a localized reading while the DIC system was able to obtain to capture the strain profile across a large area.
- A material model capable of predicting the flexural behaviour of unreinforced and GFRP-reinforced glulam beams was developed with reasonable accuracy based on two different approaches.

7.2 Recommendations for Future Work

Based on the research that has been conducted in the current thesis, the following recommendations for future work are made:

- Different patterns and depths of knurling to further investigate the effects on bond strength for both pullout out and flexural tests.
- Increase sample size for all reinforcement configurations investigated to validate and improve upon the observations and analysis methods.
- Variation of specimen geometry, cross-section, length, and span-to-depth ratio, to validate and improve upon observations and analysis methods.
- Development of analysis methods based on more precise wood-compression models to verify if and to what degree model simplifications are permissible.

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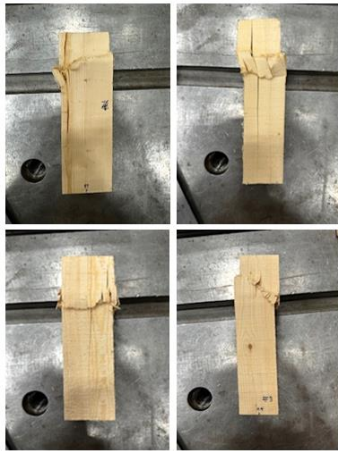
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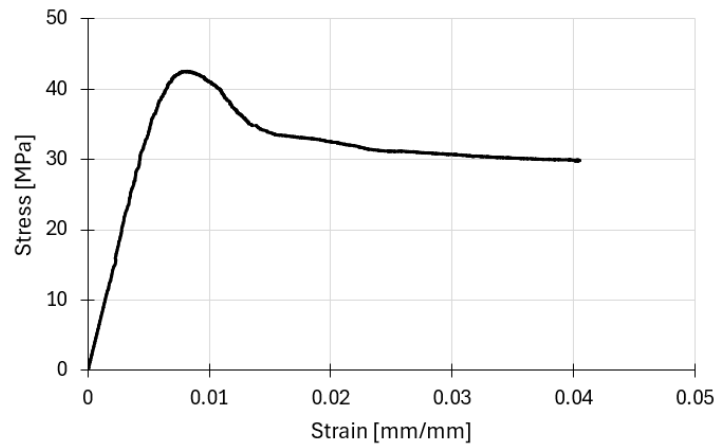
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APPENDIX A | Test Results of Wood Compression Coupons

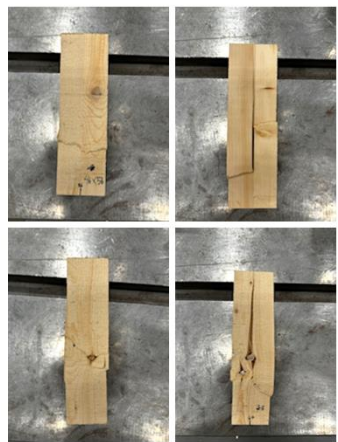


(a) Failed specimen

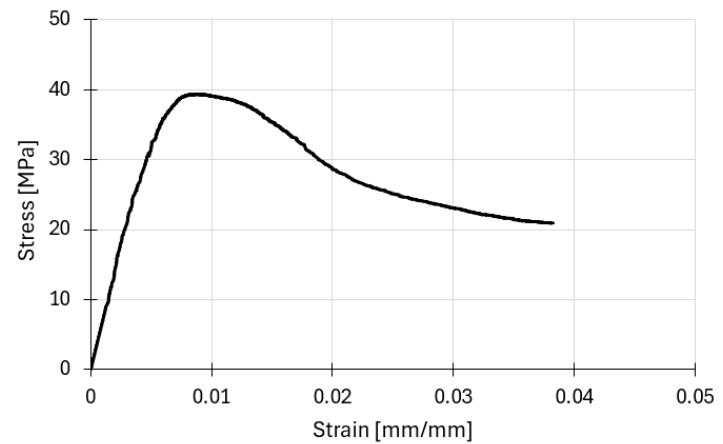


(b) Stress-strain behaviour

Figure A.1 Experimental results for C-01

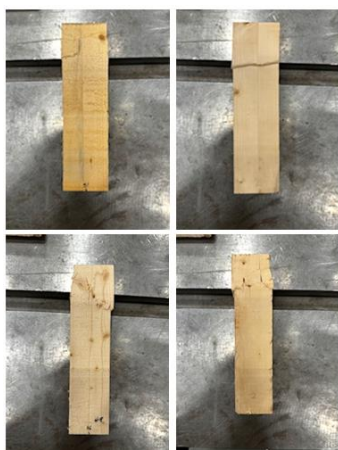


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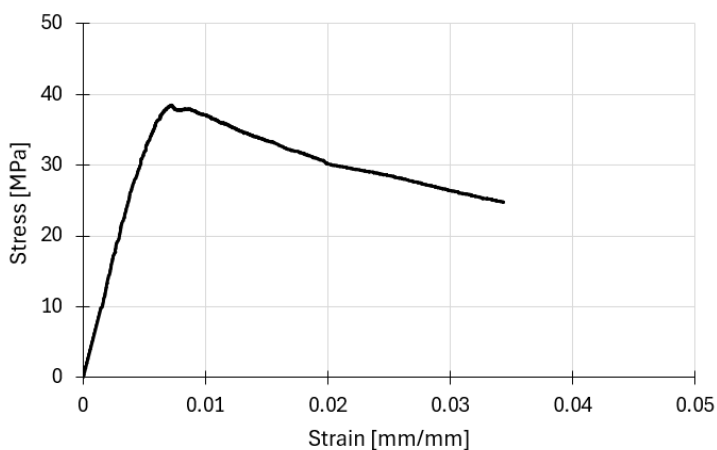


(b) Stress-strain behaviour

Figure A.2 Experimental results for C-02

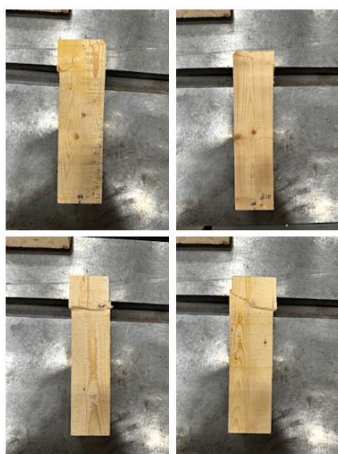


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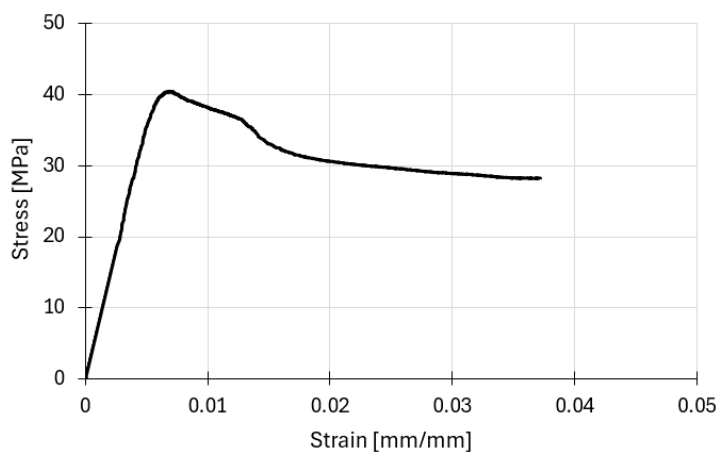


(b) Stress-strain behaviour

Figure A.3 Experimental results for C-03

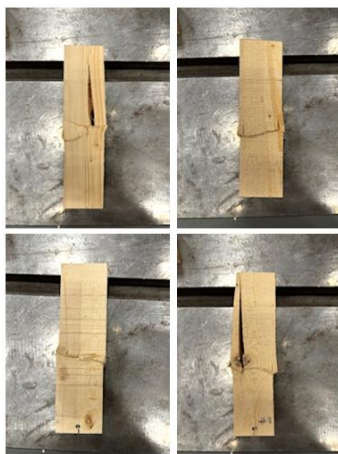


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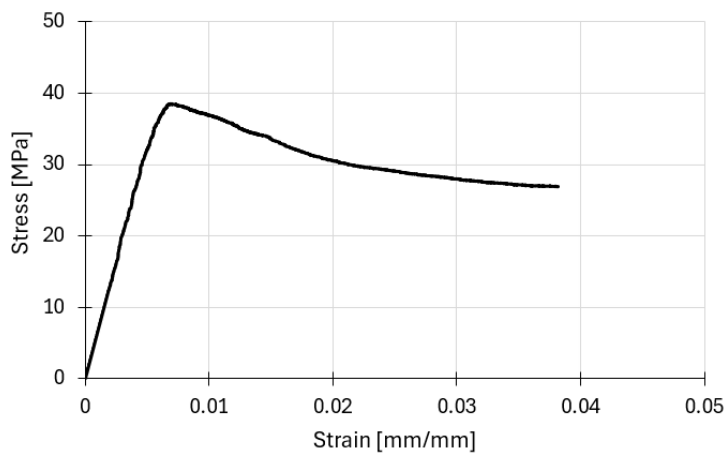


(b) Stress-strain behaviour

Figure A.4 Experimental results for C-04

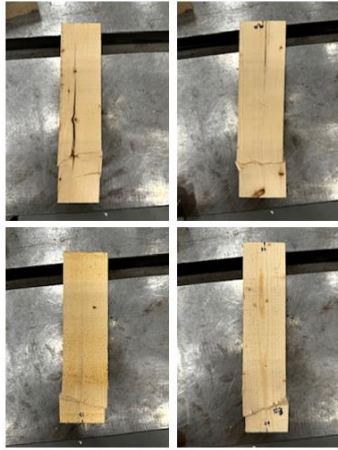


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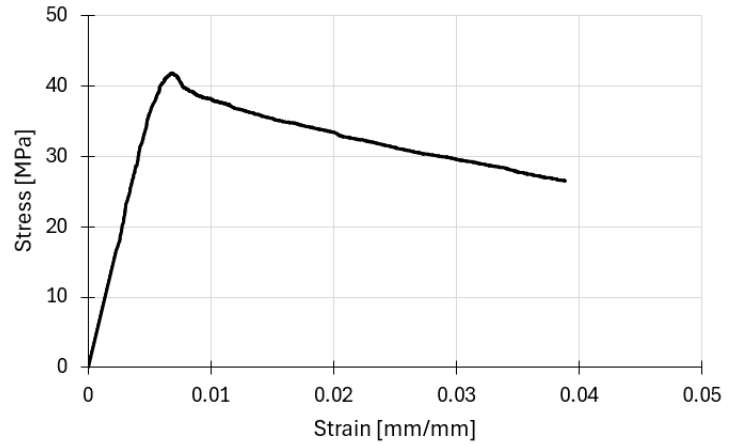


(b) Stress-strain behaviour

Figure A.5 Experimental results for C-05

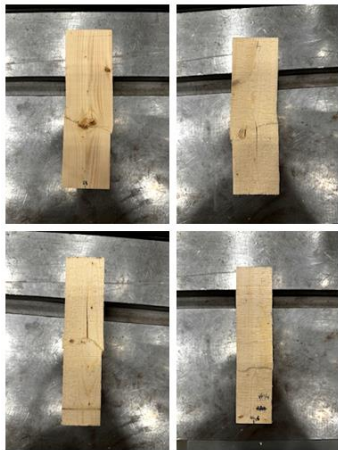


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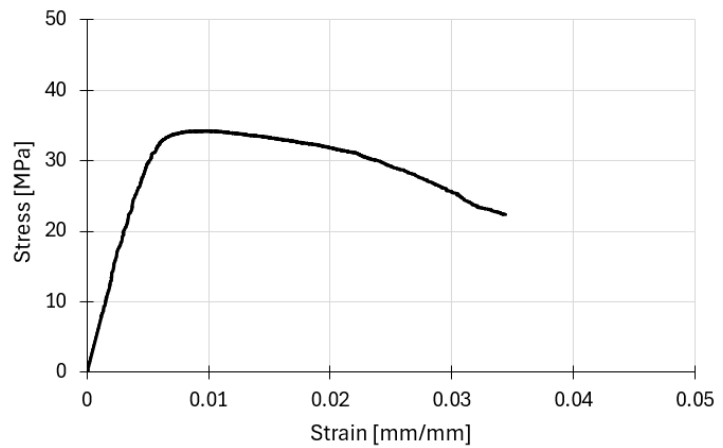


(b) Stress-strain behaviour

Figure A.6 Experimental results for C-06

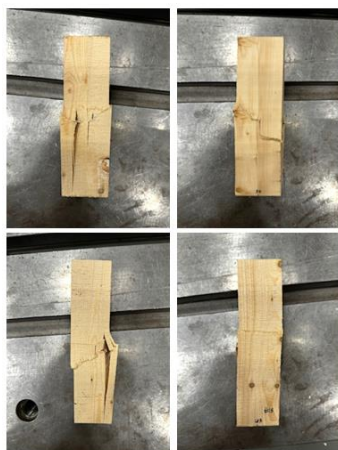


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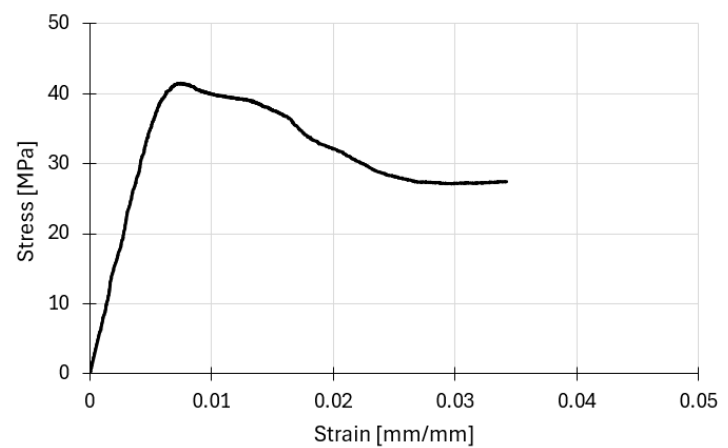


(b) Stress-strain behaviour

Figure A.7 Experimental results for C-07



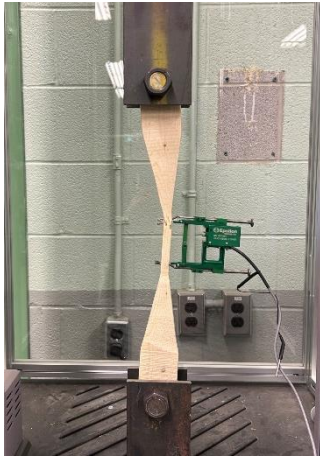
(a) Failed specimen



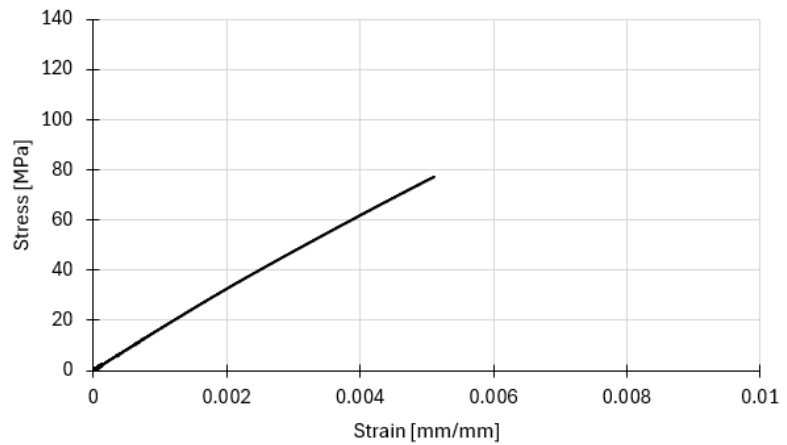
(b) Stress-strain behaviour

Figure A.8 Experimental results for C-08

APPENDIX B | TEST RESULTS OF WOOD TENSION COUPONS

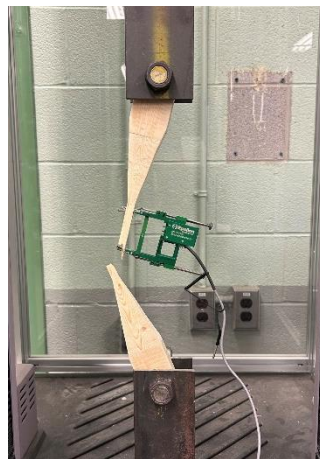


(a) Failed specimen

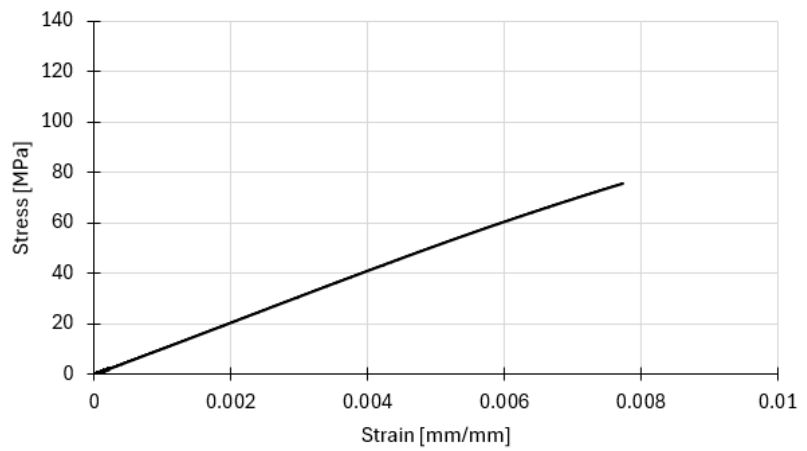


(b) Stress-strain behaviour

Figure B.1 Experimental results for T-01

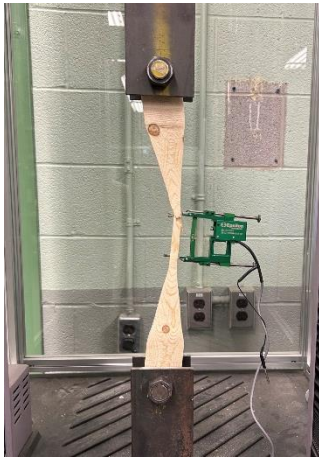


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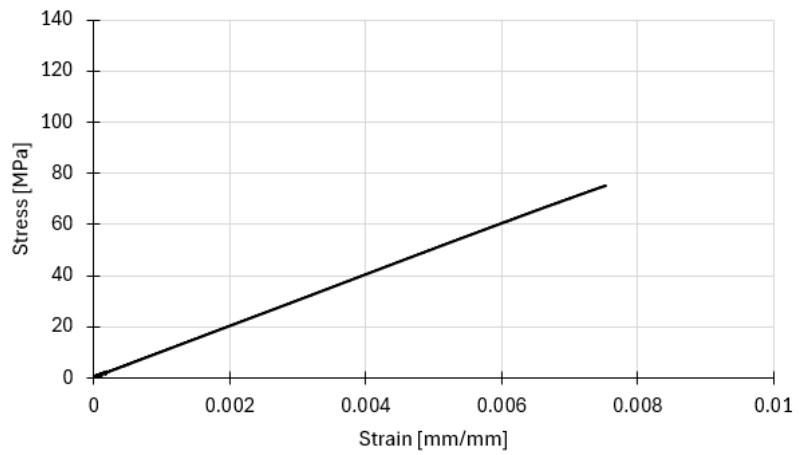


(b) Stress-strain behaviour

Figure B.2 Experimental results for T-02

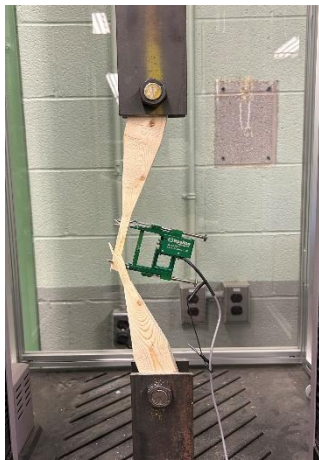


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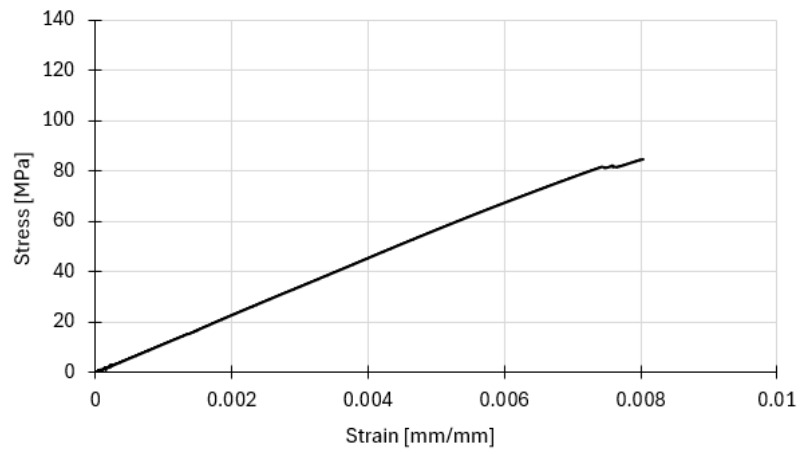


(b) Stress-strain behaviour

Figure B.3 Experimental results for T-03

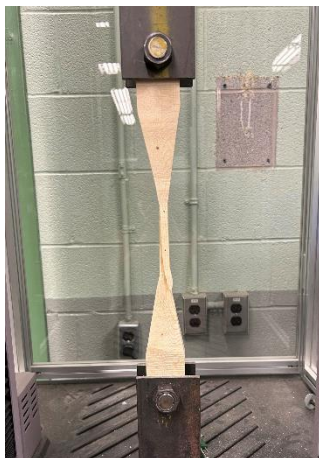


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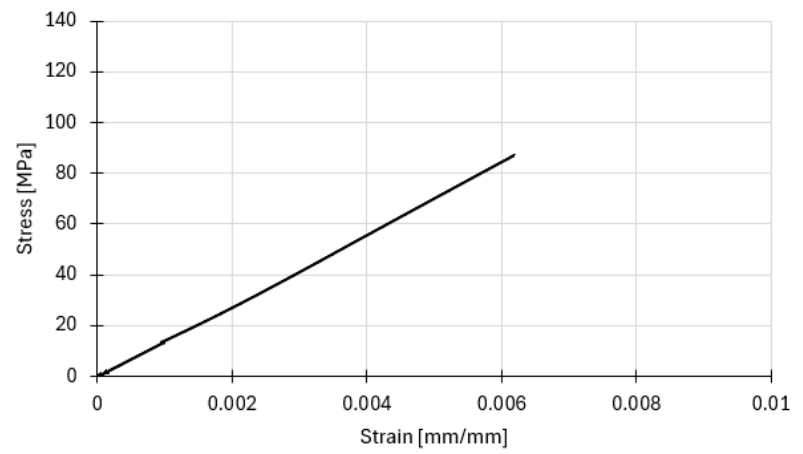


(b) Stress-strain behaviour

Figure B.4 Experimental results for T-04

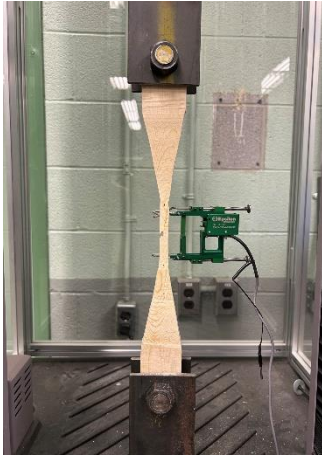


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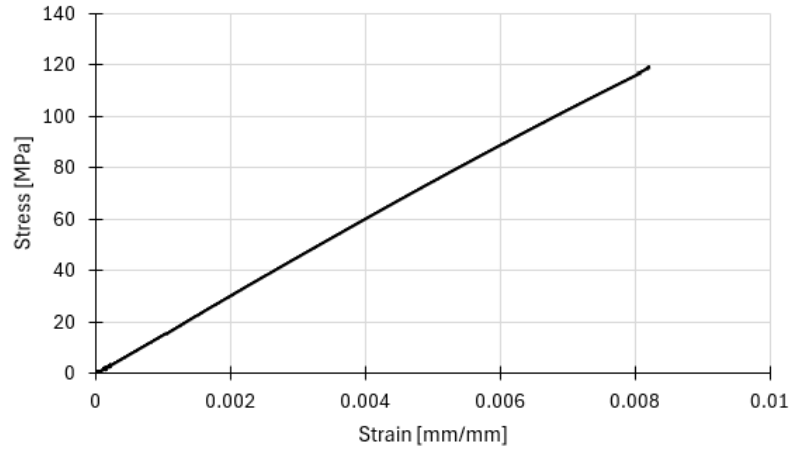


(b) Stress-strain behaviour

Figure B.5 Experimental results for T-05

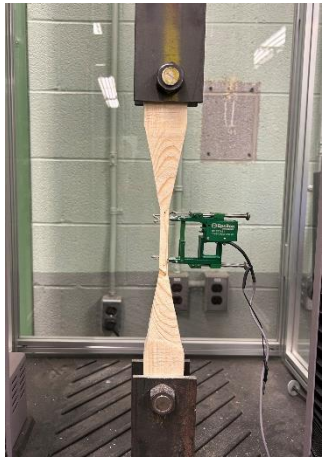


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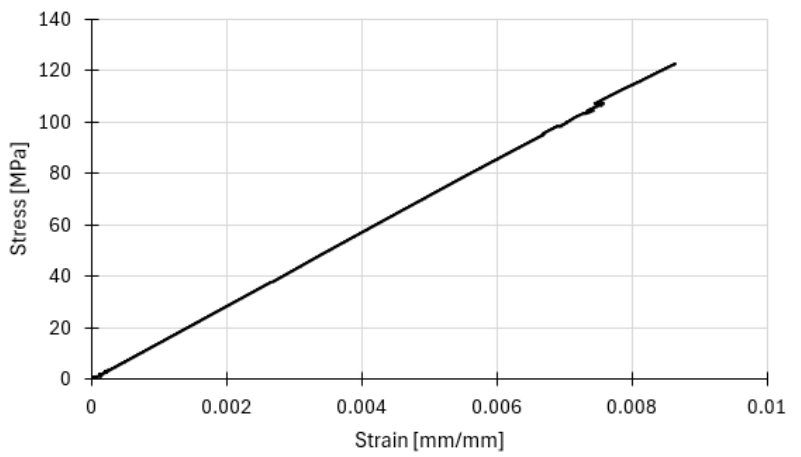


(b) Stress-strain behaviour

Figure B.6 Experimental results for T-06

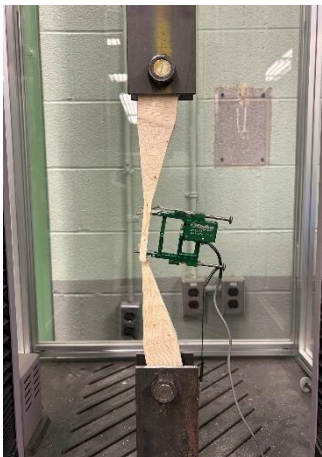


(a) Failed specimen

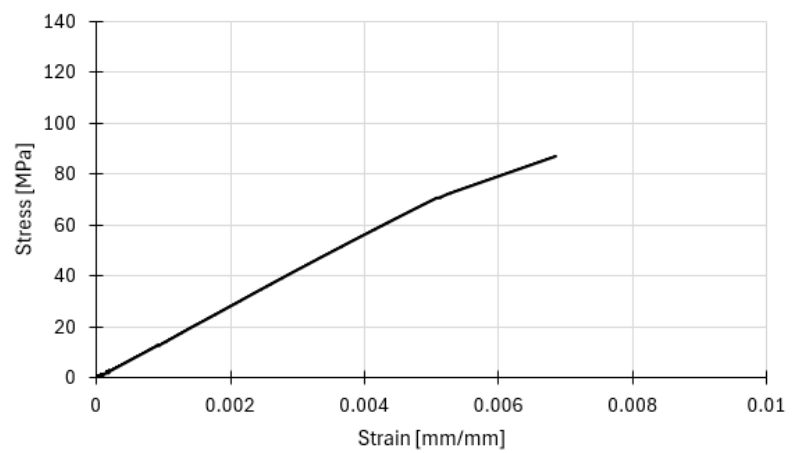


(b) Stress-strain behaviour

Figure B.7 Experimental results for T-07



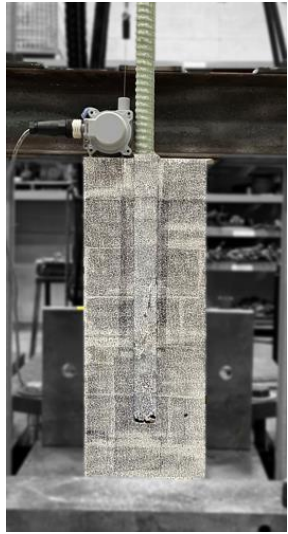
(a) Failed specimen



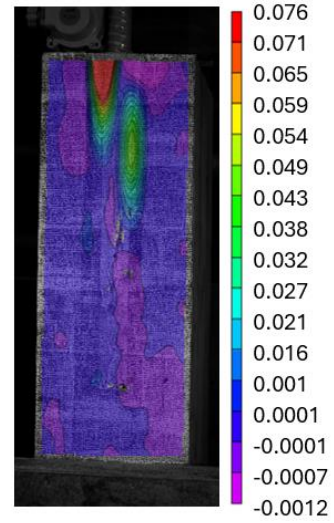
(b) Stress-strain behaviour

Figure B.8 Experimental results for T-08

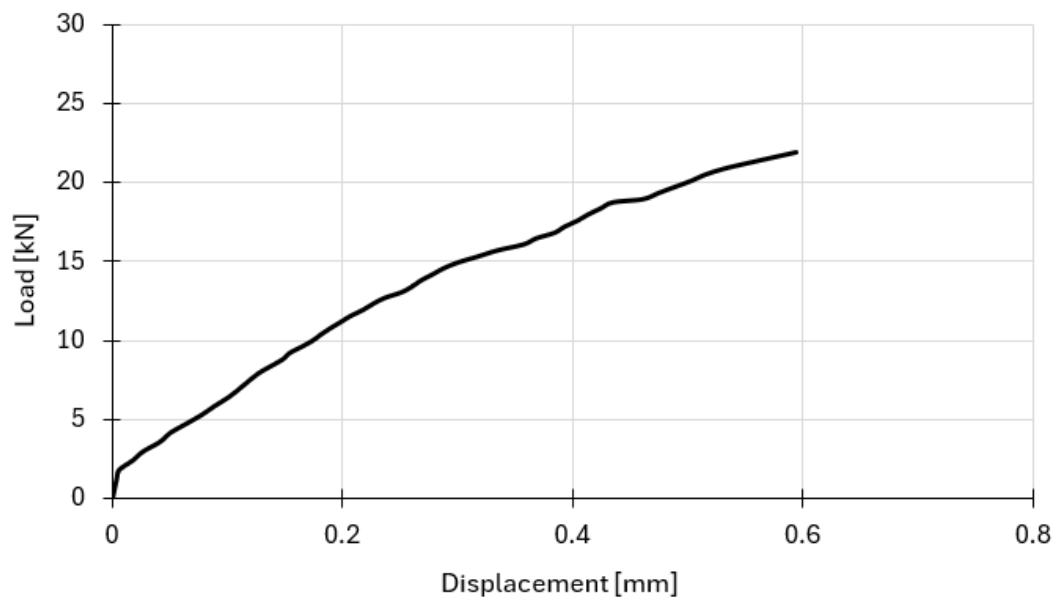
APPENDIX C | PULLOUT TEST RESULTS



(a) Failed specimen

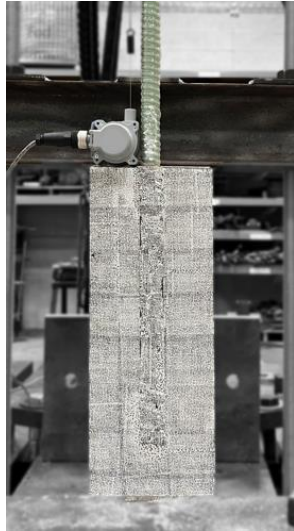


(b) Strain contours prior to failure

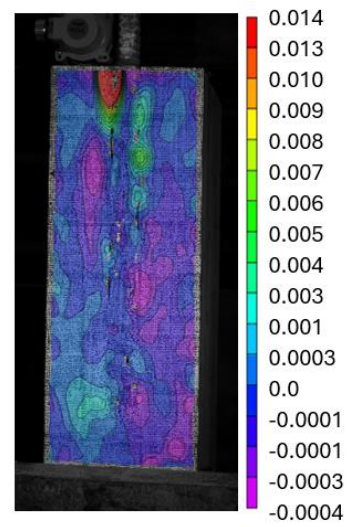


(c) Force-displacement curve

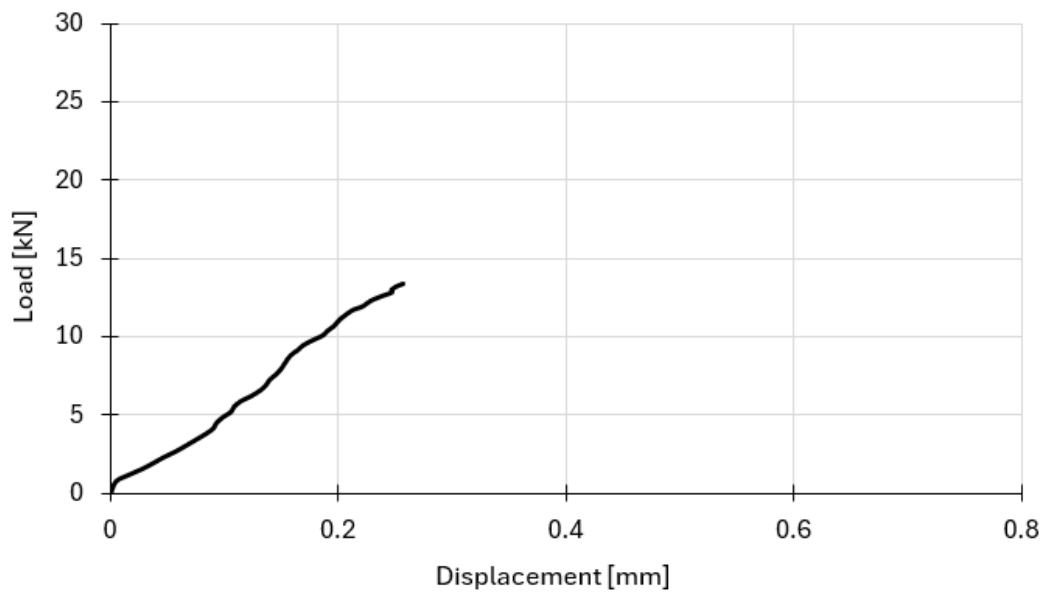
Figure C.1 Results for specimen P-01



(a) Failed specimen

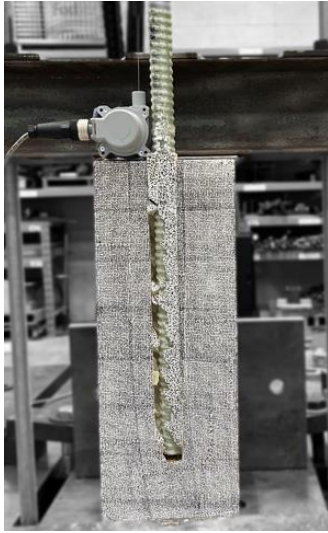


(b) Strain contours prior to failure

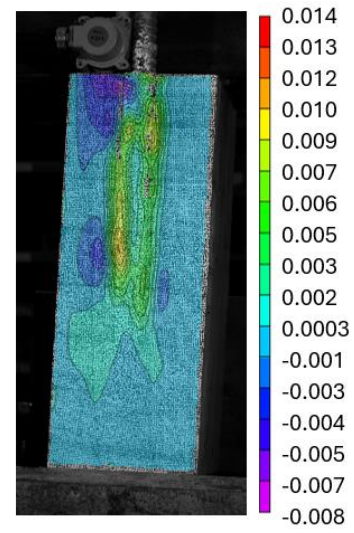


(c) Force-displacement curve

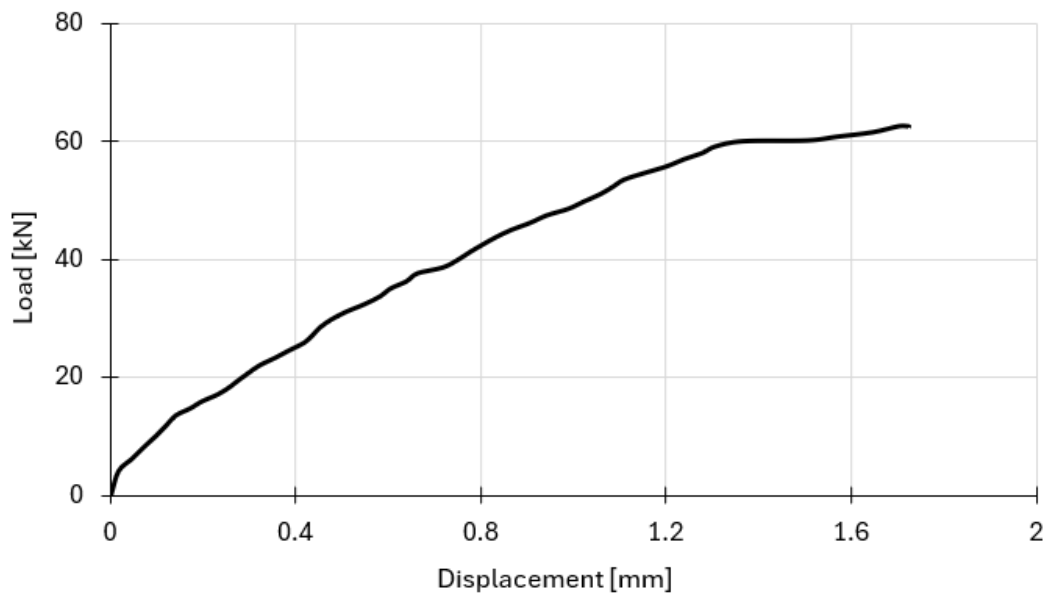
Figure C.2 Results for specimen P-02



(a) Failed specimen



(b) Strain contours prior to failure

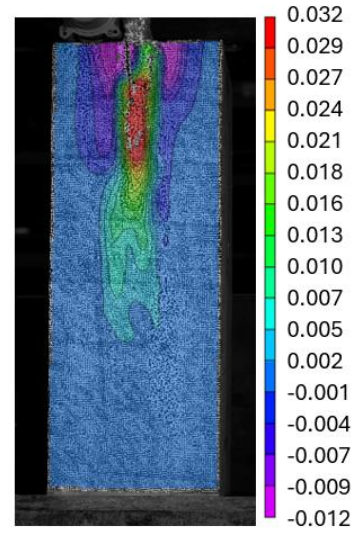


(c) Force-displacement curve

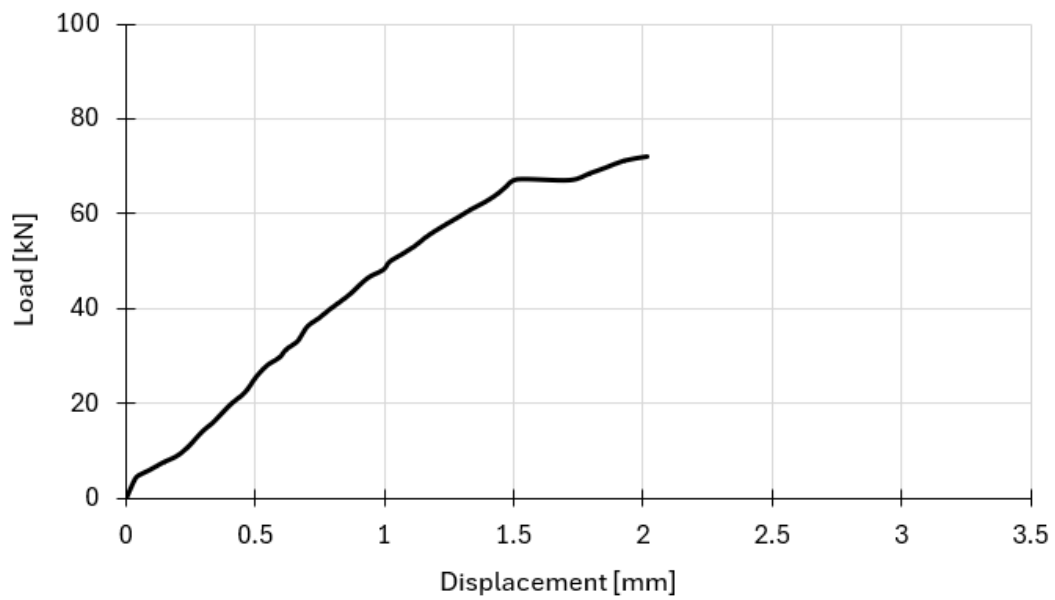
Figure C.3 Results for specimen P-03



(a) Failed specimen

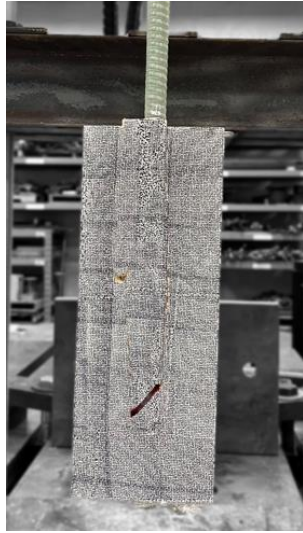


(b) Strain contours prior to failure

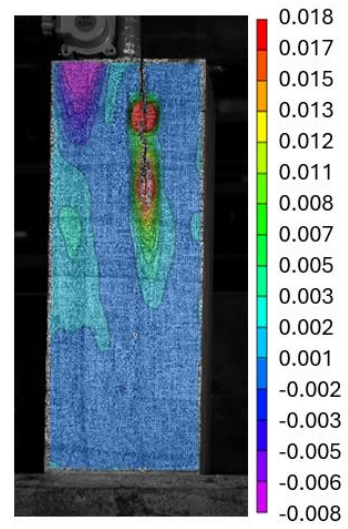


(c) Force-displacement curve

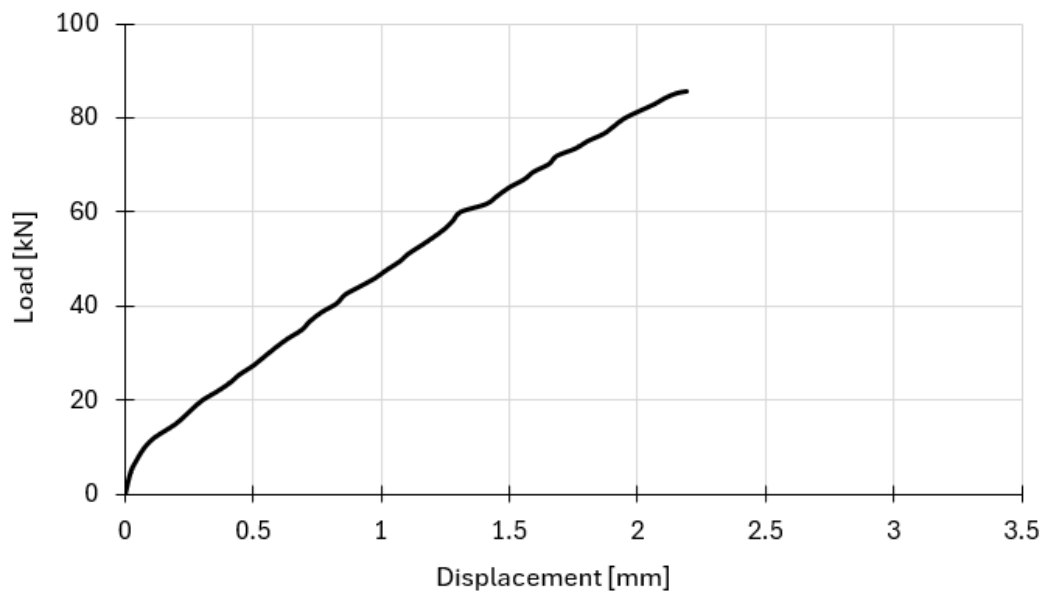
Figure C.4 Results for specimen P-04



(a) Failed specimen

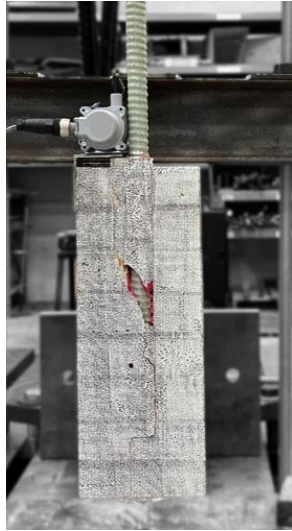


(b) Strain contours prior to failure

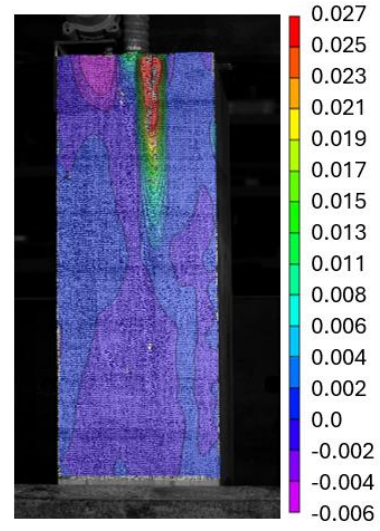


(c) Force-displacement curve

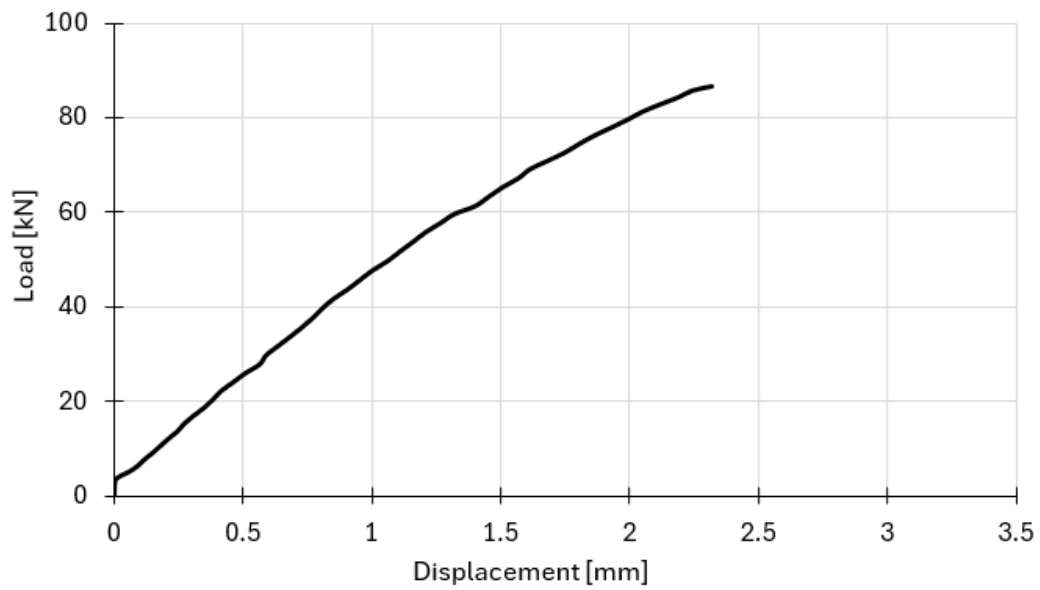
Figure C.5 Results for specimen P-05



(a) Failed specimen



(b) Strain contours prior to failure

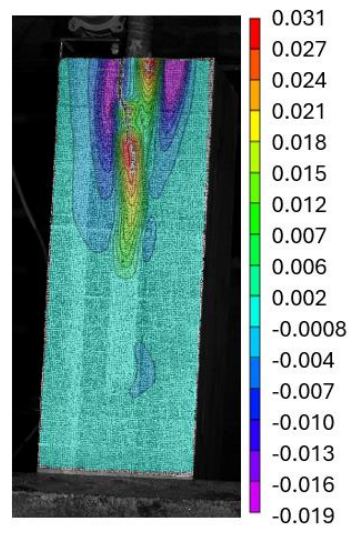


(c) Force-displacement curve

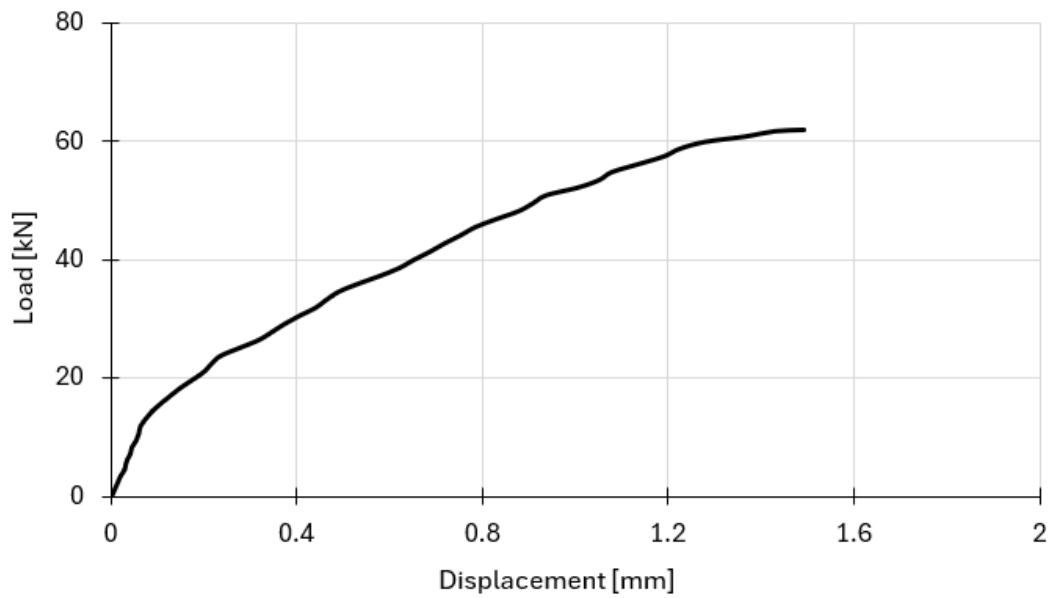
Figure C.6 Results for specimen P-06



(a) Failed specimen

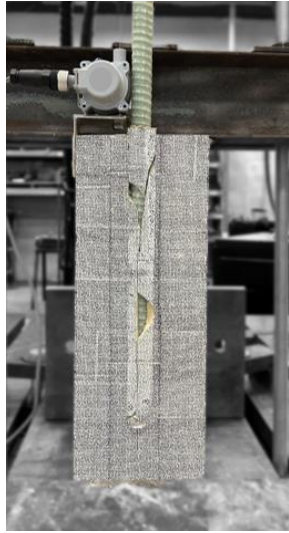


(b) Strain contours prior to failure

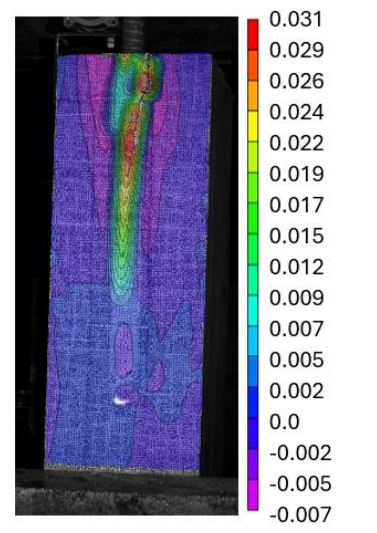


(c) Force-displacement curve

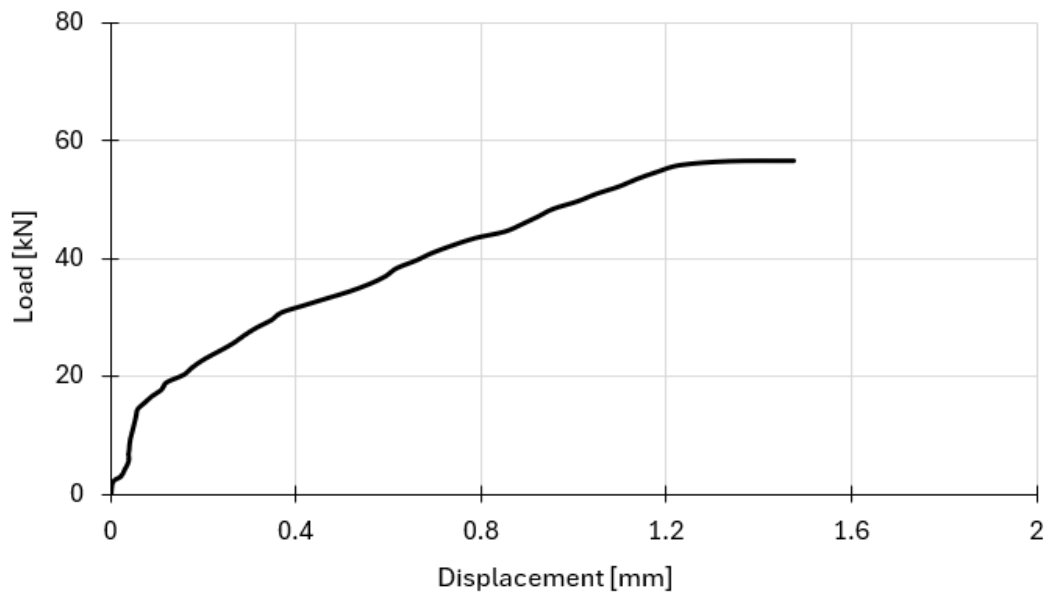
Figure C.7 Results for specimen P-07



(a) Failed specimen



(b) Strain contours prior to failure

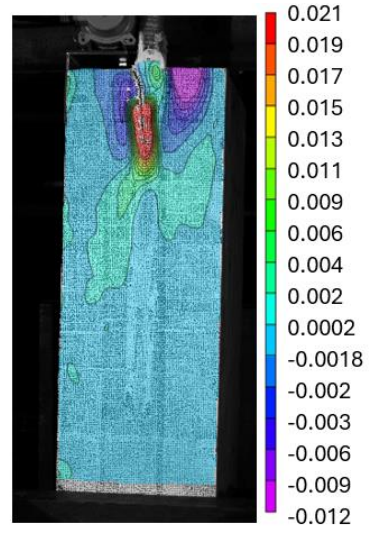


(c) Force-displacement curve

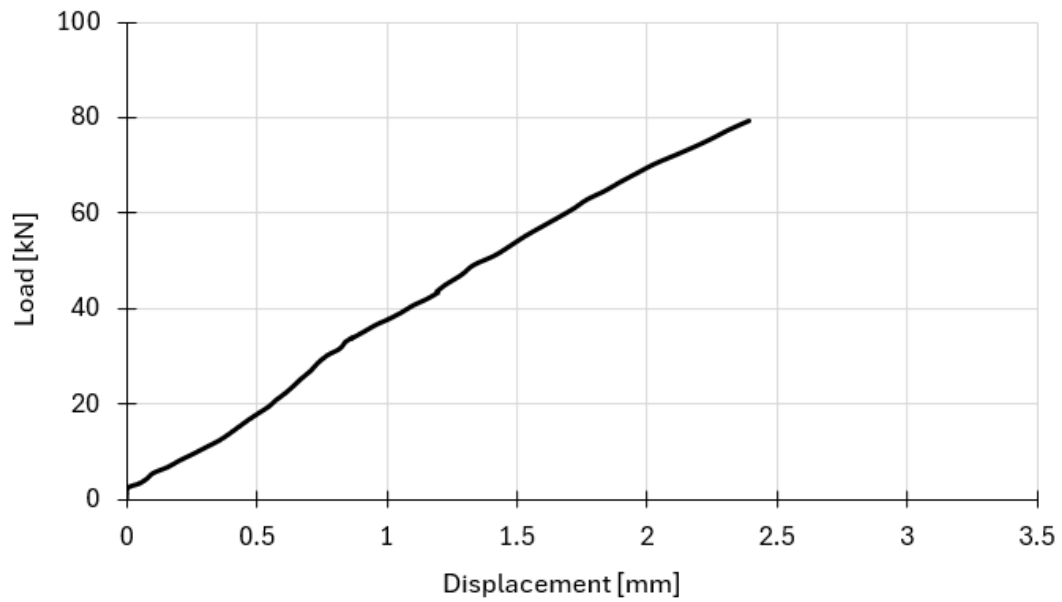
Figure C.8 Results for specimen P-08



(a) Failed specimen

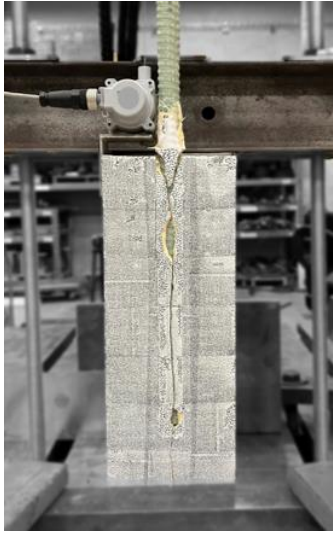


(b) Strain contours prior to failure

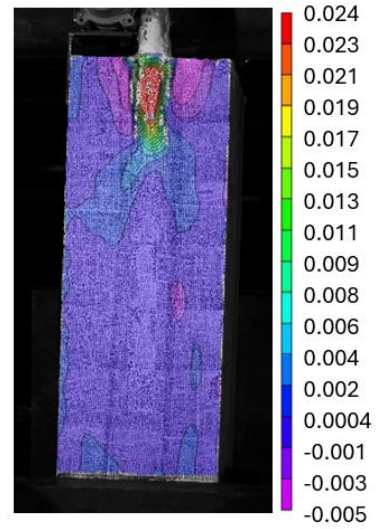


(c) Force-displacement curve

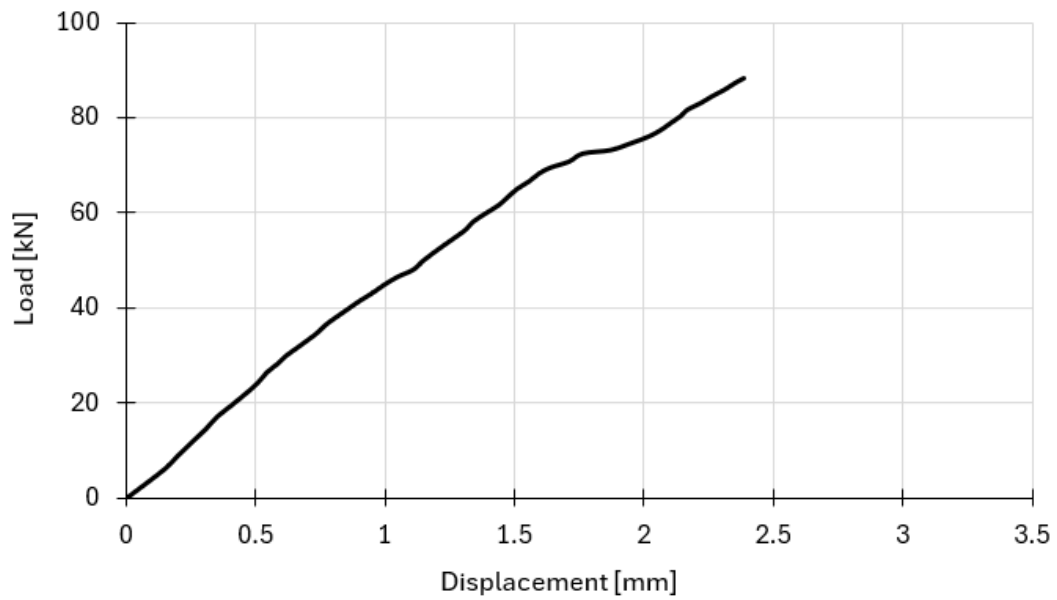
Figure C.9 Results for specimen P-09



(a) Failed specimen



(b) Strain contours prior to failure

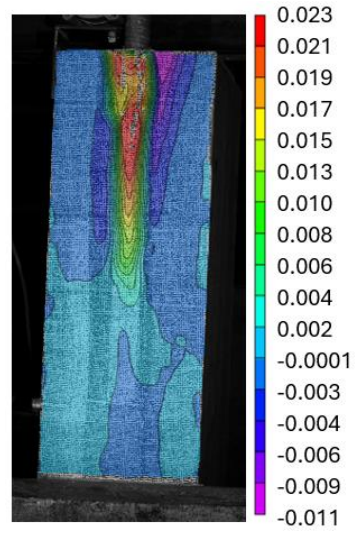


(c) Force-displacement curve

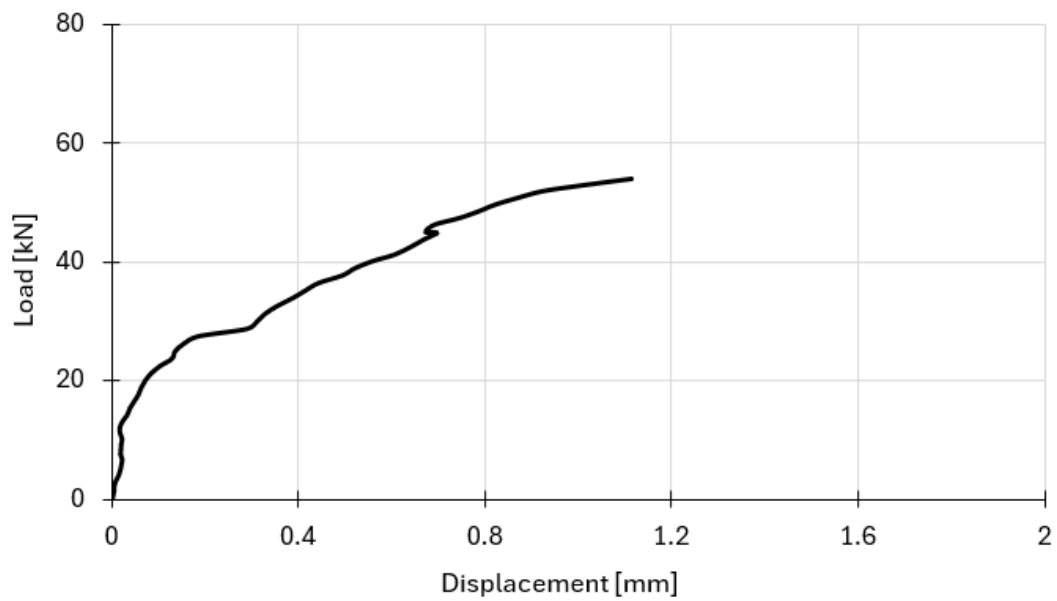
Figure C.10 Results for specimen P-10



(a) Failed specimen



(b) Strain contours prior to failure

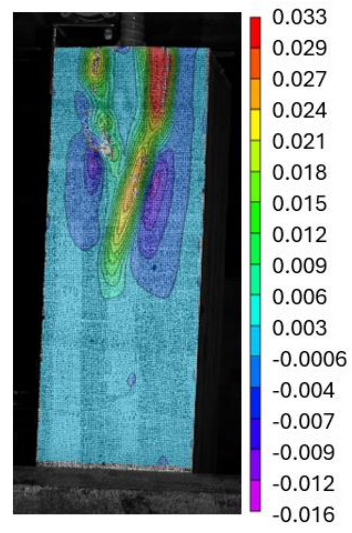


(c) Force-displacement curve

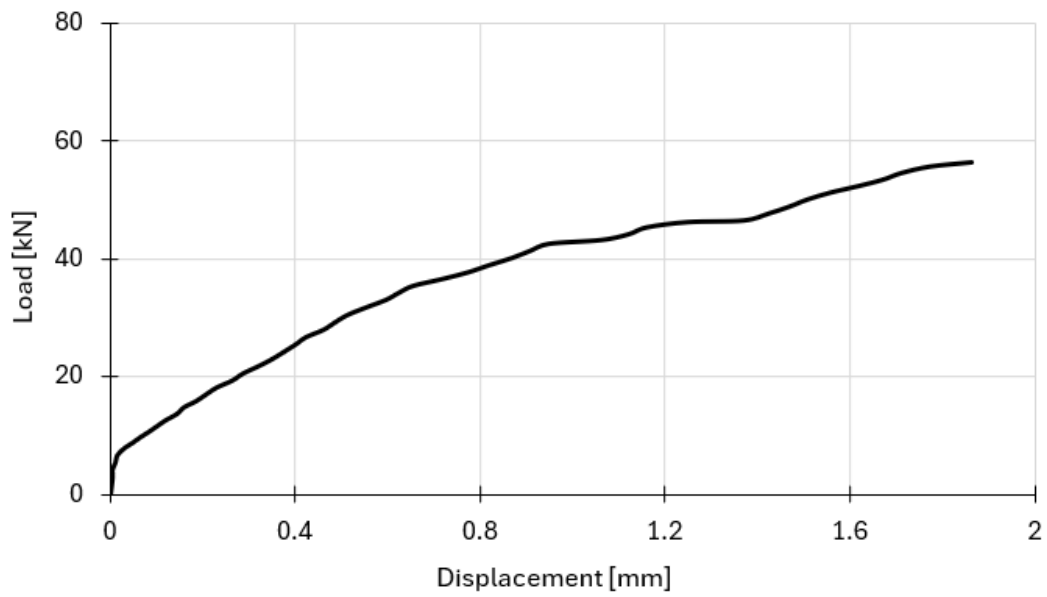
Figure C.11 Results for specimen P-11



(a) Failed specimen



(b) Strain contours prior to failure

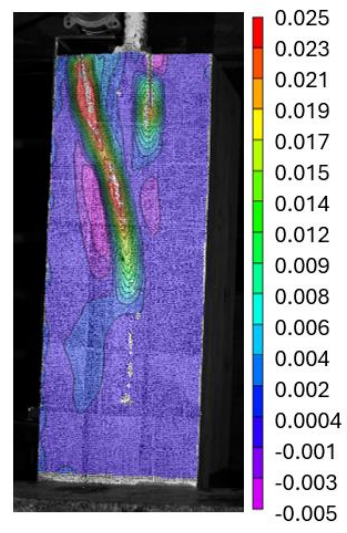


(c) Force-displacement curve

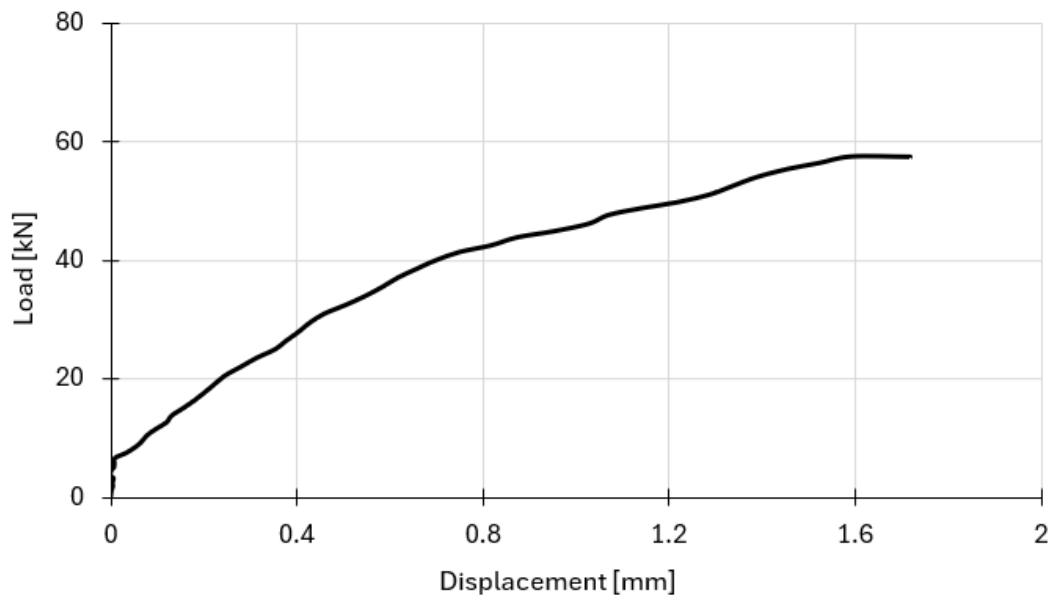
Figure C.12 Results for specimen P-12



(a) Failed specimen

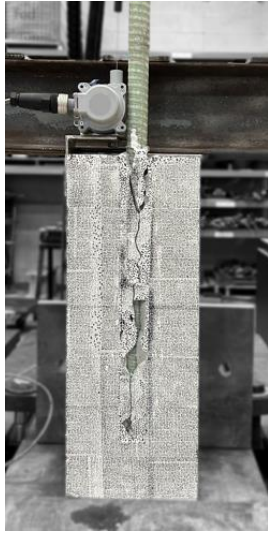


(b) Strain contours prior to failure

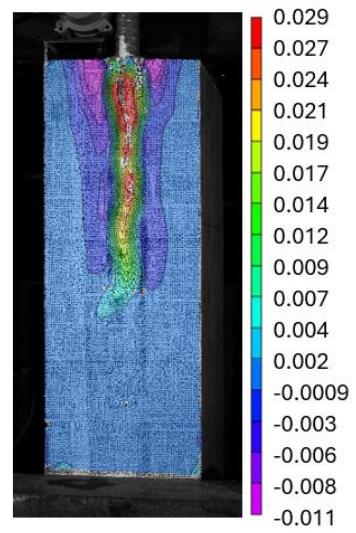


(c) Force-displacement curve

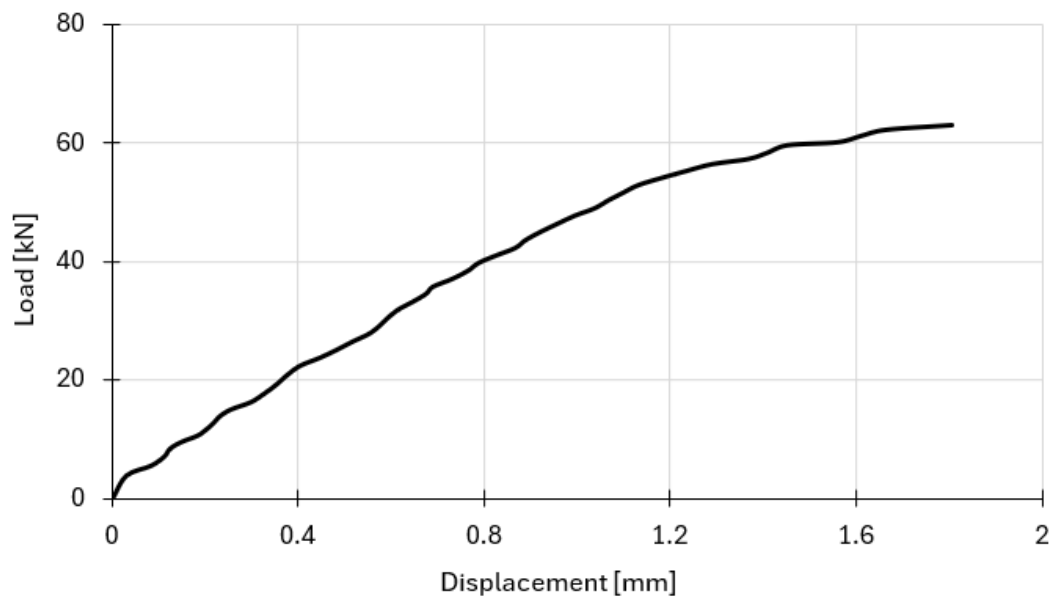
Figure C.13 Results for specimen P-13



(a) Failed specimen



(b) Strain contours prior to failure

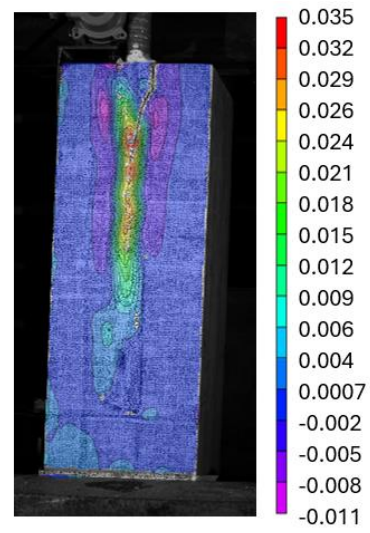


(c) Force-displacement curve

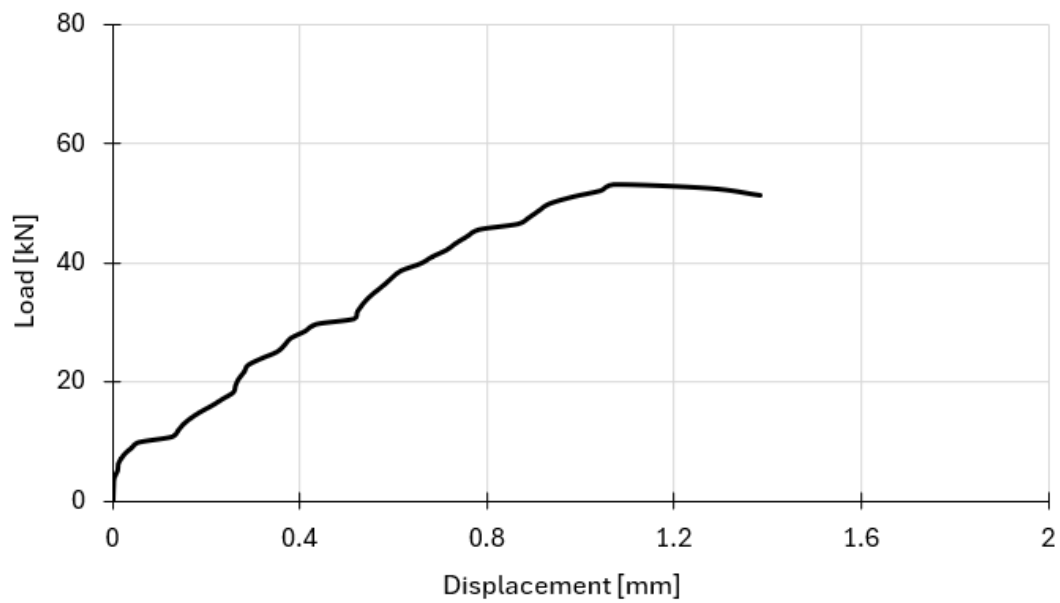
Figure C.14 Results for specimen P-14



(a) Failed specimen

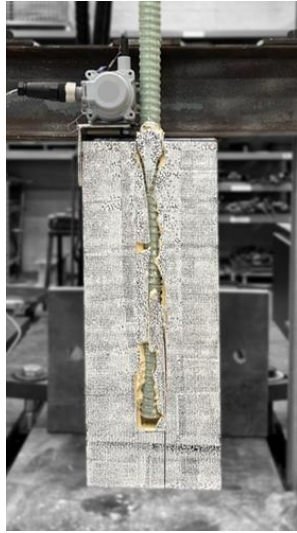


(b) Strain contours prior to failure

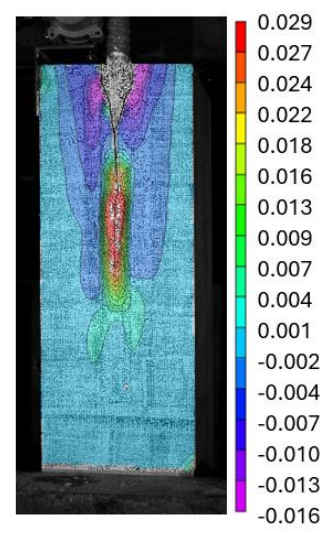


(c) Force-displacement curve

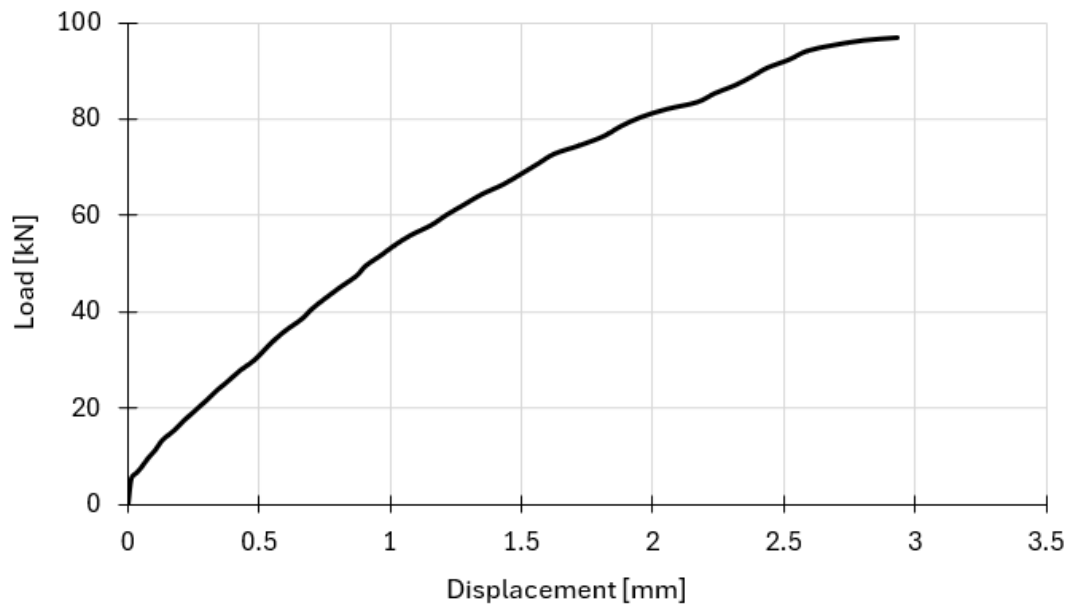
Figure C.15 Results for specimen P-15



(a) Failed specimen



(b) Strain contours prior to failure

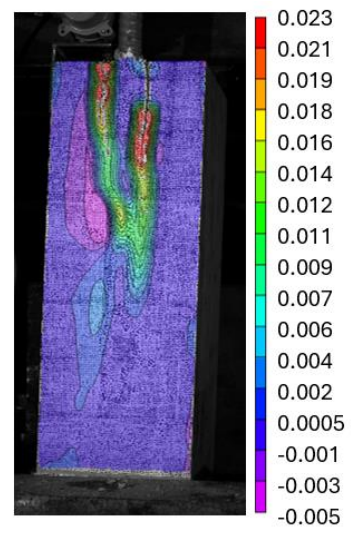


(c) Force-displacement curve

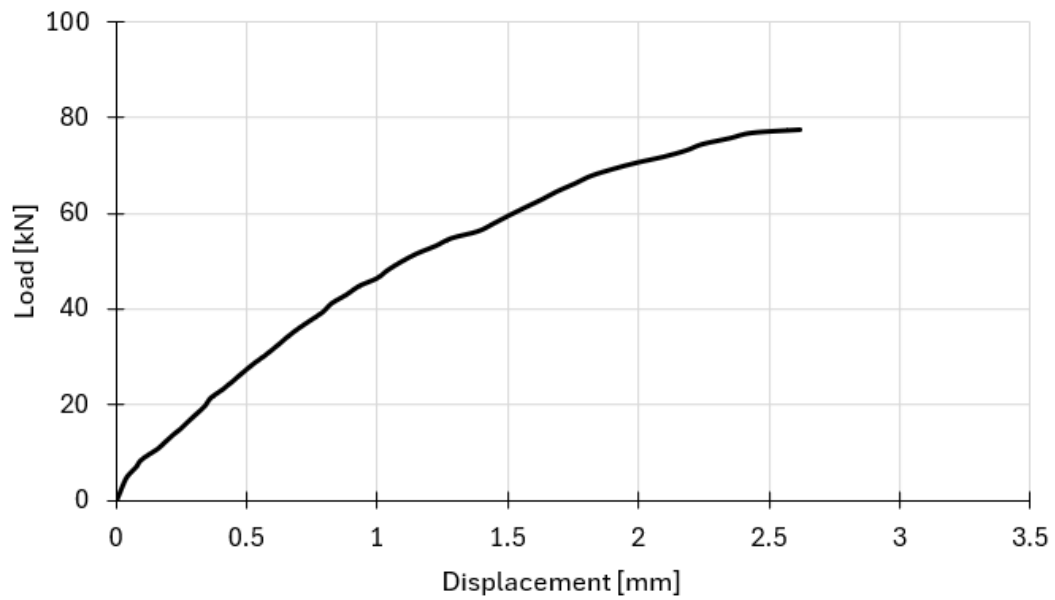
Figure C.16 Results for specimen P-16



(a) Failed specimen



(b) Strain contours prior to failure

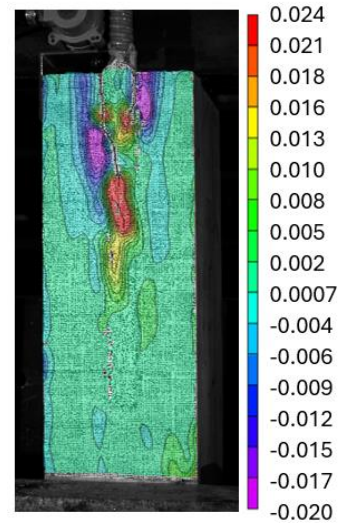


(c) Force-displacement curve

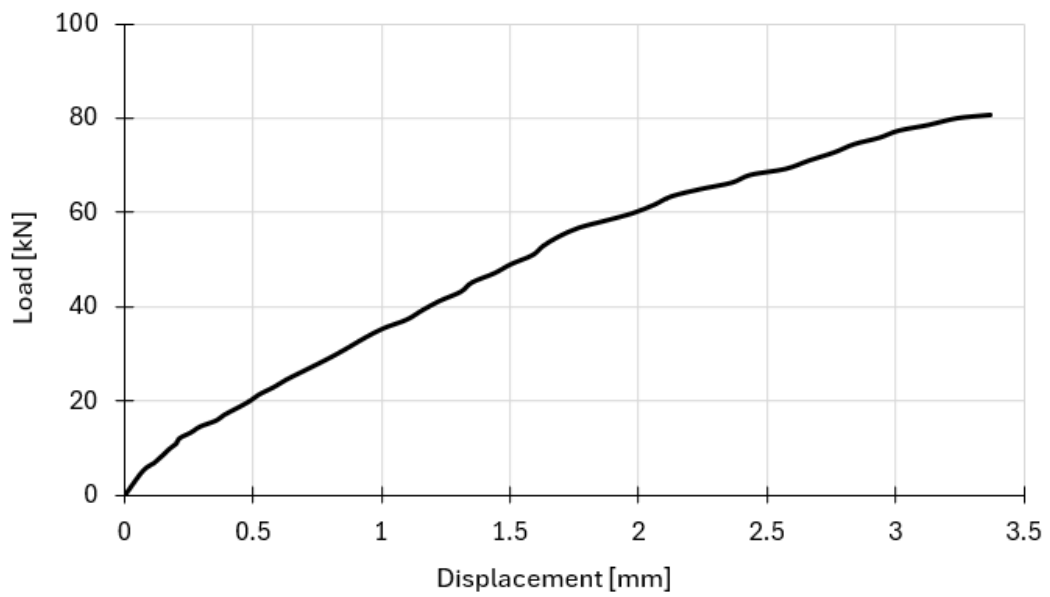
Figure C.17 Results for specimen P-17



(a) Failed specimen



(b) Strain contours prior to failure

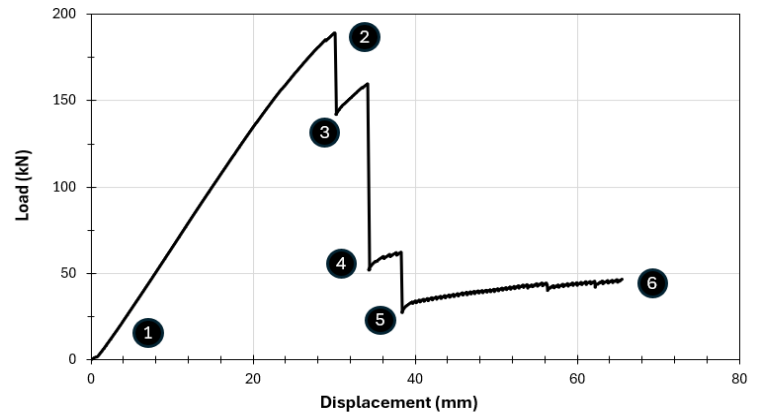


(c) Force-displacement curve

Figure C.18 Results for specimen P-18

APPENDIX D | FAILURE PROGRESSION FOR UNREINFORCED AND REINFORCED BEAMS

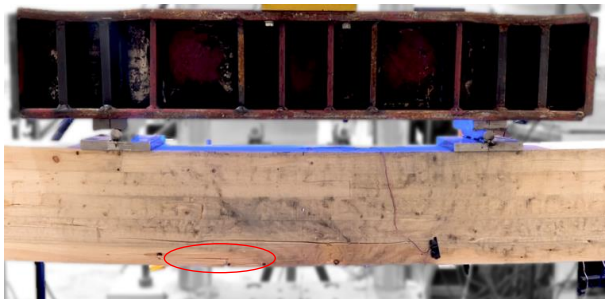
Specimen: U-01
Type: Unreinforced



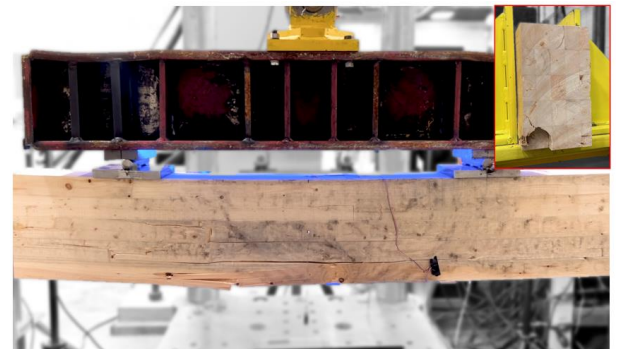
(1) Start of test



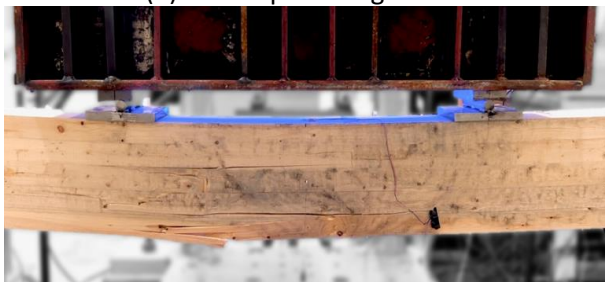
(2) Peak load



(3) Initial splintering failure



(4) Shear failure



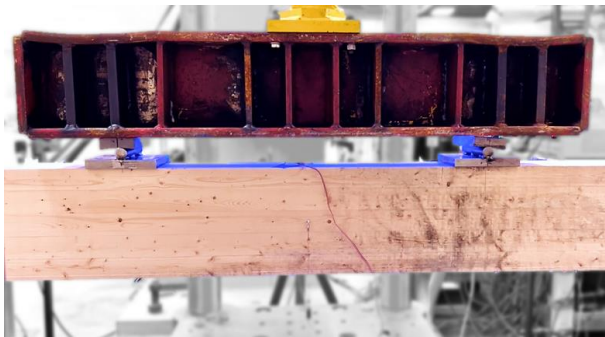
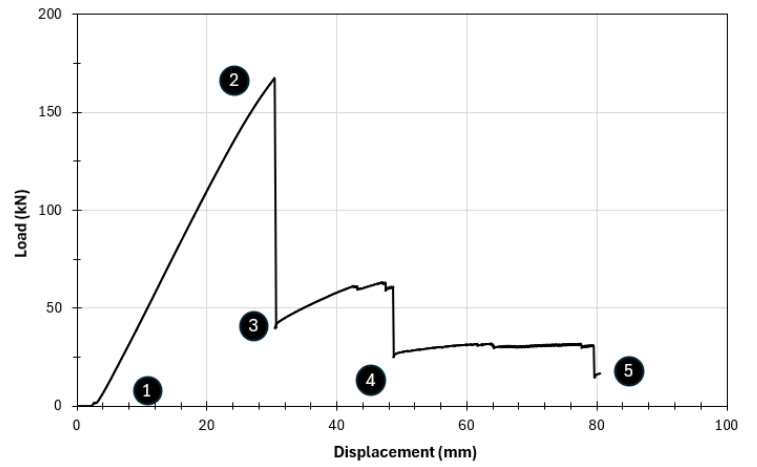
(5) Failures continue to propagate



(6) End of test

Figure D.1 Failure progression for U-01

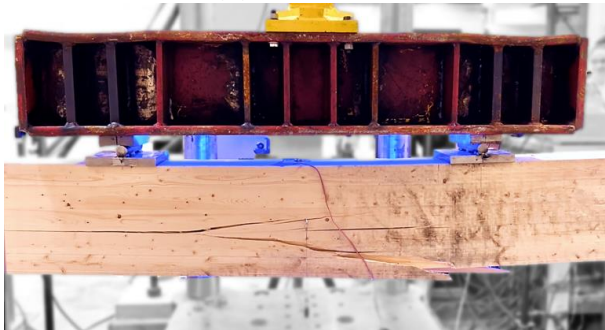
Specimen: U-02
Type: Unreinforced



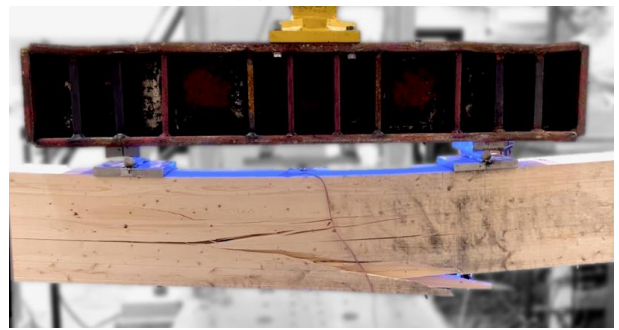
(1) Start of test



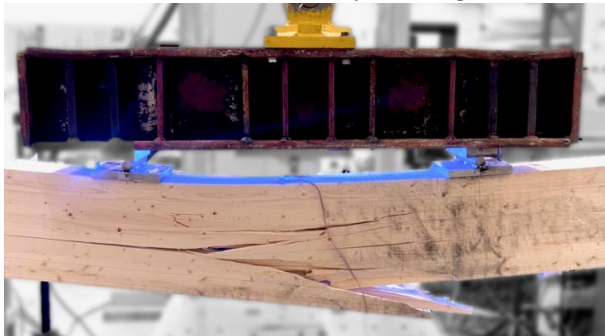
(2) Peak load



(3) Initial failure (splintering)



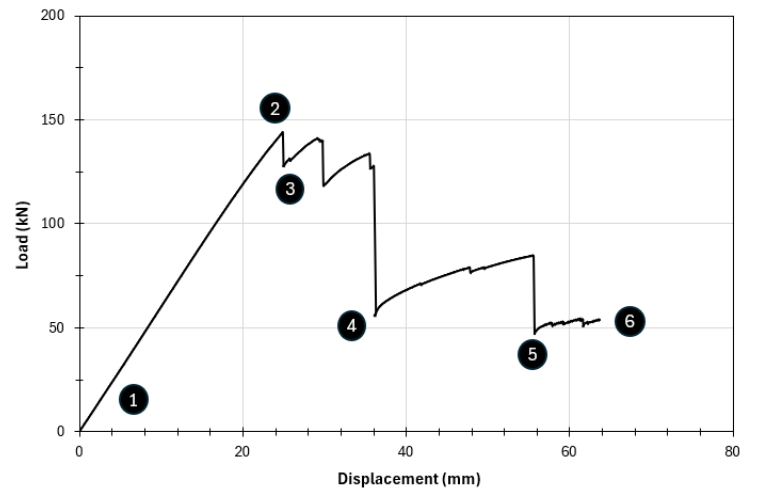
(4) Failures continue to propagate



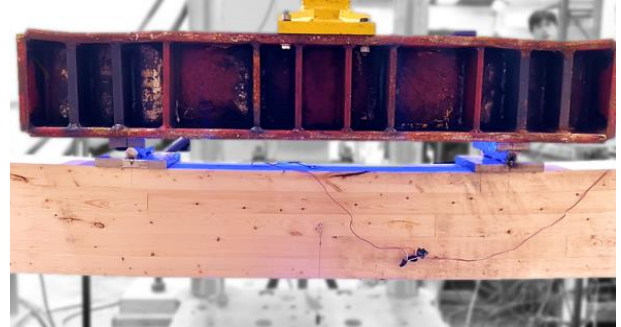
(5) End of test

Figure D.2 Failure progression for U-02

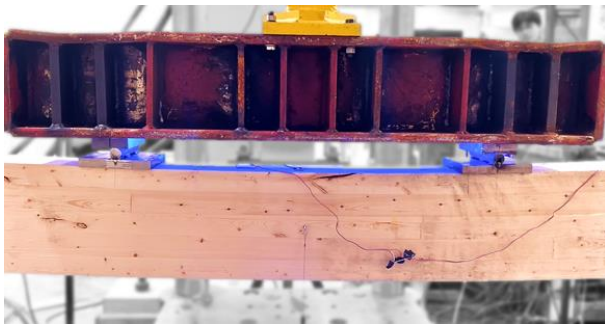
Specimen: U-03
Type: Unreinforced



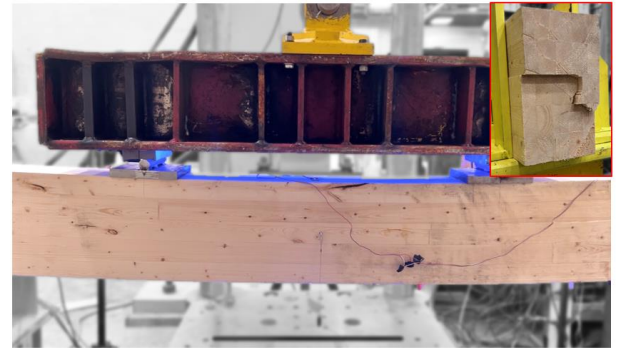
(1) Start of test



(2) Peak load



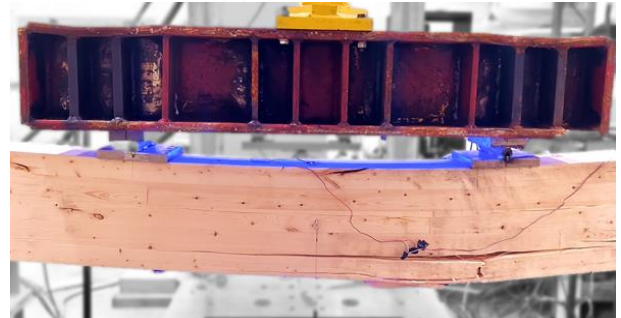
(3) Initial shear failure



(4) Shear failure



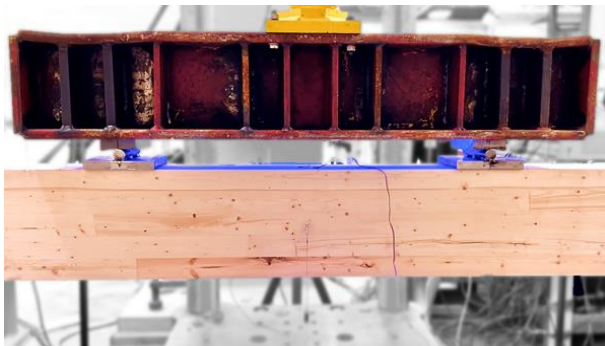
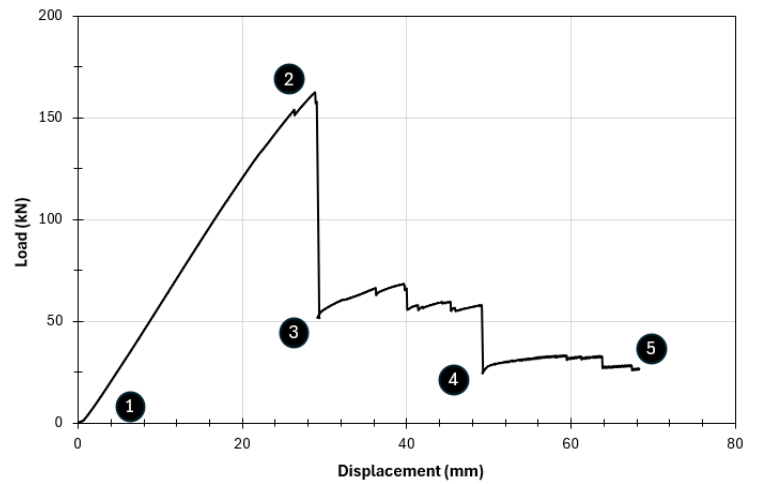
(5) Failures continue to propagate



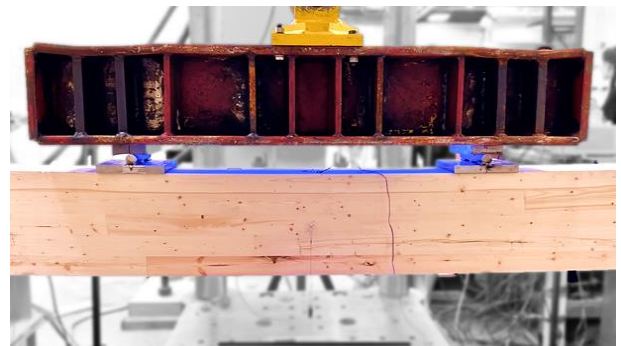
(6) End of test

Figure D.3 Failure progression for U-03

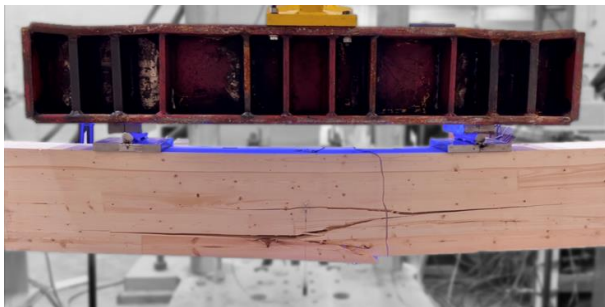
Specimen: U-04
Type: Unreinforced



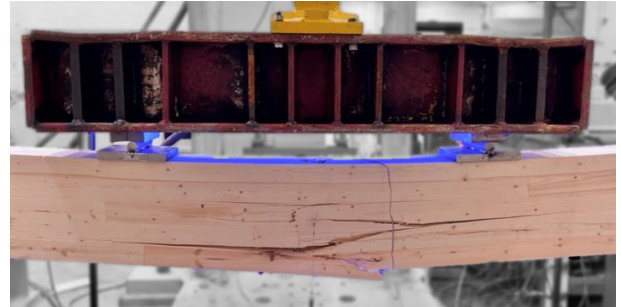
(1) Start of test



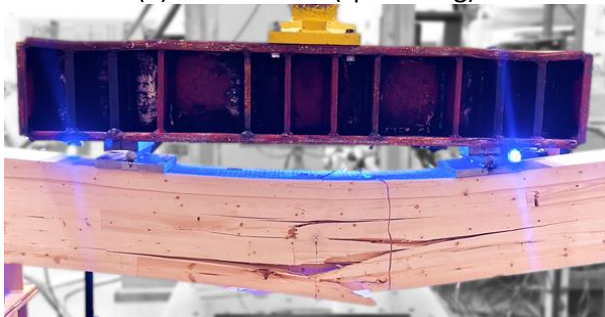
(2) Peak load



(3) Initial failure (splintering)



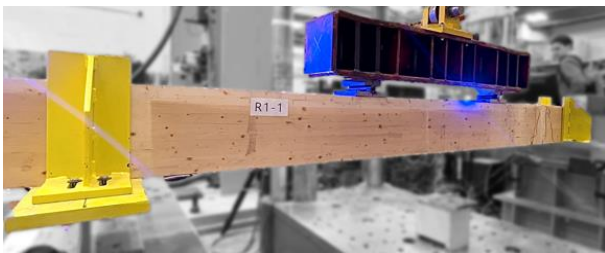
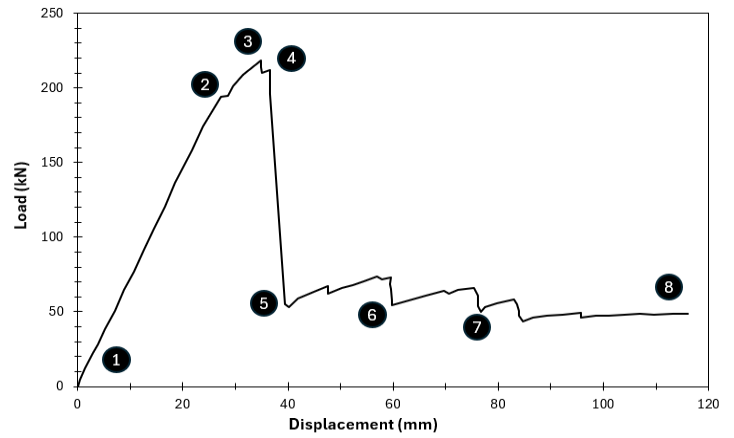
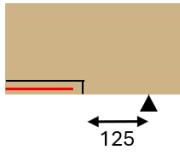
(4) Failure continues to propagate



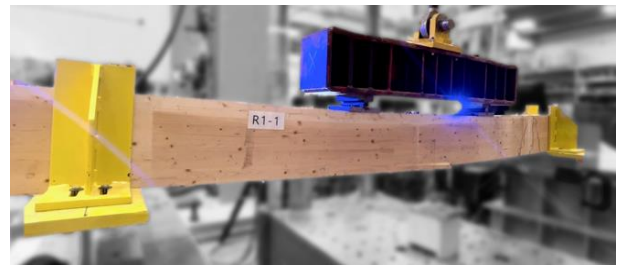
(5) End of test

Figure D.4 Failure progression for U-04

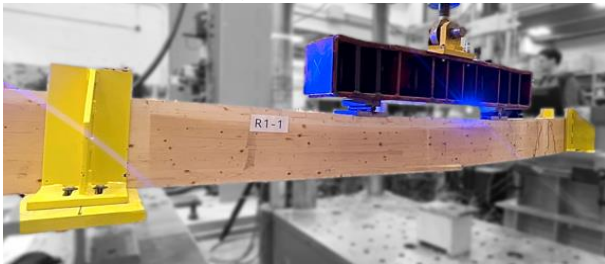
Specimen: R1-1
Type: Reinforced, short bars, straight



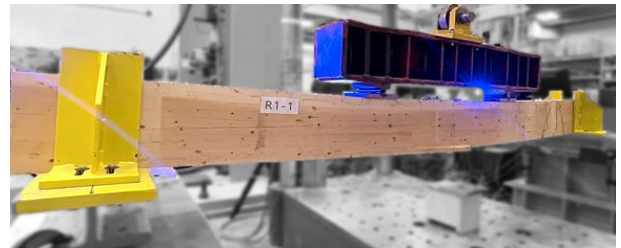
(1) Start of test



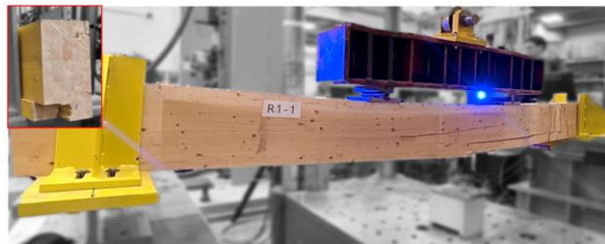
(2) Initial cracking



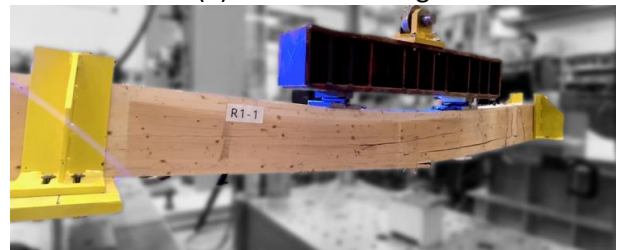
(3) Peak load



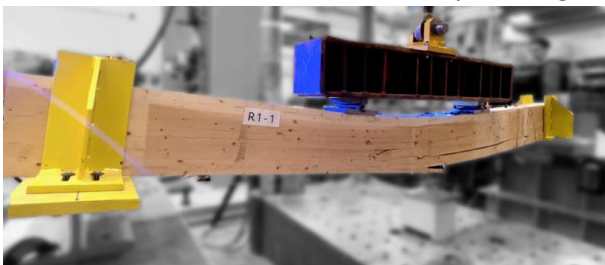
(4) Further cracking



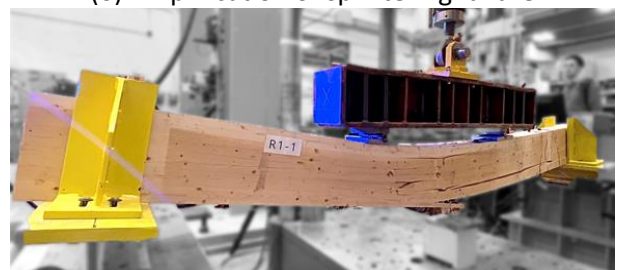
(5) Shear failure with initiation of splintering



(6) Amplification of splintering failure



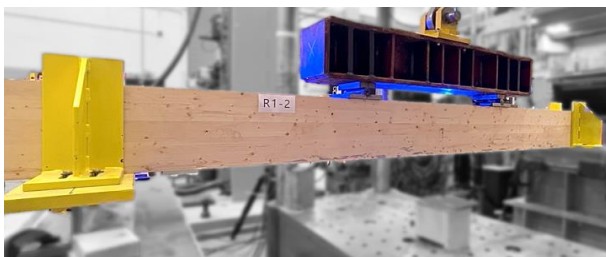
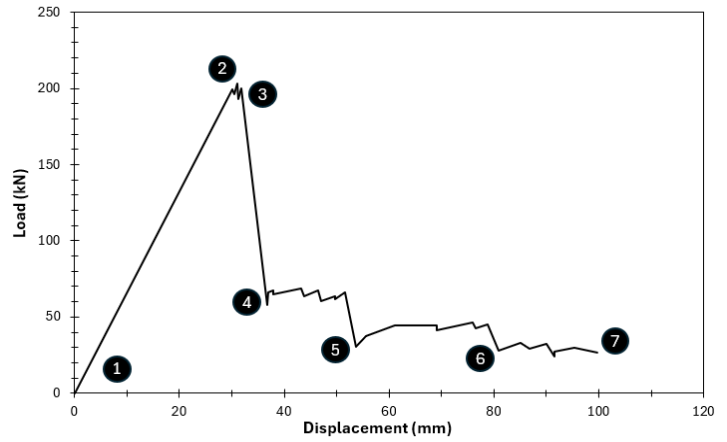
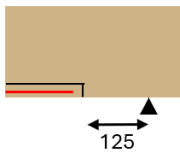
(7) Failure continues to propagate



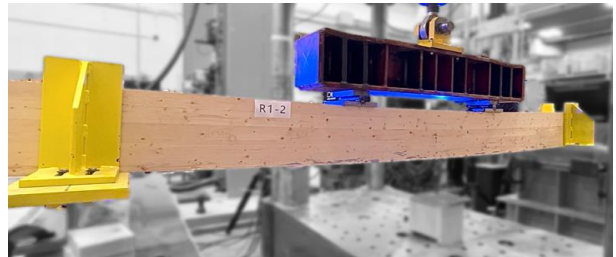
(8) End of test

Figure D.5 Failure progression for R1-1

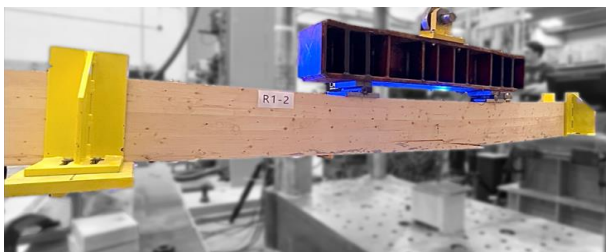
Specimen: R1-2
Type: Reinforced, short bars, straight



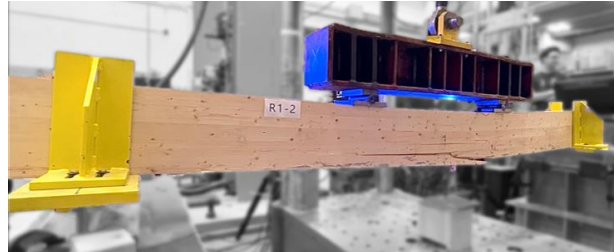
(1) Start of test



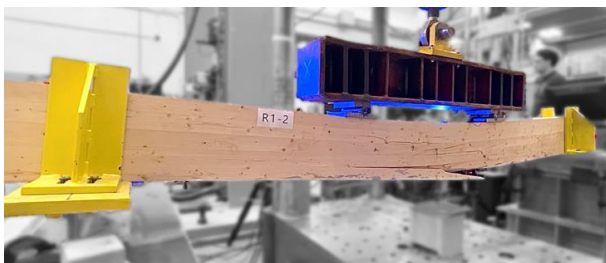
(2) Peak load



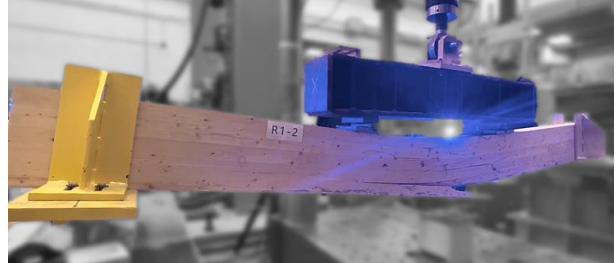
(3) Initial failure (splintering)



(4) Secondary failure



(5) Failure continues to propagate



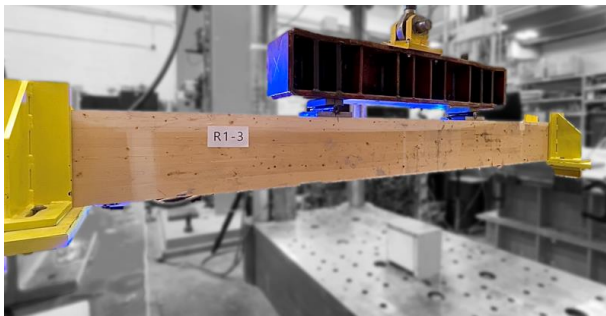
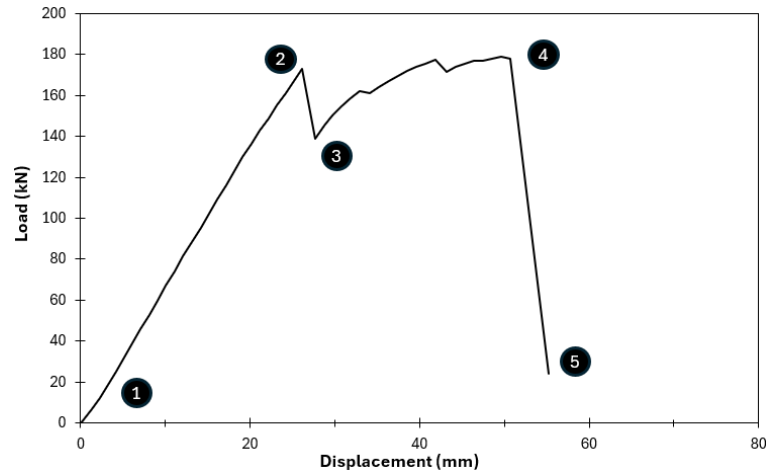
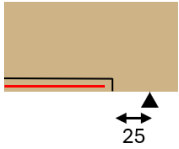
(6) Failures continue to propagate



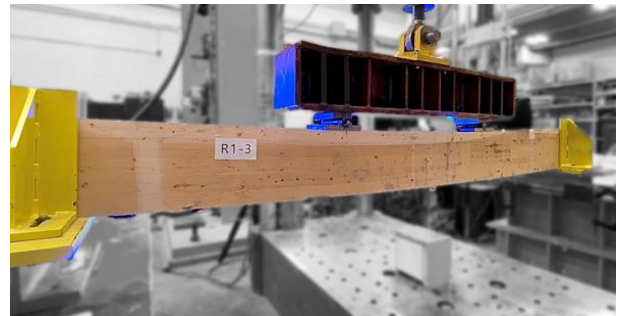
(7) End of test

Figure D.6 Failure progression for R1-2

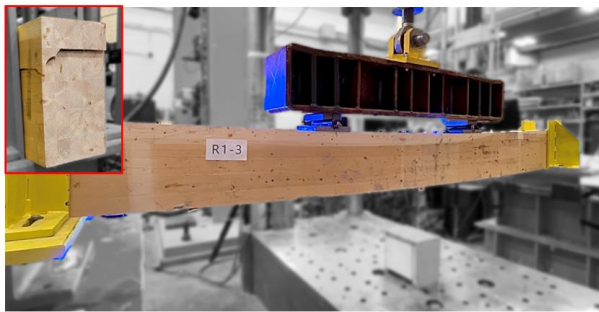
Specimen: R1-3
Type: Reinforced, long bars, straight



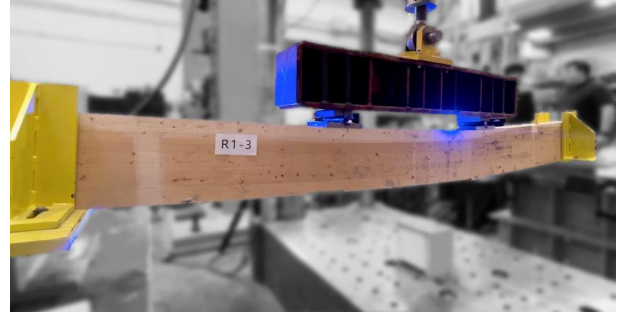
(1) Start of test



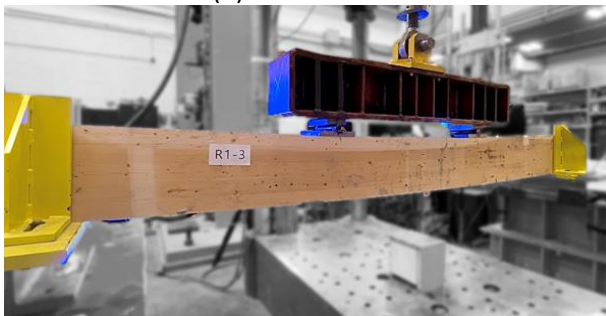
(2) Initial peak load



(3) Shear failure



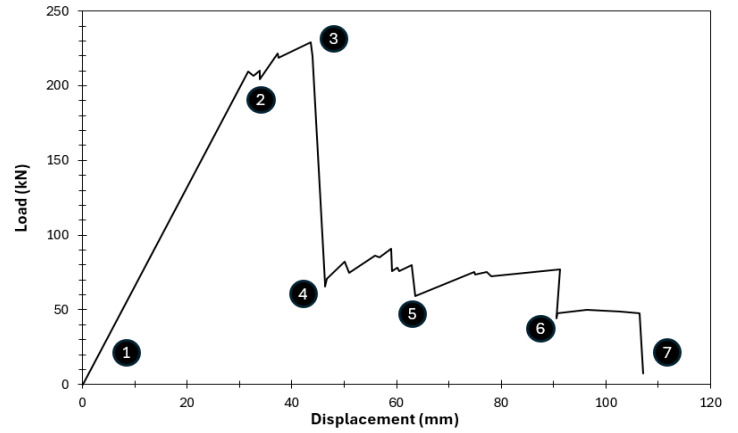
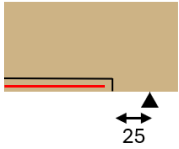
(4) Peak load



(5) End of test

Figure D.7 Failure progression for R1-3

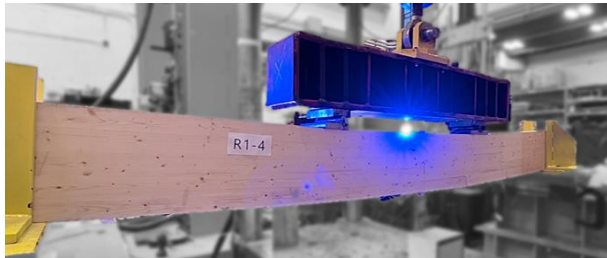
Specimen: R1-4
Type: Reinforced, long bars, straight



(1) Start of test



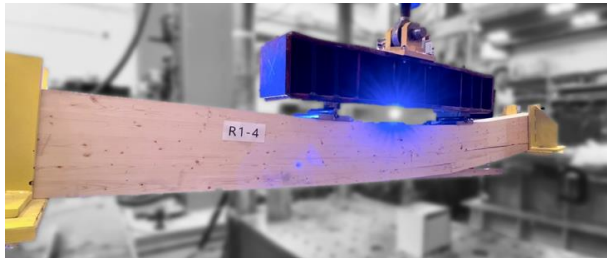
(2) Initial failure



(3) Peak load



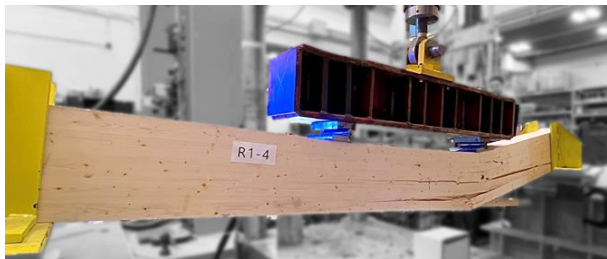
(4) Shear failure



(5) Bar comes out of beam with initiation of simple tension



(6) Failures continue to propagate

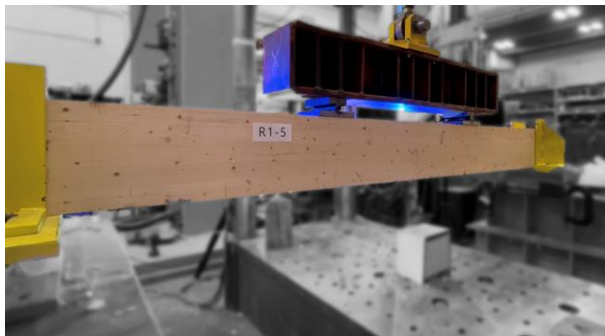
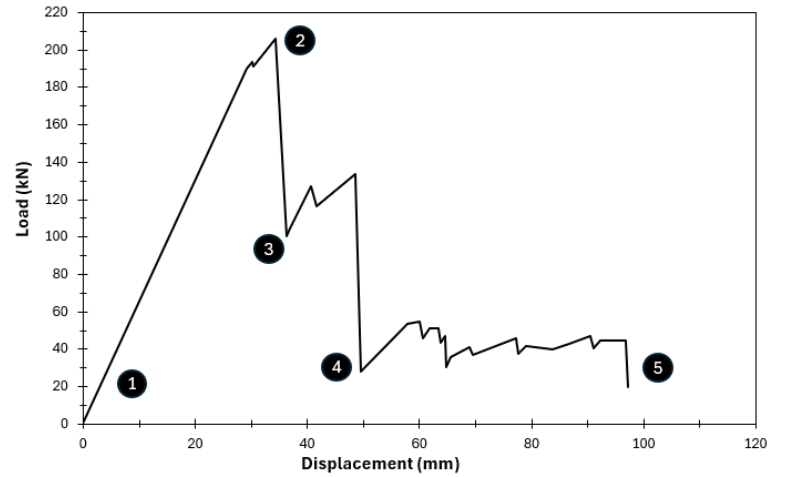
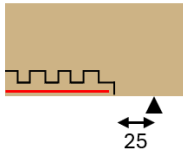


(7) End of test

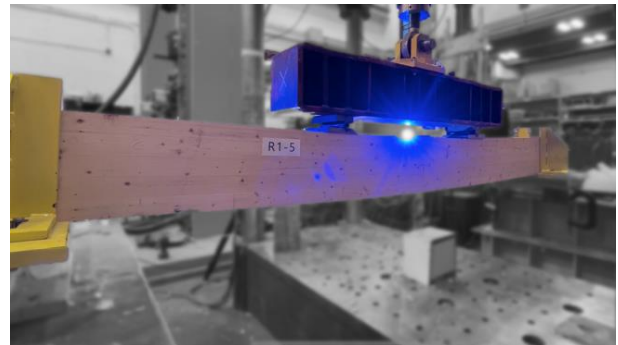
Figure D.8 Failure progression for R1-4

Specimen: R1-5

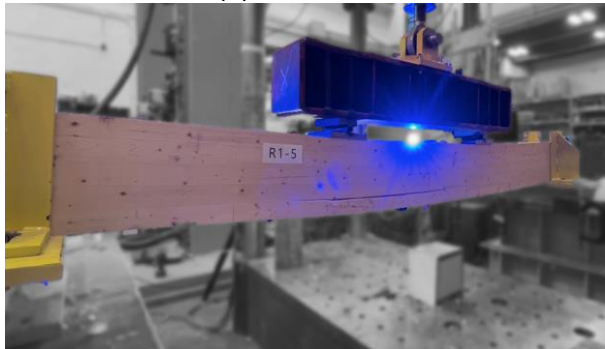
Type: Reinforced, long bars, straight,
alternating groove



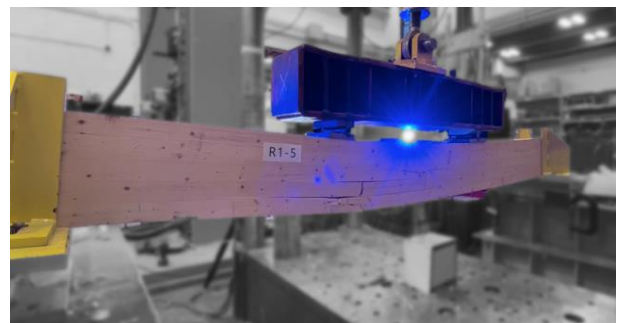
(1) Start of test



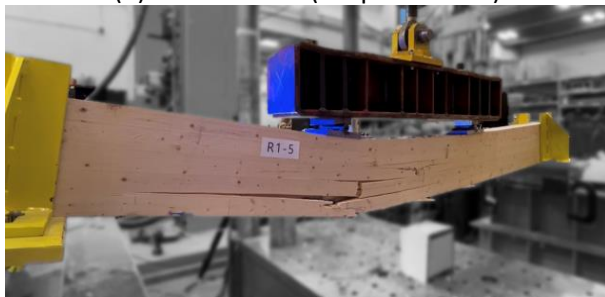
(2) Peak Load



(3) Initial failure (simple tension)



(4) Secondary failure

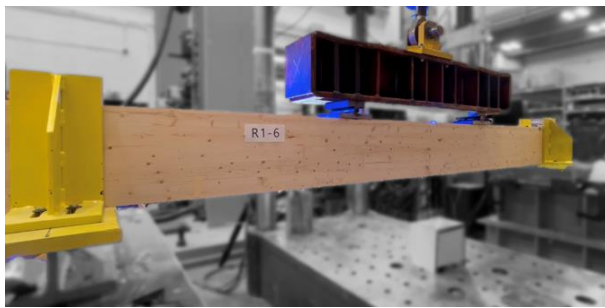
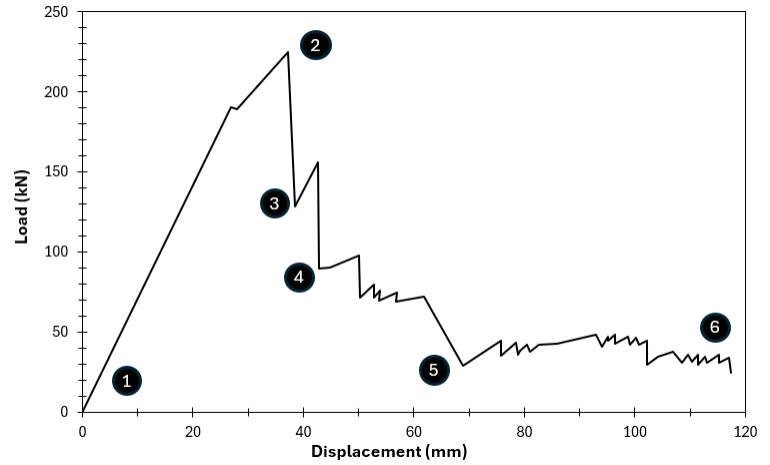
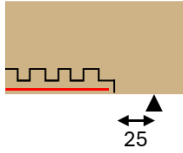


(5) End of test

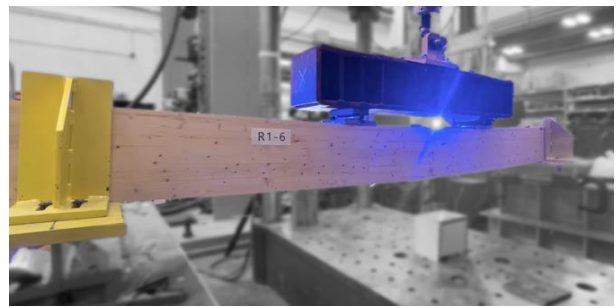
Figure D.9 Failure progression for R1-5

Specimen: R1-6

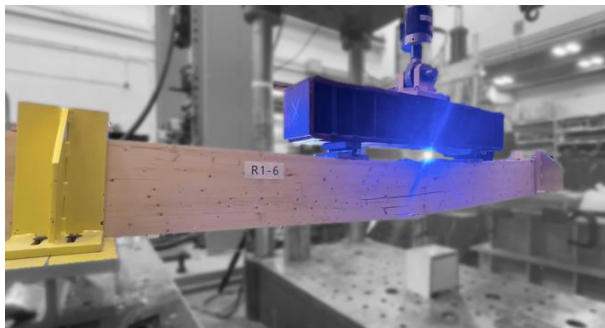
Type: : Reinforced, long bars, straight, alternating groove



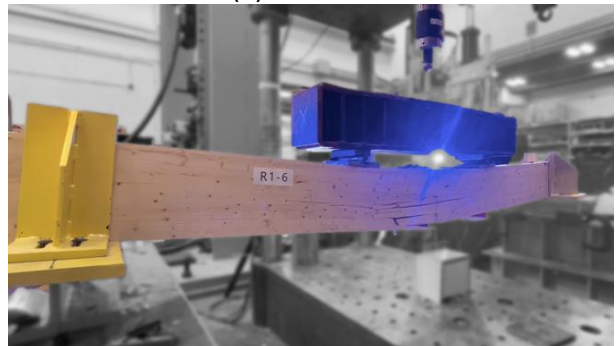
(1) Start of test



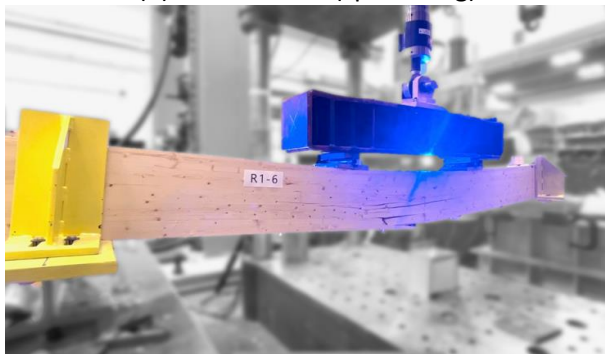
(2) Peak Load



(3) Initial failure (splintering)



(4) Secondary failure



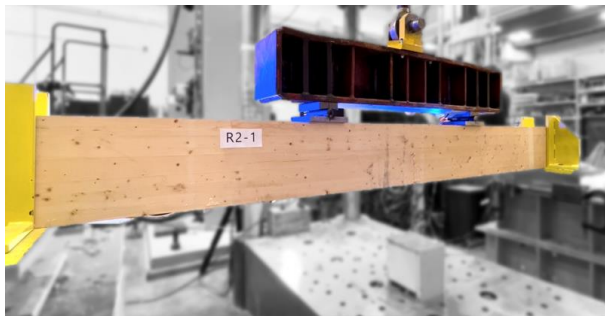
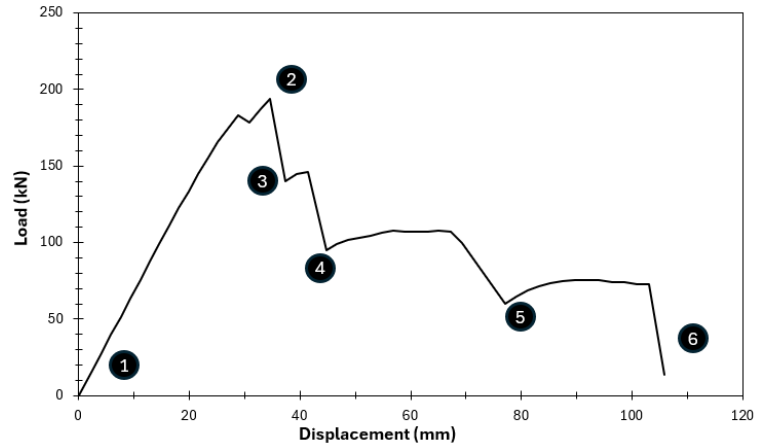
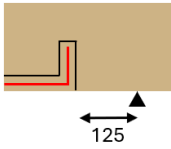
(5) Failures continue to propagate



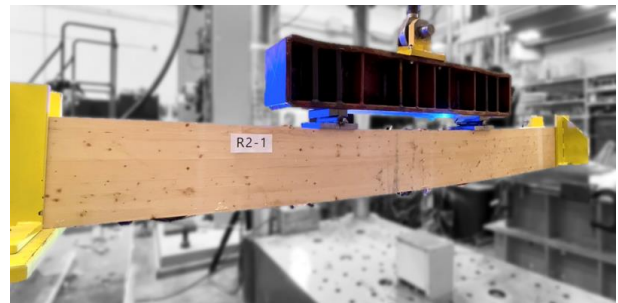
(6) End of test

Figure D.10 Failure progression for R1-6

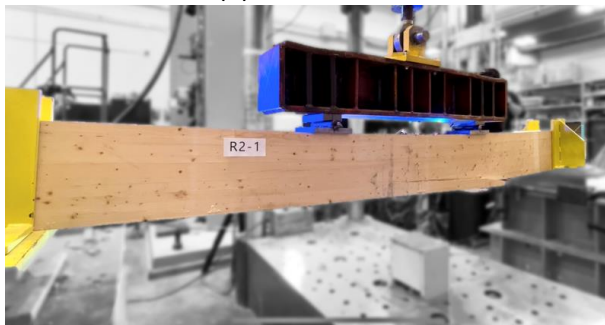
Specimen: R2-1
Type: : Reinforced, short bars, bent



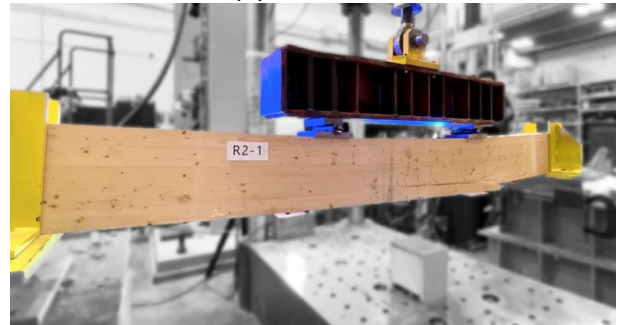
(1) Start of test



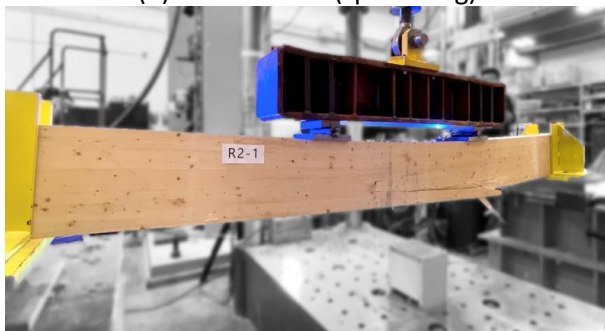
(2) Peak Load



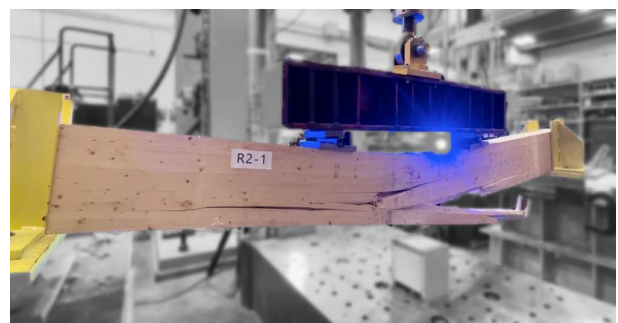
(3) Initial failure (splintering)



(4) Failures continue to propagate



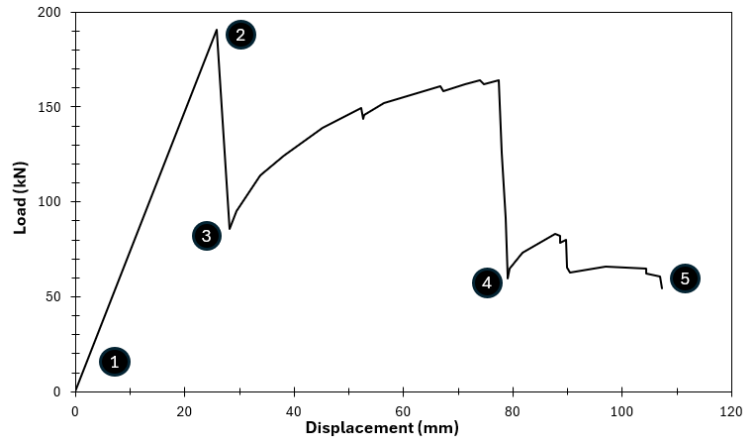
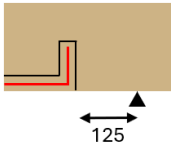
(5) Failures continue to propagate



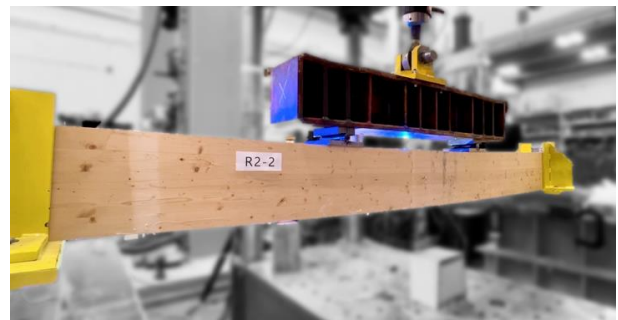
(6) End of test

Figure D.11 Failure progression for R2-1

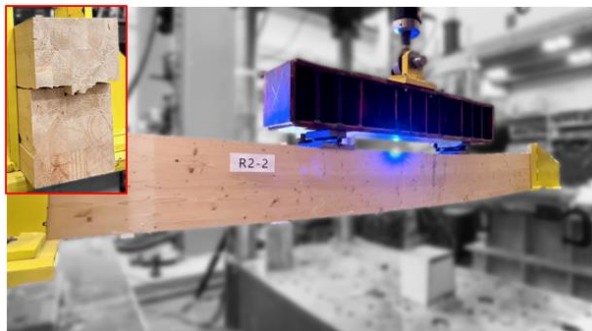
Specimen: R2-2
Type: : Reinforced, short bars, bent



(1) Start of test



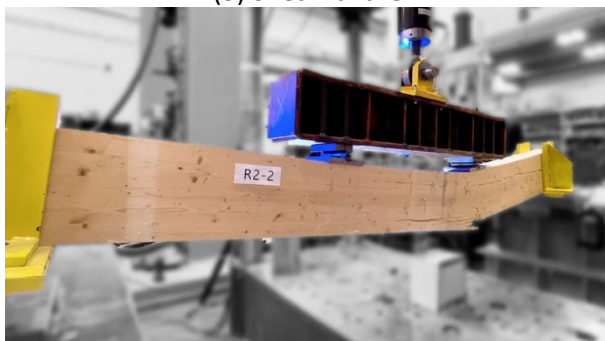
(2) Peak Load



(3) Shear failure



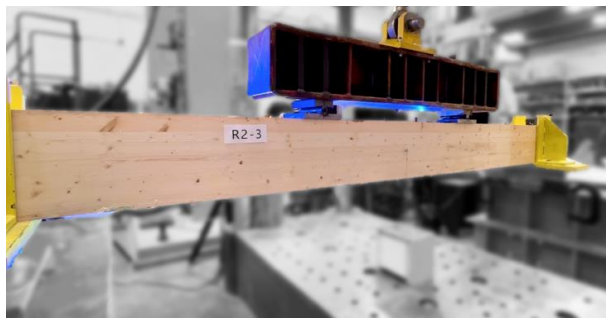
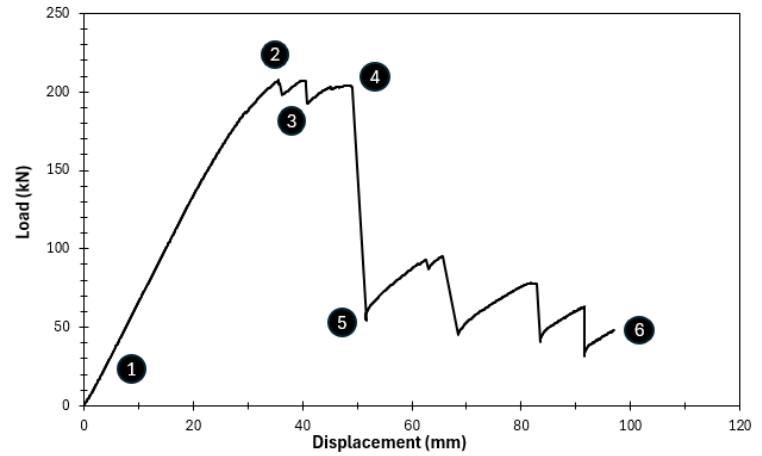
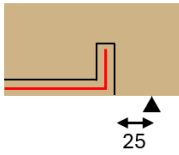
(4) Simple tension failure



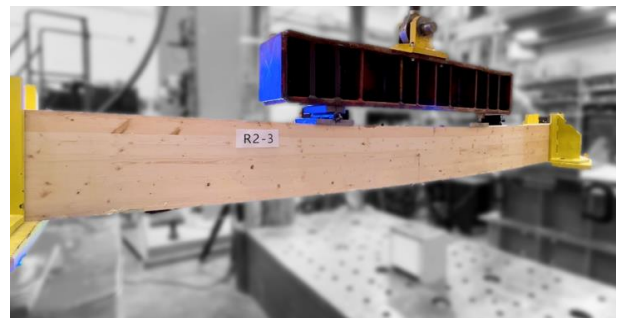
(5) End of test

Figure D.12 Failure progression for R2-2

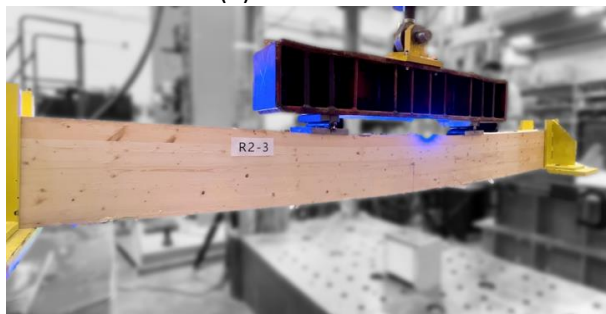
Specimen: R2-3
Type: : Reinforced, long bars, bent



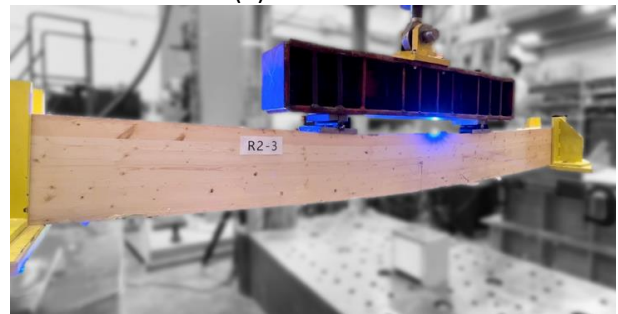
(1) Start of test



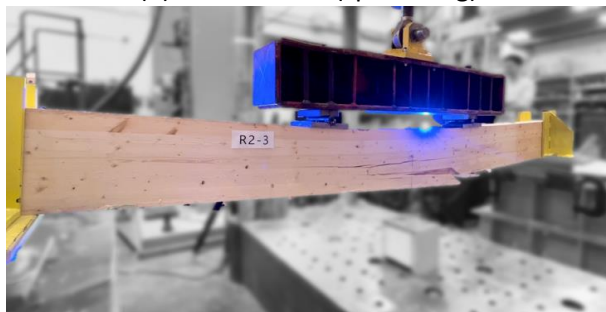
(2) Peak Load



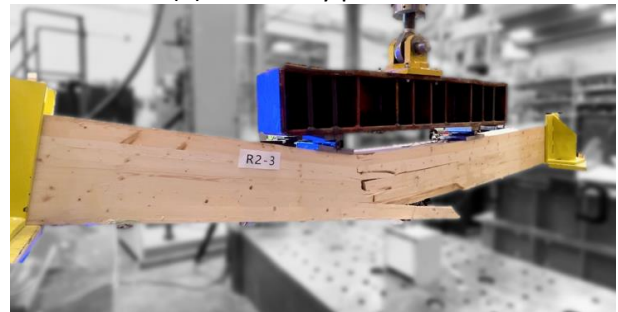
(3) Initial failure (splintering)



(4) Secondary peak load



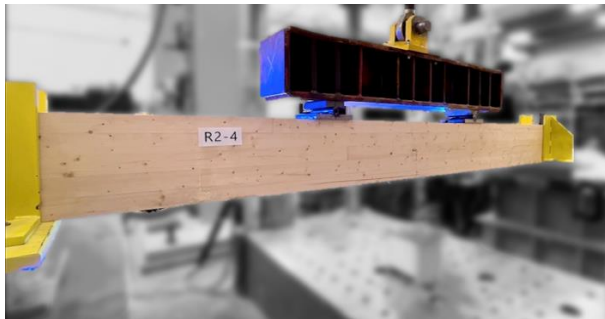
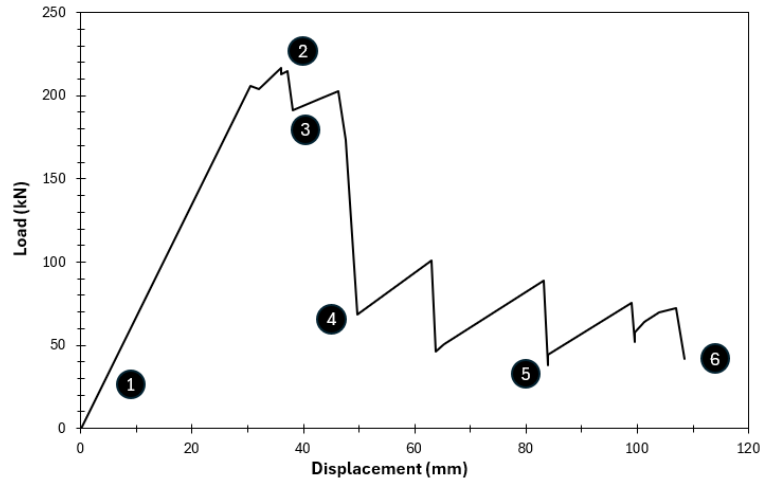
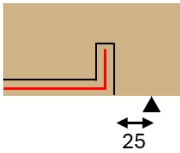
(5) Secondary failure



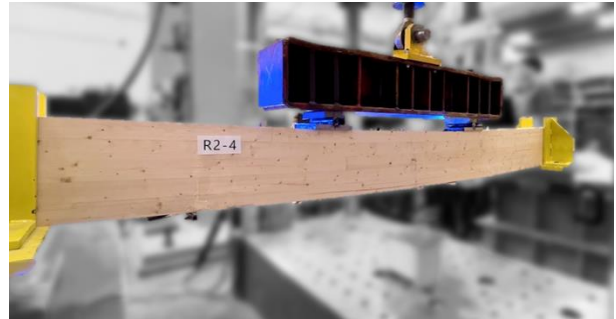
(6) End of test

Figure D.13 Failure progression for R2-3

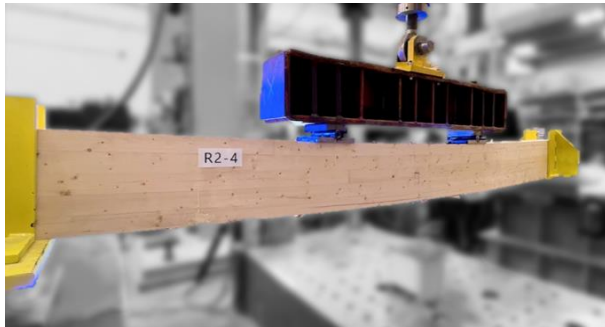
Specimen: R2-4
Type: : Reinforced, long bars, bent



(1) Start of test



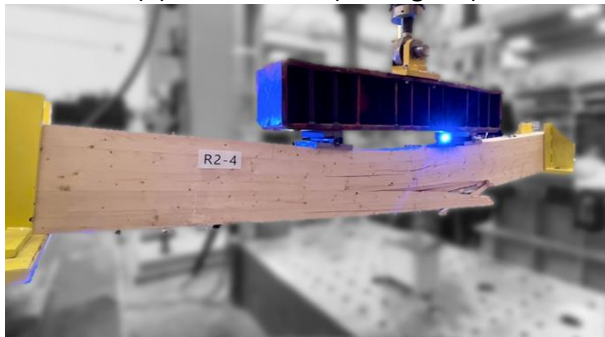
(2) Peak Load



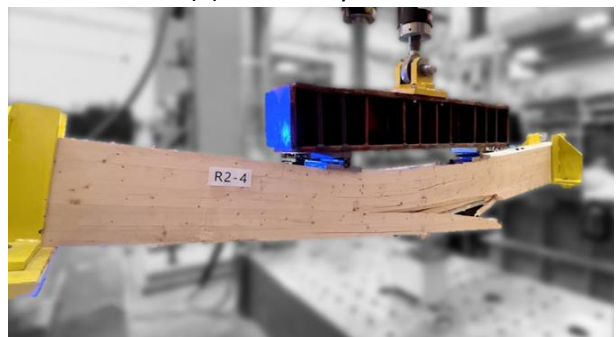
(3) Initial failure (cross-grain)



(4) Secondary failure



(5) Failures continue to propagate



(6) End of test

Figure D.14 Failure progression for R2-4

APPENDIX E | FORCE AND STRAIN DISPLACEMENTS

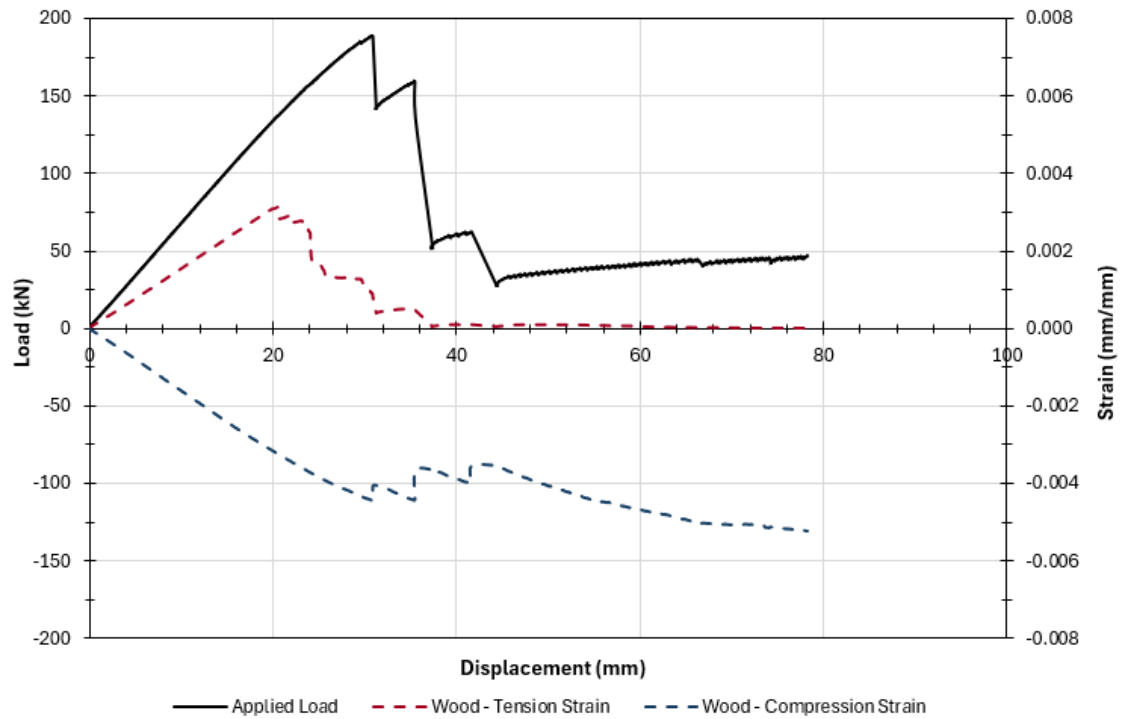


Figure E.1 Strain distribution for U-01

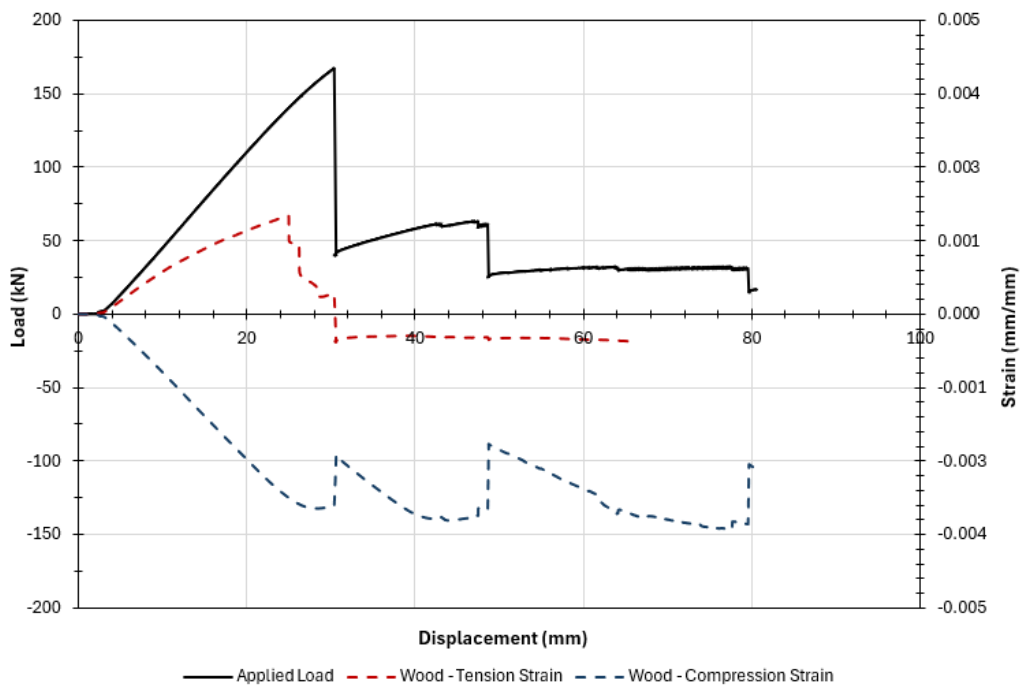


Figure E.2 Strain distribution for U-02

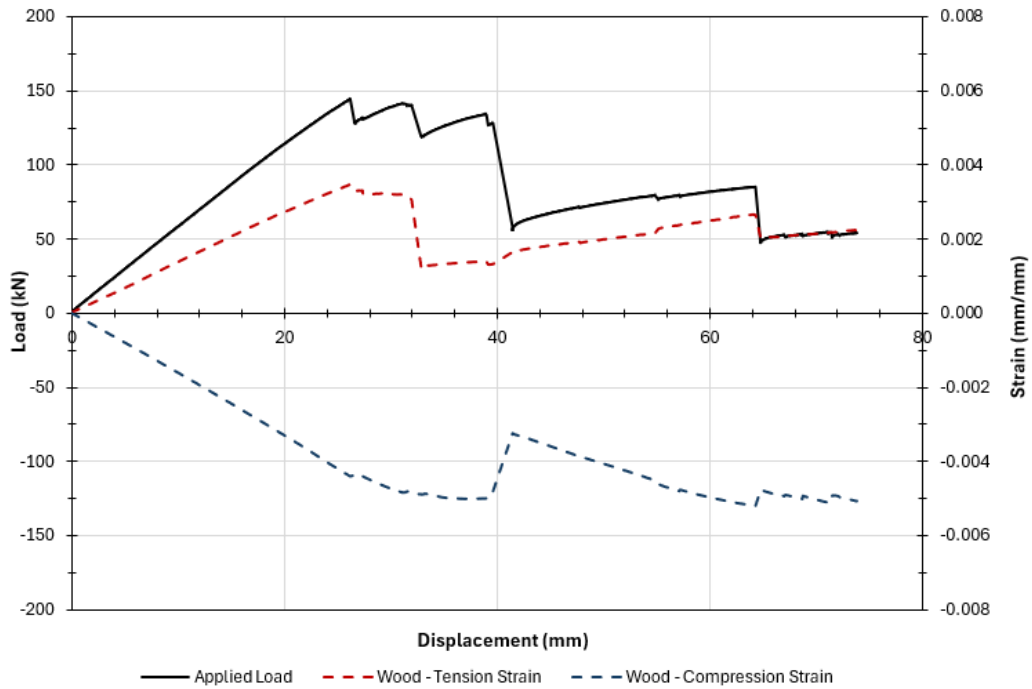


Figure E.3 Strain distribution for U-03

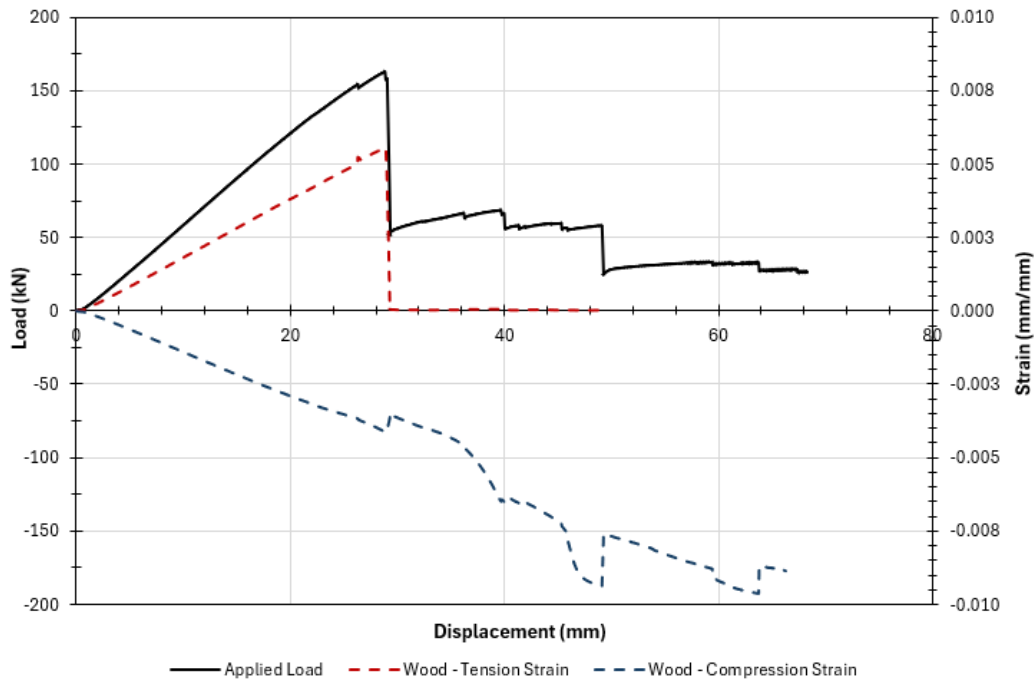


Figure E.4 Strain distribution for U-04

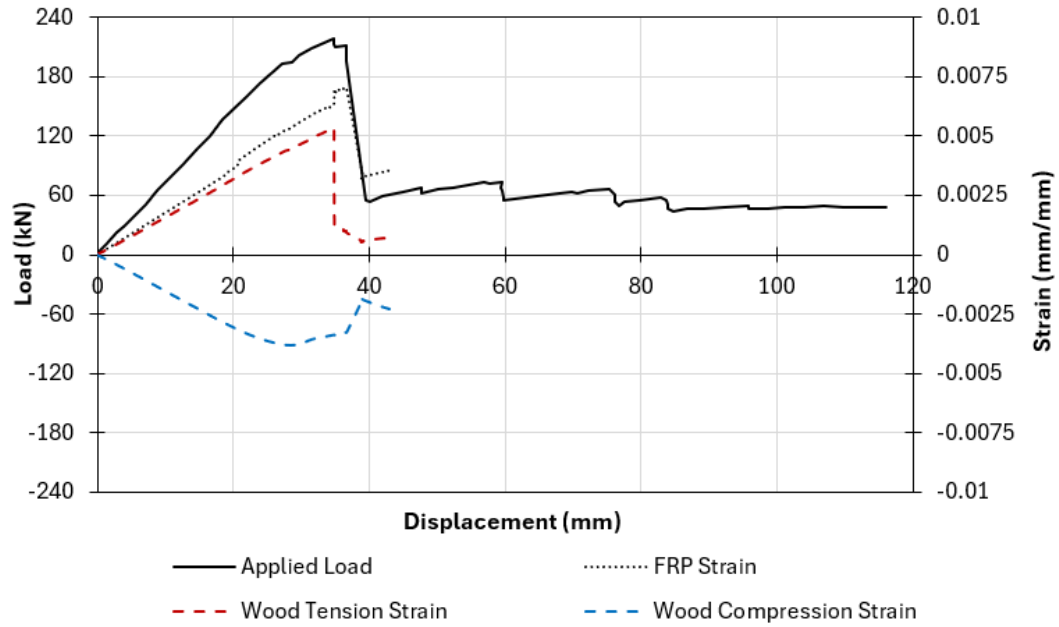


Figure E.5 Strain distribution for R1-1

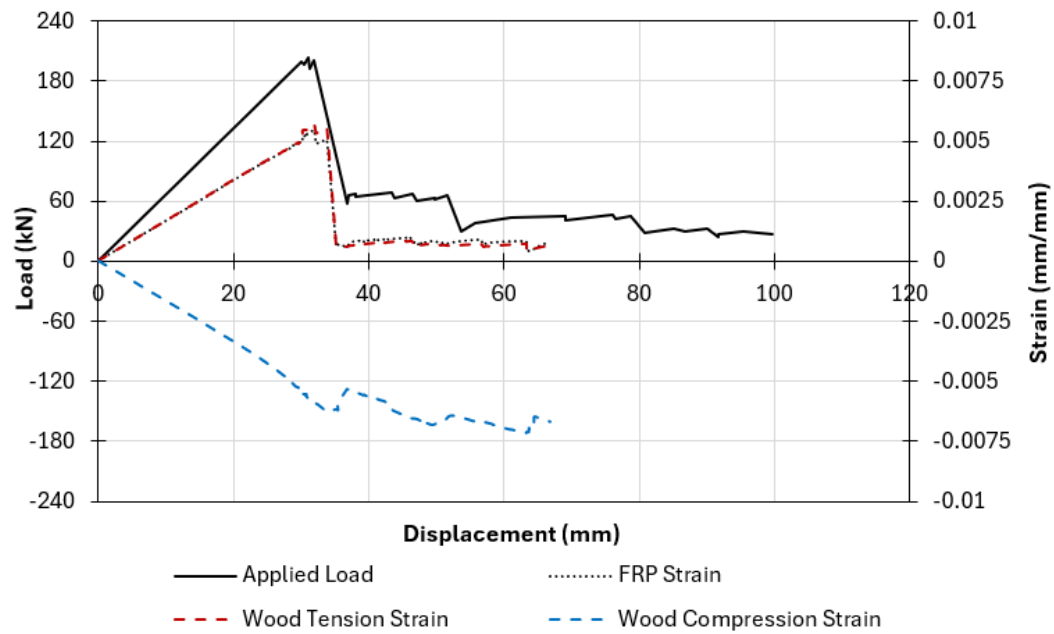


Figure E.6 Strain distribution for R1-2

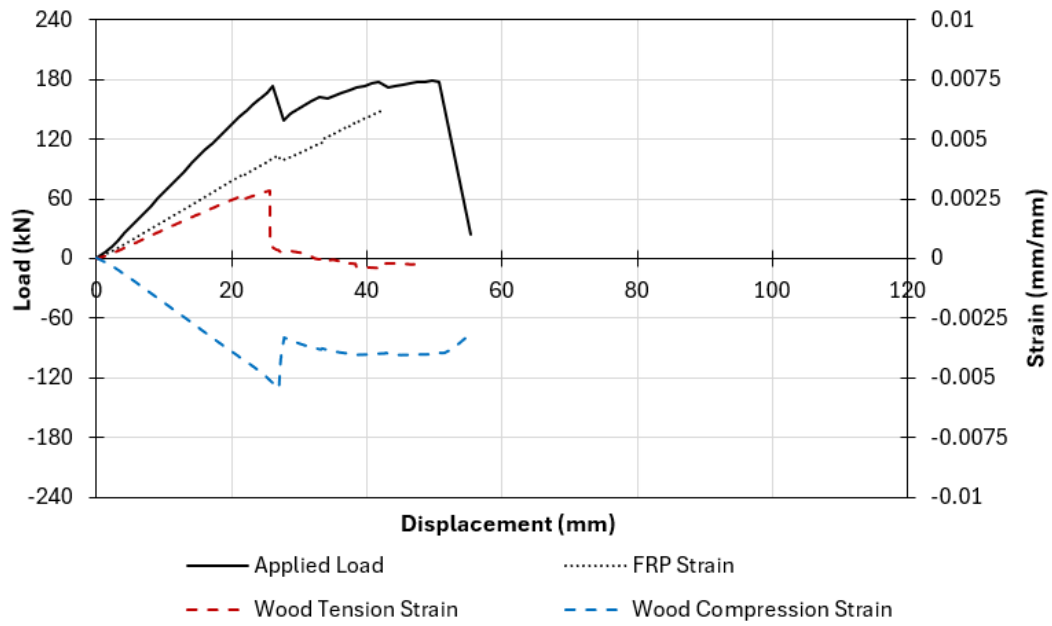


Figure E.7 Strain distribution for R1-3

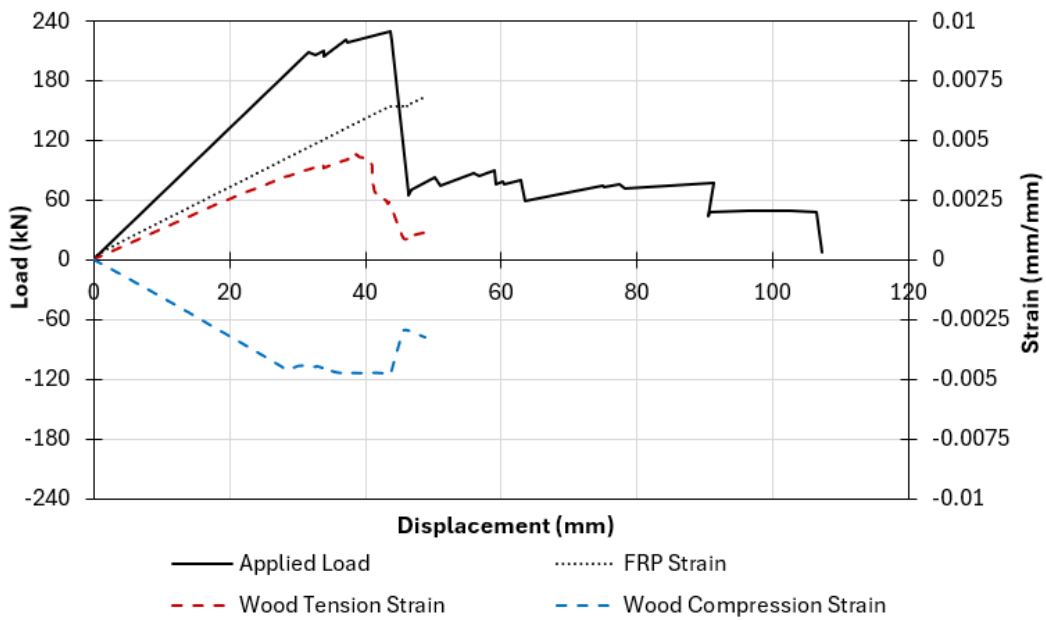


Figure E.8 Strain distribution for R1-4

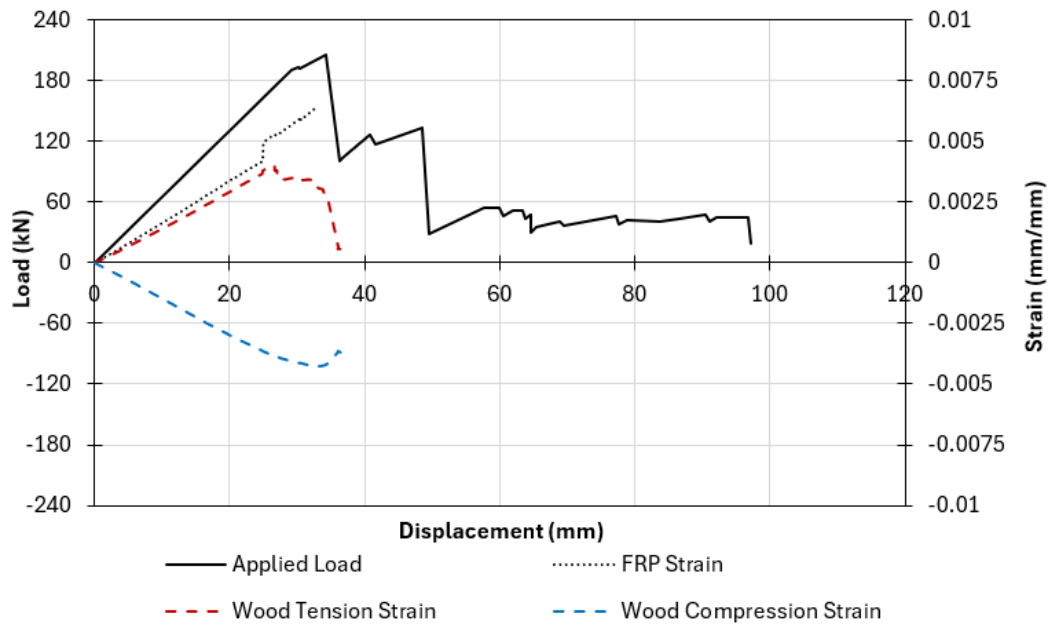


Figure E.9 Strain distribution for R1-5

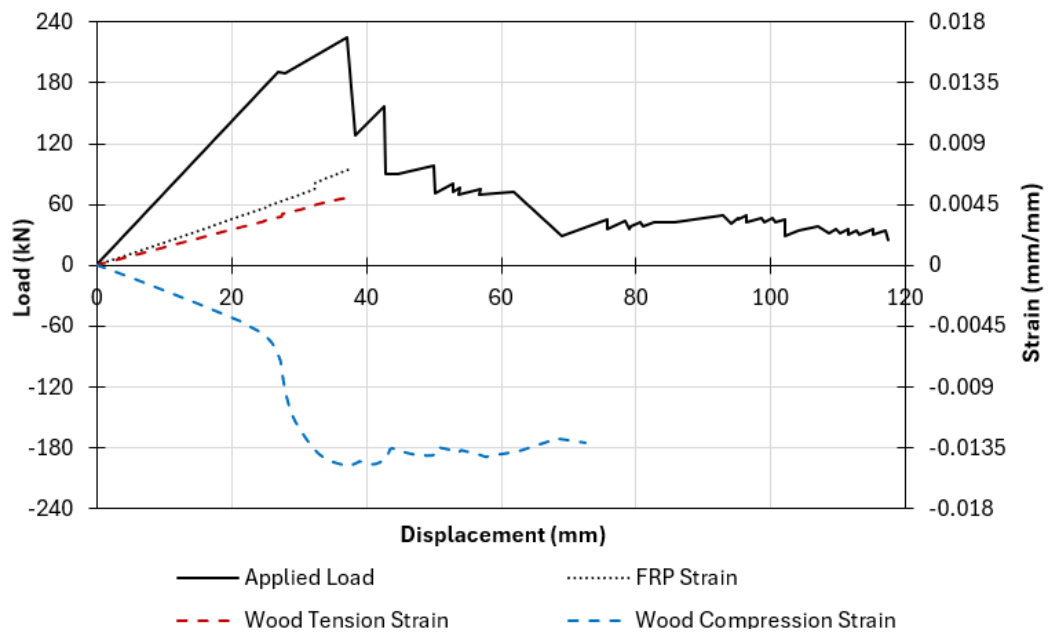


Figure E.10 Strain distribution for R1-6

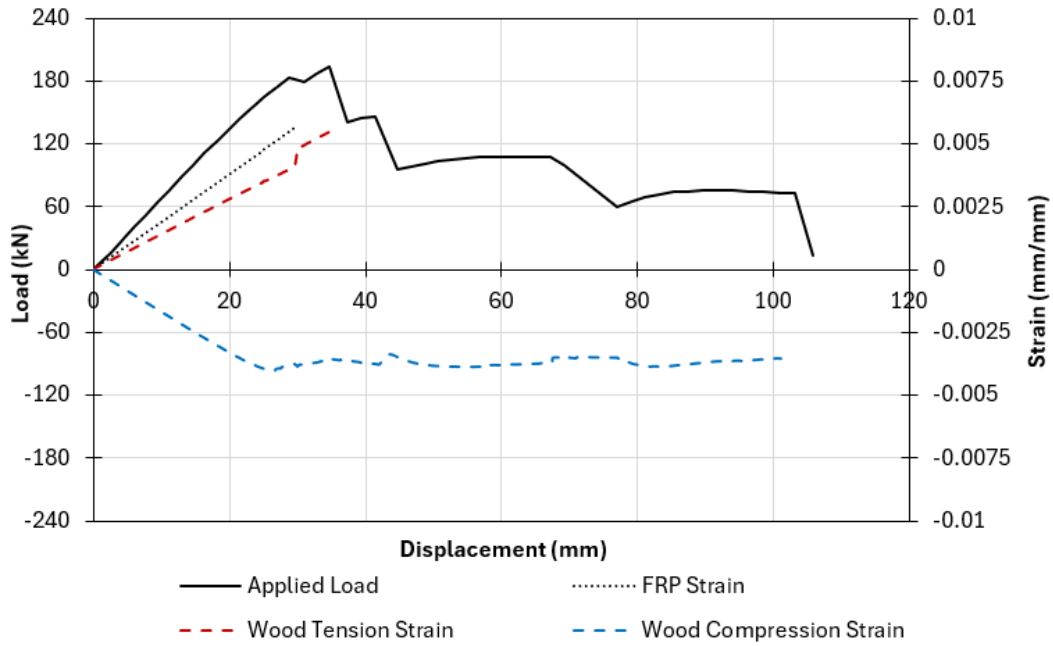


Figure E.11 Strain distribution for R2-1

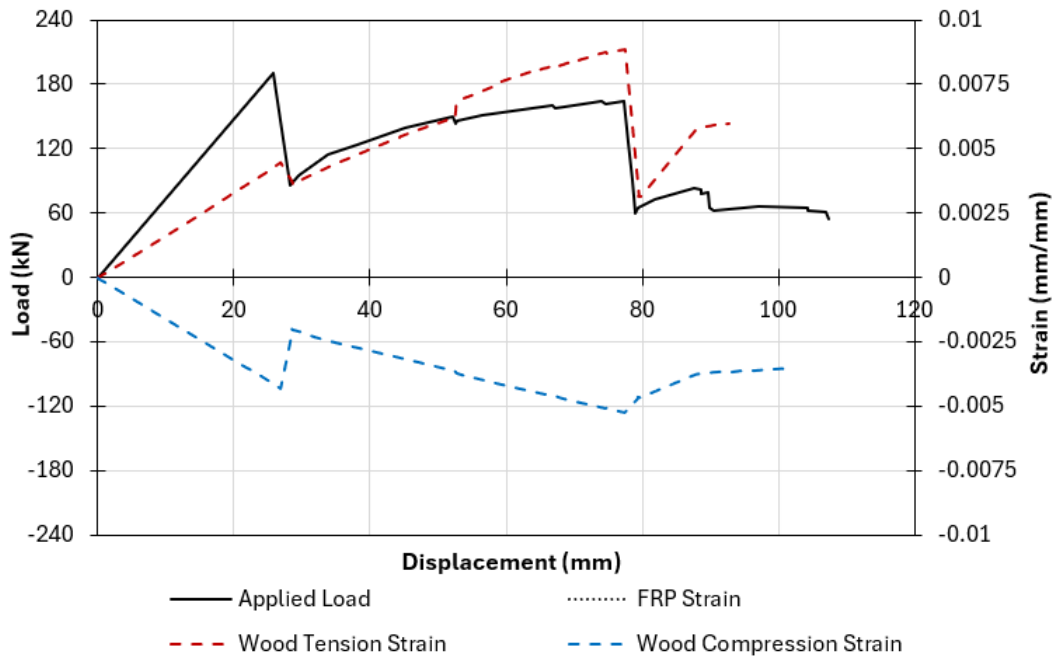


Figure E.12 Strain distribution for R2-2

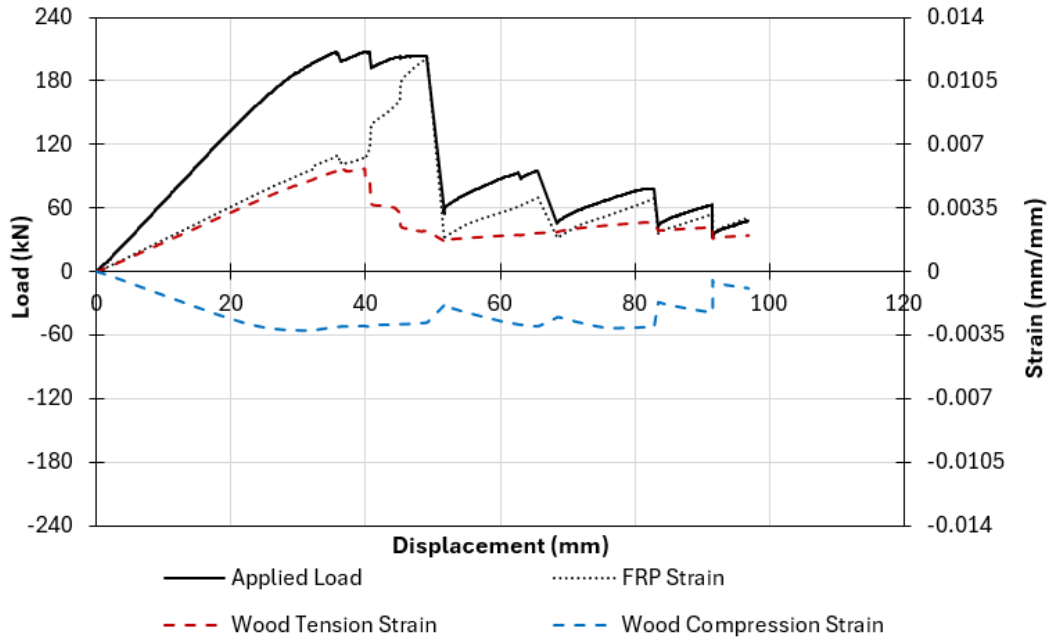


Figure E.13 Strain distribution for R2-3

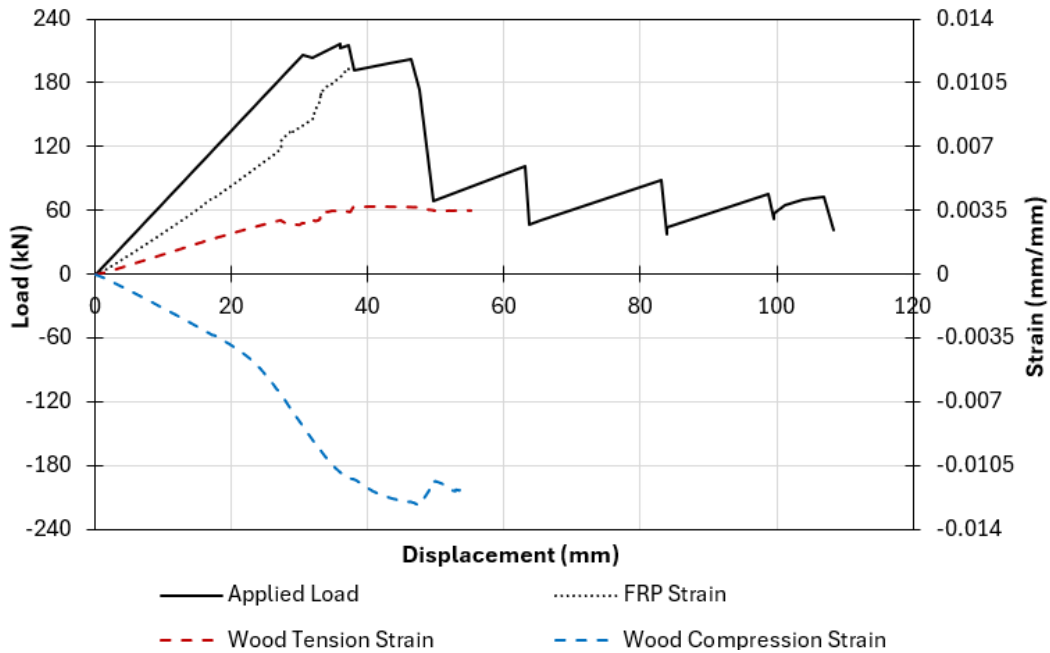
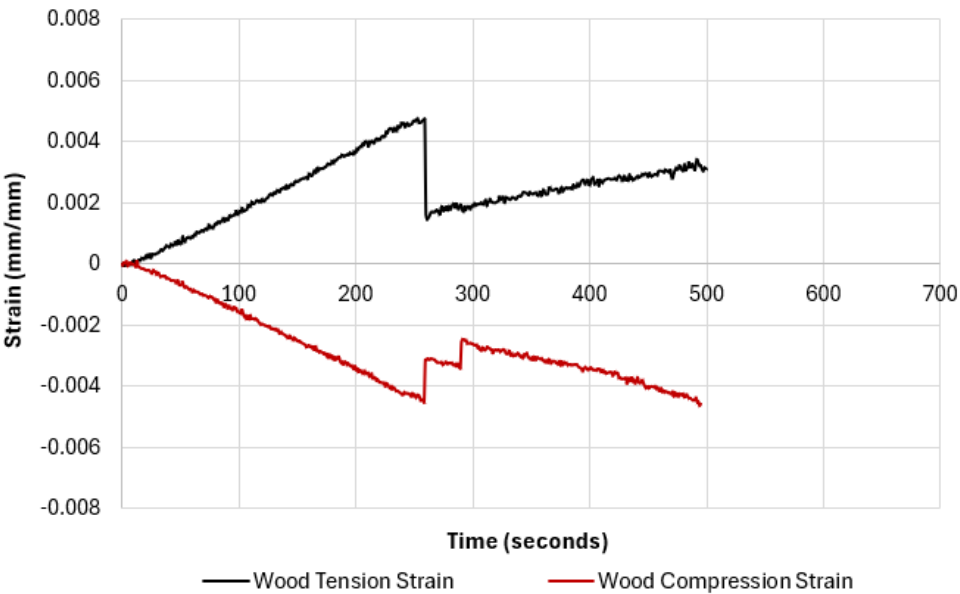
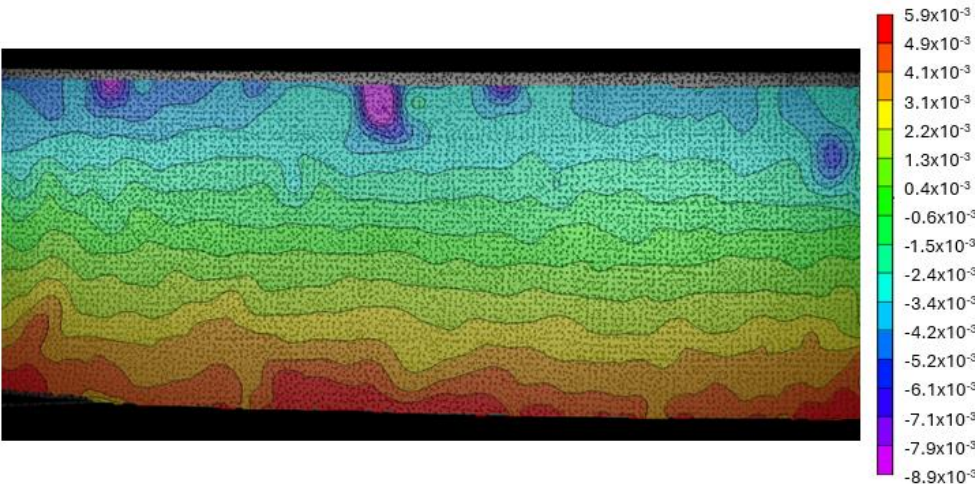


Figure E.14 Strain distribution for R2-4

APPENDIX F | DIGITAL IMAGE CORRELATION RESULTS



(a) Strain-time history



(b) Strain contours prior to failure

Figure F.1 DIC strain distribution for U-01

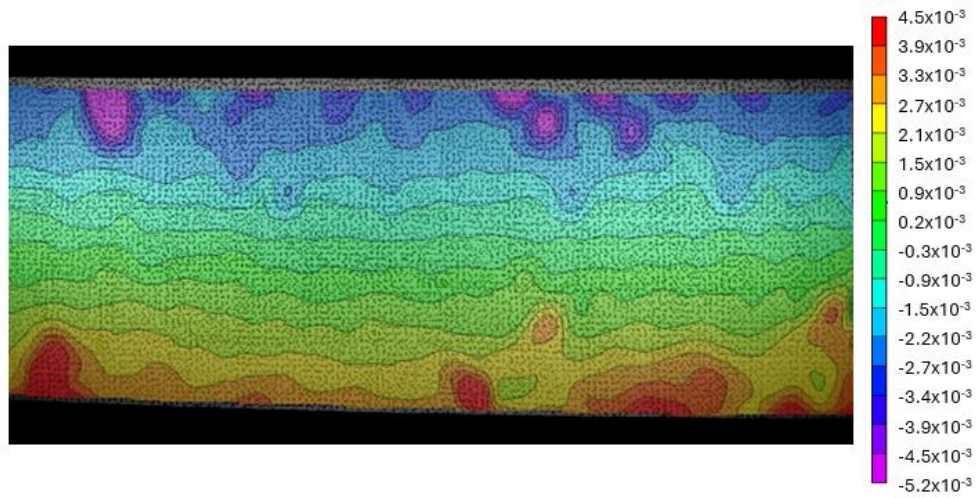
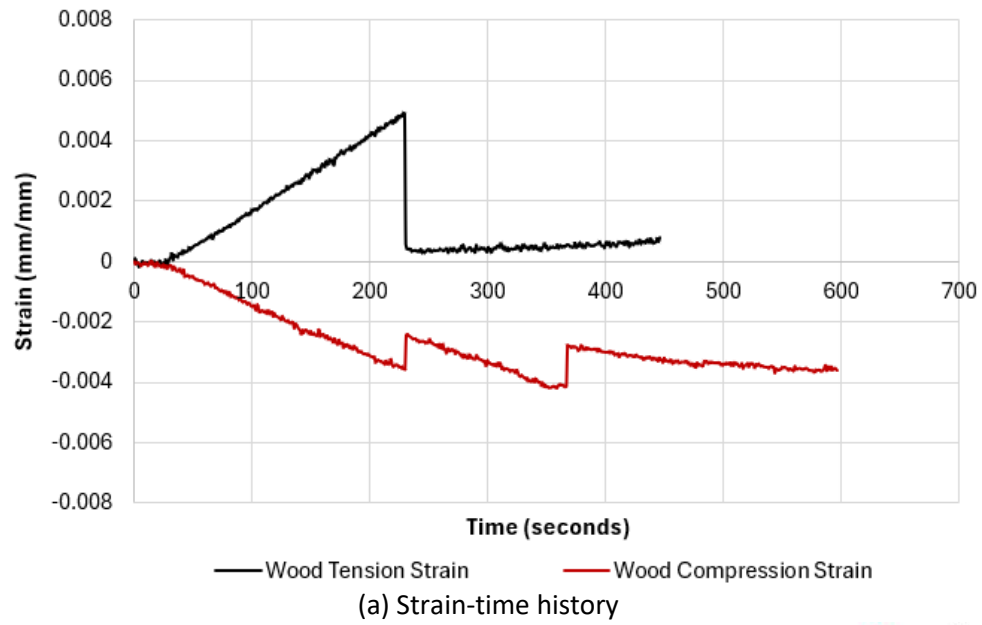
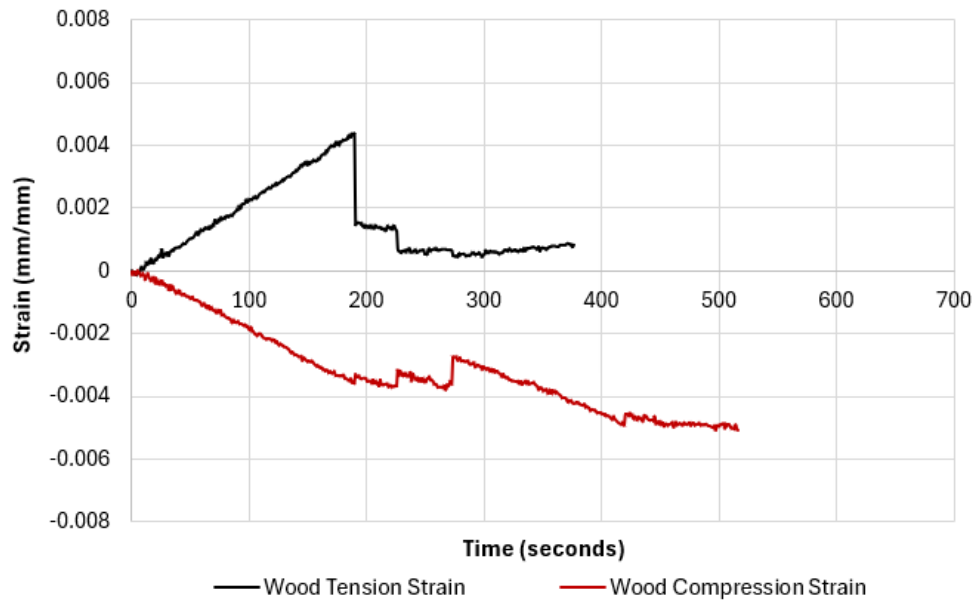
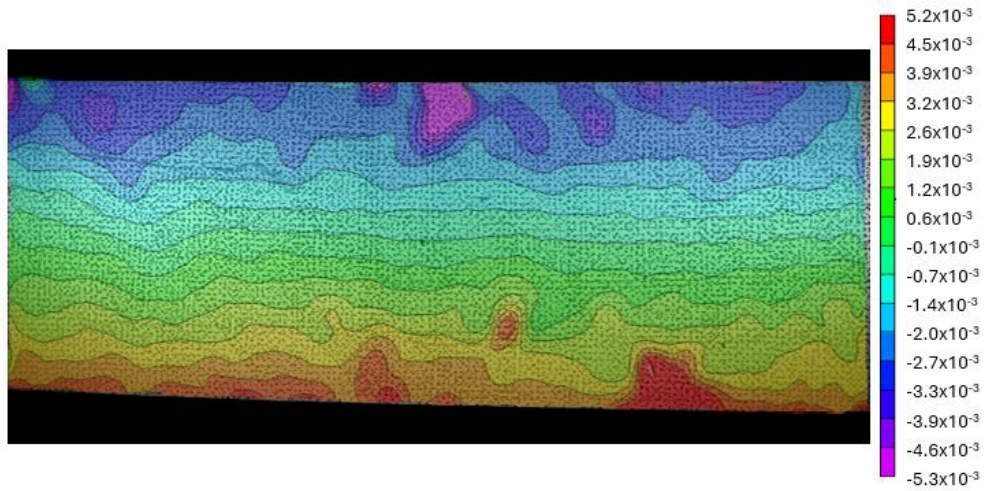


Figure F.2 DIC strain distribution for U-02

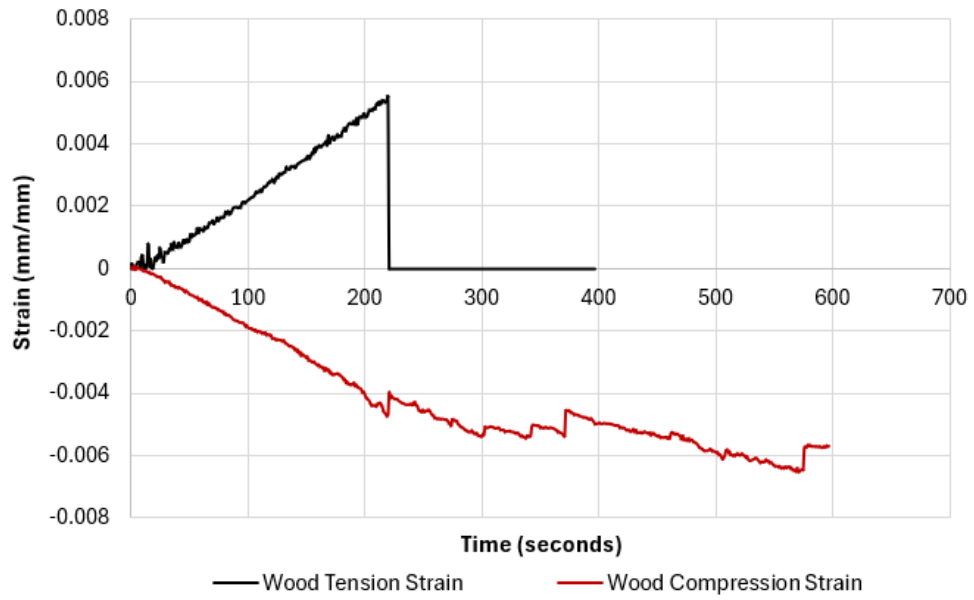


(a) Strain-time history

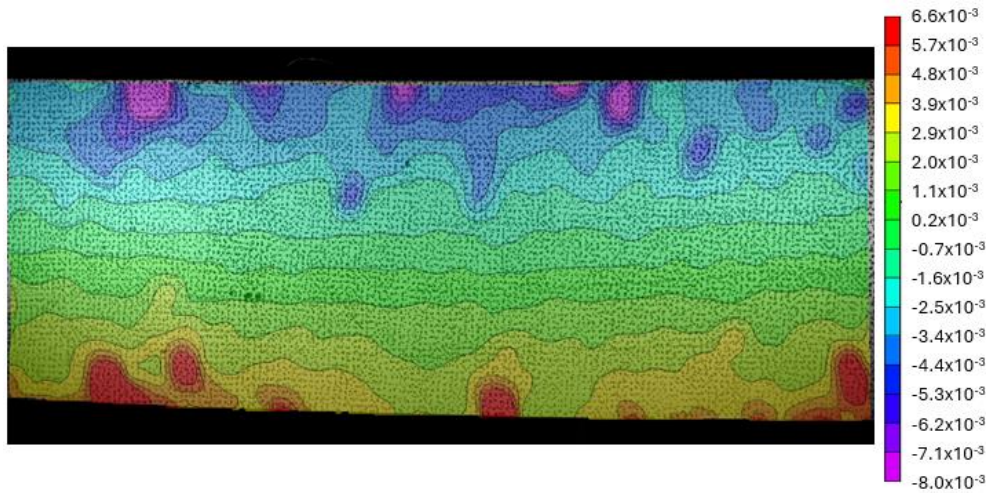


(b) Strain contours prior to failure

Figure F.3 DIC strain distribution for U-03

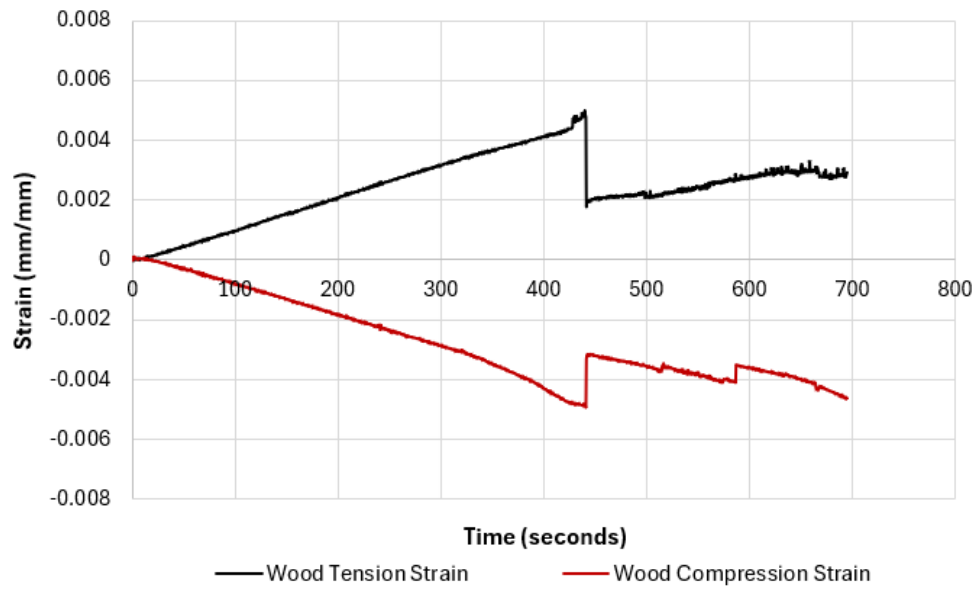


(a) Strain-time history

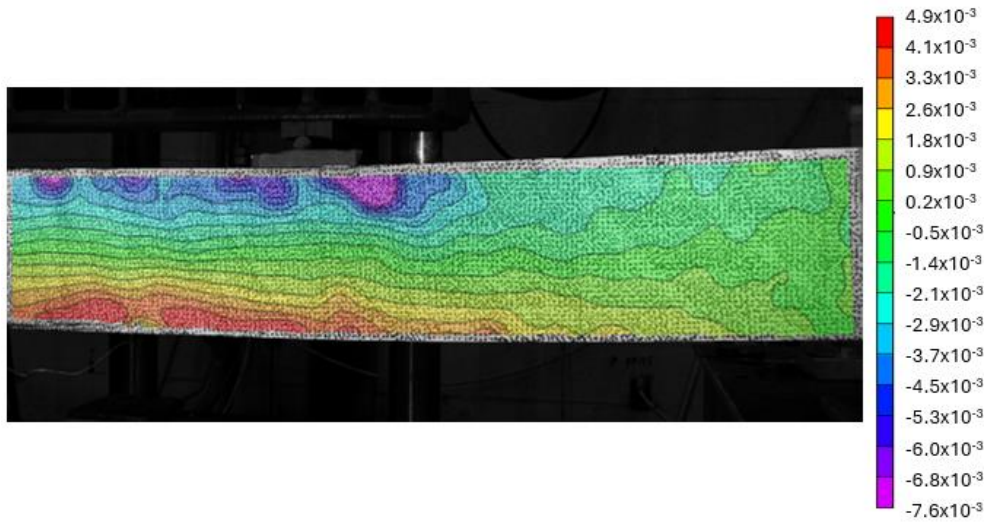


(b) Strain contours prior to failure

Figure F.4 DIC strain distribution for U-04

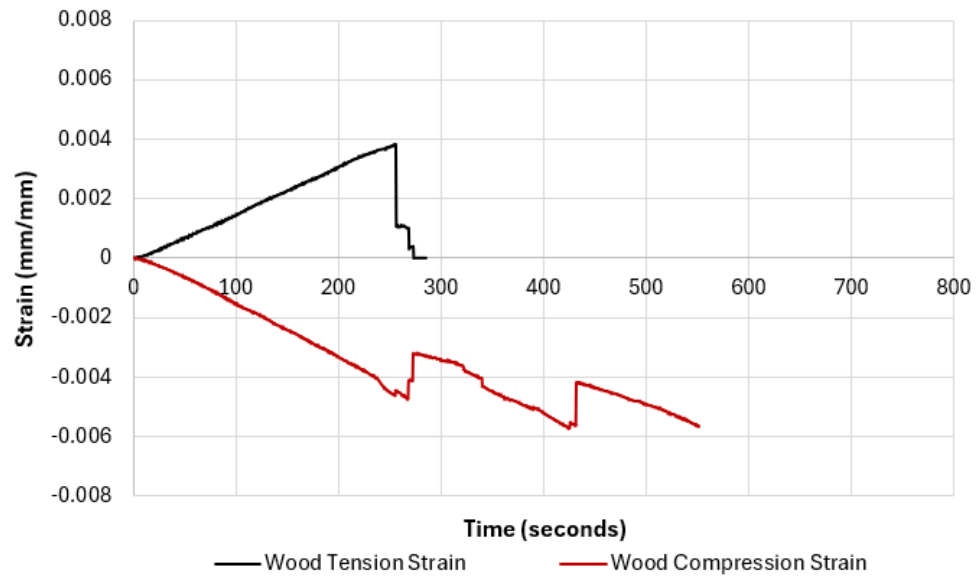


(a) Strain-time history

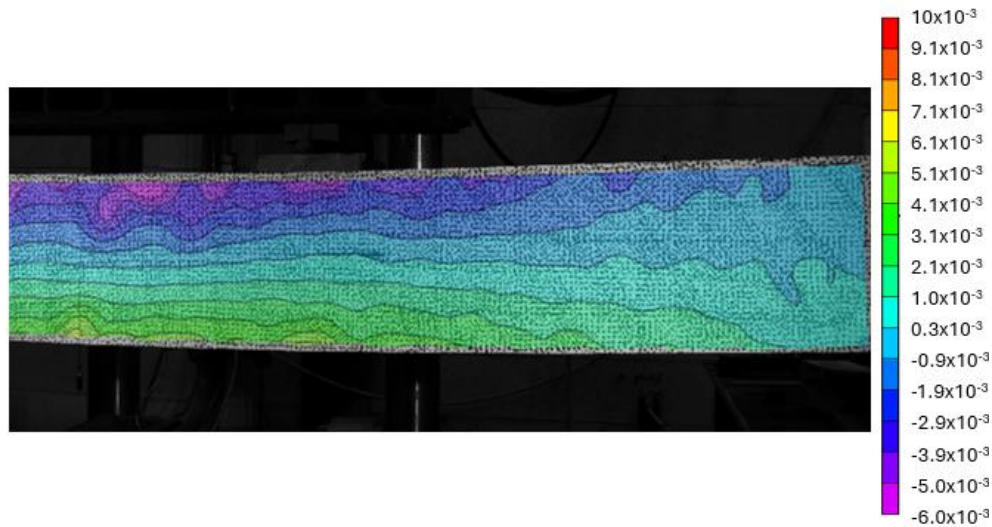


(b) Strain contours prior to failure

Figure F.5 DIC strain distribution for R1-1

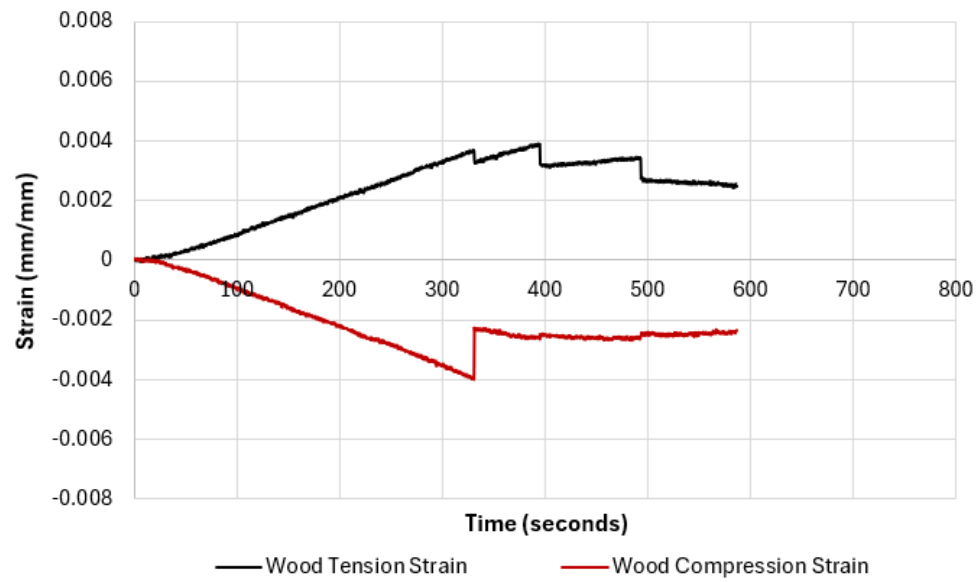


(a) Strain-time history

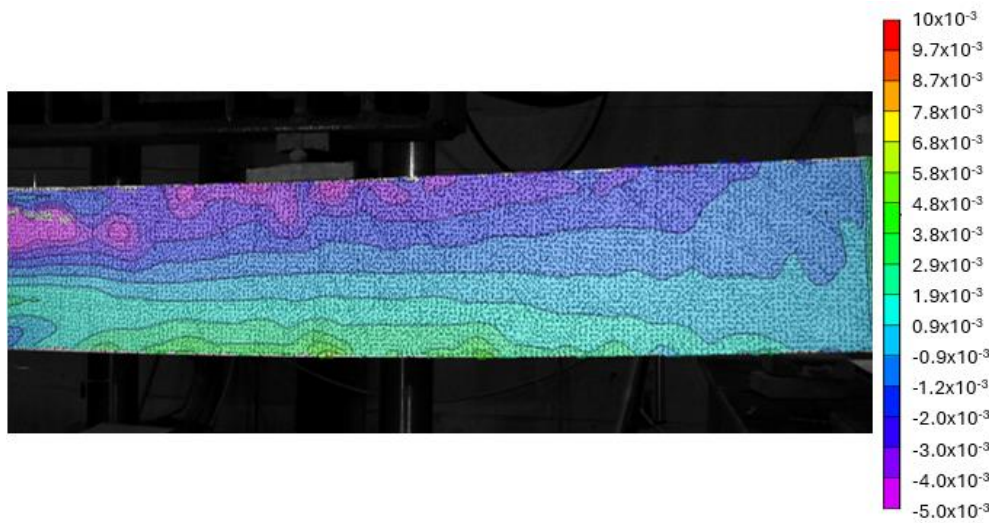


(b) Strain contours prior to failure

Figure F.6 DIC strain distribution for R1-2

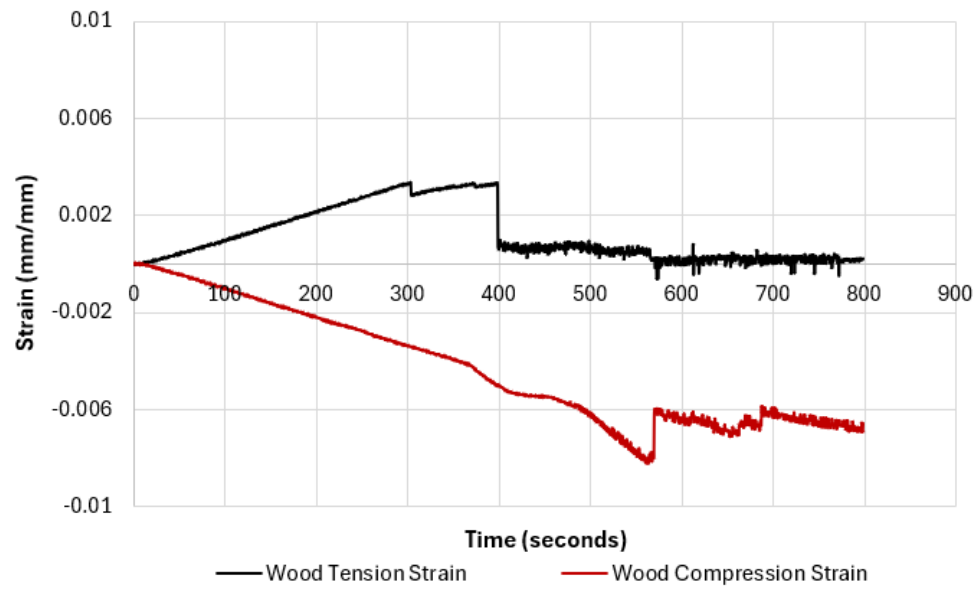


(a) Strain-time history

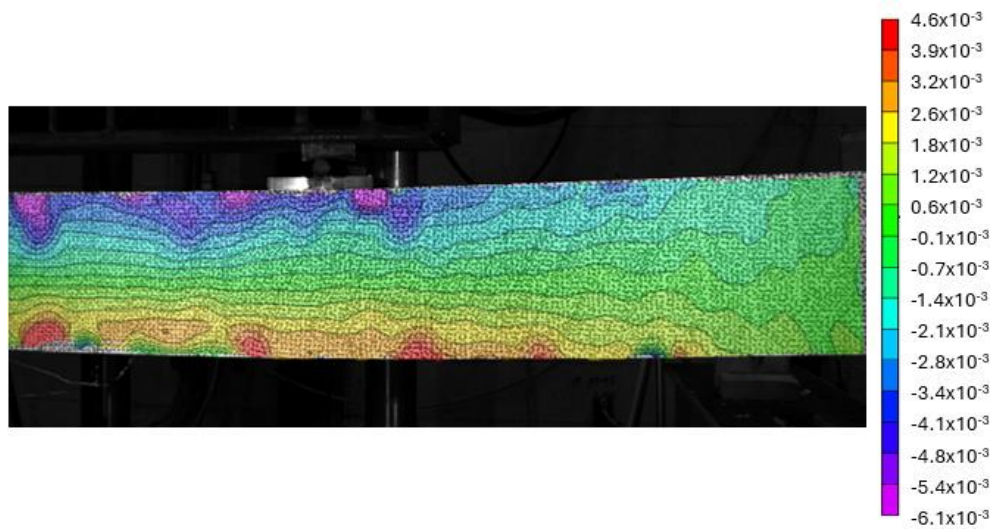


(b) Strain contours prior to failure

Figure F.7 DIC strain distribution for R1-3

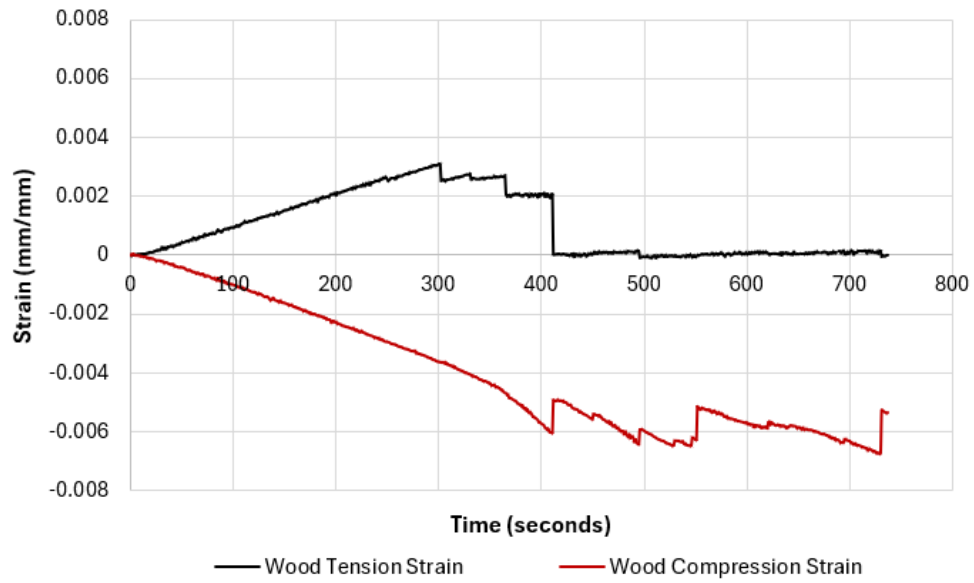


(a) Strain-time history

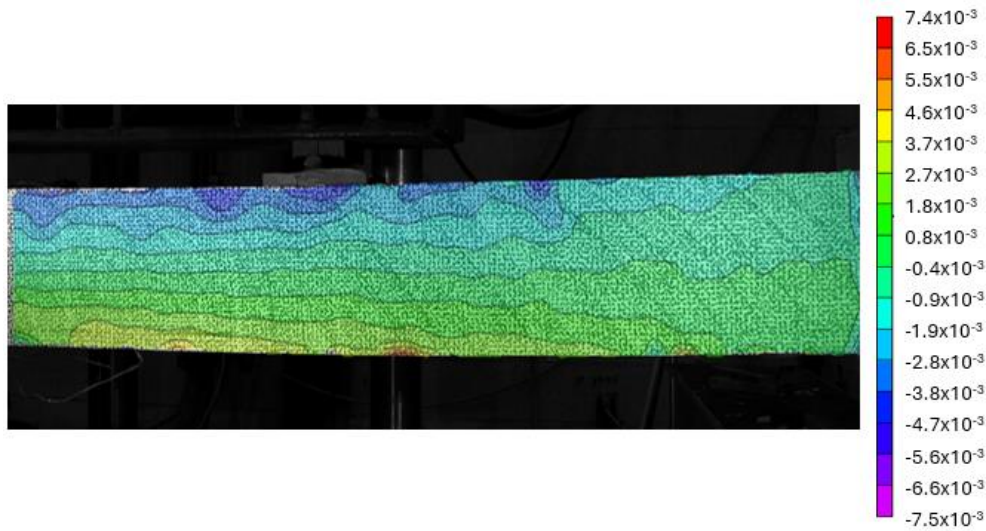


(b) Strain contours prior to failure

Figure F.8 DIC strain distribution for R1-4

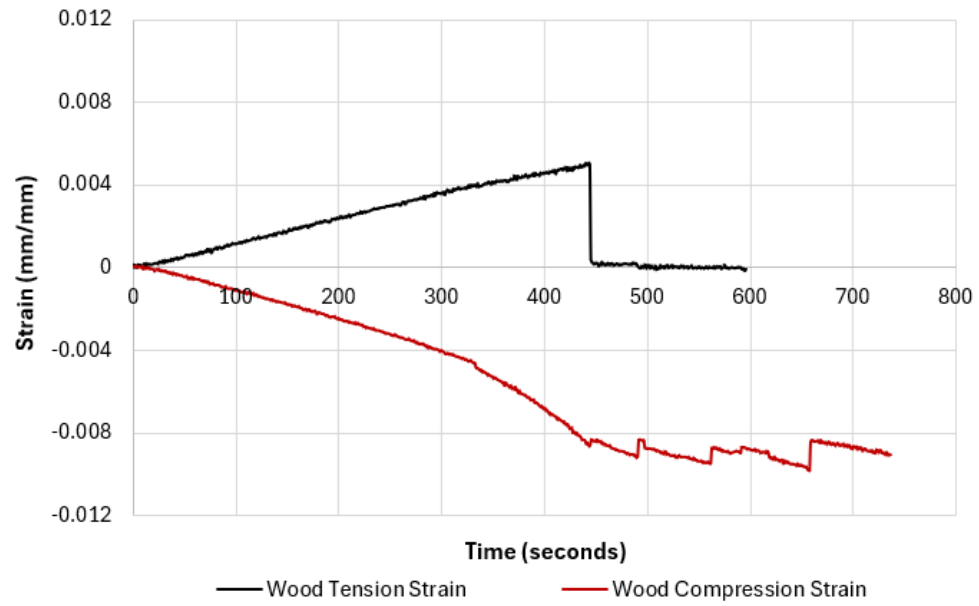


(a) Strain-time history

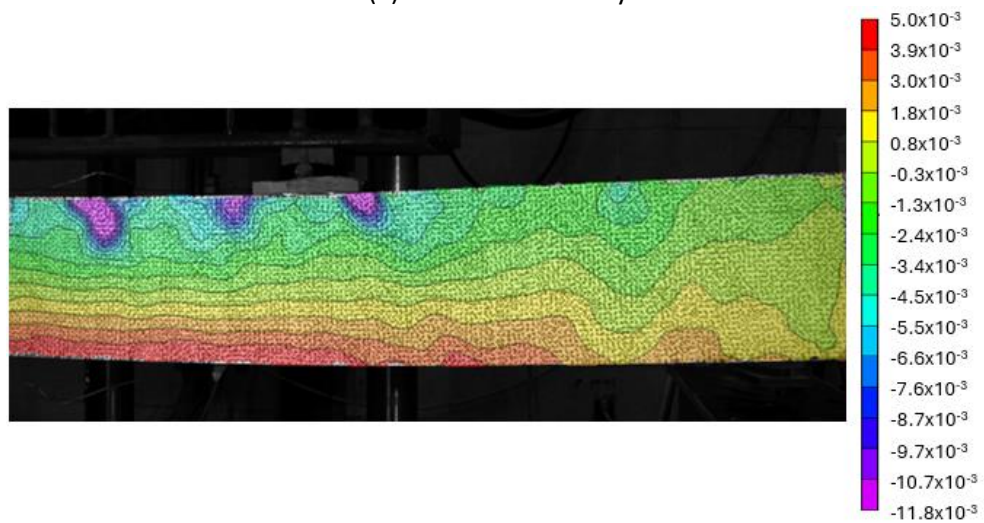


(b) Strain contours prior to failure

Figure F.9 DIC strain distribution for R1-5

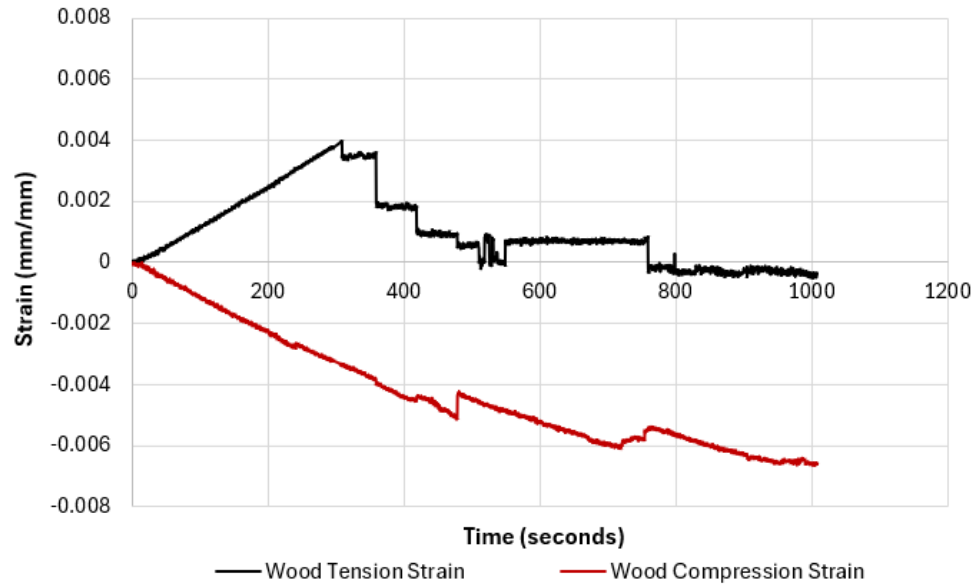


(a) Strain-time history

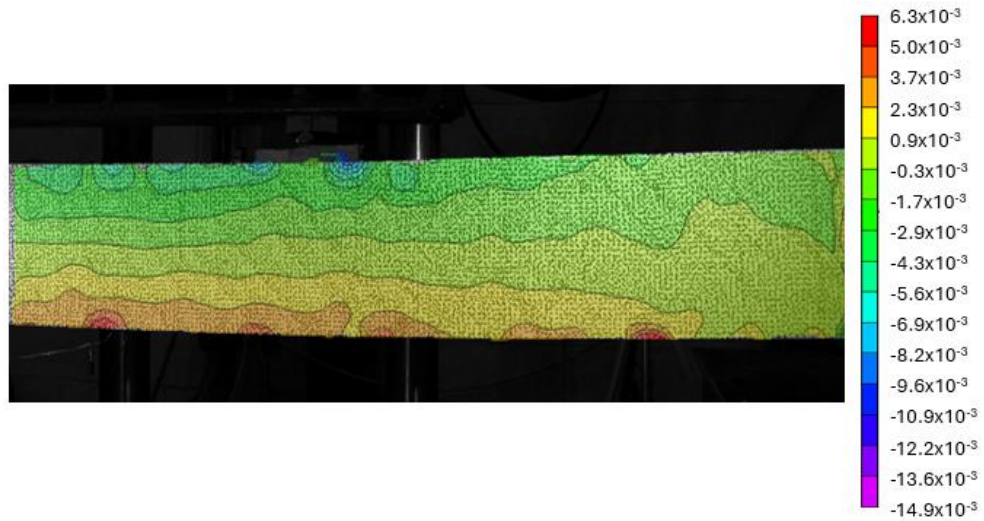


(b) Strain contours prior to failure

Figure F.10 DIC strain distribution for R1-6



(a) Strain-time history



(b) Strain contours prior to failure

Figure F.11 DIC strain distribution for R2-1

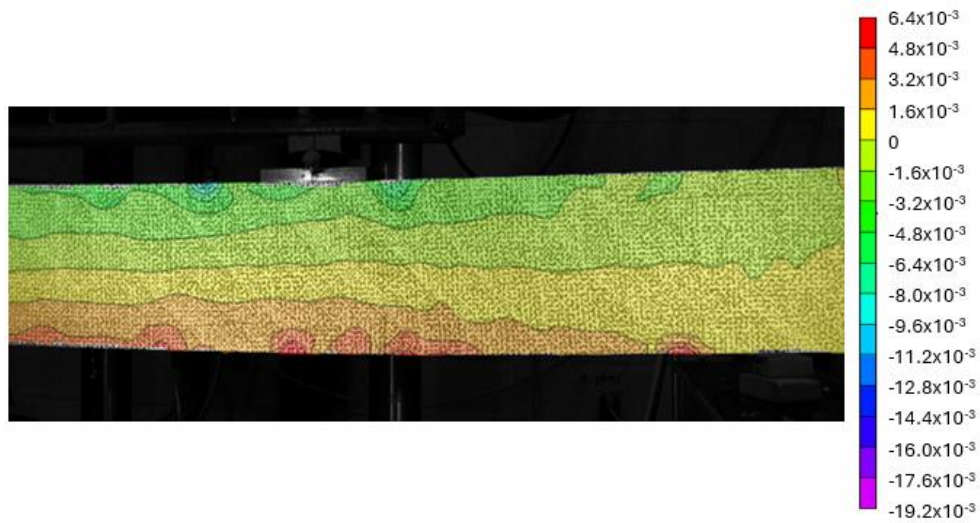
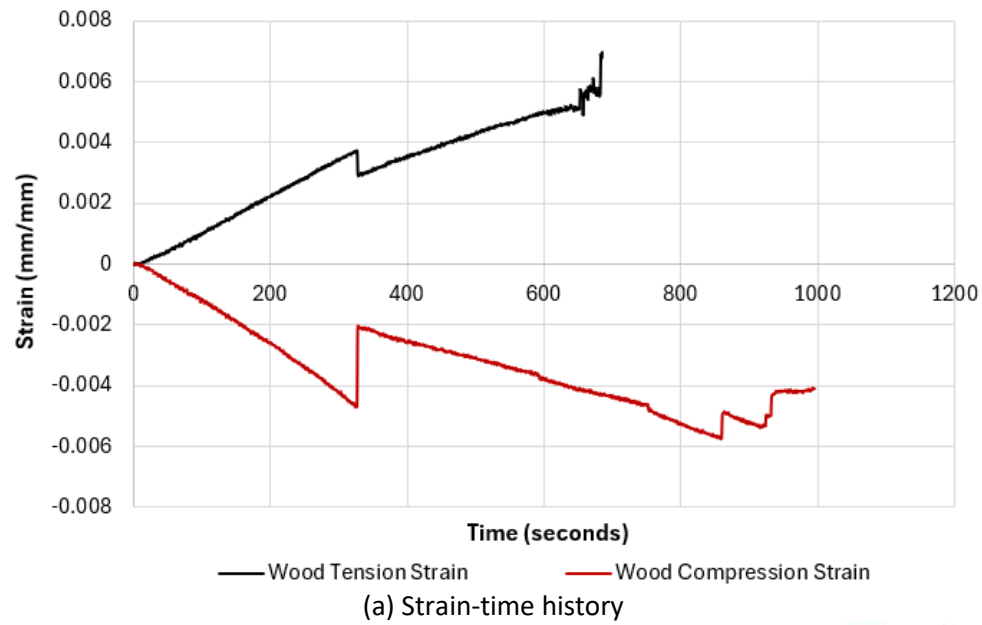
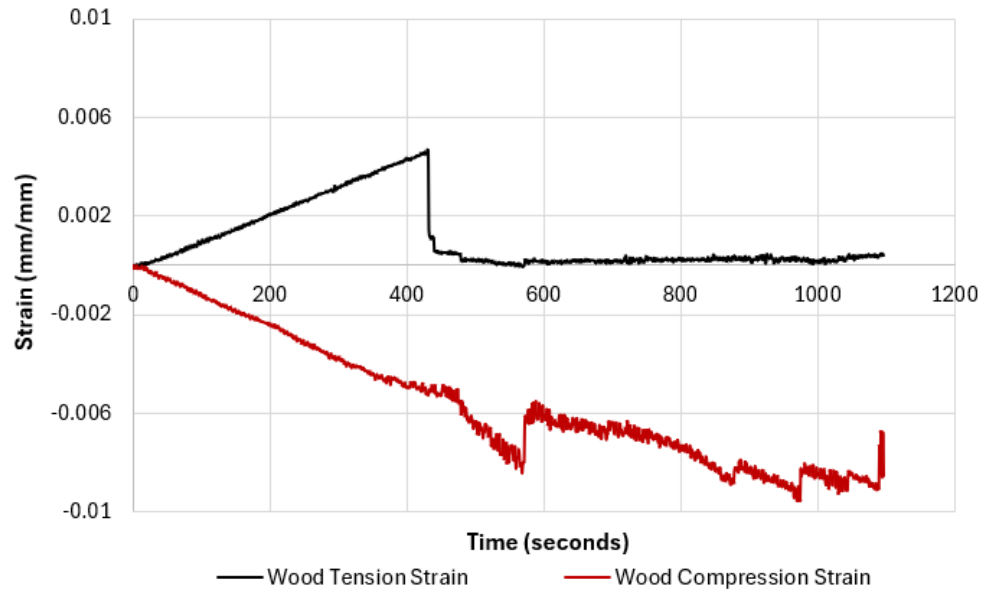
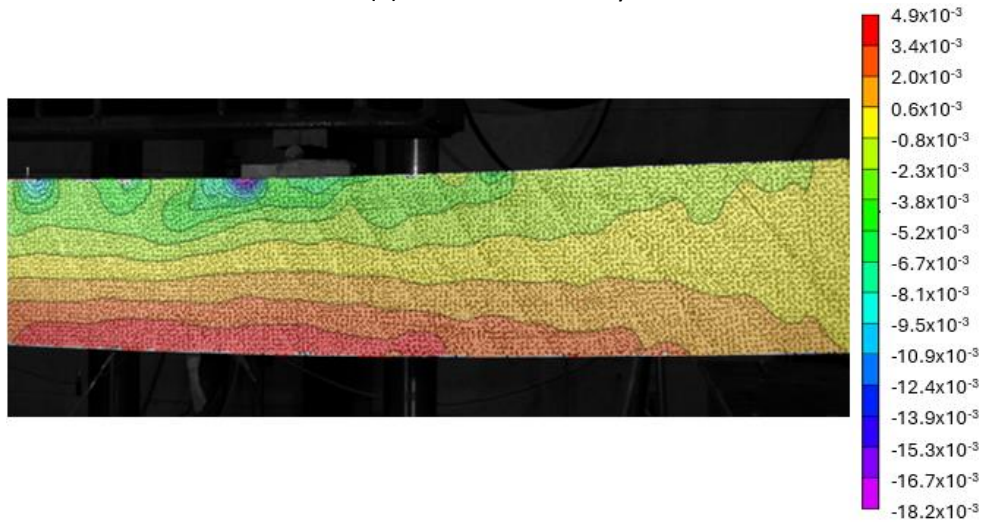


Figure F.12 DIC strain distribution for R2-2



(a) Strain-time history



(b) Strain contours prior to failure

Figure F.13 DIC strain distribution for R2-3

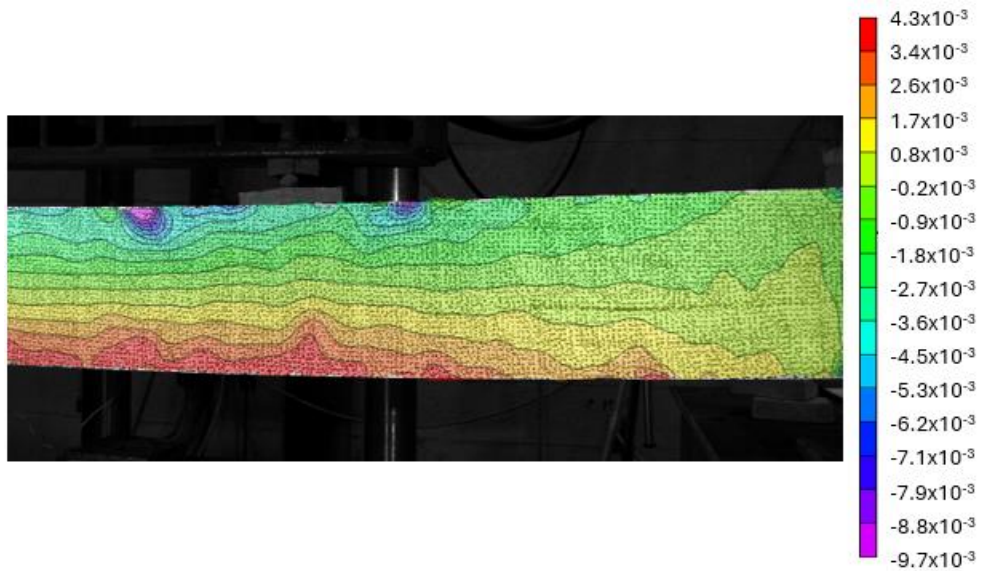
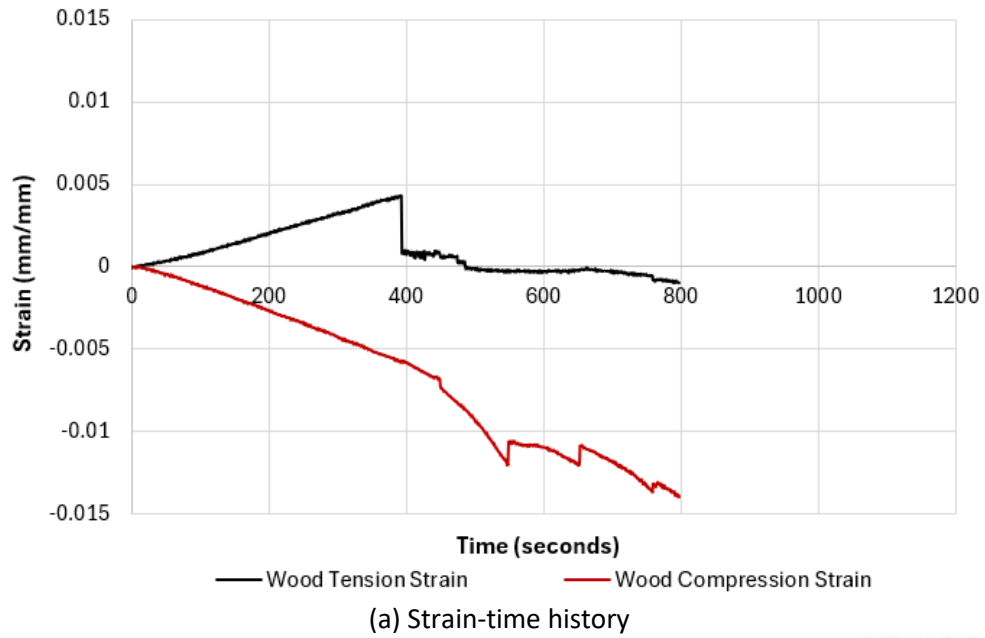


Figure F.14 DIC strain distribution for R2-4