An investigation of the viscoelastic and mechano-sorptive creep behaviour of reinforced timber elements

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An Investigation of the Viscoelastic and Mechano-sorptive Creep Behaviour of Reinforced Timber Elements

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ABSTRACT

The reinforcement of timber using fibre reinforced polymer (FRP) rods or plates is widely accepted as an effective method of increasing the strength and stiffness of members. Across Europe, this technology has been used, not only in new structures, but in the upgrading and repair of existing structures. When retrofitting these structures, changes in use of the building or, indeed, changes in building regulations often require a higher load capacity than that of the existing members. The additional capacity requirements can be successfully achieved in a timely and cost-effective manner through the use of FRP reinforcement. More widespread use of this technology has been hampered by the lack of a harmonised standard governing their design. Currently, design rules for FRP reinforcement are not included in Eurocode 5, which is the European standard governing the design of timber structures. The reasons for this are partly due to a lack of knowledge, particularly related to the long-term performance.

This body of research aims to help researchers and design engineers to better understand the long-term or creep performance of FRP reinforced timber elements and to contribute to the development of design approaches, which account for the influence of reinforcement on this behaviour. This requires an examination of the main mechanical responses that are commonly observed in timber members under long-term loading, namely, the elastic, viscoelastic, mechnano-sorptive and swelling/shrinkage responses. An experimental programme was designed to identify and characterise these individual responses on matched groups of glued laminated beams having statistically equal bending stiffness. This experimental programme involved the simultaneous loading of an equal proportion of unreinforced and reinforced beams to a common bending stress on the compression face in constant and variable climates for a period of 75 weeks. In the constant climate condition, the unreinforced and reinforced beams were subject to elastic and viscoelastic creep behaviour. In the variable climate condition, the beams were subject to elastic and viscoelastic creep behaviour and also, as a result of the variable climate condition, were subject to hygro-mechanical behaviour, which includes mechano-sorptive and swelling/shrinkage behaviour. The mid-span vertical deflection and longitudinal strain on the tension and compression faces were continuously monitored. No statistically significant difference was found between the creep deflection of unreinforced and reinforced beams in the constant climate condition. A key conclusion is
that the viscoelastic creep deflection is governed by the stress level in the timber and is independent of the reinforcement. In the variable climate, the total deflection dramatically increased due to the variable moisture content conditions. In this case, there was a statically significant difference between the creep deflection of the unreinforced and reinforced beams with a reduction in deflection due to the reinforcement. By analysing the strain measurements from beams in the constant and variable climates together with separate swelling/shrinkage measurements on matched groups, it was possible to separate and characterise each individual strain component. The test methodology was specifically designed for this purpose. Significantly, the mechano-sorptive creep component has been separately quantified unlike in previous work where the mechano-sorptive and swelling/shrinkage effects have been combined. This separation has led to a significant finding in relation the influence of these components on the response. No statistically significant difference in the mechano-sorptive strain behaviour due to the reinforcement was observed. It was found that the main difference in the hygro-mechanical response is due to the different swelling/shrinkage behaviour of unreinforced and reinforced beams. The reinforcement restrains the swelling/shrinkage behaviour in the timber resulting in less relative creep deflection in reinforced members. This means that the influence of the reinforcement on hygro-mechanical response may be characterised through short-term swelling/shrinkage tests.

A hygro-mechanical creep model was developed to predict the interaction between stress and moisture content change in timber elements when simultaneously loaded under a constant dead load and subject to a variable relative humidity condition with time. User-defined UMAT and DFLUX subroutines were utilised to define the material behaviour and variable relative humidity boundary conditions, respectively. Material characterisation tests were performed on Irish-grown Sitka spruce and BFRP rod reinforcement to provide the necessary material data for the numerical models. The numerical deflection and strain results have shown good agreement when compared to the experimental creep test results. Using this calibrated model, numerical studies to investigate a wide variety of reinforcement configurations and beam geometry are possible without having to undertake time-consuming and expensive experimental studies. To illustrate this, a parametric study was performed to examine the effect of different types of FRP reinforcement on the creep behaviour of reinforced Irish Sitka spruce beams over a 10 year period. The results of this parametric study showed that this
modelling tool can be used to predict the long-term relative creep behaviour of such beams, and it can easily be adapted to account for different timber and reinforcement types and geometry once the relevant material characteristics are available. Application of the model to a wide variety of reinforced beam configurations will enable the development of reliable modification factors for implementation in the design of FRP reinforced beams.
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**NOMENCLATURE**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>(mm) Distance between the load head and the nearest support</td>
</tr>
<tr>
<td>$A$</td>
<td>(mm$^2$) Mean cross-sectional area of the rod</td>
</tr>
<tr>
<td>$b$</td>
<td>(mm) Width</td>
</tr>
<tr>
<td>$C_e$</td>
<td>(-) Elastic compliance matrix</td>
</tr>
<tr>
<td>$C_{ms}$</td>
<td>(-) Mechano-sorptive compliance matrix</td>
</tr>
<tr>
<td>$C_{ms, irr}$</td>
<td>(-) Irrecoverable mechano-sorptive compliance matrix</td>
</tr>
<tr>
<td>$C_R$</td>
<td>(-) Relative creep</td>
</tr>
<tr>
<td>$C_T$</td>
<td>(-) Tangent operator</td>
</tr>
<tr>
<td>$C_{ve}$</td>
<td>(-) Viscoelastic compliance matrix</td>
</tr>
<tr>
<td>$d$</td>
<td>(mm) Thickness</td>
</tr>
<tr>
<td>$D_t$</td>
<td>(m$^2$/s) Diffusion coefficient</td>
</tr>
<tr>
<td>$E$</td>
<td>(N/mm$^2$) Elastic modulus</td>
</tr>
<tr>
<td>$EI$</td>
<td>(Nmm$^2$) Stiffness</td>
</tr>
<tr>
<td>$f_u$</td>
<td>(N/mm$^2$) Ultimate tensile strength</td>
</tr>
<tr>
<td>$F$</td>
<td>(N) Force</td>
</tr>
<tr>
<td>$ΔF$</td>
<td>(N) Change in force</td>
</tr>
<tr>
<td>$F_u$</td>
<td>(N) Ultimate force</td>
</tr>
<tr>
<td>$G$</td>
<td>(N/mm$^2$) Shear modulus</td>
</tr>
<tr>
<td>$h$</td>
<td>(mm) Height</td>
</tr>
<tr>
<td>$I$</td>
<td>(mm$^4$) Moment of inertia</td>
</tr>
<tr>
<td>$J_{ve,i}$</td>
<td>(-) Viscoelastic compliance ratio of the $i^{th}$ Kelvin element</td>
</tr>
<tr>
<td>$k_i$</td>
<td>(W/mK) Thermal conductivity</td>
</tr>
<tr>
<td>$l$</td>
<td>(mm) Length or span</td>
</tr>
<tr>
<td>$l_i$</td>
<td>(mm) Gauge length for the local measurement</td>
</tr>
<tr>
<td>$m$</td>
<td>(kg) Mass</td>
</tr>
<tr>
<td>$m_0$</td>
<td>(g) Dry mass at 0% moisture content</td>
</tr>
<tr>
<td>$m_u$</td>
<td>(g) Mass at moisture content, u</td>
</tr>
<tr>
<td>$q_o$</td>
<td>(kg/m$^2$s) Moisture flux across the boundary</td>
</tr>
<tr>
<td>$R_{ve,i}$</td>
<td>(με) Individual viscoelastic strain increment</td>
</tr>
<tr>
<td>$S_u$</td>
<td>(m/s) Surface emission coefficient</td>
</tr>
<tr>
<td>$t$</td>
<td>(Second, Day, Week) Time</td>
</tr>
</tbody>
</table>
\( \Delta t \) (s) Change in time
\( u \) (-) Moisture content
\( \Delta u \) (-) Change in moisture content
\( u_{eq} \) (-) Equilibrium moisture content of timber
\( u_{ref} \) (-) Reference moisture content
\( u_{surf} \) (-) Moisture content on the wood surface
\( U \) (-) Moisture content not previously attained
\( V \) (m\(^3\)) Volume
\( w \) (mm) Deflection
\( w_0 \) (mm) Initial deflection at time zero
\( w(t) \) (mm) Deflection at time, t
\( \alpha_u \) (-) Swelling/shrinkage coefficient
\( \beta \) (-) Material constant
\( \gamma \) (\( \mu e \)) Shear strain
\( \delta \) (mm) Predefined deflection
\( \delta_s \) (mm) Shear deflection
\( \varepsilon \) (\( \mu e \)) Strain
\( \Delta \varepsilon \) (\( \mu e \)) Change in strain
\( \varepsilon_{\text{creep}} \) (\( \mu e \)) Creep strain
\( \varepsilon_e \) (\( \mu e \)) Elastic strain
\( \varepsilon_{\text{mech},L} \) (\( \mu e \)) Mechanical strain in the longitudinal direction
\( \varepsilon_{ms} \) (\( \mu e \)) Mechano-sorptive strain
\( \varepsilon_{ms,\text{irr}} \) (\( \mu e \)) Irrecoverable mechano-sorptive strain component
\( \varepsilon_t \) (\( \mu e \)) Total measured creep strain
\( \varepsilon_T \) (\( \mu e \)) Total measured strain
\( \varepsilon_{\text{ULT}} \) (\( \mu e \)) Ultimate strain
\( \varepsilon_u \) (\( \mu e \)) Strain at moisture content, \( u \)
\( \varepsilon_{ve} \) (\( \mu e \)) Viscoelastic strain
\( \varepsilon_s \) (\( \mu e \)) Swelling/shrinkage strain
\( \eta \) (kg/m.s) Viscosity coefficient
\( \nu \) (-) Poisson’s ratio
\( \rho \) (kg/m\(^3\)) Density
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma$</td>
<td>(N/mm$^2$)</td>
<td>Stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>(N/mm$^2$)</td>
<td>Shear stress</td>
</tr>
<tr>
<td>$\tau_i$</td>
<td>(hr)</td>
<td>Retardation or characteristic time of the $i^{th}$ Kelvin element</td>
</tr>
<tr>
<td>$\phi_{surf}$</td>
<td>(kg/m$^3$)</td>
<td>Moisture concentration of the wood surface</td>
</tr>
<tr>
<td>$\phi_{eq}$</td>
<td>(kg/m$^3$)</td>
<td>Equilibrium moisture concentration of timber</td>
</tr>
</tbody>
</table>
1 INTRODUCTION

Structural timber products have been shown to have benefitted with regard to strength and ultimate load capacity when reinforced in strategic locations with materials of a superior stiffness (Harte & Dietsch 2015). Materials such as steel or fibre reinforced polymers (FRP) have been used in various formats to reinforce and improve the structural performance of timber elements and the applications of this technology have been seen in new and existing structures. In new structures, FRP reinforcement has been used to increase the flexural stiffness and moment capacity of timber beams, manufactured from lower grade timber and in some instances, FRP reinforcement has been used to reduce the height of structural members. In existing structures, this technology has also been successfully utilised when retrofitting or repairing these structures. In Europe, approximately 50% of all construction is in the area of maintenance, repair or redevelopment of existing structures. This is not surprising given that over 80% of structures in Europe are greater than 50 years old (Harte & Dietsch 2015). When retrofitting these structures, the change in use of the building or, indeed, the change in building regulations often requires a higher load capacity than that of the existing members. The use of reinforcement has been successfully implemented in a timely and cost-effective manner to achieve the additional capacity requirements and similar benefits have been achieved in the repair of damaged members often restoring such members to their original load bearing capacity.

More widespread use of reinforced timber elements has been hampered by the current lack of a harmonised standard governing their design. Currently Eurocode 5 (CEN 2005), which is a harmonised European standard for the design of timber structures, is under revision and the revised version will have design rules for bonded-in steel rod reinforcement. However, FRP reinforcement will not be included. The reasons for this are partly due to a lack of knowledge, particularly related to the long-term performance of such reinforcement systems. Timber as a structural material is susceptible to large creep deformations which must be accounted for in design. Eurocode 5 (CEN 2005) provides \( k_{\text{def}} \) factors which allow design engineers to account for creep behaviour of timber members. These \( k_{\text{def}} \) factors are dependent on the Service Class conditions of the structure, with Service Class 1 representing a constant climate with no significant change in relative humidity and temperature throughout a year and Service Class 3 representing
a variable climate leading to high moisture contents in the element. For reinforced members, the creep behaviour is influenced by the presence of the reinforcement and therefore the $k_{def}$ factors will require modification.

This study has been undertaken to investigate the long-term performance or creep behaviour of FRP reinforced timber elements so that the influence of the reinforcement can be quantified. An extensive literature review has highlighted key creep components and important issues relating to the experimental creep testing of both unreinforced and reinforced engineered wood products in both constant and variable climates. Under constant climate conditions, timber behaves as a viscoelastic material. This is termed viscoelastic creep. There are additional responses that must be considered under variable climatic conditions. Due to the hygroscopic nature of timber, the long-term flexural behaviour is heavily influenced by the surrounding environment and swelling and shrinkage strains develop with changing moisture content. Additionally, when timber is stressed under load, the rate of creep deflection is significantly accelerated by moisture content changes. This in known as the mechano-sorptive creep effect (Armstrong & Kingston 1960). This mechano-sorptive effect has the ability to dramatically increase the total deformation of a timber element and potentially cause failure under severe loading and variable climate conditions. The effect of reinforcement on the viscoelastic creep, mechano-sorptive creep and swelling/shrinkage behaviour of timber beams is not currently fully understood. This area of research is receiving increased attention in recent years with the use of various FRPs in different formats. However, many of these studies have quite small sample sizes and due to the lack of a common test standard, creep testing procedures vary considerably in the literature and comparison between studies has been difficult. In order to examine the effect of reinforcement on the long-term behaviour timber beams, it is proposed to simultaneously creep test unreinforced and reinforced beams in both constant and variable climates. Careful consideration is given to the experimental design focusing on the loading of both the unreinforced and reinforced beams to similar maximum compressive stress levels in order to allow for comparisons to be made between the creep performance of unreinforced and reinforced beams. It is envisaged that the experimental test set-up and procedures implemented in this study will be adopted by future researchers and standards to allow for more comparisons to be made between different studies.
In addition to the experimental test programme, a numerical model to adequately simulate the behaviour of FRP reinforced beams will be developed. The experimental testing of such beams is expensive and time consuming and a validated numerical model will allow various geometries and reinforcing schemes to be examined in a relatively short period of time. A model implementing species-specific material properties where possible will aid the development of design rules for FRP reinforced beams allowing more efficient geometries and reinforcement systems to be determined.

1.1 Research Objectives

The primary objective of the current research is to investigate the creep behaviour of reinforced glued laminated beams in both constant and variable climates. The following sub-objectives have been defined in order to achieve the primary objective.

- Characterise the elastic behaviour of unreinforced and reinforced glued laminated beams.
- Separately characterise the viscoelastic, mechano-sorptive and swelling/shrinkage behaviour of unreinforced and reinforced glued laminated beams so that the behaviour can be modelled.
- Compare the performance of the unreinforced and reinforced beams in order to identify the influence of the reinforcement on the creep response.
- Characterise the hygro-mechanical properties of fast-grown Sitka spruce timber
  - Moisture content hysteresis curve specific to Sitka spruce
  - Swelling/shrinkage coefficients specific to Sitka spruce
  - Diffusion behaviour of Sitka spruce
- Characterise the material properties of the FRP reinforcement.
  - Tensile tests on FRP reinforcement
  - Swelling/shrinkage of FRP reinforcement
  - Creep behaviour of FRP reinforcement
- Develop a coupled moisture diffusion-creep model for unreinforced and reinforced timber beams to predict the flow of moisture in timber elements coupled with the associated viscoelastic and mechano-sorptive creep behaviour in loaded elements.
  - Implement species-specific material properties where possible
- Implement FRP reinforcement properties where possible
- Compare numerical results to experimental test results

### 1.2 Research Methodology

The following section details the research methodology to allow for the separate characterisation of the elastic, viscoelastic and hygro-mechanical response of unreinforced and reinforced beams so that models of this behaviour can be developed and implemented in a numerical tool as outlined in Section 1.1. The research methodology is illustrated in the project flowchart presented in Figure 1.1.
The methodology is as follows:

- Firstly, glued laminated timber beams were manufactured from Irish-grown Sitka spruce boards. The lay-up of each beam was designed to ensure similar properties were achieved in each beam, by minimising the effects of natural variation in the timber properties.

- An extensive short-term programme was used to examine the bending stiffness of each of the manufactured beams in its unreinforced state. These tests were performed using four-point bending tests in accordance with EN 408 (CEN 2014). The four-point bending tests performed on all unreinforced beams then allowed for matched groups of statistically equal bending stiffness to be created.

- Half of the unreinforced glued laminated beams were reinforced with basalt fibre reinforced polymer (BFRP) rod reinforcement. Four-point bending tests were performed on all reinforced beams to examine the percentage increase in short-term stiffness.

- To examine the long-term performance of unreinforced and reinforced beams in constant and variable climates, two creep test frames were designed, manufactured and tested to load each beam in the test programme. Under constant climate conditions, the deformation and strains of the beams have been monitored with time. In a variable climate, beams have been similarly monitored over time and, in addition, the evolving moisture content of the beams has been recorded.

- In parallel to the long-term creep tests, the moisture-induced swelling/shrinkage deformation of non-loaded glued laminated beams has been measured. The swelling/shrinkage measurements on non-loaded beams have been performed simultaneously to that of loaded creep tests and, as a result, have been subjected to the same change in moisture content due to the variable climate.

- Statistical tests were performed on the matched groups to compare the performance of the unreinforced and reinforced beams in the constant and variable climates. This
enabled a decomposition of the total strain component into its component parts, namely the elastic, viscoelastic, mechano-sorptive and swelling/shrinkage components. A statistical analysis was used to examine and compare the strain components in unreinforced and reinforced beams.

- Material characterisation tests were performed to evaluate the moisture content hysteresis, diffusion and swelling/shrinkage behaviour of Irish-grown Sitka spruce when subjected to variable relative humidity conditions. These relationships were utilised in numerical models.

- The mechanical properties of the BFRP rod reinforcement used in this study were examined. Tensile tests were carried out to evaluate the elastic modulus and ultimate tensile strength. The swelling/shrinkage behaviour of this material was examined by placing specimens in a high relative humidity condition. In addition, creep tests were performed on BFRP rods. This material characterisation was utilised in numerical models.

- A moisture diffusion model and coupled moisture-displacement model has been developed using the finite element software package, Abaqus, to predict the flow of moisture within timber elements and to accurately simulate the resulting moisture-induced strains under a variable climate condition, respectively. These models were calibrated using the experimental results on structural size beams.

- A coupled hygro-mechanical creep model has been developed to adequately predict the creep behaviour of unreinforced and reinforced timber elements in constant and variable climates. The development of a model capable of predicting long-term deflections and strains will contribute to the further development of FRP reinforced timber allowing alternative geometries and FRP materials and formats to be modelled and simulated in an inexpensive and timely manner. To illustrate this, a parameter study was undertaken to investigate the influence of FRP type on the long-term response.
1.3 **Structure of Thesis**

This section presents the structure of the thesis. Chapter 2 introduces previous research focused on the use of fibre reinforce polymers as a reinforcing material in timber elements as well as the short and long-term hygro-mechanical behaviour of such elements. Chapter 3 details the manufacturing procedures involved in unreinforced and reinforced beams and focuses on the short-term flexural testing of beams in their unreinforced and reinforced states and the formation of matched groups for long-term comparative studies. Chapter 4 details the design, construction and commissioning of the creep test frame and the results of the long-term creep tests. Chapter 5 details the material characterisation of Sitka spruce with a particular focus on its behaviour in a variable climate and also the mechanical characterisation of BFRP specimens. Chapter 6 presents numerical models formulated to predict the behaviour of non-loaded and loaded timber elements in both constant and variable climates. Further information is given in the appendices.

**Chapter 1** – Introduction  
**Chapter 2** – Literature Review  
**Chapter 3** – Short-term Experimental Testing  
**Chapter 4** – Long-term Experimental Testing  
**Chapter 5** – Material Characterisation of Sitka Spruce and BFRP  
**Chapter 6** – Hygro-mechanical Creep Modelling  
**Chapter 7** – Conclusions  
**Appendix A** – Creep Test Rig Design  
**Appendix B** – Relative Creep Test Results  
**Appendix C** – Moisture Probe Calibration  
**Appendix D** – Moisture Probe Results
2 LITERATURE REVIEW

2.1 INTRODUCTION

Unlike more commonly used construction materials, timber is an organic material and as a result, it is a complex material. The physical and mechanical properties of timber can vary considerably in structural members as a result of the growth conditions, duration of growth and other environmental factors and silvicultural practices which affect the wood cell density and strength (Treacy et al. 2000). In Ireland, Sitka spruce is the most widely planted species and the primary species under investigation in this study. This softwood species is characterised as a fast-grown, low density timber which, when used in flexure, tends to fail in the tension zone due to the presence of knots (Gilfillan et al. 2003). This species has an average rotation length of only 35-40 years (Raftery & Harte 2014) leading to large proportions of juvenile wood content. This fast-grown timber demonstrates limited capacity to endure substantial loads, however, when combined to create a composite element such as a glued laminated beam, the capacity of this softwood timber may be greatly increased.

Engineered wood products such as glued laminated beams and laminated veneer lumber (LVL) are often manufactured from lower grade or possible by-products from larger solid products. The combination of the lower grade material with an adhesive component creates a superior engineered wood product. The load carrying capacity of these engineered wood products can be greatly increased and the variability associated with solid timber is often reduced as defects within wood are generally limited to a single lamination within the engineered wood product. Furthermore, the performance of glued laminated timber beams may be increased with the addition of fibre reinforced polymer (FRP) composite reinforcement (Raftery & Harte 2011).

This chapter presents current research to date on the use of FRP materials to reinforce timber elements with a particular interest in the long-term performance of reinforced timber beams. After a brief introduction to the structure of timber and the origins of glued laminated timber beam construction, this chapter presents studies examining the short-term behaviour of glued laminated beams reinforced with FRP of different types and formats. As there is no defined test standard for long-term creep tests
on structural timber elements, an extensive review of previous research projects examining the long-term performance of unreinforced and reinforced beams is presented with a particular interest in the test set-up, load level and climatic conditions during the long-term tests.

### 2.2 The Structure of Wood

#### 2.2.1 Introduction

Wood is a naturally grown, organic composite comprising cellulose fibres embedded in a matrix of lignin. Due to the elongated shape of the cellulose fibres (wood cells) and the orientation of the cell walls, wood is considered to be an anisotropic material, meaning its physical and mechanical properties are direction dependent.

Wood can be separated into two broad categories known commonly as hardwoods (angiosperms, deciduous) and softwoods (gymnosperms, conifers). Softwoods grow at a much faster rate than hardwoods and as a result, the cell structure which forms is often less dense than that of hardwoods. This is not always the case as Balsa wood (Ochroma lagopus), which is considered a hardwood species, uncharacteristically has a relatively low density.

![Figure 2.1. Scanning electron microscope: a) Ash (Hardwood, Fraxinus excelsior), b) Scots pine (Softwood, Pinus sylvestris) showing the axially orientated vessels (v), Fibres (f), Tracheids (t) and the Ray parenchyma (r); ew = earlywood, lw = latewood, rc = resin canal. Bar lines = 100 μm. (Eaton & Hale 1993).](image)

The majority of hardwoods are deciduous while most softwoods are evergreen (coniferous species) but there are exceptions such as eucalyptus, which is a hardwood but
uncharacteristically retains its leaves all year round, while larch is a softwood species yet loses its leaves during the winter months (Eaton & Hale 1993).

A distinction between hardwood and softwood is found when looking at the wood at a microscopic level. Figure 2.1 shows the contrasting characteristics of a), a hardwood species (Ash, Fraxinus excelsior) and b), a softwood species (Scots pine, Pinus sylvestris). Hardwoods and softwoods are made of different types of cells and this is the reason why they have different internal structures. The most distinctive differences can be seen in the large vessels, which are characteristic of hardwood species. These vessels are joined end to end forming networks to transport water throughout the tree. In softwoods, water is transported through tracheids which are elongated cells.

Within both hardwood and softwood species, there is a region known as heartwood and a region known as sapwood. In Figure 2.2, heartwood is located in the inner core of the stem and usually has a darker colour than the outer sapwood region. Sapwood is the actively growing part of the tree. This active region of living cells provides the lighter colour while the inner darker core consists of dead cells which were previously required during the juvenile growth stage of the tree.

Figure 2.2. Hardwood tree segment (Dinwoodie 2000 (©BRE, reproduced with permission))

The sapwood is responsible for upward transportation of mineral solutions. As the trunk increases in girth, the central section of the tree begins to lose water as the vessels,
tracheids and fibres of the sapwood die. This central section becomes impregnated with oils, resins and gums which accumulate within the cells. This accumulation of deposits often causes the change in colour from lighter sapwood to darker heartwood due to the oxidation of phenolic compounds but this difference in colour is negligible in some species e.g. aspen, beech and sycamore (Eaton & Hale 1993).

The cambium, as seen in Figure 2.2, is the outer vascular layer of the tree. The rate of cell production in this layer is seasonally dependent with faster growth occurring during summer months compared to that of the winter months. The faster growth rate leads to less dense material and the slower growth rate produces a more dense material. The lighter, lower density material is termed the earlywood and the darker dense wood is termed the latewood. These seasonal layers paired together form a growth ring. Fast growing trees have larger growth rings than slower grown trees and contain a higher volume fraction of juvenile wood compared to mature wood. Juvenile wood is the wood that directly surrounds the pith and generally comprises the first 15-20 growth rings in the radius of a tree. Juvenile wood is characterised as having reduced latewood, shorter fibres, thinner cell walls, increased extractive content and higher porosity (Alam 2004). Although there are few differences in the chemical makeup of juvenile and mature wood, the mechanical properties differ immensely. Juvenile wood absorbs more water, is less dense, is neither as stiff nor as strong as mature wood, has shorter tracheids, thinner cell walls and higher microfibril angles (Alam 2004). Hence, the mechanical properties of slow-grown trees are superior to their fast-grown counterparts.

2.2.2 Composition of Wood

Wood is a naturally grown composite material which consists of longitudinal fibres reinforced in a polymer matrix and cylindrically orientated about the pith, which determine the structural properties of timber. At the molecular level, the three primary constituents of wood are cellulose, hemicellulose and lignin. Lignin is the adhesive that glues the cellulose and the hemicelluloses together (Dinwoodie 2000). The cellulose molecules are the main structural component in cell walls. The cellulose fibres are very stiff and the lignin secures them into place in the form of microfibrils which stiffens the wood structure. Lignin is a complicated polymer of phenylpropane units, which are completely amorphous. Lignin increases the rigidity of a cell wall considerably due to its
less hydrophilic properties and it also influences the swelling characteristic of wood for this reason.

Figure 2.3. Wood cell showing orientation of cellulose microfibrils in each layer (Eaton & Hale 1993)

In each wood cell, an outer wall represents the primary cell wall layer and surrounds three inner layers of cellulose microfibrils known as the S_1, S_2 and S_3 layers, as seen in Figure 2.3. The orientation of these cellulose microfibrils is different in each layer. The S2 layer is dominant and its contribution to the physical properties has been shown to be more significant than the S1 and S3 layers (Mark 1967; Bodig & Jayne 1993).

2.2.3 Properties of Wood

Wood is a cylindrically orthotropic and heterogeneous material and as such, is a material that differs greatly from other materials created through man-made processes such as steel. Timber is a naturally grown, organic material with three principal axes, namely, the longitudinal axis parallel to the direction of the grain, the radial axis perpendicular to the direction of the grain and the tangential axis perpendicular to the grain direction and tangential to the growth rings of the cross section (Figure 2.4).
Orthotropic materials are symmetric about three mutually perpendicular planes and the elastic behaviour of wood is assumed to be orthogonal with respect to these principal planes. Orthogonal materials have three independent moduli of elasticity for each plane, three shear moduli in the three shear planes and six Poisson’s ratios. Equation [2.1] (Bodig & Jayne 1993) expresses Hooke’s Law in three-dimensional matrix form. This equation defines the elastic stress-strain behaviour of timber as an orthogonal material.

\[
\begin{bmatrix}
\varepsilon_L \\
\varepsilon_R \\
\varepsilon_T \\
\gamma_{RT} \\
\gamma_{LT} \\
\gamma_{LR}
\end{bmatrix} = \begin{bmatrix}
1 & -\frac{v_{RL}}{E_L} & -\frac{v_{RT}}{E_L} & 0 & 0 & 0 \\
-\frac{v_{LR}}{E_R} & 1 & -\frac{v_{TR}}{E_R} & 0 & 0 & 0 \\
-\frac{v_{LT}}{E_L} & -\frac{v_{RT}}{E_T} & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix} \begin{bmatrix}
\sigma_L \\
\sigma_R \\
\sigma_T \\
\tau_{RT} \\
\tau_{LT} \\
\tau_{LR}
\end{bmatrix}
\]

The subscripts \( L, R \) and \( T \) refer, respectively, to the longitudinal, radial and tangential material directions. Symbols \( E \) and \( G \) represent the elastic modulus and shear modulus, respectively. The symbols \( v, \varepsilon, \sigma, \gamma \) and the \( \tau \), refer to the Poisson’s ratio, the normal strain, normal stress, shear strain and the shear stress, respectively. The mechanical and physical properties of softwood species can vary considerably between wood of different species and the growth conditions of the timber and other environmental factors which affect the wood density and strength (Treacy et al. 2000). The elastic and shear modulus are also

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Figure 2.4. Orthotropic material directions of wood (Harte 2009)
dependent on the moisture content of the timber and this orthotropic representation of Hooke’s law has been shown to successfully describe the elastic response of timber under load and variable moisture content conditions.

2.2.4 Density of Wood

Desch & Dinwoodie (1996) state that density is possibly the most important material property controlling the mechanical performance of a piece of wood. Density is strongly correlated with many other wood material properties. Density ($\rho$) is the ratio of mass to volume as seen in Equation [2.2].

$$\rho = \frac{m}{V}$$

where:

$\rho =$ density  
$m =$ mass  
$V =$ volume  

The application of this equation and the interpretation of the results must be carefully examined as the density of a piece of wood is not only subject to the volume of the wood, but also the moisture content and the presence of extractives. Moisture in wood can increase the mass of the timber but it also increases the volume. As a result, it is essential that moisture content is measured in parallel with the density measurements. Extractives are organic compounds that are absent or present in small quantities in timber and are usually neglected when measuring density. In a laboratory setting, both mass and volume can be measured at zero moisture content following drying of the sample to give the dry density of the specimen.

2.2.5 Moisture in Wood

Wood is hygroscopic, so its moisture content will fluctuate depending on the surrounding environment. Wood will absorb moisture from the surrounding environment when dry (adsorption) and surrender moisture when wet (desorption) in order to reach a moisture content in equilibrium with the environment. Therefore, for any combination of relative humidity and temperature, there is a corresponding moisture content whereby no inward or outward movement of moisture will occur. This is known as the equilibrium moisture content (EMC) for that environment. Generally, it has been shown that the species of
wood has no significant effect on the moisture content of the wood. This is primarily due to the similar moisture transportation mechanisms within all species (Wadsö 1993) and as a result, many models have been created to determine the average equilibrium moisture content of timber as a function of the relative humidity and temperature (Hailwood & Horrobin 1946; Simpson 1973; Avramidis 1989). However, these models do not consider the moisture content hysteresis of wood during adsorption and desorption. This hysteresis effect refers to the phenomenon whereby at a common relative humidity, the equilibrium moisture content of a wood specimen during the adsorption phase is less than that of the desorption phase. Hence, the equilibrium moisture content is also a function of the previous history of the wood specimen.

In laboratory conditions, the oven drying method is often used to measure the moisture content of wood. The moisture content is determined by weighing a wood sample and subsequently drying it in an oven at 103°C ± 2°C until a stable mass has been reached between successive moisture measurements two hours apart in accordance with EN 13183-1 (CEN 2002). The mass of water can be easily calculated by subtracting these two masses. Equation [2.3] below is used to calculate the moisture content of the wood.

\[
Moisture\ Content\ (u) = \frac{m_u - m_0}{m_0} \times 100 \% \tag{2.3}
\]

where:

- \( u \) = moisture content
- \( m_u \) = mass of wood sample at a moisture content, \( u \)
- \( m_0 \) = dry mass of wood sample

This method is time consuming and energy intensive due to the long drying time required for the wood specimen to reach a stable weight. In the assessment of in-service timber elements, it is often impractical to remove a specimen or part of a specimen to determine the moisture content using the oven drying method. For this reason, the moisture content is often measured using an electrical resistance moisture meter in practice. The moisture meter measures the electrical resistance between two electrodes inserted into the wood. Electrical resistance moisture meters are relatively accurate for moisture contents ranging between 6-30% (Dai & Ahmet 2001). The wood species is important when determining the relationship between this measured electrical resistance and moisture content and many manufacturers provide calibration curves for different species (Forsén & Tarvainen 2000; Taylor & West 1990).
2.2.6 Swelling/shrinkage in Wood

In wood, moisture is contained in wood cells both in the cell lumens (free water) and in cell walls (bound water). As the wood dries, the free water is removed first. The point at which no free water is stored in the cell lumen and the cell walls are saturated with bound water is known as the fibre saturation point. This point occurs at a moisture content approximately between 27-30% and wood is dimensionally stable when its moisture content is higher than its fibre saturation point. As wood dries below this point, dimensional changes occur; however, the magnitude of these changes depend on the material direction, given the orthotropic nature of timber (Harte 2009). Minor swelling/shrinkage occurs parallel to the grain when compared to the radial and tangential directions.

The rate of swelling/shrinkage depends on a number of material factors. The species of wood is a primary factor, and density of the wood material also has an effect. Previous research has shown that a low density, fast-grown timber can have significantly larger swelling/shrinkage coefficients than slow-grown timber in the longitudinal direction. Ormarsson & Cown (2004) compared the swelling/shrinkage characteristics of two species, namely, a fast-grown Radiata Pine and a slow-grown Norway spruce. They reported that the swelling/shrinkage coefficient of Norway spruce was significantly greater than the swelling/shrinkage coefficient of Radiata pine in the radial and tangential directions. However, in the longitudinal direction, the swelling/shrinkage coefficient reported for Norway spruce was much less than that reported for the Radiata Pine. It is evident that species is an important parameter when determining swelling/shrinkage coefficients, but even within species there can also be an effect due to growth conditions. Bengtsson (2001c) investigated the variations between the swelling/shrinkage coefficients from a well-documented fast-grown and slow-grown stand of Norway spruce. Similar measurements were taken from both the fast and slow-grown timber. It was found that the swelling/shrinkage coefficients for the radial and tangential directions were significantly larger for the slow-grown timber; however, the opposite was seen in the longitudinal direction with a significantly larger coefficient observed for the fast-grown timber. This is of particular interest given the fast growth rate of Irish Sitka spruce used in this study. The rate of swelling/shrinkage is also dependent on the magnitude of the moisture content change, the diffusion coefficients of the timber and the size and shape of the element affected by the wetting or drying.
2.3 Glued Laminated Timber

2.3.1 Introduction

In the early 1900’s, glued laminated structural timber was integrated into the construction of archways, gymnasiums, factories and barns. It is one of the oldest engineered wood products and was officially invented in 1906 by Otto Hetzer. Hetzer’s patent described a curved timber element consisting of two or more timber elements combined with an adhesive component under pressure (Rug 2006). Glued laminated timber products created new possibilities for design as restrictions which apply to solid timber pieces can be overcome with glued laminated timber. The size of glued laminated beams are technically only restricted by the capabilities of the manufacturing plant or the mode of transport. Larger section sizes and spans can be achieved and architectural curved beams or arches can be easily designed and manufactured by bending the timber with the use of a jig. These longer spans allow for open floor plans without any need for columns leading to its popular use in gymnasiums, auditoriums and public buildings. Glued laminated beams can be tailor made to fit both load requirements and the design of the structure.

When a glued laminated timber beam is formed from a combination of timber laminations, the bending strength of the combined glued laminated beam is higher than the tensile strength of the individual laminations. This phenomenon is known as the laminating effect, which is an increase in the bending strength of timber laminations when they are bonded together in a glued laminated beam. Defects such as knots, seasoning checks, reaction wood and sloping grain may be crucial in the performance of a solid timber piece but are generally limited to one lamination in a glued laminated beam, greatly reducing the effect on the structural integrity of the beam. This lamination effect results in a material that is more homogenous than solid wood. This is also called the dispersion effect (Larsen 1982). A lamination containing low stiffness defects such as knots will also experience a reinforcing effect as stronger and stiffer adjacent laminations absorb a larger part of the tensile stresses. Nowadays, glued laminated timber is frequently used in the construction industry due to its high strength-to-weight ratio, aesthetic features and a global movement towards the use of more sustainable construction materials. Further advances have seen glued laminated beams in combination with fibre reinforced polymer composite reinforcement with glued in rods or plates to improve the performance of glued laminated elements (Plevris & Triantafillou
1995; Gustafsson & Serrano 2000; Raftery & Harte 2011). These reinforcement methods can further increase the strength, stiffness and ultimate moment capacity of timber beams in addition to allowing greater spans to be achieved.

2.3.2 Adhesives in Glued Laminated Beams

In order to optimise the performance of timber composites, it is essential that failure occurs within the timber or the reinforcement as a solid adhesive bond is paramount to ensure composite action and avoid premature failure. As discussed in Section 2.3.1, glued laminated beams were first seen in the early 1900s. However, the adhesives used at the time were not durable under fluctuating relative humidity, limiting their application to internal environments (Moody & Hernandez 1997).

This changed in 1942 with the development of Phenol-resorcinol-formaldehyde (PRF) adhesives. These new adhesives demonstrated an inherently waterproof property and the effects of variable relative humidity was no longer a prominent issue. Glued laminated timber was no longer limited to internal environments but could be used externally or internally in buildings with high moisture fluctuations such as ice rinks or swimming pools.

The bonding of the wood-wood interface is of crucial importance in glued laminated beam performance and the bond quality has been comprehensively studied. However, this primarily relates to the bonding of mature wood and limited literature is available on the subject of bonding wood with large proportions of juvenile wood present (Raftery 2010). In fast grown species with short rotation lengths, juvenile wood may be present in large quantities. This wood is characterised as having reduced latwood, shorter fibres, thinner cell walls, increased extractive content and higher porosity (Alam 2004). Raftery & Harte (2005) carried out a study to determine whether conventional wood laminating adhesives could form good quality bonds with softwood (Sitka spruce) containing large proportions of juvenile wood. Their tests also examined the performance of various adhesives under severe accelerated ageing tests. Results indicate that PRF adhesives perform extremely well for non-moisture-cycled specimens and also for moisture-cycled specimens achieving greater than 80% wood failure.

Preparation of the bond surface is important to ensure a quality bond is achieved. Jokerst & Stewart (1976) compared knife planing to abrasive planing. It was found that knife planing ensures a uniformly smooth bonding surface. They also reported that
although similar shear strengths were obtained when bonding with abrasive planed surfaces and with knife-planed surfaces, the abrasive planed surfaces experienced greater glue-line separation with accelerated ageing. When examined under a microscope, the abrasive planing showed crushing and tearing of the wood while the knife planing showed a clean cut surface. The application of adhesive to this clean cut surface allows the adhesive to penetrate the porous cellular wood structure (Raftery & Harte 2005). The short time period between planing and applying the adhesive prevents unwanted oxidation of the surface while the clean cut surface is exposed to the environment. This infiltration into the exposed surface improves mechanical interlocking (Frihart 2009).

## 2.4 FRP Reinforced Timber Elements

### 2.4.1 Introduction

FRP materials have been shown to significantly improve the ultimate capacity of timber elements when positioned in strategic positions. The use of modest proportions of FRP reinforcement in the tension zone of timber beams delays tension failure and utilises the additional capacity of the timber in the compression zone. This results in much more consistent, ductile behaviour as well as significant increase in flexural stiffness (Gilfillan et al. 2001; Gentile et al. 2002; Raftery & Harte 2011; McConnell et al. 2015). Many FRP materials can be used to increase the flexural properties of timber elements. The most common types examined in the literature are carbon fibre reinforced polymer (CFRP), aramid fibre reinforced polymer (AFRP) and glass fibre reinforced polymer (GFRP). Increasing interest in sustainable and environmentally friendly solutions has promoted the use of natural fibres such as basalt fibre. As a result, novel basalt fibre reinforced polymer (BFRP) reinforcement is becoming increasingly common. In addition to being environmentally friendly, BFRP demonstrates superior flexural properties when compared to GFRP (Lopresto et al. 2011) and is extremely cost effective when compared to CFRP and AFRP.

The addition of FRP reinforcement to timber beams has been utilised to strengthen or even repair existing timber beams. Rehabilitation of historical timber structures has benefited from advances in this technology allowing existing beams to be repaired and strengthened with the use of reinforcement in combination with an adhesive component (CNR 2004). These different types of FRP reinforcement have superior mechanical
properties compared to timber and have been shown both theoretically and experimentally to enhance the short-term performance of timber beams when positioned in strategic locations (Schober et al. 2015). Even with modest percentage reinforcement ratios of FRP materials, the reinforced glued laminated beams have been shown to demonstrate superior strength, stiffness and ultimate moment capacity when compared to glued laminated beams in their unreinforced state. This section describes the most common fibre reinforced polymers and their use in practical applications. Relevant studies which have successfully used FRP reinforcement to strengthen and stiffen timber elements are discussed along with the bonding procedures involved.

### 2.4.2 Fibre Reinforced Polymer

Fibre reinforced polymer is a composite composed of a polymer resin matrix and reinforcement fibres. The most common reinforcement fibres used are glass, carbon and aramid fibre reinforcement. FRP materials are commonly used in the aerospace, automotive and marine industries due to their low density, corrosion resistance and high strength to weight ratio (Yeboah et al. 2013; Zaman et al. 2013). Their use in the construction sector has grown over many years with the advancements in reinforced polymer composites and the reduction in cost.

Reinforcement fibres provide strength and stiffness to the composite while the polymer resin transfers the load between the fibres and provides protection to the reinforcing fibres. This combination of reinforcing fibres and polymer resins mean FRP materials are anisotropic and the best mechanical properties are in the direction of the fibre. The polymer resins are classified into two distinct groups. These are thermosetting and thermoplastics. Epoxies, vinyl esters and polyesters are examples of thermosetting resins. Thermosetting resins are formed under heat and allowed to cool and solidify. Once formed they will not melt or soften when reheated. Some examples of thermoplastic resins are polyethylene, polyurethane and polypropylene which do not change chemically during production like thermosetting resins and can easily be reformed on heating.

Fibres can be formed from a wide range of materials. As mentioned previously, the main fibres types used in the construction industry in load bearing applications are glass and carbon fibres (Hollaway 2010). Basalt fibre is less documented but has potential to rival these more commonly used fibres (Lopresto et al. 2011). Increasing interest in environmental issues has also encouraged the use of more natural fibres such as basalt.
fibre in polymer reinforcement. Basalt is one of the most common rock types in the earth’s crust. To manufacture BFRP, the basalt rock is first pre-treated and melted at approximately 1400°C. The molten rock then flows into a bushing containing hundreds of small orifices which create basalt fibres as the molten rock passes through these orifices (Yeboah et al. 2013).

BFRP has been evaluated and compared to other FRPs used in reinforced concrete applications as well as reinforced timber applications (Chen et al. 2008; Zaman et al. 2013). Lopresto et al. (2011) compared the mechanical properties of both basalt and glass fibre reinforced polymers. The results show higher tensile modulus, bending strength and compressive strength for basalt fibre. The basalt fibre showed a 35-42% higher elastic modulus when compared to glass fibre, depending on the fibre volume fraction. Other studies report lower stiffness enhancements in the order of 16% over GFRP (Parnas et al. 2007).

FRP materials face a variety of environmental conditions during their in-service life. These environmental conditions result from natural or artificial factors. They include varying temperature and relative humidity conditions, ultraviolet rays from the sun, and diverse chemical reactants (Ray 2006; Zaman et al. 2013). These conditions over time can have a serious effect on the mechanical properties of the material. Moisture effects result from natural relative humidity or rainfall and the degree of moisture penetration depends on the sensitivity of the composite. CFRP composites are usually unaffected by moisture adsorption, however, in contrast, GFRP composites are more susceptible to moisture penetration which can adversely affect the mechanical performance. The load applied has also been shown to have an effect on moisture uptake (Maron & Broutman 1981). Maron & Broutman (1981) examined the moisture movement within both CFRP and GFRP under stressed and unstressed conditions at constant environmental conditions and observed the diffusion rate and the equilibrium moisture content increased with load. Cyclic humidity conditions also have a negative effect on many FRP materials. Cyclic moisture sorption and desorption can induce residual stresses within the material leading to cracks, degradation and eventual failure. No information has been reported on the response of BFRP to moisture variations. FRP materials such as CFRP or AFRP, although affected by environmental conditions, can still outperform steel. When compared to steel their densities are lower and their strength and stiffnesses are higher. Their strengths can
be 10-15 times higher than steel and they also remain high at elevated temperatures (Gilfillan et al. 2003).

### 2.4.3 FRP Reinforcement in Timber Elements

Many forms of FRP reinforcement have been used in conjunction with timber in an effort to improve and strengthen timber properties. Timber is advantageous in its strength-weight ratio. However, its strength may be reduced due to defects present within the timber. This generally leads to brittle failure in bending and tension. The use of FRP reinforcement has been used in many cases to improve the short-term performance of timber elements (Plevris & Triantafillou 1992; Gentile et al. 2002; Alam 2004; Hansson & Kristoffer 2007; Raftery et al. 2012; Yeboah et al. 2013; McConnell et al. 2014; Schober et al. 2015; Franke et al. 2015). In a study by van de Kuilen (1991), a considerable increase in stiffness and ultimate bending moment was achieved by using GFRP plates. The use of CFRP as a reinforcement material was examined by Plevris & Triantafillou (1992) and Hansson & Kristoffer (2007). In both studies, timber beams were reinforced in the tension zone and significant improvements in bending strength and stiffness were observed with Plevris & Triantafillou (1992) reporting a stiffness improvement in the order of 60% with a percentage area reinforcement ratio of 1% and Hansson & Kristoffer (2007) reporting a stiffness increase of as much as 85% with a percentage area reinforcement ratio of 2% using CFRP reinforcement plates.

Kliger et al. (2008) performed an experimental study on nine glued laminated beams strengthened with bonded-in steel plates and CFRP plates in different configurations. Tension and compression zones were reinforced in different proportions. The beams were tested in four-point bending until failure. The results showed that for all types of reinforcement configurations, there was an increase in ultimate moment capacity and ultimate load. The observed increase in ultimate moment capacity ranged from 57% to 96%. Using an analytical non-linear model in the same study (Kliger et al. 2008), it was shown that the maximum increase in bending stiffness was achieved with 50% of the total reinforcement in tension zone and 50% in the compression zone. However, in order to optimise the ultimate moment capacity, it was recommended to reduce the proportion of compression reinforcement and attribute this to the tension zone. Similar results were observed by van de Kuilen (1991) when comparing beams reinforced with GFRP in the
tension zone to beams reinforced in both the tension and compression zone with the same total area of GFRP plate.

Gilfillan et al. (2000, 2001a, 2001b) published a series of studies on FRP reinforced Irish-grown Sitka spruce glued laminated beams and demonstrated that this low strength, lower-grade wood can be significantly enhanced by the addition of FRP plate reinforcement. Three reinforcement materials were chosen in the experimental programme, namely GFRP, CFRP and steel reinforcement and compared to unreinforced beams (Gilfillan et al. 2000). All beams were tested elastically prior to the addition of any reinforcing material. Post reinforcement, each beam was loaded to failure. All unreinforced beams failed in the tension zone. The reinforced beams with percentage reinforcement ratios ranging from 0.4% to 2.9% exhibited local failure in the compression zone prior to tensile failure. The addition of FRP reinforcement induced a ductile failure without any rupture or de-bonding of the reinforcement. The ultimate moment capacity of each reinforced beam reached considerably higher values when compared to unreinforced control specimens. With a 0.4% CFRP reinforcement on a 6 m span beam, the ultimate moment capacity was increased by 48%. CFRP greatly outperformed GFRP reinforcement mechanically, however, the economic differences are noted and deemed significant.

In another series of studies examining the short-term performance of reinforced timber beams manufactured from Irish-grown Sitka spruce, Raftery & Harte (2011) and Raftery et al. (2012) examined the use of GFRP plate reinforcement and near surface mounted GFRP rod reinforcement, respectively. In Raftery & Harte (2011), a mean percentage increase in bending stiffness of 13.15% and an increase of 38.00% in the ultimate moment capacity were observed when comparing unreinforced beams to beams reinforced with a reinforcement percentage ratio of 1.26%. In Raftery et al. (2012) a reasonable percentage area reinforcement ratio of 1.40% was adhered in the tension zone within routed grooves using a structural epoxy adhesive. They observed a mean stiffness enhancement of 13.90% and an average improvement of 68.00% in the ultimate moment capacity in comparison to matched unreinforced glued laminated beams. It was noted that in both the plate and rod format, significant improvements were observed in the bending stiffness and moment carrying capacity of this low-grade timber.

There is a limited number of studies examining the use of BFRP as a reinforcing material within timber elements. In Yeboah et al. (2013), BFRP rod reinforcement was
examined in timber joint applications. The results found that the pull-out capacity of such connections increased linearly with the bonded length up to a length of 15 times the bonded length and significantly, the common shear fracture failure mode between the timber-adhesive interfaces demonstrated the excellent bond between the BFRP reinforcement and epoxy adhesive. In one study examining the use of BFRP rod reinforcement in flexural applications by Kelly (2012), short-term four-point bending tests were performed on BFRP rod reinforced glued laminated timber beams. The test results demonstrated significant improvements in bending stiffness and ultimate moment capacity for a low percentage reinforcement ratio of 1.4%. Significantly, the reinforcement delayed tension failure in the glued laminated beams and improved utilisation of the compression properties of timber. This desirable ductile behaviour was observed in all reinforced beams with visible compression wrinkling in the top lamination and it was noted that this novel BFRP material could rival more commonly used FRP materials used to reinforce timber elements. The use of BFRP rod reinforcement has also been used in post-stressed timber applications (McConnell et al. 2015; Lloyd 2016). Short-term experimental tests of such beams demonstrated significant improvements in flexural strength and stiffness with reinforced beams demonstrating a ductile failure mode similar to that observed by Kelly (2012).

2.4.4 Structural Epoxy Adhesives

Many adhesives have been used to bond various FRP materials to timber elements (Mays & Hutchinson 1992; Broughton & Hutchinson 2001). However, structural epoxy adhesives are generally considered to be the most suitable when bonding FRP materials to timber surfaces (Rowlands et al. 1986; Davis 1997) as they demonstrate good gap filling properties and require low clamping pressure.

Rowlands et al. (1986) examined the use of epoxy adhesives and the bond created between different reinforcing materials (CFRP, GFRP and AFRP) and wood. They found an excellent bond to glass, carbon and aramid fibre reinforced materials when tested in dry conditions. When the same epoxy was subjected to severe moisture cycling, they found a reduction of approximately 50% of their dry bond strength. Regardless of this, epoxy adhesives still remain the primary choice in FRP-wood bonded connections because of their good gap filling properties, limited shrinkage during curing and their ability to achieve full cure at ambient temperatures (Raftery et al. 2009b).
The performance of FRP-wood bonds have been studied in Raftery et al. (2006; 2009a; 2009b). Numerous adhesives were tested in shear with the aim of finding the most suitable adhesive for the FRP-wood interface. In dry conditions, the epoxy adhesives performed best (Raftery et al. 2006). In a later paper by Raftery et al. (2009b), the effect of moisture cycled FRP-wood bonds was studied. These bonds were assessed using a soaking/drying cycling procedure and a block shear test. The results indicated that the quality of the bond is important but also the effect of moisture cycling. When bonding to GFRP, the structural epoxy adhesive marginally failed the 80% wood failure criterion after severe moisture cycling was performed. This result was associated with a high standard deviation given the variability inherent in timber. However, with the use of an adhesion promoter, all specimens passed the 80% wood failure criteria. The results show that strong bonds may be achieved with certain epoxy adhesives under severe moisture cycling conditions.

### 2.5 Creep Effects in Timber Elements

#### 2.5.1 Introduction

Minimisation of degradation of timber while under serviceability loads is of crucial importance to ensure satisfactory long-term performance of timber elements. Timber is susceptible to many forms of degradation namely, biological, chemical, photochemical, thermal and mechanical degradation. Mechanical degradation is the focus of this study and is commonly seen in timber elements when stressed under load for long periods of time. The most common effects of mechanical degradation are known as the duration of load (DOL) effect and the creep effect. The duration of load effect materialises as a loss of strength with time under a sustained load. Similarly with creep, there is a reduction in stiffness with time, which manifests as an increase in deflection with time under constant load. For timber structures, the creep deflection can be large relative to the instantaneous deflection. This effect is greatly magnified in the presence of cyclic environmental conditions (Armstrong & Kingston 1960). As serviceability limit state deflections often control the design of timber structures, accurate prediction of the creep deflection is essential.

The use of various FRP reinforcement materials has been shown to significantly improve the short-term performance of timber elements, however, the influence on the
long-term behaviour has received less attention and is of crucial importance to structural engineers when designing reinforced timber elements.

This section details the various creep components commonly affecting in-service timber elements and structures with a focus on relevant studies to date examining these creep components in unreinforced and reinforced timber elements.

2.5.2 Quantifying Creep Components

Creep in timber elements has been observed under all loading modes such as tension, compression, bending and shear (Schniewind 1968). The total creep can be divided into three primary components. These three creep components are time dependent or viscoelastic creep, mechano-sorptive creep due to moisture changes and pseudo-creep that is attributed to swelling and shrinkage of the timber (Hunt 1999). It is difficult to separate and monitor each component accurately as they often occur simultaneously. Under constant climate conditions, the elastic and viscoelastic components contribute to the total deformation and the mechano-sorptive creep and creep associated with swelling and shrinkage is zero. However, under a variable climate condition, moisture content changes take place within the timber. The interaction between stress and moisture content change results in additional mechano-sorptive creep. The swelling/shrinkage of the timber is also dependent on the change in moisture content and all three components contribute simultaneously to the total deformation of a timber element in a variable climate condition.

Creep is quantified by a number of time dependent parameters. Creep compliance and relative creep are two of the most common. Creep compliance ($C_C$) is defined in Equation [2.4] as the ratio of increasing strain at a time, $t$, to the applied constant stress.

$$C_C(t) = \frac{\Delta \varepsilon(t)}{\sigma_0} \quad [2.4]$$

where:

$C_C = \text{creep compliance}$

$\Delta \varepsilon = \text{change in strain with time}$

$\sigma_0 = \text{applied constant stress}$

Relative creep ($C_R$) is defined in Equation [2.5] as the increase in deflection at time, $t$, expressed in terms of the initial elastic deflection.
where:

\[ C_R (t) = \frac{w(t)}{w_0} \]  \[\text{[2.5]}\]

where:

\[ C_R \] = relative creep  \\
\[ w(t) \] = deflection at time, t  \\
\[ w_0 \] = initial deflection at time zero

For the serviceability limit state, Eurocode 5 (CEN 2005) provides deformation modification factors \((k_{\text{def}})\) for different service classes in order to account for creep effects. The service classes correspond to predefined environmental conditions and the \(k_{\text{def}}\) factor is used to increase the initial elastic deflection of the designed element depending on the service class in which the element will be situated. Equation [2.6] describes the relationship between \(k_{\text{def}}\) and relative creep.

\[ k_{\text{def}} (t-t_0) = \frac{w(t)-w(t_0)}{w(t_0)} = \text{Relative Creep} - 1 \]  \[\text{[2.6]}\]

### 2.5.2.1 Viscoelastic Creep Component

Timber can be described as a viscoelastic material as its behaviour under load is a combination of an elastic and viscous material. As a viscoelastic material, its behaviour is time dependent. Viscoelastic creep in timber is described as additional deflection with time at constant moisture content and constant environmental conditions under sustained loading. At high stress levels, viscoelastic creep may lead to failure but under normal loading conditions the increase in deflection will be moderate, however, viscoelastic creep has been observed at stress levels as low as 10% of the ultimate failure load (Senft & Suddarth 1971; Kaboorani et al. 2013). Although viscoelastic creep occurs at constant environmental conditions with no changes in moisture content, it is important to note that the magnitude of viscoelastic creep depends on the moisture content of the timber (Armstrong & Kingston 1960; Armstrong & Kingston 1962; Armstrong 1972). In a study by Hering & Niemz (2012), the viscoelastic behaviour of European beech timber subjected to four-point bending was investigated and the longitudinal creep compliance at three different moisture contents (8.14%, 15.48% and 23.2%) was examined. In Figure 2.5, the creep behaviour of each timber sample loaded to approximately 25% of the ultimate bending strength can be seen. While this study was performed over a relatively
short period of time (~200hr), it demonstrated the effect of different constant moisture levels on the viscoelastic behaviour of timber.

![Creep data vs creep compliance function fit](image)

*Figure 2.5. Creep data vs creep compliance function fit (Hering & Niemz 2012)*

The stress level has also been shown to have a significant effect on the creep behaviour of timber. Senft & Suddarth (1971) examined small specimens of Sitka spruce (Picea sitchensis) at stress levels of 10, 20, 40 and 60% of ultimate strength for load durations up to twenty days. The moisture content and relative humidity remained constant throughout in order to exclude the mechano-sorptive effect and solely examine viscoelastic creep. They found that the total creep behaviour increases with increasing stress levels and significantly, they found that creep deformation can occur at stress levels as low as 10% of ultimate strength. It was also reported that, at higher stress levels (>55%), specimens are susceptible to creep rupture resulting in failure.

Similarly to both moisture content and stress level, higher temperatures have been shown to result in higher viscoelastic creep deformations. Whether examining the effect of moisture content, stress level or temperature, the magnitude of viscoelastic creep is often correlated well with its initial elastic deformation when loaded under constant climate conditions (Hanhijarvi 1995). As a result, although stress level, temperature and moisture content have a significant effect on the total deformation, significant differences are not observed when examining these effects using the relative creep measure (Equation [2.5]) under practical applications. Practical applications refer to stress levels below 55%
of the ultimate load which is likely to results in creep rupture (Davidson 1962; Hanhijarvi 1995; Toratti 1992a).

2.5.2.2 Mechano-sorptive Creep Component

In timber elements creep deflection is accelerated under combined stress and variable relative humidity conditions. This behaviour is known as mechano-sorptive creep and is a deformation due to an interaction between stress and moisture content change in timber (Armstrong & Kingston 1960; Armstrong & Christensen 1961). Unlike viscoelastic creep, it is independent of time and only occurs during moisture content change in the timber (Armstrong 1972). It is directly related to the change in moisture content and mechanical stress.

Mechano-sorptive behaviour has been the focus of many studies, which have investigated its ability to rapidly increase deflection and cause premature failure in timber elements. Its complex nature makes it difficult to quantify and investigating the many variables involved in this behaviour requires complex and expensive resources. In one of the first studies, Hearmon & Paton (1964) examined small clear timber beams subjected to creep tests under constant and cycling relative humidity conditions.

![Figure 2.6. Small clear wooden beams subjected to constant and variable relative humidity conditions under load (Hearmon & Paton 1964)](image-url)
The test results can be seen in Figure 2.6 in which relative creep is plotted against time. The lower creep curve represents the viscoelastic creep component of a specimen tested in a constant climate condition. This beam was loaded to 37.5% of the ultimate short-term load in a constant 93% relative humidity. The higher fluctuating curve represents the total deformation of a similar beam loaded to 37.5% of the ultimate short-term load in a variable relative humidity condition ranging from 0%-93% relative humidity in 24 hour cycles. In addition to the viscoelastic component, it can be seen that the effect of mechano-sorptive creep accelerates the creep deflection of the beam and leads to premature failure after approximately 28 days.

The change in moisture content results in a significant acceleration in creep deflection over a relatively short time period. Experimental work performed by Hunt (1982) indicated three separate characteristics of mechano-sorptive behaviour as a result of variations in relative humidity. Firstly, he noticed that a decrease in moisture content results in an increase in creep. Secondly, it was observed that an increase in moisture content above the highest previously attained moisture content also results in an irrecoverable increase in creep. Thirdly, an increase in moisture content below the highest previously reached moisture content results in a decrease in creep deflection. The magnitude of this mechano-sorptive creep is also influenced by the moisture differential in each relative humidity cycle. The higher the differential, the higher the amount of creep observed (Armstrong & Kingston 1960). Hunt (1999) later suggests that the rate of this moisture content change is important in mechano-sorptive creep which indicates a possible size effect, which was previously proposed by van der Put (1989).

Many researchers have also attempted to examine various timber parameters and their influence on mechano-sorptive behaviour. In an extensive series of studies, Bengtsson (1999; 2000b; 2001b; 2001a) examined the influence of several timber material parameters on the mechano-sorptive creep behaviour in timber. Some of these parameters include modulus of elasticity, density, compression wood, slope of grain, annual ring width and knot area ratio. It was found that modulus of elasticity and annual ring width correlate best with relative creep with other comparisons demonstrating weaker correlations. In addition to the tests by Bengtsson (1999; 2000b; 2001b; 2001a), Hunt (1986) found strong correlations between mechano-sorptive creep, microfibril angle and elastic compliance (1/modulus of elasticity) when he performed numerous creep tests on small specimens made of pine. The relatively strong correlation between modulus of
elasticity and relative creep was deemed important. As the modulus of elasticity can be calculated from the results of commercial grading machines, an adequate prediction of the relative creep in graded structural timber may be calculated using such correlations.

2.5.2.3 Swelling and Shrinkage Component

Changes in moisture content of timber elements not only results in mechano-sorptive creep but also results in dimensional changes of the timber. Hygro-expansion coefficients or swelling and shrinkage coefficients describe the magnitude of these dimensional changes in timber. Such dimensional changes are often neglected when examining mechano-sorptive creep in loaded timber beams; however, the swelling/shrinkage component contributes to the total creep deformation and has been considered in a number of studies.

In an attempt to distinguish between swelling/shrinkage strain components and mechano-sorptive strain components, Bengtsson (2000b; 2001b; 2000a) monitored longitudinal shrinkage/swelling strains along four non-loaded control specimens which had been matched with bending creep test specimens and subjected to the same variable relative humidity climate conditions. These swelling/shrinkage strains were measured with 60 mm long electrical resistance strain gauges and subtracted from the total strain measured on the bending creep test specimens. In Figure 2.7, the typical strain measured on the tension face of the beam (S3, 3t) can be seen.

![Figure 2.7. Swelling/shrinkage strain component subtracted from the total strain component of a loaded beam (Bengtsson 2000a)](image-url)
This typical strain curve includes the elastic, viscoelastic, mechano-sorptive and swelling/shrinkage strain components. The compensated tension curve indicated by the dashed line in Figure 2.7, is created by subtracting the free shrinkage/swelling strains measured on the non-loaded beam from that measured on the loaded beam (S3, 3t). Interestingly the sudden increases in strain commonly attributed to mechano-sorptive creep are almost erased when subtracting the swelling/shrinkage component measured on the matched non-loaded beams. A similar trend was also observed in compression.

The swelling/shrinkage strains in timber are generally independent of the stress level induced in a timber element and simply proportional to the magnitude of moisture content change in the timber. For this reason, when timber is loaded under relatively low stress levels, as might be the case under serviceability limit state design, the swelling/shrinkage component may contribute to a significant proportion of the total strain deformation as is the case in Figure 2.7. It has also been shown that timber with high proportions of juvenile wood content, such as fast-grown Sitka spruce in Ireland, are susceptible to large swelling/shrinkage coefficients when compared to a slow-grown timber (Bengtsson 2001c; Ormarsson & Cown 2004). Meylan (1968) examined the swelling/shrinkage strains induced in timber specimens with large percentages of juvenile wood. Considerable swelling/shrinkage strains parallel to the grain were experienced in the test results when compared to timber with no juvenile wood content. It is clear that fast-grown species are susceptible to increased swelling/shrinkage strains in fluctuating moisture conditions due to the percentage of juvenile material and should be considered when attempting to quantify individual creep components which combine to give the total deformation of a timber element in a variable climate.

2.5.3 Creep Tests on Unreinforced Timber Elements

2.5.3.1 Introduction

For many structural applications, the most important mechanical property of timber is its resistance to deflection, including both elastic and creep deflection. As described in Section 2.5.2, creep can be divided into three major components, namely viscoelastic creep, mechano-sorptive creep and pseudo-creep attributed to swelling/shrinkage deformations. These three categories combine to result in the total creep deflection, the magnitude of which has been shown to be influenced by many catalysts such as temperature change, moisture content change, stress, and the duration of the applied stress.
(Morlier & Palka 1994). In addition to these catalysts, the internal cell structure and growth conditions of timber may also be a contributing factor.

In a study by Zhou et al. (1999), bending creep tests were performed on six different timber species (3 hardwood and 3 softwood species) under a varying relative humidity climate condition. Each test was performed at a constant temperature of 20°C with a relative humidity climate varying between 65% and 95% relative humidity. A constant load level corresponding to 25% of the short-term failure load was applied to each specimen in four-point bending. The results from all six species tested, show that hardwood species are more sensitive to moisture content change than softwood species and are prone to larger mechano-sorptive creep deformation. It has also been shown when studying a single species, that significant variations in the total creep deformation can be observed due to different growth conditions (Bengtsson 1999; Hunt 1997). Bengtsson (1999; 2000b; 2001b; 2001a) who studied slow-grown and fast-grown Norway spruce, found that in 81 Norway spruce specimens tested, the presence of juvenile wood in the fast-grown trees resulted in significantly larger deflections in the first moisture cycle when compared to specimens with no juvenile wood content. This first moisture cycle is often associated with large deformations whereby an increase in moisture content above the highest previously attained moisture content results in an irrecoverable increase in creep (Hunt 1982). This is of particular interest from an Irish perspective as Irish-grown Sitka spruce is associated with high proportions of juvenile wood due to the fast growth conditions and silvicultural practices.

In a select number of studies, the creep performance of glued laminated timber and sawn solid timber beams of equal dimensions have been tested and compared under similar load and climate conditions. Hoyle et al. (1986; 1994) examined relative creep in Douglas fir beams for both solid and glued laminated timber in variable relative humidity condition. Each beam was loaded to 55% of their ultimate bending strength. The relative humidity was cycled between 40%-80% in 184 hour cycles for a duration of 1000 hours. The results in Figure 2.8 show a significantly better long-term response from the glued laminated timber beams in a variable climate. When compared to similar beams in a constant climate, an increase in creep deflection of 200%-400% was observed for the solid timber beams as a result of the variable relative humidity and an increase in creep deflection of 40%-72% was observed for the glued laminated beams as a result of the variable relative humidity. They believed that the water proof properties of the adhesive
glue-lines in the glued laminated beams reduced the movement of water vapour into the timber resulting in less mechano-sorptive creep. It is also stated that glued laminated beams are more homogenous than solid timber beams and benefit from a dispersion of growth characteristics which adversely affect the behaviour of timber elements in a variable climate.

Figure 2.8. Creep test results comparing glued laminated beams to sawn solid timber beams (Hoyle et al. 1994)

In the production of timber elements for structural applications, certain practices are used to improve the performance and deliver reliable building products. A series of studies have looked at these practices and their effect on the long-term performance of these timber elements. One such practice is the drying of timber from its green state. Structural timber is carefully kiln dried to reduce the moisture content to approximately 18% in a controlled process in an attempt to reduce warping, splitting, checking or other harmful effects as a result of natural drying. Two common methods exist. One is low-temperature drying and the other is high temperature drying. In an effort to examine the effect of both methods on the long-term performance of timber elements, Bengtsson & Kliger (2003) performed bending creep tests on high-temperature dried Norway spruce and compared these to the more traditional low-temperature dried timber. The high-temperature drying was performed at 115°C and the low-temperature drying was performed at 70°C. The creep tests were performed in a varying climate with 20°C and 30%-90% relative humidity. A total of 24 specimens, measuring 45 x 70 x 1100 mm$^3$, were loaded in
bending for approximately 240 days. The results show high-temperature dried timber experiences approximate 30% less total creep deformation when compared to the low-temperature dried timber specimens. These results indicate a lower mechano-sorptive effect for high-temperature dried timber. It was also noted that the free swelling/shrinkage strains were also significantly smaller in the high-temperature dried specimens.

Another process which is often utilised in the production of structural timber elements is the application of a surface coating or impregnation of a protective substance into the timber. Timber can receive these surface coatings in order to protect against biological attack or simply for aesthetic purposes. However, timber with a protective surface coating also experiences less change in moisture content as the flow of moisture is impeded. Various studies have examined the effect of surface coatings on the long-term performance of timber elements. In a study by Mårtensson (1994a) comparisons are made between uncoated beams and beams with a protective surface coating. The mechano-sorptive creep was reduced in the coated beams due to the impeded flow of moisture within the timber. The mechano-sorptive creep component was not completely halted due to this protective treatment but a significant reduction in creep was observed (Mårtensson 1994a). Ranta-Maunus & Kortesmaa (2000) investigated the effect of various surface treatments on the creep behaviour of timber beams under low stress levels (2 MPa). The results of the experiments seen in Figure 2.9 show continuous but slow creep and a clear dependency on surface coating.

![Figure 2.9. Surface treated creep test results compared to a non-treated specimen (Ranta-Maunus and Kortesmaa 2000)](image_url)
The relative creep deformation ranges from approximately 1.3 to 2.0 after 6-7 years. The lowest creep is observed with impregnated specimens (Creosoted). In another study, Abdul-Wahab et al. (1998) examined the effect of varnish on the long-term creep behaviour of timber and observed an average reduction of 36% in relative creep (ranging from 22.1-58.8%).

It is clear to see that many material properties and practices influence the long-term behaviour of structural timber products. In an attempt to study these many parameters, the majority of studies make comparisons under common loading and climate conditions, however, these load and climate conditions vary between studies, often making comparison difficult. The following sections (Section 2.5.3.2 and Section 2.5.3.3) focus on the loading conditions and climate conditions used in various studies outlining their aims and observations.

2.5.3.2 Loading conditions for Creep Tests

The lack of a consistent test standard as mentioned previously has resulted in the implementation of various loading schemes in creep tests to date. The majority of tests are conducted under four-point bending. However, creep tests have been conducted in three-point bending (Plevris & Triantafillou 1995) and a limited number of studies have performed creep tests under evenly distributed loads (Runyen et al. 2010). The majority of studies have been conducted under stress levels a fixed percentage of the ultimate failure load, however, these are erratic and can vary from 10% of the ultimate failure load to 60% of the ultimate failure load. EN 1156 (CEN 2013a) is the only European test standard which provides a test set-up for long-term creep testing, however, this standard focuses on wood-based panel products and not solid timber or glued laminated timber products. This standard recommends for the determination of creep factors that a load corresponding to 25% of the ultimate failure load shall be used. This single stress level rests on the assumption that the material is linearly viscoelastic up to a level of at least 40% of ultimate failure load. If the validity of this assumption is in doubt, a series of varying stress levels between 10% and 40% should be applied. The stress level is an important factor in the development of creep deflection and has been the focus of many studies. Where the ultimate failure load is unknown, a percentage of the design stress or allowable design stress has been used as an alternative (Abdul-Wahab et al. 1998; Davids et al. 2000).
It was previously shown by Senft & Suddarth (1971), that the stress level had a significant effect on the total creep deflection and with increasing stress, the creep deformation increases, however, no information is given regarding the relative creep deformation in this particular study and it is difficult to observe the true effect of the stress level. Bengtsson (1999) performed creep tests on 81 Norway spruce specimens 45 x 70 mm² by 1.10 m in length. The bending test specimens were loaded to a varying bending stresses ranging from 6.0-15.4 MPa. It was observed that the difference in bending stress had little or no influence on the final relative creep behaviour. Similar results were presented by Ranta-Maunus & Kortesmaa (2000).

The vast majority of bending creep tests were carried out at high stress levels ranging from approximately 5 MPa to 15 MPa. The high stress levels were often chosen to produce measurable creep deformations in a relatively short time period (Hoyle et al. 1994). Ranta-Maunus & Kortesmaa (2000) initiated an experiment to examine the long-term creep effects at lower stress levels (2 MPa to 5 MPa). They state that these lower stress levels are more realistic of long-term, permanent loads in timber structures and significant creep deformations with time were observed in the test results at these low stress levels.

2.5.3.3 Climate Conditions for Creep Tests

The creep behaviour of timber elements can vary considerably and is dependent on the climatic conditions. The lack of a common test standard for examining creep in timber elements arises again and, as a result, many different relative humidity cycles and temperature conditions have been implemented in tests to examine the creep behaviour of timber in variable climate conditions. The duration of the variable climate cycle is another factor that influences the overall creep behaviour of timber.

A series of studies have examined the effect of temperature changes on the creep behaviour of structural timber. Schniewind (1967) performed creep tests on small timber specimens in a climate fluctuating between 15°C and 32°C in a 24 hour cycle. This variable temperature was designed to be representative of an external climate condition commonly experienced in a number of U.S localities during summer months. The results suggest that temperature cycling is of minor importance and that large changes in relative humidity at constant temperature have a greater influence on the creep behaviour of timber. A similar observation was also made by Toratti (1992a) when testing small
specimens of wood under a northern European cyclic temperature climate ranging from 20°C to -5°C. These results, coupled with the usually consistent temperature ranges experienced within internal structural applications, makes relative humidity changes the more significant component of the two.

When examining the adverse effects of relative humidity change on loaded timber beams, it is also important to remember that the consequent change in moisture content has a significant effect on the material properties of timber in an unloaded case. Bengtsson (2000b) performed bending stiffness measurements on 968 specimens of Norway spruce (11 x 11 x 200 mm³) at different equilibrium moisture contents. A dynamic test method was used to measure the stiffness. These equilibrium moisture contents correspond to 30% and 90% relative humidity, respectively. Measurements presented in this study show an average change in elastic modulus of 1.0% per percent of moisture content change. This is similar to the European standard EN 384 (CEN 2010) where an adjustment of 1.0% per percent in moisture content is recommended for tests of elastic modulus not conducted at reference conditions. Under changing moisture content conditions, the change in the stiffness of the element will also contribute to the total creep deformation and should be taken into account (Santaoja et al. 1991; Fortino et al. 2009).

Abdul-Wahab et al. (1998) performed a series of creep tests on 65 unreinforced glued laminated timber and solid timber beam specimens under different environmental conditions over an 8 year period. The beams of section sizes ranging from 100 x 30 mm² to 265 x 90 mm², were subject to four-point bending as shown in Figure 2.10. These beams were subjected to three environmental conditions. Although not deliberate, these conditions happened to coincide with Service Classes 1, 2 and 3 as defined in Eurocode 5 (CEN 2005). Environmental condition 1 was set at a constant temperature of 20°C and a constant relative humidity of 60% ± 10%. Environmental condition 2 was a variable climate with a constant temperature of 20°C and a variable relative humidity ranging between 30% and 70% in a 16 week cycle and environmental condition 3 was an external climate in a covered enclosure with a variable relative humidity ranging between 30% and 100%. Moisture measurements were recorded regularly as well as temperature and relative humidity. The mean test results from glued laminated beams for each climate condition can be seen in Figure 2.11.
In the constant Service Class 1 climate, a mean creep factor or $k_{def}$ factor ($k_{def} = \text{Relative Creep} - 1$) of approximately 0.75 can be seen after approximately 2800 days. The mean result from the beams in the external Service Class 3, experience the greatest creep, averaging a 285% increase in creep when compared to Service Class 1 beams at constant temperature and relative humidity. The Service Class 2 beams in a variable climate experienced an increase of 165% when compared to the beams examined at constant temperature and relative humidity.

Figure 2.11. Creep results for all three environmental conditions (Abdul-Wahab et al. 1998)
It is noted that for the wetting/drying cycle implemented in the variable climate, the equilibrium moisture content was not achieved and longer relative humidity cycles would result in greater moisture content change and possibly increased creep behaviour. This also indicates that the size and shape of the beam are of great importance. When expressed in terms of volume per surface area, the size and shape were shown to have a significant effect on the creep behaviour with the smaller ratio, resulting in a greater creep tendency.

Schniewind & Lyon (1973) also to examine the effect of cyclic humidity on two different sized beams. Each beam was tested at a relative humidity ranging from 35% to 87% and temperature remained constant throughout. They found a similar size effect resulting in larger specimens being affected less by changing moisture content. The movement of moisture into timber is a slow process and in larger structural sized timber beams, the rate of moisture content change will be even slower. In a study by Svensson et al. (2011), large moisture gradients were observed in timber beams when variations in relative humidity are frequent and it is stated that a complete change in equilibrium moisture content within a structural timber beam due to variations in relative humidity are only found for members subjected to varying humidity with long annual periods.

In Hoyle et al. (1986; 1994), solid Douglas fir beams of commercial size were examined under two cyclic relative humidity cycles of different durations. One short cycle of 24 hours and one long cycle of 168 hours was examined. The relative humidity differential ranged between 40% and approximately 90% during both the short and long cycle. The beam specimens in the longer relative humidity cycle of 168 hours experienced a change of 2.3% compared to a change of 0.4% for the beams in the short 24 hour cycle. There was a large time lag in the response of commercial sized beams to changes in relative humidity indicating a need to increase the cycle length to induce greater change in moisture content.

From the chosen studies presented in this section, it is clear that the relative humidity differential and duration have a significant effect of the final creep deflection of timber members. The dimensions of the timber specimens and rate of moisture content change also play a significant role. Performing such tests can be time consuming and many authors have attempted to reduce the duration of the test, however, inducing short relative humidity cycles on structural sized beams has been shown to result in low changes in moisture content. Additionally, in a proportion of studies it appears that many...
of the chosen relative humidity cycles and cycle durations are arbitrary and chosen in an attempt to induce significant mechano-sorptive deformations.

2.5.4 Creep Tests on Reinforced Timber Elements

2.5.4.1 Introduction

The addition of reinforcement to solid or glued laminated timber elements has been shown to have a significant effect on the short-term performance but studies on its effect on long-term performance have mostly been limited in scale. The variability in climate conditions and loading schemes described previously in creep tests on unreinforced beams is further complicated when considering the additional influence of reinforcement. Some authors have chosen a single dead load and have applied this load to both unreinforced and reinforced beams resulting lower maximum stress levels in the timber component of reinforced beam. Alternatively, some studies have attempted to load the unreinforced and reinforced beams to a common maximum stress level leading to higher loads on reinforced beams. This section discusses some of the relevant studies performed on reinforced beams in both constant and variable climates.

2.5.4.2 Constant Climate Creep Tests

In a constant climate, the effect FRP reinforcement on the time dependent or viscoelastic creep can be examined. In one of the first studies, Plevris & Triantafillou (1995) performed long-term load tests on CFRP reinforced beams over a 300 day period. There was a relatively small sample size of three beams, one unreinforced control beam (Beam 1) and two reinforced beams with two different percentage area reinforcement ratios of 1.18% (Beam 2) and 1.65% (Beam 3), respectively (Figure 2.12). It was also stated that the value of elastic modulus for each timber beam did not differ by more than 2-3% prior to reinforcement. The beams (1, 2 and 3) were loaded in three-point bending to 2.63 kN, 2.83 kN and 2.76 kN, respectively. The similar loading scheme demonstrated the effect of reinforcement and percentage area reinforcement ratio on the reduction of initial elastic deflection and creep deflection over a 300 day period, however, with the addition of the CFRP reinforcement there is significantly less stress in the timber component which could explain the reduced creep deformation. Similarly, Lu et al. (2012) also imposed a constant dead load on unreinforced and reinforced glued laminated beams. Ten beams were loaded in four-point bending with 460 kg load applied corresponded to 30% of the
ultimate strength of the timber. The results allowed for the comparison of different reinforcement configurations (6mm steel rebar in the compression zone, 6mm steel rebar in the tensile zone and 8mm steel rebar in the tensile zone) against the control (Unreinforced) specimens as seen in Figure 2.13a.

In this particular study the relative creep measure is used to compare the reinforced and the unreinforced specimens (Figure 2.13b). It is also stated that, after extrapolating from the experimental data, the relative creep of reinforced members after 50 years has been reduced by approximately 80% due to the addition of the FRP in various configurations. However, as was the case in the study by Plevris & Triantafillou (1995) similar load levels were applied to both unreinforced and reinforced beams and therefore, a lower stress was induced in the reinforced beams.

Figure 2.12. CFRP Reinforced Specimens (Plevris and Triantaillou 1995 (With permission from ASCE))

Figure 2.13. Creep behaviour of reinforced members: a) Total creep deflection b) Relative creep deflection (Lu et al. 2012)
Breton (1999) performed creep tests on twelve glued laminated beams. Eight of these beams were reinforced and four of these beams were unreinforced. Two percentage area reinforcement ratios were examined in this study (1.1% and 3.3%). These beams were loaded to 125% of their allowable design stress which corresponds to approximately 30-40% of the ultimate strength. The allowable design stress was calculated by taking the 5% lower tolerance limit of the ultimate strength from a previous study performed by Dagher & Lindyberg (1999) who examined the ultimate strength on 90 beams. In this study, the ultimate strength was calculated separately for the unreinforced and reinforced beams indicating more comparable stress levels in both the unreinforced and reinforced beams. Interestingly the relative creep data after 120 days is similar between all specimens, reinforced and unreinforced. This is important because the reinforced beams are subject to higher loads than the unreinforced beams of the same size. A Douglas fir beam with 3.3% GFRP carries 80% more load than its equivalent unreinforced beam.

In a study by Yahyaei-Moayyed & Taheri (2011), the creep performance of southern yellow pine (SYP) and Douglas fir (DF) timber beams reinforced with aramid fibre reinforced polymer (AFRP) was examined. These creep tests were carried out in an uncontrolled but relatively constant climate over a period of 800 hours and it is noted that the applied loads were not the same for the unreinforced and reinforced beams. In the results (Figure 2.14a), the comparison between one SYP unreinforced (S5-PIV) and one SYP reinforced beam (S5-PI) is seen and although there appears to be a reduction in creep deflection; it is not clear if this reduction is due to varying loads within the timber or the AFRP reinforcement.

![Figure 2.14. Creep deflection comparison results (Yahyaei-Moayyed & Taheri 2011)](image)
The reduced load on the reinforced beam will lead to a lower stress level within the timber. Interestingly in Figure 2.14b where one unreinforced DF beam (S10-PI) and one reinforced DF beam (S10-PIV) are compared; there is a slightly higher load on the reinforced beam and a similar creep deflection is observed. It is unclear if the stress is comparable within both beams. There is also a slight influence of the variable relative humidity and possible swelling/shrinkage or mechano-sorptive creep deformations as a result of the fluctuating moisture content (Figure 2.14).

2.5.4.3 Variable Climate Creep Tests

In variable climates, the effect of FRP reinforcement on the time dependent and moisture dependent creep behaviour can be examined. There are a limited number of studies that examine the long-term performance of reinforced beams in a variable climate. However, some of the most relevant studies were performed by Gilfillan et al. (2003), Patrick (2004), Hansson & Kristoffer (2007) and Kliger et al. (2008).

In an external but sheltered climate, Gilfillan et al. (2003) and Patrick (2004) performed creep tests on six beams manufactured from Irish-grown Sitka spruce. Three of the beams were unreinforced and three of the beams were reinforced with CFRP. The reinforced and unreinforced beams were subjected to constant three-point bending loads of 85.5kg and 62.1kg, respectively. The magnitude of the applied load was calculated using the elastic bending stress in compression derived from the ultimate bending stress from previous tests performed. The load on each beam corresponded to 25% of ultimate strength. The deflection and moisture content was monitored on a weekly basis for 160 weeks. It was not possible to draw any significant conclusions due to the low number of beams tested, however, the plot of creep deflection versus time in Figure 2.15 clearly demonstrates the reduced creep deflection in reinforced beams. This graph represents a creep deflection of approximately 50% less in the reinforced beam than the unreinforced beam.

Reinforced beams were also subjected to creep testing in studies by Hansson & Kristoffer (2007) and Kliger et al. (2008). In total, 24 beams measuring 45 x 70 x 1100 mm$^3$ were manufactured. Four groups, equal in terms of elastic modulus were created, and three of these groups were reinforced with a different reinforcing material in tension zone.
The three reinforcement schemes involved the adhering of a reinforcing material in grooves routed the entire length of the beams. This allowed beams of equal depth to be created. The three reinforcement materials consisted of a steel plate reinforcement and two CRFP plate materials of different elastic moduli. To ensure consistent exposure conditions, the unreinforced beams were sealed on the bottom surface with an epoxy resin to ensure all beams were equally exposed to the surrounding climate conditions. They loaded each beam in four-point bending to a common maximum compressive bending stress in the timber of 8 MPa. This single stress level of 8 MPa leads to higher loads on reinforced members due to their higher stiffness. The beams were initially conditioned to 65% relative humidity and directly after loading of the specimens, the climate was set to 90% relative humidity. Thereafter the climate was changed between 30% and 90% relative humidity in a 28 day cycle. The room temperature was held constant at 23°C for
the whole test period. The mid-point deflection was measured on an hourly basis. The mean relative creep results of the beam groups can be seen in Figure 2.16.

![Figure 2.16. Creep in reinforced members (Kliger et al. 2008)](image)

Greater relative creep deflection can be seen in the unreinforced timber and epoxy beams when compared to the reinforced beams. It can also be seen that the fluctuations in deflection with each relative humidity cycle are larger in the reinforced case. It is stated that this is a result of differential shrinkage on either side of the beam as the compression side swells/shrinks, the tension side is continuously restrained by the reinforcement. The results show that the addition of approximately 2% CFRP reinforcement not only improves the flexural performance of the beam but can also reduce the long-term deflection attributed to mechano-sorptive creep when compared to unreinforced beams.

2.6 **MATHEMATICAL MODELS OF CREEP BEHAVIOUR**

2.6.1 **Introduction**

This section details research to date on the use of analytical and numerical models for the prediction of long-term deformations in timber elements in both a constant and variable climate. Timber is a challenging material to model due to its natural variability in properties and anisotropic behaviour; however, in recent years, timber has been modelled successfully thereby increasing the reliability and safety in structural timber design. Experimental monitoring of the long-term behaviour of engineered wood products is a costly and time consuming process and the benefits of a validated model will provide a powerful tool to aid the further development of such engineered wood products in the
future. This is also true when examining the additional benefits of FRP reinforcement on the long-term deflection of timber elements.

The influence of the surrounding environment on timber elements is very important in understanding the behaviour of this engineering material. For this reason, special attention is given to studies modelling viscoelastic creep behaviour in a constant climate and studies modelling mechano-sorptive creep behaviour in a variable climate.

2.6.2 Modelling Viscoelastic Creep Behaviour

Rheology is the study of the flow of matter and rheological models can be used to describe time dependent viscoelastic behaviour in timber elements. The basic elements of these models are known as spring and dashpot elements. When combining springs and dashpots in series and/or parallel, rheological or viscoelastic models can be created (Simo & Hughes 1998). The spring behaviour as described in Equation [2.7] is perfectly linear elastic representation of Hooke’s law and models the elastic component of a viscoelastic material.

\[
\sigma = E\varepsilon
\]  \hspace{1cm} [2.7]

where:
\[\sigma = \text{stress}\]
\[E = \text{modulus of elasticity}\]
\[\varepsilon = \text{strain}\]

The dashpot behaviour as described in Equation [2.8] is a viscous element that extends at a strain rate proportional to the applied stress:

\[
\sigma = \eta \dot{\varepsilon}
\]  \hspace{1cm} [2.8]

where:
\[\eta = \text{viscosity coefficient}\]
\[\dot{\varepsilon} = \text{strain rate}\]

The two most common models are the known as Maxwell and Kelvin models which comprise Maxwell and Kelvin elements, respectively (Simo & Hughes 1998; Hanhijarvi 1995; Marques & Creus 2012). The Maxwell element combines a spring and a dashpot in series as seen in Figure 2.17a. The Kelvin element combines a spring and dashpot in parallel as seen in Figure 2.17b.
Figure 2.17. Rheological models: a) Maxwell element, b) Kelvin element

Figure 2.18. Element behaviour under a constant stress, $\sigma_0$: a) Maxwell element, b) Kelvin element

The behaviour of each element is heavily dependent on the arrangement of the individual elastic springs and viscous dashpots. As seen in Figure 2.18a, when a constant stress is applied to the Maxwell element, the deformation is the sum of the strain in the elastic spring and the viscous dashpot and both elements are subject to the total stress ($\sigma_0$). The viscous strain component increases with time until a point when the stress is removed resulting in an immediate decrease in the elastic strain component followed by zero change in the viscous component under zero stress. In contrast, when a constant stress is applied to a Kelvin element (Figure 2.18b), the strain in both the spring and dashpot are
the same due to the parallel arrangement and the total stress is a combination of the stress in the spring and the stress in the dashpot.

In practical applications, generalised models are required to accurately model viscoelastic material behaviour. Generalised models combine a number Maxwell or Kelvin elements, which increases the number of parameters and gives a better representation of the behaviour of the material being studied. In relation to timber elements, the viscoelastic component of creep is sometimes neglected in numerical models due to its relatively low contribution to the total creep deformation, which occurs under changing moisture content conditions, however, rheological models have been used to model the viscoelastic behaviour of timber. In 1971, Senft & Suddarth (1971) utilised a Burger model which consists of Maxwell and Kelvin elements. They examined small specimens of Sitka spruce (Picea sitchensis) at stress levels of 10, 20, 40 and 60% of ultimate strength for load durations up to twenty days. The three and four parameter models adequately simulated the time dependent viscoelastic creep behaviour of timber, however, they suggest a four-parameter model for durations longer than 24 hours or at high stress levels. This model was limited to a uniaxial direction and could only model viscoelastic creep as changes in relative humidity were omitted.

In another study by Gressel (1984), the creep behaviour was predicted and compared to experimental tests with a duration of ten years. Three of the creep models used in this study can be seen in Equations [2.9]-[2.11]

\[
\varepsilon_{\text{creep}} = \beta_1 + \beta_2 \left(1 - e^{-\beta_3 t}\right) \tag{2.9}
\]

\[
\varepsilon_{\text{creep}} = \beta_1 + \beta_2 \left(1 - e^{-\beta_3 t}\right) + \beta_4 t \tag{2.10}
\]

\[
\varepsilon_{\text{creep}} = \beta_1 + \beta_2 \left(1 - e^{-\beta_3 t}\right) + \beta_4 t^{\beta_5} \tag{2.11}
\]

where:
- \(\varepsilon_{\text{creep}}\) = creep strain
- \(\beta_i\) = model parameter
- \(t\) = time
The use of a four parameter model (Equation [2.10]) proved superior when compared to a three parameter model (Equation [2.9]) when predicting the viscoelastic creep deflection over the 10 year period. This was also observed by Dinwoodie et al. (1984), Pierce et al. (1985) and Dinwoodie et al. (1990) in a series of studies over a 15 year period quantifying and predicting the creep behaviour in particle board.

![Graph showing comparison of projected four and five parameter model curves from 24 week data (Dinwoodie et al. 1990)](image)

*Figure 2.19. Comparison of projected four and five parameter model curves from 24 week data (Dinwoodie et al. 1990)*

They determined that a three parameter model (Kelvin model) was suitable when describing creep over a short time, however, produced poor results when attempting to predict long-term creep from short-term data. This was also the case when examining a four parameter model (Burger model) and a five parameter model (Equation [2.11]: Four element model with non-linear viscous dashpot) was developed and produced significantly better results over the test period as seen in Figure 2.19.

### 2.6.3 Modelling Mechano-sorptive Creep Behaviour

When modelling the long-term behaviour of timber elements, the surrounding environment must be considered given its significant effect on the creep behaviour. Many researchers have attempted to predict the effects of variable humidity and moisture fluctuations on timber elements and to model mechano-sorptive behaviour (Leicester 1971; Ranta-Maunus 1976; Toratti 1992b); however, it has proven difficult to model all the possible phenomenon found in experiments as the individual components occur...
simultaneously (Mårtensson 1994a). Leicester (1971) and Ranta-Maunus (1975, 1976) developed some of the first models in an attempt to quantify the mechano-sorptive effect in timber. Leicester (1971) produced a uniaxial model consisting of an elastic component and an irrecoverable mechano-sorptive component. The predicted results were examined against experimental data on small samples of Eucalyptus during two weeks of drying. This rheological model was found to predict 85% of the total deformation recorded during testing. In a later study, Fridley et al. (1992) developed a sophisticated model based on the Burger model which included mechano-sorptive creep effects, but also included moisture and temperature dependent variations in stiffness. This adaption provided good agreement between modelled and experimental strain data. Mohager & Toratti (1992) presented a mechano-sorptive model incorporating different tensile and compressive compliance functions and a model including partially irrecoverable compressive strains was proposed. Mårtensson (1994a) developed another viscoelastic model that decomposed strain rate into elastic, viscoelastic, shrinkage/swelling, and mechano-sorptive creep components. This model transitioned from a uniaxial model to a three-dimensional form of a constitutive model that describes mechanical effects and moisture effects. Lu & Leicester (1997) presented a mechano-sorptive model coupled with a simple approximate solution for moisture variation within a specimen based on Fick’s law of diffusion and sinusoidal fluctuations in ambient moisture boundary conditions. Hanhijärvi & Mackenzie-Helnwein (2003) presented an orthotropic material model based on a one-dimensional model presented by Hanhijarvi (2000) incorporating elastic, viscoelastic, mechano-sorptive and swelling/shrinkage strains. The mechano-sorptive strain component is altered to incorporate permanent deformations by introducing a hardening type plasticity element. This was found to describe the deformations associated with high temperature drying in timber elements. Fortino et al. (2009) presents a three-dimensional model for analysing timber structures under variable humidity and stress conditions. They implemented the viscoelastic/mechano-sorptive constitutive model in the user subroutine UMAT of the finite element modelling code Abaqus. Fick’s law is used to describe moisture flow within the timber element. This Fickian behaviour was implemented into the Abaqus user-defined DFLUX subroutine. The model was successfully validated against experimental results performed by Leivo (1991), Toratti & Svensson (2000), Svensson & Toratti (2002) and Jönsson (2005).
2.6.4 Modelling of FRP Reinforced Timber

The addition of FRP material to timber elements has also been modelled using finite element software. Many models have been developed in an effort to fully understand the beneficial effect of FRP material in many different configurations. FRP is generally treated as an anisotropic material. The adhesives used in the manufacture of composite timber beams is treated as a linear elastic isotropic material or sometimes even excluded from the models.

While many models have been developed to examine the effect of FRP reinforcement on timber elements, the majority of these models focus on the short-term performance and are not concerned with viscoelastic creep, temperature and moisture effects. A two-dimensional model was developed by Tingley (1996) to examine the stress-strain relationship of FRP reinforced glued laminated beams. This model neglected to include plasticity, creep, temperature or moisture effects and solely focused on the short-term load response. This model achieved good agreement between bending stress obtained from experimental results and the model results.

Serrano (2001) modelled timber connections with bonded-in rods. The timber was modelled as a linear elastic orthotropic material. It was found that the stress concentrations were greatly influenced by the reinforcement stiffness and thickness. In another study, Alam (2004) implemented anisotropic plasticity in a three-dimensional model to predict flexural properties of LVL beams reinforced with bonded-in FRP reinforcement. This model compared well with experimental findings.

Raftery & Harte (2013) also employed anisotropic plasticity theory in a model to predict the behaviour of unreinforced and reinforced glued laminated beams. Extensive material characterisation experiments were performed on both the timber and FRP reinforcement. The finding of the characterisation experiments was implemented in the model. These beams were reinforced with GFRP plate reinforcement. The model results agreed strongly with experimental results. A parametric study showed that as the reinforcement percentage is increased, the stiffness and ultimate moment capacity are enhanced.

A relatively low number of studies have attempted to predict the long-term performance of FRP reinforced beams under constant or variable climate conditions and these have been limited to the uniaxial case. In a study by Plevris & Triantafillou (1995), an analytical model is employed to predict the creep behaviour of CFRP reinforced timber
beams. The model, based on a Burger element, was developed to account for moisture effects; however, the final analytical results were compared to a small sample of CFRP reinforced timber beams tested in a constant climate condition. Different percentage area reinforcement ratios were used when reinforcing each beam. The experimental and analytical data showed good agreement in a constant climate and the percentage area reinforcement ratio was deemed to provide a significant contribution to the deformation behaviour of reinforced beams. In Davids et al. (2000) a uniaxial model was developed to model the creep behaviour of GFRP reinforced timber beams in a variable climate. The model, combining viscoelastic and mechano-sorptive creep strain parameters, was fitted using the unreinforced specimen test results and then shown to accurately predict the relative creep deflection of the GFRP reinforced beams in a variable climate.

Numerical modelling can be an important tool when modelling complex materials such as timber. It has also been shown to adequately model various reinforcement materials in various configurations. These advances in numerical modelling have increased the reliability of results and as a result can aid future product development. The recent advances in modelling unreinforced timber by Hanhijärvi & Mackenzie-Helnwein (2003) and Fortino et al. (2009) are yet to be applied to reinforced timber elements under changing relative humidity conditions.

2.7 SUMMARY AND DISCUSSION

This chapter has given a brief introduction to the structure and composition of wood and has outlined studies focused on the short and long-term performance of unreinforced and reinforced timber elements. While much work has been performed to examine the short-term performance of reinforced timber beams, the long-term performance is becoming a more prominent area of investigation. It was demonstrated that a lack of a common test standard or test set-up has led to an array of test methods and results within the literature. Variations in specimen geometry, stress level, percentage area reinforcement, reinforcement type and climate conditions, to name a few, have been shown to influence the long-term performance of such elements and it is difficult for comparisons to be made between unreinforced and reinforced beams as a result. It was shown in the literature that the stress level heavily influences the creep behaviour of timber products. As the viscoelastic creep, and mechano-sorptive creep strain components are predominantly related to the timber material and heavily influenced by the stress in the timber, it is
necessary to examine creep behaviour in both unreinforced and reinforced beams at comparable stress levels. This was not always the case with some comparative studies examining unreinforced and reinforced beams at similar constant loads. Under comparable stress levels the effect of FRP reinforcement, on both the viscoelastic and mechano-sorptive creep behaviour in glued laminated beams can be examined more accurately.

Numerous stress levels have been examined and while lower stress levels are representative of realistic long-term loads in in-service timber members (Ranta-Maunus & Kortesmaa 2000), the load level corresponding to approximately 25% of the ultimate load of a timber element has been deemed more suitable when creep testing to produce notable deflection in a reasonable time scale without causing failure in the specimen as seen at higher stress levels (>55% of the ultimate stress).

When attempting to examine mechano-sorptive creep effects in timber elements, experimental tests have been performed in an external climate condition or in a relative humidity and temperature controlled room. In the controlled climate room, the relative humidity differential and duration of the relative humidity cycle may be accurately controlled. In some studies, temperature changes have been examined; however, variable relative humidity and subsequent moisture content changes produced significantly larger deformations than those related to temperature variations and as a result, the temperature was kept constant throughout many experimental tests. The numerous relative humidity differentials and cycle durations studied have made comparisons between studies difficult. Also, the severity of some of the implemented controlled climates has induced substantial and rapid increases in deflection which may not properly represent in-service structural deformations of timber elements with time.

The use of analytical and numerical methods to examine the creep behaviour of timber have been presented. From preliminary uniaxial models to more recent 3-dimensional mechanical models coupled with moisture diffusion models, the use of numerical methods to model timber has evolved and is now considered to be a reliable predictive tool that can increase the safety in timber structural design. Currently, sophisticated models are available to adequately describe the short and long-term behaviour of timber elements in both constant and variable climates; however, numerical modelling of viscoelastic and mechano-sorptive creep within reinforced timber elements has yet to be addressed.
3 SHORT-TERM EXPERIMENTAL TESTING

3.1 INTRODUCTION

The experimental test programme, outlined in Chapter 1, aims to characterise the deformation of unreinforced and reinforced timber beams under long-term loading in a variable climate. The mechanical response comprises several components, namely, elastic, viscoelastic, mechano-sorptive and swelling/shrinkage components. The test programme is designed to enable characterisation of each of these individual components. This chapter describes the design and manufacture of the glued laminated beams that will be used for the experimental test programme and also the initial short-term stiffness testing of the beams that characterises the elastic response.

In order to determine the impact of the reinforcement on the response, it is essential that the influence of the natural variability in the timber on the bending stiffness behaviour is minimised. This is achieved by careful design of the glued laminated beams to ensure that the flexural stiffness is similar in all beams. Flexural tests of each of the beams were carried out in accordance to EN 408 (CEN 2014) to determine the elastic stiffness. Based on these results, the beams are allocated to one of five groups, having statistically equal stiffness properties. Half of the beams are then reinforced with BFRP rods in the tension zone and are retested in flexure to determine the elastic stiffness of the reinforced beams. The initial stiffness values for each of the unreinforced and reinforced beams will also be used in the calculation of the relative creep in Chapter 4.

3.2 MECHANICAL GRADING OF LAMINATIONS AND BEAM DESIGN

The timber used in this study is Irish-grown Sitka spruce. Sitka spruce (Picea sitchensis), which is the most planted species in Ireland, is limited mainly to low-stress applications in domestic construction due to its low strength and stiffness values. The most common structural grade produced by Irish-grown softwoods is C16 grade. The timber sourced for this study is initially kiln dried to 18% and subsequently dried to a nominal 12% equilibrium moisture content in a conditioning chamber at a temperature of 20°C ± 2°C and at a relative humidity of 65% ± 5% prior to structural grading.
In order to minimise experimental variations due to the raw material variability, it is desirable to create beams of similar properties. To achieve this, each individual lamination is graded to establish its flexural stiffness and the laminations are then ranked in order of decreasing flexural stiffness. A total of 160 Sitka spruce laminations measuring 34 mm x 98 mm x 4200 mm were graded using a Cook Bolinder machine grader in accordance with EN 14081-4 (CEN 2009a). Figure 3.1 shows a mechanism similar to that of the Cook Bolinder stress grading machine. The Cook Bolinder applies a predetermined deflection at the mid-span and the load required to achieve this deflection across the 900 mm span is measured.

![Figure 3.1. Stress grading mechanism similar to that of the Cook Bolinder grading machine.](image)

The Cook Bolinder grader enables the elastic modulus to be determined at 100 mm intervals along the length of the lamination. Given that the timber laminations are 4200 mm long and 450 mm is required at each end to span the mechanism, an average of 33 stiffness readings are achieved along the entire length of the lamination by inducing the predetermined deflection. Each lamination is passed through the grader twice, measuring the flatwise elastic modulus on both sides of the lamination. By bending each lamination on both sides, the mean elastic modulus of each location is determined and the effects of bow and twist are minimised.

Using the readings from the Cook Bolinder, the weakest locations within each lamination were identified. The final lamination length required was 2300 mm and, where possible, weak or problematic locations were removed during the cutting process when reducing each lamination from 4200 mm to 2300 mm. Each lamination was also visually grading in accordance to IS 127 (2002). Laminations with defects such as wane, fissures or knot area ratios in excess of the limits specified were excluded. The final elastic modulus result for each lamination was taken as the average elastic modulus of the
reduced 2300 mm lamination. Equation [3.1] was used to calculate the elastic modulus ($E_{lam}$) of each lamination.

$$E_{lam} = \frac{F l^3}{4 b d^3 (\delta - \delta_s)}$$  \[3.1\]

where:
- $F$ = Force required to obtain the predefined deflection
- $l$ = the spanned length of 900 mm in the Cook Bolinder
- $b$ = the width of the lamination
- $d$ = the thickness of the lamination
- $\delta$ = the predefined deflection
- $\delta_s$ = the shear deflection

The shear deflection is calculated using Equation [3.2].

$$\delta_s = \frac{3 F l}{10 d b G}$$ \[3.2\]

where:
- $G$ = the shear modulus

Once graded, each lamination is ranked in descending order of stiffness according to the results from the Cook Bolinder (1 being the highest and 160 being the lowest). This ranked table is divided into 4 divisions with the 1st division representing the strongest laminations and the 4th division representing the weakest laminations.

![Figure 3.2. Glued laminated beam lay-up design](image)
Figure 3.2 shows the beam lay-up implemented to create the glued laminated beams. In each division, there is a maximum and minimum value and by carefully balancing between each division, 40 beams of similar properties are created. Division 1 contains the strongest 40 laminations which make up the 40 tensile bottom laminations for each beam. Division 2 is the 2nd strongest group and contains laminations ranked 41-80. These laminations are situated on the top of each beam. Division 3 contains laminations ranked 81-120 and is situated on top of the tensile lamination. Division 4 contains the weakest laminations ranked 121-160 and is located below the top lamination. The mean composite bending stiffness of each beam was calculated to ensure similar properties were observed in each beam.

3.3 Manufacture of Unreinforced and Reinforced Beams

The beams were designed as described in Section 3.2 and manufactured in the Timber Engineering Laboratory at the National University of Ireland, Galway. The timber laminations were conditioned to a temperature of 20 ± 2°C and at a relative humidity of 65 ± 5% before manufacture. To create a secure bond at the timber-timber interface, each lamination was planed in order to create a smooth surface to adhere to, free from irregularities and torn grain in accordance with EN 14080 (CEN 2013b). The planing of the laminations decreased the thickness of each lamination from approximately 34 mm to 31.25 mm on average. This process was scheduled to minimise the time between planing and adhesive application. A short interval prevents unwanted oxidation and increases the infiltration of the adhesive into the exposed surface, which improves mechanical interlocking (Frihart 2009). The adhesive applied is a 1:1 phenol resorcinol formaldehyde adhesive which is fully weatherproof as well as being suitable for structural applications such as glued laminated beams. The adhesive is applied to all interfaces to be bonded, ensuring an even spread of a minimum of 350 g/m². On average 200 g/m² was applied to each face using a rubber roller ensuring complete coverage and an even distribution (Figure 3.4). The beams were clamped in a rig applying a minimum pressure of 0.6 N/mm² in accordance with EN 14080 (CEN 2013b) as seen in Figure 3.3b. The beams were positioned flatwise on the clamping rig. Three beams were placed in the rig with a non-stick spacer between each beam. Two RHS steel sections on either side of the timber beams were used to ensure the load was applied uniformly over the entire surface of the beams.
A torque wrench applied the required pressure through five mechanical clamps spaced equally along the full 2300 mm length of the glued laminated beams. Additional clamps were used to ensure that each beam was positioned correctly and no warping or twisting of any individual lamination or beam was present. The beams remained in the clamping rig for 24 hours to cure, after which they were placed into a conditioning chamber for 5 weeks at $20 \pm 2 ^\circ C$ and at a relative humidity of $65 \pm 5\%$ in order to achieve a timber moisture content of $12\%$ prior to short-term testing. All forty manufactured beams were then tested in accordance with EN 408 (CEN 2014) to determine the bending stiffness as discussed in Section 3.4.
The results from the short-term tests were analysed and twenty glued laminated beams were carefully selected to be reinforced. These twenty beams were selected from the forty beams to create two groups of approximately equal mean stiffness prior to reinforcement. Basalt fibre reinforced polymer (BFRP) rods were chosen as suitable to reinforce Irish-grown timber. As described in Section 2.4.2, BFRP has many advantages, both economically and structurally, over more commonly used fibres such as CFRP and GFRP. Two BFRP rods measuring approximately 12 mm in diameter, were used to reinforce the timber beams.

A circular groove was routed the full length of the bottom tensile lamination, centred 30 mm from each side to house the BFRP rods. A circular groove was chosen to avoid unnecessary stress concentrations experienced in rectangular or square routs (Raftery et al. 2012). The grooves were sized to account for the BFRP rod diameter plus a 2 mm epoxy glue-line (Figure 3.5). Similar to the glue laminating process, the time between routing and adhesive application was minimised to improve adhesion. Each groove was also cleaned with compressed air to ensure it was free from dust and other impurities. A two-part thixotropic structural epoxy adhesive was chosen to bond the reinforcement to the timber as it is specially formulated for the bonding of FRP to timber (Rotafix 2014). The two parts were mixed together from their pre-packaged containers. The combined paste was mixed thoroughly and placed into a cartridge for application using an injection gun. The manufacturers report the values listed in Table 3.1 for the epoxy adhesive.

![Figure 3.5. a) Routing detail and reinforced beam geometry, b) BFRP rod diameter measurement](image)
Table 3.1. FRP/Wood epoxy adhesive data (Rotafix 2014)

<table>
<thead>
<tr>
<th>Compressive Strength (N/mm²)</th>
<th>Tensile Strength (N/mm²)</th>
<th>Tensile Modulus (GPa)</th>
<th>Flexural Strength (N/mm²)</th>
<th>*Bond Strength (N/mm²)</th>
<th>CTE (-60 + 40°C) (°C⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>68</td>
<td>38</td>
<td>3.7</td>
<td>70</td>
<td>~ 6-10</td>
<td>9.2-5</td>
</tr>
</tbody>
</table>

* Bond strength is dependent on quality of the prepared surface and adherend type

---

Figure 3.6. Reinforcing of glued laminated beams

To ensure the correct glue-line thickness is achieved, 2 mm rubber rings are placed at 300 mm centres along the length of the BFRP rod. The adhesive was applied initially to a depth of approximately two-thirds of the routed depth. The BFRP rod was then inserted into the groove forcing excess adhesive around the sides of the rout, up to the top of the rod ensuring complete coverage of the timber-adhesive and rod-adhesive interfaces. Additional adhesive was then applied to the top surface of the rod to complete the 2 mm glue-line. The beams were then placed in the conditioning chamber at a temperature of 20 ± 2 °C and at a relative humidity of 65 ± 5%, where they remained to cure for a period of three weeks. The addition of the two BFRP rods accumulated to a mean percentage reinforcement ratio of 1.85%.
3.4 Short-Term Test Programme

3.4.1 Introduction

In total, forty glued laminated beams were manufactured as described in Section 3.3. The beams consist of four laminations, each measuring approximately 98 mm x 125 mm x 2300 mm and a proportion of these were later reinforced; however, prior to reinforcement, short-term, four-point bending tests were performed to evaluate the global and local stiffness and elastic modulus of each beam in its unreinforced state. Four groups, equal in terms of local elastic modulus and local bending stiffness were then created. This section details the short-term, four-point bending test implemented on all beams in their unreinforced state and the statistical tests used to compare the means of each of the four matched groups.

Two of these matched were subsequently reinforced in the tension zone with two 12 mm BFRP rods as described in Section 3.3. The short-term, four-point bending tests were repeated on all reinforced beams to evaluate the global and local stiffness and elastic modulus of each beam in their reinforced state. The increase in local and global bending stiffness is also presented.

3.4.2 Short-Term Bending Test Set-up

In order to establish the bending stiffness of each unreinforced beam, a four-point bending test in accordance with EN 408 (CEN 2014) was implemented. A Dartec 500kN hydraulic testing machine was employed to induce four-point bending on the beams while two linear variable differential transformers (LVDTs) recorded the local and global deflection values. The local modulus is measured over a gauge length of five times the height of the member \( h \) as shown in Figure 3.7. This is suspended on a hanger beneath the bottom tensile lamination and used to measure the deflection of the neutral axis in the shear-free region located between the loading points. The global modulus is measured over the entire length of the beam and includes the shear component of deflection. Figure 3.8a shows the LVDT used to measure the global modulus of elasticity. This LVDT is externally mounted and positioned at the centre of the beam.
The beams were simply supported in the apparatus with an overhang of 70 mm achieving a span of 2160 mm. Steel plates were placed at a distance of 705 mm from each support under the loading heads to limit the effect of indentation on the timber. The distance between the steel plates was 750 mm which is equal to $6h$. The local modulus hanger was located centrally between these steel plates fixed along the neutral axis at a distance of 625 mm or $5h$, in accordance with EN 408 (CEN 2014). The beams were supported laterally using polytetrafluorethylene (PTFE) strips and packing to counteract lateral torsional buckling during bending as seen in Figure 3.8b. These strips are free to slide over one another so as not to fully restrain the beam at this location (Raftery 2010).

Figure 3.8. a) Global LVDT externally mounted and positioned at the centre of the beam, b) Packing using PTFE strips
The beams were loaded at a constant cross head rate of 0.15 mm/s (< 0.003 x \( h \) limit) to a maximum stroke of 20 mm to ensure that the deflection did not exceed the elastic limit of the beam and that the maximum load was less than 40% of the estimated ultimate failure load. Subsequent to these four-point bending tests, the recorded local and global deflection data was plotted against the applied load to obtain the stiffness values. The resulting increment in displacement \((w_{40\%} - w_{10\%})\) corresponding to the load increment \((F_{40\%} - F_{10\%})\) was substituted into Equations [3.3] and [3.4] to obtain the values for local and global stiffness, respectively.

Local Stiffness \((E_L I)\) = \(\frac{a l_1^2 (F_{40\%} - F_{10\%})}{16(w_{40\%} - w_{10\%})}\)  \[
\text{[3.3]}
\]

Global Stiffness \((E_G I)\) = \(\frac{l^3 (F_{40\%} - F_{10\%})}{12(w_{40\%} - w_{10\%})} \left[ (\frac{3a}{4l}) - \left(\frac{a}{l}\right)^3 \right] \)  \[
\text{[3.4]}
\]

where:

- \(F_{10\%}\) and \(F_{40\%}\) = are the loads corresponding to 10% and 40% of \(F_{\text{max}}\)
- \(w_{10\%}\) and \(w_{40\%}\) = are the displacements corresponding to 10% and 40% of \(F_{\text{max}}\)
- \(l_1\) = the gauge length for the local displacement measurement
- \(l\) = the span between both supports
- \(a\) = the distance between the load head and the nearest support

Figure 3.9. Global and Local modulus apparatus and beam test set-up
3.5 RESULTS

The results of the stiffness testing of the unreinforced beams are presented. Five matched groups were created and subsequently, half of the unreinforced beams were reinforced and retested. The percentage increase in stiffness due the addition of reinforcement was calculated.

3.5.1 Unreinforced Beams

Each beam was tested in accordance with EN 408 (CEN 2014) and the experimental results are reported in Table 3.2. The mean local (\(E_L\)) and global (\(E_G\)) bending stiffness are \(1.432 \times 10^{11}\) Nmm\(^2\) and \(1.312 \times 10^{11}\) Nmm\(^2\), respectively. The local bending stiffness ranges from a minimum of \(1.132 \times 10^{11}\) Nmm\(^2\) to a maximum of \(1.682 \times 10^{11}\) Nmm\(^2\). The global bending stiffness ranges from a minimum of \(1.032 \times 10^{11}\) Nmm\(^2\) to a maximum of \(1.622 \times 10^{11}\) Nmm\(^2\). The elastic modulus has been determined by dividing the bending stiffness by the second moment of inertia of the cross section. The mean local (\(E_L\)) and global (\(E_G\)) elastic modulus are \(9382.0\) N/mm\(^2\) and \(8574.5\) N/mm\(^2\), respectively. The global elastic modulus ranges from a minimum of \(7104.0\) N/mm\(^2\) to a maximum of \(10335.2\) N/mm\(^2\). The local elastic modulus ranges from a minimum of \(7774.1\) N/mm\(^2\) to a maximum of \(12343.8\) N/mm\(^2\). The variations observed between the local and global measurements are not unexpected. Firstly, there is no shear deflection measured in the local measurement due to the position of the local LVDT between the load points in the pure bending zone (Figure 3.7). Also, possible indentation at the supports and load points may be included in the global measurement over the entire span leading to reduced mean values. This low density timber is susceptible to such indentations during testing. Additionally, the inherent variability in timber leads to many defects or abnormalities that may be included in the global measurement and not necessarily in the local measurement.

As described earlier in Section 3.2, the lay-up of each glued laminated beam was carefully selected to achieve forty beams with similar properties. It can be seen from the trend lines in Figure 3.10 and Figure 3.11, that relatively similar beams have been manufactured with regard to bending stiffness.
<table>
<thead>
<tr>
<th>Beam I.D</th>
<th>$E_0$ N/mm²</th>
<th>$E_l$ N/mm²</th>
<th>$E_{gl}$ N/mm²</th>
<th>$E_{gl}$ N/mm²</th>
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<tbody>
<tr>
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</table>

**Mean**  
8574.5  
9382.0  
1.312E+11  
1.432E+11

**Std. Dev.**  
783.1  
1060.8  
1.274E+10  
1.309E+10

**Median**  
8420.9  
9182.0  
1.297E+11  
1.436E+11
In order to analyse the long-term effect of reinforcement and climate conditions on glued laminated beams, it was desirable to create matched groups to examine the long-term performance of such beams as described in Chapter 4. As a result of the deviations observed in the global and local stiffness measurements mentioned previously, the groups were matched based on the local bending stiffness results from all beams in their unreinforced state. The tested beams are sorted into five matched groups as seen in Table 3.3. Four of these matched groups consisting of nine beams each are created for the long-term test programme in Chapter 4. The remaining four beams are used to monitor moisture content change and moisture-induced strains in the variable climate. In this section, the results of each of the matched groups presented and are statistically compared.
to one another to show that no statistically significant difference exists between the mean elastic modulus or mean bending stiffness of any group prior to reinforcement.

Table 3.3. Matched Groups UC, UV, RC, RV and Group MC

For convenience each beam group was named as follows:

- **Group UC** = Un-Reinforced Constant Climate Group
- **Group UV** = Un-Reinforced Variable Climate Group
- **Group RC** = Reinforced Constant Climate Group
- **Group RV** = Reinforced Variable Climate Group
- **Group MC** = 2 Unreinforced and 2 Reinforced Beams for Monitoring Moisture Content and Moisture-induced Strain in a Variable Climate

### 3.5.2.1 Unreinforced Group UC

Group UC consists of nine unreinforced beams and is subject to long-term creep tests in a constant climate as described in Chapter 4. The local and global elastic modulus and bending stiffness results can be seen in Table 3.4 and Figure 3.12. The mean local and
global bending stiffnesses of Group UC are $1.398 \times 10^{11}$ Nmm$^2$ and $1.247 \times 10^{11}$ Nmm$^2$, respectively. The local bending stiffness ranges from a minimum of $1.132 \times 10^{11}$ Nmm$^2$ (Beam 27) to a maximum of $1.583 \times 10^{11}$ Nmm$^2$ (Beam 34) and the global bending stiffness ranges from a minimum of $1.130 \times 10^{11}$ Nmm$^2$ (Beam 40) to a maximum of $1.418 \times 10^{11}$ Nmm$^2$ (Beam 34). The mean local and global elastic modulus are 9268.3 N/mm$^2$ and 8258.3 N/mm$^2$, respectively. The global elastic moduli range from a minimum of 7583.0 N/mm$^2$ (Beam 17) to a maximum of 9693.9 N/mm$^2$ (Beam 40). The local elastic moduli range from a minimum of 7774.1 N/mm$^2$ (Beam 27) to a maximum of 11614.0 N/mm$^2$ (Beam 40).

Table 3.4. Unreinforced Group UC short-term test results

<table>
<thead>
<tr>
<th>Group UC</th>
<th>$E_G$ (N/mm$^2$)</th>
<th>$E_L$ (N/mm$^2$)</th>
<th>$E_GI$ (Nmm$^2$)</th>
<th>$E_LI$ (Nmm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 5</td>
<td>8154.63</td>
<td>9887.54</td>
<td>1.294E+11</td>
<td>1.569E+11</td>
</tr>
<tr>
<td>Beam 16</td>
<td>8291.53</td>
<td>9008.85</td>
<td>1.301E+11</td>
<td>1.413E+11</td>
</tr>
<tr>
<td>Beam 17</td>
<td>7583.00</td>
<td>9063.39</td>
<td>1.239E+11</td>
<td>1.481E+11</td>
</tr>
<tr>
<td>Beam 18</td>
<td>8112.05</td>
<td>8337.75</td>
<td>1.277E+11</td>
<td>1.313E+11</td>
</tr>
<tr>
<td>Beam 27</td>
<td>7878.22</td>
<td>7774.12</td>
<td>1.147E+11</td>
<td>1.132E+11</td>
</tr>
<tr>
<td>Beam 33</td>
<td>8082.59</td>
<td>9289.92</td>
<td>1.157E+11</td>
<td>1.330E+11</td>
</tr>
<tr>
<td>Beam 34</td>
<td>8519.60</td>
<td>9513.09</td>
<td>1.418E+11</td>
<td>1.583E+11</td>
</tr>
<tr>
<td>Beam 35</td>
<td>8099.34</td>
<td>8926.15</td>
<td>1.260E+11</td>
<td>1.405E+11</td>
</tr>
<tr>
<td>Beam 40</td>
<td>9693.85</td>
<td>11613.97</td>
<td>1.130E+11</td>
<td>1.354E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>8258.31</td>
<td>9268.31</td>
<td>1.247E+11</td>
<td>1.398E+11</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>597.46</td>
<td>1076.50</td>
<td>8.65E+09</td>
<td>3.12E+10</td>
</tr>
<tr>
<td>Median</td>
<td>8112.05</td>
<td>9063.39</td>
<td>1.260E+11</td>
<td>1.405E+11</td>
</tr>
</tbody>
</table>

Figure 3.12. Unreinforced Group UC short-term stiffness results
The percentage difference between the combined group mean, consisting of forty unreinforced beams and the Group UC mean, consisting of nine beams, can be seen in Table 3.5. The greatest percentage difference of 5.09% can be seen when comparing the global stiffness between the combined group mean and the Group UC mean. This can be attributed to balancing the groups based on local elastic modulus and local bending stiffness measurements.

Table 3.5. Percentage difference, Group mean vs Group UC

<table>
<thead>
<tr>
<th></th>
<th>Combined Mean</th>
<th>Group UC Mean</th>
<th>Percentage Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Elastic Modulus</td>
<td>8574.5</td>
<td>8258.31</td>
<td>3.76%</td>
</tr>
<tr>
<td>(E_G, N/mm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Elastic Modulus</td>
<td>9382.0</td>
<td>9268.31</td>
<td>1.22%</td>
</tr>
<tr>
<td>(E_L, N/mm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Global Stiffness</td>
<td>1.312E+11</td>
<td>1.247E+11</td>
<td>5.09%</td>
</tr>
<tr>
<td>(E_GI, Nmm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Stiffness</td>
<td>1.432E+11</td>
<td>1.398E+11</td>
<td>2.45%</td>
</tr>
<tr>
<td>(E_LI, Nmm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.5.2.2 Unreinforced Group UV

Group UV consists of nine unreinforced beams and is subject to long-term creep tests in a variable climate as described in Chapter 4. The local and global elastic modulus and bending stiffness results can be seen in Table 3.6 and Figure 3.13. The mean local and global bending stiffness of Group UV are 1.408x10¹¹ Nmm² and 1.314x10¹¹ Nmm², respectively. The local bending stiffness ranges from a minimum of 1.137x10¹¹ Nmm² (Beam 15) to a maximum of 1.648x10¹¹ Nmm² (Beam 21) and the global bending stiffness ranges from a minimum of 1.032x10¹¹ Nmm² (Beam 15) to a maximum of 1.523x10¹¹ Nmm² (Beam 21). The mean local and global elastic modulus are 9177.4 N/mm² and 8579.8 N/mm², respectively. The global elastic moduli range from a minimum of 7104.0 N/mm² (Beam 15) to a maximum of 10335.2 N/mm² (Beam 39). The local elastic moduli range from a minimum of 7824.1 N/mm² (Beam 15) to a maximum of 10546.5 N/mm² (Beam 39).
Table 3.6. Unreinforced Group UV short-term test results

<table>
<thead>
<tr>
<th>Group UV</th>
<th>$E_G$ (N/mm$^2$)</th>
<th>$E_L$ (N/mm$^2$)</th>
<th>$E_GI$ (Nmm$^2$)</th>
<th>$E_LI$ (Nmm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 6</td>
<td>7961.33</td>
<td>8184.22</td>
<td>1.272E+11</td>
<td>1.307E+11</td>
</tr>
<tr>
<td>Beam 9</td>
<td>8545.69</td>
<td>8869.44</td>
<td>1.394E+11</td>
<td>1.447E+11</td>
</tr>
<tr>
<td>Beam 11</td>
<td>9105.61</td>
<td>9573.42</td>
<td>1.316E+11</td>
<td>1.383E+11</td>
</tr>
<tr>
<td>Beam 15</td>
<td>7103.96</td>
<td>7824.07</td>
<td>1.032E+11</td>
<td>1.137E+11</td>
</tr>
<tr>
<td>Beam 21</td>
<td>9369.20</td>
<td>10142.24</td>
<td>1.523E+11</td>
<td>1.648E+11</td>
</tr>
<tr>
<td>Beam 22</td>
<td>8036.47</td>
<td>8990.42</td>
<td>1.282E+11</td>
<td>1.434E+11</td>
</tr>
<tr>
<td>Beam 23</td>
<td>8604.85</td>
<td>9366.37</td>
<td>1.458E+11</td>
<td>1.587E+11</td>
</tr>
<tr>
<td>Beam 29</td>
<td>8156.25</td>
<td>9099.50</td>
<td>1.332E+11</td>
<td>1.486E+11</td>
</tr>
<tr>
<td>Beam 39</td>
<td>10335.19</td>
<td>10546.45</td>
<td>1.218E+11</td>
<td>1.243E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>8579.84</td>
<td>9177.35</td>
<td>1.314E+11</td>
<td>1.408E+11</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>936.62</td>
<td>862.52</td>
<td>1.342E+10</td>
<td>1.525E+10</td>
</tr>
<tr>
<td>Median</td>
<td>8545.69</td>
<td>9099.50</td>
<td>1.316E+11</td>
<td>1.434E+11</td>
</tr>
</tbody>
</table>

Figure 3.13. Unreinforced Group UV bending stiffness results

The percentage difference between the combined group mean, consisting of forty unreinforced beams and the Group UV mean, consisting of nine beams, can be seen in Table 3.7. The greatest percentage difference of 2.21% can be seen when comparing the local elastic modulus between the combined group mean and the UV group mean.
Table 3.7. Percentage difference, Group mean vs Group UV

<table>
<thead>
<tr>
<th></th>
<th>Combined Mean</th>
<th>Group UV Mean</th>
<th>Percentage Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Elastic Modulus (E_G, N/mm²)</td>
<td>8574.5</td>
<td>8579.84</td>
<td>0.06%</td>
</tr>
<tr>
<td>Local Elastic Modulus (E_L, N/mm²)</td>
<td>9382.0</td>
<td>9177.35</td>
<td>2.21%</td>
</tr>
<tr>
<td>Global Stiffness (E_GI, Nmm²)</td>
<td>1.312E+11</td>
<td>1.314E+11</td>
<td>0.14%</td>
</tr>
<tr>
<td>Local Stiffness (E_LI, Nmm²)</td>
<td>1.432E+11</td>
<td>1.408E+11</td>
<td>1.72%</td>
</tr>
</tbody>
</table>

3.5.2.3 Unreinforced Group RC

Group RC consists of nine reinforced beams and is subject to long-term creep tests in a constant climate as described in Chapter 4. The results presented here are in their unreinforced state prior to reinforcement. The local and global elastic modulus and bending stiffness results can be seen in Table 3.8 and Figure 3.14. The mean local and global bending stiffnesses of Group RC are 1.434x10¹¹ Nmm² and 1.314x10¹¹ Nmm², respectively. The local bending stiffness ranges from a minimum of 1.270x10¹¹ Nmm² (Beam 36) to a maximum of 1.669x10¹¹ Nmm² (Beam 7) and the global bending stiffness ranges from a minimum of 1.198x10¹¹ Nmm² (Beam 2) to a maximum of 1.483x10¹¹ Nmm² (Beam 7). The mean local and global elastic modulus are 9176.5 N/mm² and 8402.3 N/mm², respectively. The global elastic moduli range from a minimum of 7430.1 N/mm² (Beam 13) to a maximum of 9571.9 N/mm² (Beam 7). The local elastic moduli range from a minimum of 8130.2 N/mm² (Beam 36) to a maximum of 10772.2 N/mm² (Beam 7).

Table 3.8. Unreinforced Group RC short-term test results

<table>
<thead>
<tr>
<th>Group RC</th>
<th>E_G (N/mm²)</th>
<th>E_L (N/mm²)</th>
<th>E_GI (Nmm²)</th>
<th>E_LI (Nmm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 1</td>
<td>8322.27</td>
<td>9636.16</td>
<td>1.316E+11</td>
<td>1.523E+11</td>
</tr>
<tr>
<td>Beam 2</td>
<td>8275.21</td>
<td>9966.88</td>
<td>1.198E+11</td>
<td>1.443E+11</td>
</tr>
<tr>
<td>Beam 7</td>
<td>9571.91</td>
<td>10772.24</td>
<td>1.483E+11</td>
<td>1.669E+11</td>
</tr>
<tr>
<td>Beam 12</td>
<td>8811.02</td>
<td>8944.79</td>
<td>1.372E+11</td>
<td>1.393E+11</td>
</tr>
<tr>
<td>Beam 13</td>
<td>7430.11</td>
<td>8909.37</td>
<td>1.199E+11</td>
<td>1.438E+11</td>
</tr>
<tr>
<td>Beam 26</td>
<td>8833.51</td>
<td>9375.48</td>
<td>1.422E+11</td>
<td>1.509E+11</td>
</tr>
<tr>
<td>Beam 30</td>
<td>7663.86</td>
<td>8260.94</td>
<td>1.213E+11</td>
<td>1.308E+11</td>
</tr>
<tr>
<td>Beam 32</td>
<td>8565.65</td>
<td>8592.12</td>
<td>1.347E+11</td>
<td>1.351E+11</td>
</tr>
<tr>
<td>Beam 36</td>
<td>8147.09</td>
<td>8130.21</td>
<td>1.273E+11</td>
<td>1.270E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>8402.29</td>
<td>9176.47</td>
<td>1.314E+11</td>
<td>1.434E+11</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>644.77</td>
<td>853.05</td>
<td>9.614E+09</td>
<td>1.156E+10</td>
</tr>
<tr>
<td>Median</td>
<td>8322.27</td>
<td>8944.79</td>
<td>1.316E+11</td>
<td>1.438E+11</td>
</tr>
</tbody>
</table>

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Figure 3.14. Unreinforced Group RC bending stiffness results

The percentage difference between the combined group mean, consisting of forty unreinforced beams and the Group RC mean, consisting of nine beams, can be seen in Table 3.9. The greatest percentage difference of 2.22% can be seen when comparing the local elastic modulus between the combined group mean and the Group RC mean.

Table 3.9. Percentage difference, Group mean vs Group RC prior to reinforcement

<table>
<thead>
<tr>
<th></th>
<th>Combined Mean</th>
<th>Group RC Mean</th>
<th>Percentage Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Elastic Modulus</td>
<td>8574.5</td>
<td>8402.293</td>
<td>2.03%</td>
</tr>
<tr>
<td>(E_G, N/mm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Elastic Modulus</td>
<td>9382.0</td>
<td>9176.466</td>
<td>2.22%</td>
</tr>
<tr>
<td>(E_L, N/mm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Global Stiffness</td>
<td>1.312E+11</td>
<td>1.314E+11</td>
<td>0.11%</td>
</tr>
<tr>
<td>(E_GI, Nmm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Stiffness</td>
<td>1.432E+11</td>
<td>1.434E+11</td>
<td>0.09%</td>
</tr>
<tr>
<td>(E_LI, Nmm²)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.5.2.4 Unreinforced Group RV

Group RV consists of nine reinforced beams and is subject to long-term creep tests in a variable climate as described in Chapter 4. The results presented here are in their unreinforced state prior to reinforcement. The local and global elastic modulus and bending stiffness results can be seen in Table 3.10 and Figure 3.15. The mean local and
The global bending stiffness of Group RV are $1.491 \times 10^{11}$ Nmm$^2$ and $1.400 \times 10^{11}$ Nmm$^2$, respectively. The local bending stiffness ranges from a minimum of $1.332 \times 10^{11}$ Nmm$^2$ (Beam 19) to a maximum of $1.682 \times 10^{11}$ Nmm$^2$ (Beam 14) and the global bending stiffness ranges from a minimum of $1.205 \times 10^{11}$ Nmm$^2$ (Beam 31) to a maximum of $1.622 \times 10^{11}$ Nmm$^2$ (Beam 4). The mean local and global elastic modulus are 9247.1 N/mm$^2$ and 8675.5 N/mm$^2$, respectively. The global elastic moduli range from a minimum of 7716.8 N/mm$^2$ (Beam 31) to a maximum of 9794.0 N/mm$^2$ (Beam 17). The local elastic moduli range from a minimum of 8299.1 N/mm$^2$ (Beam 19) to a maximum of 10499.8 N/mm$^2$ (Beam 14).

Table 3.10. Unreinforced Group RV short-term test results

<table>
<thead>
<tr>
<th>Group RV</th>
<th>$E_G$ (N/mm$^2$)</th>
<th>$E_L$ (N/mm$^2$)</th>
<th>$E_G I$ (Nmm$^2$)</th>
<th>$E_L I$ (Nmm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 3</td>
<td>8846.04</td>
<td>10152.67</td>
<td>1.402E+11</td>
<td>1.610E+11</td>
</tr>
<tr>
<td>Beam 4</td>
<td>9655.29</td>
<td>9637.73</td>
<td>1.622E+11</td>
<td>1.619E+11</td>
</tr>
<tr>
<td>Beam 8</td>
<td>8009.71</td>
<td>9732.45</td>
<td>1.263E+11</td>
<td>1.534E+11</td>
</tr>
<tr>
<td>Beam 10</td>
<td>8807.42</td>
<td>8822.49</td>
<td>1.375E+11</td>
<td>1.377E+11</td>
</tr>
<tr>
<td>Beam 14</td>
<td>9793.98</td>
<td>10499.81</td>
<td>1.569E+11</td>
<td>1.682E+11</td>
</tr>
<tr>
<td>Beam 19</td>
<td>7786.73</td>
<td>8299.13</td>
<td>1.250E+11</td>
<td>1.332E+11</td>
</tr>
<tr>
<td>Beam 24</td>
<td>9187.44</td>
<td>8481.92</td>
<td>1.520E+11</td>
<td>1.403E+11</td>
</tr>
<tr>
<td>Beam 28</td>
<td>8275.87</td>
<td>8922.29</td>
<td>1.397E+11</td>
<td>1.506E+11</td>
</tr>
<tr>
<td>Beam 31</td>
<td>7716.83</td>
<td>8675.21</td>
<td>1.205E+11</td>
<td>1.354E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>8675.48</td>
<td>9247.08</td>
<td>1.400E+11</td>
<td>1.491E+11</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>777.13</td>
<td>780.58</td>
<td>1.385E+10</td>
<td>1.220E+10</td>
</tr>
<tr>
<td>Median</td>
<td>8807.42</td>
<td>8922.29</td>
<td>1.397E+11</td>
<td>1.506E+11</td>
</tr>
</tbody>
</table>

Figure 3.15. Unreinforced Group RV bending stiffness results
The percentage difference between the combined group mean, consisting of forty unreinforced beams and the Group RV mean, consisting of nine beams, can be seen in Table 3.11. The greatest percentage difference of 6.50% can be seen when comparing the global bending stiffness between the combined group mean and the Group RV mean.

### Table 3.11. Percentage difference, Group mean vs Group RV prior to reinforcement

<table>
<thead>
<tr>
<th></th>
<th>Combined Mean</th>
<th>Group RV Mean</th>
<th>Percentage Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Elastic Modulus</td>
<td>8574.5</td>
<td>8675.48</td>
<td>1.17%</td>
</tr>
<tr>
<td>(E&lt;sub&gt;G&lt;/sub&gt;, N/mm&lt;sup&gt;2&lt;/sup&gt;)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Elastic Modulus</td>
<td>9382.0</td>
<td>9247.079</td>
<td>1.45%</td>
</tr>
<tr>
<td>(E&lt;sub&gt;L&lt;/sub&gt;, N/mm&lt;sup&gt;2&lt;/sup&gt;)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Global Stiffness</td>
<td>1.312E+11</td>
<td>1.400E+11</td>
<td>6.50%</td>
</tr>
<tr>
<td>(E&lt;sub&gt;G&lt;/sub&gt;I, Nmm&lt;sup&gt;2&lt;/sup&gt;)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Stiffness</td>
<td>1.432E+11</td>
<td>1.491E+11</td>
<td>4.00%</td>
</tr>
<tr>
<td>(E&lt;sub&gt;L&lt;/sub&gt;I, Nmm&lt;sup&gt;2&lt;/sup&gt;)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 3.5.2.5 Moisture Content Monitoring Group MC

Group MC consists of four beams, two unreinforced and two reinforced. These beams are matched in terms of local bending stiffness to the remaining thirty-six unreinforced glued laminated beams and are used to monitor moisture-induced strains on non-loaded glued laminated beams in a variable climate in Section 4.6.

### Table 3.12. Unreinforced Group MC short-term test results

<table>
<thead>
<tr>
<th>Group MC</th>
<th>E&lt;sub&gt;G&lt;/sub&gt; (N/mm&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>E&lt;sub&gt;L&lt;/sub&gt; (N/mm&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>E&lt;sub&gt;G&lt;/sub&gt;I (Nmm&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>E&lt;sub&gt;L&lt;/sub&gt;I (Nmm&lt;sup&gt;2&lt;/sup&gt;)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 20</td>
<td>8539.8</td>
<td>9264.5</td>
<td>1.333E+11</td>
<td>1.446E+11</td>
</tr>
<tr>
<td>Beam 25</td>
<td>9218.0</td>
<td>9624.9</td>
<td>1.325E+11</td>
<td>1.383E+11</td>
</tr>
<tr>
<td>Beam 37</td>
<td>9667.89</td>
<td>12343.82</td>
<td>1.137E+11</td>
<td>1.452E+11</td>
</tr>
<tr>
<td>Beam 38</td>
<td>10311.95</td>
<td>12224.87</td>
<td>1.217E+11</td>
<td>1.443E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>9434.4</td>
<td>10864.5</td>
<td>1.253E+11</td>
<td>1.431E+11</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>746.5</td>
<td>1646.8</td>
<td>9.352E+09</td>
<td>3.224E+09</td>
</tr>
<tr>
<td>Median</td>
<td>9442.9</td>
<td>10924.9</td>
<td>1.271E+11</td>
<td>1.445E+11</td>
</tr>
</tbody>
</table>

The results presented in Table 3.12 and Table 3.13 are for each beam in its unreinforced state prior to reinforcement. Beam 20 and Beam 25 are subsequently reinforced. In their unreinforced state, the greatest percentage difference can be seen when comparing the local elastic modulus between the combined group mean and the Group MC mean (Table 3.13). This high percentage difference is partially due to the low number of beams in this group and having given priority to the creation of Group UC, UV, RC and RV. This high
percentage difference in the local elastic modulus value is also counteracted by the 0.10% difference between the local bending stiffness of both group means.

Table 3.13. Percentage difference, Group mean vs Group MC prior to reinforcement

<table>
<thead>
<tr>
<th></th>
<th>Combined Mean</th>
<th>Group MC Mean</th>
<th>Percentage Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Elastic Modulus</td>
<td>8574.5</td>
<td>9434.4</td>
<td>9.55%</td>
</tr>
<tr>
<td>( (E_G, \text{ N/mm}^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Elastic Modulus</td>
<td>9382.0</td>
<td>10864.5</td>
<td>14.64%</td>
</tr>
<tr>
<td>( (E_L, \text{ N/mm}^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Global Stiffness</td>
<td>1.312E+11</td>
<td>1.253E+11</td>
<td>4.61%</td>
</tr>
<tr>
<td>( (E_d, \text{ N/mm}^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local Stiffness</td>
<td>1.432E+11</td>
<td>1.431E+11</td>
<td>0.10%</td>
</tr>
<tr>
<td>( (E_l, \text{ N/mm}^2) )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.5.2.6 Unreinforced Group Comparison

As mentioned previously, each group is created and matched based on their unreinforced local stiffness results. The mean group results can be seen graphically in Figure 3.16 and Figure 3.17. Figure 3.16 presents the mean local and global bending stiffness results of each group along with their corresponding standard deviations. Figure 3.17 shows a similar plot associated with the mean local and global elastic modulus and their corresponding standard deviations.
The group means appear to be relatively well balanced in terms of bending stiffness and elastic modulus. To confirm this, statistical Student’s t-tests were carried out to compare the means of each matched group to one another. Shapiro-Wilk tests were performed on each group to assess normality. The null hypothesis of this test assumes the sample is normally distributed and a p-value greater than the chosen significance level, indicates the hypothesis that the data came from a normally distributed sample cannot be rejected. To perform a Student’s t-test, each sample or group being compared should follow a normal distribution. Once normality or a normally distributed sample cannot be rejected, Levene’s test was performed to examine the homogeneity of the group or sample variances. Levene’s test is an inferential test statistic implemented to assess the equality of variances for two or more groups or samples. The null hypothesis of this test assumes the sample variances are equal. If the p-value is greater than the chosen significance level, the null hypothesis is accepted and it is concluded that there is an insignificant difference between the variances of all samples tested. In this study, all statistical tests are carried out to a significance level of 0.95 (α = 0.5). In each sample studied, normality could not be rejected and each group was assumed to follow a normal distribution. When comparing groups using Levene’s test, a proportion of the groups had equal variances and a proportion had unequal variances. For equal variances, Student’s t-test was implemented as it assumes equal variances. In the case of unequal variances, an adapted version of
Student’s t-test known as Welch’s t-test or unequal variances t-test was used to compare the means of both groups.

The results have shown no statistical evidence to suggest that the mean of any group is not equal to any other group in their unreinforced state. This is valid when examining both, mean local elastic modulus (Table 3.14) and mean local bending stiffness (Table 3.15) where the result of the statistical test and the corresponding p-value can be seen.

Table 3.14. Student’s t-test results: Elastic modulus

<table>
<thead>
<tr>
<th>Elastic Modulus</th>
<th>Unreinforced State</th>
<th>Unreinforced State</th>
</tr>
</thead>
<tbody>
<tr>
<td>p-value$\times$Sig.</td>
<td>Group UC</td>
<td>Group UV</td>
</tr>
<tr>
<td>UC</td>
<td>0</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>UV</td>
<td>0.846</td>
<td>0</td>
</tr>
<tr>
<td>RC</td>
<td>0.844</td>
<td>0.998</td>
</tr>
<tr>
<td>RV</td>
<td>0.962</td>
<td>0.860</td>
</tr>
</tbody>
</table>

Table 3.15. Student’s t-test results: Bending stiffness

<table>
<thead>
<tr>
<th>Bending Stiffness</th>
<th>Unreinforced State</th>
<th>Unreinforced State</th>
</tr>
</thead>
<tbody>
<tr>
<td>p-value$\times$Sig.</td>
<td>Group UC</td>
<td>Group UV</td>
</tr>
<tr>
<td>UC</td>
<td>0</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>UV</td>
<td>0.887</td>
<td>0</td>
</tr>
<tr>
<td>RC</td>
<td>0.569</td>
<td>0.709</td>
</tr>
<tr>
<td>RV</td>
<td>0.162</td>
<td>0.249</td>
</tr>
</tbody>
</table>

3.5.3 Reinforced Beams

Once every glued laminated beam had been tested in their unreinforced state, Group RC and Group RV were reinforced as described in Section 3.3. The short-term four-point bending test was performed on all reinforced beams using the same test set-up used to test each beam in its unreinforced state. The local ($E_L$) and global ($E_G$) bending stiffness were determined in accordance with EN 408 (CEN 2014) using Equations [3.3] and [3.4], respectively.

The bending stiffness results of all twenty reinforced beams are presented in Table 3.16. The reinforced local bending stiffness had a mean value of $1.693 \times 10^{11}$ Nmm² with a standard deviation of $0.119 \times 10^{11}$ Nmm² and the global bending stiffness had a mean value of $1.470 \times 10^{11}$ Nmm² a standard deviation of $0.113 \times 10^{11}$ Nmm². The percentage
increase in global and local bending stiffness of each reinforced beam is also presented in Table 3.16.

<table>
<thead>
<tr>
<th>Beam I.D</th>
<th>Reinforced State</th>
<th>Unreinforced State</th>
<th>Percentage Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_I$ (Nmm$^2$)</td>
<td>$E_I$ (Nmm$^2$)</td>
<td>$E_I$ (Nmm$^2$)</td>
</tr>
<tr>
<td>Beam 1</td>
<td>1.447E+11</td>
<td>1.716E+11</td>
<td>1.316E+11</td>
</tr>
<tr>
<td>Beam 2</td>
<td>1.400E+11</td>
<td>1.695E+11</td>
<td>1.198E+11</td>
</tr>
<tr>
<td>Beam 3</td>
<td>1.474E+11</td>
<td>1.757E+11</td>
<td>1.402E+11</td>
</tr>
<tr>
<td>Beam 4</td>
<td>1.668E+11</td>
<td>1.855E+11</td>
<td>1.622E+11</td>
</tr>
<tr>
<td>Beam 7</td>
<td>1.499E+11</td>
<td>1.905E+11</td>
<td>1.483E+11</td>
</tr>
<tr>
<td>Beam 8</td>
<td>1.451E+11</td>
<td>1.866E+11</td>
<td>1.263E+11</td>
</tr>
<tr>
<td>Beam 10</td>
<td>1.569E+11</td>
<td>1.592E+11</td>
<td>1.375E+11</td>
</tr>
<tr>
<td>Beam 12</td>
<td>1.482E+11</td>
<td>1.655E+11</td>
<td>1.372E+11</td>
</tr>
<tr>
<td>Beam 13</td>
<td>1.303E+11</td>
<td>1.668E+11</td>
<td>1.199E+11</td>
</tr>
<tr>
<td>Beam 14</td>
<td>1.699E+11</td>
<td>1.831E+11</td>
<td>1.569E+11</td>
</tr>
<tr>
<td>Beam 19</td>
<td>1.370E+11</td>
<td>1.503E+11</td>
<td>1.250E+11</td>
</tr>
<tr>
<td>Beam 20</td>
<td>1.518E+11</td>
<td>1.661E+11</td>
<td>1.333E+11</td>
</tr>
<tr>
<td>Beam 24</td>
<td>1.565E+11</td>
<td>1.692E+11</td>
<td>1.520E+11</td>
</tr>
<tr>
<td>Beam 25</td>
<td>1.316E+11</td>
<td>1.603E+11</td>
<td>1.325E+11</td>
</tr>
<tr>
<td>Beam 26</td>
<td>1.627E+11</td>
<td>1.810E+11</td>
<td>1.422E+11</td>
</tr>
<tr>
<td>Beam 28</td>
<td>1.435E+11</td>
<td>1.778E+11</td>
<td>1.397E+11</td>
</tr>
<tr>
<td>Beam 30</td>
<td>1.325E+11</td>
<td>1.556E+11</td>
<td>1.213E+11</td>
</tr>
<tr>
<td>Beam 31</td>
<td>1.433E+11</td>
<td>1.564E+11</td>
<td>1.205E+11</td>
</tr>
<tr>
<td>Beam 32</td>
<td>1.355E+11</td>
<td>1.609E+11</td>
<td>1.347E+11</td>
</tr>
</tbody>
</table>

| Mean     | 1.470E+11       | 1.693E+11          | 1.354E+11               | 1.458E+11       | 8.80%          | 16.30%          |
| Std. Dev.| 1.130E+10       | 1.189E+10          | 1.238E+10               | 1.203E+10       | 5.90%          | 3.66%           |
| Median   | 1.453E+11       | 1.680E+11          | 1.340E+11               | 1.440E+11       | 8.94%          | 15.95%          |

There were mean percentage increases in local and global bending stiffness of 16.30% and 8.80%, respectively. The local bending stiffness measurement produced more consistent results with a standard deviation of 3.66% compared to 5.90% for the global bending stiffness measurement. The local percentage increase in stiffness ranges from 8.86% in Beam 14 to 21.61% in Beam 8, whereas the global percentage increase in stiffness ranges from -0.62% in Beam 25 to 19.00% in Beam 31. There was also a slight reduction in the variability observed between beams in their reinforced when compared to the same beams in their unreinforced state. The large variations in the global measurements are thought to be as a result of the inherent variability in timber and shear deflections mentioned previously. Figure 3.18 displays the effect of reinforcement on the short-term stiffness of the twenty timber beams. There is a significant increase in local bending stiffness and an increase to a lesser extent in the global bending stiffness. The standard deviations are plotted with the mean values.
3.5.3.1 Reinforced Group RC

Group RC consists of nine reinforced beams, which are subjected to long-term creep tests in a constant climate as described in Chapter 4. The results presented earlier in Section 3.5.2.3, are of the same beams in their unreinforced state prior to reinforcement. The results presented here examine the mean bending stiffness of all beams in Group RC in their unreinforced and reinforced state. The percentage increase in bending stiffness is also presented.

Table 3.17. Reinforced Group RC: Percentage increase in bending stiffness

<table>
<thead>
<tr>
<th>Group RC</th>
<th>Unreinforced State</th>
<th></th>
<th></th>
<th>Reinf.</th>
<th></th>
<th></th>
<th>State</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th>Percentage Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_{01}$ (Nmm$^2$)</td>
<td>$E_{11}$ (Nmm$^2$)</td>
<td>$E_{01}$ (Nmm$^2$)</td>
<td>$E_{11}$ (Nmm$^2$)</td>
<td>$E_{01}$ (Nmm$^2$)</td>
<td>$E_{11}$ (Nmm$^2$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 1</td>
<td>1.316E+11</td>
<td>1.523E+11</td>
<td>1.447E+11</td>
<td>1.716E+11</td>
<td>9.99%</td>
<td>12.64%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 2</td>
<td>1.198E+11</td>
<td>1.443E+11</td>
<td>1.400E+11</td>
<td>1.695E+11</td>
<td>16.90%</td>
<td>17.52%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 7</td>
<td>1.483E+11</td>
<td>1.669E+11</td>
<td>1.499E+11</td>
<td>1.905E+11</td>
<td>1.11%</td>
<td>14.16%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 12</td>
<td>1.372E+11</td>
<td>1.393E+11</td>
<td>1.482E+11</td>
<td>1.655E+11</td>
<td>7.98%</td>
<td>18.81%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 13</td>
<td>1.199E+11</td>
<td>1.438E+11</td>
<td>1.303E+11</td>
<td>1.668E+11</td>
<td>8.66%</td>
<td>15.98%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 26</td>
<td>1.422E+11</td>
<td>1.509E+11</td>
<td>1.627E+11</td>
<td>1.810E+11</td>
<td>14.43%</td>
<td>19.95%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 30</td>
<td>1.213E+11</td>
<td>1.308E+11</td>
<td>1.325E+11</td>
<td>1.556E+11</td>
<td>9.21%</td>
<td>18.97%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 32</td>
<td>1.347E+11</td>
<td>1.351E+11</td>
<td>1.355E+11</td>
<td>1.609E+11</td>
<td>0.62%</td>
<td>19.15%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>1.314E+11</td>
<td>1.434E+11</td>
<td>1.433E+11</td>
<td>1.684E+11</td>
<td>9.24%</td>
<td>17.62%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>1.020E+10</td>
<td>1.226E+10</td>
<td>1.005E+10</td>
<td>1.169E+10</td>
<td>5.63%</td>
<td>2.84%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Median</td>
<td>1.316E+11</td>
<td>1.438E+11</td>
<td>1.447E+11</td>
<td>1.668E+11</td>
<td>9.21%</td>
<td>18.81%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As can be seen in Table 3.17, the mean local bending stiffness of all nine reinforced beams in Group RC has risen to $1.684 \times 10^{11}$ Nmm$^2$ from $1.434 \times 10^{11}$ Nmm$^2$ in their unreinforced
state representing a percentage increase of 17.62% in bending stiffness. The mean global bending stiffness of all nine reinforced beams in Group RC has risen to $1.433 \times 10^{11}$ Nmm$^2$ from $1.314 \times 10^{11}$ N/mm$^2$ in their unreinforced state representing a percentage increase of 9.24% in bending stiffness. In Figure 3.19, the local and global bending stiffness results of each beam in Group RC in both their unreinforced and reinforced state can be seen.

![Figure 3.19. Reinforced Group RC: Bending stiffness results](image)

### 3.5.3.2 Reinforced Group RV

Group RV consists of nine reinforced beams, which is subject to long-term creep tests in a variable climate as described in Chapter 4. The results presented earlier in Section 3.5.2.4, are the same beams in their unreinforced state prior to reinforcement. The results presented here examine the mean bending stiffness of all beams in Group RV in their unreinforced and reinforced state. The percentage increase in bending stiffness is also presented. As can be seen in Table 3.18, the mean local bending stiffness of all nine reinforced beams in Group RV has risen to $1.715 \times 10^{11}$ Nmm$^2$ from $1.491 \times 10^{11}$ Nmm$^2$ in their unreinforced state representing a percentage increase of 15.19% in bending stiffness. The mean global bending stiffness of all nine reinforced beams in Group RV has risen to $1.518 \times 10^{11}$ Nmm$^2$ from $1.400 \times 10^{11}$ N/mm$^2$ in their unreinforced state representing a percentage increase of 8.84% in bending stiffness. In Figure 3.20, the local and global bending stiffness results of each beam in Group RC in both their unreinforced and reinforced state can be seen.
Table 3.18. Reinforced Group RV: Percentage increase in bending stiffness

<table>
<thead>
<tr>
<th>Group RV</th>
<th>Unreinforced State</th>
<th>Reinforced State</th>
<th>Percentage Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_g I$ (Nmm²)</td>
<td>$E_l I$ (Nmm²)</td>
<td>$E_g I$ (Nmm²)</td>
</tr>
<tr>
<td>Beam 3</td>
<td>1.402E+11</td>
<td>1.610E+11</td>
<td>1.474E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.668E+11</td>
</tr>
<tr>
<td>Beam 4</td>
<td>1.622E+11</td>
<td>1.619E+11</td>
<td>1.474E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.451E+11</td>
</tr>
<tr>
<td>Beam 8</td>
<td>1.263E+11</td>
<td>1.534E+11</td>
<td>1.474E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.569E+11</td>
</tr>
<tr>
<td>Beam 10</td>
<td>1.375E+11</td>
<td>1.377E+11</td>
<td>1.474E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.569E+11</td>
</tr>
<tr>
<td>Beam 14</td>
<td>1.569E+11</td>
<td>1.682E+11</td>
<td>1.474E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.699E+11</td>
</tr>
<tr>
<td>Beam 19</td>
<td>1.250E+11</td>
<td>1.332E+11</td>
<td>1.435E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.370E+11</td>
</tr>
<tr>
<td>Beam 24</td>
<td>1.520E+11</td>
<td>1.403E+11</td>
<td>1.435E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.565E+11</td>
</tr>
<tr>
<td>Beam 28</td>
<td>1.397E+11</td>
<td>1.506E+11</td>
<td>1.435E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.370E+11</td>
</tr>
<tr>
<td>Beam 31</td>
<td>1.205E+11</td>
<td>1.354E+11</td>
<td>1.435E+11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.699E+11</td>
</tr>
</tbody>
</table>

Mean: 1.400E+11 1.491E+11 1.518E+11 1.715E+11 8.84% 15.19%
Std. Dev.: 1.469E+10 1.294E+10 1.131E+10 1.346E+10 6.04% 4.48%
Median: 1.397E+11 1.506E+11 1.474E+11 1.757E+11 8.26% 15.49%

Figure 3.20. Reinforced Group RV: Bending stiffness results

3.5.3.3 Reinforced Group Comparison

The results from Section 3.5.2.6 have shown that there is no statistical difference between the means of each of the matched groups in their unreinforced state. For comparable studies, it is beneficial to examine if the mean bending stiffness of Group RC and RV are similar in their reinforced state. Figure 3.21 presents the mean bending stiffness results of Group UC and Group UV in their unreinforced state and the beam bending stiffness
results of Group RC and Group RV in their unreinforced state and final reinforced state. The corresponding standard deviations are also shown in Figure 3.21.

Figure 3.21. Reinforced and unreinforced group bending stiffness comparison

Statistical Student’s t-tests were carried out to examine the hypothesis that the means of each group were equal to one another. The results have shown that there is no statistical evidence to suggest that mean values of Group UC and Group UV are not equal in terms of bending stiffness as seen in Table 3.19. This is as expected as both of these groups are not reinforced and are matched based on local bending stiffness. These results have been previously presented in Section 3.5.2.6. When analysing the mean values of Group RC and Group RV, in their reinforced state, there is a statistically significant difference between the reinforced group means and the unreinforced group means in each combination studied. This is an effect of the BFRP rod reinforcement. There is, however, no statistical evidence to suggest that the mean results of both reinforced groups, Group RC and Group RV are not equal. This provides a reliable basis for comparison of each reinforced beam group under different environmental climates in Chapter 4.

Table 3.19. Student’s t-test results: Bending stiffness of reinforced vs unreinforced groups

<table>
<thead>
<tr>
<th>Bending Stiffness</th>
<th>Unreinforced State</th>
<th>Reinforced State</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Group UC</td>
<td>Group UV</td>
</tr>
<tr>
<td>p-value\Sig.</td>
<td>0</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>UC</td>
<td>0.887</td>
<td>0</td>
</tr>
<tr>
<td>RV</td>
<td>0.000</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>0.000</td>
<td>0.001</td>
</tr>
</tbody>
</table>
3.6 Summary and Discussion

In this chapter, forty glued laminated beams were manufactured from fast-grown Irish Sitka spruce. Mechanical grading of each lamination and careful lay-up design allowed forty beams of approximately equal properties be created. Half of the beams were reinforced with two 12 mm diameter BFRP rods in the tension zone. These FRP rods were bonded with a two-part thixotropic structural epoxy adhesive into a circular groove routed the entire length of the bottom tensile lamination. The final beams measured approximately 98 x 125 x 2300 mm$^3$.

Short-term four-point bending tests have been carried out on each unreinforced beam manufactured from fast-grown Irish Sitka spruce. Local and global elastic modulus and bending stiffness results were recorded in accordance EN 408. The local deflection measurements were recorded in the pure bending zone between the load heads. This local measurement neglects shear deflection which is measured globally and was deemed the most reliable measure to categorise each beam. The local bending stiffness results were analysed and five matched groups were created, providing a reliable basis for comparable studies on the long-term performance of such beams in constant and variable climates. The five matched groups (Group UC, UV, RC, RV and MC) were shown to have no significant differences between the mean local bending stiffness of each group when compared to any other matched group in their unreinforced state.

Twenty beams were subsequently reinforced with BFRP rods in modest quantities and the bending stiffness of each was determined. The reinforcement was shown to greatly increase the bending stiffness of the glued laminated beams. An average increase in the local bending stiffness of 16.30 % for a moderate percentage reinforcement of 1.85 % was observed. Fast-grown Sitka spruce has been shown to be a suitable donor material to reinforce with FRP material, improving short-term flexural performance. A statistical analysis of two reinforced matched groups, Group RC and Group RV, which consist of nine reinforced glued laminated beams each, has shown there to be no evidence to suggest the means of both groups are not equal. However, there is a statically significant difference between the unreinforced (Group UC and Group UV) and reinforced groups (Group RC and RV) as expected. This provides a reliable basis for comparable studies on the long-term performance of both unreinforced and reinforced beams in a constant and variable climate.
4 LONG-TERM EXPERIMENTAL TESTING

4.1 INTRODUCTION

This chapter describes the creep testing of unreinforced and reinforced glued laminated beams under a constant dead load. The test programme is designed to enable the characterisation of the individual mechanical responses associated with unreinforced and reinforced beams under long-term loading in constant and variable climates. The design and manufacture of two creep test frames, that will be used to apply a constant dead load to the unreinforced and reinforced beams in constant and variable climates over a 75 week period, is presented. The matched groups with statistically equal stiffness properties, created in Chapter 3, are utilised in this chapter to ensure an equal basis for this comparative study. Four of the matched groups are creep tested in constant and variable climates.

In a constant climate, under a constant dead load, each beam group will initially experience an elastic strain due to the application of the dead load followed by viscoelastic creep strain with time. Equation [4.1] describes the total strain experienced in a timber beam when loaded in a constant climate. This allows the viscoelastic component of the unreinforced and reinforced beams to be characterised.

\[
\varepsilon_T = \varepsilon_e + \varepsilon_{ve}
\]  

where:

\(\varepsilon_T\)  = Total measured strain
\(\varepsilon_e\)  = Elastic strain
\(\varepsilon_{ve}\)  = Viscoelastic strain

In comparison, when each beam group is loaded with a constant dead load but simultaneously subjected to a variable climate condition, two additional strain components may be observed, namely, mechano-sorptive creep strain and swelling/shrinkage strains, as seen in Equation [4.2].

\[
\varepsilon_T = \varepsilon_e + \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_s
\]  

where:

\(\varepsilon_{ms}\)  = Mechano-sorptive strain
\(\varepsilon_s\)  = Swelling/shrinkage strain component on matched non-loaded beams
As outlined in the introduction, quantifying the individual swelling/shrinkage component on unreinforced and reinforced beams is essential in order to characterise the mechano-sorptive strain response. Swelling/shrinkage strains are often neglected when studying mechano-sorptive creep effects; however, they may be significant due to the hygroscopic nature of timber and are considered here. This is achieved by subjecting one matched group to the variable relative humidity under a non-loaded condition.

When focusing on the long-term changes in strain with time the initial elastic strain is subtracted from the total strain ($\varepsilon_T$) results and the total creep strain ($\varepsilon_t$) component is described using Equation [4.3].

$$\varepsilon_t = \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_s$$  \[4.3\]

where:

- $\varepsilon_t$ = Total creep strain

The total creep strain ($\varepsilon_t$) will be characterised in the variable climate creep tests results. The viscoelastic creep strain ($\varepsilon_{ve}$) will be characterised in the constant climate creep test results and the swelling/shrinkage creep strain ($\varepsilon_s$) will be characterised on matched non-loaded beams. This methodology will allow the individual mechano-sorptive creep component on unreinforced and reinforced beams to be characterised. As described in Chapter 1, statistical tests will be performed on the matched groups to compare the individual mechanical responses of the unreinforced and reinforced beams in both the constant and variable climates. The swelling/shrinkage test results and experimental creep test results will be used in Chapter 6 to calibrate the coupled moisture-displacement and hygro-mechanical creep model, respectively.

### 4.2 Long-term Test Frame Design and Testing

There have been many examples of different creep test frames and apparatus presented in the literature to date due to the lack of a common test standard. In this study, the short-term test set-up presented in Chapter 3 was in accordance with EN 408 (CEN 2014). This test set-up has been adapted in order to perform long-term tests. The geometric constraints defining span, depth and distance from the support to the load point will be as defined in
EN 408; however, the load applied will be a constant dead load. The test frame, shown in Figure 4.1a, was designed to undertake the long-term creep testing. It has the capability to load eighteen beams at once, six beams on three levels, one above the other. The constant dead load is applied through a lever arm mechanism. The lever arm is adjustable and the load can be varied as required to apply the precise stress level chosen for a given beam (Figure 4.2). A common stress level of 8 MPa, which corresponds to approximately 25% of the ultimate load, was chosen as an appropriate stress level to induce measurable creep deformation in a realistic time without causing failure in the beam. This stress level corresponds to mean loads of 6241 N and 5749 N for reinforced and unreinforced beams, respectively. CADs A3D software was used to model the creep test frame at maximum load with the additional safety factors included (Figure 4.1b).

This test frame is over designed with respect to bending and shear performance as the deflection criterion was critical in the design process. Due to the sensitivity and relatively small deformations associated with creep testing, a strict maximum deflection of 0.5 mm was set for all members regardless of length. As a result, 80 x 80 x 8 mm thick SHS sections were chosen as suitable members which conformed to the defined deflection criterion. The frame was manufactured by a steel fabricator and constructed onsite in the Timber Engineering Laboratory at the National University of Ireland, Galway. As the weight of the lever arm and load head are compulsory components when applying any

![Figure 4.1. a) Sketch of the final test frame design, b) CADs A3D model of the loaded test frame](image-url)
load, the minimum load that can be applied is approximately 2800 N. The additional load is applied in the form of 250 mm x 100 mm x 10 mm steel plates with a 10 mm hole drilled in the centre which slide onto the load hanger as seen in Figure 4.2. The load hanger is capable of holding up to thirty plates in total. The hanger may be moved thereby reducing or increasing the lever arm length depending on the load required.

![Figure 4.2. Lever arm and adjustable load hanger](image)

### 4.2.1 Creep Rig Loading Test

A series of load tests were carried out to ensure the theoretical model of the lever arm agreed with the experimentally determined load. This model calculated the load applied to the beam by taking into account the weight of the load head, the lever arm and the additional weight placed on the lever arm (Load hanger + steel plates) and the distance of this applied weight from the fulcrum of the lever arm.

In this load test, the load was recorded with a 25 kN Sensotec load cell. The weight of the lever arm was first applied to the Sensotec load cell transferred through a steel column (Figure 4.3b). The load hanger was then applied to the lever arm at the test length (1.4 m, 1.5 m, 1.6 m, and 1.7 m). The additional steel plates were then applied one by one up to a total of 16 plates for each test length. An example of one test at a lever arm length of 1.6 m is shown in Figure 4.3a. Figure 4.4a shows the theoretical load for a given number of plates applied at each lever arm test length. Figure 4.4b shows the accuracy of the model when compared to experimental results (Lever arm 1.4 m).
At each lever arm length, the experimental results underestimate the theoretical results; however, the differences are not significant. The experimental and theoretical results at 1.4 m are 96.6% in agreement, at 1.5 m are 96.4% in agreement, at 1.6 m are 96.3% in agreement, and at 1.7 m they are 97.3% in agreement.

Figure 4.3. Creep rig loading test a) Lever arm load test result (Lever arm = 1.6m), b) Lever arm load test set-up

Figure 4.4. a) Theoretical load vs weight in plates for various lever arms, b) Theoretical vs experimental results (Lever arm = 1.4m)
The model developed has been shown to agree well with measured results. This has allowed confident lever arm lengths to be calculated in order to apply a specified load. The glued laminated beams are all loaded to a maximum bending stress of 8 MPa on the compression face. This stress is dependent on the second moment of area of each beam and as a result, each beam requires a different load at a different lever arm length. The frame design allows for such variations in the lever arm length and the load (steel plates) to achieve the required dead load. This frame design also ensures that the load is applied constantly with no deterioration or relaxation of the applied load with time.

4.3 Creep Test Set-up

4.3.1 Introduction

This section defines the loading procedure involved in the long-term creep testing of four matched groups (Group UC, UV, RC and RV) and the bending stress induced in unreinforced and reinforced beams. The instrumentation used within the test programme and the constant and variable climate conditions during the creep tests are presented.

4.3.2 Loading Regime

It became apparent from the literature review that there is no consistent method of applying long-term loads to timber beams nor is there a standard for such a test. This has resulted in many different loading schemes in long-term creep tests. Creep tests have been conducted in three-point bending (Plevris & Triantafillou 1995), four-point bending and a limited number of studies have performed creep tests under uniformly distributed loads (Runyen et al. 2010). The majority of studies have been conducted under a certain percentage stress level of the ultimate failure load; however, these are inconsistent and various percentages of the ultimate failure load have been examined. EN 1156 (CEN 2013a) is the only European code which provides a test set-up for long-term testing on timber products, albeit relating to wood based panels and not structural members. This standard recommends that for the determination of creep factors a load corresponding to 25% of the ultimate failure load shall be used.

From the literature review in Chapter 2, it was decided that a constant stress level in both unreinforced and reinforced beams will demonstrate the effect of FRP reinforcement on both viscoelastic and mechano-sorptive creep behaviour in glued
laminated beams. The load chosen corresponds to approximately 25% of the ultimate load of the unreinforced and reinforced glued laminated beams which will produce measurable creep deflections in a reasonable time scale without causing failure in the specimen. A single maximum compressive bending stress of 8 MPa was chosen. This maximum stress level of 8 MPa is applied on the compression face of each beam which has led to varying loads required for each beam and greater loads in general required on the reinforced beams.

4.3.3 Instrumentation

To examine creep effects in unreinforced and reinforced timber beams, three principal measurements were taken. The vertical deflection is measured with the use of displacement dial gauges at mid-span. Longitudinal tensile and compressive strains are also measured at mid-span with the use of electrical resistance strain gauges and in the variable climate, moisture content is measured at various depths using a moisture probe system.

4.3.3.1 Displacement Dial Gauges

Thirty Mitutoyo displacement dial gauges with an accuracy of 0.01mm were used to measure the vertical global bending deflection of the unreinforced and reinforced beams. Fourteen of the displacement dial gauges were used in the constant climate chamber, seven unreinforced and seven reinforced and the other sixteen displacement dial gauges are used in the variable climate chamber, eight unreinforced and eight reinforced.

![Figure 4.5. Global displacement dial gauge positioned at mid-span](image)

The dial gauges are all positioned at mid-span as shown in Figure 4.5 and set to zero before applying any load. These dial gauges were specifically chosen due to their ability to work at high humidity levels with no risk of damage, deterioration or condensation.
4.3.3.2 Electrical Resistance Strain Gauges

Seventy two 60 mm long, TML type PLW-60-11, electrical resistance strain gauges (ERS) were used to monitor changes in strain on the unreinforced and reinforced beams in the experimental programme. These gauges are measured using a CR1000 Campbell Scientific data acquisition system. Fifty six strain gauges, which are specifically designed for long-term measurement on wood, are adhered at mid-span to the tensile and compressive faces of both unreinforced and reinforced beams in the longitudinal direction. Each gauge is centred on their respective laminations as seen in Figure 4.6. Another sixteen strain gauges were attached to non-loaded beams to monitor strains solely attributed to swelling and shrinkage. These non-loaded reference beams were supported at 200 mm centres on low friction PTFE plates which allowed the beams to swell and shrink freely.

![Figure 4.6. Position of ERS gauges on the tension and compression face of unreinforced and reinforced beams](image)

To secure the gauges correctly, the timber surface was planed and cleaned to ensure no defects or debris would affect the bond. The adhesive used was specially formulated for low-modulus applications such as timber. The two-component adhesive consisted of a main resin component and a hardener component. The weight of the hardener component used was in the order of 2-4% of the main resin component. This adhesive mixture was applied to the surface of the timber over an area slightly wider than the gauge area and was secured on top of the adhesive using a non-stick binder. The binder secured the gauge for 24 hours until the adhesive had cured, after which, the binder was removed. Due to the nature of the long-term testing and variable climate conditions that are present during
the test, it was essential that a protective surface be used to protect the gauges from deterioration. A general purpose neoprene rubber adhesive was used to provide long-term stability and a 3 mm butyl tape placed over the gauge provided moisture and water proofing from the external environments.

**4.3.3.3 Group Instrumentation**

This section describes the allocation of the instrumentation in the beam groups. Group UC contains nine unreinforced glued laminated beams and is subjected to long-term creep tests in a constant climate condition. Deflection and longitudinal strain with time are measured frequently throughout the test period. The moisture content of these beams are assumed to remain constant throughout the test and are not measured due to the constant climate conditions. The instrumentation used in this group and the beam to which it is allocated are listed in Table 4.1.

*Table 4.1. Group UC: Allocated Instrumentation*

<table>
<thead>
<tr>
<th>Group UC</th>
<th>Deflection Measurement</th>
<th>Strain Measurement</th>
<th>Moisture Content Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 5</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 16</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 17</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 18</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 27</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 33</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 34</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 35</td>
<td></td>
<td></td>
<td>✓</td>
</tr>
<tr>
<td>Beam 40</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>

Due to space constraints, not every beam can be monitored using displacement dial gauges and electrical resistance strain gauges. This occurs in each beam group. In Group UC, the vertical deflection is measured on seven beams and the longitudinal strain is measured on seven beams. Five beams have both strain and deflection measurements.

Group UV also contains nine unreinforced glued laminated beams; however, this group is subjected to long-term creep tests in a variable climate condition. In this variable climate, elastic strain and viscoelastic strain with time are measured but, in addition, mechano-sorptive creep strain and swelling/shrinkage strains occur due to the changing moisture content. The instrumentation used to monitor this group and the beam it is allocated to, are listed in Table 4.2. The vertical deflection is measured on eight beams and the longitudinal strain is measured on seven beams. Six beams have both strain and
deflection measurement. Three beams are monitored with moisture probes at various depths to observe the change of moisture content through the cross section of these beams in the variable climate condition. The installation and measurements from these moisture probes will be presented in Section 5.3.2 and Section 5.5.4.

Table 4.2. Group UV: Allocated Instrumentation

<table>
<thead>
<tr>
<th>Group UV</th>
<th>Deflection Measurement</th>
<th>Strain Measurement</th>
<th>Moisture Content Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 6</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 9</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 29</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Beam 11</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 15</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 21</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 22</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Beam 23</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 39</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

Group RC contains nine reinforced glued laminated beams and is subjected to long-term creep tests in a constant climate condition. In this group, elastic strain and viscoelastic strain are measured on reinforced beams. The moisture content of these beams are assumed to remain constant throughout the test and are not measured due to the constant climate conditions.

Table 4.3. Group RC: Allocated Instrumentation

<table>
<thead>
<tr>
<th>Group RC</th>
<th>Deflection Measurement</th>
<th>Strain Measurement</th>
<th>Moisture Content Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 1</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 2</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 7</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 12</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 13</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 26</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 30</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 32</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Beam 36</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>

The instrumentation used on beams in Group RC are listed in Table 4.3. Similar to Group UC, also in a constant climate condition, there are deflection measurements and strain measurements on seven beams in this group and moisture measurement in not required.

Group RV contains nine reinforced glued laminated beams and is subjected to long-term creep tests in a variable climate condition. Similar to Group UV, these beams
undergo creep testing in a variable climate and experience not only an elastic and viscoelastic strain but mechano-sorptive creep strain and swelling/shrinkage strains due to the changing moisture content.

Table 4.4. Group RV: Allocated Instrumentation

<table>
<thead>
<tr>
<th>Group RV</th>
<th>Deflection Measurement</th>
<th>Strain Measurement</th>
<th>Moisture Content Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 3</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Beam 4</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 8</td>
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</tr>
<tr>
<td>Beam 10</td>
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<td></td>
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</tr>
<tr>
<td>Beam 14</td>
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<tr>
<td>Beam 19</td>
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<td></td>
</tr>
<tr>
<td>Beam 24</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Beam 28</td>
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<td></td>
</tr>
<tr>
<td>Beam 31</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The instrumentation used in Group RV and the beam it is allocated to, are listed in Table 4.4. The vertical deflection is measured on eight beams and the longitudinal strain is measured on seven beams. Three beams are also monitored with moisture probes at various depths to observe the change of moisture content through the cross section of these beams in the variable climate.

Group MC contains the remaining four beams. For these beams, moisture-induced strains within non-loaded glued laminated beams due to the swelling/shrinkage of timber in a variable climate are measured. These are discussed later in Section 4.6.

It is also important to note that the bottom surface and end grain of each beam in both climate conditions is coated with a waterproof varnish. This ensures similar surface exposure conditions are experienced by both unreinforced and reinforced beams with 2-dimensional moisture flow through the top surface and both sides of each beam. These exposure conditions are commonly found in construction whereby the end of beams and one face of the beam are not exposed to the external environment.

The benefit of using matched groups allows the total measured strain to be broken down into various strain components. The elastic and viscoelastic strain will be measured in the constant climate condition and the additional mechano-sorptive strain and swelling/shrinkage strain due to moisture content change will be measured in the variable climate. The viscoelastic strain component and the swelling and shrinkage strains, measured on non-loaded beams (Section 4.6 and Section 4.6.2) in the variable climate,
will be subtracted from the total creep strain component ($\varepsilon_t$) to provide an estimate of the mechano-sorptive strain component ($\varepsilon_{ms}$). It has been suggested that the majority of mechano-sorptive creep can be attributed to swelling and shrinkage in the timber being measured (Bengtsson 2000a). The test methodology outlined aims to examine this claim and estimate the magnitude of mechano-sorptive creep not associated with swelling/shrinkage strains. This methodology will be applied to both unreinforced and reinforced beams groups.

### 4.3.4 Climate Conditions

The long-term tests are performed in conditioning chambers under two climate conditions. One of these chambers had a constant relative humidity of 65% ± 5% and a temperature of 20°C ± 2°C throughout. The second variable climate chamber has the ability to set and maintain the relative humidity at any level between 10% and 95% relative humidity with an accuracy of ± 5% and can set and maintain the temperature at any level between 10°C and 40°C with an accuracy of ± 2°C.

![Figure 4.7. Constant climate relative humidity and temperature data](image)

Group UC and Group RC are tested in the constant climate chamber. This chamber remains at a constant relative humidity of 65% ± 5% and a temperature of 20°C ± 2°C.
for the duration of the test. The relative humidity and temperature are monitored constantly and recorded every 15 minutes. The data recorded over the entire test period can be seen in Figure 4.7.

Group UV and Group RV are tested in the variable climate chamber. Figure 4.8 presents the relative humidity and temperature data recorded in this chamber for the entire duration of the test. Similar to the constant climate chamber the climate data was recorded in 15 minute intervals. The climate chambers (Constant and Variable) began at a relative humidity of 65 ± 5% and at a temperature of 20 ± 2°C for the first three weeks, after which, the relative humidity in the variable climate chamber was changed to 90% ± 5%. As a result of this change in the relative humidity, the moisture content of the beams began to change. The relative humidity cycle length of 8 weeks (4 weeks at 90% RH and 4 weeks at 65% RH) continued like this for the duration of the test as seen in Figure 4.8.

![Figure 4.8. Variable climate relative humidity and temperature data](image)

This relative humidity cycle differential and length were chosen to implement a significant moisture change within each timber beam in the variable climate. The small abnormality within the temperature data at 28 weeks can be attributed to a thermostat failure in the conditioning chamber resulting in variations from the set constant temperature of 20°C ± 2°C.
4.3.5 Commissioning Creep Test

Prior to beginning the long-term testing on the unreinforced and reinforced beams, each beam was conditioned to approximately 12% moisture content in the conditioning room at 65 ± 5% relative humidity and at 20°C ± 2°C for a period of 5 weeks. The beams were instrumented and positioned on the creep test frame. The beams were loaded on the 17/2/15. The electrical resistance strain gauges recorded the strain every 5 minutes during the loading procedure and recording continued at this frequency for the first week before reducing the recording frequency. Each beam was loaded individually and the instantaneous deflection was recorded 60 seconds after loading. The beams in both the constant and variable climate chambers remained at a constant 65 ± 5% relative humidity and at 20°C ± 2°C for the first three weeks, after which the climate in the variable climate chamber was adjusted to the 65 ± 5% - 90% ± 5% relative humidity cycle presented earlier. Delaying the variable relative humidity cycle for the first three weeks allowed the relatively rapid viscoelastic deformation, which occurs in the early stages of the test, to be accurately measured. It was beneficial to observe similar deflections across all groups prior to inducing additional deformation due to the variable climate after this period.

4.4 CONSTANT CLIMATE CREEP TEST RESULTS

4.4.1 Introduction

The long-term test results over the 75 week period are presented. Eighteen beams (9 reinforced and 9 unreinforced) were tested in the constant climate at a temperature of 20 ± 2°C and at a constant relative humidity of 65 ± 5%. The global vertical deflection at mid-span has been monitored throughout. The strain has been measured at mid-span in the longitudinal direction on the compression and tension faces.

4.4.2 Deflection Results

The deflection results for each beam in Group UC and Group RC are presented. The results are presented as total deflection (mm) but additionally, in order to focus on the long-term effects, the results are presented using the normalised measure of relative creep. Relative creep is defined as the deflection at time, $t$, over the initial elastic deflection where the initial deflection is defined as the deflection 60 seconds after the load has been applied (Equation [4.4]).
Relative Creep = \frac{w(t)}{w_0} \quad [4.4]

where:
\begin{align*}
w(t) &= \text{deflection at time, } t \\
w_0 &= \text{initial deflection at time zero} \\
t_0 &= \text{time zero is 60 seconds after loading}
\end{align*}

Group UC consists of nine unreinforced beams loaded in four-point bending to a maximum compressive bending stress of 8 MPa. Seven of these beams are monitored with vertical displacement dial gauges. The total deflection results for these beams over the 75 week test period can be seen in Figure 4.9. It can be seen that after the elastic deflection the rate of creep is quite high in all beams. This rate of creep is slowly decreasing with time. Beam 27 and Beam 34 have the highest and lowest initial elastic deformation and long term creep deflection, respectively; however, this is as expected as they have the lowest and highest bending stiffness in Group UC, respectively, measured from short-term bending tests.

Group UC: Deflection results

Group RC consists of nine reinforced beams similarly loaded in four-point bending to a maximum compressive bending stress of 8 MPa. Seven of these beams are also monitored with vertical displacement dial gauges. The total deflection results of each beam over the 75 weeks test period can be seen in Figure 4.10. Similarly to the results on the
unreinforced Group UC, the rate of creep is quite high during the early stages of the test and a creep rate is slowly decreasing with time. Beam 30 and Beam 26 have the highest and lowest initial elastic deformation and long term creep deflection, respectively; however, this is as expected as they have the lowest and highest short-term bending stiffness in Group RC, respectively. In both the unreinforced Group UC and the reinforced Group RC, the total deflection of each beam increases throughout the test period under the constant dead load. As there is no significant change in relative humidity and temperature during this time, the increase in deflection after the elastic deflection may be referred to as the viscoelastic creep component.

![Figure 4.10. Group RC: Deflection results](image)

To compare the long-term deflection results of Group UC and Group RC and to observe the effect of reinforcement on long-term deflections, the mean deflections of each group are shown in Figure 4.11.

A clear difference can be seen between the mean total deformations with time of the unreinforced Group UC and the reinforced Group RC. However, the most significant difference is due to the initial elastic deformation of these beam groups. This difference is as expected due to the difference in bending stiffness between the two groups and the similar bending stress induced in the compression zone of each group.
As a result, it is difficult to make any strong conclusions regarding the long-term deflection of both groups by examining Figure 4.11. For this reason, the mean total deflection and corresponding standard deviations of Group UC and Group RC are plotted in Figure 4.12 at a series of time points over the entire test period. For clarity, the results of Group UC and Group RC in Figure 4.12 at the same time points are offset from one another. To examine the variation in results between both groups, statistical Student’s t-tests were performed at these time points as shown in Table 4.5.
After the initial elastic loading of each beam at week 0, the percentage difference between the total deformation of the unreinforced Group UC and the reinforced Group RC was 9.29%. Student’s t-tests have shown no evidence to suggest the means of these groups are not equal at this point.

Table 4.5. Comparison between the deflection of Group UC and Group RC

<table>
<thead>
<tr>
<th></th>
<th>Week 0</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.697</td>
<td>0.719</td>
<td>0.748</td>
<td>0.754</td>
<td>0.786</td>
<td>0.804</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.654</td>
<td>0.665</td>
<td>0.674</td>
<td>0.681</td>
<td>0.695</td>
<td>0.706</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>9.29%</td>
<td>9.81%</td>
<td>10.16%</td>
<td>10.49%</td>
<td>10.41%</td>
<td>10.50%</td>
<td>10.69%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.1493</td>
<td>0.1032</td>
<td>0.0900</td>
<td>0.0789</td>
<td>0.0800</td>
<td>0.0765</td>
<td>0.0720</td>
</tr>
</tbody>
</table>

It can also be seen that the percentage difference between both groups steadily increases with each week up to a maximum of 10.69% at week 75, however, the Student’s t-tests still shown no evidence of a significant difference between the means of both groups.

To focus on the long-term or creep deflection after the initial elastic deflection the relative creep results as defined in Equation [4.4], are presented. The relative creep values for Group UC are displayed in Figure 4.13. This graph gives a measure of the long-term creep deflection after the initial loading. Consistent creep behaviour can be seen between each beam in Group UC over the 75 week period.
The relative creep values for Group RC are presented in Figure 4.14. The long-term results show similar trends in creep as previously seen in the unreinforced Group UC.

In Figure 4.15 the mean relative creep results of the unreinforced Group UC and the reinforced Group RC are compared. The relative creep results show very similar creep curves between the unreinforced and reinforced beams, loaded under the same bending stress and in the same constant climate condition over the entire test period.

To further examine these creep curves, the mean relative creep deflections and corresponding standard deviations of both groups are plotted in Figure 4.16 at a series of time points over the test period. Again, statistical Student’s t-tests were performed at a series of time points throughout the test which are presented in Table 4.6.
Figure 4.16. Group UC and Group RC: Relative creep deflection and standard deviation results

The percentage difference between the mean results of the unreinforced Group UC and the reinforced Group RC show that this difference is increasing with time. During the initial weeks (Week 0 to Week 19) the trend indicated a significant difference in the creep behaviour of unreinforced and reinforced beams is developing as seen in Table 4.6, however, this period was associated with a relatively high rate of creep deformation and after this point the statistical tests indicated an insignificant difference between the mean results of both groups. Although, after 75 weeks of creep testing, there is a reduction in the total deflection (10.69%) in the reinforced beam Group RC due to the FRP reinforcement, there is less than 1.30% difference between the mean relative creep deflections of both groups at the same time point. A statistical analysis of the group means has shown that there is no statistically significant reduction in viscoelastic creep deflection in FRP reinforced beams when compared to unreinforced beams under similar bending stresses and constant climate conditions.

Table 4.6. Comparison between the relative creep deflection of Group UC and Group RC

<table>
<thead>
<tr>
<th>Group</th>
<th>Week 0</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UC</td>
<td>1.008</td>
<td>1.122</td>
<td>1.166</td>
<td>1.190</td>
<td>1.231</td>
<td>1.263</td>
<td>1.292</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>0.014</td>
<td>0.005</td>
<td>0.007</td>
<td>0.008</td>
<td>0.011</td>
<td>0.012</td>
<td>0.014</td>
</tr>
<tr>
<td>Group RC</td>
<td>1.009</td>
<td>1.117</td>
<td>1.158</td>
<td>1.177</td>
<td>1.219</td>
<td>1.250</td>
<td>1.275</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>0.014</td>
<td>0.006</td>
<td>0.007</td>
<td>0.007</td>
<td>0.010</td>
<td>0.014</td>
<td>0.016</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>0.07%</td>
<td>0.43%</td>
<td>0.76%</td>
<td>1.10%</td>
<td>1.00%</td>
<td>1.08%</td>
<td>1.27%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.9222</td>
<td>0.1330</td>
<td>0.0337</td>
<td>0.0085</td>
<td>0.0560</td>
<td>0.0702</td>
<td>0.0663</td>
</tr>
</tbody>
</table>
4.4.3 Strain Results

The longitudinal tension and compression strain results, monitored using electrical resistance strain gauges are presented. The data from these gauges is logged using a Campbell Scientific CR1000 data acquisition system.

The tension and compression strain gauge results for each beam monitored in the constant climate condition over the 75 week test period are presented in Figure 4.17. There is a total of twenty eight strain gauges (14 tension and 14 compression) presented in this graph. The blue lines represent unreinforced beams (Group UC) and the red lines represent reinforced beams (Group RC). There is a large variation in results over the 75 week period. Three unreinforced beams, in particular, have resulted in large tension strains (Beam 18, Beam 27 and Beam 35) but these beams produced relatively low global stiffness measurements during short-term testing in Chapter 3. The long-term trends become clearer in Figure 4.18 where the mean strain results of each group on the compression and tension face can be seen.

![Figure 4.17. Strain results from all unreinforced (Blue lines) and reinforced (Red lines) beams in the constant climate condition ($\varepsilon_T = \varepsilon_e + \varepsilon_{\alpha}$)](image-url)
The strain results presented on the compression face are similar when comparing both groups, with the reinforced beam Group RC, experiencing slightly less mean longitudinal strain than the unreinforced Group UC. The difference between the strains measured in the tension face is more significant. The reinforced beams in Group RC experience less longitudinal tension strain on average. However, this large difference is as a result of the reinforcement used and its position within the tensile lamination of each reinforced beam. The strain gauge is adhered to the timber surface of the beam situated between the two routed grooves which house the BFRP rod reinforcement. The restraining effect of the BFRP rod reinforcement results in reduced strain in the timber.

To solely examine the viscoelastic strain component, the elastic strain component has been subtracted from the individual strain results of each beam and the mean of these viscoelastic strain results are presented in Figure 4.19. Similar mean viscoelastic strains components are observed on the compression face of both the unreinforced and reinforced beams groups indicating a common bending stress has been applied, as designed in the experimental programme, and as a result, a similar creep rate is observed in both beam groups. In comparison, the mean viscoelastic strains on the tension face show larger mean strain in the unreinforced beam group.
This is again as a result of the reinforcement in the reinforced beam group and its position within the tensile lamination. It is important to note that the controlled climate chamber remained at a constant temperature of 20°C ± 2°C and at a constant relative humidity of 65% ± 5% throughout the duration of the test and any minor fluctuations in strain with time are due to minimal changes in relative humidity and temperature within the tolerances presented. To examine the significance of the observed differences in viscoelastic strain, the mean and standard deviation associated with the viscoelastic strain measurement on the tensile face was examined at a series of time points over the duration of the test. The results can be seen in Figure 4.20. For clarity, these time points are offset from one another. The reinforced beams experience much more consistent viscoelastic behaviour after loading with a greater standard deviation observed within the unreinforced beams of Group UC.
In Table 4.7, a comparison is made between the mean viscoelastic strain component on the tension face of Group UC and Group RC. It can be seen that even after 3 weeks of testing, a percentage difference of 39.84% exists between the viscoelastic strain measured on the tension face of unreinforced and reinforced beams.

Table 4.7. Comparison between the mean viscoelastic strain ($\mu_e$) ($\varepsilon_{ve}$) on the tension face of Group UC and Group RC

<table>
<thead>
<tr>
<th>Tension</th>
<th>Week 3</th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 55</th>
<th>Week 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UC</td>
<td>Mean</td>
<td>71.77</td>
<td>91.79</td>
<td>112.08</td>
<td>140.56</td>
<td>142.33</td>
<td>163.72</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>30.43</td>
<td>38.90</td>
<td>44.44</td>
<td>50.56</td>
<td>56.07</td>
<td>62.00</td>
</tr>
<tr>
<td>Group RC</td>
<td>Mean</td>
<td>47.93</td>
<td>58.02</td>
<td>70.98</td>
<td>85.66</td>
<td>90.56</td>
<td>100.03</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>14.43</td>
<td>17.06</td>
<td>21.29</td>
<td>23.47</td>
<td>27.77</td>
<td>29.30</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>39.84%</td>
<td>45.08%</td>
<td>44.90%</td>
<td>48.54%</td>
<td>44.46%</td>
<td>48.30%</td>
<td>54.33%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.086</td>
<td>0.057</td>
<td>0.048</td>
<td>0.023</td>
<td>0.049</td>
<td>0.030</td>
<td>0.032</td>
</tr>
</tbody>
</table>

Statistical Student’s t-tests have shown that at this point, this percentage difference is not statically significant. The difference is not statistically significant until after week 15 with a percentage difference of 44.90%. This trend continues increasing to a maximum of 54.33% at week 71 as seen in Table 4.7.

To examine the significance of differences in viscoelastic strain on the compression face, the value at a series of time points is examined. The results can be seen in Figure 4.21 and Table 4.8. The mean viscoelastic strain results in Figure 4.21 show
similar trends in both the unreinforced Group UC and the reinforced Group RC beams on the compression face.

Figure 4.21. Mean and standard deviation of viscoelastic strain measurement on the compression face ($\varepsilon_{ve}$)

Again, it can be seen that there is a slightly higher standard deviation associated with the unreinforced Group UC beams. It can be seen in Table 4.8, that the difference between means of each group is not statistically significant at any point throughout the test.

Table 4.8. Comparison between the mean viscoelastic strain ($\mu_{ve}$) ($\varepsilon_{ve}$) on the compression face of Group UC and Group RC

<table>
<thead>
<tr>
<th>Compression</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 67</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UC</td>
<td>Mean</td>
<td>-61.94</td>
<td>-83.69</td>
<td>-69.02</td>
<td>-102.95</td>
<td>-133.48</td>
<td>-147.71</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>31.19</td>
<td>42.60</td>
<td>48.70</td>
<td>60.41</td>
<td>66.80</td>
<td>75.05</td>
</tr>
<tr>
<td>Group RC</td>
<td>Mean</td>
<td>-57.05</td>
<td>-73.42</td>
<td>-61.81</td>
<td>-91.24</td>
<td>-113.66</td>
<td>-119.87</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>20.66</td>
<td>30.39</td>
<td>33.53</td>
<td>42.06</td>
<td>49.66</td>
<td>55.11</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>8.23%</td>
<td>13.07%</td>
<td>11.02%</td>
<td>12.06%</td>
<td>16.03%</td>
<td>20.80%</td>
<td>19.64%</td>
</tr>
<tr>
<td>Student's t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.735</td>
<td>0.613</td>
<td>0.753</td>
<td>0.681</td>
<td>0.541</td>
<td>0.444</td>
<td>0.480</td>
</tr>
</tbody>
</table>

The percentage difference ranges from 8.23% at week 3 to a maximum of 20.80% at week 67. The trend is generally increasing throughout the test; however, there is no evidence to suggest the mean viscoelastic strain measured on the compression face of the unreinforced Group UC and reinforced Group RC is different.
4.5 Variable Climate Creep Test Results

4.5.1 Introduction

The variable climate creep tests follow the same format performed in the constant climate creep test. However, the beams are subjected to a variable relative humidity which adversely affects the rate of creep. Eighteen beams (9 reinforced and 9 unreinforced) are tested under load in a variable climate chamber with a relative humidity ranging from 65 ± 5% to 90 ± 5% in eight week cycles. The vertical deflection is monitored throughout along with strain measured in the longitudinal direction on the compression and tension face similarly to the constant climate creep tests. In addition to viscoelastic creep, Group UV and Group RV experience additional mechano-sorptive creep together with swelling and shrinkage deformations due to variable climate conditions. The results of the variable climate creep tests over a 75 week test period are presented in the following sections.

4.5.2 Deflection Results

The deflection results for each beam in Group UV and Group RV are presented in terms of total deflection (mm) and additionally, to focus on the long-term effects, the results are presented in the normalised measure of relative creep. Group UV consists of nine unreinforced beams loaded in four-point bending to a maximum bending stress of 8 MPa on the compression face. Eight of these beams are monitored with vertical displacement dial gauges. The deflection results for these beams over the 75 week test period can be seen in Figure 4.22. The beams are loaded and subjected to three weeks at a constant climate at a temperature of 20°C ± 2°C and at a relative humidity of 65% ± 5%. It was beneficial to delay cycling the relative humidity for the first three to observe the relatively rapid viscoelastic movement in the earlier stages of the creep test. After three weeks, the variable relative humidity cycle commenced with a change in the relative humidity from 65% ± 5% to 90% ± 5% for four weeks. The cycle continued in this manner alternating between 65% ± 5% and 90% ± 5% every four weeks. The results of the unreinforced Group UV shown in Figure 4.22, demonstrate a dramatic increase in the total deformation due to the variable relative humidity.
Typically there is an increase in the deflection during each drying phase and a decrease in deflection during the wetting phase, with the exception of the first moisture increase to a level not previously attained (Week 3-Week 7), which demonstrated a significant increase in deflection during a wetting phase. The largest increase in total deformation was observed during this first increase in moisture content, followed by a slowly increasing trend with subsequent relative humidity cycles. After 75 weeks, there is a wide variation in the final deflection results of the unreinforced Group UV. The reinforced Group RV consists of nine reinforced beams similarly loaded in four-point bending to a maximum compressive bending stress of 8 MPa on the compression face. Eight of these beams are monitored with vertical displacement dial gauges. The deflection results for these beams over test period can be seen in Figure 4.23. When compared to the deflection results of Group UV, the total deflection results show more consistent deflection behaviour with reduced variation over the 75 week test period. There also appears to be a significant reduction when comparing deflection associated with the first moisture content change (Week 3-Week 7). There is again, a trend of slowly increasing deflection with each moisture cycle similar to that observed in the unreinforced Group UV.
To compare the effect of reinforcement on the long-term deflections, the mean deflection of Group UV and Group RV are plotted together in Figure 4.24. There is a clear difference between the unreinforced Group UV and the reinforced Group RV. A large difference between the two groups develops during the first cycle change (Week 3-Week 7) where the humidity changes from 65% ± 5% to 90% ± 5% after week 3. With additional cycles, the deflection continues to increase in each beam group; however, visually, it is not clear if the margin between unreinforced Group UV and reinforced Group RV is increasing with time.
To examine the differences between both groups, the mean total deflections and corresponding standard deviations of Group UV and Group RV at a series of time points are plotted in Figure 4.25 over the entire 75 week test period. For clarity, these time points are offset from one another. Statistical Student’s t-tests are performed at these time points. The results of this statistical analysis can be seen in Table 4.9.

![Figure 4.25. Group UV and Group RV: Mean deflection and standard deviation results](image)

At week 0 and week 3, the percentage difference between Group UV and Group RV is 8.91% and 9.28%, respectively. This percentage difference at week 0 signifies the difference in mean elastic stiffness between unreinforced and reinforced beams during loading and the percentage difference at week 3 is as a result of viscoelastic creep in the unreinforced and reinforced beam groups prior to any relative humidity change. It can also be seen in Table 4.9, that statistical Student’s t-tests have shown no statistically significant difference between the means of both groups at week 0 and week 3. After this period the relative humidity cycle was initiated. At week 11, after the first relative humidity cycle, the percentage difference between the unreinforced Group UV and the reinforced Group RV beams has increased from 9.28% to 17.26%, resulting in a statistically significant difference between the mean total deflections of both groups after just one relative humidity cycle. It is significant that this occurs during the first moisture content change to a level not previously attained, which is associated with a large increase deflection (Hunt 1982). The large increase in percentage difference, however, indicates that the unreinforced Group UV has experienced a greater change during this period than
the reinforced Group RV. With subsequent relative humidity cycles, the trend shows a slowly increasing percentage difference between both groups.

Table 4.9. Comparison between the deflection of Group UV and Group RV

<table>
<thead>
<tr>
<th>Week 0</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.789</td>
<td>0.829</td>
<td>1.453</td>
<td>1.549</td>
<td>1.684</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.399</td>
<td>0.403</td>
<td>0.581</td>
<td>0.632</td>
<td>0.674</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>8.91%</td>
<td>9.28%</td>
<td>17.26%</td>
<td>17.25%</td>
<td>17.30%</td>
<td>17.47%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.1266</td>
<td>0.0909</td>
<td>0.0182</td>
<td>0.0185</td>
<td>0.0200</td>
<td>0.0203</td>
</tr>
</tbody>
</table>

The difference between the unreinforced Group UV and reinforced Group RV remains statistically significant after this first relative humidity cycle, signifying a significant reduction in the overall deflection as a result of the reinforcement within the reinforced timber beams.

To further examine these results, it is beneficial to plot the relative creep values which, similar to the comparison made between Group UC and Group RC in Section 4.4.2, gives a normalised view of all beams measured. The relative creep results of Group UV calculated using Equation [4.4], are presented in Figure 4.26. This graph provides a measure of the long-term creep deflection after the initial loading. A similar graph for the reinforced beams in Group RV can be seen in Figure 4.27 over the 75 week test period.

Figure 4.26. Group UV: Relative creep results
Figure 4.27. Group RV: Relative creep results

The comparison between the mean relative creep results in Group UV and Group RV can be seen in Figure 4.28. During the first three weeks of the test, the creep behaviour of both groups is similar. The behaviour of both groups change with a change in relative humidity after week 3. Unlike in the constant climate, where similar relative creep curves were observed, there is a significant difference between the relative creep values of the unreinforced Group UV and the reinforced Group RV in a variable climate condition.

Figure 4.28. Group UV and Group RV: Mean relative creep results

When comparing the relative creep results of both groups, the unreinforced Group UV beams appear to be affected to a greater extent than the reinforced Group RV. Similarly
to the results in the total deflection of these groups, the first change in relative humidity appears to have a significant effect on the behaviour of both groups. Interestingly, the reinforced beams in Group RV seem to experience greater mean creep fluctuations with each relative humidity cycle. This is perhaps as a result of the increased ability to recover creep deflection due to the addition of the reinforcement or an effect of the differential swelling/shrinkage on the tension face of reinforced beams.

To examine these results further, the mean relative creep deflections and corresponding standard deviations of Group UV and Group RV are plotted in Figure 4.29 at a series of time points over the entire test period. Statistical Student’s t-tests were performed at a series of these time points shown in Table 4.10.

Similarly to the total deflection results, there is no statically significant difference between Group UV and Group RV up until week 3. However, after the first relative humidity cycle at week 11, a statically significant difference of 8.21% exists between the relative creep deflection of Group UV and Group RV. The percentage difference between the two groups is increasing with time to a maximum of 8.83% after 75 weeks. Statistical Student’s t-tests have shown a statistically significant difference exists between the unreinforced Group UV and reinforced Group RV after the first relative humidity cycle. The difference remains statistically significant throughout the test period demonstrating the beneficial effect of the reinforcement in reducing the creep deformations of the FRP reinforced beams in a variable climate.
Table 4.10. Comparison between the relative creep deflection of Group UV and Group RV

<table>
<thead>
<tr>
<th></th>
<th>Week 0</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UV</td>
<td>Mean</td>
<td>1.004</td>
<td>1.126</td>
<td>1.682</td>
<td>1.788</td>
<td>1.906</td>
<td>1.981</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.017</td>
<td>0.010</td>
<td>0.053</td>
<td>0.059</td>
<td>0.070</td>
<td>0.075</td>
</tr>
<tr>
<td>Group RV</td>
<td>Mean</td>
<td>1.004</td>
<td>1.122</td>
<td>1.549</td>
<td>1.647</td>
<td>1.756</td>
<td>1.822</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.012</td>
<td>0.010</td>
<td>0.035</td>
<td>0.039</td>
<td>0.044</td>
<td>0.046</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>0.01%</td>
<td>0.40%</td>
<td>8.21%</td>
<td>8.20%</td>
<td>8.23%</td>
<td>8.38%</td>
<td>8.83%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.9881</td>
<td>0.3786</td>
<td>0.0000</td>
<td>0.0001</td>
<td>0.0001</td>
<td>0.0002</td>
<td>0.0002</td>
</tr>
</tbody>
</table>

4.5.3 Strain Results

The mid-span longitudinal strain results measured on the tension and compression faces of each instrumented beam in the variable climate condition can be seen Figure 4.30. The beams shown in red indicate reinforced beams (Group RV) and unreinforced beams (Group UV) are represented in blue. The strain gauge adhered to the compression face of the unreinforced, Beam 9, is omitted from the results as the strain gauge failed to record strain after the initial elastic deformation. A wide variation in total strain results can be seen in both the unreinforced Group UV and reinforced Group RV.

![Figure 4.30. Total longitudinal strain results from all unreinforced (Blue lines) and reinforced (Red lines) beams in the variable climate (ε_T = ε_e + ε_ve + ε_ms + ε_s)](image)
The trends become clearer when examining the mean total strain results (Equation [4.2]) on the tension and compression faces of Group UV and Group RV in Figure 4.31. Similar mean elastic strain was observed in both groups after the initial loading period. During the first three weeks of the creep test, the climate remained constant at 65% ± 5% relative humidity and there is a slight increase in longitudinal strain due to the viscoelastic behaviour of the timber. After this period the relative humidity was increased to 90% ± 5% and significant changes in longitudinal strain on the tension and compression face can be observed. In Figure 4.31, the mean strain on the tension face of the unreinforced Group UV is larger than that of the reinforced Group RV over the 75 week test period, although, similarly to the deflection results presented in Section 4.5.2, the largest changes are seen during the first moisture content change. Similarly, on the compression face, the unreinforced Group UV experienced greater mean longitudinal strain over the entire test duration when compared to the reinforced Group RV.

Figure 4.31. Mean longitudinal strain results on the tension and compression side of Group UV and Group RV ($\varepsilon_T = \varepsilon_e + \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_s$)

To solely examine the long-term creep strain component, the elastic strain component has been subtracted from the individual total strain results of each beam and the mean of these total creep strain results (Equation [4.3]) are presented in Figure 4.32. Again during the
first three weeks, similar total creep behaviour is seen on the tension and compression face of both beam groups. Once the relative humidity cycle has commenced, significant increases in strain are observed immediately as mechano-sorptive creep effects and swelling/shrinkage strains occur. In week 3-week 7, there is a significant increase in strain as a result of the combined mechano-sorptive creep component and the swelling component as the moisture content increases. In week 7-week 11, there is a reduction in the strain measured in each case as the relative humidity is changed to 65% ± 5% and the moisture content decreases. This relative humidity cycle continued over the entire 75 week test period and it can be seen that on both the tension and compression face, the unreinforced Group UV experience greater mean total creep strain.

Figure 4.32. Long-term longitudinal creep strain results of Group UV and Group RV on the tension and compression face ($\varepsilon_t = \varepsilon_{sw} + \varepsilon_{ms} + \varepsilon_s$)

To examine the variation in the means of both Group UV and Group RV, the mean total creep strains and corresponding standard deviations on the tension and compression face can be seen in Figure 4.33 and Figure 4.34, respectively. In Figure 4.33, the total creep strains on the tension side of both the unreinforced Group UV and the reinforced Group RV can be seen at a series of time points. The standard deviation is also plotted to compare the significance of the observed differences.
Figure 4.33. Mean and standard deviation of total longitudinal creep strain measurement on the tension face in a variable climate ($\varepsilon_t = \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_s$)

A proportion of these time points are chosen and presented in Table 4.11. All of the points after week 3 are chosen at peak strains associated with the end of a wetting period due to the high 90% ± 5% relative humidity condition. It can be seen that at week 3 a statistically insignificant difference of 3.70% exists between the mean total creep strain of Group UV and Group RV. The largest increase in the percentage difference between Group UV and Group RV occurs after the first wetting cycle at week 7 (27.95%). The total creep strain slowly increases with each cycle and the percentage difference between Group UV and Group RV is continuously increasing up to a maximum of 35.16% at week 71. Statistical Student’s t-tests have shown that after week 31 the difference becomes statistically significant and there is a beneficial reduction in total creep strain on the tension face due to the reinforcement in Group RV.

Table 4.11. Comparison between the mean total longitudinal creep strains ($\varepsilon_t = \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_s$) measured on the tension side of Group UV and Group RV ($\mu$)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Week 3</th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 55</th>
<th>Week 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UV</td>
<td>Mean</td>
<td>58.38</td>
<td>481.81</td>
<td>638.39</td>
<td>765.69</td>
<td>826.40</td>
<td>838.86</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>13.10</td>
<td>186.50</td>
<td>195.62</td>
<td>220.12</td>
<td>226.81</td>
<td>221.02</td>
</tr>
<tr>
<td>Group RV</td>
<td>Mean</td>
<td>56.26</td>
<td>363.64</td>
<td>467.53</td>
<td>546.21</td>
<td>583.22</td>
<td>591.72</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>34.05</td>
<td>115.30</td>
<td>139.84</td>
<td>171.40</td>
<td>182.78</td>
<td>183.12</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>3.70%</td>
<td>27.95%</td>
<td>30.90%</td>
<td>33.46%</td>
<td>34.50%</td>
<td>34.55%</td>
<td>35.16%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
<td>Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.880</td>
<td>0.179</td>
<td>0.085</td>
<td>0.059</td>
<td>0.047</td>
<td>0.042</td>
<td>0.040</td>
</tr>
</tbody>
</table>
In Figure 4.34, the total creep strains on the compression side of both the unreinforced Group UV and the reinforced Group RV can be seen at a series of time points. The standard deviation is also plotted to compare the significance of the observed differences. It can be seen that the unreinforced Group UV experiences greater total creep compression strains on average; however, there are variations associated with these mean measurements.

![Figure 4.34. Mean and standard deviation of total longitudinal creep strain measurement on the compression face in a variable climate (εₜ = εᵥₑ + εₑₑ + εₛ)](image)

A series of these points are chosen for comparison purposes and presented in Table 4.12. After week 3, these points were chosen at peak total creep compression strains associated with the end of a drying period due to the low 65% ± 5% relative humidity condition.

Table 4.12. Comparison between the mean total longitudinal creep strains (εₜ = εᵥₑ + εₑₑ + εₛ) measured on the compression side of Group UV and Group RV (με)

<table>
<thead>
<tr>
<th>Compression</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 67</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group UV</td>
<td>-46.19</td>
<td>-254.75</td>
<td>-285.76</td>
<td>-327.00</td>
<td>-359.75</td>
<td>-369.28</td>
<td>-386.80</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>10.54</td>
<td>114.54</td>
<td>143.65</td>
<td>169.07</td>
<td>194.95</td>
<td>211.52</td>
<td>213.01</td>
</tr>
<tr>
<td>Group RV</td>
<td>-59.62</td>
<td>-166.59</td>
<td>-177.53</td>
<td>-194.14</td>
<td>-204.84</td>
<td>-200.51</td>
<td>-213.78</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>18.53</td>
<td>101.85</td>
<td>119.45</td>
<td>136.35</td>
<td>148.09</td>
<td>150.81</td>
<td>151.46</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>25.38%</td>
<td>41.85%</td>
<td>46.72%</td>
<td>50.99%</td>
<td>54.87%</td>
<td>59.24%</td>
<td>57.62%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.124</td>
<td>0.156</td>
<td>0.154</td>
<td>0.134</td>
<td>0.122</td>
<td>0.114</td>
<td>0.108</td>
</tr>
</tbody>
</table>

Similar to strains measured on the tension face, the greatest increase in total creep strain on the compression face occurs after the first cycle as the percentage difference increases.
from 25.38% at week 3 to 41.85% at week 11. Although the percentage difference which reaches 59.24% at 67 weeks is generally increasing with each cycle, the difference has not been shown to be statically significant. For this reason after a 75 week test period, there is no evidence to suggest a beneficial reduction in total creep strain on the compression face of the reinforced beams.

4.6 SWELLING/SHRINKAGE STRAIN COMPONENT

4.6.1 Introduction

To investigate the dimensional stability in fast-grown glued laminated timber beams, four specimens (Group MC) measuring 98 x 125 x 1000 mm$^3$ were placed in the variable climate chamber to monitor strains development due to changing moisture content. The beams were initially conditioned to approximately 12% moisture content in a controlled climate with a relative humidity of 65% ± 5% and a temperature of 20°C ± 2°C. They were then placed in the variable climate presented in Section 4.3.4.

Each beam was monitored with four PLW-60-11 electrical resistance strain gauges aligned in the longitudinal direction on the top (SG - Top) and bottom (SG - Bottom) face and perpendicular to the grain on the left (SG - Left) and right (SG - Right) side of each beam as seen in Figure 4.35a and Figure 4.35b.

![Figure 4.35. PLW-60-11 ERS gauge, a) PL-60 ERS gauge alignment, b) PLW-60-11 ERS gauge positions](image-url)
The longitudinal strain measured on the top and bottom face of each beam represent the moisture-induced swelling/shrinkage strain component that would occur on the compression and the tension face of the loaded creep test beams, respectively.

*Figure 4.36. Non-loaded beams supported on PTFE plates at 250 mm centres*

For this reason, the swelling/shrinkage strain measured on the bottom face is referred to as the tension face and the swelling/shrinkage strain measured on the top face are referred to as the compression face. The strain was monitored using a Campbell Scientific CR1000 data logger, which recorded the strain in each gauge every 60 minutes. The specimens were supported at 250 mm centres on low-friction PTFE plates to allow swelling and shrinkage of each specimen and minimise strains due to self-weight as seen in Figure 4.36.

The beams used in this experiment are representative of the other beams in their unreinforced and reinforced states. Reinforced beams, Beam 20 and Beam 25, were matched to the 18 creep tested reinforced beams and unreinforced beams, Beam 37 and Beam 38, were matched to the 18 creep tested unreinforced beams in the test programme. These beams in Group MC were matched in terms of local stiffness results by minimising the percentage difference when compared to the average group stiffness from Chapter 3 as seen in Table 4.13. It can be seen that the maximum percentage difference of 4.07% was observed when comparing the mean local stiffness of the reinforced group (Beam 20 and Beam 25) to the mean of the 18 remaining beams in the reinforced group.
Table 4.13. Matched Group MC for Moisture Content and Strain Measurement

<table>
<thead>
<tr>
<th>Beam I.D.</th>
<th>Local Stiffness, $E_{11} \text{ (Nmm}^2\text{)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Group</td>
<td></td>
</tr>
<tr>
<td>Beam 20</td>
<td>1.661E+11</td>
</tr>
<tr>
<td>Beam 25</td>
<td>1.603E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>1.632E+11</td>
</tr>
<tr>
<td>Group Mean</td>
<td>1.700E+11</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>4.07%</td>
</tr>
<tr>
<td>Unreinforced Group</td>
<td></td>
</tr>
<tr>
<td>Beam 37</td>
<td>1.452E+11</td>
</tr>
<tr>
<td>Beam 38</td>
<td>1.443E+11</td>
</tr>
<tr>
<td>Mean</td>
<td>1.447E+11</td>
</tr>
<tr>
<td>Group Mean</td>
<td>1.403E+11</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>3.12%</td>
</tr>
</tbody>
</table>

4.6.2 Swelling/shrinkage Strain Component Results

The measured longitudinal strain on the bottom (tension face) of the beams can be seen in Figure 4.37. The unreinforced specimens, shown in blue, experience higher swelling/shrinkage strain in the longitudinal direction when compared to the reinforced specimens shown in orange. As both unreinforced and reinforced beam specimens are sealed on the bottom face, moisture flow through this tensile laminate is mostly 1-dimensional through the sides. The position of the BFRP rod reinforcement in the tension zone of the reinforced members not only provides a restraining force but also impedes moisture flow and results in a reduction in the measured swelling/shrinkage strains in reinforced members.

![Figure 4.37. Longitudinal strain measurement: Tension face ($\varepsilon_t$)](image)

Figure 4.38 and Figure 4.39 show the moisture-induced swelling/shrinkage strain on the tension face of the reinforced and unreinforced beams, respectively. In Figure 4.38, the
mean swelling/shrinkage strain in the reinforced beams is much less than that observed in Figure 4.39, which presents the average swelling/shrinkage strain in unreinforced beams. It can be seen that the reinforced beams almost completely recover with cycling relative humidity, whereas in unreinforced beams, an increasing trend can be seen in swelling/shrinkage strain on the tension face up until week 39 when the strains stabilise for subsequent moisture cycles.

![Figure 4.38. Longitudinal strain measurement: Reinforced tension face (ε_t)](image1)

To compare the swelling/shrinkage strain on the tension face of unreinforced and reinforced beams, the mean strain results and corresponding standard deviations are compared at a series of time points. These time points correspond to each maximum peak in swelling/shrinkage strain as a result of the cycling relative humidity implemented. These are illustrated graphically in Figure 4.40 and tabulated in Table 4.14. In Figure
4.40, there is no significant difference between the strains measured on unreinforced and reinforced beams after 3 weeks; however, after the first moisture cycle (Week 7), a large difference can be seen between the swelling/shrinkage strains measured on the tension face.

![Chart of Strain Comparison](chart.png)

**Figure 4.40. Strain comparison at peak longitudinal strain values on the tension face (εₜ)**

The percentage difference between the mean strain measurement on the tension face of unreinforced and reinforced beams can be seen in Table 4.14. After the first moisture cycle (Week 7) a percentage difference of 85.45% is observed between strains measured on the tension face of the unreinforced beams and reinforced beams. This percentage difference remains relatively consistent with subsequent cycles achieving percentage differences greater than 100% with a maximum of 106.01% observed at week 71.

**Table 4.14. Percentage difference between peak longitudinal tensile strains (εₜ) observed in unreinforced group and the reinforced group (με)**

<table>
<thead>
<tr>
<th>Tension</th>
<th>Week 0</th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 63</th>
<th>Week 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced Group</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>10.64</td>
<td>297.20</td>
<td>327.78</td>
<td>361.29</td>
<td>384.30</td>
<td>389.32</td>
<td>384.66</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>2.31</td>
<td>124.33</td>
<td>109.71</td>
<td>120.69</td>
<td>120.69</td>
<td>117.90</td>
<td>115.50</td>
</tr>
<tr>
<td>Reinforced Group</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>8.29</td>
<td>119.26</td>
<td>107.38</td>
<td>112.97</td>
<td>119.50</td>
<td>120.10</td>
<td>118.15</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>10.97</td>
<td>34.15</td>
<td>68.70</td>
<td>75.51</td>
<td>85.43</td>
<td>91.40</td>
<td>88.54</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>24.81%</td>
<td>85.45%</td>
<td>101.29%</td>
<td>104.72%</td>
<td>105.12%</td>
<td>105.70%</td>
<td>106.01%</td>
</tr>
</tbody>
</table>

The strain in the longitudinal direction on the top (compression face) of the beams can be seen in Figure 4.41. The top lamination is exposed to the surrounding environment on the top surface and both sides of the lamination. This 2-dimensional flow has resulted in higher swelling/shrinkage strains in the top or compression face. There was no difference
between unreinforced and reinforced beams as each electrical resistance strain gauge is aligned along the top lamination, which is negligibly affected by the reinforcement in the bottom lamination. In Figure 4.41, the measured swelling/shrinkage strain in the longitudinal direction remains at approximately 0 με for the first 3 weeks of the test as the relative humidity remained constant at 65% ± 5%. After this period the relative humidity is cycled between 65% ± 5% to 90% ± 5%.

![Figure 4.41. Strain Measurement: Longitudinal strain on the compression face (εs)](image)

This change in relative humidity causes a sudden increase in strain due to the increasing moisture content of the beams. The opposite is seen as the relative humidity is reduced from 90% ± 5% to 65% ± 5% with a sudden decrease in the swelling/shrinkage strain during the desorption phase of the cycle. After the first cycle, the swelling/shrinkage strain ranges from approximately 100 με to 600 με. It can be seen that once moisture cycling begins the mean swelling/shrinkage strain never falls below 100 με. This is believed to be a result of the moisture content hysteresis of timber in cyclic climates. This hysteresis effect means the magnitude of the moisture gradient during desorption is less than that during the initial adsorption phase. As a result, the longitudinal strain did not recover fully due to the mean increase in moisture content over the entire cross section of the beam due to the cyclic relative humidity.

To compare the moisture-induced swelling/shrinkage strain on the compression face of unreinforced and reinforced beams, the mean strain results and corresponding standard deviations are compared at a series of time points corresponding to the maximum peaks in swelling/shrinkage strain as a result of the variable climate implemented. These
are illustrated graphically in Figure 4.42 and tabulated in Table 4.15. In Figure 4.42, there is no notable difference between the strains measured on unreinforced and reinforced beams after 3 weeks and even after moisture cycling has commenced no significant difference can be seen between the strains measured on the compression face of the unreinforced and reinforced beams.

Figure 4.42. Strain comparison at peak longitudinal strain values on the compression face ($\varepsilon_s$)

The percentage difference between the mean swelling/shrinkage strain measurement on the compression face of unreinforced and reinforced beams can be seen in Table 4.15. It was not possible to perform a statistical test here due to the low sample sizes. After the first moisture cycle (Week 7), a percentage difference of 5.92% is observed between strains measured on the unreinforced beams and reinforced beams. This is the maximum percentage difference observed between the unreinforced and reinforced beams under changing moisture content. The remaining weeks all produce approximately equal swelling/shrinkage strain results as seen in Table 4.15.

Table 4.15. Percentage difference between peak longitudinal strains ($\varepsilon_s$) in the unreinforced group and the reinforced group ($\mu\%$)

<table>
<thead>
<tr>
<th></th>
<th>Week 0</th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 63</th>
<th>Week 73</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unreinforced</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>2.69</td>
<td>512.25</td>
<td>476.32</td>
<td>529.92</td>
<td>544.78</td>
<td>541.90</td>
<td>530.24</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>6.19</td>
<td>88.17</td>
<td>69.96</td>
<td>92.73</td>
<td>83.89</td>
<td>86.03</td>
<td>84.42</td>
</tr>
<tr>
<td><strong>Reinforced</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>5.47</td>
<td>482.78</td>
<td>473.58</td>
<td>541.56</td>
<td>560.27</td>
<td>564.66</td>
<td>548.98</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>6.00</td>
<td>32.53</td>
<td>29.10</td>
<td>59.89</td>
<td>66.14</td>
<td>69.97</td>
<td>64.29</td>
</tr>
<tr>
<td>Percentage</td>
<td>67.97%</td>
<td>5.92%</td>
<td>0.58%</td>
<td>-2.17%</td>
<td>-2.80%</td>
<td>-4.11%</td>
<td>-3.47%</td>
</tr>
<tr>
<td>Difference</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Perpendicular to grain, eight electrical resistance strain gauges were used to measure the moisture-induced swelling/shrinkage strain on the left and right faces of both the unreinforced and reinforced beams. The results from each of these strain gauges can be seen in Figure 4.43. As expected, the swelling/shrinkage strains measured perpendicular to the grain are orders of magnitude greater than that measured in the longitudinal direction. There is also a large variation in the strain results measured perpendicular to the grain. The swelling/shrinkage strain ranges from approximately 0 με to 6500 με during each cycle.

![Perpendicular to Grain](image)

This range of strain measurement can be attributed to the inherent variability in timber affecting material properties through the cross section. The material orientation has also been shown to have a significant effect on swelling/shrinkage strains (Angst & Malo, 2012). Small abnormalities within the measured strains at 15 weeks and 28 weeks can be attributed to a thermostat failure in the conditioning chamber resulting in variations from the set constant temperature of 20°C ± 2°C.

Figure 4.44 and Figure 4.45 present the moisture-induced swelling/shrinkage strains measured on the right and left side of the beams, respectively. As each electrical resistance strain gauge is secured across the middle two laminations and not attached to the bottom lamination, no distinction is made between unreinforced and reinforced beams.
There are variations between the mean swelling/shrinkage strains measured on the right and left side of each beam, but these are deemed to be as a result of the inherent variability within timber and the measured strain on the left and right side of the beams are combined to evaluate the average swelling/shrinkage strains perpendicular to the grain. The combined mean swelling/shrinkage strain measured perpendicular to the grain on the left and right side and corresponding standard deviations at the peak strains of each moisture cycle can be seen in Figure 4.46.
In Table 4.16 the percentage difference in observed maximum swelling/shrinkage strains measured perpendicular to the grain are compared to that measured at the end of the first cycle (week 7) to observe the change in moisture-induced strain with subsequent cycles.

Table 4.16. Percentage difference between peak strains perpendicular to the grain after Week 7 (με)

<table>
<thead>
<tr>
<th>Perpendicular to grain</th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 63</th>
<th>Week 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>4474.34</td>
<td>4270.93</td>
<td>4627.46</td>
<td>4599.86</td>
<td>4455.56</td>
<td>4282.07</td>
</tr>
<tr>
<td>Std. Dev</td>
<td>1091.41</td>
<td>1050.00</td>
<td>1220.15</td>
<td>1271.88</td>
<td>1272.91</td>
<td>1229.33</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>0.00%</td>
<td>-4.65%</td>
<td>3.36%</td>
<td>2.77%</td>
<td>-0.42%</td>
<td>-4.39%</td>
</tr>
</tbody>
</table>

The results show similar swelling/shrinkage strains are observed at the peak strain of each moisture cycle. The maximum deviation from the original mean swelling/shrinkage strain of 4474.3 με at week 7 was observed at week 15 with a percentage difference of -4.65%. The remaining results demonstrated a lower percentage difference indicating the relatively consistent and repeatable strain results with subsequent moisture cycling.

### 4.7 Climate Comparison

#### 4.7.1 Introduction

The performance of unreinforced and reinforced beams have been previously compared in similar climates; however, it is also important to compare the effect of the different climates on the behaviour of both unreinforced and reinforced beams. This section
focuses on the differences between unreinforced beams and reinforced beams when compared to similar beams in different climates.

4.7.2 Deflection Results

4.7.2.1 Unreinforced Beams

The mean deflection results of the unreinforced Groups UC and Group UV can be seen in Figure 4.47. For the first three weeks, all beams were tested at a constant relative humidity of 65% ± 5% and at a temperature of 20°C ± 2°C. The mean results from the first three weeks show good agreement between both groups prior to the change in relative humidity in the variable climate. A significant increase in the deflection can be seen in Group UV due to the first change in relative humidity in the variable climate. Although the largest change in deflection can be seen in the first relative humidity cycle, the variable climate continued to increase the total creep deflection over the 75 week test period.

![Figure 4.47. Group UC and Group UV: Mean deflection results](chart)

To examine the effect of the variable climate on unreinforced glued laminated beams manufactured from fast-grown timber, the mean deflection results and corresponding standard deviations at a series of time points are presented in Figure 4.48.
Figure 4.48. Group UC and Group UV: Mean deflection and standard deviation results

Statistical Student’s t-tests have been performed to compare the means of Group UC and Group UV at a series of these time points. The results of these statistical tests are tabulated in Table 4.17. It can be seen that at week 3, while both beam groups are creep tested in a constant climate, that the difference between both groups is not statistically significant. After the first relative humidity cycle at just week 11, the difference becomes statically significant with a percentage difference of 31.70% as a result of the variable climate. This percentage difference increases with subsequent relative humidity cycles to a maximum percentage difference of 40.30% between unreinforced beams in a constant and variable climate at week 75.

Table 4.17. Comparison between the mean deflection of Group UC and Group UV

<table>
<thead>
<tr>
<th>Week</th>
<th>Group UC</th>
<th>Group UV</th>
<th>Percentage Difference</th>
<th>Student’s t-test</th>
<th>p-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Mean 6.272, Std. Dev 0.697</td>
<td>Mean 5.952, Std. Dev 0.789</td>
<td>5.23% Not Sig.</td>
<td>0.4236</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Mean 6.979, Std. Dev 0.719</td>
<td>Mean 6.674, Std. Dev 0.829</td>
<td>4.47% Not Sig.</td>
<td>0.4632</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Mean 7.254, Std. Dev 0.748</td>
<td>Mean 9.986, Std. Dev 1.453</td>
<td>31.70% Sig.</td>
<td>0.0006</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Mean 7.397, Std. Dev 0.754</td>
<td>Mean 10.619, Std. Dev 1.453</td>
<td>35.76% Sig.</td>
<td>0.0002</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Mean 7.658, Std. Dev 0.786</td>
<td>Mean 11.321, Std. Dev 1.684</td>
<td>38.60% Sig.</td>
<td>0.0002</td>
<td></td>
</tr>
<tr>
<td>51</td>
<td>Mean 7.857, Std. Dev 0.804</td>
<td>Mean 11.769, Std. Dev 1.773</td>
<td>39.87% Sig.</td>
<td>0.0002</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>Mean 8.032, Std. Dev 0.823</td>
<td>Mean 12.086, Std. Dev 1.850</td>
<td>40.30% Sig.</td>
<td>0.0002</td>
<td></td>
</tr>
</tbody>
</table>

4.7.2.2 Reinforced Beams

The mean total deflection of the reinforced Group RC and Group RV can be seen in Figure 4.49. For the first three weeks, all reinforced beams were tested at a constant relative humidity of 65% ± 5% and at a temperature of 20°C ± 2°C. The results from the
first three weeks show good agreement between both groups prior to changing the relative humidity in the variable climate. After the first change in relative humidity a significant increase in the total deflection was observed in Group RV due to this variable climate. The variable climate continued to increase the total creep deflection in reinforced beams over the 75 week test period.

Figure 4.49. Group RC and Group RV: Mean deflection results

To quantify the effect of the variable climate on reinforced glued laminated beams, the mean creep deflection results and corresponding standard deviations are presented in Figure 4.50.

Figure 4.50. Group RC and Group RV: Mean deflection and standard deviation results
It has been shown that the reinforcement in Group RV has a significant effect in reducing the creep deformation when compared to unreinforced beams loaded under similar climate conditions (Group UV). When Group RV is compared to the reinforced beams in a constant climate (Group RC) in Figure 4.50 and Table 4.18, it is clear that a statistically significant difference exists between both groups due to the variable climate.

As was the case for the unreinforced beams, the difference between reinforced beams during the first three weeks is not statistically significant; however, the percentage difference is statistically significant after the first relative humidity cycle. The largest change in the percentage difference occurs during this first relative humidity cycle; however, the percentage difference between both groups is increasing with time to a maximum percentage difference of 33.26% after 75 weeks. This is less than that experienced in unreinforced beams (40.30% in Table 4.17) after the same period.

Table 4.18. Comparison between the deflection of Group RC and Group RV

<table>
<thead>
<tr>
<th>Group</th>
<th>Week 0</th>
<th>Week 3</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.654</td>
<td>0.665</td>
<td>0.674</td>
<td>0.681</td>
<td>0.695</td>
<td>0.706</td>
</tr>
<tr>
<td></td>
<td>Std. Dev.</td>
<td>0.399</td>
<td>0.403</td>
<td>0.581</td>
<td>0.632</td>
<td>0.674</td>
<td>0.709</td>
</tr>
</tbody>
</table>

| Percentage Difference | 4.86% | 3.94% | 24.70% | 29.15% | 31.89% | 33.10% | 33.26% |
| Student’s t-test      | Not Sig. | Not Sig. | Sig. | Sig. | Sig. | Sig. | Sig. | p-Value | 0.3428 | 0.3974 | 0.0001 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |

4.7.3 Strain Results

The total creep strain may be separated into its component parts as described previously in Equation [4.2]. In this section, the initial elastic deformation is subtracted from the strain results from each beam focusing on the long-term additional creep strain after loading. Considering Equation [4.3], the total creep strain in the constant climate comprises a viscoelastic strain component only. The total measured strain in the variable climate includes viscoelastic strain, mechano-sorptive strain and swelling/shrinkage strain components. Through the use of the matched groups created in Chapter 3, it is assumed that the viscoelastic creep measured in the constant climate is comparable with the viscoelastic component that occurs in beams in the variable climate over the same time period. The swelling and shrinkage strain component is separately measured on non-loaded reference beams in the same variable environment in Section 4.6. Therefore under
these assumptions, the mechano-sorptive creep component, which is defined as the additional creep strain, experienced in a member due to simultaneous loading and moisture transfer with the surrounding environment can be estimated using Equation [4.5].

\[ \varepsilon_{ms} = \varepsilon_t - (\varepsilon_{ve} + \varepsilon_s) \]  

\[ [4.5] \]

### 4.7.3.1 Unreinforced Beams

The mean total strain data measured on unreinforced beams is presented in Figure 4.51. The total tension and compression strains from unreinforced beams in Group UC and Group UV are presented over the 75 week test period. In addition to the strains measured on the loaded beams in Group UC and Group UV, the swelling/shrinkage strains measured on the tension and compression faces of non-loaded unreinforced beams are also presented and these represent a significant strain component.

![Figure 4.51. Mean total longitudinal strain (\(\varepsilon_T\)) and swelling/shrinkage strain (\(\varepsilon_s\)) from unreinforced beams in different climates](image)

*Figure 4.51. Mean total longitudinal strain (\(\varepsilon_T\)) and swelling/shrinkage strain (\(\varepsilon_s\)) from unreinforced beams in different climates*
To focus on the total creep strain components, the initial elastic strain is subtracted. The mean viscoelastic strain component is calculated subtracting the initial elastic component from the total results of Group UC in a constant climate. Similarly, in Group UV, the initial elastic component is subtracted, leaving the total creep strain in a variable climate, which is the sum of the viscoelastic strain component, the mechano-sorptive strain component and the swelling/shrinkage strain component. The swelling/shrinkage strain components are measured separately on non-loaded beams. Using the assumptions described previously and implementing Equation [4.5] the mean mechano-sorptive component can be estimated.

In Figure 4.52, the mean individual creep strain components on the tension face of unreinforced beams are presented. The mean mechano-sorptive strain on the tension face of the unreinforced beams is presented over the entire 75 week period. It can be seen that on the tension face the swelling/shrinkage strain component and the mechano-sorptive component contribute significantly to the total creep deformation followed by the less significant viscoelastic creep component. The largest increase in mechano-sorptive strain occurs in the first two relative humidity cycles, after which the mechano-sorptive creep component slowly increases with time.

Figure 4.52. Mean longitudinal creep strain components on the tension face of unreinforced beams ($\varepsilon_{ms} = \varepsilon_t - (\varepsilon_{ve} + \varepsilon_s)$)
In Figure 4.53, the mean individual creep strain components on the compression face of reinforced beams are presented. Although the long-term creep strains on the compression face are negative, there is a significant positive increase in total strain as a result of the swelling/shrinkage strain due to the variable relative humidity. A significant negative mechano-sorptive creep component can be seen in the unreinforced beams. Again the largest increase in mechano-sorptive strain occurs in the first two relative humidity cycles, after which the mechano-sorptive creep component slowly increases with time. There appears to be more recovery with each relative humidity cycle in the mechano-sorptive strain component when compared to that predicted on the tension face of the unreinforced beams; however, the trend shows a slowly increasing compressive mechano-sorptive strain component with time.

Figure 4.53. Mean longitudinal creep strain components on the compression face of unreinforced beams
($\varepsilon_{\text{ms}} = \varepsilon_t - (\varepsilon_{\text{ve}} + \varepsilon_s)$)

4.7.3.2 Reinforced Beams

The mean total strain data measured on reinforced beams is presented in Figure 4.54. The total tension and compression strains from reinforced beams in Group RC and Group RV are presented over the 75 week test period. In addition to the strains measured on the
loaded beams in Group RC and Group RV, the swelling/shrinkage strains measured on the tension and compression face of non-loaded reinforced beams are also presented. The swelling/shrinkage strains measured on the tension face of the reinforced beams are significantly less than those measured on the unreinforced beams as a result of the restraining behaviour of the BFRP rod reinforcement in the tension lamination.

Figure 4.54. Mean total longitudinal strain ($\varepsilon_T$) in different climates and swelling/shrinkage strain ($\varepsilon_s$) from reinforced beams

Similar to the analysis performed on the unreinforced beams, the initial elastic strain is subtracted from all total strain results to focus on the total creep strain with time after loading. The mean viscoelastic strain component is calculated subtracting the initial elastic component from the total results of Group RC in a constant climate. Similarly, in Group RV, the initial elastic component is subtracted, leaving the total creep strain in a variable climate. Using the assumptions described previously and implementing Equation [4.5] the mean mechano-sorptive component is estimated.

In Figure 4.55, the mean individual creep strain components on the tension face of the reinforced beams are presented. The estimated mean mechano-sorptive strain on the tension face of the reinforced beams is presented over the entire 75 week period. The
largest increase in mechano-sorptive strain occurs in the first two relative humidity cycles, after which the mechano-sorptive creep component slowly increases with time. The swelling/shrinkage strain component is significantly less than that observed in the unreinforced beams in Figure 4.52. The mechano-sorptive strain component is significantly larger that the swelling/shrinkage strain component on the tension face of the reinforced beams and there appears to be more recovery with each relative humidity cycle in the mechano-sorptive strain component when compared to that predicted on the tension face of the unreinforced beams.

Figure 4.55. Mean longitudinal creep strain components on the tension face of reinforced beams ($\varepsilon_{\text{ms}} = \varepsilon_t - (\varepsilon_{\text{ve}} + \varepsilon_s)$)

In Figure 4.56, the mean individual creep strain components on the compression face of reinforced beams are presented. Similarly to the unreinforced beams, there is a significant positive increase in strain on the compression face due to the swelling/shrinkage strain as a result of the variable relative humidity cycle. A significant negative mechano-sorptive creep component can be seen on the compression face of the reinforced beams. Again the largest increase in mechano-sorptive strain occurs in the first two relative humidity cycles, after which, the mechano-sorptive creep component slowly increases with time. There appears to be more recovery with each relative humidity cycle in the mechano-
sorptive strain component when compared to that predicted on the tension face of the unreinforced beams; however, the trend shows a slowly increasing compressive mechano-sorptive strain component with time.

![Reinforced - Compression](image)

**Figure 4.56. Mean longitudinal creep strain components on the compression face of reinforced beams**

\[ \varepsilon_{ms} = \varepsilon_t - (\varepsilon_{ve} + \varepsilon_s) \]

### 4.7.3.3 Mechano-sorptive Creep Strain

The mean mechano-sorptive strain components in both unreinforced and reinforced beams have been estimated and plotted in Section 4.7.3.1 and Section 4.7.3.2, respectively. There were significant differences observed in the swelling/shrinkage strain components on the tension face of the unreinforced and reinforced beams. To examine if there are significant differences between the mechano-sorptive strain components on unreinforced and reinforced beams, the standard deviation associated with such estimated values are calculated and a statistical analysis is performed to compare the strain components in both unreinforced and reinforced beams.

When calculating the mean mechano-sorptive strain component, the assumed relationship as shown in Equation [4.5] required values for the mean viscoelastic strains, swelling/shrinkage strains and total creep strains. There is a standard deviation associated
with each of these recorded mean strain components which must be considered when comparing mean mechano-sorptive strain between unreinforced and reinforced beams.

To calculate the standard deviation associated with the mechano-sorptive strain component, the combined variance of the measured strain components is considered in the analysis at various time points throughout the tests. Equation [4.6] was implemented to calculate the combined variance associated with the mechano-sorptive measurement at each time point. Equation [4.6] was adopted to implement the relationship between the mechano-sorptive component and the measured strain components to give Equation [4.7]. This equation assumes the variables are dependent on one another. As each mean strain component is measured on different sets of specimens, they are independent of one another, which, renders the covariance equal to zero when comparing the variables.

\[
\begin{align*}
\text{Var}(X + Y + Z) &= \text{Var}(X) + \text{Var}(Y) + \text{Var}(Z) \\
&\quad + 2\text{Cov}(X,Y) + 2\text{Cov}(X,Z) + 2\text{Cov}(Y,Z) \\
\text{Var}(\varepsilon_r - \varepsilon_w - \varepsilon_s) &= \text{Var}(\varepsilon_r) + \text{Var}(\varepsilon_w) + \text{Var}(\varepsilon_s) \\
&\quad - 2\text{Cov}(\varepsilon_r, \varepsilon_w) - 2\text{Cov}(\varepsilon_r, \varepsilon_s) + 2\text{Cov}(\varepsilon_w, \varepsilon_s)
\end{align*}
\]

In Figure 4.57, the mean mechano-sorptive strain components of the unreinforced Group UV and reinforced Group RV at a series of time points are presented. The standard deviations associated with the mean results calculated show a wide variation. As a result, it is visually difficult to draw any significant conclusions between the mechano-sorptive strain component of the unreinforced Group UV and reinforced Group RV. To compare the difference in means of both groups, a series of statistical Student’s t-tests were performed and the results are presented in Table 4.19.
It can be seen that the percentage difference in mechano-sorptive creep is largest at the beginning of the test with 9.47% at week 11 compared to 2.37% at week 75. The Student’s t-tests have also shown no statistically significant differences between the means of both groups on the tension face.

Table 4.19. Comparison between the mechano-sorptive strain ($\varepsilon_{ms}$) on the tension side of unreinforced and reinforced beams (με)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Week 11</th>
<th>Week 19</th>
<th>Week 35</th>
<th>Week 51</th>
<th>Week 67</th>
<th>Week 75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>200.17</td>
<td>224.12</td>
<td>285.54</td>
<td>304.65</td>
<td>319.95</td>
<td>331.73</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>119.42</td>
<td>129.94</td>
<td>150.96</td>
<td>161.80</td>
<td>170.87</td>
<td>175.26</td>
</tr>
<tr>
<td>Reinforced</td>
<td>220.07</td>
<td>239.88</td>
<td>290.17</td>
<td>307.23</td>
<td>312.93</td>
<td>323.95</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>108.20</td>
<td>132.46</td>
<td>156.39</td>
<td>169.00</td>
<td>174.92</td>
<td>187.34</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>9.47%</td>
<td>6.79%</td>
<td>1.61%</td>
<td>0.84%</td>
<td>2.22%</td>
<td>2.37%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.749</td>
<td>0.826</td>
<td>0.956</td>
<td>0.977</td>
<td>0.941</td>
<td>0.937</td>
</tr>
</tbody>
</table>

In Figure 4.58, the mean mechano-sorptive strain and associated standard deviations on the compression face of the unreinforced Group UV and reinforced Group RV beams are plotted at a series of time points. There appears to be slightly greater mechano-sorptive creep strain on the unreinforced Group UV when compared to Group RV. To further examine this, statistical Student’s t-tests were performed and a series of these are presented in Table 4.20.
At week 7 there is a percentage difference of 8.98% when comparing the mechano-sorptive creep strain on the compression face of Group UV and Group RV. The percentage difference increases throughout the test period to a maximum of 24.36% after 71 weeks. Although there is a large percentage difference between the mean results of the unreinforced and reinforced beam groups, the difference is not statistically significant at any time throughout the test.

Table 4.20. Comparison between the mechano-sorptive strain ($\varepsilon_{ms}$) on the compression side of unreinforced and reinforced beams ($\mu$)

<table>
<thead>
<tr>
<th></th>
<th>Week 7</th>
<th>Week 15</th>
<th>Week 31</th>
<th>Week 47</th>
<th>Week 55</th>
<th>Week 71</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>-303.96</td>
<td>-354.10</td>
<td>-433.09</td>
<td>-443.52</td>
<td>-465.90</td>
<td>-466.21</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>113.25</td>
<td>117.36</td>
<td>155.21</td>
<td>176.59</td>
<td>182.87</td>
<td>194.28</td>
</tr>
<tr>
<td>Reinforced</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>62.27</td>
<td>86.28</td>
<td>108.88</td>
<td>120.87</td>
<td>123.32</td>
<td>124.45</td>
</tr>
<tr>
<td>Percentage Difference</td>
<td>8.98%</td>
<td>14.17%</td>
<td>18.12%</td>
<td>20.86%</td>
<td>21.91%</td>
<td>24.36%</td>
</tr>
<tr>
<td>Student’s t-test</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
<td>Not Sig.</td>
</tr>
<tr>
<td>p-Value</td>
<td>0.603</td>
<td>0.411</td>
<td>0.335</td>
<td>0.321</td>
<td>0.291</td>
<td>0.268</td>
</tr>
</tbody>
</table>

**4.8 SUMMARY AND DISCUSSION**

The long-term creep tests have been successfully implemented on unreinforced and reinforced glued laminated beams manufactured from Irish-grown Sitka spruce. The design and manufacture of two creep test frames has allowed up to thirty six beams
(Eighteen beams in a constant climate and eighteen beams in a variable climate) to be loaded simultaneously through individual lever arms. The model developed to calculate the load applied through each lever arm has been successfully validated with experimental results allowing for precise loads to be applied to each beam. The loads required to apply a maximum bending stress of 8MPa in the compression zone of each beam was calculated based on the short-term bending stiffness results presented in Chapter 3. The number of plates and lever arm length was then accurately calculated for each beam.

In a constant climate, the long-term deflection results have shown an overall decrease in the total deflection (elastic deflection and viscoelastic deflection) of reinforced beams as a result of FRP reinforcement. However, when examining the long-term creep deflections, the statistical analysis of the relative creep results have shown no statistically significant reduction in viscoelastic creep deflection when comparing both the unreinforced and reinforced beam groups loaded to a common compressive bending stress. When examining the longitudinal strain measurement, the long-term viscoelastic strain measured on the compression face of both the unreinforced and reinforced beams have shown quite similar results and no statistically significant difference was observed. This indicates a similar compressive bending stress has been applied to each beam in the test programme as outlined in the project methodology. In comparison, on the tension face, the long-term viscoelastic strain results have shown a statistically significant reduction in viscoelastic strain in the reinforced beam group as a result of the FRP reinforcement in the tension face. This reduction in strain on the tension face is deemed beneficial, however, the primary conclusion taken from the experimental tests in a constant climate is that there is no statistically significant reduction in the relative creep behaviour of reinforced beams when compared to unreinforced beams loaded at a common maximum compressive bending stress.

In the variable climate, the creep tests have shown that reinforcing timber with a material of superior properties has a positive effect on the creep behaviour of timber. The overall mean deflection measured in reinforced beams is less than unreinforced beams, even when additional load is applied to the reinforced beams to induce a common bending stress in the compression zone of the timber. When analysing the mean relative creep result of the unreinforced beams and the reinforced beams in a variable climate, there was a statistically significant reduction in creep deflection in the reinforced beams. Interestingly, this difference was significant after just the first relative humidity cycle.
When examining the strain measurement, the long-term total creep strains measured on the compression face in a variable climate has shown a statistically insignificant difference when comparing unreinforced and reinforced beams, again indicating a similar compressive bending stress has been implemented on all test beams; however, there is a statistically significant reduction in the total creep strain measured on the tension face of reinforced beams when compared to the unreinforced beams.

As described in the project methodology, it was essential to monitor the moisture-induced or swelling/shrinkage strain component on non-loaded beams. These swelling/shrinkage strain measurements were performed on the non-loaded Group MC beams using electrical resistance strain gauges in the longitudinal and perpendicular to grain direction. The most significant differences can be seen when comparing the swelling/shrinkage strains measured on the bottom tension lamination of the unreinforced and reinforced beams. The dimensional stability of the BFRP under changing relative humidity quantified in Chapter 5, and the superior stiffness of the BFRP rod material restrain the timber in the tensile zone resulting in reduced swelling/shrinkage strain in the reinforced beams. Swelling/shrinkage strain measured on the top compression lamination experience more consistent moisture-induced strains in both the unreinforced and reinforced case. Significant strains are also measured in the perpendicular to grain direction on the left and right side of each beam. These strains will be examined and compared to a coupled moisture-displacement numerical model in Chapter 6.

When comparing the deflection results of unreinforced beams in different climates, it can be seen that the climate has a statistically significant effect with a maximum percentage difference of 40.30% at week 75 between unreinforced beams measured in a constant and variable climate. Even in the reinforced case, a maximum percentage difference of 33.26% exists at week 75 between reinforced beams measured in constant and variable climates. In an examination of the various strain components involved when creep testing unreinforced and reinforced beams in different climates, the elastic strain component, the viscoelastic strain component, mechano-sorptive strain component and the swelling/shrinkage strain component were determined for the unreinforced and reinforced beams as designed in the project methodology. Interestingly, a similar mechano-sorptive strain component was observed in both the unreinforced and reinforced beams. It is concluded that the main differences were found when examining the longitudinal swelling/shrinkage strain component of the unreinforced and reinforced
beams. The reduced swelling/shrinkage strain component on the tension face of the reinforced beams indicated an inability for the tensile lamination to swell or shrink as in the unreinforced case due to the FRP rod reinforcement and adhesive, which in turn, prevents the beam from deflecting further, rather than an actual reduction in the mechano-sorptive strain component in reinforced beams. It is concluded that under a common bending stress, the difference between the mechano-sorptive strain component in unreinforced and reinforced beams is statistically insignificant.
5 MATERIAL CHARACTERISATION OF SITKA SPRUCE AND BFRP

5.1 INTRODUCTION

The characterisation of material properties is an essential component when developing any numerical tool. In order to predict the long-term behaviour of unreinforced and reinforced glued laminated timber beams, the characterisation of Irish Sitka spruce timber and BFRP material properties is essential in the development of such a numerical tool. In this chapter, as there is no previous test data available, material characterisation tests are designed to quantify the hygro-mechanical properties of Irish-grown Sitka spruce and the material properties specific to the BFRP reinforcement used in the experimental programme.

The moisture content of any piece of timber is a function of the relative humidity and temperature of the surrounding environment and this relationship is subject to a hysteresis effect or irreversibility during increasing and decreasing moisture content and this hysteresis loop is examined. Timber is also subject to moisture-induced expansion or swelling and shrinkage strains when undergoing moisture content change below fibre saturation point. The magnitude of this swelling/shrinkage is directionally dependent and is experimentally determined in the three material directions, radial, tangential and longitudinal. Additionally, the diffusion behaviour of timber is examined in small clear specimens of Irish Sitka spruce subject to moisture content change. Tests are performed to distinguish between the diffusion behaviour of this timber in the respective material directions. BFRP rods from the same batch used to reinforce the glued laminated beams in Chapter 3 are tested in tension to failure to examine the elastic modulus and ultimate failure load. In addition, due to the variable relative humidity and long-term loading requirements of the experimental test, the swelling/shrinkage characteristics of BFRP rods in a variable climate and the creep behaviour of the BFRP rods are also examined.

These characterisation tests are performed to provide material data for numerical models to accurately simulate the behaviour of the unreinforced and reinforced beams in the experimental test programme. The material properties and the effect of moisture on both timber and reinforcement are examined and implemented into Abaqus finite element software in Chapter 6. A moisture pin system based on a resistance method, similar to most handheld moisture meters is also calibrated. This calibration on smaller specimens
allowed moisture probes to be positioned at various depths within structural size glued laminated beams. These probes which monitored the moisture content distribution in the variable climate with time is compared to a moisture diffusion model in Chapter 6.

## 5.2 Moisture Dependent Material Properties of Sitka Spruce

Wood is a hygroscopic material and will absorb or desorb moisture as required to reach an equilibrium moisture content corresponding to its surrounding environmental conditions. Moisture content is an important parameter affecting the durability, shrinkage/swelling, modulus of elasticity and strength of timber. This section describes a series of experiments designed to monitor the effect of environmental conditions on the moisture content, swelling/shrinkage properties and electrical resistance of fast-grown Sitka spruce. First, experiments are performed on small specimens of Sitka spruce at various levels of relative humidity in a climate controlled room. These specimens are used to

- Monitor the moisture content hysteresis (adsorption/desorption)
- Measure the swelling/shrinkage coefficients
- Describe the diffusion behaviour

Second, an experimental test was conducted on full-scale structural elements. The flow of moisture in glued laminated timber beams due to cycling relative humidity is examined. A calibrated moisture probe system, which is based on an electrical resistance method, was implemented on glued laminated beams at various depths to monitor the moisture content distribution with time.

### 5.2.1 Moisture Content Hysteresis of Fast-grown Sitka spruce

Moisture content in timber is commonly known to depend on the relative humidity and temperature of the surrounding environment. This relationship is also subject to a hysteresis effect or irreversibility during increasing and decreasing moisture content. This effect causes an oscillating closed loop between the upper and lower limits of moisture content as a function of relative humidity (Figure 5.1). The adsorption phase describes the lower limit of moisture content for a given relative humidity and occurs with increasing moisture content. The desorption curve describes the upper limit of moisture content for a given relative humidity and occurs with decreasing moisture content.
In order to quantify the moisture content hysteresis in Irish-grown Sitka spruce, the following experiment was devised. Small specimens were prepared and weighed. Two types of specimens were used; cubes measuring 30 x 30 x 30 mm$^3$ and sticks measuring 20 x 20 x 150 mm$^3$. For both sample types, the moisture content as a function of relative humidity was monitored during the adsorption and desorption phases.

In total, thirty cubes and sixty sticks specimens were manufactured, oven dried and had their mass measured at 0% moisture content in accordance with EN 13183-1 (CEN 2002). They were then placed in a controlled climate room. These specimens were placed on a steel mesh to aid air circulation to all faces as seen in Figure 5.2. The climate room in the Timber Engineering Laboratory at the National University of Ireland, Galway has the capability to control relative humidity and temperature. The relative humidity system ranges from 10% to 95% relative humidity with an accuracy of ± 5% and the temperature system ranges from 10°C to 40°C with an accuracy of ± 2°C. For this experiment, the temperature remains constant throughout at 20°C ± 2°C. The specimens initially oven dried to 0% moisture content and were then placed in a predefined relative humidity where the change in moisture content was monitored with time. The moisture content was then calculated using Equation [5.1].

$$u = \frac{m_u - m_0}{m_0} \times 100 \quad \text{[%]}$$  \[5.1\]

where:

- $u$ = moisture content
- $m_u$ = mass at moisture content, $u$
- $m_0$ = mass at zero percent moisture content or dry mass
The cubes and sticks were weighed three times daily using a mass balance with an accuracy of 0.001g. The specimens were allowed to accumulate moisture until the equilibrium moisture content had been reached for the current relative humidity level. The equilibrium moisture content had been deemed to have been achieved when all specimens had a difference in mass of less than 0.1% from two successive measurements taken 24 hours apart. Once the equilibrium moisture content was achieved, the relative humidity was changed to the next predefined level.

![Figure 5.2. Cube and stick samples in controlled climate chamber](image)

The chosen levels for the adsorption phase of this study were 30%, 50%, 60%, 70%, 80% and 90% relative humidity. The measured equilibrium moisture contents were used to produce the adsorption curve of Irish-grown Sitka spruce. The cubes and sticks were then soaked in water to achieve moisture contents greater than 100% and then allowed to dry at a relative humidity of 90% ± 5% and at a temperature of 20°C ± 2°C. These predefined relative humidity levels were then reversed to observe the desorption curve. Again, once equilibrium moisture content was achieved, the relative humidity was changed to the next predefined level. Combining the adsorption and desorption curves gave the complete moisture content hysteresis curve.

### 5.2.2 Swelling and Shrinkage of Fast-grown Sitka Spruce

Timber is dimensionally stable when its moisture content is above the fibre saturation point. This point occurs at a moisture content approximately between 27-30%. Below this point timber experiences swelling and shrinkage during increasing and decreasing moisture content, respectively. Due to the cylindrically orthotropic nature of timber, the magnitude of this swelling/shrinkage is different in each material direction (longitudinal, radial and tangential) (Harte 2009). The magnitude of these deformations has been shown
to depend on species and growth rate. The 20 x 20 x 150 mm³ sticks, described in Section 5.2.1 and shown in Figure 5.3, were designed to monitor moisture content but also the swelling/shrinkage strains in the longitudinal, tangential and radial directions at each relative humidity level. Over 200 specimens were initially cut and prepared from the same batch of timber used to manufacture the glued laminated beams. Sixty suitable specimens were chosen for the experiment. The criteria that determined the best sixty specimens are as follows:

- The samples must be defect free
- The slope of grain must be 0° to the longitudinal direction
- The tangential curve must be as low as possible for accurate measurement.

![Figure 5.3. Swelling/shrinkage stick specimen](image)

The dimensions of each stick specimen were measured in the longitudinal, tangential and radial directions at 0% moisture content. At each of the relative humidity levels described in Section 5.2.1, the longitudinal, tangential and radial dimensions of each specimen were measured when the equilibrium moisture content was achieved. Equation [5.2] was used to calculate the strain. This was calculated for each material direction.

\[ \epsilon_u = \frac{\Delta L}{L_o} \times 100 = \frac{L_u - L_o}{L_o} \times 100 \quad [\%] \]

where:

- \( \epsilon_u \) = total strain at moisture content, \( u \)
- \( L_u \) = length at moisture content, \( u \)
- \( L_0 \) = length at 0% moisture content
Equation [5.3] describes how the swelling/shrinkage coefficient is calculated.

\[ \alpha = \frac{\Delta \varepsilon}{\Delta u} \ \text{[\% / \%]} \]  
\[ \{5.3\} \]

where:
- \( \alpha \) = swelling/shrinkage coefficient
- \( \Delta \varepsilon \) = change in total strain
- \( \Delta u \) = change in moisture content

5.2.3 Diffusion Behaviour of Fast-grown Sitka Spruce

The diffusion of moisture in timber has been adequately described using Fick’s second law and for this reason, this diffusion behaviour is often referred to as Fickian behaviour. Two parameters govern this Fickian behaviour, namely the moisture concentration at the boundary and the diffusion coefficients of the timber.

In order to determine the appropriate diffusion coefficients for Irish Sitka spruce, a 20-week long experiment was undertaken to observe the diffusion behaviour under changing moisture content. These diffusion coefficients are also dependent on material direction. The radial and tangential directions are often comparable and are assumed equal in this study. However, the longitudinal diffusion coefficient is significantly larger than that in the radial and tangential directions. With this directional dependency in mind, three matched groups, each containing ten, 30 x 30 x 30 mm\(^3\) cubes, were placed in a climate chamber where the relative humidity was stepped between 65\% ± 5\% and 90\% ± 5\% relative humidity. The cubes were initially conditioned for four weeks at 65\% ± 5\% relative humidity followed by two, eight week cycles from 65\% ± 5\% relative humidity to 90\% ± 5\% relative humidity measuring the moisture content three times daily. The temperature remained constant at 20°C ± 2°C.

The three matched groups were designed to distinguish between the radial/tangential diffusion coefficients (assumed to be equal) and the longitudinal diffusion coefficients for Sitka spruce. Group 1 was the “uncoated” group which was free to transfer moisture through every surface. Group 2 was the “coated” group which was sealed in the longitudinal direction, only allowing moisture to travel through the radial and tangential surfaces. Group 3 was the “mixed” group which was “uncoated” for the
initial conditioning period and the first relative humidity cycle and was then sealed for the second relative humidity cycle.

5.3 **MOISTURE FLOW IN STRUCTURAL GLUED LAMINATED BEAMS**

5.3.1 **Introduction**

In Chapter 4, the swelling/shrinkage strains on non-loaded beams subjected to a variable climate have been presented. The development of such swelling/shrinkage strains is dependent on the movement of moisture through the cross section of a given element. This section describes an experiment designed to monitor the flow of moisture in structural sized, glued laminated beams manufactured from Irish-grown Sitka spruce in a variable climate. Structural sized glued laminated beams are instrumented with calibrated moisture probes to measure moisture content with time at specified depths through the cross-section over a 75 week period. The results will be used in conjunction with a numerical diffusion model, described in Chapter 6, to determine appropriate diffusion coefficients for Sitka spruce.

5.3.2 **Measurement of Moisture Content Flow in Timber**

In total, six structural size glued laminated beams, 2300 mm long with cross-sectional dimensions 98 mm x 125 mm and comprising four laminations bonded using a PRF adhesive were instrumented with calibrated moisture probes. Each moisture probe comprises two pins inserted perpendicular to the grain. The body of the pins are teflon insulated with an exposed steel tip allowing moisture content to be measured at a specified depth within the timber. The calibration of these probes is presented in Appendix C. The probes were strategically placed at various depths to measure the change in moisture content with time in a variable climate. The eleven probe positions (MC1-MC11) can be seen in Figure 5.4a. These strategic positions were chosen to help study the moisture content distribution through the cross-section of the beams in a variable climate and examine the effect of the glue-line on moisture flow within an element. The pin spacing was consistent with that of the probe calibration test, described in Appendix C and the probes were also placed perpendicular to the grain (Figure 5.4b). A total of forty-four probes were installed in six glued laminated beams. Four individual moisture probes were positioned at each of the eleven specified probe locations (MC1-MC11 in Figure 5.4a).
These probes were equally distributed between the six instrumented beams. These beams were placed in a variable climate chamber where the relative humidity was stepped between 65% ± 5% and 90% ± 5% in an 8 week cycle at a constant temperature of 20°C ± 2°C.

![Figure 5.4. a) Moisture probe positions in structural sized seam, b) Moisture probes fixed in structural sized glued laminated beams](image)

### 5.4 Characterisation of Basalt Fibre Reinforced Polymer

#### 5.4.1 Introduction

Basalt fibre reinforced polymer (BFRP) was chosen as the reinforcement element in this study. It has been shown to demonstrate significant advantages over steel and glass fibre reinforced polymer (GFRP). The manufacturers report the values listed in Table 5.1 for the BFRP used in this project.

*Table 5.1. Basalt fibre reported material data (MagmaTech Limited 2013)*

<table>
<thead>
<tr>
<th>Tensile Strength (N/mm²)</th>
<th>Tensile Modulus (GPa)</th>
<th>CTE (°C⁻¹)</th>
<th>Thermal Conductivity (W/K.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000+</td>
<td>45+</td>
<td>2x10⁻⁶</td>
<td>0.7</td>
</tr>
</tbody>
</table>

* Loading rate of 1 kN/s (Yeboah et al. 2013)

This section details experimental tests carried out to characterise this material with a focus on elastic modulus and ultimate strength, but also examines possible long-term creep
effects and swelling/shrinkage effects due to variable climate conditions when used as a reinforcing element in timber beams.

5.4.2 BFRP Tensile Tests

In order to understand the behaviour of basalt fibre reinforced polymer (BFRP), tensile tests are conducted on six test pieces. The test set-up is in accordance with ISO 10406-1 (ISO 2015). In the tensile load test, 12 mm diameter rods are used, similar to those used to reinforced glued laminated timber beams in Section 3.3. In accordance with ISO 10406-1 (ISO 2015), 700 mm long BFRP rod specimens are manufactured by bonding the rod within two 110 mm steel tubes at either end (Figure 5.5). This leaves the required 480 mm (40 x \(d\), where \(d\) is the diameter of the rod) test section of BFRP rod. A two-part epoxy adhesive was used to bond the steel tube and FRP rod. The use of steel rods at each end ensures a good connection in the grips of the tensile tests machine and also prevents crushing of the rod due to the clamping force of the mechanical grips. The test specimens are stored in a temperature controlled room at 20°C and the adhesive is allowed to cure.

The mean diameter of each test piece is measured using a Vernier callipers, with an accuracy of 0.01 mm, at three positions along the exposed 480 mm length of BFRP rod.

![BFRP tensile test specimen](image)

An LVDT was attached centrally over a gauge length of 100 mm to measure the deformation in the test piece under load. Each test piece was initially loaded to 50 % of
the ultimate tensile capacity and the elastic modulus of the test piece was calculated using Equation [5.4].

\[ E = \frac{\Delta F}{\Delta \varepsilon \times A} = \frac{(F_{50\%} - F_{20\%})}{(\varepsilon_{50\%} - \varepsilon_{20\%}) \times A} \]  

where:
- \( E \) = Modulus of elasticity
- \( \Delta F \) = Change in tensile force between 20% and 50% of the ultimate force
- \( \Delta \varepsilon \) = Change in strain for the same change in force, \( \Delta F \)
- \( A \) = Mean cross-sectional area of the rod

The BFRP rods were loaded at a rate of 1% strain per minute, which equated to approximately 0.033 mm/s or 0.14 kN/s. Each test was completed within 300 seconds in accordance with ISO 10406-1 (ISO 2015). The specimens were tested in an environment with a relative humidity of 65% ± 5% and a temperature of 20°C ± 2°C. Once the elastic modulus test was complete, the test specimen was unloaded and the LVDT was removed. The test piece was then loaded to failure to calculate the failure load, ultimate tensile strength and ultimate strain. The tensile strength, \( f_u \) was calculated using Equation [5.5]

\[ f_u = \frac{F_u}{A} \]  

where:
- \( F_u \) = the ultimate tensile force
- \( A \) = Mean cross-sectional area of the rod

The ultimate strain is the strain corresponding to the ultimate tensile capacity when strain measurements are available up to failure. In one specimen, an electronic resistance strain gauge was used to evaluate the ultimate strain up to failure; however, this was not repeated in each case. In a case where strain gauge or displacement results are not available up to failure, Equation [5.6] was used to calculate the ultimate strain, \( \varepsilon_{ULT} \).

\[ \varepsilon_{ULT} = \frac{F_u}{E \times A} \]  

5.4.3 BFRP swelling/shrinkage test

To investigate the behaviour of BFRP rods under a changing humidity environment, six specimens of 12 mm diameter BFRP rod reinforcement were cut to approximately 120
mm in length. These specimens were initially conditioned in a constant environment with a relative humidity of 65% ± 5% and a constant temperature of 20°C ± 2°C for a period of 3 weeks. The length and diameter (mean of three measurements) of each specimen were measured using a Vernier callipers. The mass of each specimen was also measured, using a mass balance with an accuracy of 0.001g. Post conditioning, each specimen was subjected to an environment with a relative humidity of 90% ± 5% and a constant temperature of 20°C ± 2°C. The specimens were placed on a steel mesh to aid air circulation, ensuring the surface of the specimen was in contact with the surrounding environment (Figure 5.6). Each specimen’s dimensions and mass were measured frequently over a period of 500 hours to investigate the effect this high humidity environment on the swelling/shrinkage properties of BFRP rods.

![BFRP swelling/shrinkage test specimens](image)

**Figure 5.6. BFRP swelling/shrinkage test specimens**

### 5.4.4 BFRP Tensile Creep Tests

The creep behaviour of BFRP has received little attention in the literature. Many of the available studies on this novel material have focused on the behaviour at high stress levels (>50% of ultimate load). Wang et al. (2014) performed creep tests on BFRP rods at stress levels of 50%-70% of the ultimate tensile strength of the rods. They found that at the higher stress levels, creep rupture occurred and at the lower limits of their tests (50-60%) the creep rate remained relatively low. In this study, the stress level is relatively low in the BFRP rods used to reinforce each timber beam. To examine the creep behaviour at lower stress levels a series of test specimens were manufactured and creep tested at low stress levels over a period of 500 hours. In order to determine the creep characteristics of BFRP rods, the following test set-up was implemented in accordance with ISO 10406-1 (ISO 2015). The creep test specimens were manufactured from BFRP rod reinforcement
with diameters varying from 8-12 mm and the test length corresponded to a gauge length of 40 times the diameter of the rod. Two hollow threaded steel bars with an outer diameter of 20 mm and a length of 110 mm were adhered to each end of the BRFP rod specimens. A structural epoxy adhesive was used to adhere the rod within the threaded steel bar. The structural epoxy was allowed to cure for a period of a week prior to testing. A strain gauge was then adhered to the BFRP rod parallel to the fibre direction. The strains were measured using a Campbell Scientific data acquisition system. Once the structural epoxy adhesive had cured, M20 lifting eyes were screwed onto the threaded bars at either end of the test specimen. The specimen was positioned in an Instron tensile testing machine as seen in Figure 5.7. The preliminary specimen (12mm) was loaded and held at a constant dead load of 3.939 kN. This load was chosen to induce a stress of 34.85 N/mm² in the BFRP rod. This stress level corresponds to 3.85% of the ultimate tensile load of the BFRP rod and is also the mean stress experienced in the BFRP rods used to reinforce glued laminated beams in the creep test programme of this study, discussed in Section 4.3.2. The subsequent creep tests were carried out at higher stress levels of 8% and 15% of the ultimate tensile load in order to examine creep at higher stress levels. A proportion of the creep tests were repeated on 8 mm diameter rods at similar stress levels to observe any effect due to the rod diameter.

Figure 5.7. BFRP rod tensile creep test set-up
The strains were initially measured every five minutes during loading of the test specimen and during the early stages of the test. The measuring frequency was slowly reduced with time until the final frequency of 1 hour was achieved. Each specimen was creep tested over a 500 hour period.

5.5 Results

This section details the results observed from experimental tests described in Sections 5.2, 5.3 and 5.4. The results of tests performed in Section 5.2 will provide reliable data for modelling the moisture diffusion and moisture-induced strain development within timber beams and will be later validated against experimental results described in Section 5.3. The results of Section 5.4 will provide material data on the BFRP rods to use in finite element models described in Chapter 6.

5.5.1 Moisture Content Hysteresis Results

The moisture content hysteresis results from the experimental tests described in Section 5.2.1 are presented. It is important to note that the values reported here were achieved under isothermal conditions and only apply to Irish-grown Sitka spruce at 20°C ± 2°C. Figure 5.8 and Figure 5.9 display the mean moisture content of the cube and stick specimens during the increasing adsorption phase and the decreasing desorption phase, respectively. The change of moisture content in the sticks and cubes can be seen at each relative humidity level (30%, 50%, 60%, 70%, 80%, and 90%). Once the cubes and sticks had reached equilibrium moisture content for a given relative humidity level, the next relative humidity level is initiated. It can be seen that the stick samples experienced a slower change in moisture content when compared to the cube samples. This is due to their elongated geometry and the different rates of moisture exchange when comparing the radial, tangential and longitudinal directions. This rate of moisture exchange is controlled by the diffusion coefficient of the material. When the concentration of moisture within the atmosphere exceeds the concentration of moisture within the wood, a gradient is present and diffusion occurs.
The coefficient is direction dependent and while no significant difference is observed between the radial and tangential directions, the longitudinal direction experiences a more rapid moisture exchange in comparison. The moisture content hysteresis curve shown in Figure 5.10 was developed using the mean moisture content all 90 specimens (30 cubes and 60 sticks) measured at each given humidity level. The blue markers indicated
moisture contents on the lower limit (adsorption) of the hysteresis loop and the red markers indicate the upper limit (desorption) of the hysteresis loop.

Table 5.2. Moisture content vs relative humidity

<table>
<thead>
<tr>
<th>Phase</th>
<th>RH (%)</th>
<th>Moisture Content (%)</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adsorption</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>4.38</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>7.67</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>9.25</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>9.98</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>10.92</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>14.23</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>18.70</td>
<td>0.42</td>
</tr>
<tr>
<td>Common</td>
<td>100</td>
<td>27.85</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>20.82</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>16.78</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>14.91</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>12.77</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>10.57</td>
<td>0.12</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>7.04</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The value at 100% relative humidity is the moisture content corresponding to the maximum measured strain (Fibre Saturation Point) in the radial, tangential and longitudinal directions. A 4th order polynomial as shown in Equation [5.7] was used to express the moisture content during the adsorption and desorption phase as a function of relative humidity and at a temperature of 20°C. The shape factors for each curve are given in Table 5.3.

\[ u(x) = r_1 x^4 + r_2 x^3 + r_3 x^2 + r_4 x + r_5 \]  

Table 5.3. Hysteresis shape factors

<table>
<thead>
<tr>
<th></th>
<th>Adsorption</th>
<th>Desorption</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_1 )</td>
<td>1.32x10^6</td>
<td>0</td>
</tr>
<tr>
<td>( r_2 )</td>
<td>-2.10x10^{-4}</td>
<td>5.02x10^{-5}</td>
</tr>
<tr>
<td>( r_3 )</td>
<td>1.09x10^{-3}</td>
<td>-6.27x10^{-1}</td>
</tr>
<tr>
<td>( r_4 )</td>
<td>-2.90x10^{-2}</td>
<td>4.00x10^{-1}</td>
</tr>
<tr>
<td>( r_5 )</td>
<td>5.87x10^{-2}</td>
<td>-1.25x10^{-1}</td>
</tr>
</tbody>
</table>

The mean sorption curve for Sitka spruce compares well to those for other species reported in the literature (Hailwood & Horrobin 1946; Simpson 1973; Avramidis 1989). The test results from this experiment provide additional information on the adsorption and desorption phases for use in the numerical modelling described in Chapter 6.
5.5.2 Swelling/Shrinkage Measurement Results

The swelling and shrinkage of sixty 20 x 20 x 150 mm³ stick samples were measured once equilibrium had been achieved at each relative humidity level. Figure 5.11a and Table 5.5 contains the mean swelling and shrinkage strain measurements taken during the adsorption and desorption phases, respectively. It can be seen in Figure 5.11b, that the mean strains measured during the adsorption and desorption phases in the radial direction are repeatable and consistent with one another. This trend also holds when examining the tangential and longitudinal directions in Figure 5.11c and Figure 5.11d, respectively.

These swelling/shrinkage coefficients are calculated using the slope of the regression lines fitted to the data in Figure 5.11a. The swelling/shrinkage coefficients in the
tangential, radial and longitudinal directions are listed in Table 5.4. In a moisture fluctuating climate, these coefficients determine the magnitude of swelling/shrinkage strains within timber.

Table 5.4. Swelling/shrinkage coefficients

<table>
<thead>
<tr>
<th>Material Direction</th>
<th>Swelling/Shrinkage Coefficient (α)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tangential</td>
<td>0.2525</td>
</tr>
<tr>
<td>Radial</td>
<td>0.1371</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>0.0122</td>
</tr>
</tbody>
</table>

The radial and tangential results are similar to many reported results in the literature; however, the swelling/shrinkage coefficient of 0.0122 (‰/‰) in the longitudinal direction is greater than would be expected. This is thought to be as a result of the fast growth conditions of this timber and this is consistent with the findings of Bengtsson (2001), where a larger longitudinal swelling/shrinkage coefficient was observed in fast-grown trees when examining swelling/shrinkage coefficients in both slow and fast-grown trees. Table 5.5 provides the average moisture content and average strain data and associated standard deviations in the radial, tangential and longitudinal directions during the adsorption and desorption phases.

Table 5.5. Swelling/shrinkage strains in the radial, tangential and longitudinal direction as a function of moisture content

<table>
<thead>
<tr>
<th>Phase</th>
<th>RH (%)</th>
<th>Moisture Content (%)</th>
<th>Radial Strain</th>
<th>Standard Deviation</th>
<th>Tangential Strain</th>
<th>Standard Deviation</th>
<th>Longitudinal Strain</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adsorption</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>4.38</td>
<td>0.540</td>
<td>0.152</td>
<td>0.668</td>
<td>0.328</td>
<td>0.005</td>
<td>0.088</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>7.67</td>
<td>0.974</td>
<td>0.183</td>
<td>1.395</td>
<td>0.381</td>
<td>0.081</td>
<td>0.100</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>9.25</td>
<td>1.112</td>
<td>0.226</td>
<td>1.806</td>
<td>0.511</td>
<td>0.058</td>
<td>0.089</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>9.98</td>
<td>1.250</td>
<td>0.287</td>
<td>1.978</td>
<td>0.547</td>
<td>0.099</td>
<td>0.093</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>10.92</td>
<td>1.495</td>
<td>0.279</td>
<td>2.313</td>
<td>0.582</td>
<td>0.139</td>
<td>0.098</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>14.23</td>
<td>1.887</td>
<td>0.351</td>
<td>3.180</td>
<td>0.668</td>
<td>0.156</td>
<td>0.103</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>18.70</td>
<td>2.531</td>
<td>0.480</td>
<td>4.440</td>
<td>0.909</td>
<td>0.183</td>
<td>0.113</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>27.85</td>
<td>3.702</td>
<td>0.826</td>
<td>7.526</td>
<td>1.744</td>
<td>0.265</td>
<td>0.121</td>
</tr>
<tr>
<td>Desorption</td>
<td>90</td>
<td>20.82</td>
<td>2.766</td>
<td>0.572</td>
<td>5.033</td>
<td>1.224</td>
<td>0.236</td>
<td>0.098</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>16.78</td>
<td>2.235</td>
<td>0.477</td>
<td>3.922</td>
<td>1.064</td>
<td>0.217</td>
<td>0.080</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>14.91</td>
<td>1.967</td>
<td>0.421</td>
<td>3.458</td>
<td>0.976</td>
<td>0.177</td>
<td>0.067</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>12.77</td>
<td>1.741</td>
<td>0.369</td>
<td>2.994</td>
<td>0.905</td>
<td>0.155</td>
<td>0.062</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>10.57</td>
<td>1.362</td>
<td>0.301</td>
<td>2.384</td>
<td>0.827</td>
<td>0.129</td>
<td>0.058</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>7.04</td>
<td>0.801</td>
<td>0.182</td>
<td>1.427</td>
<td>0.718</td>
<td>0.086</td>
<td>0.051</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
5.5.3 Diffusion Behaviour Results

The diffusion behaviour of Irish grown Sitka spruce has been experimentally examined using three matched groups, each with different exposure conditions. The mean experimental test results of each group are presented along with the recorded relative humidity during the experiment in Figure 5.12. As expected, during the first increase in moisture content, each group undergoes a large change in moisture content. However, there are noticeable differences as a result of the exposure conditions.

During the first relative humidity increase, the “uncoated” Group 1 and the “mixed” Group 3 reach an equilibrium moisture content faster than the “coated” Group 2 cubes as they are free to transfer moisture through every surface. Similarly, during the first decrease in relative humidity, the “uncoated” Group 1 and the “mixed” Group 3 reach an equilibrium moisture content faster than the “coated” Group 2 cubes. In comparison, during the second cycle, the “mixed” Group 3 is now sealed in the longitudinal direction and subject to similar exposure conditions to that of the “coated” Group 2 cubes. It can be seen that for the second increase in relative humidity, the “mixed” Group 3 mean result demonstrates a similar rate of moisture content change as seen in the “coated” Group 2 cubes.

Figure 5.12. Diffusion coefficient test results of Group 1, Group 2 and Group 3
These experimental tests results have demonstrated the different rates of diffusion associated with different exposed faces of timber and will aid the selection of suitable diffusion coefficients in Chapter 6. Using a numerical moisture diffusion model and replicating the exposure conditions of the experimental cube specimens, the diffusion coefficients in the respective material directions will be calibrated against these experimental test results.

### 5.5.4 Moisture Flow in Structural Glued Laminated Beam Results

Calibrated moisture probes were inserted into six structural size, glued laminated beams and used to monitor moisture content with time at various depths within the cross section. These instrumented beams were placed in a variable climate chamber and stepped between 65% ± 5% and 90% ± 5% relative humidity in 8 week cycles at a constant temperature of 20°C ± 2°C. The moisture content was measured at locations MC1-MC11 shown in Figure 5.13. The probe positions were chosen to focus on the outer laminations and the PRF glue-line between the outer and inner laminations.

![Figure 5.13. Moisture probe positions](image)

The results from each probe position represent the average moisture content measured from four probes at similar locations in the six instrumented beams. The mean results at each probe location (MC1-MC11) can be seen in Figure 5.14 together with the recorded relative humidity data. The moisture pins close to the surface, experience a rapid change due to the high initial moisture gradient created as a result of the change in relative humidity. The moisture probes nearer to the centre of the beam experience less change in moisture content as expected. However, the mean moisture content is seen to increase...
with repeated moisture cycles at positions closer to the centre up to approximately week 55 after which, the maximum and minimum moisture contents appear to plateau with subsequent cycles. This is of course related to the magnitude of the moisture concentration gradient and duration of the cyclic climate (65% ± 5% to 90% ± 5% relative humidity in an 8 week cycle) implemented in this test.

Figure 5.14. Moisture probe results (MC1-MC11) and relative humidity data over a 75 week period

Moisture probe 1 (MC1) is located in the top compression lamination 40 mm from the side of the beam and at a depth of 10 mm from the top surface. This lamination is subjected to 2-dimensional flow of moisture as the top and sides of this lamination are exposed to the variable relative humidity environment.

Figure 5.15. Probe MC1: Mean result
The mean measured moisture content from probe location MC1 can be seen in Figure 5.15. MC1 is located 10 mm from the surface of the beam resulting in the rapid change in moisture content with changing relative humidity. The mean result at probe location MC1 is the mean of four moisture probe measurements (P1, P12, P19 and P38) which can be seen in Figure 5.16. This averaging process is repeated for all other probe locations (MC2-MC11) and the mean results of each are presented in this chapter. The individual measurements of each moisture probe (P1-P44) are graphed against time in Appendix D.

Moisture probe 2 (MC2) is located in the top compression lamination 40 mm from the side of the beam and at a depth of 25 mm from the top surface. This lamination is subjected to 2-dimensional flow of moisture as the top and sides of this lamination are exposed to the fluctuating relative humidity.

MC2 is the mean result from four moisture probes (P2, P13, P20, and P39) and shows a more gradual increase in moisture content when compared to MC1. The overall mean
values from these probes over the 75 week test period can be seen in Figure 5.17. Moisture probe 3 (MC3) is located in the second lamination from the top, 40 mm from the edge of the beam and at a depth of 37.5 mm. The sides of this lamination are exposed to the fluctuating relative humidity; however, flow through the top and bottom surfaces of this lamination is impeded by the PRF adhesive used during the manufacture of these glued laminated beams. The mean moisture content of probe MC3 is the mean value from four moisture probes (P3, P14, P21, and P40) and can be seen in Figure 5.18.

Figure 5.18. Probe MC3: Mean result

The mean values of MC1, MC2 and MC3 with time can be seen in Figure 5.19. As expected, MC3 being the furthest point of measurement from the surface, experiences less change in moisture content due to the low gradient experienced at this depth. It is also affected by the PRF adhesive to an unknown extent. The effect of adhesive glue-lines are often neglected; however, they may play an important role in moisture transport within small and large engineered wood products. The impact of the adhesive on moisture flow affects how stresses develop within these engineered wood products.

The moisture content can be seen to change with time with subsequent relative humidity cycles. MC1 shows approximately consistent and repeatable results with increasing cycles; however, with increasing depth, the internal measurement points (MC2 and MC3) appear to be subjected to an increasing trend in mean moisture content up until week 35. After this point, the mean moisture content appears to reach a maximum and minimum moisture content value with each adsorption and desorption phase, respectively.
MC4-MC9 are located along the sides of the beams at a depth of 10 mm from the surface of the timber (Figure 5.21). The mean results from MC8 and MC9 similar to those from MC6 and MC7 due to their similar exposure surface conditions and depth within the timber and are not discussed further. The bottom of the beam is sealed and the bottom lamination is only exposed to the external environmental conditions on the sides of the lamination. In Figure 5.20, the mean moisture content results from MC4-MC7 are presented together with the recorded relative humidity data over the 75 week test period. Probes MC4 and MC5 are located in the top lamination which is exposed to the external climate conditions on the top surface and both sides of the beam. These probes experience greater moisture content change due to the combination of moisture movement from both the top surface and the sides of the beam, whereas in comparison, MC6 and MC7 are located in the second lamination from the top and a reduced change in moisture content is observed. This is possibly due to the effects of the glue-line on moisture flow through the cross section.
MC10 and MC11 are located in the bottom lamination. Half of the beams are reinforced with BFRP rods secured with an epoxy adhesive in the bottom tensile lamination as illustrated in Figure 5.21.

The BFRP rods and epoxy adhesive can be seen to affect the moisture flow within these beams. Figure 5.22 and Figure 5.23 display the differences observed between unreinforced and reinforced beams measured for MC10 and MC11, respectively. In both
cases, the unreinforced beams experience a greater change in moisture content when compared to that in the reinforced beams. This is expected due to the position of MC10 and MC11 in the centre of the bottom lamination and the effect of the reinforcement and epoxy adhesive impeding the flow of moisture in this lamination.

![Figure 5.22. Probe MC10: Mean results in unreinforced and reinforced beams](image)

![Figure 5.23. Probe MC11: Mean results in unreinforced and reinforced beams](image)

### 5.5.5 BFRP Test Results

This section details the results of the tensile tests, swelling/shrinkage tests and creep test performed on specimens of basalt fibre reinforced polymer rod reinforcement.

#### 5.5.5.1 BFRP Tensile Test Results

Six samples were prepared as outlined in Section 5.4.2 in accordance with ISO 10406 (ISO 2015). As seen in Figure 5.24a, the deformation was measured using an LVDT over a gauge length of 100 mm. In addition to measuring the elastic modulus, the rods were
also tested to failure to observe the tensile strength and ultimate strain as seen in Figure 5.24b and Figure 5.24c.

![Figure 5.24. BFRP Rod: a) Strain gauge and LVDT, b) Failed specimen, c) Comparison between failed and un-failed specimens](image)

The tensile test results for each test specimen up to a tensile load of 50 kN can be seen in Figure 5.25. When tested to failure, the rods demonstrated brittle failure (Figure 5.24c) in all cases with the exception of Rod 4, which experienced adhesive failure through pull-out of the rod from the steel anchor. This failure occurred after the initial elastic modulus
test at a load greater than 50% of the ultimate failure load. The results in Table 5.6, provide the average and standard deviations of the experimentally determined, elastic modulus, failure load, tensile strength and ultimate strain.

Table 5.6. BFRP elastic modulus and tensile strength results

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Young's Modulus (N/mm²)</th>
<th>Failure Load (N) (Rate =0.14 kN/s)</th>
<th>Tensile Strength (N/mm²)</th>
<th>Ultimate Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rod 1</td>
<td>46978.3</td>
<td>99442</td>
<td>879.71</td>
<td>1.87%</td>
</tr>
<tr>
<td>Rod 2</td>
<td>49552.6</td>
<td>94636</td>
<td>837.19</td>
<td>1.69%</td>
</tr>
<tr>
<td>Rod 3</td>
<td>55251.1</td>
<td>105052</td>
<td>929.33</td>
<td>1.68%</td>
</tr>
<tr>
<td>Rod 4</td>
<td>52464.7</td>
<td>54629*</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rod 5</td>
<td>49794.4</td>
<td>99353</td>
<td>878.92</td>
<td>1.77%</td>
</tr>
<tr>
<td>Rod 6</td>
<td>50589.3</td>
<td>112875</td>
<td>998.54</td>
<td>1.97%</td>
</tr>
<tr>
<td>Mean</td>
<td>50771.7</td>
<td>102722</td>
<td>904.74</td>
<td>1.80%</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>2821.2</td>
<td>6982</td>
<td>61.77</td>
<td>0.13%</td>
</tr>
</tbody>
</table>

* Values omitted from mean and standard deviation due to pull-out failure of Rod 4

The average measured elastic modulus of 50772 N/mm² is consistent with the value provided by the manufacturer of 45000+ N/mm² but the tensile strength of 905 N/mm² is lower than that provided by the manufacturer of 1000+ N/mm². This was previously observed by Yeboah et al. (2013) whereby the load rate of the test heavily influences the test result. Yeboah et al. (2013) used a load rate of 0.2 kN/s which is comparable to the load rate used in this study (0.14 kN/s) but is much lower than that used by the manufacturer (1 kN/s).

5.5.5.2 BFRP Swelling/shrinkage Test Results

A study was undertaken to determine the swelling/shrinkage characteristics of BFRP under variable climate conditions. The results are tabulated in Table 5.7. When examining the change in length, strain, diameter and mass throughout the test it became apparent that the variable climate had little effect on the structural stability of this BFRP rod material. The manufacturing process of this material involved the binding of BFRP fibres within an epoxy matrix to create the composite. It is believed that in this format the epoxy resin impedes the flow of moisture and provides a stable material under changing relative humidity conditions. This study focuses on vapour transfer from the surrounding environment and neglects additional moisture transport mechanisms that may occur if this material is in contact with or submerged in water. The experimental test results are tabulated in Table 5.7 and the mass of each specimen throughout the test period is
illustrated in Figure 5.26. There are no obvious changes in the mass as a result of moisture content uptake over the 500 hour test period.

Table 5.7. BFRP swelling/shrinkage test results

<table>
<thead>
<tr>
<th>Time (Hours)</th>
<th>Length (mm)</th>
<th>Longitudinal Strain (ε) (%)</th>
<th>Mean Diameter (mm)</th>
<th>Mass (g)</th>
<th>Density (kg/m³)</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>117.09</td>
<td>0.0000</td>
<td>12.36</td>
<td>28.506</td>
<td>2030.2</td>
<td>0.084%</td>
</tr>
<tr>
<td>25.1</td>
<td>117.10</td>
<td>0.0100</td>
<td>12.38</td>
<td>28.509</td>
<td>2022.5</td>
<td>0.094%</td>
</tr>
<tr>
<td>55.6</td>
<td>117.09</td>
<td>0.0071</td>
<td>12.37</td>
<td>28.508</td>
<td>2027.7</td>
<td>0.091%</td>
</tr>
<tr>
<td>96.8</td>
<td>117.09</td>
<td>0.0028</td>
<td>12.35</td>
<td>28.507</td>
<td>2035.1</td>
<td>0.088%</td>
</tr>
<tr>
<td>143.1</td>
<td>117.09</td>
<td>0.0071</td>
<td>12.35</td>
<td>28.507</td>
<td>2034.5</td>
<td>0.088%</td>
</tr>
<tr>
<td>190.6</td>
<td>117.08</td>
<td>-0.0014</td>
<td>12.33</td>
<td>28.507</td>
<td>2039.3</td>
<td>0.085%</td>
</tr>
<tr>
<td>238.9</td>
<td>117.09</td>
<td>0.0085</td>
<td>12.32</td>
<td>28.507</td>
<td>2042.1</td>
<td>0.087%</td>
</tr>
<tr>
<td>310.8</td>
<td>117.10</td>
<td>0.0171</td>
<td>12.30</td>
<td>28.508</td>
<td>2048.4</td>
<td>0.092%</td>
</tr>
<tr>
<td>382.5</td>
<td>117.08</td>
<td>-0.0043</td>
<td>12.34</td>
<td>28.506</td>
<td>2036.8</td>
<td>0.083%</td>
</tr>
<tr>
<td>458.3</td>
<td>117.08</td>
<td>-0.0015</td>
<td>12.35</td>
<td>28.505</td>
<td>2034.0</td>
<td>0.081%</td>
</tr>
<tr>
<td>500.0</td>
<td>117.08</td>
<td>-0.0056</td>
<td>12.35</td>
<td>28.505</td>
<td>2032.7</td>
<td>0.080%</td>
</tr>
</tbody>
</table>

Figure 5.26. Mass of each specimen (Rod 1-6) over a 500 hour period

In an attempt to investigate this further, the percentage increase in mass, the density of each sample and the change in strain was examined over the entire test period. Although some increases were observed initially, the trends did not continue and no significant effects were observed throughout the test as seen in Table 5.7.

5.5.5.3 BFRP Tensile Creep Test Results

The result from each test specimen loaded under different tensile loads is presented. The tensile load applied is a percentage of the ultimate tensile strength measured previously in Section 5.4.2. The creep strain results from each test specimen can be seen in Figure 5.27. At 3.85% of the ultimate tensile load, the creep strain in the BFRP rod is negligible.
Any fluctuations are deemed to result from small temperature fluctuations (± 2°C) during the test period. This stress level corresponds to the mean load level in the BFRP rods in the reinforced beams when loaded in the creep test frame in Chapter 4 and there was no significant creep strain observed over the 500 hour period.

The creep test results are similar at all higher stress levels measured. At 8% of the ultimate tensile strength, the strain increases to approximately $15 \times 10^{-6} \varepsilon$ during the first 100 hours after loading and levels off at this point for the duration of the test; however, this is not deemed significant. Similarly at 15% of the ultimate tensile strength, the strain increases to approximately $25 \times 10^{-6} \varepsilon$ during the first 100 hours after loading and levels off at this point for the duration of the test. Although there is a trend of increasing creep strain in the BFRP rods with increasing stress, these relatively low stress levels have resulted in negligible creep strain within the BFRP rod. There was also no apparent influence due to the rod diameter when loaded to similar stress levels.

The use of such BFRP rods within reinforced timber beams results in under-utilisation of the ultimate tensile capacity of the rods. In a previous study by Wang et al. (2014), BFRP rods underwent sustained loading at stress levels from 50% to 70% of the
ultimate tensile strength. The experimental results of this study show relatively low creep rates in BFRP rods when loaded between 50% and 60% of the ultimate tensile failure and at higher stress levels, creep rupture was reported. The stress levels of 15% ULT or lower in the BFRP rods used in this study is much lower than the lower limit tested by Wang et al. (2014). The experimental test results have indicated that the creep behaviour of BFRP rods at such a low stress levels is negligible.

5.6 SUMMARY AND DISCUSSION

The properties of timber are heavily influenced by moisture content and in order to model the long-term effects of timber in variable climates, it is important to quantify these moisture dependent material properties. As no previous data relating to the hygro-mechanical behaviour of fast-grown Irish Sitka spruce was available, a series of experimental tests were performed.

It has been shown that timber under a variable relative humidity condition is subject to a hysteresis effect. Experimental tests on ninety small specimens of Irish Sitka spruce have been used to examine the upper and lower limits of the moisture content hysteresis curve at 20°C ± 2°C. This hysteresis effect has been successfully characterised for this species and will allow accurate definition of the surrounding environment in the boundary conditions of the numerical models in Chapter 6. It has also been shown that timber swells and shrinks under variable relative humidity conditions at moisture contents below fibre saturation point. To quantify this behaviour, the dimensional changes in sixty specimens of Irish Sitka spruce were monitored at various moisture content below the fibre saturation point. Each specimen was studied and the mean swelling/shrinkage coefficients specific to fast-grown Sitka spruce have been determined in the radial, tangential and longitudinal direction. It was noted that a large longitudinal swelling/shrinkage coefficient characteristic of fast-grown species was also observed in fast-grown Sitka spruce. These experimental results will be used in the evaluation of moisture-induced strain in Chapter 6. Experimental tests have also been performed to examine the diffusion behaviour of timber in the respective material directions. Many authors have reported different diffusion coefficients for various species of timber; however, none have been shown to successfully describe moisture diffusion in Irish Sitka spruce. The experimental test results on three matched groups have demonstrated the differences in moisture adsorption and desorption associated with different exposure
conditions and will aid the selection of appropriate direction-dependent diffusion coefficients using a numerical moisture diffusion model in Chapter 6.

Insulated moisture probes were inserted into glued laminated timber beams and have measured the change in moisture content with time in a variable climate. The results show various moisture content and moisture gradient changes with depth at the various pin locations. These test results on structural sized timber elements will be compared to a numerical moisture diffusion model in Chapter 6 utilising the species-specific material properties determined in this chapter.

A series of tests on BFRP rod reinforcement have been conducted to quantify material properties for use in numerical studies in Chapter 6. The mean elastic modulus and ultimate tensile strength have been determined through tensile tests. The mean elastic modulus for the rods was 50772 MPa. Specimens of BFRP rod reinforcement have also been subjected to a high relative humidity environment to examine the swelling/shrinkage characteristics. These tests found no evidence of any substantial moisture uptake when exposed to a high humidity environment over a 500 hour period and may be assumed negligible. Creep tests were also performed on specimens of BFRP rod. It was found that the when used to reinforce timber elements, FRP reinforcement is generally under-utilised in terms of ultimate tensile capacity. In this particular study, the BFRP rods are loaded to approximately 3.85% of their ultimate tensile capacity. Previous research has shown relatively low levels of creep strain at 50-60% of the ultimate tensile strength and as a result the experimental study of creep has shown no significant creep strain in BFRP rods tested up to 15% of their ultimate tensile strength.
6 HYGRO-MECHANICAL CREEP MODELLING

6.1 INTRODUCTION

Numerical modelling can be an important tool when modelling complex materials such as timber. There are often many challenges involved in modelling timber due to its anisotropic nature and inherent variability. Modelling timber is further complicated due to its hygroscopic nature. As this work is focused on long-term effects of unreinforced and reinforced glued laminated beams subjected to both mechanical loads and time varying relative humidity loads, a coupled moisture-displacement model is required to calculate the moisture content distribution in timber elements due to the variable relative humidity loads and the associated mechanical strains. Utilising such models may contribute significantly to the understanding and further development of reinforced timber elements. Examining long-term effects of timber, as the name suggests, requires long durations of experimental testing to observe the desired behaviour. In many cases, the longer the test, the more reliable the result; however, such tests are expensive not only in terms of time but also from an economic point of view, require substantial early investment in instrumentation, specimen manufacture and test apparatus in addition to the maintenance costs throughout the duration of the test. The long test durations and high costs associated generally limit the number of specimens that can be tested and as a result, it is often not feasible to examine a wide variety of reinforcement methods or materials. In the long-term experimental tests presented in Chapter 4, only one reinforcement scheme and material were investigated. The development of a numerical model to successfully describe this long-term experimental behaviour will allow many variations in geometry, reinforcement scheme, reinforcement format, and reinforcement type to be modelled, further increasing the development and reliability of such beams with no substantial time or economic constraints. In addition, many different environmental conditions may be implemented in the model to examine such beams in various different applications.

The models presented in this chapter are developed using Abaqus finite element analysis software (Abaqus 2014). This versatile software can be adapted to monitor moisture movement within timber with the use of a thermo-hygro analogy. This analogy allows the standard heat transfer or coupled temperature-displacement analysis available
in Abaqus finite element software to be used to describe the moisture diffusion or coupled moisture-displacement analysis in timber as similar equations govern both heat and moisture transfer. Both transport phenomena are driven by diffusion. In the case of mass transfer, this is called mass diffusion and in the case of heat transfer, this is called conduction. The similarity becomes clear when one compares Fick’s law of mass diffusion (Equation [6.1]) and Fourier’s law of heat conduction (Equation [6.2]), which in three-dimensional form read,

\[
\frac{\partial \phi}{\partial t} = D_x \left( \frac{\partial^2 \phi}{\partial x^2} \right) + D_y \left( \frac{\partial^2 \phi}{\partial y^2} \right) + D_z \left( \frac{\partial^2 \phi}{\partial z^2} \right) \tag{6.1}
\]

\[
\frac{\partial T}{\partial t} = k_x \left( \frac{\partial^2 T}{\partial x^2} \right) + k_y \left( \frac{\partial^2 T}{\partial y^2} \right) + k_z \left( \frac{\partial^2 T}{\partial z^2} \right) \tag{6.2}
\]

where:

\begin{align*}
D &= \text{Diffusion coefficient} \\
\phi &= \text{Moisture concentration} \\
k &= \text{Thermal conductivity} \\
T &= \text{Temperature}
\end{align*}

In this chapter, the thermo-hygro analogy is utilised to create three primary models each utilising species-specific material properties determined in Chapter 5 where appropriate:

- Moisture diffusion model
- Coupled moisture-displacement model
- Hygro-mechanical creep model

First, a 3-dimensional non-linear moisture diffusion model is created to determine appropriate non-linear diffusion coefficients for fast-grown spruce. This is achieved by matching various diffusion coefficients reported in the literature to the experimental tests on the diffusion behaviour of Sitka spruce presented in Chapter 5. Subsequently, a 2-dimensional non-linear moisture diffusion model utilising the appropriate diffusion coefficients is created to predict the moisture content distribution in a structural sized glued laminated beam. This model is then compared to experimental test results previously presented in Section 5.5.4. Both models focus on the moisture transfer between timber and the surrounding environment. Essential to this is the definition of the
boundary conditions with time in the model. This is achieved using a user-defined DFLUX subroutine, which allows for the variable relative humidity boundary conditions or interaction between the timber surface and the surrounding environment to be defined with time.

Second, a 3-dimensional non-linear coupled moisture-displacement model is created to simulate the development of moisture-induced swelling/shrinkage strains in timber due to differential swelling and shrinkage in a variable climate. This utilises the moisture diffusion model in addition to experimentally measured swelling/shrinkage coefficients specific to fast-grown Sitka spruce. The moisture-induced swelling and shrinkage strain results are compared to the experimental test results presented in Section 4.6.2.

Third, as mechano-sorptive creep is not available in Abaqus, a user-defined UMAT subroutine is required to define mechano-sorptive behaviour during loading and simultaneous moisture movement within timber. Such models have been shown to successfully replicate and deflection and stress development in fluctuating environments by Mirianon et al. (2008) and Fortino et al. (2009). Their model is based on the 1-dimensional model provided by Toratti (1992a). In this chapter, the UMAT subroutine is adapted to better simulate creep behaviour of Irish-grown Sitka spruce and is compared to the experimental creep tests results previously presented in Chapter 4. The final calibrated model incorporating mechano-sorptive creep is used in a parametric study to examine the effect of reinforcement type on the long-term behaviour of reinforced timber beams in a variable climate.

### 6.2 Moisture Diffusion Model

#### 6.2.1 Introduction

Moisture diffusion models are created to predict the flow of moisture within timber elements manufactured from Irish-grown Sitka spruce. The definition of the surrounding environment and rate of moisture exchange between the surrounding environment and timber are essential in the solution of such models. In this section, the implementation of a variable relative humidity boundary condition is discussed and numerical simulations are carried out to select appropriate direction-dependent diffusion coefficients to describe the diffusion behaviour observed in the experimental tests presented in Section 5.2.3. The
experimental test previously presented in Section 5.3 is then simulated in the developed moisture diffusion model. The numerical results are then compared to the experiment test results previously presented in Section 5.5.4.

6.2.2 Environmental Load and Rate

The environmental load or relative humidity load is implemented in Abaqus through a DFLUX subroutine in order to apply a defined relative humidity boundary condition as a function of time. This relative humidity, which corresponds to a specific moisture content in the timber, is applied to the exposed surfaces of the timber. The previously determined moisture content hysteresis curve (Section 5.2.1) is utilised, which adjusts the moisture content to the lower curve during adsorption and to the upper curve during desorption. The shape factors for both curves have been experimentally determined in Section 5.5.1 and are given in Table 5.3. The rate of moisture flow from the surrounding environment to the exposed surface of the timber is governed by Equation [6.3] (Mirianon et al. 2008; Hanhijarvi 1995; Toratti 1992a).

\[
q_n = S_u (\phi_{eq} - \phi_{surf}) = \rho S_u (u_{eq} - u_{surf})
\]  

[6.3]

where:

- \(q_n\) = Moisture flow across the boundary
- \(S_u\) = Surface emission coefficient
- \(\phi_{surf}\) = Moisture concentration of the wood surface
- \(\phi_{eq}\) = Equilibrium moisture concentration of timber corresponding to the relative humidity of the surrounding environment
- \(\rho\) = Density at 0% moisture content
- \(u_{surf}\) = Moisture content on the wood surface
- \(u_{eq}\) = Equilibrium moisture content of timber corresponding to the relative humidity of the surrounding environment

The surface emission coefficient, \(S_u\), defines the rate of moisture content exchange across the boundary. This material property is a function of the environment, the characteristics of the timber surface and is also dependent on the velocity of the circulating air (Hanhijarvi 1995). The value used in this model of \(3.2 \times 10^{-8} \ e^{4.0u} \text{ m/s}\) was previously reported by Hanhijarvi (1995).
6.2.3 Diffusion Coefficients

The selection of appropriate material direction dependent diffusion coefficients for timber is fundamental when examining moisture flow within timber elements. The diffusion coefficients are dependent on material direction. In this study, the differences between the diffusion coefficients in the radial and tangential directions are assumed negligible and no distinction is made; however, the longitudinal diffusion coefficient is significantly larger (Rosen 1978; Sjödin 2006).

In order to determine suitable diffusion coefficients for fast-grown Sitka spruce, a 30 x 30 x 30 mm³ specimen is modelled based on the cube specimens used to observe the diffusion coefficients as described previously in Section 5.2.3. The material properties of this cube are cylindrically orientated to best replicate the orientations and orthotropic nature of timber. This allows for directional dependent diffusion coefficients to be applied in the radial, tangential and longitudinal direction. The developed finite element mesh (Mesh 4) which can be seen Figure 6.1 is seeded as determined by a mesh sensitivity study (Table 6.1). In total, 8000 heat transfer DC3D8 elements were used ranging from a 0.6 mm at each corner to 3 mm in the centre. This mesh size chosen provided consistent moisture distribution results with time and was not demanding regarding computational time or memory usage.

Table 6.1. Diffusion coefficient model mesh sensitivity study

<table>
<thead>
<tr>
<th>Mesh 1</th>
<th>Mesh 2</th>
<th>Mesh 3</th>
<th>Mesh 4</th>
<th>Mesh 5</th>
<th>Mesh 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Elements</td>
<td>6600</td>
<td>7000</td>
<td>7500</td>
<td>8000</td>
<td>8500</td>
</tr>
<tr>
<td>Moisture content</td>
<td>17.47%</td>
<td>17.50%</td>
<td>17.53%</td>
<td>17.54%</td>
<td>17.55%</td>
</tr>
</tbody>
</table>

* mean moisture content at 4.5 weeks

Figure 6.1. Diffusion coefficient specimen model
The relative humidity load is applied to the exposed surfaces via a DFLUX subroutine implemented within Abaqus finite element software. Various reported softwood timber diffusion coefficients were incorporated into the model and investigated. A comparison of the radial and tangential diffusion coefficients from some of the selected studies can be seen Figure 6.2 (Siau 1984; Toratti 1992a; Hanhijarvi 1995; Sjödin 2006; Fortino et al. 2009).


The majority of the diffusion coefficients studied were moisture dependent and have been shown to provide more accurate results when studying timber elements in variable environments. For example, Toratti (1992) incorporated a moisture dependent diffusion coefficient of $1.2 \times 10^{-10} e^{2.28u} \text{m}^2/\text{s}$ where $u$ is the moisture content. Hanhijarvi (1996) later increased the moisture dependency of the diffusion coefficient used by Toratti (1992) to $8.0 \times 10^{-11} e^{4.0u} \text{m}^2/\text{s}$. The various reported diffusion coefficients for timber were examined in this model and compared to the experimentally determined results. The simulated moisture content results of the numerical model are compared to experimental test results of Group 1, Group 2 and Group 3 in Figure 6.3.
Figure 6.3. Moisture diffusion experimental results: a) Group 1, Coated b) Group 2, Uncoated c) Group 3, Mixed

In Figure 6.3a, the radial/tangential diffusion coefficients presented by Toratti (1992) are paired with the longitudinal diffusion coefficient provided by Sjödin (2006) and compared to the uncoated Group 1 experimental results. The rapid increase and decrease in moisture content due to the relative humidity cycle has been adequately simulated. This
combination of diffusion coefficients was found to be the most appropriate to adequately describe the moisture flow in fast-grown Sitka spruce exposed on all surfaces. In Figure 6.3b, the coated Group 2 results are compared to the numerical model. The radial/tangential diffusion coefficients previously presented by Toratti (1992) agreed best with the experimentally determined results when moisture movement is limited to the radial/tangential surfaces. The mixed Group 3 results are shown in Figure 6.3c. Under the combined conditions (Coated and Uncoated), the model has successfully described the behaviour of the Group 3 cubes in a variable climate when subjected to moisture flow through various faces and material directions.

The figures presented shows good agreement with the experimentally determined curves and the diffusion coefficients provided by Toratti (1992) and Sjödin (2006) and are deemed suitable for Irish Sitka spruce in the tangential/radial and longitudinal directions, respectively.

6.2.4 Moisture Diffusion Model Application

In this section, the experimental tests presented in Section 5.3 are simulated. The diffusion coefficients already validated have been implemented in the model to examine the flow of moisture in larger structural size glue laminated beams. The simulated results are compared to the experimental results presented in Section 5.5.4. The effect of a fully permeable and impermeable glue-line on the flow of moisture in structural size glued laminated beams is also examined.

6.2.4.1 Model Geometry

The experimentally measured flow of moisture within glued laminated beams has previously been presented in Section 5.3.2. In order to replicate the test set-up presented and model the moisture flow through the cross section of the beam, a 2-dimensional moisture transfer model was created. This simplification to two dimensions is justified as the ends of the beam are sealed limiting moisture flow through the radial and tangential faces of the beam. The model mesh consisting of 3536 heat transfer DC2D4 elements was determined using a mesh sensitivity study (Table 6.2). The model comprises four cylindrically orientated timber laminations, each separated by a 0.1 mm adhesive glue-line. The pith of each timber lamination was centred along the top surface of each lamination. The beams are sealed along the bottom surface and moisture exchange is
limited to the top surface and both sides of each beam. The directional-dependent diffusion coefficients determined in Section 6.2.3 are applied in the cylindrically orientated radial and tangential directions. The glue-line is modelled as either a fully permeable and fully impermeable interface, in order to examine the glue-lines effect on moisture flow in glued laminated beams as seen in Figure 6.4.

Table 6.2. Moisture diffusion model mesh sensitivity study

<table>
<thead>
<tr>
<th>Mesh</th>
<th>No. of Elements</th>
<th>Mesh 1</th>
<th>Mesh 2</th>
<th>Mesh 3</th>
<th>Mesh 4</th>
<th>Mesh 5</th>
<th>Mesh 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture content</td>
<td>15.11%</td>
<td>15.25%</td>
<td>15.29%</td>
<td>15.31%</td>
<td>15.32%</td>
<td>15.32%</td>
<td></td>
</tr>
</tbody>
</table>

* mean moisture content at week 5

Figure 6.4. 2-D Finite element model of moisture content distribution at 7 weeks: a) timber beam comprising four adhesively bonded laminations b) fully permeable glue-line c) fully impermeable glue-line

In Figure 6.4b the glue-line is assumed to be fully permeable with the diffusion coefficients of the glue-line equal to that of the timber, whereas, in Figure 6.4c the glue-line is modelled as fully impermeable with the diffusion coefficients of the adhesive equal to zero allowing no moisture movement through the glue-line. The main differences between the model predictions occur in the top two laminations. When the fully impermeable glue-line is modelled, the flow of moisture is impeded limiting all three lower laminations to 1-dimensional diffusion leading to reduced moisture flow in the second lamination. The moisture content distribution through the cross section was calculated with time while examining both glue-line conditions. The moisture content profile through the cross section was determined with time and the appropriate node positions corresponding to moisture probe locations (MC1-MC11) were identified.
6.2.4.2 Moisture Diffusion Model Results

The simulated results are compared to experimental results previously presented in Section 5.5.4. In Figure 6.5, the mean of the experimental measurements from moisture probe MC1 are compared to the simulated results from the numerical model. The numerical results for both the fully permeable glue-line and impermeable glue-line condition are plotted against the experimental results for the 75 weeks of testing. It can be seen that the numerical results of both the permeable glue-line and impermeable glue-line condition adequately match the experimental data for MC1 which is located within the top laminate 10 mm from the top surface and 40 mm from the side of the beam (Figure 5.13).

![Moisture Probes MC1](image)

*Figure 6.5. Probe MC1: Experimental vs Numerical results*

The glue-line has an insignificant effect on the flow of moisture at the MC1 position; however, this is as expected as this point is close to the top surface of the beam. The largest simulated differences in moisture content as a result of the impermeable glue-line occurs during the initial moisture cycles and the differences diminish with repeating cycles.

The experimental and numerical results for moisture probe MC2 are plotted in Figure 6.6. The increased depth of this moisture probe has resulted in a more gradual increase in moisture content at this position. The numerical and experimental results show similar trends of increasing and decreasing moisture content with repeated moisture cycles. The numerical results have shown a greater difference between the fully permeable and impermeable glue-line; however, these differences are largest within the
initial moisture cycles and reduce with repeated moisture cycling. There is a phase lag between the experimental and numerical results presented. Under ideal conditions, a slight delay in moisture content change would be expected as simulated by the model. This abrupt change in moisture content observed experimentally is believed to be as a result of the multiple factors influencing the experimentally measured moisture content with depth using the resistive method. However, considering the factors influencing the probes (type, spacing, contact issues, temperature) with depth, the experimental results provide a reasonable estimation of the true moisture content at each measurement point.

Figure 6.6. Probe MC2: Experimental vs Numerical results

There is a similar phase lag when examining moisture probe MC3 which is located within the second laminate from the top, 37.5 mm from the top surface and 40 mm from the side of the beam. The simulated results in Figure 6.7 show a similar trend of increasing moisture content with each moisture cycle. Given the position of this probe, MC3 is heavily influenced by the glue-line which is apparent when examining the simulated results in Figure 6.7.
Figure 6.7. Probe MC3: Experimental vs Numerical results

The model has shown good agreement during the adsorption phase; however, experimental results have shown a more rapid decrease in moisture content during desorption phase. This would indicate differences between the diffusion coefficients during adsorption and desorption. In a study by Peralta & Lee (1995), the measured diffusion coefficient during desorption was approximately twice that measured during adsorption. It would be beneficial to incorporate different diffusion coefficients to further refine the model. It is also worth noting that moisture flow in timber is not the solely attributed to moisture diffusion and other mechanisms may contribute to the flow of moisture.

When the numerical simulations with a fully permeable and impermeable glue-line are compared to the experimental data in moisture probe MC2 and MC3, a slightly higher correlation coefficient is observed for the fully permeable glue-line model. For moisture probe MC2, a correlation coefficient of 0.79 was observed when comparing the experimental data to that of the model with a fully permeable glue-line compared to a correlation coefficient of 0.76 was observed when the experimental data was compared to that of the model with an impermeable glue-line. Similar trends were seen when comparing results in MC3. A correlation coefficient of 0.63 was observed when comparing the experimental data to that of the model with a fully permeable glue-line compared to a correlation coefficient of 0.60 when the experimental data was compared to that of the model with an impermeable glue-line. This would indicate that the inclusion of the glue-line within the model may add to the computational time and not necessarily contribute to additional accuracy. It must also be considered that the size of the beams in
the test programme is 98 mm x 125 mm and the effect of the glue-line may be more significant in larger sections.

In Figure 6.8 and Figure 6.9, the experimentally determined average moisture content at probes MC4 and MC5 are compared to the results of the numerical model. In both MC4 and MC5, the numerical results have shown good agreement with the experimental findings with repeated cycles. There is no significant difference observed between the fully permeable and impermeable glue-line model.

![Figure 6.8. Probe MC4: Experimental vs Numerical results](image)

![Figure 6.9. Probe MC5: Experimental vs Numerical results](image)

Significantly, the top lamination is subject to 2-dimensional flow through the top surface and each side of the timber lamination, hence MC4 and MC5 have experienced greater moisture content change than probes at similar depths below the surface in laminations subjected predominantly to 1-dimensional flow (MC6, MC7, MC8 and MC9) as
previously seen in Figure 5.13. This greater change in moisture content has been accurately predicted by the finite element model.

In MC6-MC9, a reduced change in moisture content can be seen as such probe locations are subjected to 1-dimensional flow and positioned 10 mm from the side of the beam. The numerical results of probe MC6-MC9 have also shown good agreement with the experimental results as seen in Figure 6.10-Figure 6.13, respectively. MC6 similar to MC7 has shown a better correlation coefficient when comparing the experimental data to that of the model with a fully permeable glue-line, however, the most significant difference between both numerical results occur during the first three cycles. MC8 and MC9 have shown no difference between the model incorporating an impermeable glue-line and the model incorporating a fully permeable glue-line. This is due to the position of moisture probes MC8 and MC9 in the bottom lamination which is sealed on the bottom surface.

![Moisture Probe MC6](image)

*Figure 6.10. Probe MC6: Experimental vs Numerical results*

![Moisture Probe MC7](image)
MC10 and MC11 are situated in the bottom tensile lamination of each beam. As the bottom of each beam is sealed, the rate of moisture content change is relatively slow in this lamination. As a result, a slowly increasing trend in moisture content is observed during the first four relative humidity cycles after which the cyclic moisture content appears to reach an equilibrium state with repeated cycles. There is no significant influence of the adhesive layer separating the bottom tensile lamination and the lamination above as seen in Figure 6.14 and Figure 6.15. There is also an additional effect due to the rod reinforcement in a proportion of the beams hindering the flow of moisture in this lamination. The reinforcement results in a slower rate of change than that shown in the unreinforced case in Figure 6.14 and Figure 6.15; however, after 5-6 relative
humidity cycles, a relatively consistent mean result is achieved in both unreinforced and reinforced beams.

The experimental results close to the surface have shown good agreement with the numerical model results. At such depths, the boundary conditions have been shown to dominate the numerical response with time. At greater depths, the defined diffusion coefficients influence the internal response of moisture flow in timber. It has been shown that MC2, MC3, MC10 and MC 11 which are located at depths closer to the centre of the beam have resulted in less accurate results leading to the conclusion that the diffusion coefficients may require further examination. It was previously observed that the adsorption phases of MC3 in Figure 6.7 had shown good agreement with the numerical model; however, experimentally a more rapid change in moisture content was observed during the desorption phase. Investigating this theory, faster diffusion coefficients were
implemented into the moisture diffusion model and better agreement was observed at greater depths. However, this negatively affected the results closer to the surface. Given the importance of moisture content change in the development of swelling/shrinkage strains and mechano-sorptive creep strains, it was deemed more important to describe the change in moisture content closer to the surface of the beam rather than that close to the centre of the beam and the original diffusion coefficients have been adopted and implemented in all models. However, an additional test programme to study the moisture diffusion is recommended based on in-grade timber specimens to account for the influence of knots and other anatomical features in order to identify separate and more appropriate diffusion coefficients for the adsorption and desorption phases.

6.3 **Coupled Moisture-Displacement Model**

6.3.1 Introduction

Under changing moisture content conditions, timber swells and shrinks. Such moisture-induced swelling/shrinkage strains may be simulated utilising the thermo-hygro analogy described in Section 6.1, and the coupled thermal-displacement transient analysis capability in Abaqus. In this section, the developed model predicts mechanical strains associated with timber element under changing relative humidity conditions. The moisture diffusion model presented in Section 6.2 is utilised in this coupled model to describe the moisture content change, and the definition of the elastic material properties of the timber along with the direction dependent shrinkage/swelling coefficients are defined to describe the associated mechanical strains. The experimental test previously presented in Section 4.6 is then simulated in the developed coupled moisture-displacement model. The numerical results are then compared to the experimental test results previously presented in Section 4.6.2.

6.3.2 Elastic Material Properties

In this study, timber is modelled as an orthotropic elastic material. The fast-grown Irish timber used in this project was graded to strength class C16. Eurocode EN 338 (CEN 2009b) provides a mean value of 8000 MPa for the elastic modulus of C16 timber in the longitudinal direction. In this study, the experimentally determined elastic modulus in the longitudinal direction from the short-term experimental tests in Chapter 3 is used.
Experimentally the elastic moduli in the radial and tangential directions were not measured; however, Bodig & Jayne (1993) expressed a general relationship between the elastic modulus, $E$ and shear modulus, $G$ in the three material directions. These ratios are reproduced in Equations [6.4]-[6.6].

$$E_L : E_R : E_T \approx 20 : 1.6 : 1$$  \[6.4\]

$$G_{LR} : G_{LT} : G_{RT} \approx 10 : 9.4 : 1$$  \[6.5\]

$$E_L : G_{LR} \approx 14 : 1$$  \[6.6\]

These ratios are widely accepted for modelling the orthotropic properties of timber. However, it is noted that these ratios are not without their inaccuracies. The size of a timber element and the location it was cut from a log are factors in the accuracy of the assumption. More information on the timber material properties is given in Table 6.4 on page 219 in Section 6.5. Additionally, these material properties are also dependent on the moisture content, temperature and density of the timber. Equations [6.7] and [6.8] are implemented to adjust the elastic and shear modulus properties when the environmental conditions are different from the prescribed, reference conditions (Santaoja et al. 1991).

$$E_i = E_{i,\text{ref}} \left\{1 + \alpha_1 (\rho - \rho_{\text{ref}}) + \alpha_2 (T - T_{\text{ref}}) + \alpha_3 (u - u_{\text{ref}})\right\}$$  \[6.7\]

$$G_{ij} = G_{ij,\text{ref}} \left\{1 + \alpha_1 (\rho - \rho_{\text{ref}}) + \alpha_2 (T - T_{\text{ref}}) + \alpha_3 (u - u_{\text{ref}})\right\}$$  \[6.8\]

$E_{i,\text{ref}}$ and $G_{ij,\text{ref}}$ are the reference elastic modulus and shear modulus, respectively, which correspond to the reference temperature ($T_{\text{ref}}$), moisture content ($u_{\text{ref}}$) and density ($\rho_{\text{ref}}$); $T$, $u$ and $\rho$ refer to the current temperature, moisture content and density, respectively, at any given time step and $\alpha_1 = 0.0003$, $\alpha_2 = -0.007$ and $\alpha_3 = -2.6$ are material constants provided by Santaoja et al. (1991) and Mirianon et al. (2008).

### 6.3.3 Swelling/shrinkage Coefficients

Using the thermo-hygro analogy, the swelling/shrinkage coefficients define the development of strain in the coupled moisture-displacement model, just as the thermal expansion coefficients define the development of strain in a coupled temperature-displacement analysis. The experimentally determined swelling/shrinkage coefficients in
Section 5.5.2 and Table 5.4, specific to Irish grown Sitka spruce, are used in this model. The radial, tangential and longitudinal swelling/shrinkage coefficients have been determined and can be applied in a cylindrically coordinate system to accurately model the orthotropic nature of timber and the direction-dependent swelling/shrinkage characteristics.

6.3.4 Coupled Moisture-Displacement Model Validation

In this section, the simulation of the experimental tests presented in Section 4.6 is described. The elastic material properties, swelling/shrinkage coefficients and the directional dependent diffusion coefficients described previously are implemented in a 3-dimensional coupled moisture-displacement model.

6.3.4.1 Model Geometry

A 3-dimensional coupled moisture-displacement model has been developed to simulate moisture-induced swelling/shrinkage strain within non-loaded glued laminated beams incorporating material properties representative of fast-grown Irish Sitka spruce. To reduce the computational time, it is assumed that all material properties and orientation remain uniform in the longitudinal direction. This permits a shortened slice of the beam to be modelled (Hassani et al. 2015). The boundary conditions allow for free expansion and contraction. A 60 mm slice of the entire beam comprising of four laminations each measuring 98 mm x 125 mm is modelled as shown in Figure 6.16. The glue-lines have been omitted in this coupled moisture-displacement model as they have been found to have little influence on the moisture diffusion. This coupled moisture-displacement model (Mesh 4) consists of 9500 8-noded coupled thermal-displacement C3D8T elements. The mesh size was determined from a mesh sensitivity study (Table 6.3).

<table>
<thead>
<tr>
<th>No. of Elements</th>
<th>Mesh 1</th>
<th>Mesh 2</th>
<th>Mesh 3</th>
<th>Mesh 4</th>
<th>Mesh 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Strain</td>
<td>417.30</td>
<td>424.89</td>
<td>433.51</td>
<td>436.36</td>
<td>436.69</td>
</tr>
<tr>
<td>Perpendicular Strain</td>
<td>3703.36</td>
<td>3740.78</td>
<td>3767.89</td>
<td>3774.88</td>
<td>3776.04</td>
</tr>
</tbody>
</table>

* Strain at 5 weeks

Each of the four laminations is considered as a separate section with individual material properties and cylindrically orientated coordinate systems. The beam was subjected to variable relative humidity cycles similar to that reported in Section 4.3.4. The humidity
cycles were implemented within Abaqus using a user defined DFLUX subroutine as described previously.

Figure 6.16. 60 mm slice through glued laminated beam, a) Timber section, b) Model mesh, c) moisture content distribution and d) moisture-induced longitudinal (S33) strain distribution
In Figure 6.16, the idealised symmetric case where the pith of each timber lamination is situated in the centre along the top surface of each lamination is presented. The moisture content distribution at 4 weeks can be seen in Figure 6.16c and the corresponding strain distribution can be seen in Figure 6.16d where longitudinal strain discontinuities can be seen along the bond lines as expected due to the different material properties and the change in material orientation at this junction. This was also observed numerically when examining the radial and tangential strains. These strain discontinuities can lead to the development of cracks between adjacent laminations due to the low tensile strength perpendicular to grain with repeated moisture cycling. This model assumes an ideal symmetric geometry cylindrically orientated around the pith. This assumption is adequate in practical circumstances; however, the position of the pith within the timber model has been shown to have a significant influence on the strain distribution through each lamination. In a numerical study by Angst & Malo (2012), 35-50% lower stresses were observed when the piths were located outside the laminations compared to a configuration where piths were located in the laminations. For this reason, a case study of a particular beam (Beam 37) was chosen and this was mapped and replicated as closely as possible within the finite element model. This mapping process can be seen in Figure 6.17. The tangential material orientation input within the Abaqus finite element environment can be seen to be in agreement with the mapped beam (Figure 6.17c). This particular beam was chosen to be representative of all four beams measuring moisture-induced strains on non-loaded beams in the test programme due to the similar orientation of the individual laminations and the lack of a pith in any lamination within the experimental test specimens.
6.3.4.2 Coupled Moisture-Displacement Model Results

The results from Section 4.6 have shown relatively consistent and repeatable fluctuations in moisture content and swelling/shrinkage strain with repeated cycles. The strains were measured in the longitudinal and perpendicular to grain directions. The parallel to grain results from the numerical model are presented in Figure 6.18 together with the experimental data previously presented in Section 4.6.2. There is no difference between the mapped model and the idealised model predictions of strain in the longitudinal direction as the mapping procedure presented in Figure 6.17 only alters the radial and tangential material orientation. The relatively high experimentally determined longitudinal swelling/shrinkage coefficient (0.0122) appears to adequately predict the response of the timber in this cyclic climate over the 75 week test period. Although this swelling/shrinkage coefficient is associated with a high standard deviation (Table 5.5), the average value is deemed suitable when describing the swelling/shrinkage characteristics of fast-grown Sitka spruce in a variable climate.
Figure 6.18. Parallel to grain strain: Numerical result vs experimental result

The perpendicular to grain results from the numerical model are presented in Figure 6.19 together with the experimental data previously presented in Section 4.6.2. The numerical strain measurement presented is an average of the simulated results on the left and right side of the beam model. The mapping process has been shown to produce significantly more accurate results when compared to the results of the idealised model. The perpendicular to grain strains have been adequately predicted using the experimentally determined radial and tangential swelling/shrinkage coefficients of 0.1371 and 0.2525, respectively. The variability inherent within timber elements is clearly visible in the presented experimental strain results. The model has been shown to predict with good accuracy, the moisture-induced swelling/shrinkage strain development in the longitudinal and tangential directions in Irish Sitka spruce subjected to repeated relative humidity cycles.

Figure 6.19. Perpendicular to grain strain: Numerical result vs experimental result
6.4 HYGRO-MECHANICAL CREEP MODEL

6.4.1 Introduction

The moisture content distribution and moisture-induced swelling/shrinkage strains have been accurately modelled using Abaqus finite element software. However, under long-term loading conditions, additional viscoelastic creep takes place and additional mechano-sorptive creep effects result due to the interaction between long-term loads and changing moisture content. A mechano-sorptive creep modelling capability is not available in Abaqus; however, with the use of a user-defined UMAT subroutine, this material behaviour may be defined.

In this section, the constitutive model is established to simulate the long-term creep of timber beams in both constant and variable climates. The formulation presented follows the approach implemented by Santaoja et al. (1991), Mårtensson (1994b), Mirianon et al. (2008) and Fortino et al. (2009) but is adapted to represent Irish-grown Sitka spruce using experimental data. A 3-dimensional finite element model is implemented to determine the total strain experienced in timber elements when subjected to mechanical stress and simultaneous moisture content changes with time. This simulated total strain can be subdivided into five separate strain components. These strain components are solved for in an incremental analysis with time. The total strain $\varepsilon_T$, is given in Equation [6.9].

$$
\varepsilon_T = \varepsilon_e + \sum_{i=1}^{k} \varepsilon_{ve} + \varepsilon_{ms} + \varepsilon_{ms,irr} + \varepsilon_s
$$

where:

- $\varepsilon_T$ = Total strain component
- $\varepsilon_e$ = Elastic strain component
- $\varepsilon_{ve}$ = Viscoelastic strain component
- $\varepsilon_{ms}$ = Mechano-sorptive strain component
- $\varepsilon_{ms,irr}$ = Irrecoverable mechano-sorptive strain component
- $\varepsilon_s$ = Swelling/shrinkage strain component

The $k$ term denotes the number of viscoelastic Kelvin elements used in the formulation of the viscoelastic creep component as described in Section 6.4.4.
6.4.2 Elastic Strain Component

The elastic component ($\varepsilon_c$) follows the generalised Hooke’s law as seen in Equation [6.10] where $\sigma$ is the stress and $C_e$ is the elastic compliance matrix of an orthotropic material. In 3-dimensional matrix form, this equation defines the orthogonal elastic stress-strain behaviour of timber (Equation [6.11]).

$$\varepsilon_c = C_e \sigma \quad \quad [6.10]$$

$$C_e = \begin{bmatrix}
\frac{1}{E_R} & -\frac{v_{TL}}{E_T} & -\frac{v_{LR}}{E_L} & 0 & 0 & 0 \\
\frac{v_{RT}}{E_R} & \frac{1}{E_T} & -\frac{v_{LT}}{E_L} & 0 & 0 & 0 \\
-\frac{v_{RL}}{E_R} & -\frac{v_{TL}}{E_T} & \frac{1}{E_L} & 0 & 0 & 0 \\
0 & 0 & 0 & \frac{1}{G_{RT}} & 0 & 0 \\
0 & 0 & 0 & 0 & \frac{1}{G_{RL}} & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{1}{G_{TL}}
\end{bmatrix} \quad \quad [6.11]$$

The elastic compliance matrix is symmetric and the Poisson’s ratios are related according to Equation [6.12].

$$v_{RL} = \frac{E_R}{E_L} v_{LR} \quad v_{TL} = \frac{E_T}{E_L} v_{LT} \quad v_{TR} = \frac{E_T}{E_R} v_{RT} \quad [6.12]$$

The subscripts $L$, $R$ and $T$ refer, respectively, to the directions of assumed elastic symmetry, longitudinal, radial and tangential directions. $E$ and $G$ represent the elastic modulus and shear modulus, respectively. Bodig & Jayne (1993) expressed a general relationship between the moduli $E$ (elastic) and $G$ (shear) in the three orientations as presented in Equations [6.4]-[6.6]. These material properties of timber are not constant and are dependent on the moisture content, temperature and density of the timber as shown previously in Section 6.3.2. Equation [6.7] and [6.8] previously presented in Section 6.3.2 are also implemented to adjust the elastic and shear modulus properties when the environmental conditions are different from the prescribed, reference conditions (Santaoja et al. 1991).
6.4.3 Swelling/shrinkage Strain Component

The swelling/shrinkage strain component \( (\varepsilon_s) \) occurs within the timber during moisture content change. This change in strain depends on the material direction and magnitude of the change in moisture content. The increment in \( \varepsilon_s \) due to a moisture content change, \( \Delta u \) is described by Equation [6.13].

\[
\Delta \varepsilon_s = \alpha_u \Delta u
\]  \[6.13\]

Where the moisture swelling/shrinkage coefficient \( \alpha_u \) is described by Equation [6.14]

\[
\alpha_u = \begin{bmatrix}
\alpha_R \\
\alpha_T \\
\alpha_L \\
0 \\
0 \\
0
\end{bmatrix}
\]  \[6.14\]

The values of the swelling/shrinkage coefficient \( (\alpha_u) \) used in the study were determined experimentally in Section 5.5.2. It has been found that different loading conditions (tension, compression or bending) result in different longitudinal swelling/shrinkage behaviour of timber. Wood tends to swell/shrink less when in tension and swell/shrink more in compression (Hunt & Shelton 1988). Toratti (1992b) implements an adapted formula (Equation [6.15]) to account for variations in longitudinal strain when in a state of tension and compression as is the case in bending.

\[
\Delta \varepsilon_{s,L} = \left( \alpha_L - \beta \varepsilon_{mech,L} \right) \Delta u
\]  \[6.15\]

where:

\( \beta \) = Material constant, 1.3 (Mirianon et al., 2008)

\( \varepsilon_{mech,L} \) = mechanical strain vector in the longitudinal direction.

This is applied to the longitudinal direction in the developed model. A similar approach has been used in Mohager & Toratti (1992), Hanhijarvi (1995) and Davids et al. (1999).
6.4.4 Viscoelastic Strain Component

The additional deformation with time in timber elements loaded under constant stress is known as viscoelastic creep. In this study, the viscoelastic strain component \( \varepsilon_{ve} \) is an extension of the one-dimensional model presented by Toratti (1992a) consisting of a sum of Kelvin type elements as illustrated in Figure 6.20.

![Figure 6.20. A system of four Kelvin elements connected in series: Each Kelvin element comprises a spring (E) and viscous dashpot (\( \eta \)) in parallel.](image)


\[
\dot{\varepsilon}_{ve,i} + \frac{1}{\tau_i} \varepsilon_{ve,i} = \frac{1}{\tau_i} \mathbf{C}_{ve,i}^{-1} \sigma
\]

where:
- \( \varepsilon_{ve,i} \) = the viscoelastic strain rate = \( \frac{\partial}{\partial t} \varepsilon_{ve,i} \)
- \( \mathbf{C}_{ve,i} \) = the viscoelastic compliance matrix of the \( i \)th Kelvin element
- \( \sigma \) = the total stress of the \( i \)th Kelvin element
- \( \tau_i \) = the retardation or characteristic time of the \( i \)th Kelvin element

It has been shown by Hanhijarvi & Mackenzie-Helnwein (2003) and Fortino et al. (2009) that for stress driven problems, the solution to the rate equation (Equation [6.16]) can be written as,

\[
\varepsilon_{ve,i,n+1} = \varepsilon_{ve,i,n} \exp \left( \frac{-\Delta t}{\tau_i} \right) + \int_{t_n}^{t_{n+1}} \frac{\mathbf{C}_{ve,i}^{-1} \sigma(t)}{\tau_i} \exp \left( \frac{(t_{n+1} - t_n)}{\tau_i} \right) dt \quad [6.17]
\]

where:
- \( \varepsilon_{ve,i} \) = viscoelastic strain of the \( i \)th Kelvin element
- \( \sigma(t) \) = the total stress of the \( i \)th Kelvin element at time \( t = t_n \) or \( t = t_{n+1} \)
- \( \Delta t \) = the time increment \( (t_{n+1} - t_n) \)
By integrating this expression for viscoelastic strain (Equation [6.17]) over time for the \( i \)th Kelvin element, the following increment of elemental viscoelastic strain is obtained (Mackenzie-Helnwein & Hanhijärvi 2003; Mirianon et al. 2008).

\[
\Delta \epsilon_{ve,i,n+1} = C_{ve,i} T_{i,n+1} \left( \frac{\Delta t}{\tau_i} \right) \Delta \sigma_{n+1} - \left( \epsilon_{ve,i,n} - C_{ve,i} \sigma_n \right) \left( 1 - \exp \left( - \frac{\Delta t}{\tau_i} \right) \right) \tag{6.18}
\]

where:

\[
T_{i,n+1} \left( \frac{\Delta t}{\tau_i} \right) = 1 - \frac{\tau_i}{\Delta t} \left( 1 - \exp \left( - \frac{\Delta t}{\tau_i} \right) \right) \tag{6.19}
\]

\[
C_{ve,i} = J_{ve,i} C_e \tag{6.20}
\]

where \( J_{ve,i} \) represents the dimensionless viscoelastic compliance ratio of the \( i \)th Kelvin element. This dimensionless viscoelastic compliance ratio is a constant of proportionality between the viscoelastic compliance matrix \( (C_{ve,i}) \) and the elastic compliance matrix \( (C_e) \). The same ratios have been applied in the longitudinal, radial and tangential direction to simplify the model (Fortino et al. 2009). Four Kelvin elements in series with different retardation (or characteristic) times describe the viscoelastic creep function. The compliance parameters of this viscoelastic creep function were validated by comparing the numerical result to the unreinforced experimental results performed in the constant climate condition.

6.4.5 Combined Mechano-sorptive Strain Component

The total mechano-sorptive strain component \( (\epsilon_{ms,\text{Total}}) \) consists of two individual components. Both of these components occur under coupled stress and moisture content change associated with relative humidity variations in the surrounding environment.

\[
\epsilon_{ms,\text{Total}} = \epsilon_{ms,\text{irr}} + \epsilon_{ms} \tag{6.21}
\]

where:

\[
\epsilon_{ms,\text{irr}} = \text{irrecoverable mechano-sorptive strain component}
\]

\[
\epsilon_{ms} = \text{mechano-sorptive strain component}
\]
6.4.5.1 Irrecoverable Mechano-sorptive Strain Component

The increment of the irrecoverable component of mechano-sorptive strain is calculated using Equation [6.22] (Svensson & Toratti 2002; Fortino et al. 2009). The irrecoverable mechano-sorptive component is independent of stress rate and is a permanent deformation proportional to the stress state and change in moisture content (Hanhijarvi 1995).

\[
\Delta \varepsilon_{ms,irr,n+1} = C_{ms,irr} \sigma_n | \Delta U |
\]

where:
- \( \Delta \varepsilon_{ms,irr} \) = irrecoverable mechano-sorptive strain increment
- \( C_{ms,irr} \) = irrecoverable mechano-sorptive compliance matrix (Equation [6.23])
- \( \sigma_n \) = stress state at time \( t=t_n \)
- \( \Delta U \) = increase to moisture content not previously attained.

The moisture content increment (\( \Delta U \)) refers to the moisture levels not yet attained during previous moisture cycles. The value of \( \Delta U \) will be zero for all moisture levels previously attained during a loaded state. The irrecoverable mechano-sorptive matrix previously presented by Fortino et al. (2009) was concerned with irrecoverable deformations in the radial and tangential directions. It was shown in the experimental strain results presented in Section 4.5, that the development of irrecoverable mechano-sorptive creep strain during moisture content change to levels not previously attained can also been observed in the longitudinal direction. As a result, the irrecoverable mechano-sorptive matrix must be modified to account for irrecoverable strains in the longitudinal direction. This is achieved with the introduction of the term, \( m_L \), in addition to the \( m_v \) term previously provided by Fortino et al. (2009). The modifications have been highlighted in Equation [6.23]. Through careful calibration of this irrecoverable strain component in the respective material directions, the large increase associated with the first change in moisture content beyond a previously attained value described by Hunt (1982) has been satisfactorily predicted. As a result, the value of the irrecoverable coefficient, \( m_v \), presented by Fortino et al. (2009) has also been amended.
where:

\( m_L \) = Longitudinal irrecoverable strain coefficient

\( m_r \) = Radial/tangential irrecoverable strain coefficient

### 6.4.5.2 Mechano-sorptive Strain Component

When describing the mechano-sorptive strain associated with repeating cycles of relative humidity, the schemes previously implemented by Santaoja et al. (1991), Mårtensson (1994b) and Ormarsson (1999) have been implemented. The solution is found using the absolute value of moisture content, which is assumed to be constant over the time increment.

\[
\Delta \varepsilon_{ms,n+1} = C_{ms} \sigma_n |\Delta u|
\]  

where:

\( \Delta \varepsilon_{ms} \) = mechano-sorptive strain increment

\( C_{ms} \) = mechano-sorptive compliance matrix (Equation [6.25])

\( \sigma_n \) = stress state at time \( t=n \)

\( |\Delta u| \) = the moisture increment \( |u_{n+1} - u_n| \)

The elemental mechano-sorptive compliance matrix assumes the given form in Equation [6.25] (Santaoja et al. 1991; Mårtensson 1994b; Ormarsson 1999).
The coefficients within the matrix \( C_{ms} \) are based on the mechano-sorptive compliance coefficients \( (m_{ms,i}) \) in the respective material directions and, using the ratios \( v_{ms,i} \), describe the coupling of the mechano-sorptive strain between the different directions. The values used have been based on previous research by Santaoja et al. (1991) and Ormarsson et al. (1999) and finally matched to experimental test results.

\[
C_{ms} = \begin{bmatrix}
  m_{msR} & -m_{msL}v_{msRT} & -m_{msL}v_{msLR} & 0 & 0 & 0 \\
  -m_{msR}v_{msRL} & m_{msL} & -m_{msL}v_{msLT} & 0 & 0 & 0 \\
  -m_{msR}v_{msRL} & -m_{msL}v_{msLT} & m_{msL} & 0 & 0 & 0 \\
  0 & 0 & 0 & m_{msRT} & 0 & 0 \\
  0 & 0 & 0 & 0 & m_{msRL} & 0 \\
  0 & 0 & 0 & 0 & 0 & m_{msIL} \\
\end{bmatrix}
\] [6.25]

\[\Delta \varepsilon_T = \Delta \varepsilon_v + \Delta \varepsilon_s + \Delta \varepsilon_{ms,ir} + \Delta \varepsilon_{ms} + \Delta \varepsilon_{ve} \] [6.26]

The implementation of this equation has been well described by Fortino et al. (2009). The computational analysis of creep strain in wood is solved in an incremental analysis such that at the beginning of the time step \( t=t_{n+1} \), the moisture content \( u_{n+1} \), the moisture increment \( \Delta u_{n+1} \) and the total strain increment \( \Delta \varepsilon_{T,n+1} \) have been calculated by the moisture-displacement analysis in the Abaqus software. These values stored in Abaqus are called into the UMAT subroutine when required. The values of total stress and elemental viscoelastic strain at the beginning of the current time step \( t=t_{n+1} \), have been provided by the UMAT subroutine at the end of the previous time step \( t=t_n \).

In the computation of the total strain increment \( \Delta \varepsilon_T \), the individual components are partitioned into two states, the first of which, \( \Delta \varepsilon_T^1 \), depends on the state of the stress at the beginning of the time increment and the second of which, \( \Delta \varepsilon_T^2 \), is dependent on the increment of stress during the time increment, \( At \) such that:
\[ \Delta \varepsilon_T = \Delta \varepsilon_T^1 + \Delta \varepsilon_T^2 \]  \hfill [6.27]

The elastic strain increment (\( \Delta \varepsilon_e \)) may also be separated into two separate states. One of which is due to an increment of the elastic compliance matrix at the beginning of the time increment due to its dependency on moisture content (\( \Delta \varepsilon_e^1 \)) and the second of which is due to a stress increment to be calculated at the current time step (\( \Delta \varepsilon_e^2 \)) as seen in Equation [6.28].

\[ \Delta \varepsilon_{e,n+1} = \Delta \varepsilon_e^1 + \Delta \varepsilon_e^2 \]  \hfill [6.28]

where:

\[ \Delta \varepsilon_e^1 = \Delta C_e \sigma_{n+1} \]  \hfill [6.29]

\[ \Delta \varepsilon_e^2 = C_e \Delta \sigma_{n+1} \]  \hfill [6.30]

Implementing the moisture dependent material correction factors given in Equation [6.7] and Equation [6.8], \( \Delta \varepsilon_e^1 \) can be given by Equation [6.31] (Santaoya et al. 1991; Mirianon et al. 2008). In the solution, the temperature, moisture content and density are assumed to have a constant value over the time increment.

\[ \Delta \varepsilon_e^1 = -C_e \sigma_{n+1} \frac{\alpha_2 (T - T_{ref}) + \alpha_3 (u - u_{ref})}{1 + \alpha_1 (\rho - \rho_{ref}) + \alpha_2 (T - T_{ref}) + \alpha_3 (u - u_{ref})} \]  \hfill [6.31]

The swelling/shrinkage component and the mechano-sorptive components are as described in Equations [6.13]-[6.15] and Equations [6.21]-[6.25], respectively. In the determination of these strain components, the value of moisture content change is also assumed to be constant during the time increment.

The viscoelastic strain component, previously described in Equations [6.16]-[6.20], is similar to \( \Delta \varepsilon_e^2 \) in that it is dependent on the stress increment (\( \Delta \sigma_{n+1} \)) rather than the current stress state (\( \sigma_{n+1} \)). At the beginning of each time increment, the stress state (\( \sigma_{n+1} \)) and total strain increment (\( \Delta \varepsilon_{T,n+1} \)) are known for the current time step. The objective of the UMAT is to update the stress, viscoelastic strain and the Jacobian matrix at the end of each time increment.
As seen previously in Equation [6.27] the total strain increment \( \Delta \varepsilon_T \) is divided into two states, those dependent on the stress state and those dependent on the stress increment. Based on Equations [6.10], [6.13], [6.18], [6.22], [6.24], and [6.28], these two states may be written as:

\[
\Delta \varepsilon_T^1 = \Delta \varepsilon_e^1 + \Delta \varepsilon_s + \Delta \varepsilon_{ms} + \Delta \varepsilon_{ms,irr} \tag{6.32}
\]

\[
\Delta \varepsilon_T^2 = \Delta \varepsilon_e^2 + \Delta \varepsilon_{ve} \tag{6.33}
\]

Because the terms in Equation [6.32] are calculated with known variables passed into the UMAT, the calculation of \( \Delta \varepsilon_T^1 \) is relatively simple. By arranging Equation [6.27] to solve for \( \Delta \varepsilon_T^2 \), and substituting Equations [6.32] and [6.33], Equation [6.34] is produced incorporating each strain component.

\[
\Delta \varepsilon_T^2 = \Delta \varepsilon_e - \Delta \varepsilon_e^1 - \Delta \varepsilon_s - \Delta \varepsilon_{ms,irr} - \Delta \varepsilon_{ms} \tag{6.34}
\]

Solving for the strain increment related to the change in stress over the time increment \( \Delta \varepsilon_T^2 \) using Equation [6.34] together with Equation [6.18], the total increment of stress can be calculated as shown in Equation [6.35] at a given time step.

\[
\Delta \sigma_{n+1} = C_T \left( \Delta \varepsilon_T - \Delta \varepsilon_e^1 - \Delta \varepsilon_s - \Delta \varepsilon_{ms,irr} - \Delta \varepsilon_{ms} + \sum_{i=1}^{k} R_{ve,i} \right) \tag{6.35}
\]

where:

\[
R_{ve,j} = \left( \varepsilon_{ve,i,n} - C_{ve,i} \sigma_n \right) \left( 1 - \exp \left( -\frac{\Delta t}{\tau_j} \right) \right) \tag{6.36}
\]

\[
C_T = \left( C_e + \sum_{i=1}^{n} C_{ve,i} T_{i,n+1} \left( \frac{\Delta t}{\tau_i} \right) \right)^{-1} \tag{6.37}
\]

\( R_{ve,i} \) is the individual viscoelastic strain increment calculated in the previous increment \( t=t_n \) and is a function of \( \varepsilon_{ve,i} \) and \( \sigma_n \). In this particular UMAT, it is the sum of each of the four Kelvin elements. \( C_T \) is the tangent operator, which the sum of the elastic and
viscoelastic creep matrices used to calculate the stress increment for a given change in strain.

The stress must be updated at the end each time step, hence the stress at the end of the increment, defined as $\sigma_{n+1,E}$, must be calculated using Equation [6.38]. The change in stress, $\Delta \sigma_{n+1}$ calculated using Equation [6.35], must be added to the stress at the beginning of the time step, defined as $\sigma_{n+1,B}$. A similar calculation must be made for the viscoelastic strain increment as seen in Equation [6.39]. These new quantities (Equations [6.38] and [6.39]) and the tangent operator ($C_T$) are used in the next time step of the analysis.

$$\sigma_{n+1,E} = \sigma_{n+1,B} + \Delta \sigma_{n+1}$$  \hspace{1cm} [6.38]

$$\varepsilon_{we,i,n+1,E} = \varepsilon_{we,i,n+1,B} + \Delta \varepsilon_{we,i,n+1}$$  \hspace{1cm} [6.39]

### 6.5 Hygro-mechanical Creep Model Implementation

In this section, the implementation of the UMAT subroutine in a non-linear coupled hygro-mechanical creep model is described. The numerical results are compared to the experimental creep test results presented in Chapter 4. The numerical model examines the behaviour of timber elements in both constant and variable climate conditions. In the constant climate conditions, the model focuses on elastic deformation of both unreinforced and reinforced beams and long-term viscoelastic creep associated with such beams. The simulated results in the constant climate condition are compared to the mean deflection and creep strain results of the unreinforced Group UC and the reinforced Group RC presented in Section 4.4. In the variable climate condition, the model will simulate the elastic deformation and the long-term viscoelastic creep deformation of both the unreinforced and reinforced beams, but will also include the swelling/shrinkage strains associated with the variable climates and the additional mechano-sorptive creep component. The simulated results of this model are compared to the mean deflection and creep strain results of the unreinforced Group UV and the reinforced Group RV presented in Section 4.5.
Additionally, utilising the model, parametric studies are performed to observe the effect of reinforcement type on the long-term performance of timber elements in a variable climate. The effect of GFRP, AFRP and CFRP rod reinforcement on the long-term performance of such beams is examined and compared to BFRP reinforced beams.

6.5.1 Model Geometry

The geometry and creep test set-up on the glued laminated beam loaded in four-point bending previously described in Section 4.3 is modelled using Abaqus finite element software. As seen in Figure 6.21, half symmetry is utilised allowing half of the 2300 mm long beam to be modelled in order to reduce the computational time. The 98 x 125 mm\(^2\) beam comprises four laminations. The glue-line has been omitted in this mechanical model. Each of the four laminations is assigned material properties in a local cylindrically orientated coordinate system. The reinforced beam model is created similar to that of the unreinforced beam; however, in the bottom tensile lamination, two routed grooves are created to accommodate the two 12 mm diameter BFRP rods and the 2 mm structural epoxy adhesive. Although there are differences in the unreinforced and reinforced models, each model uses 8-noded coupled thermal-displacement C3D8T elements. The respective mesh sizes for the unreinforced and reinforced beam models were determined from mesh sensitivity studies to provide accurate results in a reasonable time frame. The beam is simply supported on 80 mm x 100 mm plates which are free to rotate about their central axis. These plates are modelled using 2-dimensional shell S3 elements. Hard contact is defined between the surface of the beam and the steel plate with a tangential friction coefficient of 0.4. Similar plates are used to apply the dead load in tests. These are also modelled using S3 shell elements and the load is applied as a uniform pressure over the plate area. As in the experimental test, the load applied to the unreinforced beams differed from that applied to the reinforced beams.
Mesh 1 | Mesh 2 | Mesh 3 | Mesh 4 | Mesh 5  
---|---|---|---|---  
No. of Elements | 7950 | 9580 | 12060 | 14950 | 19030  
Deflection (mm)  | 4.18 | 5.90 | 6.79 | 6.85 | 6.90  
* mid-span deflection at 3 weeks

Figure 6.21. Finite element coupled hygro-mechanical creep model (Mesh 4) of the unreinforced glued laminated beam and mesh sensitivity study: The red arrows represent the load point and support point on the top and bottom surface, respectively, and the blue arrows represent the relative humidity load shown on the exposed surfaces.

Regarding the numerical models, a load of 5749 N was applied to the unreinforced beams and a load of 6241 N was applied to the reinforced beams to induce a common maximum compressive bending stress of 8 MPa in each beam.

In both the constant and variable climate condition, the mechanical loading situation and test geometry remain unchanged. In a variable climate, the only addition is a relative humidity load which is applied through the exposed surfaces of the beam. The exposed surfaces include both sides of the glued laminated beam and the top surface. This applies to both the unreinforced and reinforced beam model.

6.5.2 Material Data

The experimentally determined material data for both timber and BFRP are implemented in the numerical models. The timber material data is summarised in Table 6.4. The BFRP rod reinforcement and epoxy adhesive material data implemented into the mechanical model are presented in Table 6.5.
<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
<th>Property Value</th>
<th>Unit</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>E_R</td>
<td>Radial elastic modulus</td>
<td>663.28</td>
<td>MPa</td>
<td>Equation [6.4]-[6.6] (Bodig &amp; Jayne 1993)</td>
</tr>
<tr>
<td>E_T</td>
<td>Tangential elastic modulus</td>
<td>414.55</td>
<td>MPa</td>
<td></td>
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<tr>
<td>E_L</td>
<td>Longitudinal elastic modulus</td>
<td>9222.30</td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td>\nu_{RT}</td>
<td>Poisson's ratio radial-tangential</td>
<td>0.558</td>
<td>-</td>
<td>(Santaoja et al. 1991; Mirianon et al. 2008; Fortino et al. 2009)</td>
</tr>
<tr>
<td>\nu_{RL}</td>
<td>Poisson's ratio radial-longitudinal</td>
<td>0.038</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>\nu_{TL}</td>
<td>Poisson's ratio tangential-longitudinal</td>
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<td>-</td>
<td></td>
</tr>
<tr>
<td>G_{RT}</td>
<td>Shear modulus radial-tangential</td>
<td>65.87</td>
<td>MPa</td>
<td>Equation [6.4]-[6.6] (Bodig &amp; Jayne 1993)</td>
</tr>
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<td>Shear modulus radial-longitudinal</td>
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<td>MPa</td>
<td></td>
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<td>Alpha density</td>
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Table 6.5. BFRP and epoxy adhesive material data

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6.5.3 Constant Climate Model Results

In this section, the results of the long-term numerical simulation of both the unreinforced beam model and reinforced beam model in a constant climate are compared to the mean experimental test results of Group UC and Group RC, respectively. As described in Section 4.3.4, the constant climate condition is at a relative humidity of 65% ± 5% throughout and at a constant temperature of 20°C ± 2°C. In Figure 6.22, an example of the longitudinal (S33) bending stress distribution in the unreinforced beam can be seen at week 3.

![Creep model of an unreinforced glued laminated beam (Week 3) (Longitudinal stress in Pa)](image)
6.5.3.1 Unreinforced Creep Model Results

The total vertical deflection of the unreinforced numerical model is compared to the mean experimental data of Group UC in Figure 6.23. The model predicted the short-term behaviour well with an experimental mean initial elastic deflection of 6.27 mm compared to the numerical model result of 6.09 mm. Over the 75 week test period, it can also be seen that the total long-term vertical deflection of the unreinforced beams are simulated accurately with an experimental mean result of 8.03 mm compared to the simulated result of 7.90 mm.

![Figure 6.23. Mean deflection result of Group UC vs the numerical result](image)

To further examine the long-term deformation, the relative creep of both the numerical simulation and mean experimental data are presented to focus solely on the additional viscoelastic creep occurring after the initial elastic loading. The mean experimental relative creep as defined in Equation [4.4] is presented in Figure 6.24 and compared to the numerical result. The mean relative creep has been modelled accurately over the 75 week period in unreinforced beams with an experimental mean result of 1.292 compared to a numerical relative creep result of 1.296. It is important to note, the viscoelastic compliance coefficients defined in Table 6.4 and used in this unreinforced model have been matched to the mean experimental deflection data of the unreinforced beams.
Figure 6.24. Mean unreinforced Group UC relative creep result vs the numerical Group UC result

It was important to accurately define these viscoelastic compliance coefficients of the unreinforced beams prior to simulating creep in beams reinforced with BFRP rod reinforcement. The numerical model has demonstrated the ability to predict long-term deflection behaviour of timber beams in a constant climate.

Figure 6.25. Mean unreinforced experimental vs numerical longitudinal strain result
The mean total longitudinal strain measured on the tension and compression faces of the unreinforced beams have also been examined and are compared to the numerical model in Figure 6.25. It can be seen that the results of the numerical model agree well with the mean experimental strain results measured on the tension and compression faces. The initial mean elastic strain has been accurately predicted on both the tension and compression faces of the unreinforced beams. On the tension face, the experimental result of 691 με compared well to the numerical result of 697 με. On the compression face the experimental result of -715 με compared well to the numerical result of -716 με. The model also appears to accurately predict the viscoelastic strain on the tension and compression faces. After 75 weeks, on the tension face, the experimental result of 853 με compared well to the numerical result of 904 με and on the compression face the experimental result of -864 με compared well to the numerical result of -929 με. To focus the long-term performance of the unreinforced specimens, the mean initial elastic deformation is subtracted from the mean total strain, providing the additional viscoelastic strain component with time. A similar procedure is performed with the results of the numerical model and presented in Figure 6.26.

![Figure 6.26. Mean unreinforced Group UC experimental vs numerical longitudinal creep strain result](image)

The numerical model has predicted the additional mean viscoelastic strain development on both the tension and compression faces of the unreinforced beams. In the longitudinal
direction, the total viscoelastic strain component is relatively small with an experimental result of 162 με on the tension face and -149 με on the compression face after 75 weeks. The numerical model has been shown to over estimate the strain component over the 75 week period with a numerical result of 207 με and -213 με on the tension and compression face, respectively. However, given the experimental difficulty associated with measuring such strain components, the result can be described as accurate and the slight over estimation has been deemed conservative in the prediction of long-term strains.

6.5.3.2 Reinforced Creep Model Results

In the reinforced creep model, the same timber material properties used to model the unreinforced beam have been used, in addition to the reinforcement and adhesive material properties, to predict the total deflection and longitudinal strain development in reinforced beams. In Figure 6.27, the numerical longitudinal stress (S33) result of the reinforced beam model at week 3 can be seen.

![Figure 6.27. Creep model of a reinforced glued laminated beam (Week 3) (Longitudinal stress in Pa)](image)

The longitudinal stress in the BFRP reinforcement is approximately 6 times higher than the maximum tensile timber stress. As very little detailed information can be seen in Figure 6.27, the stress distribution at mid-span through the cross section is presented in
Figure 6.28. In Figure 6.28a, the total stress in the cross section can be seen. The stress is highest in the BFRP rod reinforcement with a maximum stress of 33.96 MPa indicated by the numerical model at 3 weeks. In Figure 6.28b, the rod reinforcement and epoxy adhesive are removed to focus on the stress distribution in the timber material. When examining the numerical stress result, a maximum compression bending stress of -7.83 MPa was observed on the compression face of the reinforced timber beam. This is in agreement with the maximum compressive bending stress on 8 MPa implemented on all reinforced and unreinforced beams in the experimental creep tests in Chapter 4.

![Stress Distribution Images]

Figure 6.28. Creep model of a reinforced glued laminated beam (Week 3): a) Longitudinal stress in reinforcement, b) Longitudinal stress in timber (Longitudinal stress in Pa)

The simulated numerical deflection result for the reinforced beam model is compared to the mean experimental result of Group RC in Figure 6.29. The addition of the BFRP reinforcement has resulted in a reduced initial elastic deflection and reduced long-term deflection after 75 weeks. This reduction in initial elastic deflection as a result of the reinforcement has been accurately predicted using the model. The mean experimental elastic deflection of 5.72 mm was previously observed and the numerical model of the reinforced beam resulted in an elastic deflection of 5.76 mm. After this point, the numerical model simulates slightly reduced creep behaviour when compared to the experimental result; however, the experimental results have still been accurately predicted with a numerical result of 7.10 mm compared to the experimental mean result of 7.22 mm after 75 weeks.
Figure 6.29. Mean deflection result of Group RC vs the numerical result

The simulated relative creep result is compared to the mean experimental relative creep result of the reinforced beams in Figure 6.30. The initial relative creep behaviour of the numerical and experimental result agree well with one another over the first 5 weeks. This period is associated with relatively rapid creep deformation and the simulated model has described this behaviour accurately. After this period the numerical relative creep results tends to predict less relative creep deflection than has been presented experimentally.

Figure 6.30. Mean reinforced Group RC relative creep result vs the numerical Group RC result
There was a slight but statistically insignificant reduction in the experimental relative creep of the reinforced Group RC beams when compared to the unreinforced Group UC beams in Chapter 4. The model has predicted this slight reduction in relative creep in the reinforced beams when compared to unreinforced beams. After 75 weeks the numerical result of 1.246 compared well to the experimental result of 1.275 demonstrating the ability to accurately predict the relative creep behaviour of reinforced timber elements.

The total measured longitudinal strain on reinforced beams is compared to that of the numerical model on the tension and compression faces in Figure 6.31. On the tension face, the elastic strain of 597 με simulated in the model is slightly higher than the elastic strain of 565 με determined experimentally. The simulated longitudinal strain results on the tension face have demonstrated the beneficial reduction in elastic strain due to the BFRP rod reinforcement in the reinforced beam when compared to that unreinforced beam. On the compression face, the numerical result of -718 με compares well to the experimental result of -670 με. Although the elastic strain on the compression face is slightly larger than that measured experimentally, the difference is not significant.

Figure 6.31. Mean reinforced Group RC experimental vs numerical longitudinal strain result
To focus the long-term performance of the reinforced specimens, the mean initial elastic deformation is subtracted from the mean total strain, providing the additional viscoelastic strain component with time. A similar procedure is performed with the results of the numerical model and presented in Figure 6.32. Experimentally, a significant reduction in viscoelastic creep was observed on the tension face of reinforced beams when compared to the same measure on unreinforced beams. This behaviour has been successfully simulated in the numerical model. After 75 week, the numerical viscoelastic strain result of 115 με compares well to the 94 με determined experimentally on the tension face as seen in Figure 6.32. On the compression face, the numerical model has been shown to over estimate the viscoelastic strain component over the 75 week period with a numerical result of -186 με compared to an experimental result of -122 με. Similarly to the results presented in the unreinforced beam, the over estimation of the results are deemed conservative in the prediction of the longitudinal viscoelastic strain behaviour.

![Figure 6.32. Mean reinforced Group RC creep strain result vs the numerical result](image)

The reinforced beam model has adequately simulated the experimental deflection and strain results in a constant climate condition. The addition of the BFRP rod reinforcement into the model has been shown to simulate the beneficial reduction of longitudinal strain on the tension face in reinforced beams.
6.5.4 Variable Climate Model Results

In this section the long-term results of both the unreinforced beam model and reinforced beam model in a variable climate are compared to the experimental results previously presented in Chapter 4. The variable climate as described in Section 4.3.4 is implemented in a DFLUX subroutine. In the variable climate, changes in moisture content result in additional strains as a result of swelling and shrinkage strains and mechano-sorptive creep strains. These additional strain components are calculated in the user-defined UMAT subroutine and are dependent on stress and change in moisture content in the beam. The numerical results of the unreinforced and reinforced beam models are compared to the mean experimental results of Group UV and Group RV, respectively, presented in Chapter 4.

6.5.4.1 Unreinforced Creep Model Results

In a variable climate, the change in relative humidity has been shown to have a significant effect on the creep behaviour on unreinforced beams due to changing moisture content. The total vertical deflection of the unreinforced numerical model is compared to the mean Group UV experimental data over the 75 week test period in Figure 6.33. It can be seen visually that the mean vertical deflection of the unreinforced beams is in good agreement with the simulated result. It is important to note, during the first three weeks of the experimental test and numerical model, the relative humidity remains constant and as a result, this period is solely associated with elastic and viscoelastic creep. Consequently, the numerical result in the variable climate produced the same elastic deformation and viscoelastic creep as was produced in constant climate model during this three week period. The mean experimental elastic deflection of Group UV is 5.95 mm. The simulated numerical model result of 6.09 mm is in good agreement with the elastic deflection of Group UV. Significant changes in the model and experimental data occur after three weeks when the relative humidity is changed resulting in an increase in moisture content. This first moisture cycle has been previously characterised by significant increases in deflection and strain due to changes in moisture content higher than that previously attained under a loaded condition (Hunt 1982). As described in Section 6.4.5, the irrecoverable mechano-sorptive creep matrix has been adapted to include an irrecoverable strain component in the longitudinal direction to accurately describe this unique behaviour during changes to moisture contents not previously attained under loaded
conditions. Examining Figure 6.33, the significant change in deflection during the first relative humidity change between week 3 and week 7 has been accurately simulated using the improved irrecoverable mechano-sorptive creep matrix defined in Section 6.4.5.

![Figure 6.33. Mean deflection result of Group UV vs the numerical result](image)

After week 7 the relative humidity reverted back to 65% ± 5% from 90% ± 5% and the relative humidity continued this 8 week cycle for 75 weeks. The fluctuations in each relative humidity cycle observed in the experimental results have also been simulated in the numerical model; however, the magnitude of the fluctuations in the numerical model are larger than that observed experimentally. It was found that the large swelling/shrinkage coefficient (0.0122) defined in the longitudinal direction was a contributing factor to the magnitude of the fluctuations with each cycle. When examining the effect of the longitudinal swelling/shrinkage coefficient, a smaller longitudinal swelling/shrinkage coefficient (0.005) previously used by Toratti (1992a) and Fortino et al. (2009) was examined. Implementing this longitudinal swelling/shrinkage coefficient which is more commonly associated with slow-grown timber, into the numerical model, did not result in such large fluctuations with each relative humidity cycle. Although the numerical deflection result was improved using the smaller longitudinal swelling/shrinkage coefficient, the numerical longitudinal strain result was adversely affected and it was deemed more suitable to use the larger longitudinal swelling/shrinkage
coefficient experimentally determined for Irish Sitka spruce. The numerical deflection result after 75 weeks of 12.48 mm agreed well with the mean experimentally determined Group UV deflection of 12.09 mm demonstrating the effectiveness of the numerical model in the prediction of the creep behaviour of timber elements.

To further examine the long-term deformation in a variable climate, the relative creep from both the numerical model and experimental data are presented to focus solely on the additional creep occurring after the initial elastic loading. The relative creep from the numerical and experimental results as defined in Equation [4.4] is presented in Figure 6.34. Again, during the first three weeks of the experimental and numerical tests, the climate remains constant and there is no significant increase in creep. The significant increase in relative creep associated with the first relative humidity cycle has been accurately simulated by the numerical model along with the fluctuations associated with repeated changes in relative humidity.

![Figure 6.34. Mean unreinforced Group UV relative creep result vs the numerical Group UV result](image)

At 75 weeks, the numerical relative creep of 2.049 agreed well with the mean experimental relative creep value of 2.034 for unreinforced beams. The numerical model has successfully demonstrated the ability to predict long-term deflection behaviour of timber beams in a variable climate.

The total mean strains measured on the tension and compression faces of the unreinforced beams in Group UV have been measured and are compared to the numerical
model results. These results can be seen in Figure 6.35. The mean initial elastic strain has been slightly over estimated on both the tension and compression faces of the unreinforced beams. On the tension face the mean experimental result of 615 με is less than that predicted by the numerical result of 697 με. On the compression face, the experimental result of -674 με showed better agreement with the numerical result of -716 με. After the initial elastic component, the model appears to adequately predict the total strain on the tension and compression face of the unreinforced beam in a variable climate. The large increase in longitudinal strain during the first moisture content change can be seen in Figure 6.35. This irrecoverable mechano-sorptive component has been simulated using the improved irrecoverable mechano-sorptive matrix defined in Section 6.4.5.

![Figure 6.35. Mean reinforced Group UV total strain measurement vs the numerical prediction](image)

The fluctuations in moisture content with repeated cycles has also been successfully simulated with the numerical model. On the tension face, the change in strain with each relative humidity cycle is more gradual than that experienced on the compression face. This is as a result of the different exposure conditions of the tension and compression face. The bottom (tension) face is sealed from the surrounding environment resulting in a reduced rate of moisture content change and more steadily increasing and decreasing rate of strain development in the tension face. In comparison, the compression face is exposed to the surrounding environment and results in a more rapid change in moisture content.
content and strain is observed at the beginning with a slowly reducing rate of change with each relative humidity cycle. These different behaviours have been observed experimentally and have also been simulated in the numerical model results. The accurate prediction of this contrasting behaviour on the tension and compression face, suggests the moisture content distribution with time has been simulated sufficiently accurately in the model and the moisture diffusion coefficients have been successfully replicated. After 75 weeks, the experimental tensile strain of 1294 με compared well to the numerical result of 1535 με and the experimental compressive strain of -1069 με compared well to the numerical result of -1174 με. It can be seen that the results of the numerical model slightly over predict the mean experimental strain results measured on the tension and compression faces of the unreinforced beams, however, a significant proportion of this over prediction may be as a result of the initial differences in the elastic strain component. To examine the long-term performance of the unreinforced specimens, the initial elastic deformation is subtracted from the total strain, providing the additional total creep strain with time. This procedure is performed on the results of the numerical model and presented in Figure 6.36.

![Figure 6.36. Mean reinforced Group UV creep strain measurement vs the numerical prediction](image)

On the tension face of the beam, much greater agreement can be seen in the mean experimental and numerical result when focusing on the total creep strain. Similarly, the
prediction of strain on the compression face has been accurately predicted when focusing on the total creep strain. Again, the irrecoverable mechano-sorptive creep strain component and the characteristic strain behaviour associated with the exposure conditions are accurately predicted on the tension and compression faces. After 75 week, the numerical total creep result of 838 με compares well to the 679 με determined experimentally on the tension face. On the compression face, the numerical model has been shown to slightly over estimate the viscoelastic strain component over the 75 week period with a numerical result of -458 με compared to an experimental result of -396 με. Similarly to the results presented in the unreinforced beam in the constant climate, the over estimation of the results are deemed conservative when predicting the total creep strain behaviour.

6.5.4.2 Reinforced Creep Model Results

In the reinforced creep model, the same variable climate used for the Group UV case has been implemented to predict the total deflection and longitudinal strain development in the reinforced Group RV beams. The numerically predicted deflection of the reinforced beam is compared to the mean experimental deflection results in Figure 6.37. Similarly to the results presented in the constant climate condition, the addition of the BFRP rod reinforcement has resulted in a reduced initial elastic deflection. This reduction in elastic deflection as a result of the reinforcement has been accurately predicted using the model.

![Figure 6.37. Mean experimental deflection result of Group RV vs the numerical prediction](image)

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The mean experimental Group RV elastic deflection of 5.44 mm was previously observed in Chapter 4 and the numerical model of the reinforced beam resulted in an elastic deflection of 5.76 mm. Once the relative humidity was changed, the deflection increased rapidly. The first relative humidity cycle demonstrated a significant increase in deflection. Experimentally the increase in creep deflection associated with the first increase in moisture content to a level not previously attained was less in the reinforced Group RV when compared to the unreinforced Group UV. The simulated result has accurately described this experimentally observed behaviour in the reinforced beam as seen in Figure 6.37. With additional relative humidity cycles, the fluctuations in deflection have been successfully predicted; however, greater fluctuations in the numerical deflection result have been observed. This is due to the contribution of the large longitudinal swelling/shrinkage coefficient characteristic of fast-grown Sitka spruce and similar to the result observed in the unreinforced beams, the use of a reduced longitudinal swelling/shrinkage coefficient in the numerical model adversely affected the longitudinal swelling/shrinkage strains and it was deemed appropriate to use the experimentally determined longitudinal swelling/shrinkage coefficient for Irish-grown Sitka spruce. After 75 weeks, the mean experimental Group RV deflection result of 10.10 mm has been accurately predicted by the numerical model with a simulated deflection result of 10.09 mm.

To further examine the long-term deformation in a variable climate, the relative creep of both the numerical and experimental Group RV results are presented in Figure 6.38 to focus solely on the total creep deflection occurring after the initial elastic loading. Again, during the first three weeks of the experimental and numerical tests, the climate remains constant and there is no significant increase in creep. The significant increase in relative creep associated with the first relative humidity cycle has been accurately simulated by the numerical model. The fluctuations associated with repeated changes in relative humidity have been accurately predicted in the numerical model. There was a statistically significant reduction in the experimental relative creep of the reinforced Group RV beams when compared to the unreinforced Group UV beams as described in Chapter 4. The numerical model has predicted this significant reduction in relative creep in reinforced beams when compared to unreinforced beams in a variable climate.
Figure 6.38. Mean experimental reinforced Group RV relative creep result vs the numerical Group RV prediction

The relative creep of reinforced beams in a variable climate has been modelled over the 75 week period with a numerical relative creep value of 1.753 in good agreement with the mean experimental relative creep value of 1.862 for reinforced beams.

Similarly to the longitudinal strains measured on the unreinforced beams, the total measured creep strain on the reinforced Group RV beams is compared to the numerical model strain results on the tension and compression faces. The results of the mean experimental tests and numerical model predictions can be seen in Figure 6.39. On the tension face, the elastic strain of 597 με simulated in the model is slightly higher than the mean elastic strain of 577 με determined experimentally. The simulated longitudinal strain results on the tension face have demonstrated the beneficial reduction in elastic strain due to the BFRP rod reinforcement in the reinforced beams when compared to that unreinforced beams. On the compression face, the numerical elastic strain result of -718 με is larger than the experimental result of -643 με. Although the elastic strain on the compression face is larger than that measured experimentally, the difference is not deemed significant.
The large increase in longitudinal strain during the first moisture content change can be seen. This irrecoverable mechano-sorptive component has been simulated accurately using the improved irrecoverable mechano-sorptive matrix defined in Section 6.4.5. Experimentally, the irrecoverable mechano-sorptive creep component was shown to be less significant in the reinforced beams when compared to the unreinforced beams. This reduction in irrecoverable mechano-sorptive creep has also been observed in the numerical model due to the addition of the BFRP rod reinforcement. The fluctuations in strain with repeated cycles have also been successfully simulated with the numerical model on both the tension and compression faces. On the tension face, the numerical and mean experimental results are quite similar. The more gradual increase and decrease in strain due to the cyclic relative humidity is characteristic of the exposure conditions on this tension face and similar behaviour has also been observed in the unreinforced beams. On the compression face, the behaviour is again similar to that experienced on the unreinforced beams as the compression face is exposed to the surrounding environment and results in a more rapid change in strain is observed at the beginning with a slowly reducing rate of change with each relative humidity cycle. These different behaviours have been observed experimentally in reinforced beams and have been simulated successfully in the numerical model results. After 75 weeks, the mean experimental strain of 1030 με measured on the tension face of the reinforced beams compared well to the

![Figure 6.39. Mean experimental reinforced Group RV strain vs the numerical prediction](image-url)
numerical result of 1059 με. On the compression face the experimental strain result of -866 με was less than the numerical result of -1068 με, however, this over estimation is deemed conservative.

To examine the long-term performance of the reinforced beams, the initial elastic strain component is subtracted from the total strain on the tension and compression face, providing the total creep strain with time. This procedure is performed on the results of the numerical model and presented in Figure 6.40.

![Figure 6.40. Mean experimental reinforced Group RV creep strain vs the numerical prediction](image)

On the tension face, the simulated result can be seen to agree well with the mean experimental result. After 75 weeks, the mean experimental creep strain experienced on the tension face was 453 με. This experimental result compared well to the numerically simulated creep strain result of 462 με. On the compression face, the simulated result has also been shown to agree well with the mean experimental creep stain result. After 75 weeks, the mean experimental creep strain experienced on the compression face was -224 με, which compared reasonably well to the numerically simulated creep strain result of -350 με. The model has been shown to adequately predict the total longitudinal creep strain which comprises the viscoelastic, mechano-sorptive and swelling/shrinkage strain, on both the tension and compression faces of the reinforced beams.
6.5.5 Parametric study: Effect of Reinforcement Type on Creep Behaviour

The presented model has demonstrated the ability to predict the creep behaviour of the timber beams reinforced with BFRP rod reinforcement and a significant reduction in creep deflection and longitudinal creep strain in reinforced beams was observed from the analysis of the experimental and numerical results. In an effort to examine the effect of the reinforcement type, the calibrated model is adapted to examine the influence of different FRP materials on the hygro-mechanical creep behaviour. The different materials used are presented in Table 6.6. Where a range of values is provided, the average value is taken.

Table 6.6. Parametric study: FRP materials

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<td>GPa</td>
<td></td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon fibre rod reinforcement</td>
<td>120-300</td>
<td>GPa</td>
<td></td>
</tr>
</tbody>
</table>

The material properties for the timber and the mechanical loading conditions remained unchanged. A constant dead load of 6241 N was applied to all beams under examination. The climate conditions implemented is a sinusoidal curve with a period of one year and an amplitude cycling between 65% and 90% relative humidity as seen in Figure 6.41.

![Parametric Study Relative Humidity Cycle](image)

*Figure 6.41. Sinusoidal relative humidity cycle implemented to examine the performance of different types of FRP materials*
The sinusoidal curve was chosen to be a crude representation of an annual change in relative humidity with a mean relative humidity of 65% in the middle of summer transitioning slowly to a mean of 90% relative humidity in the middle of winter. This relative humidity is applied to the exposed surfaces of the glued laminated beam for a duration of 10 years.

The total deflection results for the unreinforced and reinforced beams in the parametric study are presented in Figure 6.42. Additionally, the peak deflections at a series of relative humidity cycles were chosen and have been presented in Table 6.7.

When examining the results in Figure 6.42 and Table 6.7, it is important to remember that the beams are loaded to a common dead load of 6241 N and each beam is subject to the same relative humidity cycle and only the stiffness of the reinforcement has been changed in each reinforced model. In Figure 6.42 and Table 6.7, it can be seen that initially, there is a large difference due to the elastic behaviour of the various beams. It can be seen that the unreinforced beam has experienced the greatest creep deflection as expected. The magnitude of the creep deflection behaviour of FRP reinforced beams can be seen to be influenced by the stiffness of the FRP material with the least stiff GFRP experiencing a greater creep deflection than the stiffer BFRP, AFRP and CFRP reinforcing materials. The GFRP and BFRP reinforced beams have demonstrated similar creep behaviour as a result of their similar stiffness values of 46 GPa and 50.8 GPa, respectively. The creep behaviour of the AFRP reinforcement results in a large decrease in creep deflection due

Figure 6.42. Parametric study: Creep deflection results of unreinforced and FRP reinforced beams over a 10 year period under a constant dead load
to the stiffness of the FRP material, however the general behaviour is similar to that of the less stiff FRP materials. When examining the creep behaviour of the CFRP reinforcement, it can be seen that the creep behaviour is different to that seen previous as there is an overall decrease in deflection during the first increase in deflection. A similar result was observed by Hansson & Kristoffer (2007) and Kliger et al. (2008) when experimentally comparing the creep behaviour of various types of reinforcement to the creep behaviour of an unreinforced member.

Table 6.7. Parametric study: Peak deflection (mm) during the chosen relative humidity cycles

<table>
<thead>
<tr>
<th>Loading</th>
<th>Peak 1</th>
<th>Peak 2</th>
<th>Peak 4</th>
<th>Peak 6</th>
<th>Peak 8</th>
<th>Peak 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>6.37</td>
<td>14.20</td>
<td>14.46</td>
<td>14.78</td>
<td>15.04</td>
<td>15.29</td>
</tr>
<tr>
<td>GFRP</td>
<td>5.70</td>
<td>10.44</td>
<td>10.64</td>
<td>10.96</td>
<td>11.25</td>
<td>11.54</td>
</tr>
<tr>
<td>BFRP</td>
<td>5.58</td>
<td>10.17</td>
<td>10.36</td>
<td>10.65</td>
<td>10.91</td>
<td>11.15</td>
</tr>
<tr>
<td>AFRP</td>
<td>4.85</td>
<td>8.37</td>
<td>8.54</td>
<td>8.85</td>
<td>9.15</td>
<td>9.45</td>
</tr>
<tr>
<td>CFRP</td>
<td>4.06</td>
<td>6.85</td>
<td>7.01</td>
<td>7.30</td>
<td>7.58</td>
<td>7.86</td>
</tr>
</tbody>
</table>

They found that higher stiffness materials demonstrated such behaviour. This is believed to be a result of the longitudinal swelling of the timber while the stiffer CFRP reinforcement does not swell. There also appears to be a phase lag in the response of each beam to the sinusoidal relative humidity cycle in both the unreinforced and reinforced beams. During the first relative humidity cycle, which peaked at 90% relative humidity after 26 weeks and returned to 65% relative humidity after 52 weeks, the unreinforced beam reached its peak deflection after 40 weeks. A similar trend was observed in the FRP reinforced beams. The stiffness of the FRP material has been shown to further delay the response of each beam due to the sinusoidal relative humidity cycle with the GFRP reaching its first peak deflection after 45 weeks, the BFRP after 46 week, the AFRP after 51 weeks and the CFRP after 53 weeks. For this reason, it was decided to compare the results of each modelled beam at the peak deflection of each relative humidity cycle.

The creep deflection results have demonstrated the effect of FRP reinforcement type on the total creep behaviour of such beams. Using the normalised relative creep measure, the ratio of total deflection to elastic deflection may be examined allowing the long-term behaviour to be compared. The relative creep results are presented in Table 6.8. It can be seen that regardless of FRP material, the relative creep behaviour increases with time. The unreinforced beam has the largest relative creep result throughout with a final result of 2.435 after 10 years. Interestingly, although the FRP materials demonstrate different magnitudes of total creep deflection over time, when examining the relative
creep results after 10 relative humidity cycles, the relative creep behaviour of all reinforced beams, regardless of FRP type, have shown a similar relative creep result with the GFRP resulting in a value of 2.073, BFRP with 2.038, AFRP with 2.010 and CFRP with 2.006.

Table 6.8. Parametric study: Peak relative creep result during the chosen relative humidity cycles

<table>
<thead>
<tr>
<th>Loading</th>
<th>Peak 1</th>
<th>Peak 2</th>
<th>Peak 4</th>
<th>Peak 6</th>
<th>Peak 8</th>
<th>Peak 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>1.000</td>
<td>2.228</td>
<td>2.268</td>
<td>2.319</td>
<td>2.360</td>
<td>2.398</td>
</tr>
<tr>
<td>GFRP</td>
<td>1.000</td>
<td>1.831</td>
<td>1.866</td>
<td>1.922</td>
<td>1.973</td>
<td>2.023</td>
</tr>
<tr>
<td>BFRP</td>
<td>1.000</td>
<td>1.821</td>
<td>1.855</td>
<td>1.908</td>
<td>1.954</td>
<td>1.997</td>
</tr>
<tr>
<td>AFRP</td>
<td>1.000</td>
<td>1.725</td>
<td>1.762</td>
<td>1.825</td>
<td>1.887</td>
<td>1.948</td>
</tr>
<tr>
<td>CFRP</td>
<td>1.000</td>
<td>1.687</td>
<td>1.726</td>
<td>1.796</td>
<td>1.865</td>
<td>1.935</td>
</tr>
</tbody>
</table>

These simulated relative creep results have shown that it may be possible to apply a single factor ($k_{def}$ factor) to predict the long-term performance of reinforced beams regardless of FRP type. It is noted that these results only apply for beams with a similar percentage area reinforcement ratio and under a common dead load. Future examination of the effect of stress level and different percentage area reinforcement ratios is required prior to the provision of reliable modification factors to predict creep behaviour of FRP reinforced beams.

6.6 SUMMARY AND DISCUSSION

In recent times, there has been a movement in the construction industry towards using more sustainable construction materials and a unique opportunity exists for timber to be used to build larger and taller buildings. Numerical models have played a key role in an attempt to understand and predict the behaviour of this sustainable material in taller buildings under more demanding loading conditions. Numerical models, with the use of reliable material data, are being increasingly used to model the short and long-term behaviour of this complex material. In this chapter, some of the key influences affecting the long-term performance of unreinforced and reinforced glued laminated timber beams under constant and variable climate conditions have been examined using numerical methods.

Firstly, the effect of the surrounding climate on the behaviour of timber has been examined. Specific attention has been given to fast-grown Irish timber in this study as presented in the material characterisation in Chapter 5. The presented moisture diffusion
model, implementing direction-dependent diffusion coefficients and utilising a thermo-hygro analogy has been shown to provide reliable moisture content distribution results through the cross section of structural size timber elements. The accuracy of the model has shown better agreement with measurements taken close to the surface of the timber beam. With greater depth, the prediction of moisture content was less accurate. It was observed that the change in moisture content during the desorption phase was not accurately simulated and an increased diffusion coefficient was required to simulate this behaviour. It has also been shown by Peralta & Lee (1995) that the diffusion coefficient during the desorption phase can be approximately twice that measured during the adsorption phase. This difference in diffusion coefficients during adsorption and desorption was not implemented into the model at this stage. It was decided that the results closer to the surface were simulated accurately and deemed more influential when studying moisture-induced swelling and shrinkage strains and mechano-sorptive creep strains in the coupled moisture-displacement models. Using the diffusion coefficients, which were found to adequately describe moisture diffusion in Irish Sitka spruce, the effect the PRF glue-line on the flow of moisture in glued laminated beams was also examined. A fully permeable glue-line was compared to a fully impermeable glue-line. The numerical model results have shown negligible differences between the moisture content at a series of locations due to the different glue-line conditions and it is concluded that the inclusion of the adhesive glue-line does not result in better accuracy and only adds to the computational time of the model. For this reason, the glue-line was omitted in subsequent models. It is also noted that, in this study, the beam is of structural size; however, in larger block glued laminated beams of CLT products, the effect of the glue-line may be more significant and should be investigated.

It has been shown that under non-loaded conditions and variable climate conditions, timber is subject to moisture-induced strains. For this reason, a coupled moisture-displacement model was created to examine the dimensional stability of glued laminated beams under changing moisture content conditions. The developed model does not include viscoelastic or mechano-sorptive strain components with occur under loaded conditions. This model utilised the coupled thermal-displacement transient analysis available in Abaqus finite element software and with accurate definition of the radial, tangential and longitudinal material properties, the moisture-induced strains were simulated. The material properties in the respective material directions and their
dependency on moisture content were accurately defined together with the experimentally determined longitudinal, radial, and tangential swelling/shrinkage coefficients for Irish Sitka spruce, presented in Chapter 5. The accurate definition of the material geometry was found to be important when modelling a complex cylindrically orthotropic material such as timber. The material properties may be easily defined in a cylindrical coordinate system in Abaqus mimicking the cylindrical nature of timber; however, the definition of this cylindrical coordinate system must be carefully considered. The position of the pith or centre of this coordinate system has a considerable effect on the numerical moisture-induced strain result. The idealised symmetric case where the pith of each timber lamination is situated in the centre along the top surface of each lamination was examined and compared to a mapped case where the pith of each lamination was accurately mapped and modelled in Abaqus finite elements software. In the mapped case, the pith was located outside each lamination as was the case in the experimental test specimens. The results presented have shown the mapped model to produce significantly more accurate results when compared to the idealised model. Implementing the mapped model, the moisture-induced strain behaviour of Irish-grown Sitka spruce parallel and perpendicular to the grain has been accurately modelled using the experimentally determined directional dependent swelling/shrinkage coefficients.

The final model described in this chapter was created to examine the performance of unreinforced and reinforced timber beams when under loaded conditions for long periods of time. The hygro-mechanical creep model, which has been developed in a user-defined UMAT subroutine, incorporates viscoelastic and mechano-sorptive strain components due to the external dead load and variable relative humidity conditions. The material characterisation of fast-grown Sitka spruce included in the moisture diffusion model and the coupled moisture-displacement model have also been incorporated into the mechanical creep model. The formulation of the UMAT subroutine follows the approach implemented by Santaoja et al. (1991), Mårtensson (1994b), Mirianon et al. (2008) and Fortino et al. (2009); however, a significant alteration was made to the irrecoverable mechano-sorptive creep matrix to include irrecoverable deformations in the longitudinal direction when stressed under loaded conditions and subject to moisture contents not previously attained. This irrecoverable longitudinal strain component was observed experimentally which was the catalyst for such an alteration. The user-defined UMAT was applied to an unreinforced and reinforced beam in constant and variable climates and
compared to experimental results previously presented in Chapter 4. In the constant climate condition, the viscoelastic behaviour of both unreinforced and reinforced beams was examined. The definition of the parameters in the viscoelastic creep matrix was calibrated and matched to the experimental viscoelastic creep data of the unreinforced beams. This was performed as no data was previously available to describe the viscoelastic creep behaviour of Irish Sitka spruce. Applying these species-specific material properties within the model, the deflection and relative creep behaviour of the unreinforced and reinforced beams were accurately modelled over the 75 week period. The total longitudinal strain and viscoelastic strain on the tension and compression faces of the unreinforced and reinforced beams were over estimated; however, this has been deemed conservative.

In a variable climate, the additional strain as a result of moisture fluctuations with time has been examined. In addition to the swelling/shrinkage strains, the mechano-sorptive strain component is calculated in the user-defined subroutine. When examining the total deflection and longitudinal strain of unreinforced and reinforced beams, it was found that the initial change in moisture content to a level not previously attained, resulted in a large increase in deflection and strain. This behaviour was observed experimentally in the unreinforced beams and to a lesser extent in the reinforced beams and due to the significant alteration to the irrecoverable mechano-sorptive creep matrix in the UMAT subroutine, this behaviour has been simulated in the numerical models. Prior to this alteration, the irrecoverable mechano-sorptive creep matrix did not adequately describe the long-term strain in the longitudinal direction. The agreement of the experimental and simulated results demonstrate the need for such irrecoverable strains in the longitudinal direction. The hygro-mechanical creep model has also provided reliable predictions when examining the long-term creep behaviour of unreinforced and reinforced beams with subsequent relative humidity cycles. Experimentally in Chapter 4, the reinforced beams in a variable climate experienced reduced creep behaviour. This reduced creep behaviour has also been simulated numerically using the hygro-mechanical creep model.

Using this validated model, a parametric study was designed to examine the effect of different types of FRP on the long-term behaviour of reinforced timber. Glass, basalt, aramid and carbon FRP materials were chosen and implemented into the reinforced beam model and subject to a common dead load under an annual sinusoidal relative humidity cycle ranging between 65% relative humidity during summer months to 90% relative
humidity during winter months. The results were compared to an unreinforced beam under a common dead load. The influence of stiffer FRP materials resulted in a reduced creep deflection behaviour over the simulated 10 year period. As a result of the difference in stiffness of the FRP materials, there were large differences in the elastic deflection, and the normalised relative creep measure was used to compare the long-term performance of each beam. It was found that the FRP reinforced beams performed better than the unreinforced beam throughout. When focusing on the FRP reinforced beams, it was found that after 10 years, there was very little difference between the relative creep results of all the FRP reinforced beams regardless of FRP type. This indicates that a single modification factor may be suitable to describe the long-term relative creep behaviour of FRP reinforced beams regardless of FRP type. It is noted that the simulated beams are loaded under a common bending load and in future studies, the influence of bending stress must be examined in addition to examining different percentage areas of reinforcement prior to the provision of reliable modification factors. This validated model is a useful tool that can be used to investigate such influences on the long-term behaviour of FRP reinforced beams, resulting in significant economic and time savings normally associated with experimental tests.
7 CONCLUSIONS

7.1 INTRODUCTION

This study has been performed with the objective to investigate the viscoelastic and mechano-sorptive creep behaviour of FRP reinforced timber elements. The methodology outlined in Chapter 1 has been successfully implemented. Experimental tests have been carried out to examine the short-term and long-term behaviour of unreinforced and reinforced beams. Material characterisation tests have been performed on fast-grown Sitka spruce and basalt fibre reinforced polymer. These tests have provided the necessary material properties for implementing a hygro-mechanical creep model, which has been shown to reliably predict the long-term behaviour of both unreinforced and reinforced timber elements manufactured from this fast-grown species in both constant and variable climates. This chapter discusses the main conclusions from both the experimental and numerical work performed in this thesis.

7.2 EXPERIMENTAL TEST CONCLUSIONS

7.2.1 Short-term Test Conclusions

The short-term tests presented in Chapter 3 were successfully performed on unreinforced and reinforced glued laminated beams. These tests results provide a reliable basis for long-term comparative studies in Chapter 4. This is achieved by manufacturing beams with similar properties thus minimising the variability inherent in timber. This procedure has proven to be important in the comparison of unreinforced and reinforced members and is recommended in future studies.

A total of 40 beams formed the basis of the test programme, half of which were reinforced. The addition of BFRP rod reinforcement in the tensile lamination of half the manufactured beams has also been examined. Short-term four-point bending tests have demonstrated that the addition of BFRP rod reinforcement in modest quantities of 1.85% by area can increase the flexural stiffness of glued laminated beams by 16.30%. To the author's knowledge, this is the largest sample size of reinforced beams utilising BFRP. The results demonstrated the suitability of this material for timber reinforcement applications.
7.2.2 Long-term Test Conclusions

The long-term creep behaviour of unreinforced and reinforced beams has been successfully characterised in both constant and variable climates using the matched groups created in Chapter 3. It had been shown in previous research studies that the use of FRP reinforcement resulted in reduced relative creep behaviour in a constant climate condition. In this study, the long-term mid-span deflection results of the unreinforced and reinforced beams in the constant climate have shown similar mean relative creep behaviour. A statistical analysis has shown a statistically insignificant difference exists between the mean relative creep behaviour of the unreinforced and reinforced beam groups. In this study, each beam is loaded to a common maximum bending stress in the compression zone. This was often not the case in previous studies and a common maximum bending stress in the compression zone is essential to make comparisons between unreinforced and reinforced members. It is recommended that this methodology is applied in future studies of FRP reinforced beams. These results in a constant climate condition indicate that the main driver of viscoelastic creep behaviour occurs within the timber and the effect of the FRP reinforcement is negligible. This indicates that the current modification factors provided for solid or engineered wood products in Eurocode 5 may be adequate in describing the creep behaviour of FRP reinforced beams in a constant climate.

In a variable climate, the relative creep of the unreinforced beams has been compared to that of the reinforced beams. It was demonstrated that the variable climate resulted in greatly increased creep behaviour of both beam groups when compared to similarly unreinforced and reinforced beam in a constant climate condition. This large increase in relative creep is due to a combination of the swelling/shrinkage behaviour and mechano-sorptive behaviour of the timber. Interestingly, the magnitude of this increase in relative creep was not the same in unreinforced and reinforced beams. The unreinforced beams experienced greater relative creep on average than the reinforced beams even when stressed to a common maximum bending stress. This difference was shown to be statistically significant demonstrating the beneficial effect of reinforcement in reducing the relative creep behaviour of FRP reinforced beams in a variable climate. Currently there are no modification factors available to account for this reduced creep behaviour.

In addition to the measurement of mid-span vertical deflection, the longitudinal strain has been measured on the tension and compression faces of the unreinforced and
reinforced beams throughout the 75 week test period. In the constant climate, these tests results have allowed the viscoelastic strain component to be characterised. Although there was no reduction in the relative creep deflection, there was a beneficial mean reduction in longitudinal strain on the tension face of the reinforced beams as a result of the position of the FRP reinforcement in the bottom tensile lamination. This beneficial reduction of longitudinal strain in the tensile lamination may be utilised in future design approaches.

When examining the mid-span longitudinal strain results on the tension and compression faces of the instrumented beams in the variable climate, there was a significant increase in longitudinal strain as a result of the variable climate on both the unreinforced and reinforced beams. In the variable climate, the total creep strain, comprising viscoelastic, mechano-sorptive and swelling/shrinkage strain components, was characterised. A statistical analysis has shown a statistically significant difference exists between the longitudinal creep strain results measured on the tension face of unreinforced and reinforced beams.

As discussed in Chapter 1, the swelling/shrinkage behaviour of timber is often neglected when examining the creep performance of timber beams in a variable climate (Armstrong & Kingston 1960; Hunt 1982; Zhou et al. 1999; Runyen et al. 2010; Yahyaei-Moayyed & Taheri 2011). In this study, the swelling/shrinkage strain component was characterised by tests on non-loaded beams. The use of matched groups allowed the individual strain components to be characterised. The mechano-sorptive component was calculated by subtracting the viscoelastic strain component, measured in the constant climate, and the swelling/shrinkage strain component from the total creep component, measured in the variable climate. This methodology has successfully isolated the mechano-sorptive strain component on the tension and compression faces of unreinforced and reinforced beams. This mechano-sorptive effect is attributed to the coupled interaction between stress and moisture content change and interestingly, a statistical analysis has shown no evidence to suggest that the mechano-sorptive behaviour on the tension and compression faces of unreinforced and reinforced beams are not equal. This study has shown, when both unreinforced and reinforced beams are loaded to a common bending stress, and under-go moisture content changes as a result of a common variable climate, the mechano-sorptive strain experienced in unreinforced and reinforced beams are similar.
The main difference between the unreinforced and reinforced beams was seen in
the respective swelling/shrinkage components and most noticeably on the tension face.
The restraining behaviour of the BFRP reinforcement has resulted in significantly
reduced timber swelling/shrinkage strains in the tension zone of the reinforced beams.
This reduction in swelling/shrinkage strain appears to be the most significant influence
on the long-term deflection of the reinforced beams. This conclusion demonstrates the
importance of including and accurately characterising the swelling/shrinkage behaviour
of timber when examining the long-term performance of timber elements. It is envisaged
that in future studies, the accurate characterisation of the restrained swelling/shrinkage
behaviour due to the addition of a reinforcing material may be used to accurately predict
the long-term behaviour of FRP reinforced members.

7.2.3 Material Characterisation Conclusions

In order to develop a numerical tool to predict the behaviour of FRP reinforced timber
beams, it is important to accurately characterise the material properties of both the timber
and FRP materials. For this reason, a series of experiments was designed to examine the
hygro-mechanical material properties of fast-grown Sitka spruce and the mechanical
properties of BFRP reinforcement.

Firstly, the moisture content hysteresis at 20°C for fast-grown Sitka spruce grown
in Ireland was successfully characterised. In this study, the hysteresis loop which
characterises the moisture content during adsorption and desorption has been
characterised as this is much more accurate for modelling of moisture effects instead of
the average of both isotherms. This material data was previously unavailable for this
timber and may now be used in future studies. This data was used to accurately define the
relative humidity boundary conditions within the numerical models in Chapter 6.

The mean radial, tangential and longitudinal swelling/shrinkage coefficients have
been determined for Irish grown Sitka spruce. Irish grown timber is characterised by a
fast growth rate and fast-grown timber has been reported in some studies to result in larger
longitudinal swelling/shrinkage coefficients than that associated with slow-grown timber.
The experimental test results have found this to be the case in Irish Sitka spruce. The test
results have provided species-specific material properties that were not previously
available.
As there had been no previous study on the diffusion behaviour of Irish Sitka spruce, a 20 week long experiment on 30 x 30 x 30 mm$^3$ timber cube specimens was conducted to observe the radial/tangential and longitudinal diffusion behaviour of Irish-grown Sitka spruce. The experimental tests have distinguished different diffusion behaviour associated with different exposure surfaces. This diffusion behaviour is used in Chapter 6 to determine radial/tangential and longitudinal diffusion coefficients suitable for Irish Sitka spruce.

A series of tests has also been performed to characterise the behaviour of BFRP reinforcement for use in numerical models. Tensile tests in accordance with ISO 10406-1 (ISO 2015) were used to establish a mean elastic modulus of the BFRP rod reinforcement. The mean elastic modulus was found to be 50772 N/mm$^2$. Swelling and shrinkage tests were also performed on specimens of BFRP rod reinforcement to determine its behaviour under variable relative humidity conditions. It was found that there was no moisture uptake or measurable swelling/shrinkage behaviour on the BFRP rod reinforcement when placed in a high relative humidity condition. The creep behaviour of BFRP rod reinforcement was also examined at loads ranging to 3.85%-15% of the ultimate tensile strength. It was found that there was no significant change in creep strain over a 500 hour test period. These results have demonstrated that swelling/shrinkage behaviour and creep behaviour of BFRP rod reinforcement may be omitted when studying the long-term performance of FRP reinforced beams.

The material characterisation tests performed have provided new data relating to the effect of moisture content and variable relative humidity on Irish-grown Sitka spruce and BFRP reinforcement. The experimental characterisation performed to fill this gap in the knowledge was fundamental in the analysis and numerical study of FRP reinforced beams in Chapter 6. It is hoped that this data will be utilised in future studies, across a wide range of applications, to promote the use of this timber species and FRP material.

### 7.3 Numerical Modelling Conclusions

Finite element models have been developed to predict the behaviour of FRP reinforced beams within constant and variable climates. These models incorporating material properties specific to Sitka spruce and BFRP have been shown to accurately model the long-term behaviour of unreinforced and reinforced beams.
The first model created was a moisture diffusion model to aid the selection of suitable diffusion coefficients for fast-grown Sitka spruce. The diffusion coefficients in the radial/tangential direction presented by Toratti (1992a) and the longitudinal diffusion coefficients presented by Sjödin (2006) have been shown to accurately describe the diffusion behaviour of small clear specimens of fast-grown Sitka spruce. These experimentally matched diffusion coefficients have also been utilised in a 2-dimensional model used to examine the moisture flow in larger structural sized timber beams. The results are accurate close to the surface but the accuracy of the model reduces at greater depths. It was observed that the change in moisture content during the desorption phase was not accurately simulated and an increased diffusion coefficient was required to simulate this behaviour.

The effect of the glue-line between the timber laminations on the flow of moisture within glued laminated beams has also been examined. It was concluded that the inclusion of the glue-line increases computational time and does not result in increased accuracy of the model. As a result, the glue-line was not included in subsequent models. For larger engineered wood products such as CLT or block glued laminated beams, the influence of these glue-lines may be significant and should be examined.

The moisture-induced strain, which has been shown to heavily influence the long-term performance of FRP timber elements, has also been accurately predicted in a 3-dimensional coupled moisture-displacement numerical model with the use of experimentally determined shrinkage/swelling coefficients. It was shown that the accuracy of the model can be greatly increased by carefully defining the orientation of the material. This was evident when the location of the pith along the longitudinal axis and cylindrically orientated radial and tangential directions of the material were accurately defined for the individual laminations. A similar practice is recommended in future studies.

The final numerical model developed is a 3-dimensional hygro-mechanical creep model created to predict the long-term behaviour of unreinforced and reinforced beams under loaded conditions in constant and variable climates. Following the approach by Santaoja et al. (1991), Mårtensson (1994b), Mirianon et al. (2008) and Fortino et al. (2009), a user-defined UMAT subroutine was developed. In this study, a significant modification to the existing approach was the inclusion of an irrecoverable mechano-sorptive creep component in the longitudinal direction, which was neglected in previous
studies. This modification was found to better describe the long-term deflection and strain results of both the unreinforced and reinforced beams. It is recommended to include this modification in future studies but it is noted that the parameters provided in this study have been calibrated against experimental test results on fast-grown Sitka spruce and should be checked when implemented on different species of timber. This model, incorporating material data obtained from the material characterisation tests, described in Chapter 5, has compared well to the experimental creep tests results. In the experimental programme, described in Chapter 4, it was found that there was a significant reduction in the relative creep behaviour of reinforced members when compared to unreinforced beams in a variable climate. This reduction in relative creep behaviour was also simulated by the numerical model. This validated model incorporating experimentally determined properties of fast-grown Sitka spruce and BFRP reinforcement provides a unique design tool which can be used to examine various FRP reinforced engineered wood products in different climate conditions. A parameter study was carried out to examine the influence of FRP type on the creep behaviour of such beams. It was shown that the elastic modulus of the FRP material has a significant effect on the total deflection behaviour; however, when examining the relative creep behaviour, the results were similar for each FRP type tested. The similar relative creep result indicates that it may be possible to develop a single modification factor to be used to describe the creep behaviour of FRP reinforced beam regardless of FRP type; however, it was noted that further simulations are required to investigate the influence of bending stress and percentage area of reinforcement prior to the provision of reliable modification factors.

7.4 SUMMARY OF KEY RESEARCH CONTRIBUTIONS

This study was undertaken to address the current lack of knowledge in relation to the viscoelastic and mechano-sorptive creep behaviour of reinforced timber elements. This lack of knowledge relating to long-term performance in variable climate conditions has been a major impediment to the development of harmonised standards for the design of such reinforced timber elements. The test methodology developed in this research, which was designed to provide a reliable basis for long-term comparable studies, has been implemented successfully and has allowed the elastic, viscoelastic, mechano-sorptive and swelling/shrinkage components of the total hygroscopic-flexural response to be individually characterised. Additionally, an investigation utilising finite element software
has seen the development of a numerical tool which has been shown to successfully predict the long-term behaviour of reinforced timber elements. The key novel research contributions are as follows:

- The use of FRP reinforcement has resulted in a reduction in the total deflection of reinforced beams when compared to similarly-stressed unreinforced beams. The reduction in total deflection is primarily due to the increase in elastic stiffness of the reinforced beams and the restraining effect of reinforcement on the swelling/shrinkage behaviour.

- The stress in the timber is the main driver of viscoelastic creep behaviour and the effect of FRP reinforcement is negligible. This leads to the conclusion that the modification factors currently available in Eurocode 5 for solid or glued laminated timber elements may be suitable for the design of reinforced timber elements in constant climate conditions.

- The stress level and changing moisture content in the timber are the main drivers of mechano-sorptive creep behaviour and the FRP reinforcement does not heavily influence this flexural response.

- The restraining behaviour of the BFRP rod reinforcement has heavily influenced the swelling/shrinkage behaviour of the reinforced beams and has significantly contributed to the reduced relative creep behaviour observed in cyclic humidity conditions. To conclude, subject to further validation on different species, it will be possible to predict the long-term creep behaviour from the short-term swelling/shrinkage behaviour of FRP reinforced beams making expensive and time consuming experimental tests unnecessary.

- A coupled hygro-mechanical creep model has been created that successfully predicts the flow of moisture and associated viscoelastic, mechano-sorptive and swelling/shrinkage behaviour in reinforced timber beams. The calibrated model provides a unique tool to examine different FRP reinforced timber elements in various climate conditions and is ideally suited to aid in the development of design rules for FRP reinforced beams, which will in turn support the increased use of FRP reinforcement in the reinforcement, repair and upgrade of timber structures.

- As most previous studies on moisture effects in timber have focused on slow-grown species, the characterisation of these effects in fast-grown species is
important in light of changing forestry practices across Europe that promote fast growth rates.

7.5 RECOMMENDATIONS AND FUTURE WORK

The lack of a test standard for creep testing of structural timber and engineering wood products has resulted in an inconsistency in the test procedures reported in the literature. The test set-up implemented in this study used a constant dead load to measure creep deflection and strain with time. This test methodology, outlined in Chapter 4, has been successfully used to characterise the creep behaviour of unreinforced and reinforced beams in constant and variable climates. It is suggested that the test methodology outlined should be considered in future studies and standards allowing for more comparisons to be made between different studies.

In this study, while one species and FRP material combination has been investigated experimentally, the conclusions may be of significant importance in the future development of design standards for all reinforced timber beams. The short-term measurement of the restrained swelling/shrinkage behaviour has been found to be the most significant factor controlling the long-term relative creep behaviour for a given FRP reinforced timber element. Experimental tests, implementing the methodology developed in this study on different species and FRP materials, should be conducted to further validate this finding.

The diffusion coefficients in this study have adequately described the moisture content distribution close to the surface of structural sized beams in a variable climate. However, the accuracy reduces with depth. It was observed that the diffusion coefficients were less accurate in describing the desorption behaviour. It has been reported that the diffusion coefficient during desorption may be approximately twice that measured during the adsorption phase (Peralta & Lee 1995). It is recommended that a more extensive diffusion characterisation study be undertaken that separately characterises the adsorption and desorption behaviour. It must also be noted that the diffusion coefficients selected in this study are based on the behaviour of small clear samples of Irish Sitka spruce. Consideration should be given to investigating the diffusion behaviour using larger in-grade specimens. In-grade specimens may encompass effects due to knots and other defects that have not been considered in this study.
In this study, only the effect of passive reinforcement has been examined. In recent studies, there have been significant advances achieved through the pre-stressing of FRP reinforcement in timber elements which utilises additional capacity in the FRP material, while at the same time increasing the ultimate load capacity (Kliger et al. 2016). Future experimental and numerical studies should also focus on the long-term performance of such beams.

In this particular study, the longitudinal strain along the length of the BFRP rod reinforcement was not monitored and, although no measurable creep was evident at the load levels investigated here, in future studies, it would be beneficial to observe the change in strain with time of the FRP reinforcement particularly for pre-stressed FRP applications.
REFERENCES


Raftery, G. & Harte, A., 2013. Nonlinear numerical modelling of FRP reinforced glued laminated...


APPENDIX A

RELATIVE CREEP RESULTS

This appendix presents the individual relative creep deflection curves of each beam over the 75 week period in their respective groups.

Group UC

Figure A.1. Beam 5

Figure A.2. Beam 17
Figure A.3. Beam 18

Figure A.4. Beam 27

Figure A.5. Beam 33
Figure A.6. Beam 34

Figure A.7. Beam 40

Group RC

Figure A.8. Beam 2
Figure A.9. Beam 12

Figure A.10. Beam 13

Figure A.11. Beam 26
Figure A.12. Beam 30

Figure A.13. Beam 32

Figure A.14. Beam 36
Group UV

Figure A.15. Beam 6

Figure A.16. Beam 11
Figure A.17. Beam 15

Figure A.18. Beam 21

Figure A.19. Beam 22
Figure A.20. Beam 23

Figure A.21. Beam 29

Figure A.22. Beam 39
Group RV

Figure A.23. Beam 3

Figure A.24. Beam 4
Figure A.25. Beam 8

Figure A.26. Beam 10

Figure A.27. Beam 4
Figure A.28. Beam 19

Figure A.29. Beam 24

Figure A.30. Beam 28
APPENDIX B

CREEP FRAME DESIGN
Figure B.1. Creep test frame (dimensions in mm)

Figure B.2. Lever arm detail (dimensions in mm)
Figure B.3. Parts A-H (dimensions in mm)
Figure B.4. Part A (dimensions in mm)
Figure B.5. Part B (dimensions in mm)
Figure B.6. Part C (dimensions in mm)
Figure B.7. Part D (dimensions in mm)
Figure B.8. Part E (dimensions in mm)
Figure B.9. Part F and Part G (dimensions in mm)
Figure B.10. Part G (dimensions in mm)
Figure B.11. Part H (dimensions in mm)
**APPENDIX C**

**MOISTURE PROBE CALIBRATION**

In order to determine the moisture content at a specific depth in large glued laminated beams, it was advantageous to investigate the electrical resistance between two pins within this fast-grown Sitka spruce and the effect of moisture content on this resistance. Similarly to the most commonly used hand-held moisture meters, this method works best between 6-30% moisture content (Dai & Ahmet 2001). In two studies by Forsén & Tarvainen (2000) and Taylor & West (1990), the influence of moisture content meter, species and type of electrode used, was shown to have a significant influence when measuring moisture content using the resistance method. For this reason, it was necessary to calibrate the moisture pins system used in this study, for Irish Sitka spruce.

Similar to the sticks specimens in Section 5.2.2, the cubes also had a dual purpose. The cubes had a pair of moisture pins inserted into their centre with a view to recording the electrical resistance of the timber at various moisture content levels and calibrating the moisture probe system to use in larger timber beams. The body of the moisture pins are teflon insulated and the tip of the pins are uninsulated. This prevents any surface moisture content readings and allows moisture content measurement at a specified depth within the timber. The 2 mm diameter pins were inserted 15 mm into the core of each cube. The holes were predrilled 10 mm into the cubes to prevent splitting and driven an extra 5 mm into position as seen in Figure C.1.

![Figure C.1. Moisture probe calibration sample manufacture](image)

There was a fixed distance of 15 mm between each pin pair and they were inserted perpendicular to the grain. The cubes were placed in various relative humidity levels and
allowed to reach the equilibrium moisture content. A known voltage was applied across the pins and the voltage drop was recorded with the use of a voltmeter. Equation [C.1] was used to calculate the electrical resistance of the timber between the two pins.

\[ R_{\text{wood}} = R_{\text{meter}} \left( \frac{V_{\text{battery}}}{V_{\text{meter}}} - 1 \right) \]  

Figure C.2. Resistance method diagram

where:
- \( R_{\text{wood}} \) = resistance of the wood (MΩ)
- \( R_{\text{meter}} \) = resistance of the voltmeter = 10 MΩ
- \( V_{\text{battery}} \) = known voltage applied across pins
- \( V_{\text{meter}} \) = measured voltage

The resistance between each pin was measured at each equilibrium moisture content and a table of log resistance (MΩ) vs moisture content (%) was produced. There are many factors that influence the measurement of moisture content using the electrical resistance method. Timber species, pin spacing, pin type and also the positioning of the pins perpendicular or parallel to the grain influence the result. Due to these factors the calibration of the moisture probes system was required for accurate moisture measurements. Due to the influence of pin type, pin spacing, temperature and species, the curve produced is only suitable to use when measuring moisture content in fast-grown Sitka spruce at 20°C and the pins are positioned as previously stated.

**Results**

Electrical resistance was measured between thirty identically insulated moisture pin pairs inserted into specimens of Irish Sitka spruce. The specimens were exposed to various levels of relative humidity and remained at each level until equilibrium moisture content was achieved. The electrical resistance was measured at each equilibrium moisture
content. Figure C.3 presents the curve of measured electrical resistance vs measured moisture content. It is important to remember that each pin pair was identical to one another and installed with a constant spacing of 15 mm perpendicular to the grain. The measured curve is very different to that provided by James (1988) of the Forest Products Laboratory (FPL) for Sitka spruce; however, there are many differences in the test setup, and pin type. In addition, the FPL used uninsulated stainless steel pins, measuring surface moisture contents and were not concerned with moisture content at specific depths. The various factors that influence the relationship between moisture content and electrical resistance, further justify the importance of this calibration for accurate, reliable experimental data.

![Figure C.3 Moisture probe calibration curve: Resistance vs moisture content](image)

*Figure C.3 Moisture probe calibration curve: Resistance vs moisture content*
APPENDIX D

MOISTURE PROBE RESULTS

This appendix presents the individual probe measurements at each probe location (MC1-MC11).

Figure D.1. Moisture probe location MC1

Figure D.2. Moisture probe location MC2
Figure D.3. Moisture probe location MC3

Figure D.4. Moisture probe location MC4

Figure D.5. Moisture probe location MC5
Figure D.6. Moisture probe location MC6

Figure D.7. Moisture probe location MC7

Figure D.8. Moisture probe location MC8
Figure D.9. Moisture probe location MC9

Figure D.10. Moisture probe location MC10

Figure D.11. Moisture probe location MC11