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DEPARTMENT OF CIVIL ENGINEERING

MSc.ENG THESIS

Continuum Damage Mechanics to Model Glulam Beams Reinforced With Glued-In Steel Cables

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ABSTRACT

This study aims to investigate the use of Continuum Damage Mechanics (CDM) for the computational modelling of timber, a material known for its high variability and anisotropy. Timber has not had a standard way of performing computational mechanics like that of steel and concrete, which has made engineers hesitant to design with the material. Many other concerns, including durability and fire resistance have also hindered its use in modern construction. However, the benefits of timber construction on the environment are well known, and therefore, there is a need for a reliable and accurate computational model for timber.

This study utilizes CDM to model the fibrous portion of timber, similar to Fibre Reinforced Polymers (FRPs), in the hopes of encouraging engineers to build sustainable structures with timber as the main structural material. The material model is implemented in the software ABAQUS using a User-Defined Material UMAT written in Fortran coding. This UMAT allows for the simulation of the nonlinear behavior of timber under different loads and boundary conditions.

To validate the model, experimental tests on glulam beams internally reinforced with steel cables were conducted. The beams had varying diameters of steel cables (3mm, 5mm and 8mm) to investigate the effect of the diameter of the cables on the strength of the beams. The results were compared to the results obtained from a similar setup in the ABAQUS/CAE software. The results show that the UMAT closely represents the failure, strength, and displacement readings of the control experimental tests. However, it was unable to accurately represent interactions between the reinforcing steels and timber.

Despite the lack of accuracy in the interactions between the reinforcing steels and timber, this research concludes that the UMAT is accurate and suitable for solid timber and glulam beams. The UMAT can be used to model the fibrous portion of timber in a similar way to FRPs, and therefore, it will allow engineering predictions to be made with regards to the strength of SA pine timber. This will lead to more sustainable structures and more confidence in timber as a structural material. The model can be used to design and analyze members with more accurate predictions of the structural behavior of timber.

Overall, this research aims to encourage a sustainable future by providing engineers with a reliable and accurate computational model for timber, which will lead to more widespread use of timber in construction. The use of CDM for timber modeling opens up new possibilities for sustainable and environmentally friendly construction. Further research in this field can lead to even more accurate and reliable models for timber, making it a viable alternative to traditional building materials.

Table of Contents

List of Figures	vii
List of Tables	xi
List of Graphs	xii
1 Introduction	1
1.1 Research Aim	2
1.2 Objectives	2
1.3 Background	2
1.4 Research Importance	3
1.5 Scope	3
1.6 Thesis Structure	4
2 Timber Literature Review	5
2.1 Introduction	6
2.2 Timber and Structural Timber in SA	6
2.3 Timber structure	7
2.4 Glulam and Cross-Laminated-Timber	11
2.5 Timber Misconceptions	14
2.5.1 Sustainability	14
2.5.2 Fire	14
2.5.3 Strength	16
2.6 Mechanical Properties of Glulam	17
2.6.1 Modulus of Elasticity	19
2.6.2 Modulus of Rigidity	22
2.6.3 Poisson's Ratio	23
2.6.4 Modulus of Rupture	24
2.7 Modes of Failure	25
2.8 Finger Joints	25
2.9 Timber-Steel Adhesive Bond Strength	27
2.9.1 Adhesive Type	28
2.9.2 Groove Type	29
2.9.3 Steel Bar Variables	29

2.9.4	Failure Modes	30
3	Testing Literature Review	32
3.1	Introduction	33
3.2	Non-Destructive Testing	33
3.3	Destructive Tests	34
3.4	Previous Timber Reinforcement	34
3.4.1	Fiber Reinforced Polymers and Steel	35
3.4.2	Cambering	35
3.5	Previous Tests on Pinus Patula	36
3.5.1	Tensile strength Tests	36
3.5.2	Compressive Strength Tests	37
4	Steel Cables	39
4.1	Introduction	40
4.2	Steel-wire Rope	40
5	Digital Image Correlation - DIC	43
5.1	Introduction	44
5.2	Measurement Principles in a DIC system	44
5.3	Set-up and Results	46
5.4	Navigating the software	47
5.4.1	Pylon Viewer	48
5.4.2	Istra4D	50
6	Modelling	54
6.1	Introduction	55
6.2	Previous Models	55
6.3	ABAQUS Modelling	60
6.3.1	USDFLD-Subroutine	60
6.3.2	Mechano-Sorptive and Rheological Model	61
6.4	Continuum Damage Mechanics	63
6.4.1	Mathematical Failure Criteria	65
7	The UMAT and CDM model	68
7.1	Introduction	69
7.2	Material Definitions	69
7.2.1	Orientation	69
7.2.2	Failure Modes	71
7.3	Damage Evolution	72
7.4	Geometry	73
7.4.1	Mesh Dependency	74
7.5	Input Material Properties	76

8	Theoretical Calculations	80
8.1	Ultimate Moment Resistance - SANS 10163-1	81
8.2	Theoretical Deflection calculations	82
8.2.1	Transformed sections	83
9	Methodology	85
9.1	Introduction	86
9.2	Experimental tests	86
9.2.1	Pull-out Glue Tests	86
9.2.2	Beam Tests	89
9.3	Abaqus CDM Experiments	92
9.3.1	Test Setup:	93
9.3.2	Material Properties:	95
9.3.3	Model Calibration	95
9.3.4	Data Analysis:	100
9.3.5	Conclusion:	100
10	Results	101
10.1	Introduction	102
10.2	Bond Strength Results	103
10.3	Load vs. Deflection Results	104
10.3.1	DIC Deflection Images	106
10.4	Load vs. Strain Results	113
10.4.1	DIC Strain Images	114
10.5	Abaqus Results	121
10.5.1	Load Deflection Analysis	122
10.5.2	Damage Evolution	124
11	Discussion	127
11.1	Introduction	128
11.2	Experimental Results	128
11.2.1	Failure Modes	128
11.2.2	Summary	129
11.3	Computational Results	130
11.3.1	Outcome	130
11.3.2	Material Properties	130
11.4	Summary	131
12	Conclusion and Recommendations	132
12.1	Conclusion	133
12.2	Recommendations	133

13 Bibliography	135
Appendices	
A Linking Abaqus, Visual Studio and Intel OneAPI	145
B UMAT code	150
C Job Safety Analysis	165
D Ethics	168

List of Figures

2.1	Softwood cell structure	8
2.2	Hardwood cell structure	8
2.3	Structural elements of a typical tree	9
2.4	Timber cross-section	10
2.5	CLT and Glulam comparison	11
2.6	Stress distribution across a rectangular beam section	12
2.7	Balanced vs. unbalanced glulam layups	12
2.8	Mid-rise office building from glulam and CLT	12
2.9	charring of glulam after a standard fire test	15
2.10	Steel strength in a fire test	16
2.11	Probability density comparison between glulam and solid timber	17
2.12	Three major axes of a tree	18
2.13	Stress-strain curve for a typical material	20
2.14	Elongation and Contraction under axial loading	23
2.15	Typical finger joint used in glulam production	26
2.16	Finger joint orientation in glulam beams	26
2.17	SANS 10096 dimensions of a finger joint	27
2.18	Groove types for bonded in steel rods	29
2.19	Failure methods of the pull-out tests	31
3.1	Non-destructive test grading machines in South Africa	33
3.2	cambered glulam beam	35
3.3	cambered glulam beam in the factory	36
3.4	SANS 6122 Tensile test	37
3.5	Tensile test used by (Burdzik, 2004)	37
3.6	SANS 6122 Compression test	38
4.1	Steel rope diagram	41
4.2	6×7 FC cross section	41
4.3	6×19 FC cross section	42
5.1	Speckle pattern on the surface of a test specimen	44
5.2	Transformation of an area using DIC algorithms	45

5.3	Components of a Q400 DIC system	46
5.4	Final DIC setup and schematic	47
5.5	Visual comparison of a DIC analysis vs. an Abaqus model	47
5.6	Pylon viewer navigation menus	48
5.7	Recording settings	48
5.8	Pylon viewer camera settings	49
5.9	how to capture images in Pylon viewer	50
5.10	35mm 9x9 black and white calibration target	50
5.11	Istra4D importing data	51
5.12	Istra4D data selection	51
5.13	Istra4D Calibration settings	52
5.14	Istra4D starting a new calibration	52
6.1	Karlsruhe Failure criterion III	57
6.2	Flow chart of Monte Carlo simulation	58
6.3	Tension vs. compression material degradation for USDFLD	62
6.4	Rheological model schematic	62
6.5	Different constitutive damage functions used in modelling of timber	64
6.6	Basic flowpath for a UMAT	66
7.1	Stress components and material directions	70
7.2	Separate stages of the damage evolution	72
7.3	Geometry of the mesh dependency test with 15mm elements	75
7.4	Mesh dependency test for different element sizes	75
7.5	Influence of mesh size on numerical results	76
8.1	Reinforced glulam beam transformed section	83
9.1	Experimental beam and camera setup	86
9.2	Process by which the pull out tests were laminated	87
9.4	Distribution of A blocks in a 6m span	88
9.5	Pull out test setup	89
9.6	Experimental beam dimensions	89
9.7	Groove dimensions for beams B, C and D	90
9.8	Beam preparation at OT Joiners factory	90
9.9	The painting process and resulting speckle pattern achieved	91
9.10	Experimental beam test assembly in the lab	91
9.11	Selecting the material constants for a UMAT in Abaqus	93
9.12	Model Generated In Abaqus CAE For a Bending Test	94
9.13	Selecting a UMAT File In Abaqus	95
9.14	Model Calibration: Compression Experimental Results	97
9.15	Model calibration: Compression Tests Comparison	98
9.16	Model Calibration - Tension Test	99

10.1	Setup of experimental tests with a view through the DIC camera	102
10.2	Beam A1 at 10 kN and 0.5s before failure	106
10.3	Beam A2 at 10 kN and 0.5s before failure	107
10.4	Beam A3 at 10 kN and 0.5s before failure	107
10.5	Beam A4 at 10 kN and 0.5s before failure	107
10.6	Beam A5 at 10 kN and 0.5s before failure	108
10.7	Beam B1 at 10 kN and 0.5s before failure	108
10.8	Beam B2 at 10 kN and 0.5s before failure	108
10.9	Beam B3 at 10 kN and 0.5s before failure	109
10.10	Beam B4 at 10 kN and 0.5s before failure	109
10.11	Beam B5 at 10 kN and 0.5s before failure	109
10.12	Beam C1 at 10 kN and 0.5s before failure	110
10.13	Beam C2 at 10 kN and 0.5s before failure	110
10.14	Beam C3 at 10 kN and 0.5s before failure	110
10.15	Beam C4 at 10 kN and 0.5s before failure	111
10.16	Beam C5 at 10 kN and 0.5s before failure	111
10.17	Beam D1 at 10 kN and 0.5s before failure	111
10.18	Beam D2 at 10 kN and 0.5s before failure	112
10.19	Beam D3 at 10 kN and 0.5s before failure	112
10.20	Beam D4 at 10 kN and 0.5s before failure	112
10.21	Beam D5 at 10 kN and 0.5s before failure	113
10.22	Line along which strain data was exported and graphed	113
10.23	Beam A1 global strain vs failure point, 0.5s before failure	115
10.24	Beam A2 global strain vs failure point, 0.5s before failure	115
10.25	Beam A3 global strain vs failure point, 0.5s before failure	115
10.26	Beam A4 global strain vs failure point, 0.5s before failure	116
10.27	Beam A5 global strain vs failure point, 0.5s before failure	116
10.28	Beam B1 global strain vs failure point, 0.5s before failure	116
10.29	Beam B2 global strain vs failure point, 0.5s before failure	117
10.30	Beam B3 global strain vs failure point, 0.5s before failure	117
10.31	Beam B4 global strain vs failure point, 0.5s before failure	117
10.32	Beam B5 global strain vs failure point, 0.5s before failure	118
10.33	Beam C1 global strain vs failure point, 0.5s before failure	118
10.34	Beam C2 global strain vs failure point, 0.5s before failure	118
10.35	Beam C3 global strain vs failure point, 0.5s before failure	119
10.36	Beam C4 global strain vs failure point, 0.5s before failure	119
10.37	Beam C5 global strain vs failure point, 0.5s before failure	119
10.38	Beam D1 global strain vs failure point, 0.5s before failure	120
10.39	Beam D2 global strain vs failure point, 0.5s before failure	120
10.40	Beam D3 global strain vs failure point, 0.5s before failure	120
10.41	Beam D4 global strain vs failure point, 0.5s before failure	121
10.42	Beam D5 global strain vs failure point, 0.5s before failure	121

10.43	Abaqus deflection contours for Beam C	123
10.44	Damage Prediction for the Control Beam at Failure	125
10.45	Damage Prediction for the Control Beam at Failure - Centre Portion	126
10.46	Damage pattern on a control beam	126
11.1	Failure Mode Beam B	129
A.1	Visual Studio Configuration	146
A.2	OneAPI configuration	146
A.3	How to change the environment paths	147
A.4	Edited file for abq2019.bat	148
A.5	edited portion of the abaqus_v6.env file	148
A.6	Abaqus linking verification	149

List of Tables

2.1	Burning process of timber	15
2.2	Strength retention after fire tests	15
2.3	Strength comparison between different materials	17
2.4	Weakening ratio for finger joint length	27
2.5	Adhesive bond strength compared to epoxy resins	28
2.6	Single groove vs. Double groove	29
2.7	Minimum edge distance for embedded steel	31
3.1	MoR non destructive predictors	34
4.1	Steel wire strength grades	40
4.2	Minimum breaking force factors	42
6.1	Advantages and Disadvantages of using CDM	65
8.1	Beam names and reinforcing configurations	84
9.1	Material Property Assigned Values For Pinus Patula	96
10.1	Pull-out test results	103
10.2	Beam names and reinforcing configurations	122
C.1	Production and Delivery JSA	166
C.2	Testing JSA	167

List of Graphs

2.1	Degradation of Poisson ratio over time when a force is applied	24
8.1	Theoretical deflections using equation 8.2	84
9.1	Calibration - Parallel Compression	98
9.2	Calibration - Perpendicular Compression	98
9.3	Calibration - Parallel Tension	99
10.1	Pull-out glue strength for blocks with glued in cables	104
10.2	Un-reinforced glulam beams	104
10.3	Five 3mm steel cables	104
10.4	Four 5mm steel cables	105
10.5	Three 8mm steel cables	105
10.6	Average Max Load vs. Average Max deflection for glulam beams	105
10.7	Maximum moment vs. Max deflection for glulam beams	106
10.8	Load-Strain for unreinforced beams	114
10.9	Load-Strain for 5 x 3mm cables	114
10.10	Load-Strain for 4 x 5mm cables	114
10.11	Load-Strain for 3 x 8mm cables	114
10.12	Abaqus Deflection - Beam A	123
10.13	Abaqus Deflection - Beam B	123
10.14	Abaqus Deflection - Beam C	124
10.15	Abaqus Deflection - Beam D	124

Nomenclature

Abbreviations

ABAQUS/CAE	Complete Abaqus Environment
AISC	American Institute of Steel Construction
AITC	American Institute of Timber Construction
ALE	Arbitrary Lagrangian-Eularian
APA	The Engineered Wood Association
ASTM	American Society for Testing and Materials
CBM	Crack Band Model
CDM	Continuum Damage Mechanics
CLT	Cross-Laminated Timber
CORRIM	Consortium for Research on Renewable Industrial Materials
EPO	Epoxy resins
FC	Fibre Core
FEA	Finite Element Analysis
FEM	Finite Element Modelling
FM	Failure Modes
FRP	Fibre Reinforced Polymer
FSC	Forest Stewardship Council
Glulam	Glue-Laminated Timber
KAR	Knot-Area-Ratio
LEFM	Linear Elastic Fracture Mechanics

MoE	Modulus of Elasticity
MoR	Modulus of Rupture
MUF	Melamine-Urea-Formaldehyde
NDT	Non-Destructive Tests
NLFM	Non-Linear Fracture Mechanics
PRF	Phenol-Resorcinol-Formaldehyde
PROLAM	Probabilistic modeling of glued-laminated timber
PUR	Polyurethane
ReLAM	Non-linear probabilistic modeling of reinforced laminated timber
SA	South Africa(n)
SC	Steel Core
UMAT	User-defined material model
USDFLD	Defines field variables at a material point
UTC	Ultimate Compression Stress
UTS	Ultimate Tensile Stress

Symbols

α_e	Modular ratio
$\bar{C}$	Local compliance matrix
$\bar{\sigma}$	Local elastic stress
$\bar{\varepsilon}_e$	Local elastic strain
δ	Deflection
$\forall$	For all values
ν	Volume
λ	Wavelength
ρ, ρ^c	Density, Density power function for bond strength with $0 < c < 1.5$
Φ	Curvature
ν	Poisson ratio

A	Cross-sectional Area
A_s	Area of reinforcing steel
$c_{\perp}, c_{\parallel}$	Compression perpendicular and parallel to-the-grain
D	Stiffness matrix
E	Young's modulus / Elastic Modulus
E_{ac}, E_{dyn}, E_{th}	Modulus of elasticity, actual, dynamic and theoretical
f	Frequency
F_{min}	Steel wire breaking force
G	Modulus of Rigidity/Shear Modulus
I_g	Gross moment of inertia
I_k	Moment of inertia of all knots within 15cm of the critical cross section
K	Steel wire breaking force factor
M	Bending moment
R^2	Coefficient of correlation
R_r	Rope grade
T	Wave period
$t_{\perp}, t_{\parallel}$	Tension perpendicular and parallel to-the-grain
V	Shear
$\varnothing_d$	Diameter

Units

$^{\circ}C$	Degrees Celcius
kN	kiloNewtons
kNm	kiloNewton-meter
MPa	MegaPascals
W/mK	Watts per meter-Kelvin

Chapter 1

Introduction

1.1 Research Aim

The objective of the research is to investigate the increase in flexural strength and stiffness of SA pine glulam beams which have been reinforced with high tensile steel wires similar to those used in pre-stressed concrete. The study also aims to develop a constitutive model for SA pine based on continuum damage mechanics. The computational model will be compared to experimental tests carried out on full scale beam layups. By developing a computational model specific for SA Pine it is envisaged that this will encourage the use of timber in the south African construction industry.

1.2 Objectives

The objectives of this study are:

1. Collect data from experimental tests on the reinforcement of glulam timber beams with steel cables of different diameters.
2. Evaluate the feasibility of the selected reinforcing method in terms of costs, sustainability, and practicality in the South African market.
3. Validate a User Defined Material Model in Abaqus Finite Element Analysis software to accurately model results from experimental tests.
4. Develop a reliable computational modelling approach for the timber species *Pinus Patula* using the validated User Defined Material Model.

1.3 Background

A previous study was conducted by the author where solid-sawn timber was externally reinforced with different reinforcing configurations of steel and fibre reinforced polymers (Rhodes, 2019). One of the suggestions for future research was that steel bars be used in a glued-laminated beam layup, for internal reinforcing of timber that could be achieved during the production phase or as a repair mechanism. A further suggestion was that a method for computational modelling of SA Pine be developed in order to help with future design standards with the goal of encouraging future designs to include structural members made from timber.

In recent years, the main goal of engineers has been to develop new innovative ways by which to provide sustainable solutions for the world in order to reduce the effects of global warming. To do this engineers are trying to reduce their carbon footprint by finding new materials with which to build structures such as high-rise buildings, which are necessary to meet infrastructure needs in the world.

Concrete is often used for infrastructure in the built environment however, the carbon emissions from producing cement are one of the highest contributors to the global warming crisis. Finding a material that requires little to no cement will allow engineers to significantly reduce the carbon footprint of the construction industry.

One of the latest materials to be used is glued-laminated timber (glulam). Timber has been used in construction for thousands of years, however in modern times it is only used for menial purposes and

not to support heavy loads, with roofs largely being the remaining exception in South Africa. With the development of glulam the uses for timber has increased and there are now an increasing number of mid-rise buildings, decks and small bridges being constructed from glulam products. Many engineers, as well as, architects are calling these glulam structures a necessity if the construction industry is to reduce its carbon footprint.

Finding an efficient way to reinforce and model reinforced glulam structures will encourage many designers to consider the use of timber as a structural material.

There have been a large number of research studies conducted on reinforcing timber with fibre reinforced polymers (FRPs) such as carbon fibre (CFRP) and glass fiber (GFRP) these have been found to be effective but are considered an expensive alternative.

1.4 Research Importance

Fiber reinforcing options were considered as expensive due to the high price of the fiber material. However, with the increased strength the amount of high grade timber needed along the tensile face can be reduced. This will result in a reduced volume of material needed for the construction of buildings. Reinforced glulam components have a lower product variability than that of regular timber as their strength is not significantly dependent on natural growth characteristics, such as knots.

By using steel as a reinforcing material, the benefits of fiber reinforcing could still be retained while reducing the overall cost, as steel is cheaper than fiber polymers, especially high end polymers such as the carbon fibres.

A cheaper option of reinforcement, coupled with the many other added benefits of reinforced glulam members, may encourage more designers to use this as a viable construction material and in turn lead to sustainable material use within the construction industry.

The development of a 3D constitutive CDM material model for wood has only been undertaken for a few timber species. *Pinus Patula* is the most common wood used in South African construction and the research into it's properties and development of FEM models has been very limited which results in the material being under utilised. Using a material model that can easily have input properties changed according to the species or timber grade available is a useful aspect of any computational model. Abaqus finite element analysis software allows for such an input method that, once the material model has been coded, can be changed quickly for efficient analysis.

1.5 Scope

This thesis will have two main parts, an experimental approach and a modelling approach. For both cases the the timber that will be used is of the species *Pinus Patula* (SA pine). The FEM model that will be used and refined for the purpose of this thesis was originally developed for three different species; Beech, Spruce and Azobé. The experimental tests will be used to calibrate the FEM model which will be developed in ABAQUS using FORTRAN coding language to write a user defined material (UMAT) input for the ABAQUS software.

Wherever possible the South African National Standards (SANS) design codes will be used for testing procedures and calculations. Where these standards are not sufficient other international standards,

mainly Eurocode 5 and ASTM standards, will provide the missing information. In the SANS documents it should be noted that material properties for SA pine in all directions is not defined and these values will have to be taken from other sources.

Although only one species of timber is being tested, the sample size for the experimental aspect is relatively small due to the specialized manufacturing procedure and limited budget. The small sample size may lead to statistical inaccuracies when calibrating the computational model.

1.6 Thesis Structure

The thesis is divided in to 12 chapters:

Chapter 1 serves as an initial overview of the thesis, outlining the goals and objectives of the research work. The chapter emphasizes the importance and relevance of the study and provides a thorough examination of the background knowledge and current state of research within the field.

Chapter 2 provides an in depth look at timber as a material by focusing on its internal structure, and the available engineered timber products. Then focusing on glued-laminated timber the material properties will be discussed, as well as the importance of finger joints, adhesive bond, and different test procedures. Finally, previous studies of timber reinforcing are summarised here.

Chapter 3 looks at and types of non-destructive and destructive test procedures done on timber specimens and how these are used to determine material properties for a timber species.

Chapter 4 summarises the properties and selection process behind using high tensile steel cables over other materials.

Chapter 5 explains the Digital Image Correlation (DIC) procedure which is used for experimental data analysis instead of the standard strain gauges and dial gauge setup. Discussed in this chapter are the mechanics behind how the software tracks the movement as well as a step-by-step guide on how to navigate through the software menus.

Chapter 6 is focused on investigating previous modelling techniques used by different researchers. Many models exist for timber depending on what is being investigated and it is important to determine which of these models is suitable for the current thesis. The modelling literature review provided in this chapter will help with making a suitable choice for the model to be developed in this research.

Chapter 7 will give a full description of the Continuum-Damage-Mechanics model that has been selected for analysis. Also discussed in the chapter will be the UMAT code for ABAQUS which will allow the material to be modelled using Finite Element Analysis.

Chapter 8 will present theoretical calculations to determine the size of reinforcement required and the predicted bending strength of the beams.

Chapter 9 will describe the experimental test setup used to determine the adhesive strength and the actual bending strength of the control and reinforced beams.

Chapter 10 will present the results for both the experimental tests and the FEM results. Comparisons will be drawn and the reliability of the CDM model will be discussed

Chapter 11 and **12** will summarise and discuss the experimental and computational testing and provide a conclusion to the research.

Chapter 2

Timber Literature Review

2.1 Introduction

Extensive research into the properties of steel and concrete have made them the foremost material choice for many construction projects. Reinforced concrete is a combination of the two materials which is able to take the best components of each material and combine them into a composite material that can be used in almost any situation. The production of this composite material requires the production of cement which is one of the main contributors to carbon (CO₂) emissions from the construction industry. With many countries now calling for sustainable alternatives to be used wherever possible, the responsibility now falls on structural engineers and architects to produce new buildings and structures with a lower carbon footprint.

One possible alternative to the extensive use of reinforced concrete is that of timber, more specifically reinforced glulam timber. Using timber reduces the amount of cement required, and although steel is still used with this composite material the decreased weight of the structure and the increased bending strength of the timber (provided the bending force is parallel to the grain) should ensure that the amount of steel required is reduced. Many people have their doubts with regards to timber as a suitable material. This is addressed in section 2.5 where the stigma surrounding the material is addressed.

Presented here is a summary of literature for glued-laminated timber (glulam). In order to develop a three-dimensional continuum damage mechanics (CDM) model for a material it is important to have a good understanding of the material's stiffness and strength properties. For timber and glulam the material properties are difficult to understand as the material is considered to be anisotropic. Assumptions can be made to simplify these properties; however, knowing the impact of how these assumptions will affect the CDM model will result in the correct choice of failure criterion being chosen. Modelling will be discussed in further detail in section 6.

In this chapter, the anatomy of the tree will be discussed. Although both hardwoods and softwoods are investigated at first, with a comparison provided between the two in section 2.3 it should be noted early on that the species of timber being researched, *Pinus Patula*, is a softwood.

Timber testing procedures from multiple standards are discussed. Both destructive and non-destructive are considered and will be used in a variety of ways. Non-destructive tests allow for an estimation of mechanical properties such as modulus of elasticity, moisture content and bending strength to be determined. When conducted, the non-destructive test values can be used for theoretical calculations and as inputs for material properties in the CDM model.

Timber cell structure, growth patterns, and mechanical properties of timber are covered extensively in literature by; Silvester and Kape (1967); Green et al. (1999); Thelandersson and Larsen (2003); Smith et al. (2003); Harte (2009) and Sandhaas et al. (2020).

For further information on SA pine in particular the reader is directed to the work of Dr. Brand Wessels (Wessels et al., 2014, 2015) who conducts research on *Pinus patula* with regards to its surroundings, age, properties and the sawmills in which it is manufactured for commercial use.

2.2 Timber and Structural Timber in SA

Timber is a structural material that has been used for thousands of years. Traditionally it has been used as solid timber pieces to provide structural supports as far back as 9000BC (Foliente et al.). In modern

times solid timber is still used in construction; however, a relatively new engineered form of timber called glue-laminated timber (referred to as glulam) has been invented.

South Africa, similar to the rest of the world, uses reinforced concrete as its main building material with timber used as a secondary material, often only for aesthetic purposes. In South Africa the main species of timber used for structural purposes is of the genus *Pinus* (SANS1783-1, 2018) and further classified as *Pinus Patula* (Dowse and Wessels, 2013). This is a softwood tree and is used mainly for its high strength:weight ratio as well as its relatively short growth time of 20 years. Softwoods and hardwoods will be discussed further in section 2.3

Timber is not a material that can be manufactured reproducibly in order to fit a certain specification or grade. Since the material is grown naturally, deviations in strength and many another mechanical properties, even amongst trees of the same species, will occur. These deviations are due to the fact that trees are highly susceptible to environmental and loading conditions.

Timber is different to many other materials used for engineering purposes. It is an anisotropic material meaning that the material has varying properties depending on the orientation of its molecular axis (growth direction). Therefore in the strength testing of timber the angle of loading with respect to the angle of wood grain is an important aspect to consider. Timber, in general has a high strength and stiffness parallel to the grain direction but is weaker in directions perpendicular to the grain (Harte, 2009).

2.3 Timber structure

Although the scope of this study does not require further research on timber growth mechanisms as these are well documented, it is important still, to have a relatively thorough understanding of what is inside a tree. This refers to both a macro-level as well as a micro-level. This will provide a deeper understanding as to how to model timber by looking at its physical make up.

Trees can be divided into two main groups; softwoods and hardwoods. In general softwoods are less dense than hardwoods and grow at a faster rate. Due to this they are used more extensively for structural use as the trees can be replenished faster but also have a high weight to strength ratio. The most common softwood used in construction is Pine. Pine can be treated to resist decay and rot, and it is ideal for the construction of frames (PineTimberProducts, 2015).

A more academic approach to the differentiation between the two types of wood is defined by Silvester and Kape (1967). They differentiate between a hardwood and softwood by using the cell structure of the tree.

A softwood tree is built from long, needle-like cells called ‘tracheids’ which are orientated along the longitudinal axis of the tree. These tracheids are separated along the radial axis by smaller cells which are referred to as ‘rays’. Tracheids are responsible for the vertical conduction of water and the rays are responsible for the lateral conduction of water. Figure 2.1 shows the cell structure of a softwood tree in regards to its relative axis.

Hardwoods follow a similar cell pattern to that of softwoods. Tracheids are now replaced with a mixture of large and small cells known as ‘vessels’ and ‘fibres’ respectively. These are orientated in the longitudinal axis of the tree. However, unlike in softwoods where each cells main objective is the conduction of water, in a hardwood tree the main function of vessels is the conduction of water and

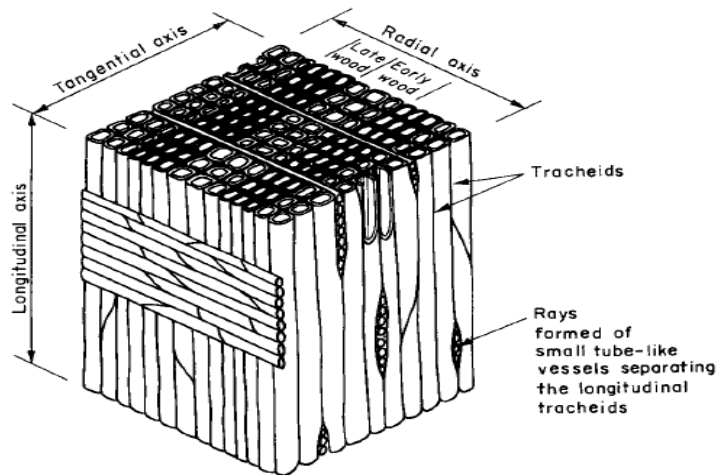


Figure 2.1: Softwood cell structure
(Silvester and Kape, 1967)

the main function of fibers is to increase structural strength. Figure 2.2 shows the cell structure of a hardwood tree.

Figure 2.3 shows how the cells align within a softwood and hardwood tree, to develop the internal structural elements.

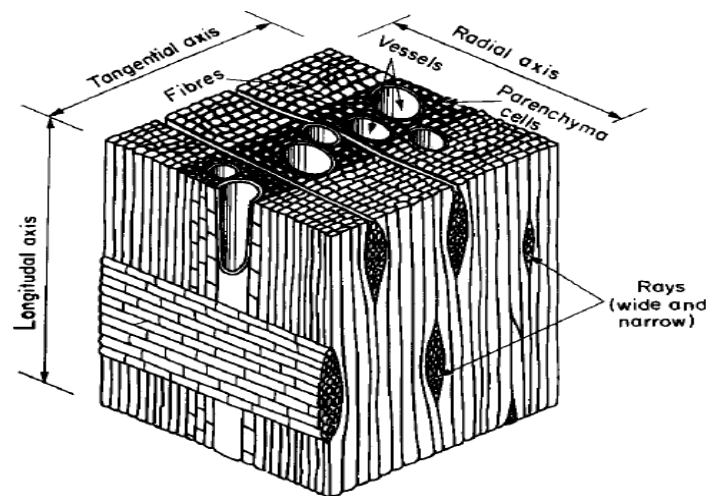


Figure 2.2: Hardwood cell structure
(Silvester and Kape, 1967)

A timber cross section can be classified into different sections, each performing a different role within the structure of a tree. Figure 2.4 shows the different features within the cross section. Working from the innermost feature outwards, these are the core or pith, heartwood, sapwood, cambium and the bark (Harte, 2009). The heartwood is usually darker in colour and is generally stronger and more resistant to decay than other parts of the tree (Encyclopædia-Britannica, 2012a). The sapwood and cambium layers vary in width depending on the growth rate and age of the tree, commonly they comprise 20 % to 60 % of the cross sectional area (Dinwoodie, 2000). It is considered to be the living layers which are responsible for the transportation of minerals and water molecules and these are lighter in colour (Encyclopædia-

Britannica, 2012b). The cambium layer consists of actively dividing cell tissue which results in an increase in tree diameter and growth of stems and roots which protrude from the tree (Encyclopædia-Britannica, 2014).

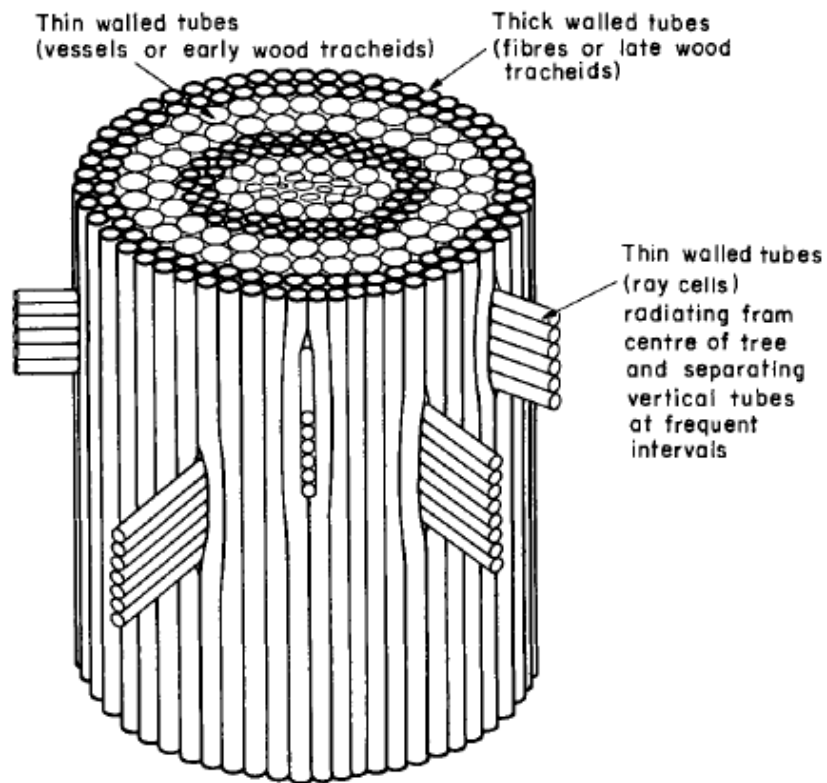


Figure 2.3: Structural elements of a typical tree
(Silvester and Kape, 1967)

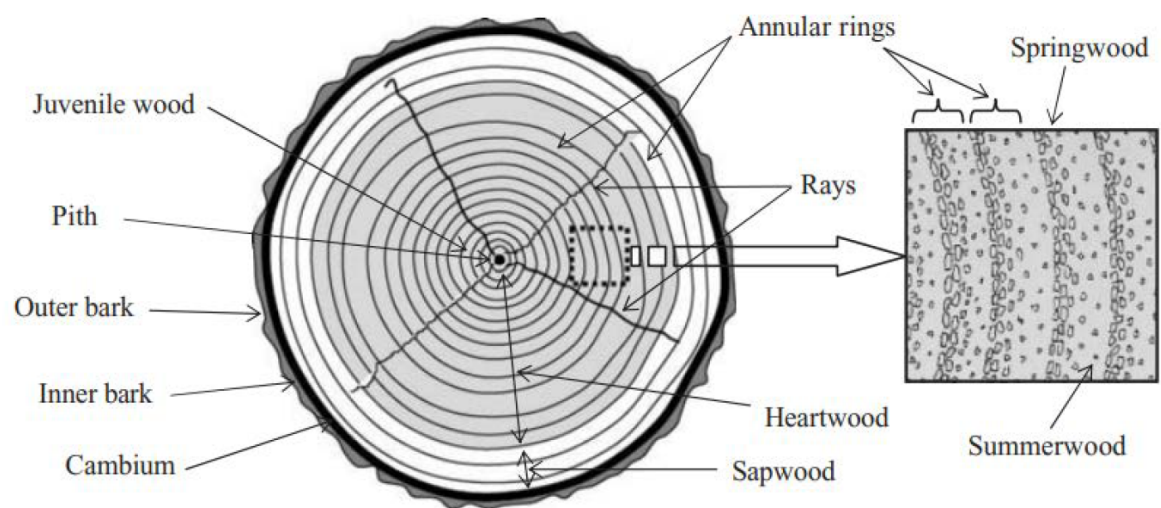


Figure 2.4: Timber cross-section
(Harte, 2009)

2.4 Glulam and Cross-Laminated-Timber

The current study is focused on engineered timber and not solid timber. More specifically the research focuses on glue-laminated timber (glulam), which is one of many different types of engineered timber products. The two most common types of engineered timber products used in structural engineering are glulam and cross-laminated timber (CLT). These two types of engineered timber are often confused with each other, therefore although CLT is not within the scope of this research it is important to differentiate between the two products.

The two products are manufactured in much the same way, using the lamination of multiple timber boards to create a member that is larger and stronger than solid timber. The main benefit of these two products is that it mitigates most of the inconsistencies that occur within solid timber.

Cross-laminated timber (CLT), shown in figure 2.5 is manufactured by taking timber and aligning the grain at 90 degrees to each other in alternating layers (WMSH, 2019). Due to the alternating grain direction the strong anisotropic properties of the timber are reduced. CLT is generally strong in both the longitudinal and transverse directions as the timber grain (tracheids) runs in both directions, and is thus mainly used for large 2-way spans.

CLT usually has an odd number of layers with the outer layers being orientated in the same direction (Pei et al., 2013). This ensures that expansion and contraction due to moisture are consistent through the member. The direction with the higher number of layers is called the primary axis (Longitudinal Layers) and the other direction is referred to as the secondary axis (Transversal Layers)

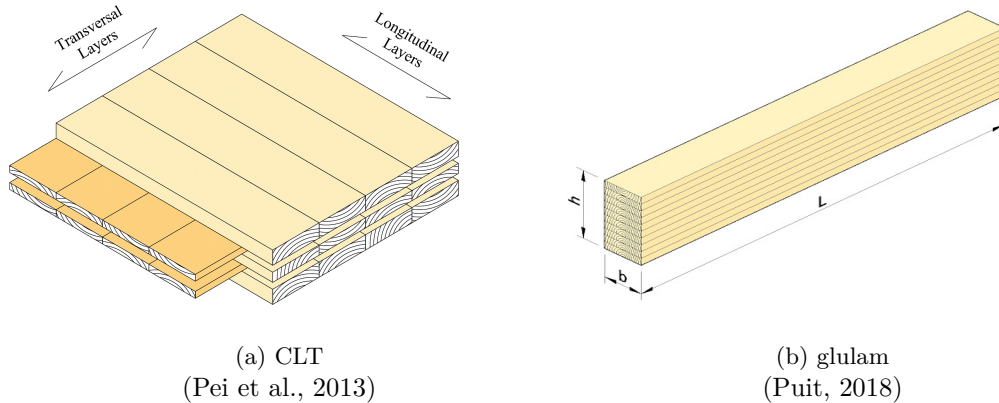


Figure 2.5: CLT and Glulam comparison

Glue-laminated timber (Glulam), shown in figure 2.5, is manufactured by taking multiple timber boards and aligning the grain in the same direction. These boards are then covered in an adhesive and clamped together. To create long sections each beam is end-jointed to another beam using finger joints (see more in section 2.8). The Glulam beams expand and contract evenly and in each direction due to the orientation of the grains in each layer being kept constant (WMSH, 2019). The glulam beams have increased strength along the primary direction, which makes them the main choice for load-bearing beams and columns (HyneTimber, 2019).

Glulam members can be manufactured with the highest quality timber placed at the external surfaces, where the applied stresses are greatest, and with a lower grade timber in the middle of the beam where stresses are low (Yeh, 2003). Figure 2.6 shows a standard elastic stress distribution across an unreinforced

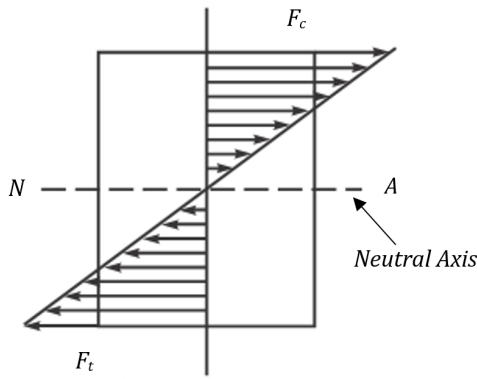


Figure 2.6: Stress distribution across a rectangular beam section

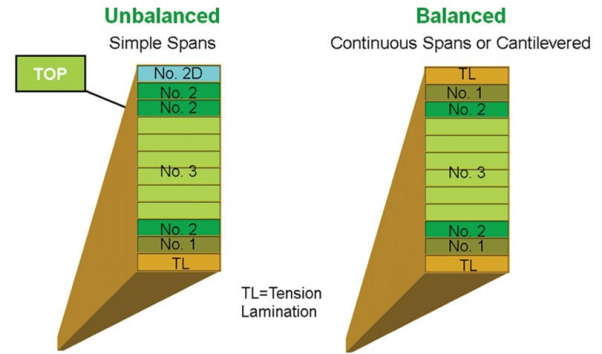


Figure 2.7: Balanced vs. unbalanced glulam layups

beam cross section. A further classification of the beam composition can determine whether the glulam beam is balanced or unbalanced (APA, 2020). An unbalanced section (see figure 2.7) has the strongest laminations along only either the tension or compression face of the beam, depending on where the highest stress concentrations will be found, and a balanced beam has laminations which are symmetrical about the neutral axis of the beam. The benefit of manufacturing beams in this manner is that by using lower grade timber in the middle of the beam, manufacturers are able to reduce overall costs without compromising structural stability (APA, 2020).

Using a combination of glulam and CLT an entire mid-rise building can be built, this has been done on occasion in the construction industry and is becoming a popular choice in recent times (Zumbrunnen and Fovargue, 2012). The combination of these two engineered products can be seen in Brisbane, Australia. 25 King street, shown in figure 2.8 is constructed almost entirely from CLT and glulam (Aurecon, 2018).



(a) Exterior



(b) Interior

Figure 2.8: 25 King Street, Brisbane, Australia: Mid-rise office building from glulam and CLT (Aurecon, 2018)

Due to the variability of timber, many countries have differing design standards and requirements for timber products and design. For solid-sawn timber european countries use the Eurocode 5 1995 design code. For glulam testing and design procedures often, the *American Society for Testing and Materials* - *ASTM* standards are generally more complete. The *South African National Standards* - *SANS* derive their content heavily from the *Eurocode* standard. The following standards, although not all have been

referenced directly, are all relevant for the design and testing of glulam timber:

IS EN: Eurocode 5

- 338:** Structural timber - strength classes
- 385:** Finger jointed structural timber - requirements
- 390:** Glued laminated timber - sizes - permissible deviations
- 1194:** Glued laminated timber - strength classes
- 14080:** Glued laminated timber - requirements
- 14081:** Strength graded rectangular cross-section structural timber

ASTM: American Society for Testing and Materials

- D198-15:** Standard Test Methods of Static Tests of Lumber in Structural Sizes
- D1101-97:** Standard Test Methods for Integrity of Adhesive Joints in Structural Laminated Wood Products for Exterior Use
- D2559-12:** Standard Specification for Adhesives for Bonded Structural Wood Products for Use Under Exterior Exposure Conditions
- D3737-18:** Standard Practice for Establishing Allowable Properties for Structural Glued Laminated Timber (Glulam)
- D4688-14:** Standard Test Method for Evaluating Structural Adhesives for Finger Jointing Lumber
- D6874-12:** Standard Test Methods for Nondestructive Evaluation of Wood-Based Flexural Members Using Transverse Vibration
- D7199-07:** Standard Practice for Establishing Characteristic Values for Reinforced Glued Laminated Timber (Glulam) Beams Using Mechanics-Based Models
- D7341-14:** Standard Practice for Establishing Characteristic Values for Flexural Properties of Structural Glued Laminated Timber by Full-Scale Testing
- D9-12:** Standard Terminology Relating to Wood and Wood-Based Products

SANS: South African National Standards

- 1460:** Laminated timber (glulam)
- 6083:** Modulus of elasticity and modulus of rupture in static bending of solid and glulam timber in structural sizes
- 6121:** Bond quality of laminated timber products
- 6122:** Structural timber - Characteristic values of strength-graded timber

2.5 Timber Misconceptions

The main purpose of this research is to find a cost effective way of reinforcing timber for various structural applications including high-rise buildings when compared to previous reinforcements of FRPs, this will thus encourage the use of glulam timber in structures. However, in modern times the cost of reinforcing is not the only contributing factor that affects the use of a material and although not in the experimental scope of this research, these other factors need to be addressed in order to encourage the switch from concrete and steel structures.

Dr. Brand Wessels, head of the forest and wood science department at Stellenbosch University, insists that removing the misconceptions about timber as a construction material is a key aspect that needs to be addressed in order to increase the use of timber.

2.5.1 Sustainability

The most common timber used for construction in South Africa is *Pinus Patula*, which has growing area of 338 923 ha. in the country, of which 70% is used for construction (Dowse and Wessels, 2013; DAFF , Department of Agriculture and Fisheries; Crickmay and Associates, 2009). The misconception that the use of this timber is bad for the environment stems from the mass deforestation that takes place in natural forests around the world. The timber used for construction is produced in plantations that are maintained by regulatory bodies such as the Forest stewardship Council (FSC). The forests are regulated so that more trees are planted than what are harvested, which ensures the timber reserves are never depleted. Wessels (2019) states that these plantations are like the factories that have the sole purpose of producing timber in a controlled manner that does not contribute to global warming or deforestation. Rodrigues et al. (2017) and CORRIM (Consortium for Research on Renewable Industrial Materials) examined the life cycle of a timber building, and when taking a ‘cradle to grave’ approach found that a timber building sequesters more CO₂ than what it takes to produce (APA, 2020). This includes harvesting the raw material, transportation, production and construction phases of the life cycle. Studies suggest that with efficient design and the use of local timber, the sequestering of CO₂ can be greatly improved (Crafford et al., 2017). One study conducted a ‘cradle to grave’ case study on timber building weighing 721 tons, and found that the trees used sequestered a total of 1192 tons of CO₂. Transport and production of the timber resulted in 47 tons of CO₂ production and construction resulted in 45 tons of CO₂. Overall 1100 tons of CO₂ was sequestered by the timber used in the construction of the building (Darby et al., 2013).

2.5.2 Fire

Although the burning process of timber is very well researched and understood, it is often the first concern that people have when discussing the idea of a timber structure, there are many questions posed by designers and the public alike (Law and Hadden, 2020).

Timber is a charring material, and the rate of charring is the dominant factor in determining a beams resilience to strength loss when in a fire (Yang et al., 2009). To determine the charring rate of timber a standard fire test is performed according to CNS12514, here a beam is placed inside a furnace with temperature readings of 540, 705, 840, 705, 840, 895, and 925°C at interval times of 5, 10, 30, 45, and 60 minutes respectively. Beams are then allowed to cool to room temperature while still inside the furnace. Law and Hadden (2009) go on to discuss what happens to timber at specific temperatures and the findings

are summarised in table 2.1. A standard glulam beam will char at a rate of 0.35 - 0.70 mm/min, resulting in only 21mm of damage occurring 30 minutes after the initial onset of charring at around 300°C (Frangi and Fontana, 2003), (Bartlett et al., 2015). For further details on fire resistance/protection designers should consult SANS10400-T (2011).

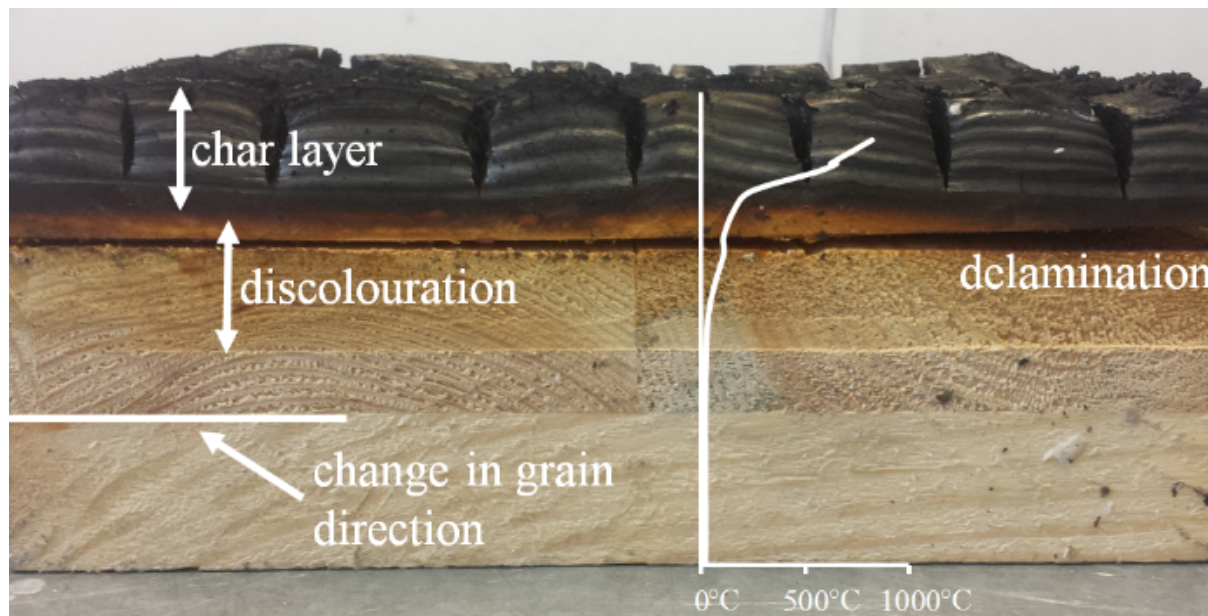


Figure 2.9: charring of glulam after a standard fire test (60 minutes)
(Bartlett et al., 2015)

Table 2.1: Burning process of timber

Temperature °C	Process	Effect
>100	Drying	Mass loss due to moisture loss
100 - 200	-	-
200 - 350	Pyrolysis	Rapid mass loss due to loss of pyrolysis gasses
350 +	Char-oxidation	Ignition and charring of the outer layers

The beam outer layers begin to char and insulate the inner layers from the flames keeping them cooler for a longer period of time (APA, 2020). The charred layers provide no structural strength and thus the reduction in cross sectional area causes a loss of strength. Research conducted by Yang et al. (2009) found the strength retention rates for five different timber species after a standard CNS12514 fire test, and the results are summarised in table 2.2

Table 2.2: Strength retention for specific test procedures after a CNS12514 fire test.

Test Procedure	Retention Rate (%)
MoE	61 - 92.7
MoR	42 - 75.9
Compression	~ 95
Shear	No significant difference

The glulam beams they tested were classified as balanced (see section 2.4) and for buildings that may have a fire risk an unbalanced section is advised. A balanced section will have stronger laminations in the outer cross section layers and will have lower strength retention rates than an unbalanced beam where all timber laminations are the same strength classification (Yang et al., 2009).

Many steel and concrete associations will quickly point out that timber does contribute to the fuel of a fire once the member reaches a temperature of 350°C (MBMA; AISC, 2020). This is true, but as pointed out by APA (2009) “wood’s low thermal conductivity” allows it to remain cooler for a longer period of time. Steel has a thermal conductivity almost 400 times higher than wood (55 W/mK vs. 0,15 W/mK). This means that the steel will heat up much faster than the wood. Although the steel may not contribute to the fire, it is well documented that steel strength declines rapidly once it has reached a temperature of 250°C (Kirby, 1997; Wang, 2000). Figure 2.10 shows the rapid decline of steel strength in a sustained fire. After 30 minutes of sustained fire glulam timber retains 75% of its structural capacity, while steel retains only 10% (Yang et al., 2009; AITC, 2004). Steel reinforcing beams embedded in the glulam beams will experience temperatures significantly lower than ambient temperatures due to the low thermal conductivity of the protective layer of timber (Aicher et al., 1999).

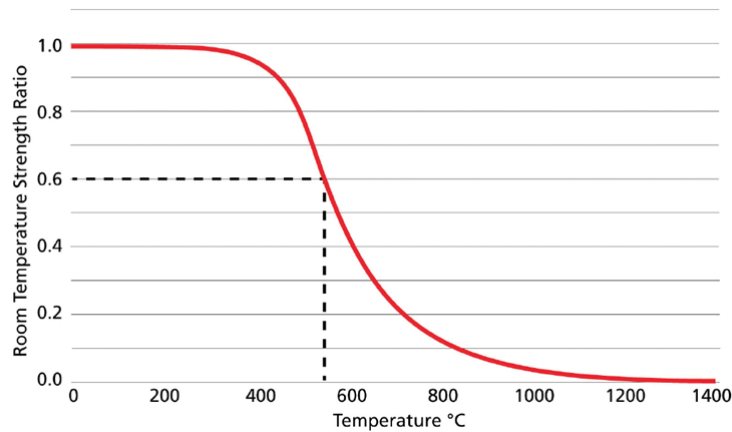


Figure 2.10: Degradation rate of steel with regards to internal core temperature of untreated steel

Timber (and all other building materials) can also be treated to resist fire damage, although the industry standard fire protection has been used in the experiments conducted in section 9, as further research on this is not within the scope of this study.

2.5.3 Strength

The variability in timber strength has led to many people disregarding timber as a suitable construction material. Concrete can achieve consistent strengths of 20 - 120 MPa in compression depending on the mix design used (The Concrete Institute, 2013). Timber on the other hand has inconsistencies, such as knots, that may lead to a structural member failing at unsafe loading. As shown in figure 2.11 the distribution of timber strengths is spread out over a wide range. The use of glulam timber is meant to not only increase strength, but also result in a lower standard deviation of strength values. The reason for the reduced variability is given by Thelandersson and Larsen (2003) as “the growth defects of solid wood are either removed in the production process or distributed in the finished product so that the strength

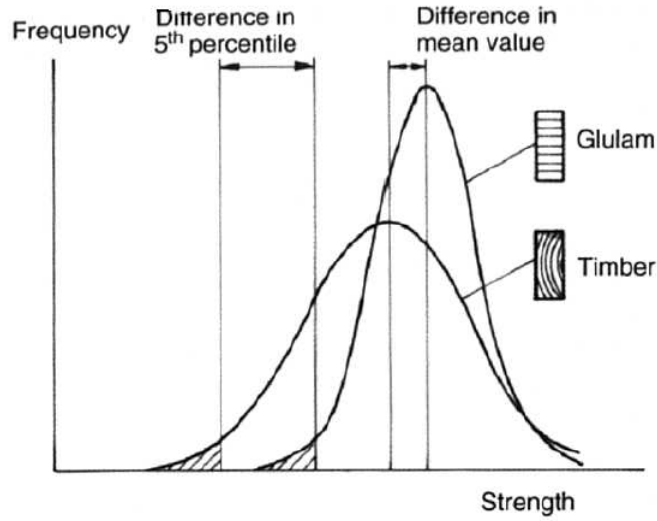


Figure 2.11: Probability density comparison between glulam and solid timber (Thelandersson and Larsen, 2003)

Table 2.3: Strength comparison between different materials with respect to their density

Material	Density (kg/m ³)	Strength (MPa)	Specific Density (m ³ /kg)
Structural steel	7800	400 - 1000	50 - 130
Aluminium	2700	100 - 300	40 - 110
Concrete, Comp.	2300	30 - 120	13 - 50
Timber, Tension	400-600	15 - 40	30 - 80
Glass fiber			500
Carbon fiber			1000

(Thelandersson and Larsen, 2003)

is affected less and the variability of the product becomes smaller". The reduction in variability is shown in figure 2.11.

Comparatively, concrete may have a higher compressive strength than timber; however, timber has a higher bending strength than unreinforced concrete. Table 2.3 compares the strength and density of different materials used in member design and the reinforcing of members.

2.6 Mechanical Properties of Glulam

It is important to fully understand the mechanical properties of timber, as these will be used as input variables for the finite element model (FEM). Timber, as an anisotropic material, has varying mechanical properties in each direction. One classification that is used by many researchers, is that timber is orthotropic. An orthotropic material has mechanical properties that are unique along three mutually perpendicular axes (Green et al., 1999; Dinwoodie, 2000; Cai and Ross, 2010). A further assumption that is often used for timber is that it may be considered as a cylindrically orthotropic material (Saracoglu, 2011; Khorasani, 2012). However, other studies have classified timber as transversely isotropic, which

follows similar properties to orthotropic material except it looks at each plane of symmetry individually instead of three orthogonal planes (Davalos et al., 1991; Sandhaas, 2012; Gharib et al., 2017).

The three axis, shown in figure 2.12, are: longitudinal (L), radial (R) and tangential (T). The longitudinal axis runs parallel to the grain of the timber, the radial axis is perpendicular to the grain in the direction of radial growth, and the tangential axis is perpendicular to the grain but tangent to the growth rings (Harte, 2009; Mirianon et al., 2008; Green et al., 1999; Dinwoodie, 2000).

These assumptions result in a timber element having a total of twelve different mechanical property values - three principle material directions, each with four material values (Khorasani, 2012). Figure 2.12 shows the three major axes of a timber beam and their relation to the grain and the tree.

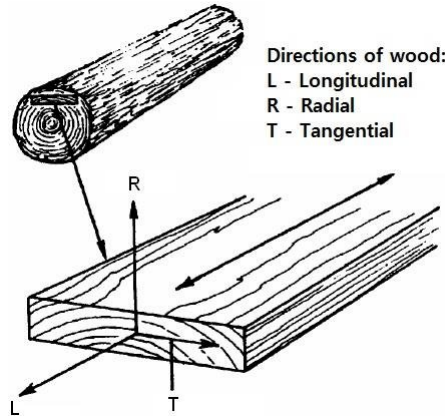


Figure 2.12: Three major axes of a tree
(Alves et al., 2015)

These twelve constants are used in a stress tensor shown in equation 2.1 (Harte, 2009). The use of this matrix is used extensively in continuum damage finite element modelling of orthotropic materials which is discussed in further detail in section 6.4

$$\begin{bmatrix} \varepsilon_L \\ \varepsilon_R \\ \varepsilon_T \\ \gamma_{RT} \\ \gamma_{LT} \\ \gamma_{LR} \end{bmatrix} = \begin{bmatrix} \frac{1}{E_L} & -\frac{\nu_{RL}}{E_R} & -\frac{\nu_{TL}}{E_T} & 0 & 0 & 0 \\ -\frac{\nu_{LR}}{E_L} & \frac{1}{E_R} & -\frac{\nu_{TR}}{E_T} & 0 & 0 & 0 \\ -\frac{\nu_{LT}}{E_L} & -\frac{\nu_{RT}}{E_R} & \frac{1}{E_T} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{RT}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{LT}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{LR}} \end{bmatrix} \begin{bmatrix} \sigma_L \\ \sigma_R \\ \sigma_T \\ \tau_{RT} \\ \tau_{LT} \\ \tau_{LR} \end{bmatrix} \quad (2.1)$$

Where:

G_i is the Modulus of Rigidity with directional subscript,

E is the Modulus of Elasticity with directional subscript,

ν is the Poisson ratio with directional subscript.

Equation 2.1 equates to equation 2.2 as defined by Kuai (2017). Many researchers have simplified the equation in similar, equivalent ways:

$$\bar{\epsilon}_e = \bar{C}\bar{\sigma} \quad (2.2)$$

Where:

$\bar{\epsilon}_e$ is the local elastic strain,

$\bar{\sigma}$ is the local elastic stress,

$\bar{C}$ is the local compliance matrix.

The modulus of elasticity can be related to Poisson's ratio by the expression detailed by Green et al. (1999) in equation 2.3. However, to use this equation requires assumptions, that although make calculations and data collection easier, results in less accurate results and equation 2.18, with explanation in section 2.6.3, should rather be followed.

$$\frac{\nu_{ij}}{E_i} = \frac{\nu_{ji}}{E_j}, \quad i \neq j \quad i, j = L, R, T \quad (2.3)$$

Green et. al (1999) highlights that all materials have variation in their properties, not only anisotropic materials. Concrete for example has different mechanical properties depending on the mix design used. However, for timber, as a natural material it is subjected to constantly changing influences. These influences include, but are not limited to: moisture, soil conditions, and growing space (Green et al., 1999). Even in clear timber pieces (no defects such as knots) high degrees of variability are still prevalent (Harte, 2009; Green et al., 1999).

2.6.1 Modulus of Elasticity

“The Modulus of Elasticity (MoE) is calculated from the slope of the load-against-deflection curve in the elastic (linear) region, the span and the full cross section of the test piece.” (SANS-6083, 2020)

Cheremisinoff and Rosenfeld (2010) state that the modulus of elasticity is the rate of change of unit stress with respect to unit strain under uni-axial loading within the proportional (or elastic) limits of the material. The proportional limit occurs when a material is put under a certain amount of stress (loading) and a corresponding strain (deflection) is experienced. The stress-strain relationship is considered as proportional up until the proportional limit (elastic limit) is reached, after which plastic deformation occurs (Silvester and Kape, 1967). Timber is often more variable than most structural materials, and will generally conform to a standard stress-strain curve as shown in figure 2.13, however variations and inconsistencies may occur at certain stress values (Silvester and Kape, 1967).

The MoE is the single greatest predictor of a timber specimens bending and tensile strength (see section 3.2) (Dowse and Wessels, 2013). For this reason, it may be beneficial to determine a beams dynamic MoE (E_{dyn}), which can be measured in a production factory in order to quickly and accurately grade each timber beam individually. The dynamic MoE can be calculated using a frequency resonator such as the Brookhuis Micro-Electronics Timber Grader MTG (see section 3.2) which takes into account the density of the beam and the frequency at which a sound wave travels through the beam (Barrett

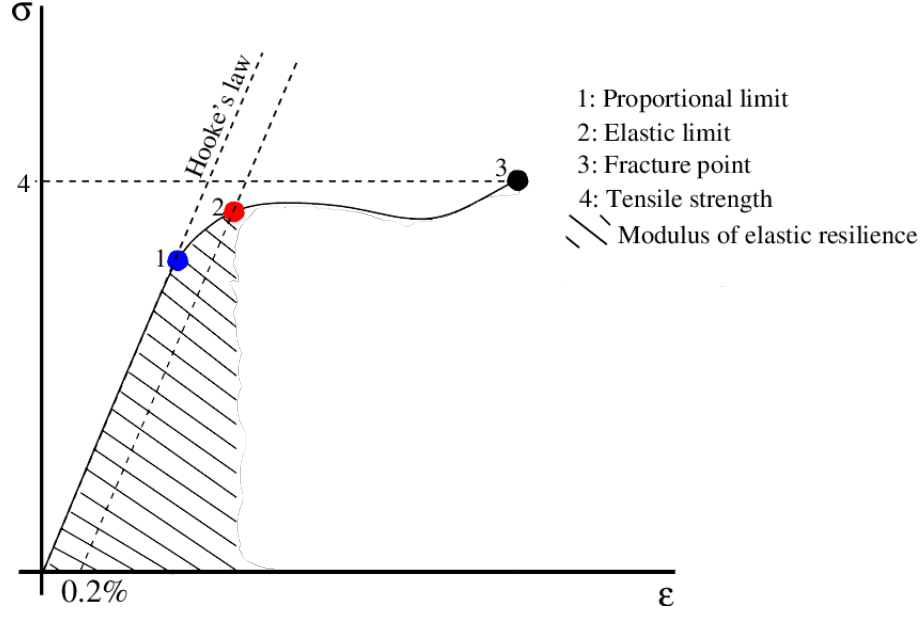


Figure 2.13: Stress-strain curve for a typical material
(González-Viñas and Mancini, 2003)

et al., 2008). The E_{dyn} is calculated using equation 2.9 which is derived by Blaß and Sandhaas (2017) as follows:

$$\sigma = \frac{F}{A} = E \cdot \varepsilon \Rightarrow A \cdot E \cdot \frac{\partial u}{\partial x} \Rightarrow m \cdot \frac{\partial^2 u}{\partial t^2} \quad (2.4)$$

And assuming that the Area (A) and Young's modulus (E) are kept constant:

$$\frac{\partial^2 u}{\partial t^2} = \frac{A \cdot dx \cdot E}{m} \cdot \frac{\partial^2 u}{\partial x^2} \quad (2.5)$$

And $A \cdot dx = \text{volume } (\nu)$, density ($\rho = m/\nu$)

$$\frac{\partial^2 u}{\partial t^2} = \frac{E}{\rho} \cdot \frac{\partial^2 u}{\partial x^2} = c_{long}^2 \cdot \frac{\partial^2 u}{\partial x^2} \quad (2.6)$$

And for long, thin bars:

$$c_{long}^2 = \sqrt{\frac{E}{\rho}} \quad (2.7)$$

Where c_{long}^2 is the sound velocity and the wavelength is denoted as (λ):

$$\lambda = c_{long} \quad \text{where} \quad T = \text{wave period} \quad (2.8)$$

Now considering the bar length (ℓ) and resonance within the beam, and knowing that the frequency (f) $= 1/T$ and by substitution we get the formula for E_{dyn} as:

$$E_{dyn} = 4 \cdot \ell^2 \cdot f^2 \cdot \rho \quad (2.9)$$

These can be compared as well to the mechanical grading tests stipulated in SANS 10149 (2002). The value obtained from this non-destructive prediction test can be compared to the destructive testing methods given by SANS 6083 (2020).

Three point loading:

SANS 6083

Clause: 4.2.a

$$E = \frac{L^3 \cdot \Delta F}{4b \cdot h^3 \cdot \Delta S} \quad (2.10)$$

Four point loading:

SANS 6083

Clause: 4.2.b

$$E = \frac{L^3 \cdot \Delta F}{4,7b \cdot h^3 \cdot \Delta S} \quad (2.11)$$

where:

E is the elastic modulus,

L is the length of the beam,

ΔF is a chosen load increment on the straight line portion of a load vs. deflection curve,

ΔS The deflection value at midspan, corresponding with the ΔF value

For beams that are subjected to point loading and not pure bending test setups, and are not considered as slender beams ($L/b > 12$) the effects of shear are considered significant (Kliger and Pellicane, 1996). Kliger et. al then go on to derive the deflection due to shear as:

For three point loading:

$$\delta_s = \int_0^{\frac{L}{2}} \frac{V\beta}{GA} dx = \frac{0,3PL}{GA} \quad (2.12)$$

and for four point loading:

$$\delta_s = \int_0^{\frac{L}{3}} \frac{V\beta}{GA} dx = \frac{0,2PL}{GA} \quad (2.13)$$

where:

δ_s is the deflection due to shear,

β is 1.2 and is rectangular cross section geometric factor,

G is the modulus of rigidity,

A is the cross sectional area,

V is the shear force,

L is the beam length,

P is the force exerted on the beam.

Kliger and Pellicane (1996) then states the formula for the total combined deflection to be calculated as:

For three point loading:

$$\delta_t = \frac{PL^3}{48EI} + \frac{0.3PL}{GA} \quad (2.14)$$

and for four point loading:

$$\delta_t = \frac{23PL^3}{1296EI} + \frac{0.2PL}{GA} \quad (2.15)$$

Where:

δ_t is the total deflection at the given loading.

Equation 2.14 and 2.15 can be used to calculate the theoretical MoE (E_{th}) by substituting δ_s into equations 2.10 and 2.11 respectively for ΔS . This can then be compared to the actual MoE (E_{ac}) obtained using 2.10 and 2.11 and the values obtained for deflection during testing.

Due to the many species of trees and lack of testing, often the MoE values in all directions is unknown for any given species. In instances where a species' data is incomplete for MoE it is safe to assume the following in order to predict the directional MoE; the MoE in the longitudinal direction (L) is roughly 20 times larger than the MoE in the radial direction (R), which in turn is two times more than that of the MoE in the tangential direction (T) (Sandhaas, 2012; Neuhaus, 1994)

2.6.2 Modulus of Rigidity

The modulus of rigidity (G), better known as the shear modulus, of a material expresses the resistance it has to deflections caused by external shear forces (Green et al., 1999). The three values for the rigidity moduli are denoted as G_{LR} , G_{LT} , and G_{RT} .

Mirianon et al. (2008) describe the way in which these are related to their relative planes. G_{RT} expresses the shear modulus due to shear strain in the RT-plane, and shear stresses in the LR and LT-planes.

The modulus of rigidity is calculated using equation 2.16 and is a relationship between the elasticity of a material and its Poisson's ratio (see section 2.6.3).

$$G = \frac{E}{2(1 + \nu)} \quad (2.16)$$

Where:

G is the Modulus of Rigidity,

E is the Modulus of Elasticity,

ν is the Poisson ratio.

2.6.3 Poisson's Ratio

Poisson's ratio is a dimensionless value that represents the deformation of an object in a particular direction when a load is applied perpendicular to that direction (Greaves et al., 2011). Without the Poisson ratio of a material, accurate mathematical models cannot be generated as the stress states are not calculated properly (Sandhaas, 2012).

The ratios are denoted by ν and the respective planes in which the stress and strain are being measured. The denotations are as follows: ν_{LR} , ν_{RL} , ν_{LT} , ν_{TL} , ν_{RT} , and ν_{TR} (Green et al., 1999; Greaves et al., 2011), with the first of the two directional indices representing the direction of the applied stress and the second representing the perpendicular direction in which the deformation is measured.

The ratio is calculated by applying an axial force in one direction, which causes an elongation in that direction and a contraction in the other as shown in figure 2.14. The resulting strain components are related in equation 2.17 to give Poisson's ratio (Skatulla, 2017).

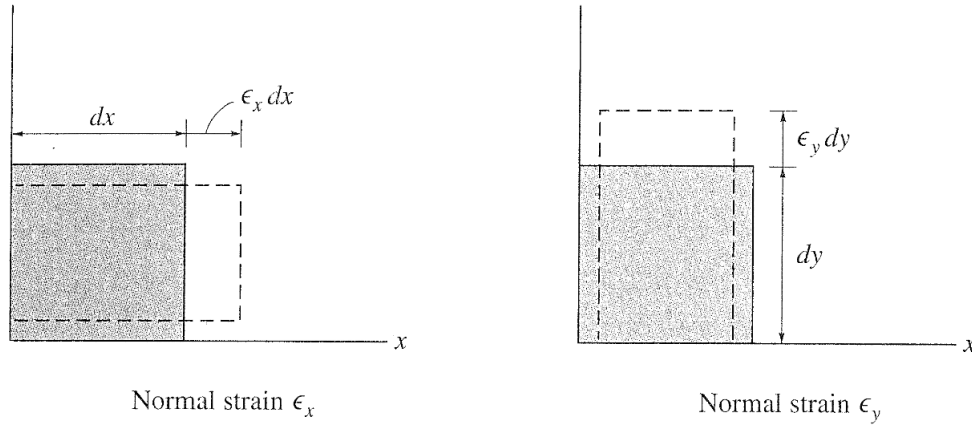


Figure 2.14: Elongation and Contraction under axial loading (Skatulla, 2017)

$$\nu = \frac{\epsilon_{transverse}}{\epsilon_{axial}} \quad (2.17)$$

Like all properties of timber, the Poisson ratio of a tree depends heavily on the moisture content and specific gravity, and can vary dramatically between species and even within the same tree depending on growth conditions. Therefore it is important for research that is to be conducted, to generate Poisson ratio's specific to the tree that is being used in the testing, or use an averaged value found from literature (Green et al., 1999), which is often not fully documented for all directions (Greaves et al., 2011).

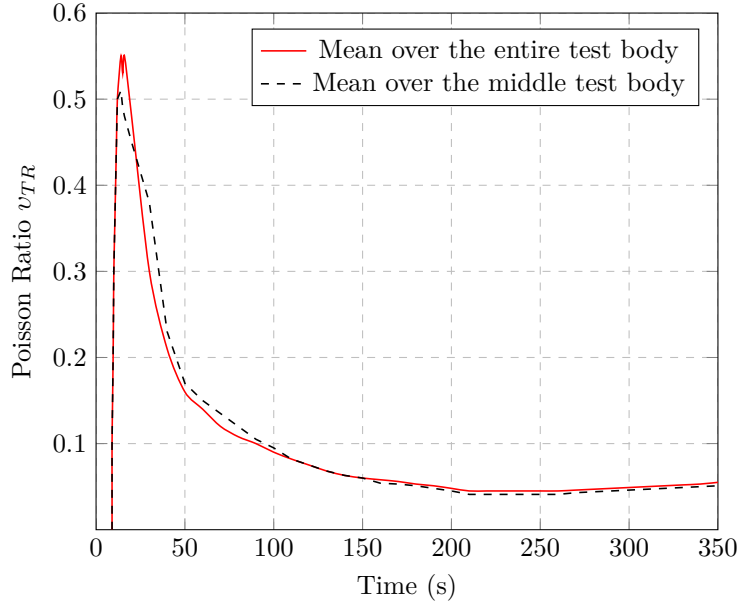
Greaves et al. (2011) then goes on to investigate the symmetry of Poisson ratios in timber, due to the ratios being symmetrical in isotropic materials in regards to equation 2.18. Multiple researchers (Neuhaus, 1983; Garab et al., 2010; McBurney et al., 1962) found that timber does not follow this symmetry and all six Poisson ratios for timber should be used independently when they are available in the literature or should be determined when within the scope of a research paper.

$$\frac{\nu_{TR}}{E_T} \neq \frac{\nu_{RT}}{E_R} \quad (2.18)$$

It should also be noted, that although not in the scope of this research, studies have been conducted on

the degradation of the Poisson ratio in timber after a load has been applied (Franke, 2008; Kashtalyan and Soutis, 2000). Figure 2.1 shows the degradation of the Poisson ratio in timber when stressed perpendicular to the grain (ν_{TR}). The research shows a significant change in Poisson ratio of timber as more force is applied. The implication of not including a changing Poisson ratio in the computational model is that damage propagation and elongation of the material as time passes may be inaccurate.

Graph 2.1: Degradation of Poisson ratio over time when a force is applied adapted from (Franke, 2008)¹



2.6.4 Modulus of Rupture

Modulus of rupture (MoR) is a measure of the maximum load-carrying capacity or strength of a structural member and is defined as the stress at which a material breaks or ruptures. It assumes that a material is considered elastic until the point at which failure occurs (Cheremisinoff and Rosenfeld, 2010)

“The modulus of rupture is determined by the bending moment at the maximum load and the full cross section of the test piece.” (SANS-6083, 2020)

The MoR can be calculated by taking readings at one of two points along the beams length, depending on the test setup. The two test setups recommended in all standards is either a centre point load test (3 point loading), or a third point load test (4 point loading). However, the preferred method of testing is the four point loading test (see section 9). Equations 2.19 and 2.20 show the calculations for the modulus of rupture for their respective test procedures. These equations are for an ultimate stress loading test, and not for the allowable stresses.

Three point loading:

SANS 6083

Clause: 4.1.a

$$\sigma = \frac{3FL}{2bh^2} \quad (2.19)$$

¹Graph translated using Google Translate

Four point loading:

SANS 6083

Clause: 4.1.b

$$\sigma = \frac{F.L}{b.h^2} \quad (2.20)$$

The MoR is a widely accepted strength classification for all materials, although it is not valid for the plastic behaviour, but rather to the elastic limit only (Green et al., 1999).

2.7 Modes of Failure

As previously stated, timber is an orthotropic material with fibrous properties, it is sometimes defined as a transversely isotropic material depending on the expected outcome or modelling procedure adopted. The mechanical properties differ with respect to grain direction and the type of loading. In tension the material behaviour under loading can be described as following linear elastic properties followed by a brittle softening effect, this effect is experienced in both perpendicular and parallel loading directions ($t_{\perp}$ and $t_{\parallel}$, respectively). For compression loading there is a significant difference between perpendicular and parallel to-the-grain loading ($c_{\perp}$ and $c_{\parallel}$, respectively). For $c_{\perp}$ there is a hardening effect due to a possibility of wood densification; whereas $c_{\parallel}$ is characterised by non-linear actions. In $c_{\parallel}$ the plasticity of the material experiences a small hardening effect followed by a region of ductility, once the ultimate stress has been reached, the material undergoes a softening effect. Attempts at modelling the failure of timber using a mathematical procedure should properly differentiate the types of failure with respect to grain direction such as given later in section 7.2.2.

2.8 Finger Joints

Finger joints are used in engineered timber products to produce lengths that are otherwise not possible with natural sawn timber (see figure 2.15). Using adhesive and the increased surface area of the beam edge, two pieces of timber are pressed together using an axial load until the adhesive has cured. This creates a fairly strong bond between two pieces of timber but still causes a discontinuity in the grain and can create weaknesses along the length of the member. For this reason, it is important to know the impact of finger joints on a beams structural stability, especially when trying to create an accurate computer model (Brunetti et al., 2019). Finger joint positions in the outer lamination layers, and the mechanical properties of the outer laminations (especially in the tension zone), are a very big factor when trying to model a glulam beam accurately (Colling, 1990). Failure often occurs at points of discontinuity, such as knots and finger joints (Serrano, 2003).

When a finger joint does not appear in outer layer of laminations at points of high stress, then typically the beam would fail as normal. Therefore many tests have been conducted to determine the amount of beams affected by the finger joints in zone of max bending. These tests were conducted on beams from a standard production line with no prejudice on beam selection. Colling (1990) conducted bending on 1767 beam sections, of which 277 had finger joints present in the outer laminations in the zone of max bending (15.7%). Of these 277 beams, 220 of them (79%) failed at the finger joints, resulting in a 12.5% chance



Figure 2.15: Typical finger joint used in glulam production, before adhesive application

of failure at a finger joint from a randomly selected set of glulam beams. Other tests were conducted (Källsner et al., 1997; Källsner and Ditlevsen, 1994), one with 197 beams that all had finger joints in the required region. Of these beams 42% of them failed at the finger joints. A further study was conducted in which 32 beams with finger joints in the required region (Johansson, 1990) and 10 of these beam (31%) failed at the finger joints. These tests have been summarised and a reasonable estimation on the amount of times a beam will fail at a finger joint, should it fall in the zone of max bending in the outer lamination, is given as 40% - 60% (Serrano and Larsen, 1999).

SANS 1460 (2015) states that no other type of joinery may be used in structural glulam timber, such as a planed sloping scarf joint. A scarf joint retains 85% - 90% of the clear timber strength, however are more wasteful than the finger joints (Lam and Prion, 2003). Therefore since finger joints are not as strong, but are required, it is necessary to determine the strongest type of finger joint for structural purposes. Finger joints are to be machined in accordance to SANS:10096 (2013).

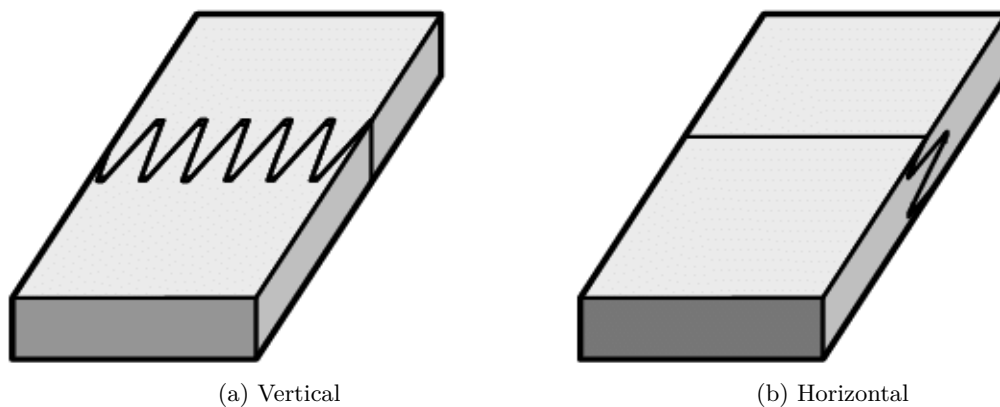


Figure 2.16: Finger joint orientation in glulam beams
(Brandner, 2013)

Finger Joint orientation, (figure 2.16) has been the speculation of many research projects. For the pure tensile strength test there was no difference in strength between the two orientations (Pellerin,

1970). For a bending test it is well documented that the vertical orientation has a slight increase in bending strength over the horizontal orientation (Dupont, 1964; Dawe, 1965; Strickler, 1973; THADA, 1977; Pellerin, 1978). More recently new studies have been conducted that confirm these findings even with new adhesives used (Yeh and Lin, 2012; Zulfahmi et al., 2014; Ahmad et al., 2017). The reason for the increase in strength is provided by Jokerst (1981) but is outside the scope of this research. A finger joint orientation of 45° was developed in Sweden (Jokerst, 1981) and although these have been shown to be significantly better they are not widely used.

Studies where conducted to determine the effect of the finger length on the bending strength of the glulam beam, when the joint appears in the area of max bending (Yeh and Lin, 2012; Mantanis et al., 2010; Ahmad et al., 2017). These studies found that the finger length is a significant factor for the bending strength, a longer finger results in a higher strength in both orientations. Manufacturers would preferably use a shorter finger as it results in less waste. SANS 10096 (or equivalent, IS EN 385) must be used to determine a suitable finger length depending on the application of the beam. Figure 2.17 shows the dimensions necessary for finger joint production and typical values can be found within the SANS 10096 standard. Table 2.4 (adapted from SANS 10096) shows the weakening ratio of having shorter finger lengths

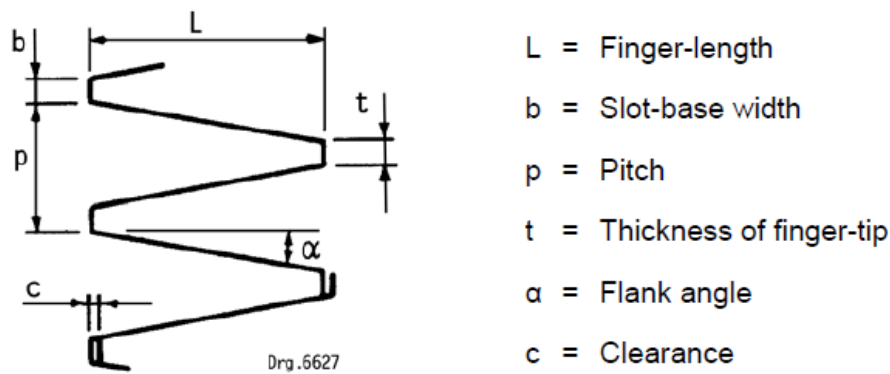


Figure 2.17: SANS 10096 dimensions of a finger joint

Table 2.4: Weakening ratio for finger joint length

Finger Length, L (mm)	Weakening ratio, $v = b/p$
7.5	0.08
7.5 - 35	0.16
35 +	0.20

2.9 Timber-Steel Adhesive Bond Strength

Although the interaction between the adhesive, steel, and timber is not the direct aim of the research it is important to ensure that the steel bonds adequately to the timber. For this reason the literature looked at was based on steel-timber connections as these have in depth prediction models and have tested many different types of epoxies consisting of different chemical compositions. Studies have been conducted on

many types of timber species, and very limited information was available Pinus Patula timber. For this reason it will be important to conduct small-scale pull-out strength tests to determine a suitable adhesive for the specific species being bonded to steel. A suitable bond will allow the connection to be modelled as a perfect bond (Brunetti et al., 2019). All adhesives used must comply with SANS:10183-2 (2019) for the required exposure class.

It is unclear as to whether the density of timber contributes to the bond strength. Serrano (2001); Kemmsies (1999); Bengtsson and Johansson (2002); Bernasconi (2001) have found there is no correlation between density and bond strength, while Gerold (1993); Aicher et al. (1999); Widmann et al. (2007); Steiger et al. (2007) say that density has a direct correlation with bond strength. Many design codes take the density to be a contributing factor but only partially and use it in a power function in the form ρ^c with $0 < c < 1,5$ (Riberholt, 1988; Steer, 2001; Gustafsson et al., 2001; Kangas, 2000). This means that the density of Pinus Patula cannot be used to determine a general bond strength without conducting experimental tests.

Before conducting experiments, or any gluing procedures, the steel must be cleaned according to the adhesive manufacturers specifications (Eickner and Blomquist, 1964), in lieu of this information Yeboah et al. (2011) used methanol and De Luca and Marano (2012) used a non-ionic surfactant and detergent.

2.9.1 Adhesive Type

The typical wood-wood adhesives (casein, urea resin, phenol resin, and resorcinol) do not have the same bond strength when bonding wood-metal (Eickner and Blomquist, 1964). The reason for this is that the two materials have different mechanical properties and the usual wood-wood adhesives are not elastic enough to compensate for the difference, and in general do not bond well to the steel (Eickner and Blomquist, 1964). To increase the elastic properties of the adhesive additives vinyl resins, synthetic and natural rubbers have been combined with phenol-formaldehyde to produce the new epoxy resins.

Although these adhesives have been deemed ineffective for wood-steel bonding by Wheeler and Hutchinson (1998) and Eickner and Blomquist (1964), many researchers are testing this theory as using two different gluing processes in the glulam manufacturing operation results in additional steps and costs that could potentially be unnecessary. For that reason the epoxies tested in the literature are as follows: Epoxy resins (EPO), Polyurethane (PUR), Phenol-Resorcinol-Formaldehyde (PRF), and Melamine-Urea-Formaldehyde (MUF). The consensus on the best bond strength is very well supported as nearly all studies find similar strength deviations between the different adhesives. Table 2.5 shows the experimental comparison of these adhesives (Sardiko et al., 2017; Yeboah et al., 2011; Serrano, 2001). The adhesive type used by South African glulam manufacturers is most commonly an MUF or PRF adhesive.

Table 2.5: Adhesive bond strength compared to epoxy resins

Adhesive Type	Bond Strength (%)
EPO	100
PUR	~ 65
PRF	60 - 65
MUF	55 - 60

Sardiko et al. (2017) and De Luca and Marano (2012) both state that MUF, the weakest of the adhesives, is strong enough to allow all gluing processes to be conducted with a single glue type. Therefore,

if during the minor pull-out procedures a value of 60% of the epoxy resin failure load is reached, using the MUF or PRF adhesives, then a single step gluing process will be sufficient for this research. If this condition is not satisfied a two step adhesive procedure should be used and EPO resin would be used for the timber-steel connection, and MUF/PRF adhesive will be used for timber-timber connections.

2.9.2 Groove Type

Two types of grooving methods were investigated to determine which was the most efficient, the results are shown in table 2.6 and is adapted from Sardiko et al. (2017). Figure 2.18 shows the layout of the grooving methods. Along with the groove types the other factor is the groove diameter compared to the bar diameter (bond thickness). A bond thickness between 1.5mm - 2mm has been shown to produce the best results (Harvey and Ansell, 2003; Yeboah et al., 2011; Za'ba et al., 2012)

Table 2.6: Single groove vs. Double groove

Adhesive Type	Single Groove (kN)	Double Groove (kN)	Strength Increase (%)
EPO	1865	4886	162
PUR	3294	5285	60.4
MUF	7975	8252	3.5

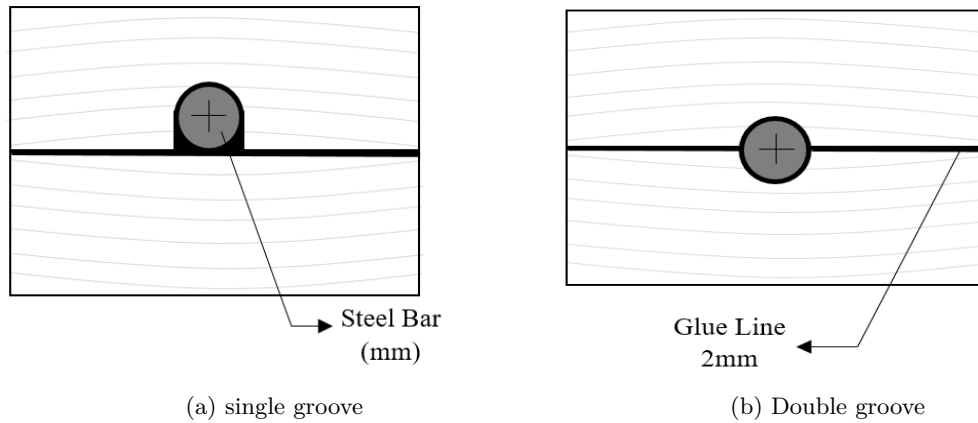


Figure 2.18: Groove types for bonded in steel rods

From these results, it is clear to see that a double groove joint in glulam timber provides a significantly higher tensile pull-out strength. This is due to the stresses being more evenly distributed in the beam and the adhesive (Sardiko et al., 2017).

2.9.3 Steel Bar Variables

Threaded vs. Smooth

Threaded bars are generally considered to provide the best bond, followed by deformed bars, both of which have been found to perform significantly better than the smooth steel bar counterpart (Yeboah et al., 2011). Deng (1997) suggests that when using the bar type (K_b) as a variable in calculations values

of $K_b = 1$ and $K_b = 0.79$ should be used for threaded and deformed bars respectively and Thustochowicz et al. (2011) giving very similar values of $K_b = 1$ and $K_b = 0.8$.

Bonded Length

Bonded length refers to the length of steel bar that has been embedded in the wooden block with epoxy applied. This is regarded as one of the most significant factors, providing almost linear correlation between the bonded length and strength (Broughton and Hutchinson, 2001; Yeboah et al., 2011). This is backed by many design models which take the bonded length into account in their prediction calculations (Riberholt, 1988; Steer, 2001; Steiger et al., 2007; Feligioni et al., 2003). Buchanan and Moss (1999) found that once a value of $\ell > 20 \cdot \phi_{bar}$ was reached there were diminishing returns on increasing strength.

Bar Diameter

The pull-out strength is a subject of debate, particularly regarding the effects of bar diameter. While Yeboah et al. (2011) found that larger bar diameters significantly increase pull-out strength, they have no impact on the developed shear stress in the region. However, they also mention that larger bar diameters increase the likelihood of failure within the timber, while smaller diameters tend to fail at the adhesive/timber interface. Conversely, Za'ba et al. (2012) argues that larger bar diameters lead to failure at the rod/adhesive interface, while smaller diameters result in failures at the adhesive/timber interface. Broughton and Hutchinson (2001) contradicts both studies, stating that bar diameter has no effect on strength. This variation in findings underscores the complexity and variability of factors affecting pull-out strength and highlights the need for further experimental tests pertaining to this research.

2.9.4 Failure Modes

A successful bonding is classified by a failure in the wood itself, and not the adhesive. Therefore the timber must fail in shear according to figure 2.19(3). To increase the bond strength of the adhesive to the timber, which will encourage failure in the timber, the adhesive should be applied to the wood when a moisture content $<20\%$ is reached (Broughton and Hutchinson, 2001). Deng (1997) suggests similar importance on the moisture content however states a value of $<22\%$ should not be exceeded.

Once failure is reached the load can be recorded and the shear stresses can be calculated using equations 2.21 and 2.22.

$$\tau_1 = \frac{P}{\pi \cdot \phi_{hole} \cdot \ell} \quad (2.21)$$

$$\tau_2 = \frac{P}{\pi \cdot \phi_{bar} \cdot \ell} \quad (2.22)$$

Where:

- P is the failure load (N),
- ϕ_{hole} is the diameter of the grooved hole (mm),
- ϕ_{bar} is the diameter of the steel bar (mm),
- ℓ is the bonded length of steel (mm),

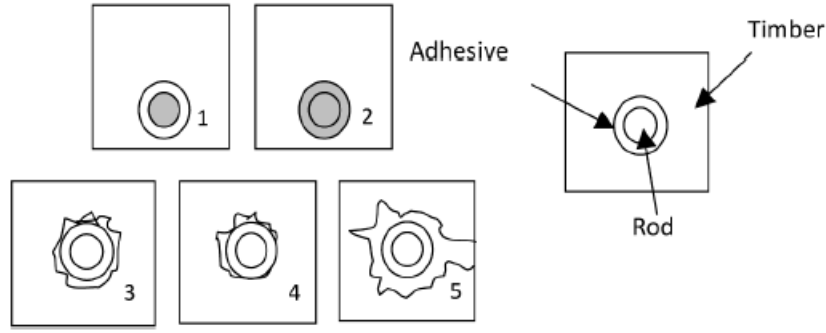


Figure 2.19: Failure methods of the pull-out tests
(Za'ba et al., 2012)

(1): failure in rod/adhesive interface, (2): failure at the adhesive/timber interface, (3): is a failure in the timber away from the adhesive/timber interface, (4): is a failure in the timber close to the adhesive/timber interface, (5): is similar to that of (3)

τ_1 is the stress at the adhesive/timber interface (MPa),

τ_2 is the stress at the adhesive/rod interface (MPa).

For the modelling of the of the timber-steel interface, gluing in the rods allows a selection of virtually infinite friction coefficients between the two surfaces, as long as the edge distances are in accordance with the design standards in table 2.7 (Johansson et al., 1995).

Table 2.7: Minimum edge distance for embedded steel

	Minimum Distances
Rods glued in parallel to the grain	
Spacing	$a_2 = 5 \cdot d$
Edge distance	$a_{2,c} = 2.5 \cdot d$
Rods glued in perpendicular to the grain	
Spacing (along beam width)	$a_1 = 4 \cdot d$
Edge distance (along beam width)	$a_{1,c} = 2.5 \cdot d$
Spacing (along beam length)	$a_2 = 4 \cdot d$
Edge distance (along beam length)	$a_{2,c} = 2.5 \cdot d$

Chapter 3

Testing Literature Review

3.1 Introduction

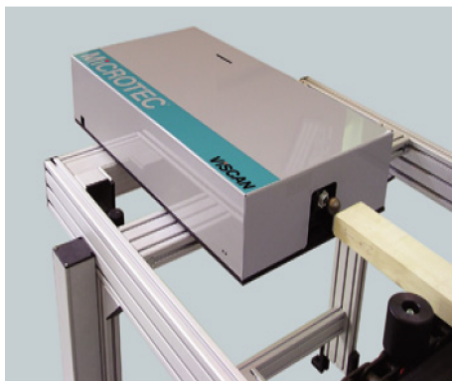
Timber testing shares both commonalities and distinctions with assessments applied to other materials. It aligns with standard test procedures on specimens, but diverges in terms of researchers' objectives. Precisely determining unit values for timber's mechanical properties presents a challenge due to the pronounced variability in results, even with consistent use of identical wood planks and testing equipment. Consequently, research has focused on establishing average values for timber species, along with quantifying the range of variation above and below these averages, and the governing principles of this variability (Record, 1914).

Generally, testing can be divided into two types of testing; destructive and non-destructive testing, both of which will be explored briefly in this chapter. Non-destructive testing is generally conducted on beams in the warehouse before selling to clients and is used for grading timber, in some cases these can be done on a large scale for a relatively low cost. Destructive tests are done on a small representative sample size as the beams are generally discarded after testing and are costly.

3.2 Non-Destructive Testing

The main non-destructive test (NDT) machines available in South Africa are the MiCROTEC Stress Grader and the Brookhuis Timber Grader MTG. The MiCROTEC is a fairly large machine, and in South Africa they are all currently in-use in high speed timber grading systems and are not available for public use. The Brookhuis MTG is a handheld device that allows the operator to test a specific beam once off the production line. The two machines are shown in figure 3.1 and both test for the MoE of a beam. Neither of these machines, or any NDT test procedures, can be used to accurately or reliably predict the Poisson ratios of the timber (Bachtiar et al., 2015). To determine the Poisson ratios for use in equation 2.16, destructive mechanical tests need to be used for this (see section 3.3)

A MiCROTEC machine uses a laser to measure the frequency of vibration within the beam. This natural frequency and density (both calculated by the machine) are used to calculate the MoE. An MTG scanner uses a hammer that sends a soundwave through the beam. The wave frequency and the beam density are used to calculate the beams MoE using equation 2.9 which was derived in section 2.6.1.



(a) MiCROTEC machine



(b) Handheld Brookhuis MTG

Figure 3.1: Non-destructive test grading machines in South Africa

The MoE is the single greatest predictor of a specimens bending and tensile strength, achieving the highest coefficient of correlation between bending strength and non destructive indicators (R^2)(Dowse and Wessels, 2013; Yang et al., 2015). Combining the coefficient of different predictors can result in a more accurate result. Dowse and Wessels (2013) compiles many of these predictors from multiple studies with the main ones shown in table 3.1.

Table 3.1: Coefficients of determination (R^2) for MoR by different predictors (Dowse and Wessels, 2013)

Characteristics that can be measured non-destructively	Coefficient of Determination (R^2)
Knots	0.15 - 0.35
Density	0.20 - 0.40
Frequency, ultrasonic speed	0.30 - 0.55
MoE	0.40 - 0.65
Knots and density	0.40 - 0.60
Knots and MoE	0.55 - 0.75
Knots, density and frequency	0.55 - 0.80

A study was conducted by Barrett et al. (2008) in which comparison tests were done to determine the accuracy of the MiCROTEC and the MTG. The study began by using a Metriguard Model E340 computer. This system is known to calculate the MoE accurately. It uses natural frequency, density and beam length to get its measurements. It has a strong correlation with the MoE results obtained from tests conducted according to ASTM:D4761-19 (2019) ($E_{ASTM4761} = 0.991 E_{Metriguard}$). The results obtained from the MiCROTEC machine where found to be 2.2% greater than that of the Metriguard and the results obtained from the MTG were found to be “essentially equivalent” (Barrett et al., 2008).

However the the test procedures in ASTM:D4761-19 (2019) do not account for shear stresses, unlike those in ASTM:D198-15 (2015).

3.3 Destructive Tests

Destructive testing must follow the procedure of testing as laid out by SANS-6083 (2020) and ASTM:D198-15 (2015), with the ASTM standard going into further detail on each test method, including: Flexure, Compression (Short Specimen), Compression (Long Specimen), Tension, Torsion¹, and Shear Modulus. The ASTM:D198-15 (2015) standard is written specifically for scientific studies and the advancement of new design standards. The characteristic stresses obtained during these tests can be converted to allowable stresses for further design standards to be developed. Destructive testing methods used in this study are discussed further in section 9.2.

3.4 Previous Timber Reinforcement

The reinforcing of timber and glulam has seen a significant increase in research in recent years, current methods of reinforcing are; fiber reinforced polymers (FRP’s), steel sheets, cambering and pre-stressing.

¹Torsional testing and any secondary effects of torsional stresses are not in the scope of the research and will be neglected

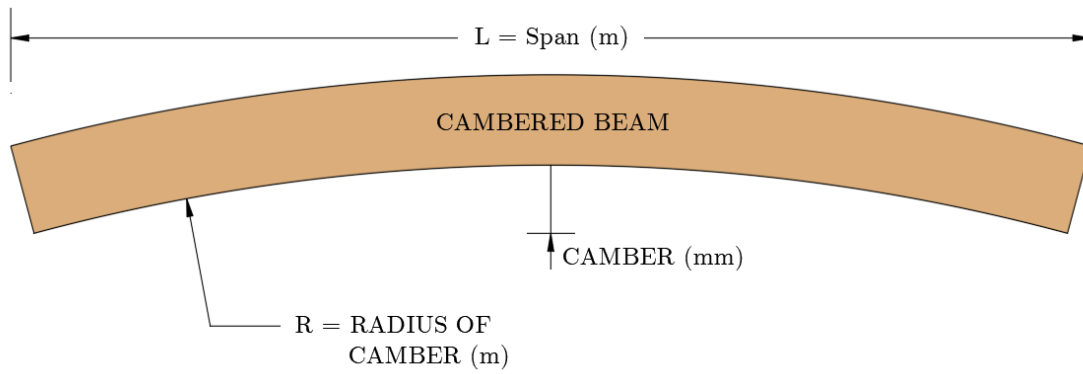


Figure 3.2: cambered glulam beam adapted from (APA, 2020)

3.4.1 Fiber Reinforced Polymers and Steel

When a beam fails in the outer lamination, the load bearing capacity of the beam is usually always lost, and the entire beam fails (Serrano and Larsen, 1999). To increase the failure load many researchers have tried to reinforce the outer layers of beams with FRPs and steel sheeting, often with significantly increased flexural strength and stiffness. The increase in these two factors has been recorded between 39.19% - 64.12% and 16.6% - 50.62% respectively (Yang et al., 2016; Nadir et al., 2016; Johnsson et al., 2007; Brunetti et al., 2019). However, most of the reinforcement used in these tests have either been expensive (CFRP) and not viable for large scale construction, or the reinforcement has been externally applied and the aesthetic appeal of the timber/glulam is lost.

3.4.2 Cambering

Although not common practice, nor a topic that has been researched heavily, for larger spans of glulam that may be subject to large deflections it is possible to specify cambered beams. For these larger spans the deflection is often the limiting design factor and by cambering the beam, as shown in figure 3.3 the aesthetic effect of in service deflections may be significantly reduced (APA, 2020). As shown in figure 3.2 glulam camber is often specified in “inches of camber” or “mm of camber” and is dependant on the span length.



Figure 3.3: cambered glulam beam in the factory

3.5 Previous Tests on Pinus Patula

Although many tests have been conducted on timber, a fairly limited amount of research has been conducted specifically on Pinus Patula. This is due to the fact that Pinus Patula in South Africa is used primarily as a secondary construction material and not as a core element of structures. For this reason, more modern research studies have investigated steel and concrete materials rather than timber.

Although test procedures are the same for timber species, the results from one species are not necessarily true for another species.

3.5.1 Tensile strength Tests

The standard tensile test for South African timber is described in SANS6122 section 5.7, the test diagram is shown in figure 3.4 and important procedural steps are noted as:

1. A standard specimen length of $8w + 2000\text{mm}$ should be used
2. This test is for tension parallel to the grain
3. Apply constant load
4. If loading to failure the test should be completed in 3 to 4 minutes

Burdzik (2004) followed the ISO CD 13910 (2001) standard tensile test shown in figure 3.5. There are no key differences between these two test methods and both use equation 3.1 to determine the tensile test of the material.

$$F_t = \frac{P_f}{tw} \quad (3.1)$$

Where:

F_t is the tensile strength parallel to the grain, MPa

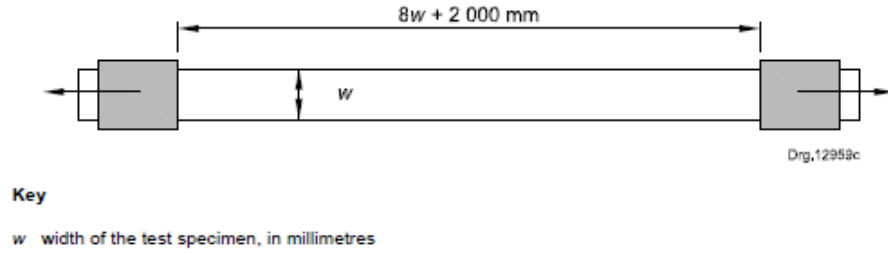


Figure 3.4: SANS 6122 Tensile test

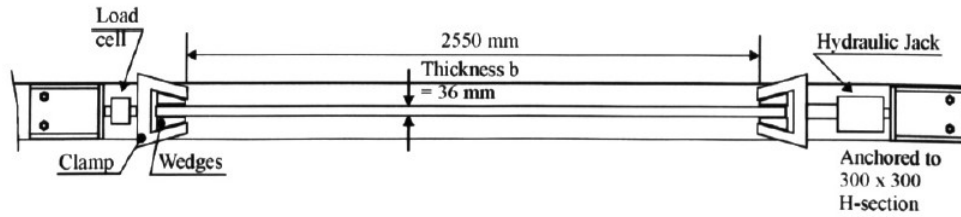


Figure 3.5: Tensile test used by (Burdzik, 2004)

P_f is the failure load, N

t is the timber thickness, mm

w is the timber width, mm

3.5.2 Compressive Strength Tests

Similarly to that of the tensile test, a compression test parallel to the grain is given in SANS 6122 section 5.8. An apparatus and test setup is shown in figure 3.6

1. A standard specimen length of $8t + 2000\text{mm}$ should be used,
2. This test is for compression parallel to the grain
3. Apply constant load
4. If loading to failure the test should be completed in 3 to 4 minutes
5. rollers or other supports are used to prevent any out-of-plane bending or buckling

Once again, Burdzik (2004) and the ISO CD 13910 (2001) standard use a similar test setup to test all species of timber in compression. With this setup the compressive strength parallel to the grain is given in equation 3.2

$$F_c = \frac{P_f}{tw} \quad (3.2)$$

Where:

F_t is the compressive strength parallel to the grain, MPa

P_f is the failure load, N

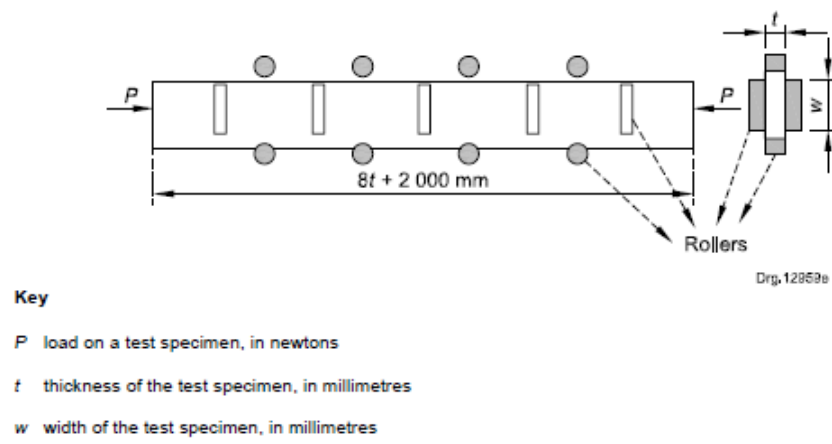


Figure 3.6: SANS 6122 Compression test

t is the timber thickness, mm

w is the timber width, mm

Chapter 4

Steel Cables

4.1 Introduction

Selecting a suitable reinforcing material for the internal reinforcing of timber is a difficult task in itself. Previously, carbon fibre and other FRPs have been found to be effective at either increasing the timber strength or reducing the global deflection, but these were often found to not be viable due to the significant costs involved. Regular reinforcing steel, such as that found in reinforced concrete, has multiple downfalls with regards to its ability to reinforce timber. The steel is susceptible to corrosion which would easily cause de-bonding between any form of glue and the steel. Additionally, small diameter reinforcing steel ($< \varnothing 5\text{mm}$) tends to be easily bent out of shape, and difficult to keep in straight lines, making it almost impossible to get the steel in the grooved timber. Mild steel provides some challenges of its own due to the smooth surface. As noted in section 2.9.3 a notched or rough surface provides better adherence for the glue.

Since the weakest point of the timber in flexure is the tension lamellae in the beam, it is important to use a reinforcing material that is strong in tension, while still being able to be resistant to corrosion, have a notched or rough surface, and for ease of production, it needs to be workable in order for it to be pressed into the grooves.

Three standard sizes of steel wire ropes (3mm, 5mm and 8mm) were selected as a suitable material as they fit the criteria stated. The ropes are often used in reinforcing for pre-tensioned concrete as well as in lifting gear in cranes and hoists.

4.2 Steel-wire Rope

Steel-wire rope is designed to have high tensile strength, while still being flexible and having a zero compressive strength. The tensile strength is often more than 3 times higher than that of normal rebar used in reinforced concrete. Figure 4.1 shows a typical illustration of the wire rope composition. Cables are named according to 'strands $\times$ wires per strand' followed by the core type. Two main core types are steel (SC) and fibre (FC). The fibre core is a polypropylene polymer that runs along the length of the rope. Table 4.1 summarizes the different grades of steel-wire rope that are available.

Table 4.1: Steel wire strength grades

Grade	Tensile strength range (MPa)
1570	1370 to 1770
1770	1570 to 1960
1960	1770 to 2160
2160	1960 to 2160

The breaking force, F_{min} , of a cable can be calculated using equation 4.1 given in SANS2408.

$$F_{min} = \frac{d^2 \times R_r \times K}{1000} \quad (4.1)$$

Where:

d is the nominal diameter of rope (mm)

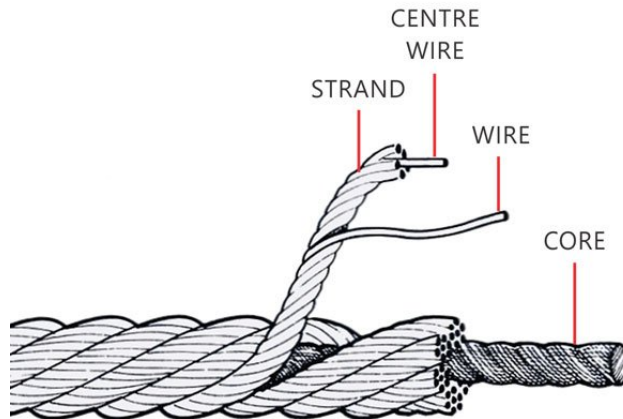


Figure 4.1: Steel rope diagram

R_r is the rope grade

K is the minimum breaking force factor for a given rope class

6×7 FC

The 3mm wire rope is comprised of 6 strands, each consisting of 7 wires, resulting in a total of 42 wires per cable. Each of the wires are coated in zinc for corrosion resistance. A fibre core is used for the 3 mm cable. A typical cross section of a 6×7 FC cable is shown in figure 4.2

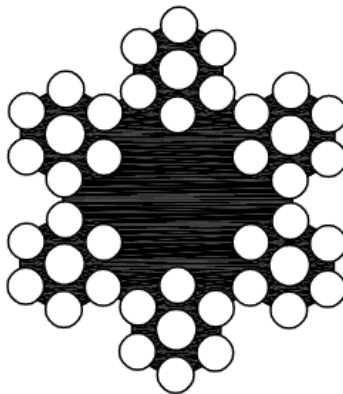


Figure 4.2: 6×7 FC cross section

6×19 FC

The 5mm and 8mm wire ropes are comprised of 6 strands, each consisting of 19 wires, resulting in a total of 114 wires per cable. Each of the wires are coated in zinc for corrosion resistance. A fibre core is used for the both the 5mm and 8mm cables. A typical cross section of a 6×19 FC cable is shown in figure 4.3

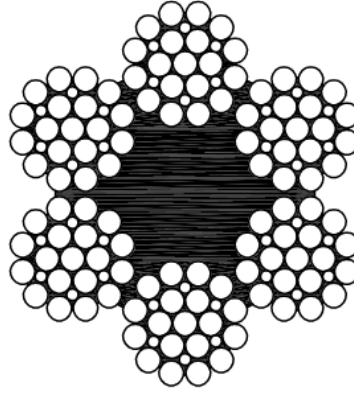


Figure 4.3: 6×19 FC cross section

The breaking force factors, K , for all wire types is given in SANS2408, The K values for the wires used in this research, as well as their respective F_{min} values are provided in table 4.2.

Table 4.2: Minimum breaking force factors

Class	Breaking force factor	Diameter (mm)	F_{min} (kN)
6 x 7 w/ fibre core	0.332	3	5.3
6 x 19 w/ fibre core	0.330	5	14.6
6 x 19 w/ fibre core	0.330	8	37.4

Chapter 5

Digital Image Correlation - DIC

5.1 Introduction

Strain gauge use was initially considered for experimental data capturing for this study, this however, was discarded as it was found to be ineffective at capturing the overall beam behaviour. The strain gauges provide data for a very localised area, and using multiple strain gauges along the beams length is impractical with regards to budget constraints. The use of DIC has been adopted as a suitable alternative.

DIC is an optical non-contact method of measuring full field strains and displacements, and can be used in both 2D and 3D spaces. The technology is widely used in the automotive and aerospace industries and has recently found its way into structural engineering for its ability to accurately track crack and fatigue behaviour (James, 2020).

5.2 Measurement Principles in a DIC system

The DIC process requires the use of one or more high resolution charge coupled device (CCD) monochrome cameras. A 2D system requires only one camera whereas as a 3D system requires a minimum of two cameras and a maximum of eight cameras. The shutter speed of these cameras may be varied in order to capture images at a different rate depending on the tests being conducted.

To track the changes of the material, a stochastic speckle pattern is painted onto the material surface. The speckle, as shown in figure 5.1, is divided into smaller subareas using a grid pattern, called facets. Each facet has a unique speckle pattern which is tracked in successive images and compared to the initial untransformed image taken under no loading (Frankovsky et al., 2013).

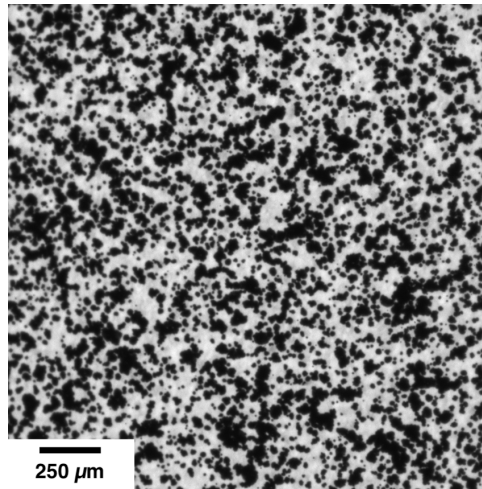


Figure 5.1: Speckle pattern on the surface of a test specimen

The facets are compared by applying a correlation algorithm. The algorithm is based on tracking the grey scale factors of the speckle pattern in the initial position (P, Q) to the deformed state (P_1, Q_1) . The changes are tracked over small linear sections using equations 5.1 and 5.3 and a graphical representation of the process is presented in figure 5.2

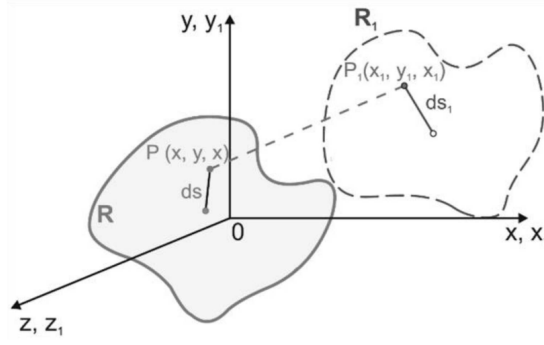


Figure 5.2: Transformation of an area using DIC algorithms
(Domański et al., 2021)

$$P_1 = (x_1, y_1, z_1) \quad (5.1)$$

$$= (x + u(P), y + v(P), z + w(P)) \quad (5.2)$$

$$Q_1 = (x_1 + dx_1, y_1 + dy_1, z_1 + dz_1) \quad (5.3)$$

$$= (x + u(P) + u(Q) - u(P) + dx, y + v(P) + v(Q) - v(P) + dy, z + w(P) + w(Q) - w(P) + dz) \quad (5.4)$$

Where:

u, v, w are the displacements in the respective axis directions x, y, z

The section lengths are determined with equations 5.5 and 5.6

$$|PQ|^2 = (ds)^2 = dx^2 + dy^2 + dz^2 \quad (5.5)$$

$$|P_1Q_1|^2 = (ds_1)^2 = dx_1^2 + dy_1^2 + dz_1^2 \quad (5.6)$$

The final deformation state components are then given as mathematical transformations shown in equations 5.7 - 5.9 (Domański et al., 2021).

$$\varepsilon_{xx} \cong \frac{du}{dx} + \frac{1}{2} \left[\left(\frac{\partial u}{\partial x} \right)^2 + \left(\frac{\partial v}{\partial x} \right)^2 \right] \quad (5.7)$$

$$\varepsilon_{yy} \cong \frac{dv}{dy} + \frac{1}{2} \left[\left(\frac{\partial u}{\partial y} \right)^2 + \left(\frac{\partial v}{\partial y} \right)^2 \right] \quad (5.8)$$

$$\varepsilon_{xy} \cong \frac{1}{2} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) + \frac{1}{2} \left[\frac{\partial u}{\partial x} \frac{\partial u}{\partial y} + \frac{\partial u}{\partial x} \frac{\partial v}{\partial y} \right] \quad (5.9)$$

Using this method of measurement, displacements can be measured to an accuracy of 1/100000 of the measurement field, a 100mm visual field can have displacement readings accurate to the nearest 1μm.

Strains can be calculated with an accuracy of around 0.01 pixels depending on the camera used for data capturing.

The results and the relative accuracies can be achieved across the entire length of the test specimen while using DIC, whereas using a strain gauge can often be less accurate and only at a local area.

5.3 Set-up and Results

The set-up and apparatus used for the DIC used in this research makes use of the analysis software 'Istra4D' provided by Dantec Dynamics, and video capturing software 'Pylon Viewer' from Basler AG.

The correlation system is based off of a Q-400 setup, as shown in figure 5.3. The final setup is shown in figure 5.4 and the following apparatus was be used:

- Camera: Basler Ace 2 Basic - acA2500 - 60 μ m
- Lens: Fujinon CF12.5HA-1
- Wide angle lens: Kowa LM8HC
- Lighting rig
- Calibration board (35mm 9 $\times$ 9 chequerboard pattern)
- Windows 10 PC with relevant software

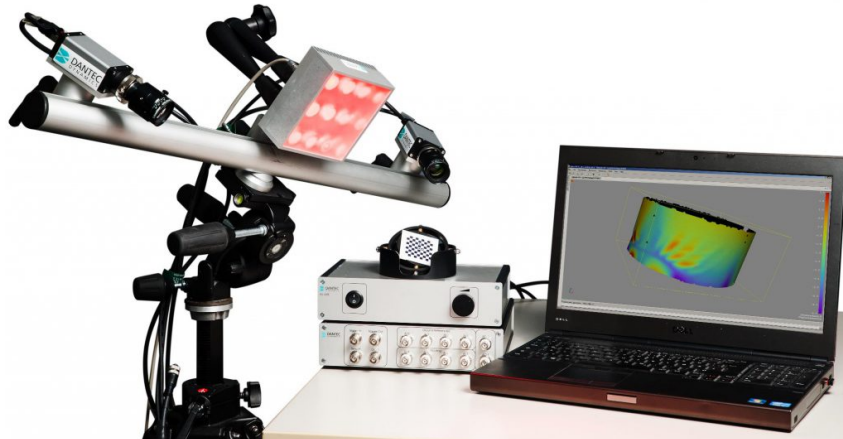


Figure 5.3: Components of a Q400 DIC system

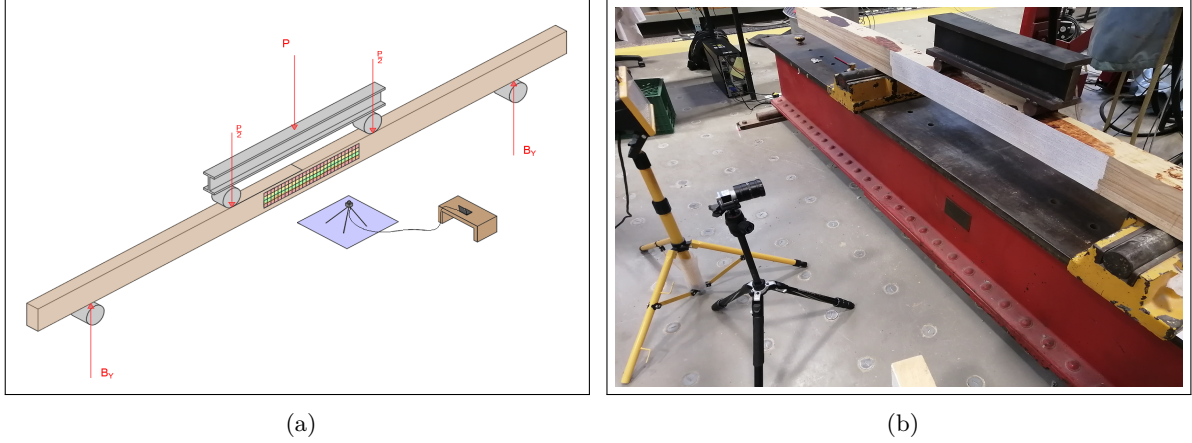


Figure 5.4: Final DIC setup and schematic

The results provided by the post-processing software , Istra4D, can be interpreted numerically, graphically and visually. The visual presentation can be used to easily compare the physical tests to a computational model provided the strain/displacements are adjusted accordingly, this can be seen in figure 5.5 where a dog bone tensile test is compared visually in Abaqus and the DIC software.

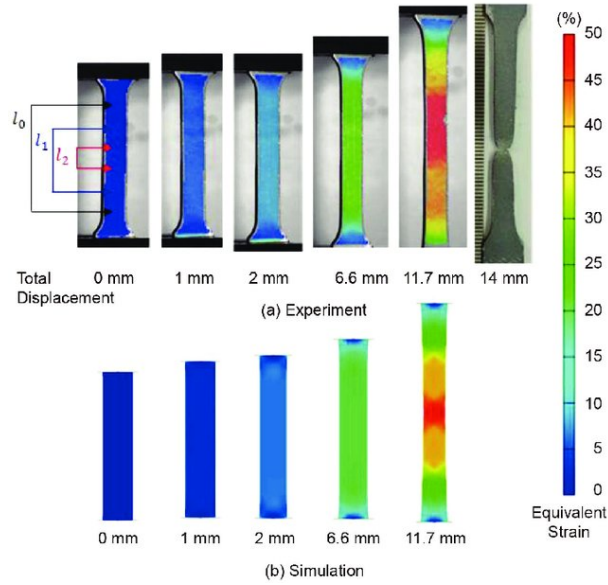


Figure 5.5: Visual comparison of a DIC analysis vs. an Abaqus model (Yoon et al., 2016)

5.4 Navigating the software

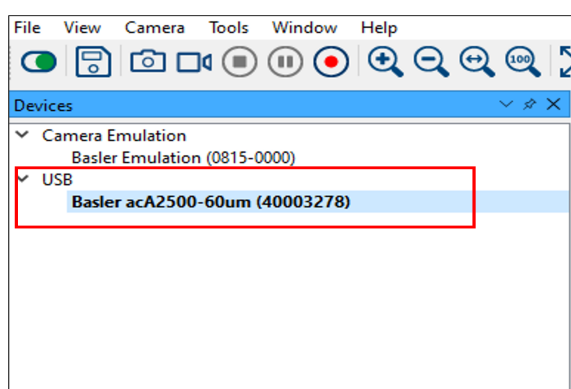
As mentioned previously in section 5.3 two different softwares were used for the DIC analysis. Image acquisition and data analysis had to be done separately as the trigger cable for the camera assembly was unavailable at the time of testing however, under normal circumstances the entire DIC process is possible within Istra4D.

5.4.1 Pylon Viewer

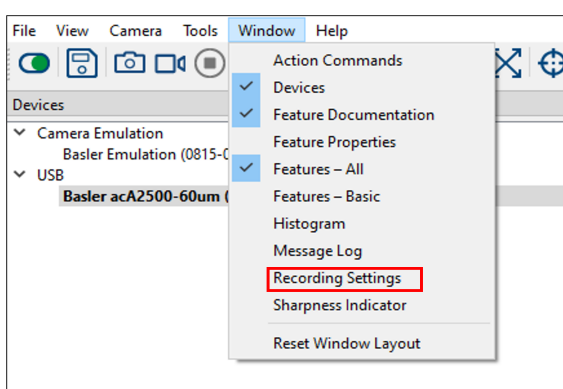
Pylon Viewer is a freely available image acquisition camera software suite provided by Basler AG (available at: <https://www.baslerweb.com/en/sales-support/downloads/software-downloads/\#type=pylonsoftware;language=all;version=all>)

Once installed onto a PC, the camera is connected via the USB 3.0 cable provided and proceed as follows to capture images.

1. **Open** the Pylon Viewer software
2. In the top left of the window that opens, select the camera you have just plugged into the PC (see figure 5.6a)
3. Next select the **Recording settings** from the Windows toolbar as shown in figure 5.6b



(a) Pylon viewer camera selection



(b) How to access the recording settings

Figure 5.6: Pylon viewer navigation menus

4. When using the images in Istra4D they must be in the format of **.tiff or .bmp** (8 and 16-bit)
5. In the pop-up menu select the correct output format as well as the frame rate and output folder (see figure 5.7)

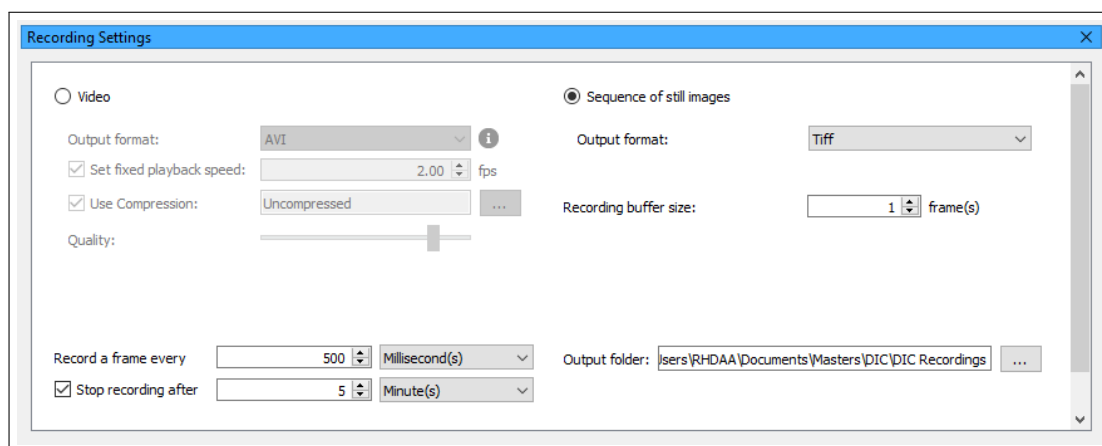


Figure 5.7: Recording settings

6. The next step is to select **camera settings** (figure 5.8). These are located in the bottom left of the Pylon Viewer home screen.
7. These features are **user-specific** and will not be discussed here.

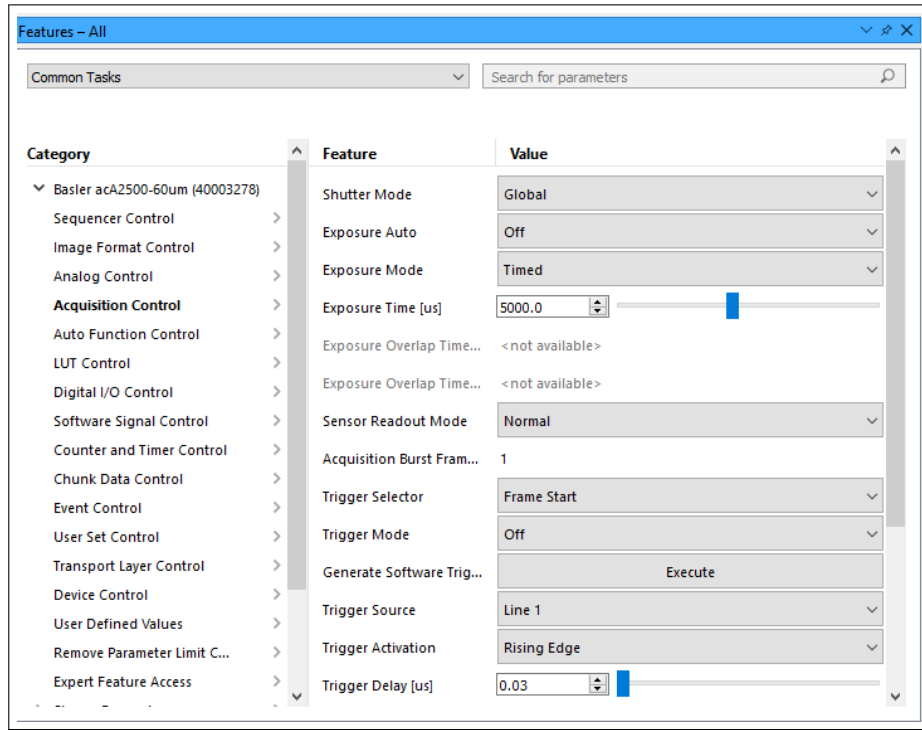


Figure 5.8: Pylon viewer camera settings

8. Once everything has been set-up correctly image acquisition can commence. To do this refer to image 5.9 where:
 - i Shows the live view through the camera lens (use to adjust camera for focus and exposure)
 - ii Takes a snapshot of the current view (take a snapshot after each loading point)
 - iii Saves the snapshot to a specified folder
 - iv Records multiple snapshots at a rate determined in figure 5.7

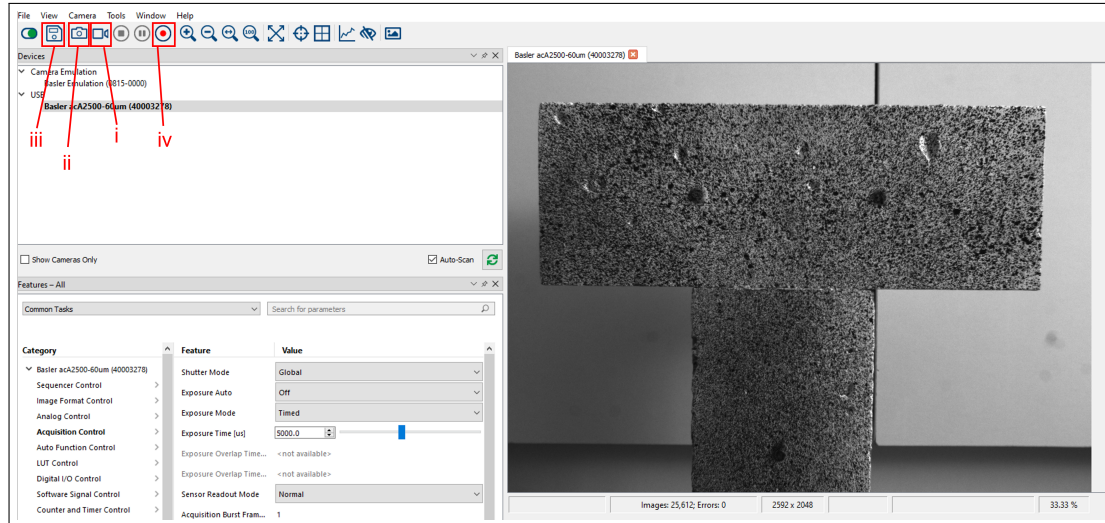
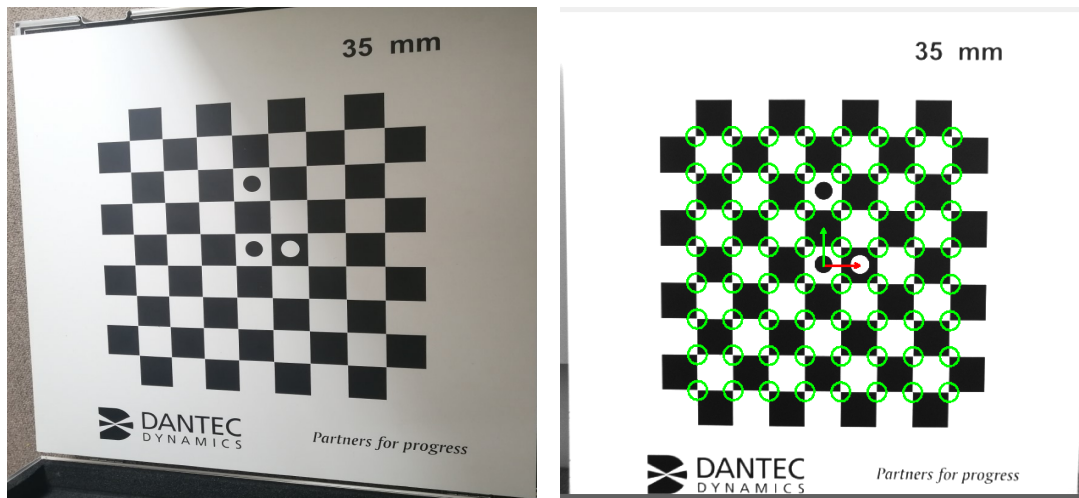


Figure 5.9: how to capture images in Pylon viewer

In Pylon Viewer it is possible to take snapshots of the calibration target as well. The calibration target (seen in figure 5.10) must be in the field of view of the camera and it has to be in focus. Thus it should be positioned at the same distance as the measurement specimen. The target must still be in focus, even if it is tilted in different orientations. Roughly 6 -8 images of the calibration should be taken.



(a) Calibration target

(b) Calibration target in Istra4D

Figure 5.10: 35mm 9x9 black and white calibration target

5.4.2 Istra4D

Istra4D is an image capturing and post-processing DIC software provided by Dantec Dynamics. As stated earlier, Istra4D was used solely for post-processing. The .tiff images taken in Pylon Viewer are then imported and analysed, the process to do this is explained below:

1. Select your desired **repository**
2. To import your data click **File < Import < New Correlation Series**

3. To import your calibration images click **File < Import < New Project Calibration**

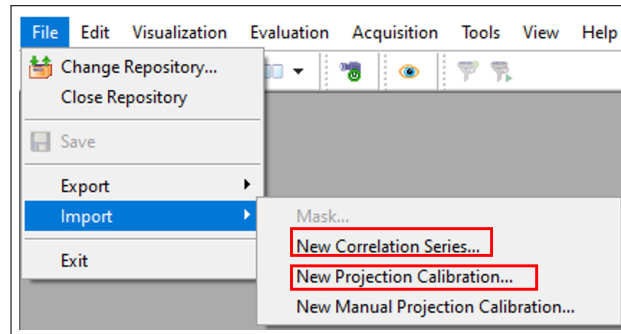


Figure 5.11: Istra4D importing data

4. For both the correlation series and calibration imports, the next step is the same.

5. Select the **number of cameras** and then click **Camera Position**

6. Then select the data from the pop-up window (see figure 5.12)

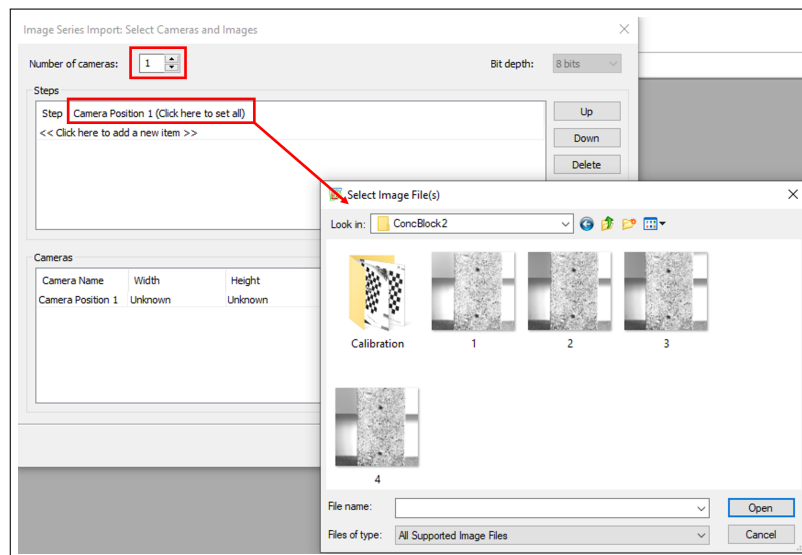


Figure 5.12: Istra4D data selection

7. When selecting the calibration data a parameter selection window will appear (see figure 5.13)

- i Select the correct **calibration target** size
- ii click **Extract markers**
- iii Wait for all the points to get a green circle - if not all points have a green circle the calibration images must be retaken
- iv Click **Connect**
- v Click **Done**

8. To start a **new evaluation** follow the process shown in figure 5.14
9. **Select** the data which you are to evaluate
10. Click the evaluation icon
11. Select **Universal correlation** and click **Next**
12. Add in the calibration data you generated in figure 5.13 and click **Next**
13. Before selecting Finish select the necessary items in the tick boxes - in this case only **Correlation Parameters** needs to be selected

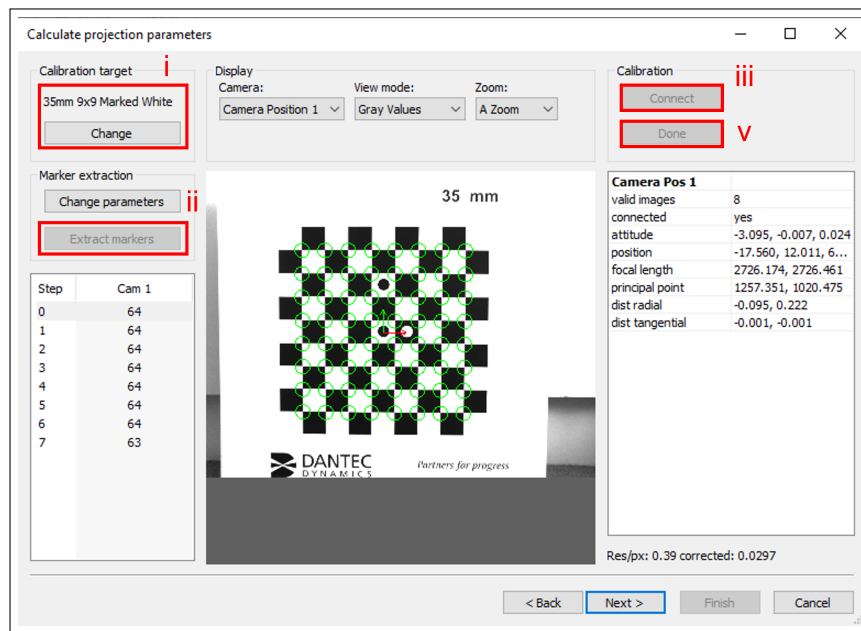


Figure 5.13: Istra4D Calibration settings

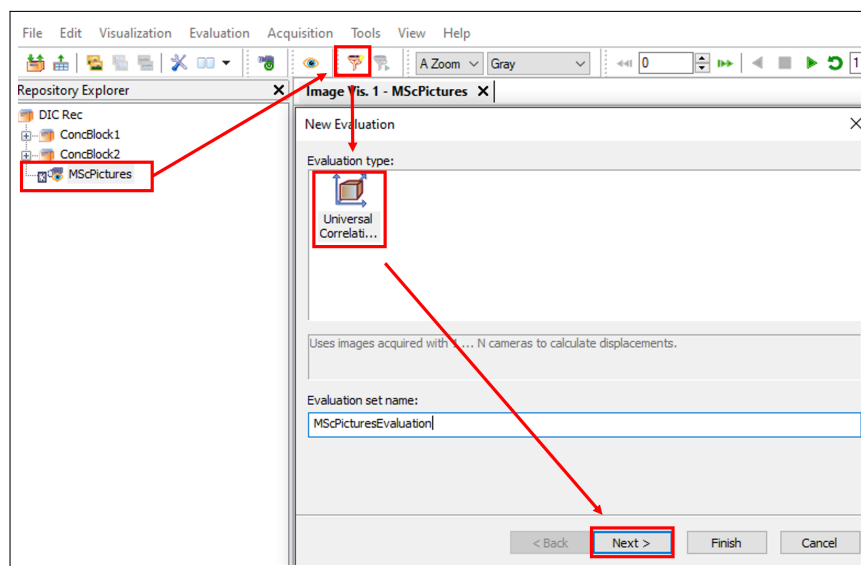


Figure 5.14: Istra4D starting a new calibration

The next steps of the evaluation depend on the accuracy required as well as the area which is to be evaluated. A more in depth approach to the menus and settings can be found in the Istra4D user manual, specifically chapter 7 and 8. The purpose of this chapter is to only show the process of importing Pylon Viewer data into Istra4D and how to begin the DIC evaluation.

Chapter 6

Modelling

6.1 Introduction

Finite Element Modelling (FEM) is used extensively for computational mathematics problems in the engineering field. It is often used to calculate, but not limited to; heat transfer, structural analysis, fluid dynamics and mass transport. For this research, FEM was used for structural analysis on the steel reinforced glulam beams. ASTM:D7341 (2014) defines the minimum requirements that all modelling reports should address:

1. Timber Species
2. Modulus of Elasticity
3. Creep
4. Finger joint strength
5. Laminae arrangement (balanced or unbalanced)
6. Prediction of properties

There are many different computer softwares that are capable of Finite Element Analysis (FEA). Two common ones used are ABAQUS/CAE (ABAQUS) and ANSYS. Each software has its own set of advantages and disadvantages, and of these two softwares, ABAQUS was used for this research as it was readily available to the Author.

6.2 Previous Models

Many different computational models have been used to predict the behaviour of timber, some are more complex than others. Some models build on previous ones and add in properties that were excluded from the previous version (Serrano, 2003).

Monte Carlo and Weibull Theory

Monte Carlo simulations (Metropolis and Ulam, 1949) and the Weibull Theory (Weibull, 1939) are two statistical methods that are often used in the modelling of glulam. These are two commonly used models as it allows researchers to randomly generate lengths of timber pieces (useful for finger joint placement in glulam beams), to model the random placement of cracks, knots and mechanical properties of timber. Well developed models use these statistical distributions to generate random properties for each cube element of the FEM model.

Foschi and Barret

The model used by Foschi et al. (1980) used the Monte Carlo simulations to predict the strength of glulam beams. They did this by generating random values for Knot-Area-Ratio (KAR), density, MoE, and strength. These were determined using probability density functions. The values were entered into an FE program and the behaviour of the beam was recorded and its strength was determined. The assumptions used in the model are summarised by (Serrano, 2003):

1. Tension and Compression are both linearly elastic properties.
2. The failure criterion used was brittle weakest link failure, this did not allow for progressive failures of the timber.
3. The values used in the statistically generated values for strength and stiffness were based on full sized specimens which are significantly larger than the FE mesh.
4. Finger joints were neglected

This was one of the first FEM simulations that was widely accepted for glulam beams, and many more recent models use this as a starting position and build on it as a foundation of understanding.

Karlsruhe Model

This model is named after the institution where it was developed (Karlsruhe Institute of Technology) however, it was first investigated by Ehlbeck et al. (1985). It is similar to the Foschi-Barret model, as it randomly assigns the timber mechanical properties based on statistical analysis.

The benefit of this model is that finger joints are included in the analysis. The joint placements are determined by measuring the average length between finger joints in a beam lay-up and using statistical distributions to randomly assign a length to each lamination and thus the placement of the finger joints. Whereas the Foschi-Barret Model used the Monte Carlo simulation to determine properties for each cell, the Karlsruhe model instead provided each length of timber between finger joints with its own properties. Using regression equations a KAR-value and density are assigned to each length. The KAR-values are then further multiplied by a factor, taken from more statistical variations, in order to determine the effect of multiple knots in a lamination. The MoE for a given lamination, i , is then calculated using equation 6.1. By multiplying the KAR-value by the factor, even if multiple laminations are assigned the same KAR-values they need not have the same MoE.

$$\ln(E_i) = \ln(E_{reg}) + \Delta_{B,i} + X_i \quad (6.1)$$

Where:

- E_i is an expression of the MoE as a function of density and KAR-values
- $\Delta_{B,i}$ is a normally distributed measure of the average regression line
- X_i is a measure from the average regression line, to the regression line of the current lamination

Improvements on the Foschi-Barret model are as follows (Serrano, 2003):

1. The inclusion of finger joints
2. Using a plasticity model, the compression failure was modelled as non-linear.
3. The failure of the beam was given an iterative progressive failure with four failure criterion. Elements which had previously failed are removed from the next iterative calculation.

I When one element fails, if the stress redistribution leads to the failure of another element, the beam is said to undergo brittle tension failure

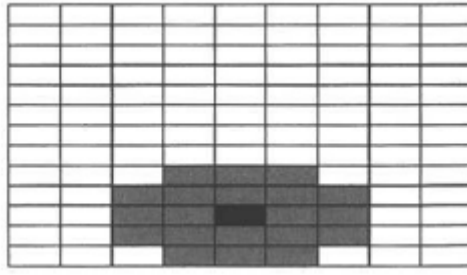


Figure 6.1: A beam will fail if; an already failed cell (black region) is in close proximity to another cell that fails (grey region). (Colling, 1990)

- II If an element fails, in tension, close to a previously failed element the beam is said to collapse. This collapse simulates a shear stress failure in that region. (see figure 6.1)
- III If a finger joint fails, the beam fails in tension.
- IV If none of the above criterion are met, the beam will fail after a 5th element has failed in tension

Serrano Model

The model developed by Serrano et al. (2001) focuses on modelling the effect that finger joints have on a glulam beam. Unlike other models it allows for the simulation of progressive failure and crack propagation within a lamination. Saracoglu (2011) and Khorasani (2012) conducted further research on the effects of cracks and their propagation within a beam, however their research was based on natural cracks, using Linear Elastic Fracture Mechanics (LEFM), and not finger joints. The model of Serrano et al. (2001) was based on Non-Linear Fracture Mechanics (NLFM). In order to isolate the effect of the finger joints, the timber in surrounding areas was modelled as a homogeneous material with deterministic properties. The finger joints were modelled as multiple parallel non-linear spring elements, each with an initial length of zero. The elongation of the spring element can be associated with the formation of cracks and represents the finger joint failure. The model is closely related to a fictitious crack model and describes the gradual damage of a material.

The benefit of this model is that it can be implemented into many different models, which use a more complex system of modelling the anisotropic/ orthotropic properties of wood, as it focuses on just one isolated section of the beam.

PROLAM

Probabilistic modeling of glued-laminated timber beams - referred to as PROLAM - was developed by Bender (1985) and refined by Hernandez et al. (1992). PROLAM uses a 'transformed section' analysis and standard beam theory to determine the strength of a glulam beam. A Monte Carlo simulation (see figure 6.2) is used to determine the glulam lay-up and the carrying capacity of the beam is given as the carrying capacity of the weakest cross section (Serrano, 2003). The beam is assumed to have failed once the tensile strength parallel to the grain of a lamination is exceeded (Bender, 1985)

A computational model was developed by Brown and Suddarth (1977) which performs the necessary calculations for transformed section analysis. The program calculates the maximum moment (m) using

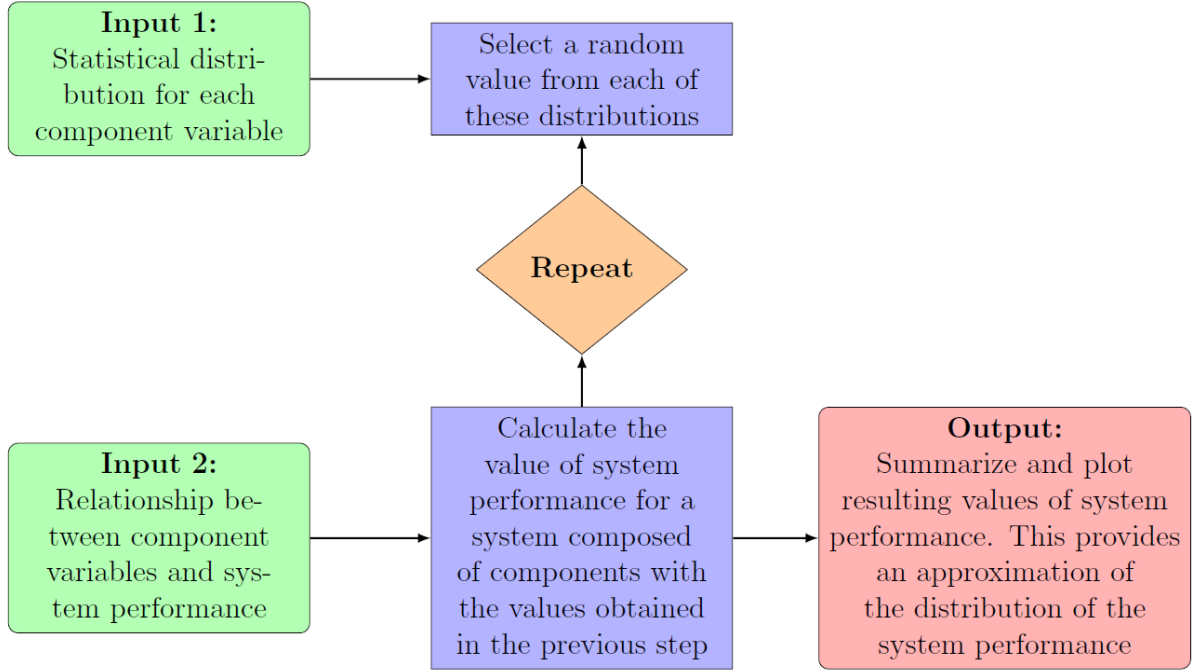


Figure 6.2: Flow chart of Monte Carlo simulation used by Bender (1985)

equation 6.2

$$m_i = \frac{E_s}{E_i} \left(\frac{I_s F_i}{C_i} \right) \quad (6.2)$$

Where:

E_s is the MoE for the standard material

E_i MoE for the i -th lamination

F_i Allowable stress of the i -th lamination

I_s Is the moment of inertia of the transformed section

C_i “Distance between the neutral axis of the transformed section and the extreme fiber of the i -th lamination”

Hernandez et al. (1992) further refines the PROLAM model and uses a virtual work calculation (see equation 6.3) to calculate the deflection (Δ) which can be used in standard elastic deflection MoE calculations (see equation 2.11).

$$\Delta = \sum_{i=1}^n \left[\left(\frac{M_x m_x}{E_c I_i} + \frac{k V_x v_x}{A_i G_i} \right) \times \Delta_x \right] \quad (6.3)$$

Where:

Δ is the total glulam beam deflection at x ,

M_x is the bending moment at x caused by actual loading,

m_x is the bending moment at x caused by a unit load at the midspan of the beam,

V_x is the shear at x caused by actual loading,

v_x is the shear at x caused by a unit load at the midspan of the beam,

k is a form factor (1.2 for rectangular section),

E_c is a constant MoE value used in the transformed cross section,

I_i is the moment of inertia of the i th transformed cross section at x ,

A_i is the transformed area at the i th transformed cross section at x ,

G_i is the shear modulus of the i th transformed cross section at x ,

Δ_x is the increment at which calculations are performed,

n is the total number of increments along the beam length.

According to Hernandez et al. (1992) using the midspan deflection values, an E_{app} is determined and this more accurately predicts values of MoE published in design standards.

ReLAM

Relam, developed by Lindyberg (2000); Lindyberg and Dagher (2012), is a non-linear probabilistic model which is used to analyse reinforced glulam beams in bending and determine the MoE and MoR. Unlike the PROLAM model, which is used for unreinforced beams only, ReLAM is used for reinforced beams only. ReLAM was shown to be accurate to within 1.8% of PROLAM (Lindyberg and Dagher, 2012), however it has not been used extensively in the academic field as a material model for timber. It has however, still been published in the appendix of (ASTM:D7199, 2012) as a possible mechanics based model for reinforced glulam.

Variable mechanical properties of timber, ultimate tensile stress (UTS), ultimate compression stress (UTC), E , and m , are treated as random variables and determined using a three-parameter Weibull distribution. Then using moment curvature (Φ) and standard beam theory to calculate MoE using equation 6.4 and equation 2.11. For the beam to remain in equilibrium the section must maintain rotational and horizontal equilibrium.

$$\sum F_x = 0 \Rightarrow \int_{Area} \sigma(y) dA = 0$$

$$M_{Applied} = M_{internal} = C(or T) \cdot Arm = \int_{Area} -y \cdot \sigma(y) \cdot dA$$

$$f(Y) = \int \sigma(y) dA$$

And the curvature at the cross section is given as:

$$\Phi(x) = \frac{d^2 \Delta}{dx^2}$$

And to calculate the deflection over the beams length equation 6.4 is used, and then substituted into equation 2.11.

$$\Delta(x) = \iint_{span} \Phi(x) dx \quad (6.4)$$

where:

Δ is the beam deflection,

Y is the distance from the compressive face to the neutral axis,

Φ is the beam curvature.

6.3 ABAQUS Modelling

ABAQUS is a commercially available Finite Element Software that is often used in both academic and engineering consulting institutions. The software is able to perform computational mechanics on a wide variety of problems; from linear to non-linear materials, elastic to plastic properties, and is used for all types of engineering problems including fluid dynamics and structural engineering.

For ABAQUS modelling with materials such as steel and concrete, often used in structural engineering, there are predefined materials and material properties that make the simulation of these materials relatively straightforward. The simulation of a material such as timber, that has a high variety of properties, is often more complex as there is no preset material within the ABAQUS database. This is due to the fact that not all timber species are of equal quality, and even within a timber species there are many different factors that result in different properties which often have to be user-defined. Along with the differing properties, the non-linear failure of timber in multiple directions requires the input of different failure criterion. There are many different failure criterion that may be used depending on the expected outcome of the experimental tests. Many of these failure criterion are based on the failure of fibre composites which on a micro-scale have similar structural compositions. The commonly used models are based on the the Hills failure criterion (Hill, 1948), the Tsai-wu criterion (Tsai and Wu, 1971), the Hashin failure model (Hashin, 1980), and the failure conditions identified by Mackenzie-Helnwein et al. (2003). These have been discussed in brief detail in section 6.4.1.

6.3.1 USDFLD-Subroutine

An article written by Iraola and Cabrero (2016) highlights the use of ABAQUS FEM in order to solve for a wooden beam in bending and for a nail embedment test as described by Eurocode EN:383 (2007). The algorithm used in the modelling focuses on the timbers elastic and post-elastic responses to loading. It focuses on the different behaviours experienced in tension and compression, both parallel and perpendicular to the grain. By using a USDFLD-subroutine, which is written in the computer language FORTRAN and added to the ABAQUS file, Iraola and Cabrero (2016) were able to define an application of different failure criteria and model the post-elastic behaviour by using stress and damage reduction parameters.

Previously, the USDFLD-subroutine was used to model the progressive failure in composite materials, but its use is fairly limited. It was deemed to be a suitable technique by (André et al., 2014) and Iraola and Cabrero (2016) found that with a reasonable mesh density and increment size, the results were in good agreement to those of experimental results.

According to the article the USDFLD-subroutine that was developed is able to use any failure criterion chosen by the user, however, for the study Iraola and Cabrero (2016) made use of the Hill criterion which

is shown in equation 6.5

$$1 = F(\sigma_R - \sigma_T)^2 + G(\sigma_t - \sigma_L)^2 + H(\sigma_L - \sigma_R)^2 + 2L\tau_{RT}^2 + 2M\tau_{LT}^2 + 2N\tau_{LR}^2 \quad (6.5)$$

Where:

$$\begin{aligned} F &= \frac{1}{2} \left[\frac{1}{S_R^2} + \frac{1}{S_T^2} - \frac{1}{S_L^2} \right], \\ G &= \frac{1}{2} \left[\frac{1}{S_T^2} + \frac{1}{S_L^2} - \frac{1}{S_R^2} \right], \\ H &= \frac{1}{2} \left[\frac{1}{S_L^2} + \frac{1}{S_R^2} - \frac{1}{S_T^2} \right], \\ L &= \frac{1}{S_{LR}^2}, \\ M &= \frac{1}{S_{LT}^2}, \\ N &= \frac{1}{S_{RT}^2}, \end{aligned}$$

And Where:

S is the strength of the material for each direction and stress type,
 L, R, T are the wood direction Longitudinal, Radial and Tangential.

The material being modelled is said to have failed once the expression (6.5) is equal to or higher than a value of one. After the initial onset of failure, the material undergoes a progression of damage. Three progressions of failure are proposed by Murray and Schwer (1990); instantaneous, gradual unloading, and constant stress. The progression of damage causes the materials properties to degrade and change over time.

Iraola and Cabrero (2016) used two different degradations factors (d_i), one for compression loading which may be used for both parallel and perpendicular direction (equation 6.6) and one for tension loading in both directions (figure 6.3). Here, d_L , is the damage parameter for elements failed to longitudinal compression and tension.

$$d_i = \frac{\sigma_{post,i}}{\epsilon_i E_i} \quad \text{with} \quad 10^{-4} \leq d_i \leq 1 \quad (6.6)$$

And,

$$\sigma_{post,i} = r_i S_i \quad (6.7)$$

Where:

r_i is a parameter dependent on loading direction

As shown in figure 6.3 the compression failure was modelled as a slow ductile failure whereas the tension failure was instantaneous and considered to be a brittle failure.

6.3.2 Mechano-Sorptive and Rheological Model

A modelling aspect that has been extensively researched is the cyclic moisture distribution within a timber member. This modelling technique is usually based on the long term effects of moisture on a structure. To develop a mechano-sorptive model a user-defined material (UMAT) must be written in FORTRAN for

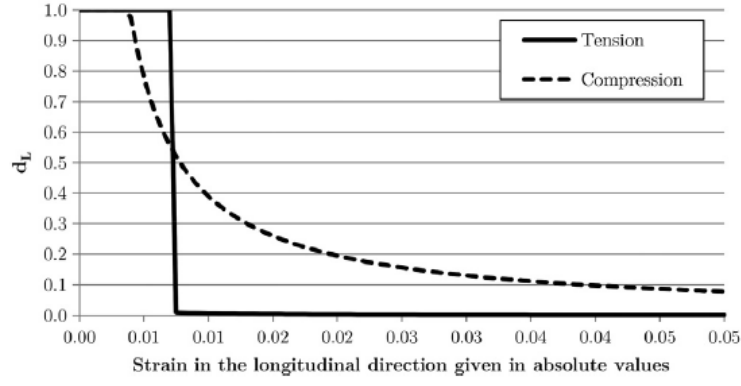


Figure 6.3: Tension vs. compression material degradation for USDFLD
Iraola and Cabrero (2016)

timber, similar to that of the USDFLD in section 6.3.1, and then another subroutine, DFLUX, is written to model the flow of moisture. A relevant UMAT is presented by Santaoja et al. (1991) and Mirianon et al. (2008) present a study describing how one would be able to develop their own UMAT and DFLUX subroutines. O’Ceallaigh (2016) uses the UMAT to model the long term effects of mechano-sorptive creep on timber elements in a built-up frame.

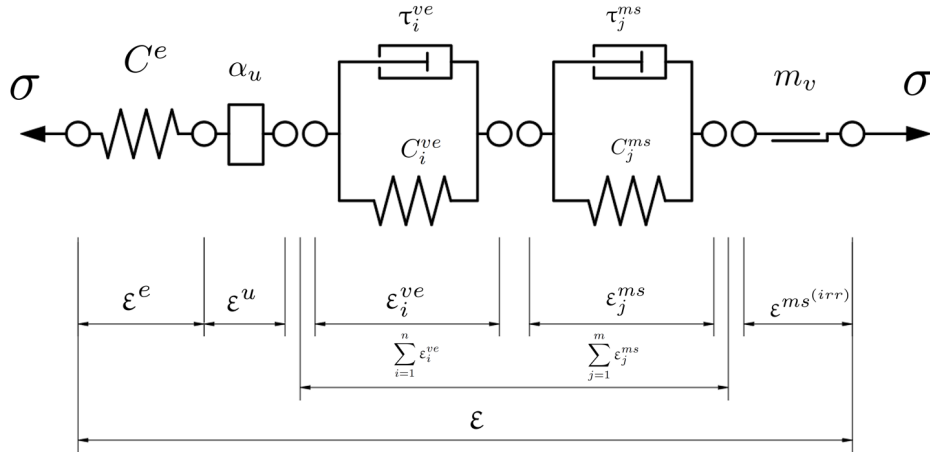


Figure 6.4: Rheological model schematic as adapted from Mirianon et al. (2008)

This is a complex model as it deals with fluid mechanics and timber over extended periods of time. It focuses on modelling five different mechanisms, which are presented by Mirianon et al. (2008). The five mechanisms determine the overall strain and is calculated using equation 6.8

$$\epsilon = \epsilon^e + \epsilon^u + \epsilon^{ms} + \epsilon^{ms(irr)} \quad (6.8)$$

Where:

- ϵ is the total strain vector
- ϵ^e is the elastic strain vector

- ε^u is the hygroexpansion strain vector
- ε^{ms} is the the recoverable mechano-sorptive strain vector
- $\varepsilon^{ms^{(irr)}}$ is the irrecoverable mechano-sorptive strain vector

Determining each of these mechanisms with equations 6.9 to 6.14 the **UMAT** and **DFLUX** are able to be implemented into the ABAQUS software to calculate the moisture creep within the timber.

Elastic Strain Increment:

$$\Delta \varepsilon_{n+1}^e = \Delta C^e \sigma_{n+1} + C^e \Delta \sigma_{n+1} = \Delta \varepsilon_1^e + \Delta \varepsilon_2^e \quad (6.9)$$

Hygroexpansion Strain Increment:

$$\Delta \varepsilon^u = \alpha_u \Delta u \quad (6.10)$$

Which when calculated along the longitudinal direction, and applying a strain dependence on shrinkage, equates to equation 6.11 (Toratti, 1992)

$$\Delta \varepsilon_{33}^u = (\alpha_{u,3} - 1.3 \varepsilon_{33}) \Delta u \quad (6.11)$$

Mechano-sorptive Strain Increments:

The total mechano-sorptive creep strain is given as:

$$\Delta \varepsilon^{ms^{(total)}} = \Delta \varepsilon_{n+1}^{ms^{(irr)}} + \sum_{j=1}^m \Delta \varepsilon_{j,n+1}^{ms} \quad (6.12)$$

where the irrecoverable vector is given as:

$$\Delta \varepsilon^{ms^{(irr)}} = C_j^{ms} \sigma_{n+1} |\Delta U| \quad (6.13)$$

and the recoverable vector is given as:

$$\Delta \varepsilon_{j,n+1}^{ms} = C_j^{ms} T_{j,n+1} \left(\frac{\Delta u}{\tau_j^{ms}} \right) \Delta \sigma_{n+1} - (\varepsilon_{j,n}^{ms} - C_j^{ms} \sigma_n) \left(1 - \exp \left(-\frac{|\Delta u|}{\tau_j^{ms}} \right) \right) \quad (6.14)$$

This method of modelling is described extensively by Mirianon et al. (2008) in sections 2.3 to 2.4 and by O'Ceallaigh (2016) in section 6.4. These papers go into detail of the necessary calculations and methodology required, both to write the **UMAT** and **DFLUX** subroutines and how to implement them into the ABAQUS software.

6.4 Continuum Damage Mechanics

Continuum Damage Mechanics (CDM) is based on the early works of French mathematician Augustin-Louis Cauchy and developed into what we know today by Janson and Hult (1977); Krajcinovic (1984); Kachanov (1986) and is used for modelling of different damage methods, usually in brittle materials. It does this by focusing on the macroscopic level, by means of modelling the material as a continuous mass rather than discrete particles. When continuum mechanics describes a point on a material it does not

refer to an atomic space, but rather an idealized area of the material occupying that point. Following this method produces an accurate macroscopic model which can make predictions on the crack initiation, propagation and fracture of the material. The use of damage mechanics has been extensively used in the modelling of fibre composite materials.

In recent years, research by Sandhaas (2012) developed a subroutine for hardwoods based on the CDM process. The process of CDM was selected as it is a non-linear elastic approach which adjusts the stiffness matrix (D) or its inverse, the compliance matrix ($\bar{C}$) to account for non-linear behaviour. The work of Sandhaas (2012) was extended by Gharib et al. (2017) where special focus was given to the hardening behavior of timber in transverse compression and efforts in resolving the localization issues of brittle failure modes. Both Sandhaas (2012) and Gharib et al. (2017) used a UMAT subroutine, like that used in section 6.3.2 by Mirianon et al. (2008). Sandhaas (2012) found that the CDM model accurately predicted the results of stiffness degradation and that the theoretical damage due to stresses was similar to that of the experimental results. However, problems occurred when steel dowels were placed in the timber model. The model was unable to accurately transfer loads from strongly distorted elements to nearby elements. Gharib et al. (2017) tested their model in multiple scenarios such as; dog-bone tensile tests, embedding tests, and a dowel-joint connection. The model was compared to results obtained experimentally and found that stiffness degradation, modes of failure and the non-linear behaviour were accurately modelled using the improved UMAT.

The failure of materials when modelled using continuum mechanics is based on the fundamental Hooke equation (see equation 6.15). After determining the damage variable ($d, 0 \leq d \leq 1$) it is inserted into the equation to determine whether the material at that point has failed. Figure 6.5 shows the different functions that have been used in previous UMAT subroutines.

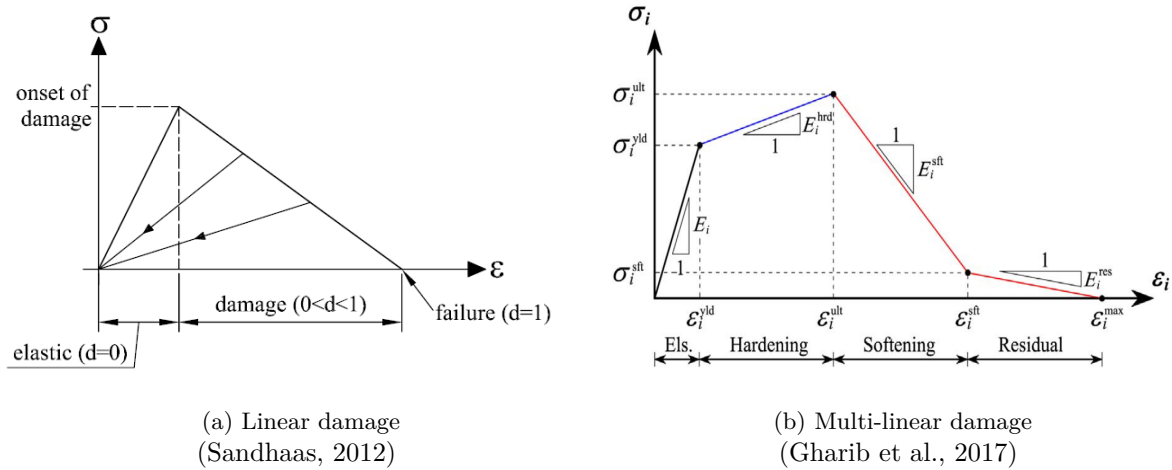


Figure 6.5: Different constitutive damage functions used in modelling of timber

$$\sigma_{ij} = (1 - d)D_{ijkl} \cdot \varepsilon_{kl} \quad (6.15)$$

Where:

$d = 0$ Means that no damage is present

$d = 1$ Means that full failure is present in the material

The damage variable begins to increase once a specified strain limit has been reached. For anisotropic materials, such as timber, there are several damage variables that need to be defined.

Table 6.1 shows the advantages and disadvantages of using a CDM model as layed out by Sandhaas (2012). By looking at the relevance of these advantages and disadvantages with regards to this research, it can be seen why a CDM model has been deemed as a viable option to model timber.

Table 6.1: Advantages and Disadvantages of using CDM (Sandhaas, 2012)

Advantages	Disadvantages
Implementable in standard FE packages	No permanent deformations
Few material parameters needed	Mesh dependent
Material model easily implementable	Convergence problems when used in implicit code

It is relatively easy to implement into ABAQUS, which is a FEM software that is readily available to the author. Material parameters for Pinus Patula are fairly limited, therefore a model that uses few of these parameters is preferable. The disadvantage of being mesh dependent means that the results are heavily dependent on the geometry of the mesh used in the FEA, whereby a smaller mesh geometry results in a higher degree of accuracy. Further development of these properties is seen in section 7.4.1.

Not all UMAT's are equal in complexity, ones created for simple isotropic materials are straightforward and require little coding and few input parameters; however, ones created for orthotropic and/or transversely isotropic materials can be fairly complex. The UMAT code may not all be the same, but the general idea and flowpath of the UMAT is the same. Figure 6.6 shows the flowpath outline that all UMAT's must comply to independent of which type of material has been selected.

6.4.1 Mathematical Failure Criteria

Many different mathematical models exist that are able to predict the different failure methods of materials. One of the initial mathematical damage criterion that was developed by Hill (1948) is often the starting block for new models. Later versions of this model try and address some of its limitations. The main limitation with using the original Hill-criterion is the fact that it is based on a quadratic format. This means that whether stresses are positive or negative the model is unable able to determine whether the material has failed in tension or compression (Sandhaas, 2012).

Tsai-Wu criterion

The Tsai-Wu criterion is an iteration of the Hill criterion developed by Tsai and Wu (1971). It provides an additional parameter and extends the the definition to include the interaction of normal stresses. The criterion is given in equation 6.16 as:

$$\Phi = A_i \sigma_i + B_{ij} \sigma_i \sigma_j - 1 \quad (6.16)$$

Where:

Φ is the criterion,

A_i is a vector which contains 1st order contributions of the different stress components,

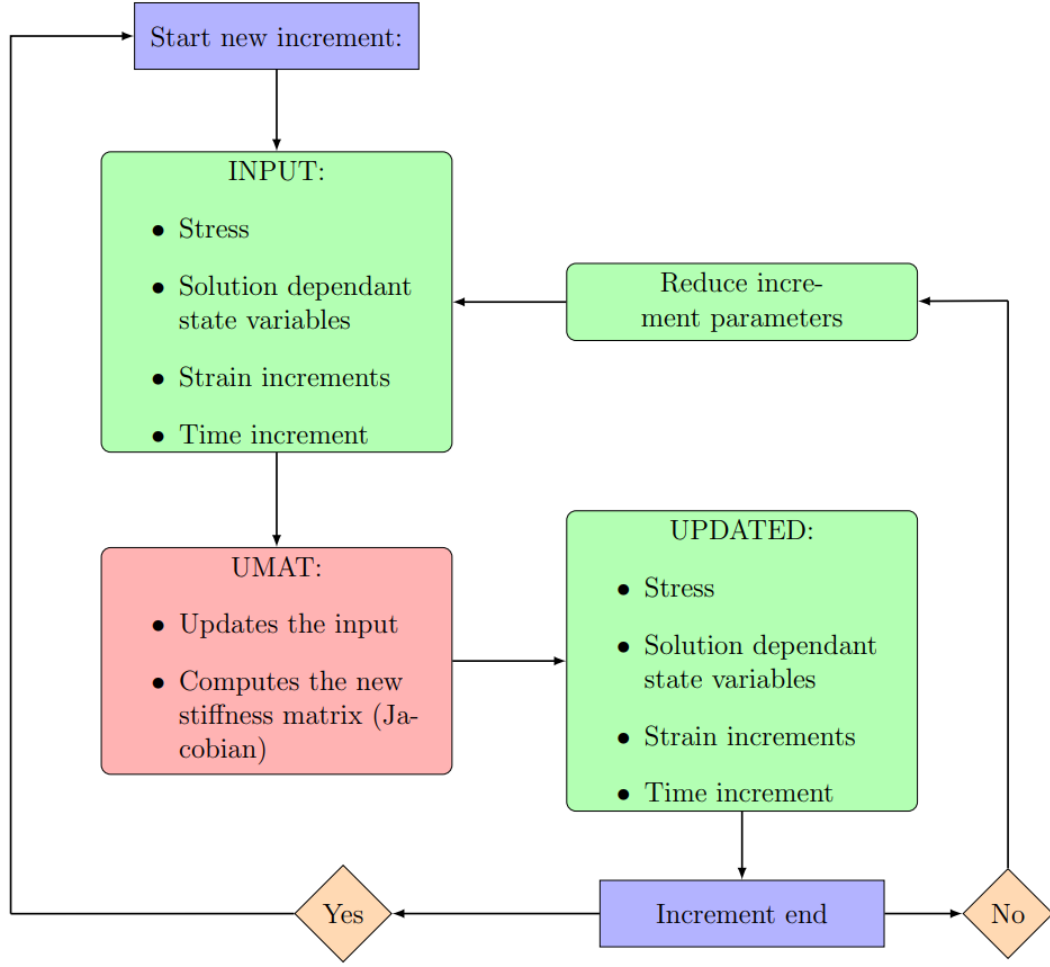


Figure 6.6: Basic flowpath for a UMAT

B_{ij} is the 2^{nd} order effects of the interaction between different stress components.

Expanding on equation 6.16 Gharib et al. (2017) explains that the vector, A_i , allows for an asymmetrical solution in terms of each stress component ($A_i = 0$ for $i \geq 4$). The model also allows any shear components to be neglected (B_{ij} for $i, j \geq 4$ and $i \neq j$). Additionally, the normal and shear component interactions can be neglected ($B_{ij} = 0$ for $i \leq 3$ and $j \geq 4$). This produces an expanded version of equation 6.16 and is shown in equation 6.17 which describes the Tsai-Wu criterion for an orthotropic material:

$$\begin{aligned}
 \Phi = & A_1\sigma_1 + A_2\sigma_2 + A_3\sigma_3 \\
 & + B_{11}\sigma_1^2 + B_{22}\sigma_2^2 + B_{33}\sigma_3^2 + B_{44}\sigma_4^2 + B_{55}\sigma_5^2 + B_{66}\sigma_6^2 \\
 & + 2B_{12}\sigma_1\sigma_2 + 2B_{23}\sigma_2\sigma_3 - 1 \quad (6.17)
 \end{aligned}$$

Gharib et al. (2017) expands equation 6.17 further by separating it into two separate equations. This is done by neglecting the interaction of the fibre strength and transverse pressure ($B_{12} = B_{13} = 0$). This produces the two equations for the failure criteria, equation 6.18 is the criteria for the fibre direction whereas equation 6.19 is the criteria for the matrix direction (radial and tangential directions).

$$\text{Fibre direction: } \Phi^{(1)} = A_1\sigma_1 + B_{11}\sigma_1^2 + B_{55}\sigma_5^2 + B_{66}\sigma_6^2 - 1 \quad (6.18)$$

$$\begin{aligned} \text{Matrix direction: } \Phi^{(2)} = & A_2\sigma_2 + A_3\sigma_3 + B_{22}\sigma_2^2 + B_{33}\sigma_3^2 \\ & + B_{44}\sigma_4^2 + B_{55}\sigma_5^2 + B_{66}\sigma_6^2 + 2B_{23}\sigma_2\sigma_3 - 1 \end{aligned} \quad (6.19)$$

The matrix direction mentioned in equation 6.19 is further decomposed by Davila et al. (2005) in which the conditions of an orthotropic material are preserved along with an assumption of $B_{23} = 0$. The matrix direction splits into the radial direction and the tangential direction. This leaves a failure criterion of the fibre direction (equation 6.18), the radial direction (equation 6.20) and the tangential direction (equation 6.21).

$$\text{Radial direction: } \Phi^{(2)} = A_2\sigma_2 + B_{22}\sigma_2^2 + B_{44}\sigma_4^2 + B_{55}\sigma_5^2 - 1 \quad (6.20)$$

$$\text{Tangential direction: } \Phi^{(3)} = A_3\sigma_3 + B_{33}\sigma_3^2 + B_{44}\sigma_4^2 + B_{66}\sigma_6^2 - 1 \quad (6.21)$$

Determining the values for the 1st order contributions produces:

$$A_1 = \frac{f_1^c - f_1^t}{f_1^t \cdot f_1^c}, \quad A_2 = \frac{f_2^c - f_2^t}{f_2^t \cdot f_2^c}, \quad A_3 = \frac{f_3^c - f_3^t}{f_3^t \cdot f_3^c}$$

And determining the values for the 2nd order contributions produces:

$$\begin{aligned} B_{11} &= \frac{1}{f_1^t \cdot f_1^c}, & B_{22} &= \frac{1}{f_2^t \cdot f_2^c}, & B_{33} &= \frac{1}{f_3^t \cdot f_3^c}, \\ B_{44} &= \frac{1}{f_4^2}, & B_{55} &= \frac{1}{f_5^2}, & B_{66} &= \frac{1}{f_6^2}, \end{aligned}$$

Where:

- f_i^a is the uniaxial strength of timber in direction a (t - tension, c - compression) in direction i for $i \leq 3$,
- f_4 is the rolling shear strength in the plane -23
- f_5, f_6 is the splitting shear strength in the plane -12 and -13

Chapter 7

The UMAT and CDM model

7.1 Introduction

Continuum damage mechanics is a method which can be used to model different behaviours of a material, specifically softening and hardening. It does this by modifying the stiffness matrix (D), and in turn its inverse, the compliance matrix ($\bar{C}$). The biggest disadvantages of this modelling technique is its mesh dependency and problems related to convergence. This Chapter will outline the mathematical basis of the CDM model which is based on the works of Sandhaas (2012) and Gharib et al. (2017). The model will be coded into a UMAT and implemented into ABAQUS. The model provides a method by which the mesh sensitivity and convergence problems may be reduced, but not alleviated completely. Appendix B shows sections of the UMAT code used and section 9.3.1 shows a visual representation of the implementation process within the ABAQUS menus.

7.2 Material Definitions

The first stage of the CDM model is to clearly define the material and its properties. This includes the conventions used for the axes orientation and the type of failures that are expected to be modelled. Only once this has been determined can the mathematics by which to model the material be properly demonstrated.

Throughout section 2, timber was looked at in terms of an orthotropic material, as well as a transversely isotropic material. For modelling purposes it is easier to model a transversely isotropic material due to it only having five constants as opposed to the nine constants of the orthotropic material. This being said, it is pointed out by Sandhaas (2012) and Blaß and Sandhaas (2017) that the five constants are insufficient to create an accurate mathematical model. In a transverse isotropic material G_{23} is not an independent constant, but rather a function of the E_{22} and ν_{23} constants. For timber, this assumption produces an incorrect value of G_{23} (= rolling shear). Gharib et al. (2017) uses the orthotropic material definition as well for the mathematical model produced. For failure criterion of an orthotropic material the decomposition of equation 6.19 by Davila et al. (2005) as an extension of the Tsai-Wu failure criterion can be used, opposed to the Hashin-model that would be used for a transversely isotropic material.

Therefore, with this in mind the CDM model provided in this research will characterize Pinus Patula timber as an orthotropic material with nine independent material constants.

7.2.1 Orientation

As previously determined, the material directions are; Longitudinal (L), Radial (R) and Tangential (T). These directions are related to the grain direction as shown in figure 7.1. These directions can be related to the stress components using the definition:

$$\begin{aligned}\sigma_L, \sigma_R, \sigma_T &= \sigma_{11}, \sigma_{22}, \sigma_{33} \\ \sigma_{LR}, \sigma_{LT} &= \sigma_{12}, \sigma_{13} \\ \sigma_{RL}, \sigma_{RT} &= \sigma_{21}, \sigma_{23} \\ \sigma_{TL}, \sigma_{TR} &= \sigma_{31}, \sigma_{32}\end{aligned}$$

The notation is confusing due to the high number of material directions, as is expected with an

orthotropic material, and in many research papers the indices are readily interchangeable. For the purposes of the UMAT being written the indices will be written in the form σ_{ij} where i and j are numerals.

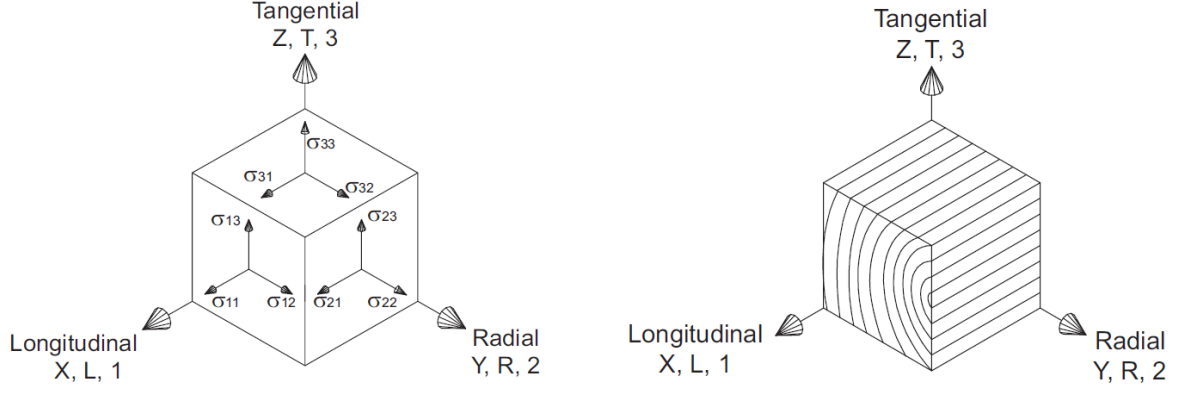


Figure 7.1: Stress components and material directions
(Sandhaas et al., 2012)

Equation 7.1 is the same as that of equation 2.2, now with the relevant indices according to figure 7.1. This follows that the stiffness matrix is the inverse of the compliance matrix ($D = C^{-1}$). These matrices are a 6×6 matrix that follow Hooke's law as described by Harte (2009).

$$\varepsilon_i = \bar{C}_{ij} \bar{\sigma}_j \quad (7.1)$$

Where:

$$\begin{aligned} \varepsilon &= \{\varepsilon_{11}, \varepsilon_{22}, \varepsilon_{33}, 2\varepsilon_{23}, 2\varepsilon_{12}, 2\varepsilon_{13}\}^T \\ \bar{\sigma} &= \{\sigma_{11}, \sigma_{22}, \sigma_{33}, \sigma_{23}, \sigma_{12}, \sigma_{13}\}^T \end{aligned}$$

And C_{ij} is given in equation 7.2 as:

$$\bar{C}_{ij} = \begin{bmatrix} \frac{1}{E_1} & -\frac{\nu_{21}}{E_1} & -\frac{\nu_{31}}{E_1} & & & \\ -\frac{\nu_{12}}{E_2} & \frac{1}{E_2} & -\frac{\nu_{32}}{E_2} & & & \\ -\frac{\nu_{13}}{E_3} & -\frac{\nu_{23}}{E_3} & \frac{1}{E_3} & & & \\ & & & \frac{1}{E_4} & & \\ 0 & & & & \frac{1}{E_5} & \\ & & & & & \frac{1}{E_6} \end{bmatrix} \quad (7.2)$$

Where:

$$\begin{aligned} E_1, E_2, E_3 &= E_{11}, E_{22}, E_{33} \\ E_4, E_5, E_6 &= G_{23}, G_{12}, G_{13} \end{aligned}$$

And the Poisson ratios in equation 7.2 are given as such that they follow the definition of symmetry due to limited research into all six Poisson ratios of Pinus Patula:

$$\begin{aligned}v_{12}E_1 &= v_{21}E_2 \\v_{13}E_1 &= v_{31}E_3 \\v_{23}E_2 &= v_{32}E_3\end{aligned}$$

7.2.2 Failure Modes

Similar to that of the Karlsruhe model in section 6.2 it is necessary to define the modes of failure in which the material is expected to fail. Due to a fibrous orthotropic material being modelled failure along multiple planes should be expected, each with their own behaviour and orientation. The mechanical behaviour of these failures with respect to the material direction can be found in section 2.7. Failure modes I - VIII are given by Sandhaas (2012) (FM I - VIII).

- FM I: *Tension parallel-to-the-grain* - ($t_{\parallel}$)
Tension failure in the direction of the grain is considered as a brittle failure. The stress responsible for this type of failure is that of σ_{11} (σ_L).
- FM II: *Compression parallel-to-the-grain* - ($c_{\parallel}$)
Opposite to the behaviour in FM I, parallel compression is considered to have a ductile failure response. The stress component responsible for this failure is also σ_{11} . It should be noted that once a timber member has reached the point at which compression strength is lost due to the buckling of fibres there is also a reduction in $t_{\parallel}$. The same can not be said for that of FM I i.e. as $t_{\parallel}$ strength decreases, $c_{\parallel}$ strength does not necessarily decrease.
- FM III: *Tension perpendicular-to-the-grain, tangential splitting (in LT-plane)*
This FM is due to tensile stresses in σ_{22} (R), longitudinal shear stresses in σ_{12} (in plane - LT) as well as the rolling shear stresses from σ_{23} . Activation of this failure mode results in a degradation in three stiffness components namely, E_{22} , G_{12} , G_{23} . Damage in tension perpendicular to the grain must occur during FM III.
- FM IV and FM V: *Compression perpendicular-to-the-grain, radial direction (R)*
FM IV refers to the transverse compression and FM V refers to shear. Both of these are caused due to compression loading in σ_{22} . This compression loading is perpendicular to the grain and causes a ductile failure in the wood.
- FM VI: *Tension perpendicular-to-the-grain, radial splitting (in LR-plane)*
Tensile stresses in σ_{33} , shear stresses in σ_{13} and rolling shear stresses in σ_{23} result in a brittle failure in wood. The degradation of stiffness factors is similar to that of FM III.
- FM VII and VIII: *Compression perpendicular-to-the-grain, tangential direction*
FM VII and VIII are comparable to that of FM IV and FM V.

7.3 Damage Evolution

As shown previously in figure 6.5b the damage evolution is given as a multi-stage process. This multi-stage process can be divided into the separate stages in order to reduce the complexity of the modelling process. Figure 7.2 shows the different stages with figure 7.2a displaying the graph used for elastic-plastic hardening for ductile failure modes and figure 7.2b displaying the graph used for linear softening of brittle failure modes.

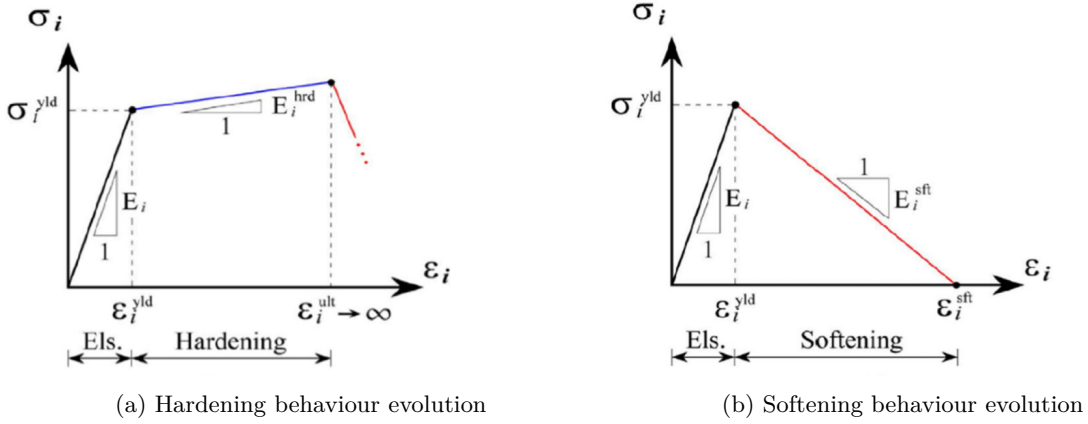


Figure 7.2: Separate stages of the damage evolution proposed by Gharib et al. (2017)

The equations that define these failure behaviours are presented as a piecewise linear-stress function as shown in equation 7.3.

$$\begin{aligned}
 E_i^{\text{hrd}} &= \frac{\sigma_i^{\text{ult}}(2\hat{\sigma}_i^{\text{ult}} - \sigma_i^{\text{ult}}) - 2E_i U_i^{\text{ult}}}{\varepsilon_i^{\text{ult}} \hat{\sigma}_i^{\text{ult}} - 2U_i^{\text{ult}}} \\
 E_i^{\text{sft}} &= \frac{[1 - (\alpha_i^{\text{sft}})^2](\sigma_i^{\text{ult}})^2}{2(1 - \beta_i^{\text{res}})(U_i^{\text{max}} - U_i^{\text{ult}})} \\
 E_i^{\text{res}} &= \frac{(\alpha_i^{\text{sft}})^2 (\sigma_i^{\text{ult}})^2}{2\beta_i^{\text{res}}(U_i^{\text{max}} - U_i^{\text{ult}})}
 \end{aligned} \tag{7.3}$$

Where:

- $\hat{\sigma}_i^{\text{ult}}$ is an effective stress which relates to $\varepsilon_i^{\text{ult}}$,
- $\varepsilon_i^{\text{ult}}$ is the ultimate strain,
- α_i^{sft} is a ratio between residual strength and the ultimate capacity ($\sigma_i^{\text{sft}} : \sigma_i^{\text{ult}}$)
- β_i^{res} is a value pertaining to the post hardening strain energy for the remaining strength ($U_i^{\text{max}} - U_i^{\text{ult}}$)

There are a total of four possible stages in the failure response of timber. The initial elastic phase followed by a hardening effect that occurs when timber undergoes perpendicular compression and the onset of crack propagation (E^{hrd}). The final two stages account for bi-linear softening of the timber as it begins to fail and develop open cracks (E^{sft} and E^{res}).

Using the decomposition of the Tsai-Wu criterion in section 6.4.1, accounting for the multi-axial stress states of the material, the interaction between transverse pressure and the strength in fibre direction

can be disregarded. Using equation 7.4 given by Nouri et al. (2019) and Gharib et al. (2017) a redefined damage criteria can be adopted at the stage of initial damage.

$$d_i = \begin{cases} \left(1 - \frac{E_i^{\text{hrd}}}{E_i}\right) \left(1 - \frac{1}{k_i}\right) & \forall \quad 1 \leq k_i \leq k_i^{\text{ult}} \\ d_i^{\text{ult}} \cdot \frac{k_i^{\text{ult}}}{k_i} + \left(1 + \frac{E_i^{\text{sft}}}{E_i}\right) \left(1 - \frac{k_i^{\text{ult}}}{k_i}\right) & \forall \quad k_i^{\text{ult}} \leq k_i \leq k_i^{\text{sft}} \\ d_i^{\text{sft}} \cdot \frac{k_i^{\text{sft}}}{k_i} + \left(1 + \frac{E_i^{\text{res}}}{E_i}\right) \left(1 - \frac{k_i^{\text{sft}}}{k_i}\right) & \forall \quad k_i^{\text{sft}} \leq k_i \leq k_i^{\text{max}} \end{cases} \quad (7.4)$$

Where:

$k_i^{\text{ult}}, k_i^{\text{sft}}, k_i^{\text{max}}$ are upper bounds of the respective state variable stages,
 $d_i^{\text{ult}}, d_i^{\text{sft}}$ are the damage parameter values for $k_i^{\text{ult}}, k_i^{\text{sft}}$

After damage parameters along the three principle directions ($\vartheta = 1, 2, 3$) have been determined for all active actions (compression/tension) the final stress component (σ_i) in the direction can be calculated using equation 7.5

$$d_i^{\vartheta} = \frac{\langle \sigma_i \rangle}{|\sigma_i|} d_i^{(\vartheta, t)} + \frac{\langle -\sigma_i \rangle}{|\sigma_i|} d_i^{(\vartheta, c)} \quad (7.5)$$

Where a Macaulay operation is given by $\langle \rangle$ and the values of d_i^{ϑ} are not summed over the index i . The final stage is taking into account all different failure modes described in 7.2.2 across all directions in order to determine the damage parameter d_i for a multi-directional stress state which is obtained through the superposition of the multiple damage parameters in all directions and is finally given in equation 7.6.

$$d_i = 1 - \prod_{\vartheta} (1 - d_i^{\vartheta}) \quad (7.6)$$

Where pi-notation is similar to that of sigma-notation, whereby terms are now multiplied together across an index value instead of the normal summation of sigma-notation.

The value obtained for d_i can be substituted into the Hooke's law damage equation originally given in equation 6.15 and given again here. This produces a new value for σ_{ij} which changes the damage parameter d_i . This is the mechanic behind the iterative process within the CDM model.

$$\sigma_{ij} = (1 - d) D_{ijkl} \cdot \varepsilon_{kl}$$

7.4 Geometry

Computational modelling of timber is difficult and can often require large amounts of resources and time to finish the calculations, this is especially true when introducing glulam in its multiple layers. There are multiple ways in which the computations can be reduced, two of which have been mentioned before as mesh dependency and convergence, which are discussed further in section 7.4.1. Another method is used by Nouri et al. (2019) in which CLT is modelled as a solid timber material, where the lamellae were assumed to have a perfect bond between each other. As shown by the experimental and numerical data

found in the research, this assumption is reasonably accurate and can be improved further by using the correct bonding agent discussed in section 2.9.1. Brunetti et al. (2019) model all laminations of the glulam layup and ensured that the beam lamellae were symmetrical, as well as the test setup, and modelled only half the beam. This significantly reduced the required number of computational calculations and the results were found to be within an acceptable margin of error.

The method used by Brunetti et al. (2019) requires a large degree of control on the beam production and for the purpose of this research is not deemed a suitable method. Therefore, the method used by Nouri et al. (2019) in which the multiple layers are bonded perfectly and modelled as a solid material will be used.

7.4.1 Mesh Dependency

The softening behaviour discussed in section 6.4 presents a significant problem when it comes to continuum modelling with regards to the mesh dependency of the results.

Mesh refers to the size of each element within a finite element model. It is preferable to have a larger mesh as it results in a faster computational solution. However, if too large a mesh is selected then optimal results may not be obtained and the solution may be considered inaccurate. The mesh dependency of a model is mathematical in nature and refers to the stability of the results as the mesh size is increased.

A common method to solve the mesh dependency problem is by using a crack band model (CBM) as seen in the models of Sandhaas (2012); Gharib et al. (2017) and Nouri et al. (2019). The crack band model is also used in the fracture theory of concrete (Bažant and Oh, 1983). The CBM alleviates this issue by normalising an elements fracture energy.

The crack band model is given as:

$$g_i^{frc} = \frac{G_i^{frc}}{l_c} \quad (7.7)$$

Where:

- g_i^{frc} is the characteristic fracture energy,
- G_i^{frc} is the post peak fracture energy given as $U_i^{max} - U_i^{ult}$,
- U_i^{max} is the strain energy at complete failure,
- U_i^{ult} is the strain energy at the ultimate state,
- l_c is the typical element length

A mesh sensitivity test was conducted by Nouri et al. (2019) on a layup of CLT and laminated veneer lumber (LVL) boards in ABAQUS. One of the specimens tested is shown in figure 7.3 and the results of the test are given in figure 7.4.

The results of the analysis show that there is no significant difference when determining the peak bending moment capacity with the difference between the three tests being 3.6% and 5.2% respectively. However, post fracture bending capacity shows a significant difference in results. Therefore, depending on the computational time available and the accuracy of the results required the 15mm mesh is recommended. Sandhaas (2012) presented similar results and gives a simplified graph (figure 7.5) based on the damage evolution graph previously shown in figure 6.5a.

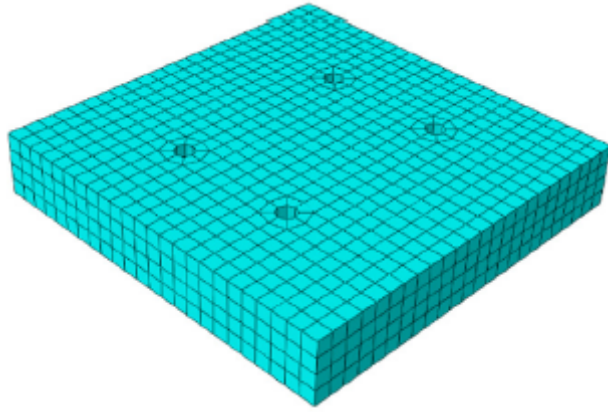


Figure 7.3: Geometry of the mesh dependency test with 15mm elements (Nouri et al., 2019)

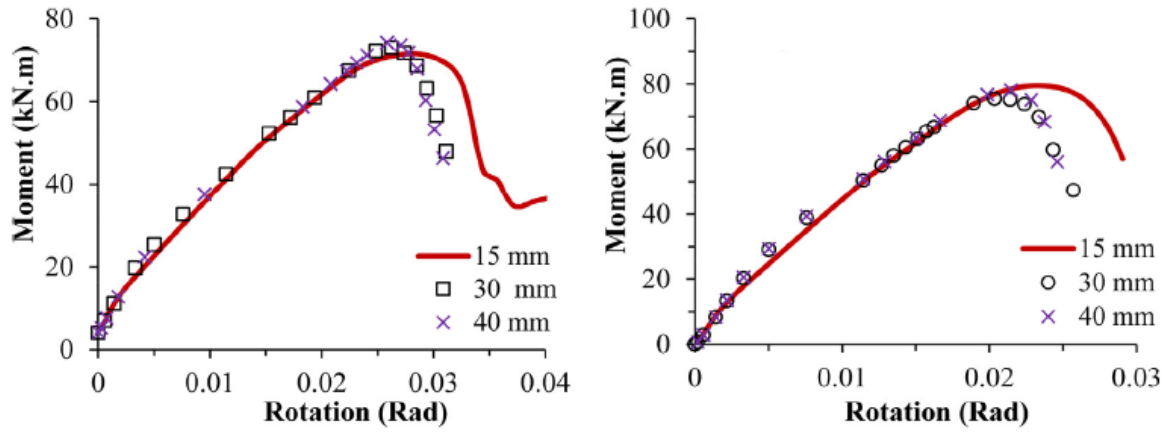


Figure 7.4: Results of a mesh dependency test for a CLT specimen with 15, 30, and 40mm element sizes (Nouri et al., 2019)

Problems with regards to convergence may also be encountered, especially when there is a significant change with regards to the damage parameter of a specific time parameter. This problem is usually handled by ABAQUS using the arbitrary Lagrangian-Eulerian (ALE) formulation which is natively adopted by the ABAQUS software.

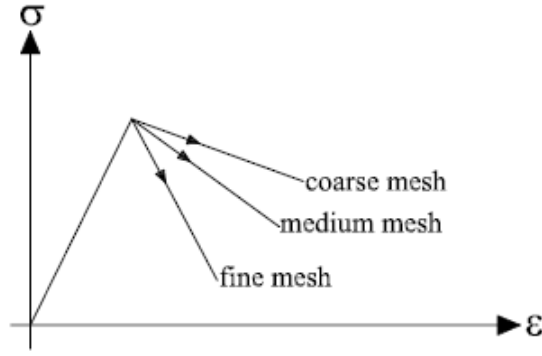


Figure 7.5: Influence of mesh size on numerical results
(Sandhaas, 2012)

7.5 Input Material Properties

To model the timber as accurately as possible (Sandhaas, 2012) used a total of 18 material properties (referred to as **PROPS** in the **UMAT** code) to define its behaviour, (Gharib et al., 2017; Nouri et al., 2019) expanded on the work of Sandhaas and improved the **UMAT** to include a total of 72 defineable material properties.

Using the 72 material property approach has its advantages where the model is significantly improved with regards to accuracy of the material behaviour however, for many timber species it often difficult to find a value for all the required timber parameters. In most cases values for the elastic, plastic, ultimate strength and directional Poisson ratios can readily be found in literature, and for the lesser known properties, such as viscous parameters, there are generalised formulas which can determine approximate values based on the more common known strength values.

ELASTIC PROPERTIES

E_{11}	=	PROPS (1)	MOE Parallel-to-grain
E_{22}	=	PROPS (2)	MOE Perpendicular-to-grain radial
E_{33}	=	PROPS (3)	MOE Perpendicular-to-grain tangential
G_{12}	=	PROPS (4)	Splitting shear modulus
G_{13}	=	PROPS (5)	Splitting shear modulus
G_{23}	=	PROPS (6)	Rolling shear modulus
ν_{12}	=	PROPS (7)	Poisson ratio
ν_{13}	=	PROPS (8)	Poisson ratio
ν_{23}	=	PROPS (9)	Poisson ratio

ULTIMATE STRENGTH PROPERTIES

$\sigma_{t,11}$	=	PROPS (10)	Tensile strength parallel-to-grain
$\sigma_{t,22}$	=	PROPS (11)	Tensile strength perpendicular-to-grain radial
$\sigma_{t,33}$	=	PROPS (12)	Tensile strength perpendicular-to-grain tangential

$\sigma_{c,11}$	=	PROPS (13)	Compressive strength parallel-to-grain
$\sigma_{c,22}$	=	PROPS (14)	Compressive strength perpendicular-to-grain radial
$\sigma_{c,33}$	=	PROPS (15)	Compressive strength perpendicular-to-grain tangential
$\sigma_{v,2}$	=	PROPS (16)	Radial shear strength
$\sigma_{v,3}$	=	PROPS (17)	Tangential shear strength
$\sigma_{v,23}$	=	PROPS (18)	Rolling shear

ULTIMATE STRAIN PROPERTIES

$\epsilon_{t,11}$	=	PROPS (19)	Peak tensile strain parallel-to-grain
$\epsilon_{t,22}$	=	PROPS (20)	Peak tensile strain perpendicular-to-grain radial
$\epsilon_{t,33}$	=	PROPS (21)	Peak tensile strain perpendicular-to-grain tangential
$\epsilon_{c,11}$	=	PROPS (22)	Peak compressive strain parallel-to-grain
$\epsilon_{c,22}$	=	PROPS (23)	Peak compressive strain perpendicular-to-grain radial
$\epsilon_{c,33}$	=	PROPS (24)	Peak compressive strain perpendicular-to-grain tangential
$\epsilon_{v,2}$	=	PROPS (25)	Peak radial shear strain
$\epsilon_{v,3}$	=	PROPS (26)	Peak tangential shear strain
$\epsilon_{v,23}$	=	PROPS (27)	Peak rolling shear strain

CONSUMED ENERGY

$Gu_{t,11}$	=	PROPS (28)	Parallel tensile fracture energy up to ϵ
$Gu_{t,22}$	=	PROPS (29)	Radial tensile fracture energy up to ϵ
$Gu_{t,33}$	=	PROPS (30)	Tangential tensile fracture energy up to ϵ
$Gu_{c,11}$	=	PROPS (31)	Parallel compression fracture energy up to ϵ
$Gu_{c,22}$	=	PROPS (32)	Radial compression fracture energy up to ϵ
$Gu_{c,33}$	=	PROPS (33)	Tangential compression fracture energy up to ϵ
$Gu_{v,2}$	=	PROPS (34)	Radial shear fracture energy up to ϵ
$Gu_{v,3}$	=	PROPS (35)	Tangential shear fracture energy up to ϵ
$Gu_{v,23}$	=	PROPS (36)	rolling shear fracture energy up to ϵ

TOTAL FRACTURE ENERGY

$Gf_{t,11}$	=	PROPS (37)	Total parallel tensile fracture
$Gf_{t,22}$	=	PROPS (38)	Total radial tensile fracture energy
$Gf_{t,33}$	=	PROPS (39)	Total tangential tensile fracture energy
$Gf_{c,11}$	=	PROPS (40)	Total parallel compression fracture energy
$Gf_{c,22}$	=	PROPS (41)	Total radial compression fracture energy

$Gf_{c,33}$ = PROPS (42) Total tangential compression fracture energy

$Gf_{v,2}$ = PROPS (43) Total radial shear fracture energy

$Gf_{v,3}$ = PROPS (44) Total tangential shear fracture energy

$Gf_{v,23}$ = PROPS (45) Total rolling shear fracture energy

RESIDUAL STRAIN ENERGY RATIO - BETA PARAMETER

$\beta_{t,11}$ = PROPS (46)

$\beta_{t,22}$ = PROPS (47)

$\beta_{t,33}$ = PROPS (48)

$\beta_{c,11}$ = PROPS (49)

$\beta_{c,22}$ = PROPS (50)

$\beta_{c,33}$ = PROPS (51)

$\beta_{v,2}$ = PROPS (52)

$\beta_{v,3}$ = PROPS (53)

$\beta_{t,23}$ = PROPS (54)

RESIDUAL STRENGTH RATIO - ALPHA PARAMETER

$\alpha_{t,11}$ = PROPS (55)

$\alpha_{t,22}$ = PROPS (56)

$\alpha_{t,33}$ = PROPS (57)

$\alpha_{c,11}$ = PROPS (58)

$\alpha_{c,22}$ = PROPS (59)

$\alpha_{c,33}$ = PROPS (60)

$\alpha_{v,2}$ = PROPS (61)

$\alpha_{v,3}$ = PROPS (62)

$\alpha_{t,23}$ = PROPS (63)

VISCOUS PARAMETER

$\gamma_{t,11}$ = PROPS (64)

$\gamma_{t,22}$ = PROPS (65)

$\gamma_{t,33}$ = PROPS (66)

$\gamma_{c,11}$ = PROPS (67)

$\gamma_{c,22}$ = PROPS (68)

$\gamma_{c,33}$ = PROPS (69)

$\gamma_{v,2}$ = PROPS (70)

$$\gamma_{v,3} = \text{PROPS (71)}$$

$$\gamma_{t,23} = \text{PROPS (72)}$$

The **PROP** values are properties of the unloaded timber and are considered to be the strength properties of the timber. Once the timber undergoes loading the cell structure of the timber begins to change and some of the strength properties begin to change. When these properties begin to change they are said to be damaged and must then be recalculated and run through the **UMAT** calculations again, which in turn then returns a new damaged variable and the process begins again. In the **UMAT** these parameters are referred to as **STATEV** - dependent state variables. In total for this research there are 35 dependent variables that have been activated in the model.

DEPENDENT STATE VARIABLES

$$d_{t,11,old} = \text{STATEV (1)}$$

$$d_{t,22,old} = \text{STATEV (2)}$$

$$\vdots$$

$$d_{v,23,old} = \text{STATEV (35)}$$

These damage parameters combine together to form a single damage parameter calculated in equation 7.6 to give a final d_i value for each increment of the analysis. As stated in section 7.3 d_i is then substituted into the Hooke's equation (equation 6.15) and thus the new σ can be calculated.

Chapter 8

Theoretical Calculations

8.1 Ultimate Moment Resistance - SANS 10163-1

The ultimate moment resistance (M_r) for a beam with no lateral support provided is given in SANS10163-1 section 13.6.

Test Setup, Beam Dimensions and Material Properties

Breadth (b) = 105 mm

Height (h) = 130 mm

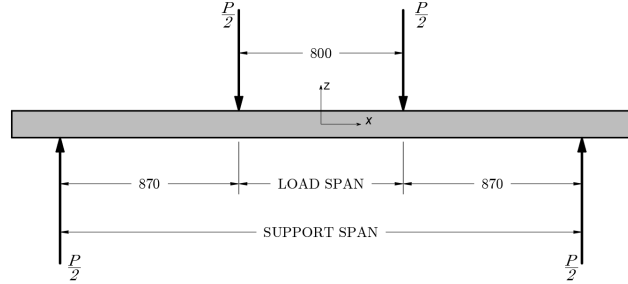
Length (L) = 2850 mm

Timber strength (f_b) = 11.5 MPa

Reinforcing yield strength (f_y) = 1770 MPa

Section Modulus (Z_e) = 295 750

Resistance Factor (φ) = 0.68



Slenderness Factor C_s

The slenderness factor, C_s , is given as:

$$C_s = \sqrt{\frac{L \cdot h}{(1 - 0.1125 \cdot \omega_3) \cdot b^2}}$$

Where ω_3 is extrapolated from table 6 in the code and is a positive value due to a destabilising load

$$\omega_3 = 2.185$$

Therefore:

$$\begin{aligned} C_s &= \sqrt{\frac{(2850) \cdot (130)}{(1 - 0.1125 \cdot 2.185) \cdot (105)^2}} \\ &= 6.68 \end{aligned}$$

Bending in Rectangular Sections

The formula for M_r depends on the the value of C_s and C_k .

When $0 < C_s \leq 10 \cdot M_r$ is given as:

$$M_r = \varphi \cdot Z_e \cdot \frac{f_b}{\gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{m3} \cdot \gamma_{m4} \cdot \gamma_{m5}}$$

Partial Material Factors

γ_{m1} is the material factor for load duration is calculated from section 13.1.2.1 - 13.1.2.5 and table 3 in the code:

$$\begin{aligned}\gamma_{m1} &= \frac{C_{fd} \cdot W_{DU} + C_{fi} \cdot W_{IU} + C_{fw} \cdot W_{WU}}{W_{DU} + W_{IU} + W_{WU}} \\ &= \frac{(1.26) \cdot (1.2) + (0.62) \cdot (1.6) + (0) \cdot (1)}{(1.6) + (1.2) + (1)} \\ &= 0.659\end{aligned}$$

γ_{m2} is the material factor for load sharing: This partial material is not applied to glulam beams according to section 13.1.2.2 (commentary)

γ_{m3} is the material factor for stressed volume: For beams, the γ_{m3} , is given by

$$\begin{aligned}\gamma_{m3} &= 0.85 + 0.03L \\ &= 0.85 + 0.03(2850) \\ &= 0.9355\end{aligned}$$

γ_{m4} is the material factor for moisture content: When the moisture content for a beam is below 200g/kg this factor is taken as 1

γ_{m5} is the material factor for pressure treatment: A treatment factor of 1.11 is applied to members that have been pressure treated

Resisting Moment M_r

$$\begin{aligned}M_r &= \varphi \cdot Z_e \cdot \frac{f_b}{\gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{m3} \cdot \gamma_{m4} \cdot \gamma_{m5}} \\ &= (0.68) \cdot (295750) \cdot \frac{11.5}{(0.659) \cdot (1) \cdot (0.9355) \cdot (1) \cdot (1.11)} \\ &= 3.38kNm\end{aligned}$$

The Resisting moment of the glulam beam, without reinforcement, is 3.38kNm

8.2 Theoretical Deflection calculations

The theoretical deflection calculations will use standard design values for the beam and will assume the glulam beam to have perfect glue bonds. This reduces the variables required and allows for the industry standard beam to be calculated for. A more accurate approach can be undertaken using the initial non-destructive testing, as mentioned in section 3.2, and equation 2.9 (or similar depending on the machine used). This method would involve finding the E_{dyn} for each glulam lamina that composes the final beam and then determining the beams overall EI with equation 8.1

8.2.1 Transformed sections

In this research the beams are to be reinforced with high tensile steel wire of diameter ($\varnothing_d$) 3, 5 and 8mm. The steel wires will be placed at different distances across the width of the beam in order to achieve different quantities of steel reinforcing (A_s). The recommended distance for spacing is $>4 \cdot \varnothing_d$ to ensure there is adequate bond between the wooden surfaces, however, a variety of distances will be tested.

For the purpose of the theoretical calculations the steel will be converted to an equivalent area of wood, seen in figure 8.1 in order to assume a homogeneous material, a similar approach is used for reinforced concrete. The PROLAM model in section 6.2, Sardiko et al. (2017) and Yang et al. (2016) all use a similar approach for the modelling of glulam. This allows the distance to the neutral axis, x , and the moment of inertia (I_i) to be determined.

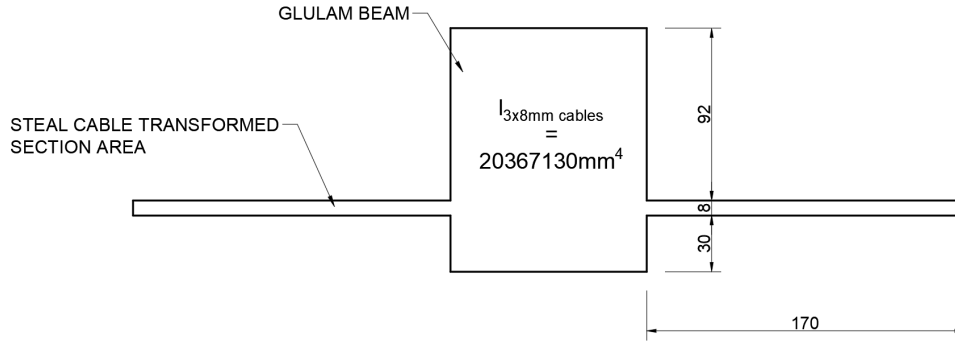


Figure 8.1: Reinforced glulam beam transformed section

$$E_{dyn}I = \sum E_i \cdot (I_i + A_i \cdot d_i^2) \quad (8.1)$$

Using either the values obtained from design standards or those from the non-destructive testing it is possible to calculate a theoretical deflection, δ_{theo} , using equation 8.2

$$\delta_{theo} = \frac{P \cdot a}{48EI} \cdot (3L^2 - 4a^2) \quad (8.2)$$

Where:

- P is the applied load (N)
- L is the beam span (mm)
- a is the distance between the support and loading points (mm)

In practice a strain gauge or dial gauge is used to take readings of the beam deflections. SANS 6122 clause 5.5.3.6 states that a pre-load of 200N must be applied to the beam before initial deflection readings are taken. This is to allow for settling at the supports which could reduce the accuracy of the readings. If a manual dial gauge is used, a minimum of five readings per beam must be taken, after which the apparatus may be removed and the beam may be tested until failure.

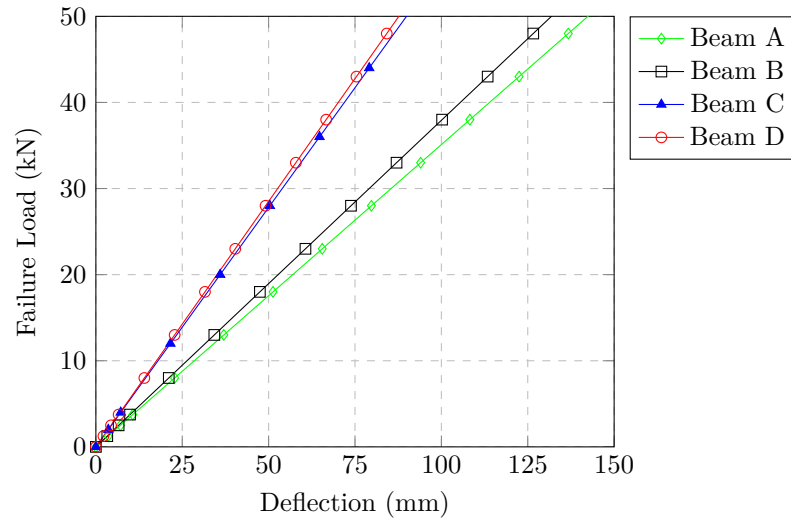
Table 8.1 gives a brief description of each beams reinforcing configuration and graph 8.1 shows the theoretical deflection results of the unreinforced and reinforced beams. As the area of steel in the tension zone increases, the expected deflections decrease. It is important not to place steel too close to the edge

Table 8.1: Beam names and reinforcing configurations

Beam	No. of Steel Bars	Bar Diameter (mm)	Reinforcing Cross Sectional Area (mm ²)
A	-	-	-
B	5	3	35.34
C	4	5	98.17
D	3	8	150.80

of the beams as this may cause internal shear forces which may lead to the timber splintering at the edges. It can be seen that by adding more steel reinforcing rods, and increasing the area of steel, the deflections begin to get smaller.

Graph 8.1: Theoretical deflections using equation 8.2



Chapter 9

Methodology

9.1 Introduction

Beam specimens were prepared, where possible, using the manufacturing standards currently in place in the South African glulam industry. This includes sizing, glue type, processes and seasoning. The beams were made by OT Joiners, situated in Cape Town, South Africa. The testing procedures adopted for the research are from SANS 6122, and where ambiguities were encountered ASTM D4761 and ASTM D198 were consulted for clarification.

This chapter will focus on the beam production, small scale test setup for bond strength, beam test procedure and the model setup in Abaqus FEA software with the inclusion of the UMAT code.

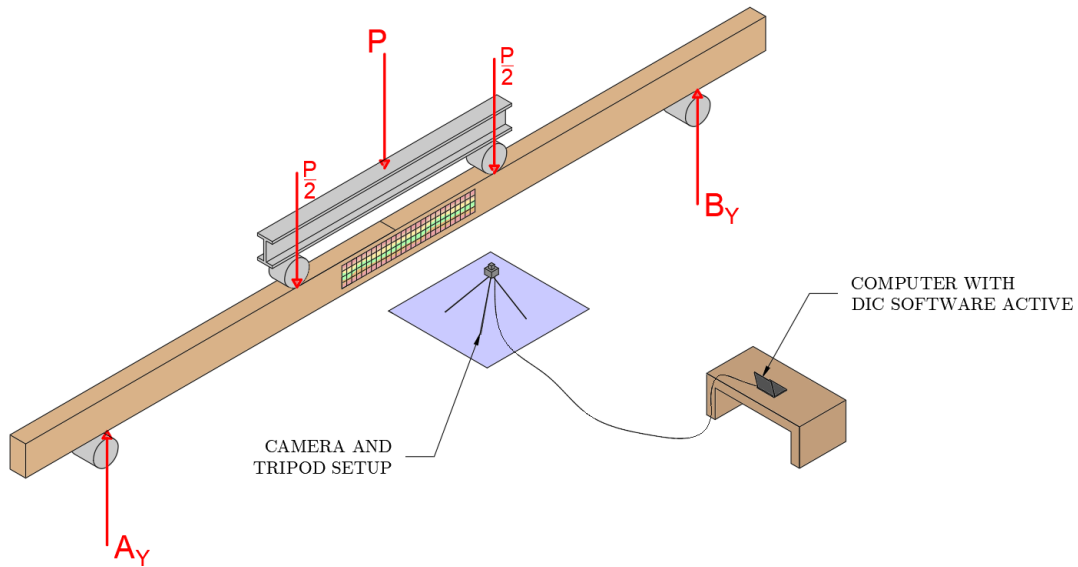


Figure 9.1: Experimental beam and camera setup

9.2 Experimental tests

All testing was conducted at the University of Cape Town (UCT) with the appropriate calibrated testing machines. A requirement of the University is that any physical tests require a Job Safety Analysis (JSA) which will highlight any expected dangers and the mitigation measures that will be taken. The JSA documents can be found in appendix C. The experimental side of the research covers thirty-five (35) pull out glue tests, and twenty (20) full scale beam tests.

9.2.1 Pull-out Glue Tests

As discussed previously in section 2.9 the bond strength between the steel and the timber is an important factor in the effectiveness of the reinforcing. The literature suggests the use of an EPO glue for the best bond strength between the two materials; however, in the South African glulam production mills an MUF or PRF glue is used. Therefore a pull out test was conducted using the different available glue types to determine whether the MUF or PRF glue is sufficient according to the literature. Sardiko et al. (2017) and De Luca and Marano (2012) both suggest that the MUF or PRF glue strength should be 60% of the EPO glue in order for the MUF or PRF glue to be used as the only adhesive throughout the reinforced

beam production. Achieving 60% strength of the EPO allows for the timber-timber connection, as well as the timber-steel connection, to be simplified in the modelling process as a perfect connection.

Section 2.9.2 discusses the type of grooves that would commonly be used or this type of application. In the tests conducted for this research three groove types were tested, each using both glue types:

1. A single sided groove that is slightly smaller than the cable to allow for a close bond between steel and timber.
2. A single sided groove that; is the same size as the cable in width, and slightly smaller in depth to allow the timber to crush, and mould around the cable.
3. A double sided groove that suits the literature found in section 2.9.2.

Sample Preparation

The samples were prepared in such a way as to try mitigate as many variables as possible. This was done by using a single 6 meter span of timber that was cut into five 1.2m spans. The spans were then labelled and cut according to the schematic shown in figure 9.2. This figure only shows two of the five beams however, the labelling is consistent throughout until on the 5th span a final piece, F52, is acquired. After testing the initial samples, A-F, another set, G, was made using the strongest groove type and the EPO glue for comparison.

The cut blocks were then grooved using a saw blade of width 2.4mm. The different groove types are given in figure 9.3 where blocks A and B shared the same groove with differing glue types, blocks C and D shared the same groove with differing glue types, and the same with blocks E and F.

Once grooved, the blocks were glued and clamped together according to figure 9.2. Where A11 is glued to A12, following this numbering system, block F51 is glued to F52.

The reasoning behind this type of numbering system was to achieve an even distribution of blocks along the original 6m span. The distribution of A blocks can be seen in figure 9.4.

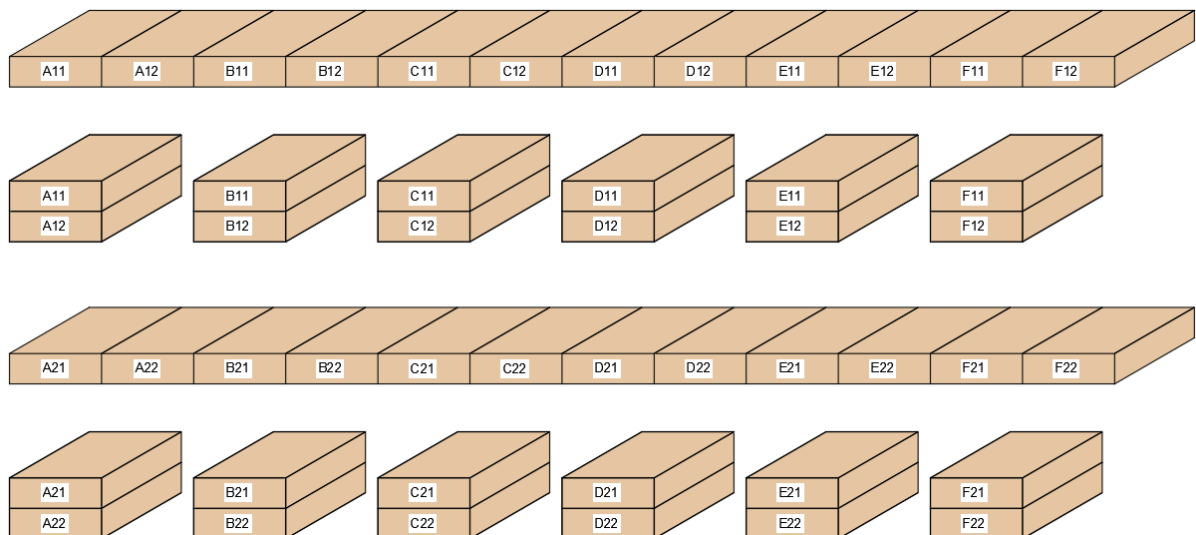


Figure 9.2: Process by which the pull out tests were laminated

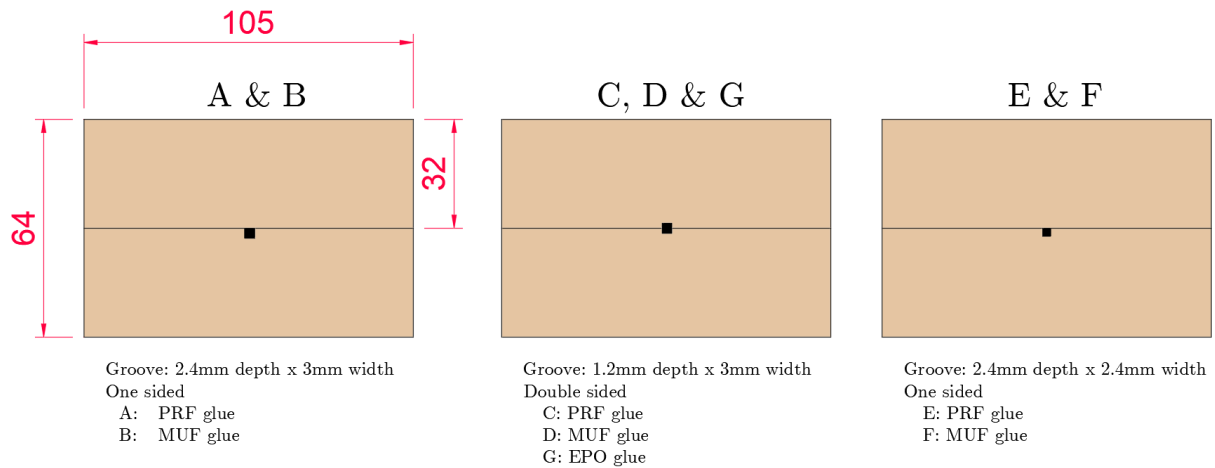


Figure 9.3: Three groove types used for the pull out tests



Figure 9.4: Distribution of A blocks in a 6m span

The PRF glue used was a mixture of lambond resin and hardener mixed to the supplier specifications. The resin to hardener ratio was 5:1 and the mixture was hand mixed for 5 minutes. A pre-curing time of 5 minutes was allowed to ensure the glue would stick to the cables correctly.

The MUF glue used was a mixture of Chemres CR MUF30 with hardener CR MXF30. The resin to hardener ratio was 10:1 and the mixture was hand mixed for 3 minutes. A pre-curing time of 5 minutes was used once again.

The glue was applied, the 3mm cable was cleaned with a degreasing agent and inserted into the groove, the blocks were clamped together using mechanical clamps. After 15 minutes the clamps were re-torqued to ensure a tight bond. It was assumed that there was sufficient clamping pressure, as evidenced by the glue bleed-out on the specimens, which occurred at a clamping force of 0.7-1.5 MPa. The finished specimens were cured in ambient conditions for a minimum of 7 days.

Test Procedure

Pull-out tests were conducted in the same manner that the bond strength between rebar and concrete is determined as shown in figure 9.5. The timber blocks were placed in between two steel base plates, one of which has a hole in the middle for the reinforcing wire to pass through. The base plates are secured in place with bolts and the wire is placed in a grip. Once everything is secured and in place, a tensile load, P , is applied and gradually increased at a rate of 50N/s until the reinforcing wire slips free from the timber block.

The test is repeated for blocks A through G and the results are shown in table 10.1. The strongest glue and groove combination will be used in the final beam layup.

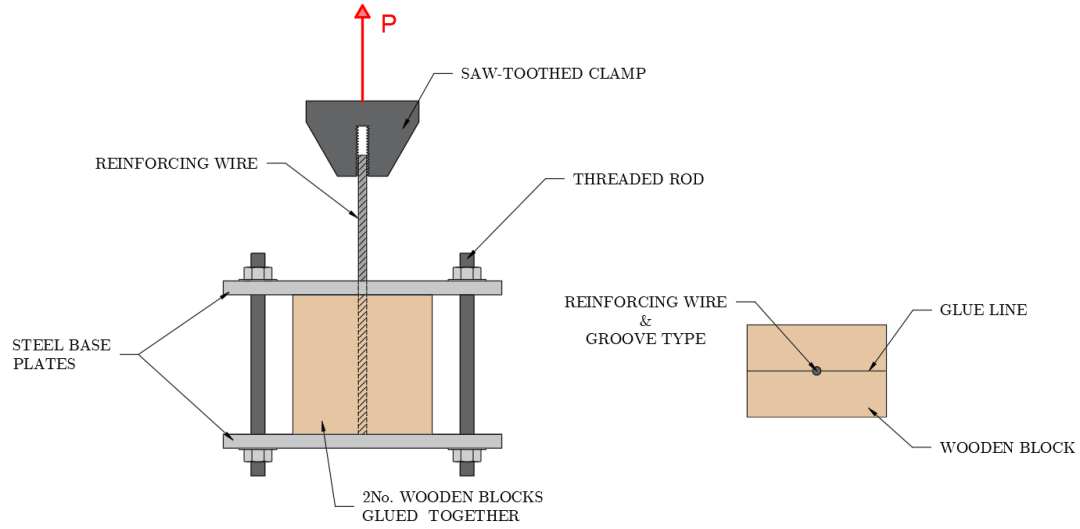


Figure 9.5: Pull out test setup

9.2.2 Beam Tests

Beams were tested in flexure until failure and compared with results obtained from the CDM computational model. The changing variable in the experimental tests will be the Area of Steel (A_s). This is achieved by changing the bar diameter and the number of bars placed within the laminae.

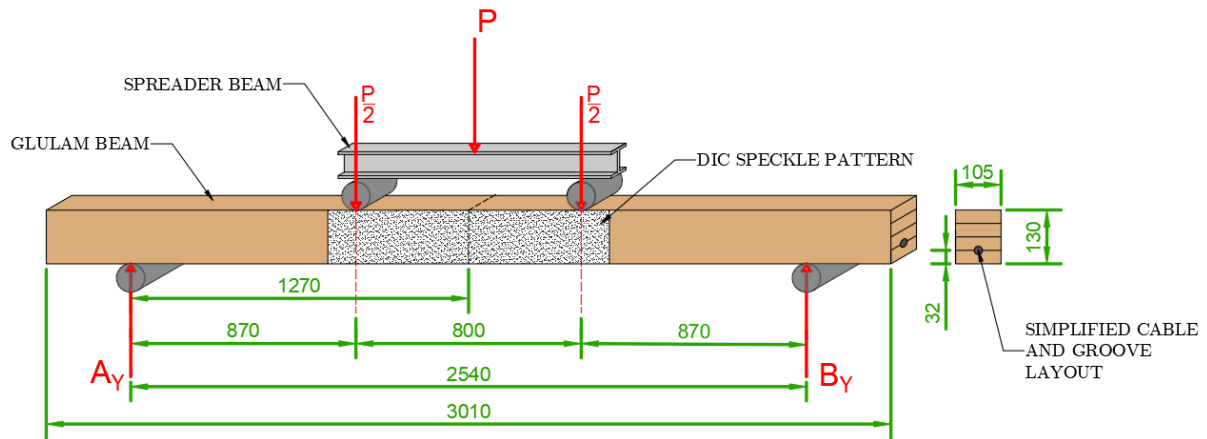


Figure 9.6: Experimental beam dimensions

Sample Preparation

Glulam beams were produced using grade 5 Pinus Patula timber, four laminates each with cross sectional area measuring roughly 32x105mm, an MUF glue and high tensile steel cables with diameter ($\varnothing$) 3, 5 and 8mm, the dimensions for the lower two (2) laminates and grooves can be seen in figure 9.7

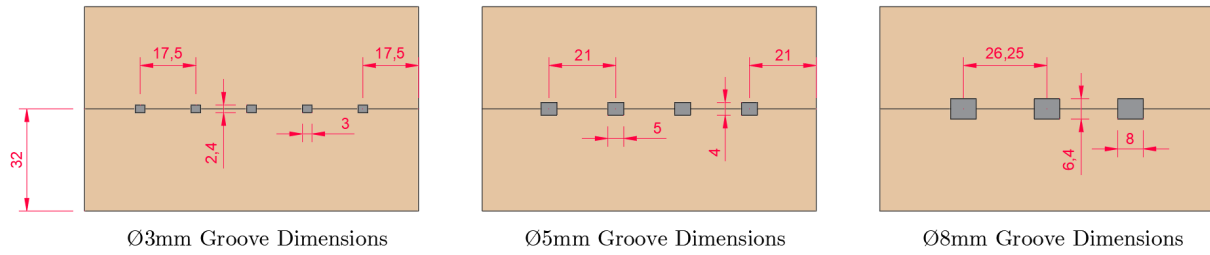


Figure 9.7: Groove dimensions for beams B, C and D

The beams were assembled following standard South African glulam factory guidelines and procedures as closely as possible. Two additional steps were added into the production process, where the third and fourth laminates were taken off the assembly line and grooved, before being placed back in the production line. The second additional step, was where the steel cables are placed within the grooves and pulled tight¹ as shown in figure 9.8a. The beams were then placed in a 50T compressed air clamping system for 16 hours until the MUF glue had cured, shown in figure 9.8b.



(a) Ø 8mm Cables lying in the grooves



(b) Beams in the 50T press while curing

Figure 9.8: Beam preparation at OT Joiners factory

The twenty (20) beams were delivered to the testing laboratory where they were stored for 8 weeks in order to acclimatise to ambient conditions and ensure moisture content was between 10-20%. Once acclimatised the beams were prepped for painting, required for the use of the DIC software.

The centre 1 meter portion of the beam was sanded on one side and painted with two coats of a white acrylic paint. A black speckle pattern was painted over the white paint using an epoxy spike roller with its tips dipped in black paint. The roller was then moved in horizontal and vertical paths across the white painted surface to create a high contrast speckle. The painting process and resulting speckle pattern on a beam can be seen in figure 9.9.

¹The steel cables were pulled tight using a handmade jig and minimal tensile force to ensure there was no prestressing force induced into the beam.

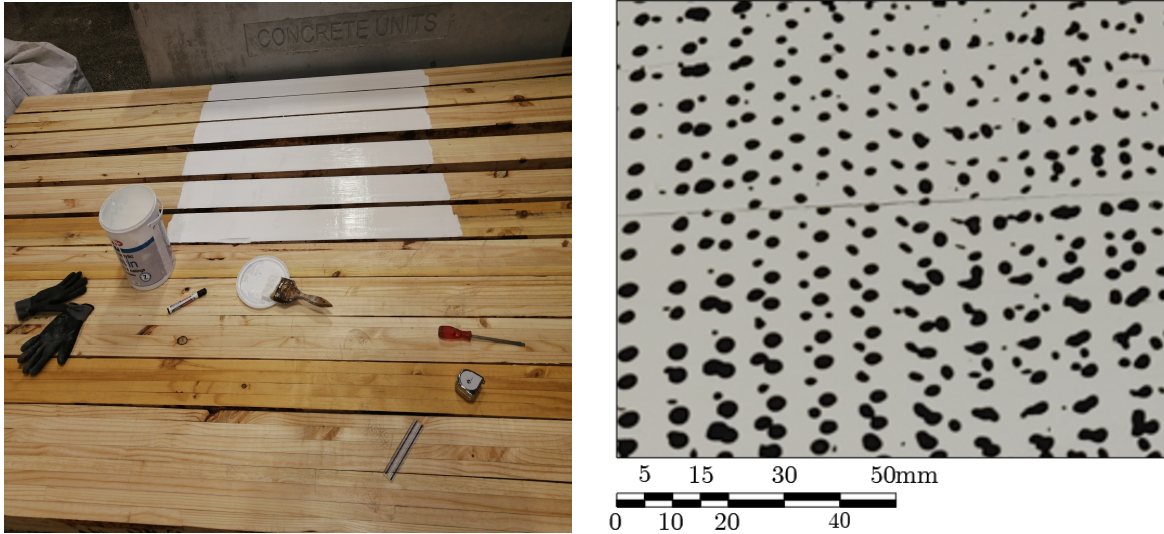


Figure 9.9: The painting process and resulting speckle pattern achieved

Test Procedure

After painting and drying, the beams were loaded onto the test assembly with the speckle pattern facing the DIC camera setup. A steel spreader beam, weighing 40kgs, was placed on the timber and this accounted for the preload required by SANS 6122 clause 5.5.3.6 The test assembly consisted of a 230kN Instron actuator connected to a non-sway steel frame. This setup can be seen in figure 9.10.

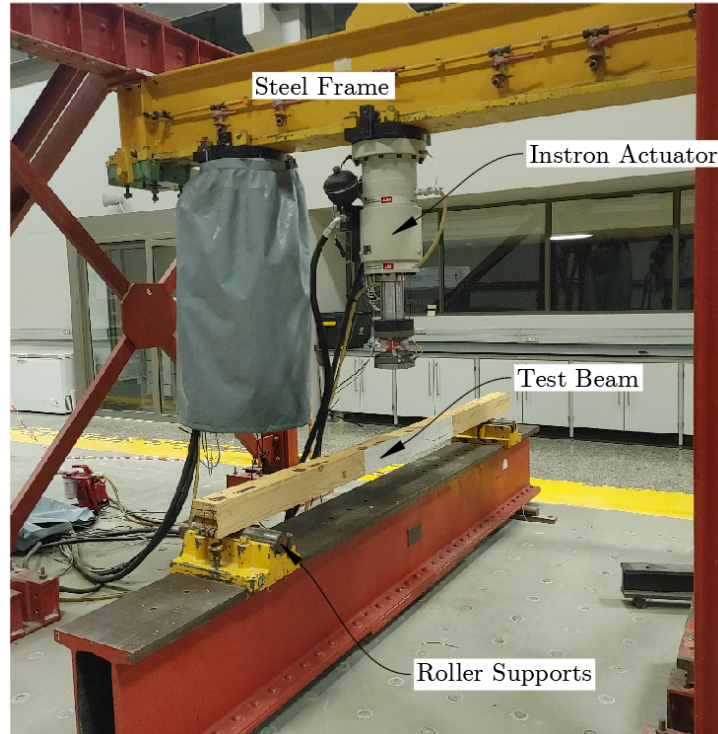


Figure 9.10: Experimental beam test assembly in the lab

The glulam beam rested on two roller supports, covered with a bearing pad, which allowed for horizontal slippage during the bending test. Due to the relatively small loads in the experiment, the bearing pads were assumed to be incompressible.

Once the beam was in place, the DIC camera was setup on a tripod at a distance that allowed the entire painted section of the beam to be visible within the captured images. Calibration of the DIC camera could then commence. This was done by following the steps outlined in section 5.4.1, and ensuring there were no shadows cast onto the beam, with the use of spotlights.

The image capture rate in Pylon Viewer was set to 500 milliseconds resulting in two (2) DIC images/readings every second. The Instron actuator was displacement controlled, as opposed to load controlled, meaning the actuator could move up or down a specified distance within a given amount of time. This allowed the loading rate of the beams to be set at 0.2mm/s. This load rate allowed for minimal shock loading to occur and for the average test duration being 4-5 minutes long. This resulted in an average of 500 DIC images being captured per beam for post-processing.

The load from the Instron actuator is passed through a spreader beam of length 800mm in order to run a 4-point flexural test. The beam was said to have failed once a significant crack formed that resulted in a large decrease in applied load. Smaller cracking noises were heard during the testing and small reductions in load were observed at those instances, however, load recovery was observed and the test was allowed to continue.

Four (4) beam configurations were tested and the results are presented in chapter 10. The beam layups are as follows:

- BEAM A1-A5: Control beam with no reinforcing steel, and dimensions 130x105x3000mm, consisting of 4 laminations.
- BEAM B1-B5: consists of five (5) high tensile steel cables $\varnothing$ 3mm
- BEAM C1-C5: consists of four (4) high tensile steel cables $\varnothing$ 5mm
- BEAM D1-D5: consists of three (3) high tensile steel cables $\varnothing$ 8mm

After the test was complete the images were transferred from Pylon Viewer into the Istra4D software for post processing and data extraction. This was done following the steps outlined in section 5.4.2 The analog voltage signal from the Instron actuator was synchronised with the captured images and this was used to extract the corresponding load vs. deflection and load vs. strain curves for each beam. The graphs are presented as graphs 10.2 - 10.6 and graphs 10.8 - 10.11 respectively.

9.3 Abaqus CDM Experiments

In this study, a 4 point bending test was conducted on four timber beams using Abaqus CAE software and a UMAT to simulate the non-linear behaviour of the beam material. The UMAT was used to describe the constitutive behaviour of the beams, which was made of four timber laminates each. In order to save computational time, the beams were modelled as solids rather than individual layers.

The control beam has no reinforcing cables, while the other three beams have progressively larger cable diameters. All four beam types were analysed using the 4 point bending test to determine the effect

of the reinforcing cables on the beam's strength and stiffness. This setup of the Abaqus experiments matched that of the experimental tests found in 9.2.2

The results of this study are used to better understand the non-linear behaviour of timber beams under 4 point bending, and to explore the potential benefits of using reinforcing cables in timber beam construction. By accurately simulating the constitutive behaviour of the beam material using a UMAT, it will be possible to predict the performance of different beam configurations and make informed design decisions.

9.3.1 Test Setup:

The beam will be modelled as a 3D solid in Abaqus CAE with dimensions and boundary conditions based on standard 4 point bending test specifications. The supports were fixed in all directions, and the load selected was pressure load applied along two solid steel sections placed on the top surface of the beam. The load increased incrementally and the deflection of the beam at the midpoint of the bottom surface was recorded at each load step.

Usually, the material properties in Abaqus are assigned in the material module within the Abaqus menus. Using a UMAT is slightly different, the steps are listed here:

1. Go to the **Material** module
2. Click **General > User Material**
3. Now add in the values for the PROPS from section 7.5.
4. Now to add the solution dependent variables click **General > Depvar**
5. With the scroll arrows, increase to the required number of state variables, in this case 35.

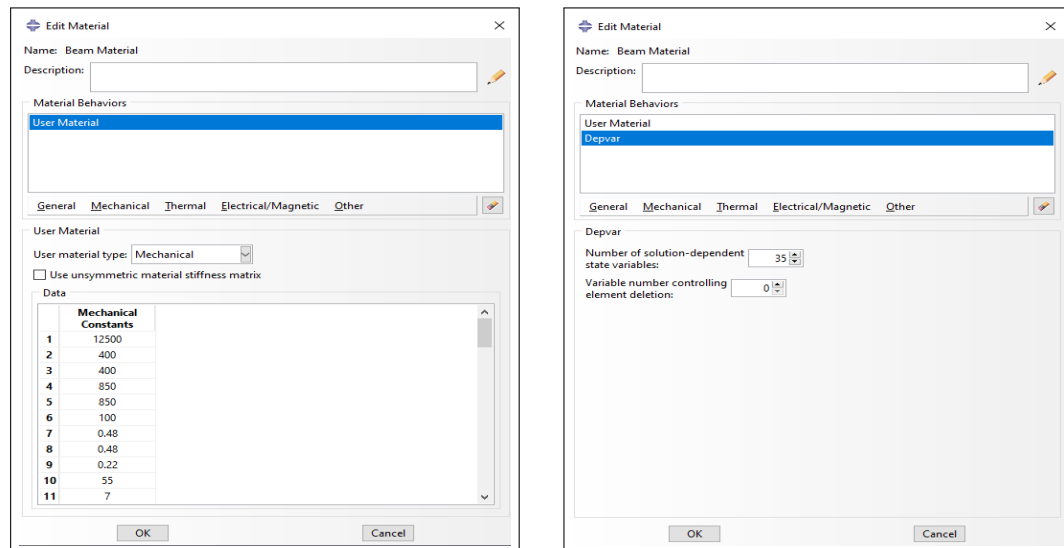


Figure 9.11: Selecting the material constants for a UMAT in Abaqus

The load was applied in the form of a distributed pressure load over a small area at the locations of two steel sections on the top surface of the beam. This load application is chosen to approximate

the behaviour that would be expected in the experimental tests, where the steel section would compress slightly into the timber beam and not apply a direct line load at the point of contact.

The boundary conditions for the beam analysis will be established utilizing the constraint options available in Abaqus. In order to accurately reflect the experimental setup, the supports were modeled as roller supports. The experimental beams were not laterally restrained and were permitted out-of-plane sliding, while the Abaqus beams were not modeled in such a manner. Instead, a pure roller support was utilized which only allowed horizontal sliding as necessary.

In order to accurately capture the non-linear behaviour of the timber beam, the beam was meshed with non-linear, 3D continuum elements. The selected element type depends on the desired level of accuracy and computational efficiency. Some commonly used elements for non-linear analysis of solids in Abaqus include the C3D8R, C3D8, and C3D4. The appropriate element type is selected based on the size and complexity of the model, as well as the desired level of accuracy. For this study a C3D8R element was used. A C3D8R element is described as an 8-node linear brick with reduced integration and hourglass control.

The beam model was generated and assembled using the Abaqus preprocessing tools. The beam dimensions, boundary conditions, and load application were defined in the respective modules in Abaqus. The model was then meshed, with the element type and size chosen based on the desired level of accuracy and computational efficiency. As stated in section 7.4.1, the work of Sandhaas (2012) suggests a mesh size 15mm to be the recommended size to balance accuracy and efficiency. The final model in Abaqus is shown in figure 9.12

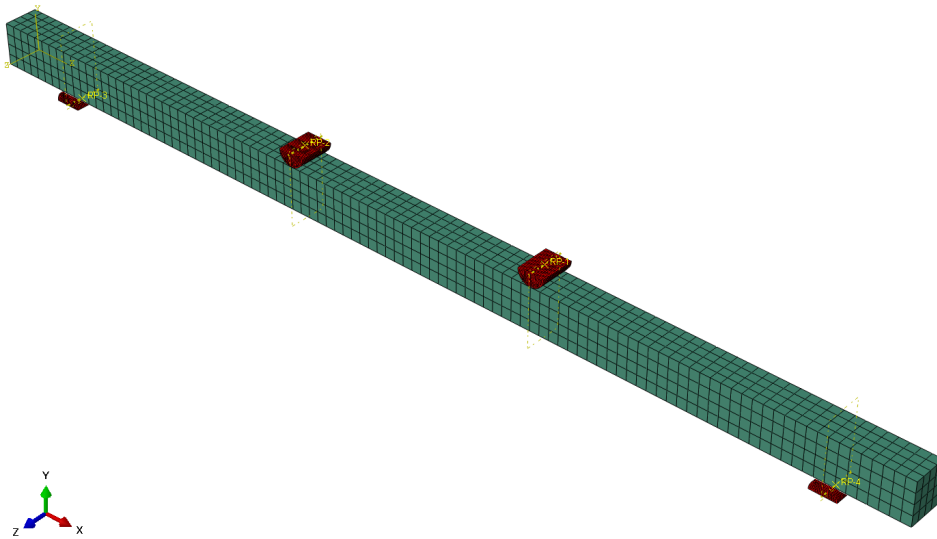


Figure 9.12: Model Generated In Abaqus CAE For a Bending Test

In order to load a **UMAT** into an Abaqus simulation, the code needs to be written in Fortran code and activated through the Abaqus Job Module. Before doing this it is important to ensure that Abaqus and Fortran code compilers are linked correctly. For information on how to link the programs, and how to check that the programs are compatible and working together, see Appendix A. To select a **UMAT** code for a simulation the following extra steps are required when creating a job:

1. Click on the **Job** module in Abaqus

2. Create a job file as usual
3. In the pop-up window, open the **General** tab
4. At the bottom of the window, select the path for the required UMAT, as shown in figure 9.13.

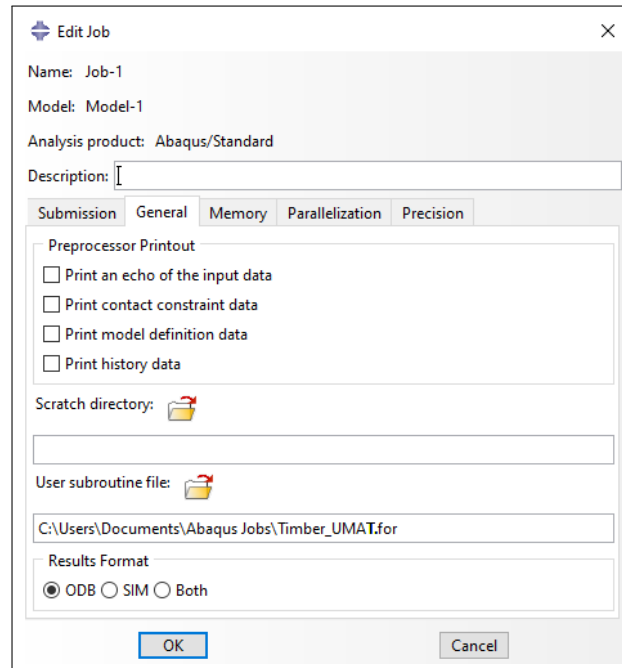


Figure 9.13: Selecting a UMAT File In Abaqus

9.3.2 Material Properties:

The material properties used in the UMAT code were obtained from literature. Where values specific to Pinus Patula can not be found, an average value from different tree species are used. Further studies would need to be conducted on Pinus Patula specifically to make the UMAT as accurate as possible for the species. Material properties are labelled in the UMAT as PROPS and are defined in section 7.5. The value of each of these properties is listed in table 9.1:

9.3.3 Model Calibration

The calibration of the developed UMAT code for timber is crucial to ensure that the computational simulations will produce accurate and reliable results. In this thesis, the calibration process will involve comparing the simulation results with experimental data obtained from literature. Specifically, three types of calibration experiments were conducted: compression parallel and perpendicular to the grain, as well as tension parallel to the grain test. These experiments will provide a comprehensive evaluation of the model's performance under various loading conditions. The results of calibration will then be used to assess the model's suitability for use in a bending test and to produce realistic data for the simulation of timber structures.

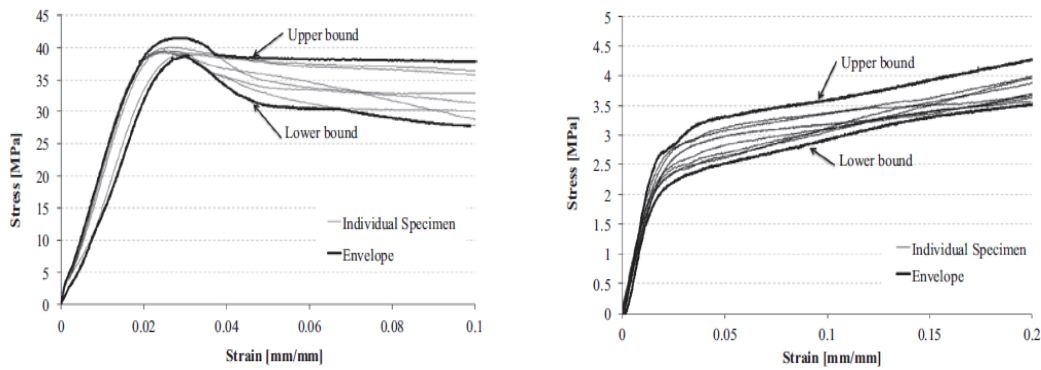
Table 9.1: Material Property Assigned Values For Pinus Patula

Material Property Name	Symbol	Assigned Value	Material Property Name	Symbol	Assigned Value
PROPS (1)	E_{11}	7800	PROPS (37)	$Gf_{t,11}$	6
PROPS (2)	E_{22}	115	PROPS (38)	$Gf_{t,22}$	14
PROPS (3)	E_{33}	115	PROPS (39)	$Gf_{t,33}$	14
PROPS (4)	G_{12}	490	PROPS (40)	$Gf_{c,11}$	36
PROPS (5)	G_{13}	490	PROPS (41)	$Gf_{c,22}$	24
PROPS (6)	G_{23}	50	PROPS (42)	$Gf_{c,33}$	24
PROPS (7)	v_{12}	0.48	PROPS (43)	$Gf_{v,2}$	8
PROPS (8)	v_{13}	0.48	PROPS (44)	$Gf_{v,3}$	8
PROPS (9)	v_{23}	0.2	PROPS (45)	$Gf_{v,23}$	101
PROPS (10)	$\sigma_{t,11}$	67	PROPS (46)	$\beta_{t,11}$	0.1
PROPS (11)	$\sigma_{t,22}$	36	PROPS (47)	$\beta_{t,22}$	0.1
PROPS (12)	$\sigma_{t,33}$	36	PROPS (48)	$\beta_{t,33}$	0.1
PROPS (13)	$\sigma_{c,11}$	18	PROPS (49)	$\beta_{c,11}$	0.1
PROPS (14)	$\sigma_{c,22}$	47	PROPS (50)	$\beta_{c,22}$	0.1
PROPS (15)	$\sigma_{c,33}$	47	PROPS (51)	$\beta_{c,33}$	0.1
PROPS (16)	$\sigma_{v,2}$	16	PROPS (52)	$\beta_{v,2}$	0.1
PROPS (17)	$\sigma_{v,3}$	16	PROPS (53)	$\beta_{v,3}$	0.1
PROPS (18)	$\sigma_{v,23}$	36	PROPS (54)	$\beta_{v,23}$	0.1
PROPS (19)	$\epsilon_{t,11}$	0.00218	PROPS (55)	$\alpha_{t,11}$	0.1
PROPS (20)	$\epsilon_{t,22}$	0.270	PROPS (56)	$\alpha_{t,22}$	0.1
PROPS (21)	$\epsilon_{t,33}$	0.270	PROPS (57)	$\alpha_{t,33}$	0.1
PROPS (22)	$\epsilon_{c,11}$	1	PROPS (58)	$\alpha_{c,11}$	0.1
PROPS (23)	$\epsilon_{c,22}$	0.4	PROPS (59)	$\alpha_{c,22}$	0.1
PROPS (24)	$\epsilon_{c,33}$	0.4	PROPS (60)	$\alpha_{c,33}$	0.1
PROPS (25)	$\epsilon_{v,2}$	0.145	PROPS (61)	$\alpha_{v,2}$	0.1
PROPS (26)	$\epsilon_{v,3}$	0.145	PROPS (62)	$\alpha_{v,3}$	0.1
PROPS (27)	$\epsilon_{v,23}$	2	PROPS (63)	$\alpha_{t,23}$	0.1
PROPS (28)	$Gu_{t,11}$	0.026	PROPS (64)	$\gamma_{t,11}$	0.001
PROPS (29)	$Gu_{t,22}$	13.5	PROPS (65)	$\gamma_{t,22}$	0.001
PROPS (30)	$Gu_{t,33}$	13.5	PROPS (66)	$\gamma_{t,33}$	0.001
PROPS (31)	$Gu_{c,11}$	35	PROPS (67)	$\gamma_{c,11}$	0.001
PROPS (32)	$Gu_{c,22}$	23.2	PROPS (68)	$\gamma_{c,22}$	0.001
PROPS (33)	$Gu_{c,33}$	23.2	PROPS (69)	$\gamma_{c,33}$	0.001
PROPS (34)	$Gu_{v,2}$	7.25	PROPS (70)	$\gamma_{v,2}$	0.001
PROPS (35)	$Gu_{v,3}$	7.25	PROPS (71)	$\gamma_{v,3}$	0.001
PROPS (36)	$Gu_{v,23}$	100	PROPS (72)	$\gamma_{t,23}$	0.001

Compression Tests

The experimental data used for the UMAT calibration in compression is taken from Karagiannis et al. (2016). The first test was a set of eight Scandinavian Spruce cubes measuring 45mm x 45mm x 45mm that were loaded parallel to the grain at a rate of 1mm/min. Karagiannis et al. (2016) measured the data to within an accuracy of ± 0.1 kN. The stress-strain data was recorded and is presented in figure 9.14a

Another experimental test was conducted on nine wooden blocks measuring 45mm x 70mm x 90mm. These were loaded in compression perpendicular to the grain. As with the previous tests, the load was increased at a rate of 1mm/min and the results were recorded within an accuracy of ± 0.1 kN. The stress-strain data was recorded and is presented in figure 9.14b



(a) Parallel Compression - Experimental Results (b) Perp. Compression - Experimental Results

Figure 9.14: Compression Experimental Results Taken From (Karagiannis et al., 2016)

The experimental tests from (Karagiannis et al., 2016) were replicated in Abaqus FEA software with the assigned material properties coming from the UMAT used in this study. The test setups are shown in figure 9.15 which shows the dimensions, grain direction and the applied load. The values for PROPS 1-72 were adjusted as necessary to match that of the Scandinavian Spruce species, the values were taken from Eslami et al. (2021) and for PROPS values where no additional data was found on Scandinavian Spruce, values for Pinus Patula were used.

The results of FEA compression tests were compared to the experimental compression tests and are presented in graphs 9.1 and 9.2. The parallel compression tests show an initial linear elastic behaviour until the compression limit is reached, followed by a plastic stage as predicted in the damage evolution model for ductile failures presented in figure 7.2a. The perpendicular compression tests show a similar linear elastic behaviour, now followed by a plastic hardening stage. Both FEA results correlate well within the upper and lower bounds of the experimental results from Karagiannis et al. (2016), and the stress-strain graphs show similar trends. This suggests that the UMAT can accurately predict the failure of timber in both parallel and perpendicular compression.

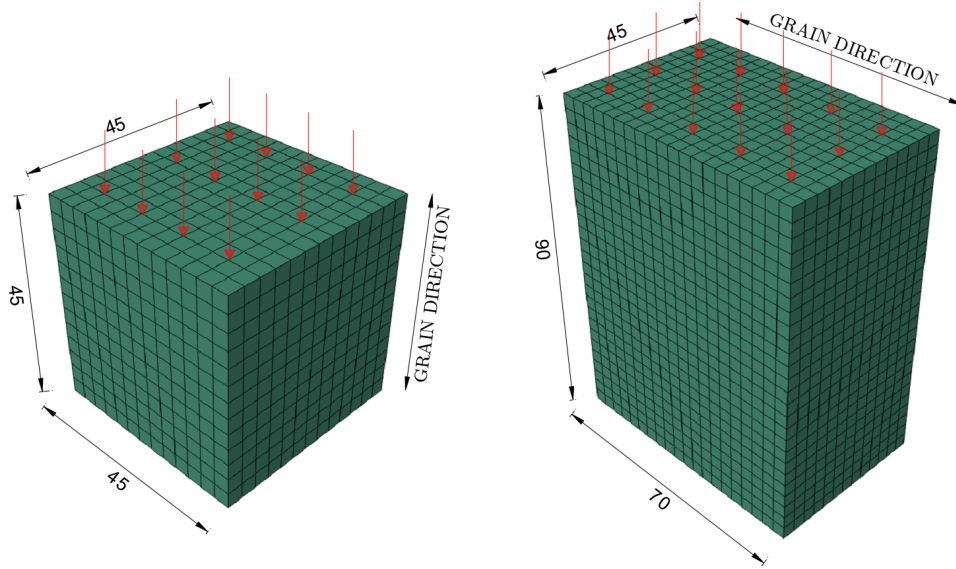
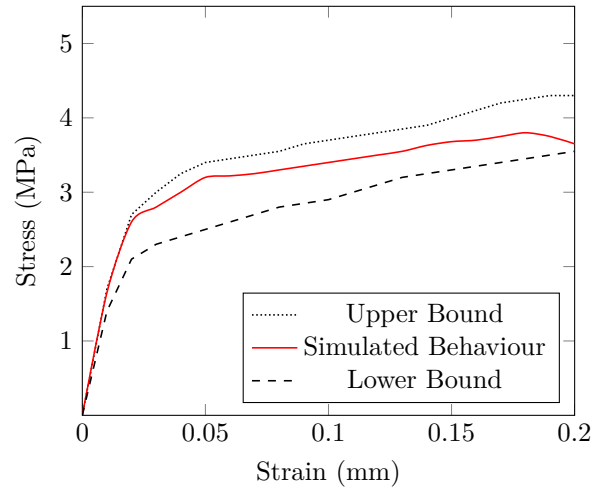
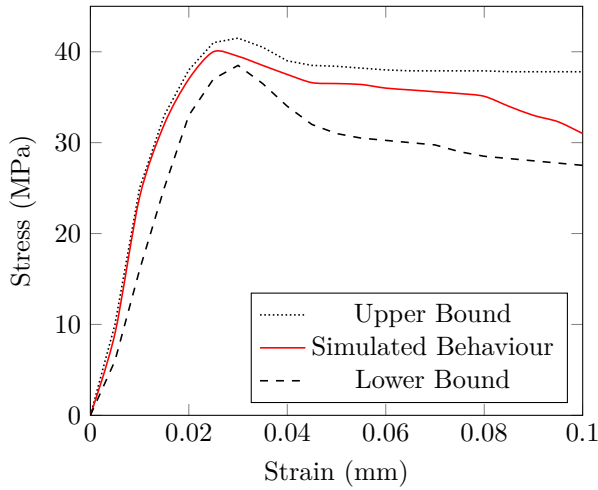


Figure 9.15: Model calibration: Compression Tests Comparison

Graph 9.1: Calibration - Parallel Compression

Graph 9.2: Calibration - Perpendicular Compression



Tension Tests

To calibrate the accuracy of the UMAT, tension parallel to the grain needs to be accurate as this is the most significant stress experienced by a beam in bending. The experimental data used for tensile calibration is taken from Khennane et al. (2014). The test was performed on a Pine specimen measuring 40mm x 10mm x 2mm that was loaded parallel to the grain at a rate of 1mm/min and the load vs. displacement data was recorded.

The experimental test described by Khennane et al. (2014) was replicated using the Abaqus FEA software. The test setup was designed to reflect the dimensions, grain direction, and applied load as shown in figure 9.16. In order to accurately simulate the behavior of the Pine species, the values for the material properties, represented as PROPS 1-72, are taken from data provided by Khennane et al. (2014). However, for those values where no additional data was found, values used for Pinus Patula, another

species of pine, were used instead.

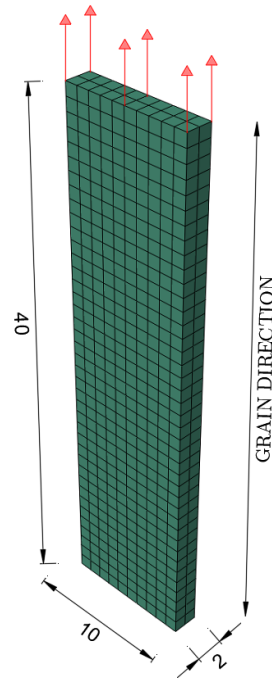
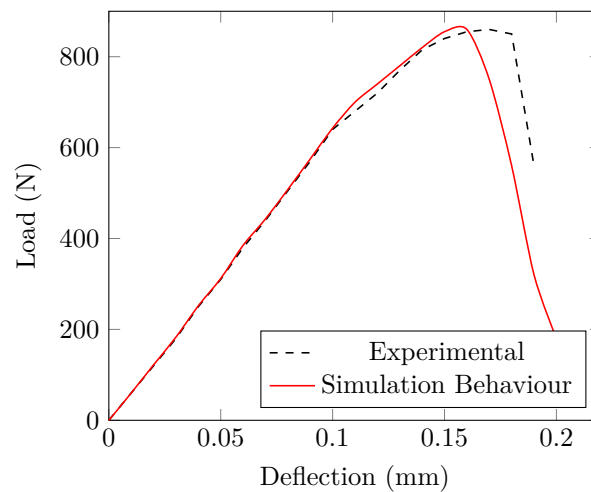


Figure 9.16: Model Calibration - Tension Test

The results of FEA tension test was compared to the experimental tension test and are presented in graphs 9.3. The parallel tension test shows an initial linear deformation until which point the yield strength is reached, followed by a sudden brittle failure as predicted in the damage evolution model for ductile failures presented in figure 7.2b. The FEM model predicts, with a good accuracy, both the failure load and failure mechanism of a timber specimen load in tension parallel to the grain.

Graph 9.3: Calibration - Parallel Tension



The use of the UMAT in simulating a 4 point bending test is a reliable and accurate approach, as it has been validated through multiple tests and showed to model the mechanical properties of timber in

parallel compression and tension, as well as perpendicular compression.

9.3.4 Data Analysis:

The recorded deflection and applied load data was used to calculate the bending moment and stress at the midpoint of the beam cross section. The results of the simulation can be compared to experimental measurements or other computational results to validate the accuracy of the model. The sensitivity of the results to changes in the material properties and meshing parameters can also be evaluated to ensure the robustness of the model.

9.3.5 Conclusion:

The 4 point bending test using Abaqus CAE and a UMAT code allows for the accurate simulation of the mechanical behaviour of timber beams under applied loads. The resulting data can be used to determine the material's yield strength and modulus of elasticity, providing valuable information for engineering design and analysis. By using nonlinear, 3D continuum elements and a user material subroutine, the behaviour of the timber beam can be accurately captured in the simulation. The load application, boundary conditions, and model generation and assembly are all important considerations in the setup and execution of testing while using Abaqus CAE.

Chapter 10

Results

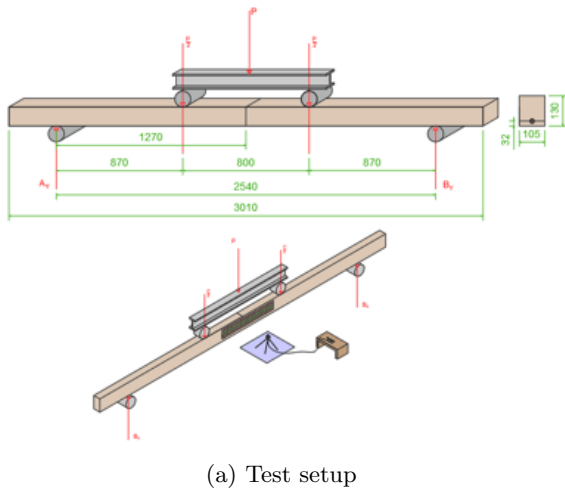
10.1 Introduction

In this section the experimental, DIC and Abaqus data will be presented. For the experimental and DIC data the procedure described in section 9.2.2 was carried out on glulam beams with dimensions 105 x 130 x 3000mm, each with differing sizings of reinforcing cables. The Abaqus data is presented and compared to the experimental results. The comparison between the two sets of data was used to determine whether the **UMAT** code implemented into the Abaqus model is sufficient to model glulam beams in 4 point bending.

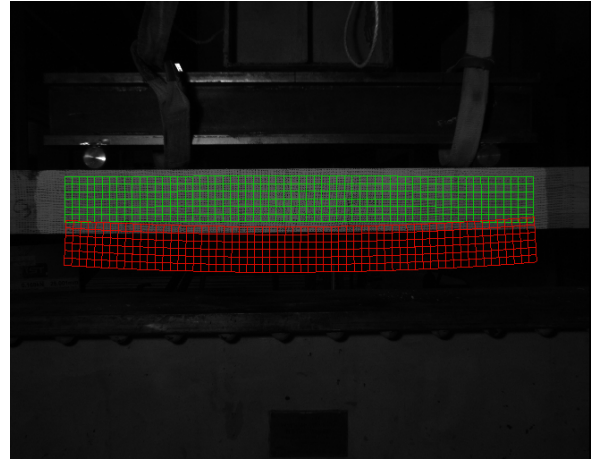
The tests were carried out in order to determine the strains (ε) and deflections (δ) of glulam beams. This was done using DIC software to capture the experimental data, and then using Fortran and Abaqus softwares to produce FEA of the system and comparing these results to the experimental ones.

Tests were carried out on four different reinforcing configurations:

1. Control test with no reinforcing (Beam A)
2. Five 3mm steel cables (Beam B)
3. Four 5mm steel cables (Beam C)
4. Three 8mm steel cables (Beam D)



(a) Test setup



(b) DIC grid overlay with scaled deflection

Figure 10.1: Setup of experimental tests with a view through the DIC camera

Prior to beam production, an adequate adhesive was determined using pull-out tests described in section 9.2.1. The results of these tests can be found in table 10.1 and neatly summarised in graph 10.1

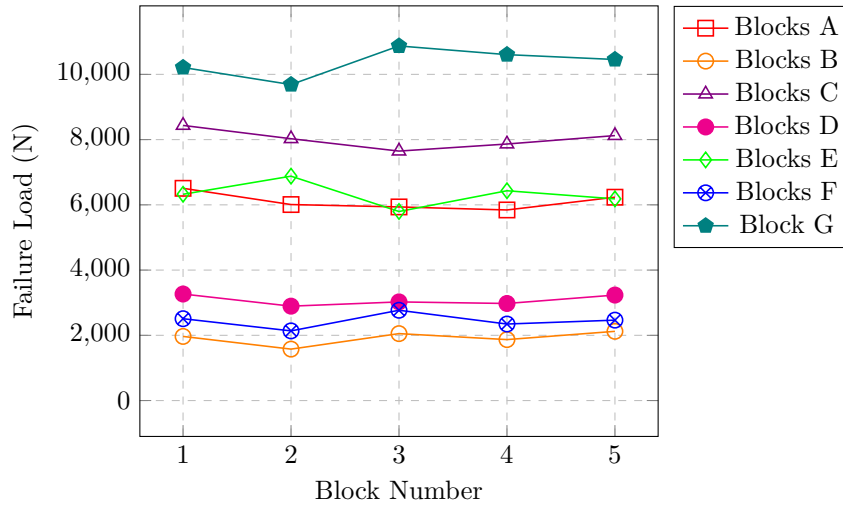
10.2 Bond Strength Results

Table 10.1: Pull-out test results

Block Division	Block no.	Groove Type d x w	Glue Type	Failure load (N)	F_{mean} (N)	Std. dev. (N)	Coef. of Var. (%)	Failure mode
A	1	1 sided 3mm x 2.4mm	PRF	6507	6106	267	4.4	Adhesive
	2			6009				
	3			5936				
	4			5843				
	5			6234				
B	1	1 sided 3mm x 2.4mm	MUF	1966	1917	212	11.1	Adhesive
	2			1577				
	3			2052				
	4			1869				
	5			2122				
C	1	Double sided 1.5mm x 2.4mm	PRF	8435	8020	294	3.7	Cohesive
	2			8027				
	3			7648				
	4			7865				
	5			8123				
D	1	Double sided 1.5mm x 2.4mm	MUF	3266	3079	163	5.3	Cohesive
	2			2894				
	3			3024				
	4			2976				
	5			3234				
E	1	1 sided 2.4mm x 2.4mm	PRF	6324	6323	393	6.2	Adhesive
	2			6879				
	3			5796				
	4			6432				
	5			6183				
F	1	1 sided 2.4mm x 2.4mm	MUF	2509	2445	231	9.5	Adhesive
	2			2136				
	3			2768				
	4			2346				
	5			2465				
G	1	Double sided 1.5mm x 2.4mm	EPO	10211	10366	449	4.3	Cohesive
	2			9687				
	3			10871				
	4			10605				
	5			10456				

From these results we can see that the double sided groove was stronger than a single groove independent of glue type. This matches with the findings from 2.9.2. Then ranked in order the glue strengths were EPO, PRF, MUF consistent with finding in section 2.9.1. Furthermore, the double sided PRF glue was within a 60% range of the EPO glue, which allows the use of a single adhesive type during the production phase, making production of such beams more efficient and cost effective. Looking at the failure mode, shows us that a double sided groove allows for better bonding to the timber as a cohesive failure plane within the glue was observed as a opposed to single sided grooves which always experienced adhesive failure between the timber and glue. The beams in the current study use the single adhesive layup, with the PRF glue type and a double-groove along the entire length of the beam.

Graph 10.1: Pull-out glue strength for blocks with glued in cables

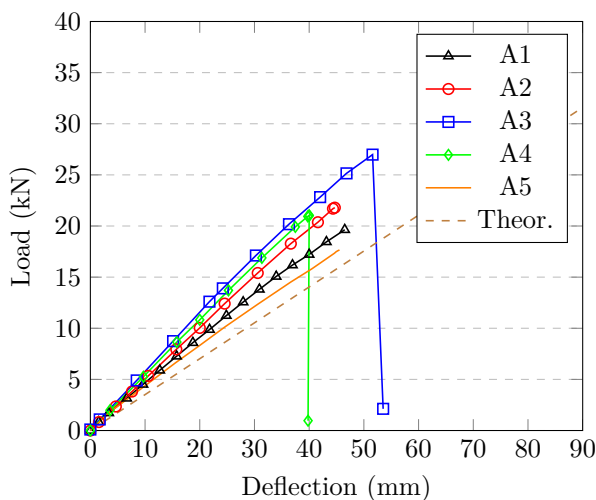


10.3 Load vs. Deflection Results

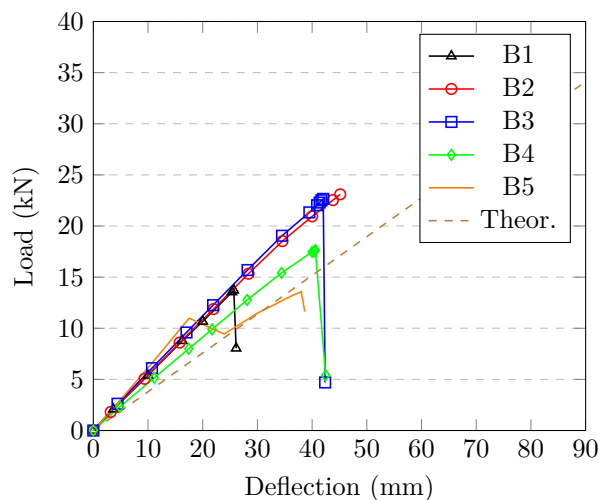
The load vs. deflection curves for the 20 beams are presented in graphs 10.2 - 10.5. The theoretical deflection curves for each configuration, calculated from section 8.2 and shown in graph 8.1, is also included in each graph as a reference. The axes scales for each graph are identical to allow for direct comparison between the graphs.

Data points for the curves were collected every 0.5 seconds of the tests which lasted between 3-4 minutes on average. Load data was collected directly from the load cell whereas the deflection data was collected from both the load cell and the DIC software, which would differ in readings by $\pm 0.2\text{mm}$. Therefore, an average of the two data points was used for the graphs.

Graph 10.2: Un-reinforced glulam beams

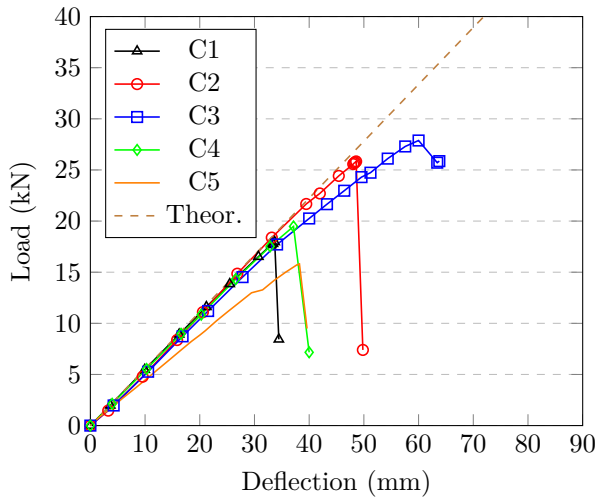


Graph 10.3: Five 3mm steel cables

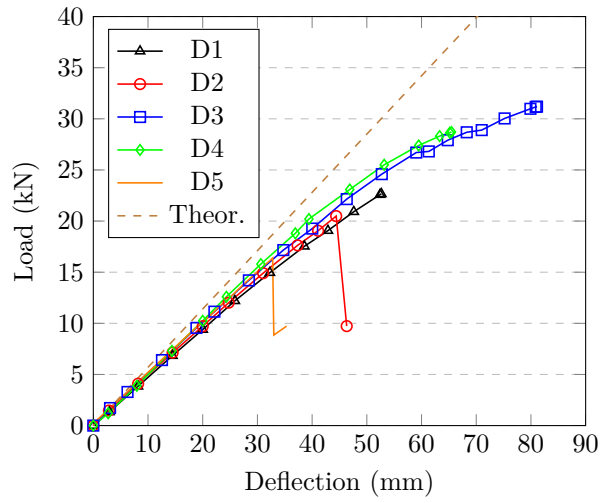


To summarise the load vs. deflection curves, graph 10.6 shows the average max load and the average max deflection for each beam type. The point of failure was taken as the reading 0.5s before failure.

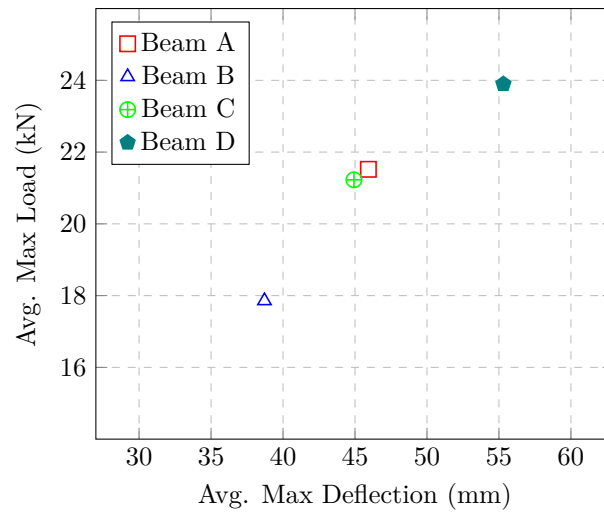
Graph 10.4: Four 5mm steel cables



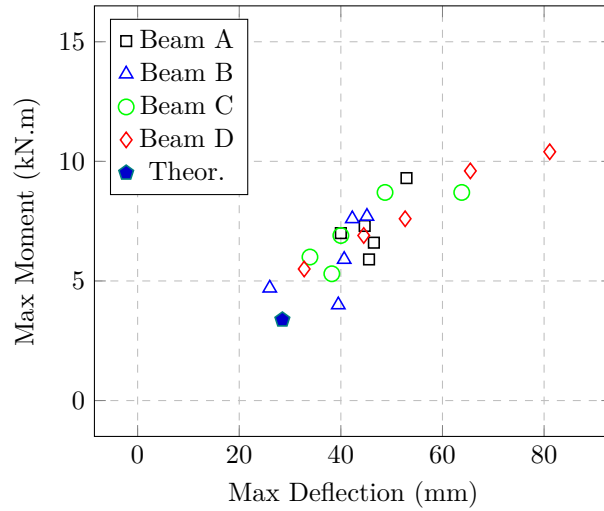
Graph 10.5: Three 8mm steel cables



Graph 10.6: Average Max Load vs. Average Max deflection for glulam beams



Graph 10.7: Maximum moment vs. Max deflection for glulam beams



10.3.1 DIC Deflection Images

The DIC data for all the beams is presented in figures 10.2 - 10.21. The deflection readings at a load of 10kN is shown for comparison purposes, and the final deflection and load readings are presented as well. The colour overlay on each beam shows the deflection contours across the centre 1 meter portion of the beam, similar to that of how a FEM would work.

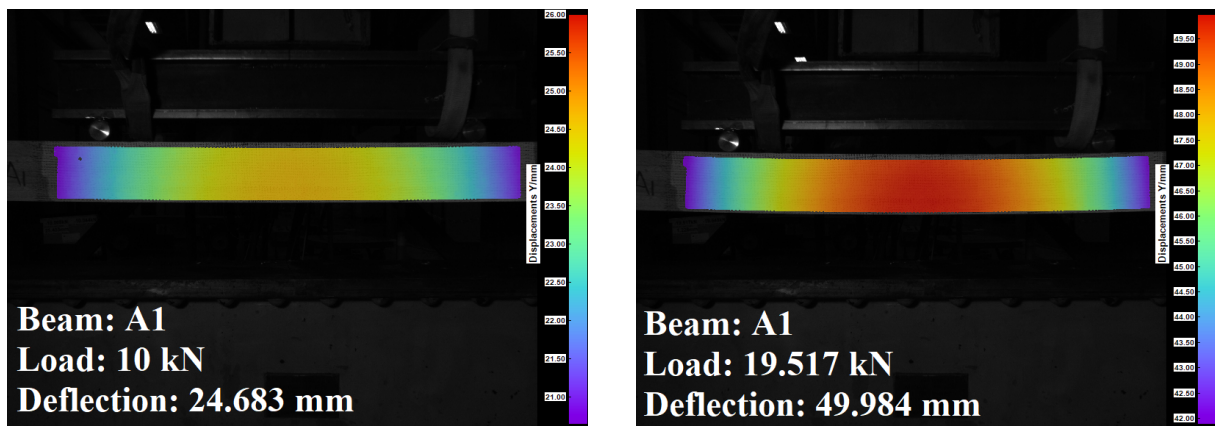


Figure 10.2: Beam A1 at 10 kN and 0.5s before failure

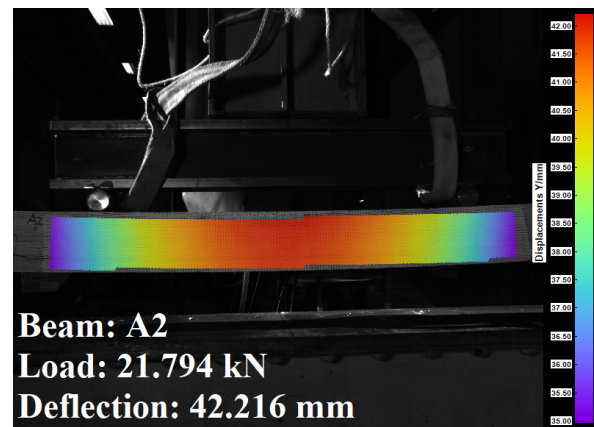
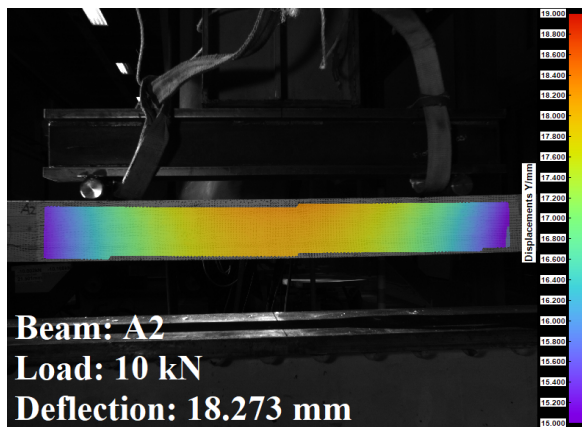


Figure 10.3: Beam A2 at 10 kN and 0.5s before failure

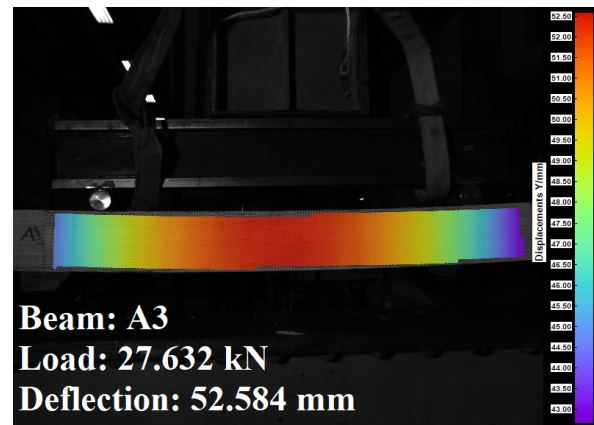
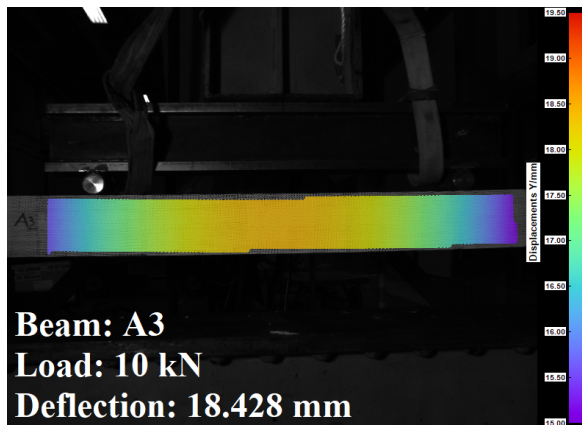


Figure 10.4: Beam A3 at 10 kN and 0.5s before failure

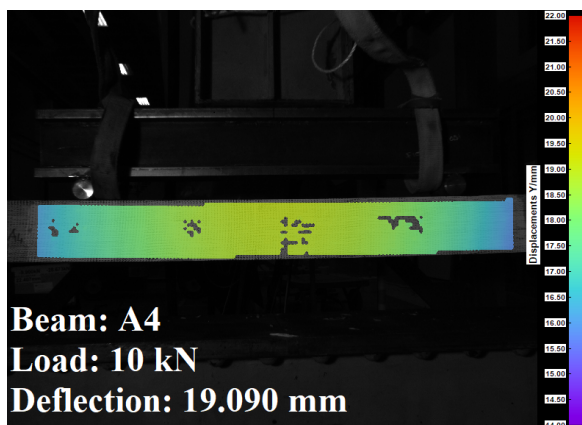


Figure 10.5: Beam A4 at 10 kN and 0.5s before failure

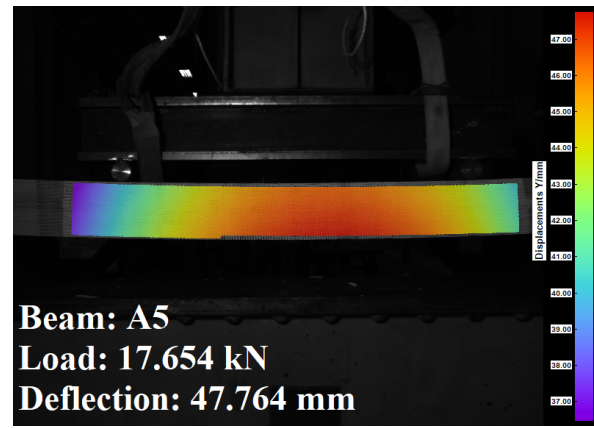
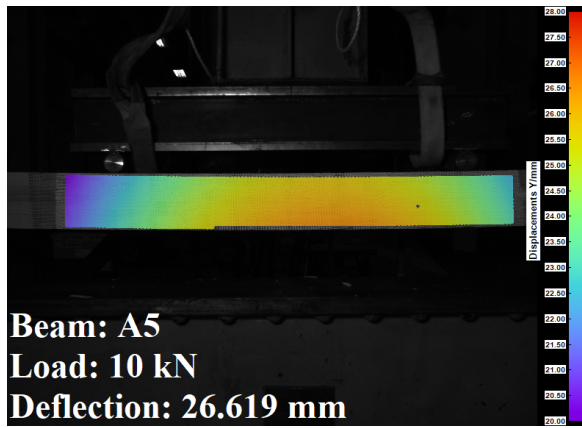


Figure 10.6: Beam A5 at 10 kN and 0.5s before failure

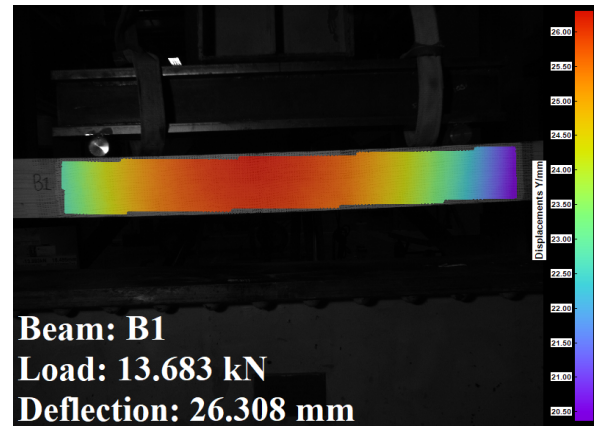
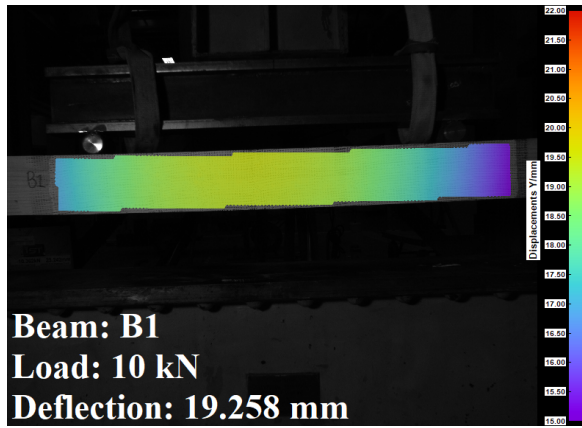


Figure 10.7: Beam B1 at 10 kN and 0.5s before failure

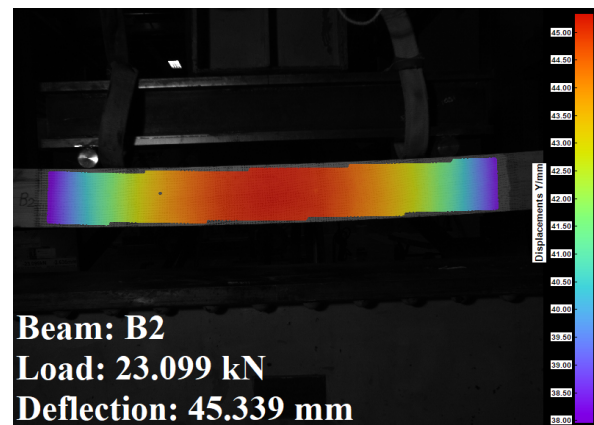
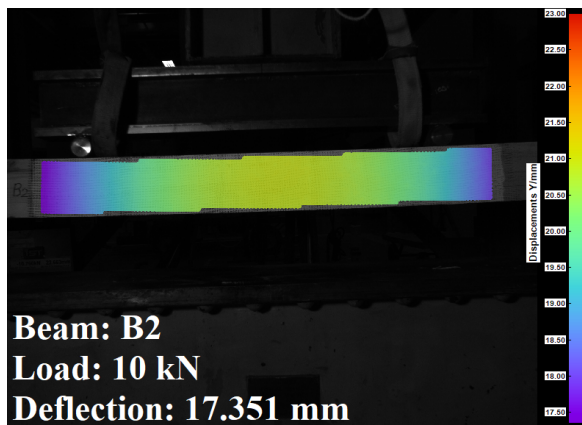


Figure 10.8: Beam B2 at 10 kN and 0.5s before failure

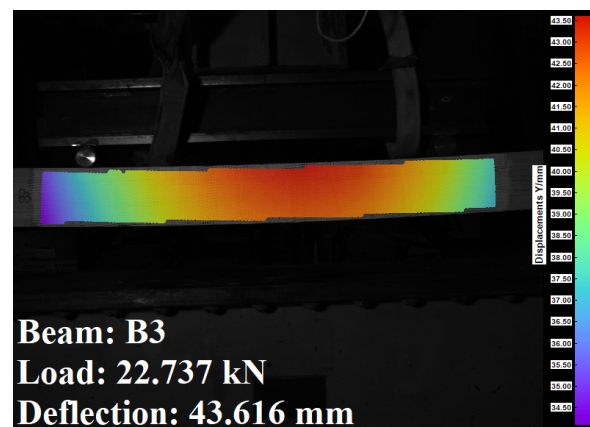
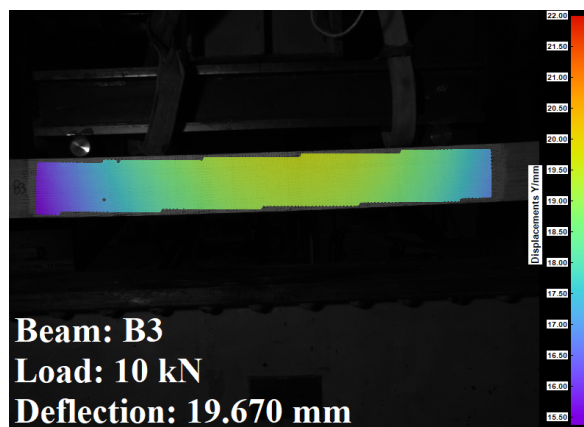


Figure 10.9: Beam B3 at 10 kN and 0.5s before failure

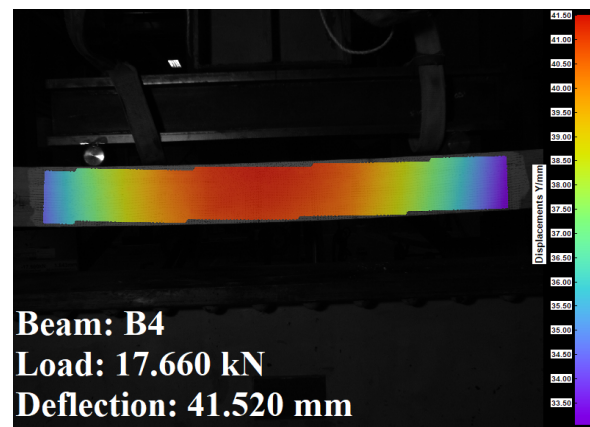
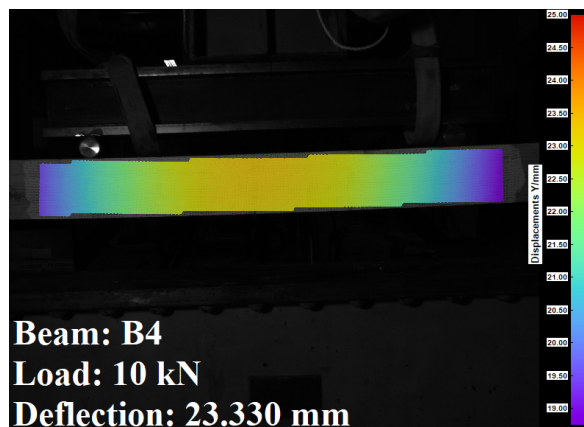


Figure 10.10: Beam B4 at 10 kN and 0.5s before failure

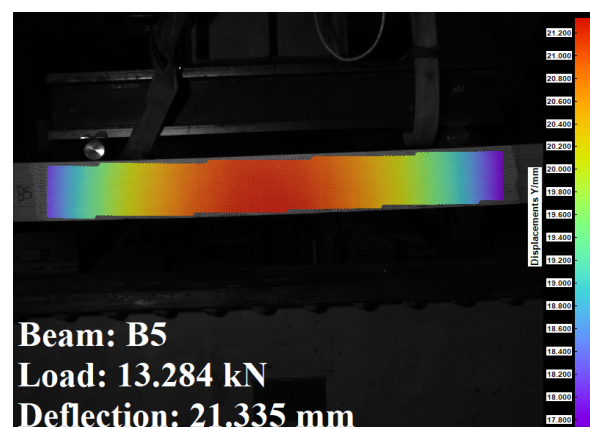
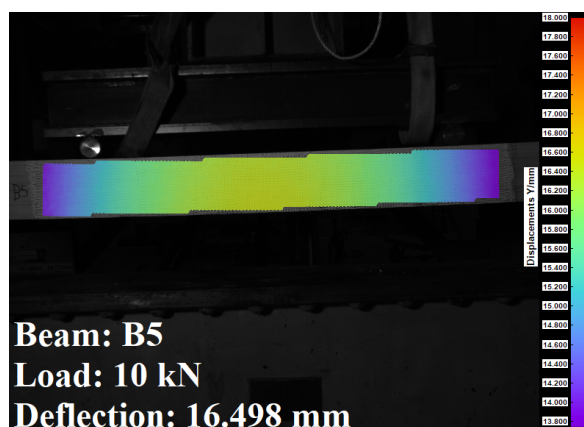


Figure 10.11: Beam B5 at 10 kN and 0.5s before failure

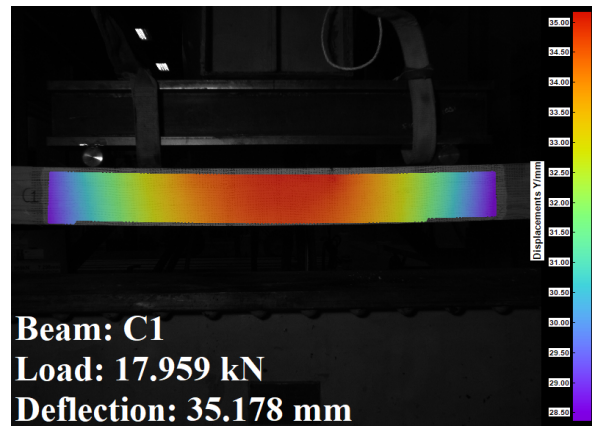
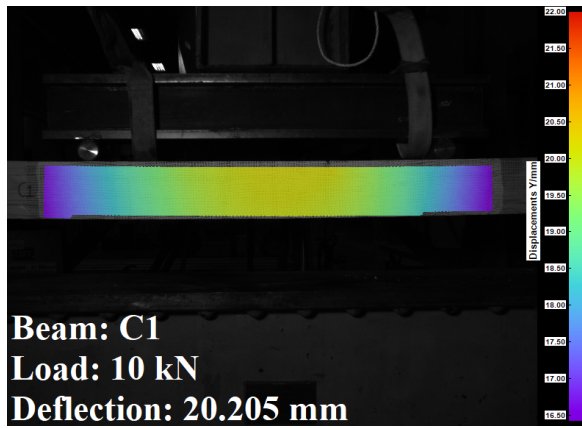


Figure 10.12: Beam C1 at 10 kN and 0.5s before failure

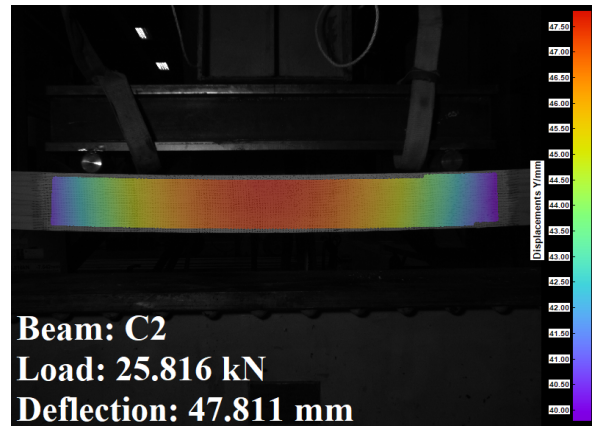
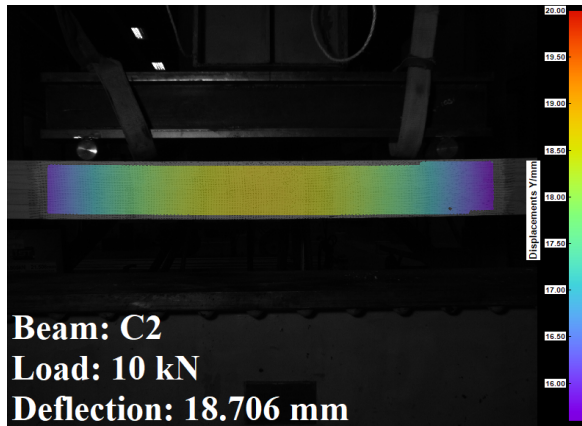


Figure 10.13: Beam C2 at 10 kN and 0.5s before failure

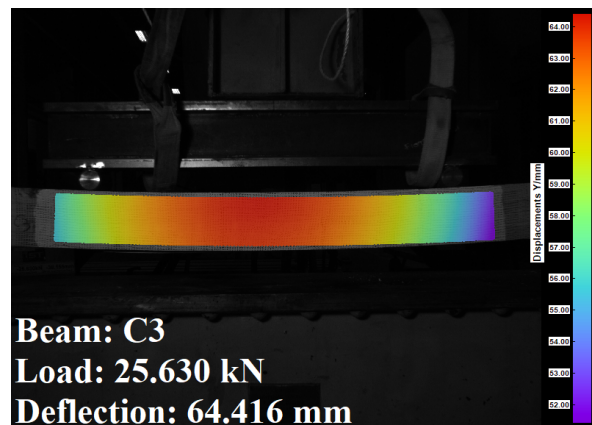
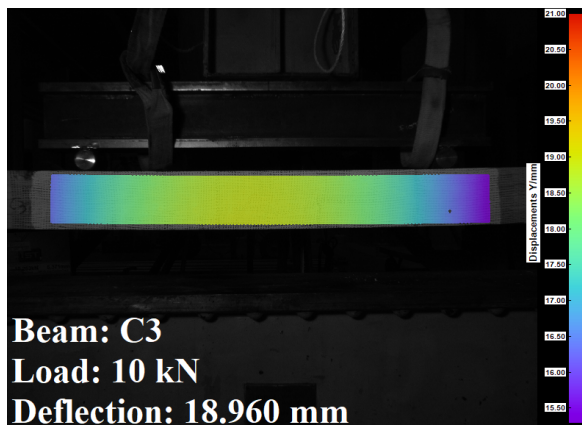


Figure 10.14: Beam C3 at 10 kN and 0.5s before failure

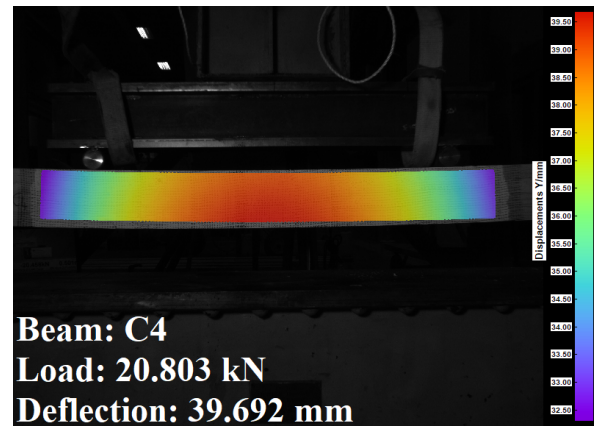
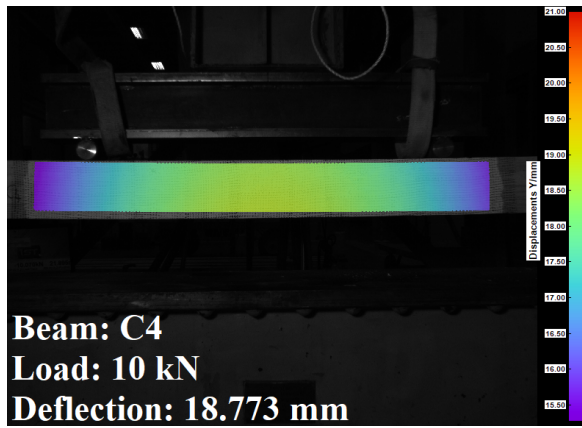


Figure 10.15: Beam C4 at 10 kN and 0.5s before failure

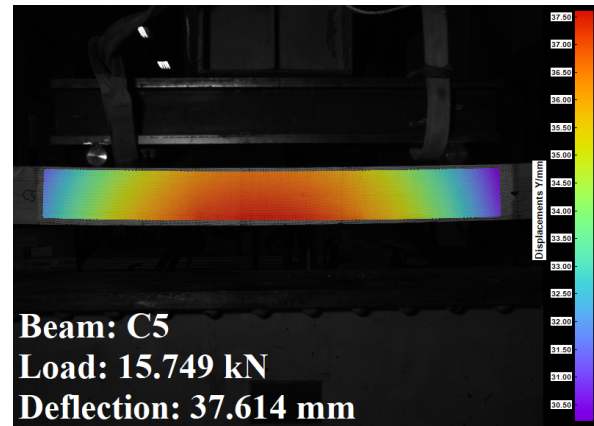
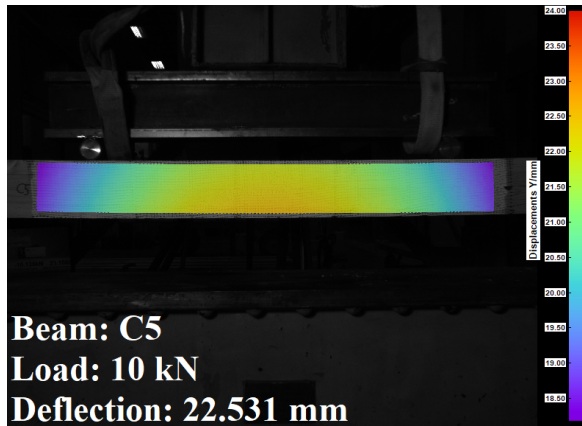


Figure 10.16: Beam C5 at 10 kN and 0.5s before failure

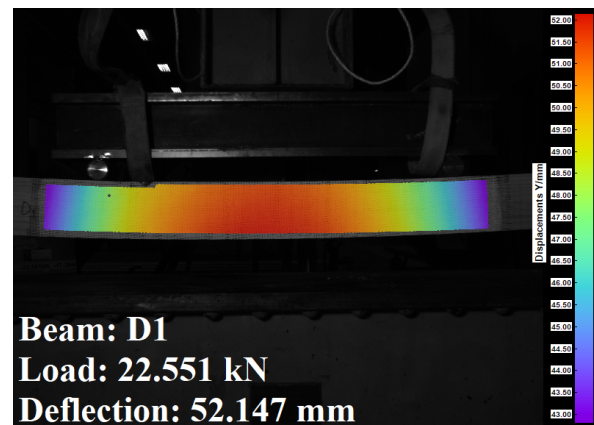
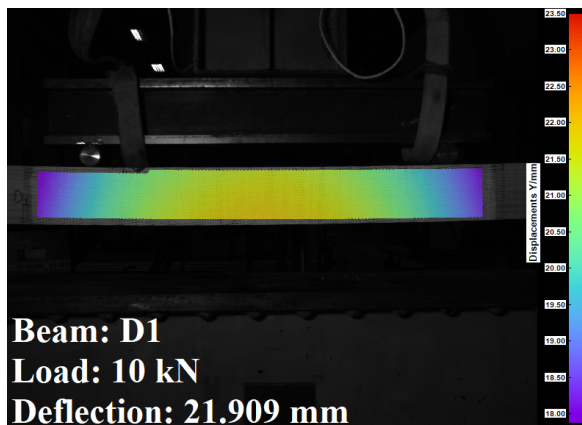


Figure 10.17: Beam D1 at 10 kN and 0.5s before failure

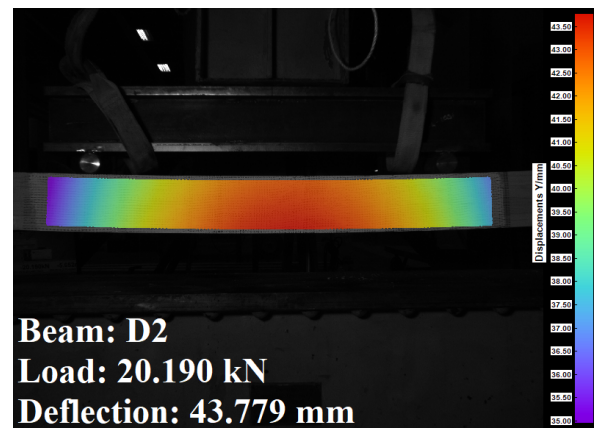
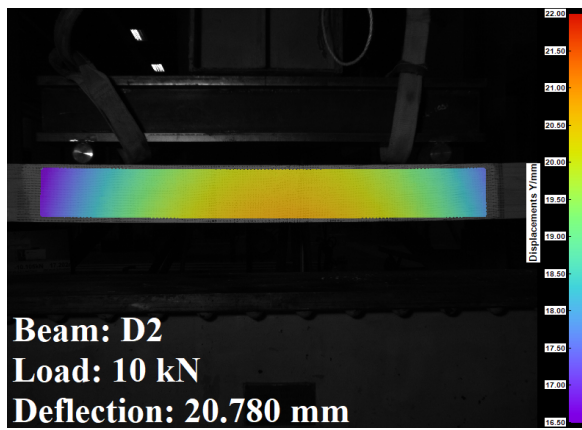


Figure 10.18: Beam D2 at 10 kN and 0.5s before failure

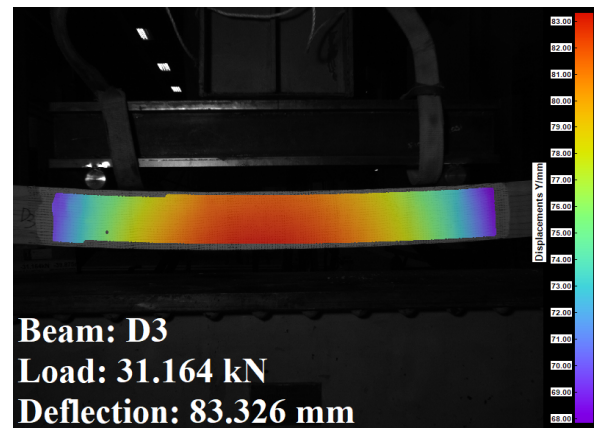
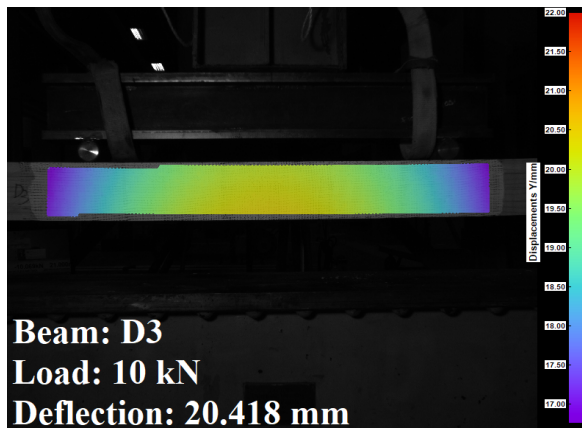


Figure 10.19: Beam D3 at 10 kN and 0.5s before failure

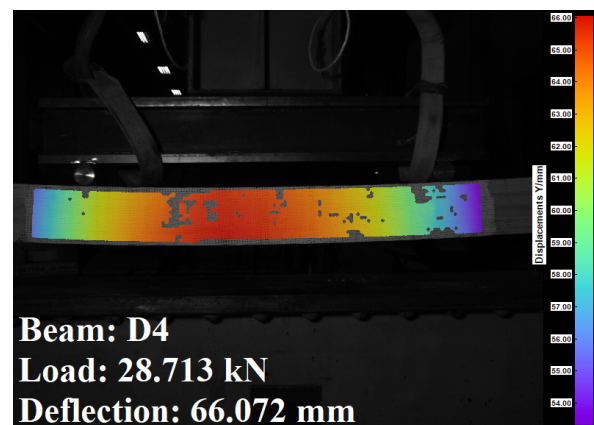
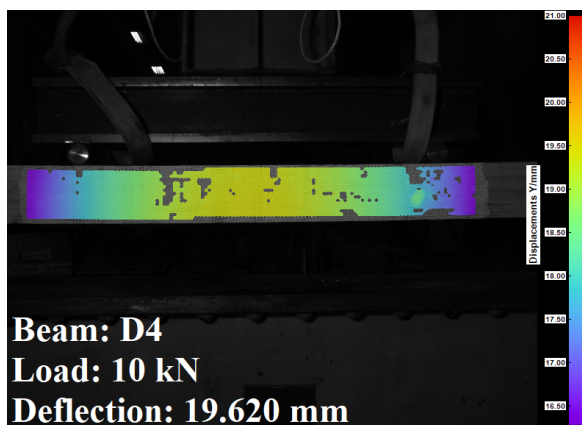


Figure 10.20: Beam D4 at 10 kN and 0.5s before failure

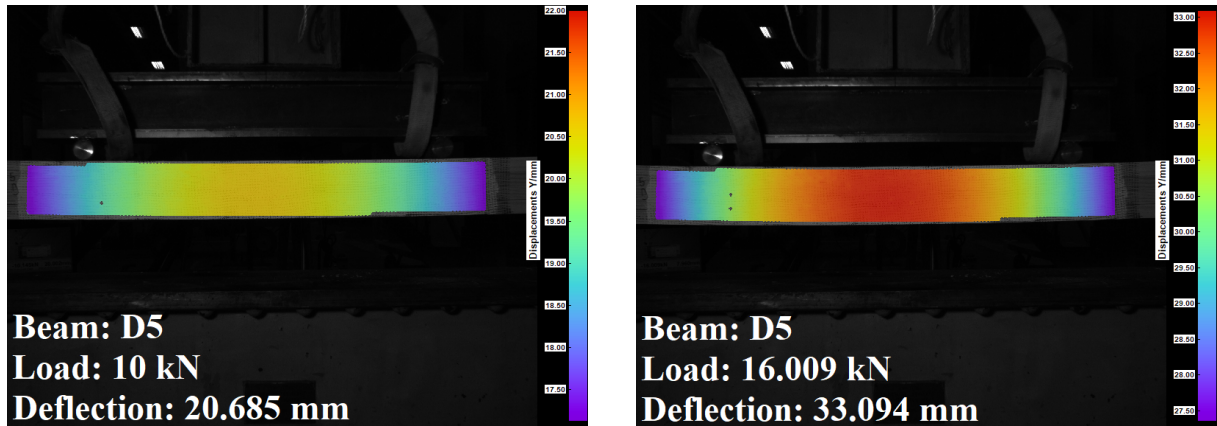


Figure 10.21: Beam D5 at 10 kN and 0.5s before failure

The deflection behavior of timber beams was analyzed through DIC techniques. The results demonstrate a clear correlation between the deflection readings obtained from the DIC setup and those produced by the load actuator, with a deviation of less than 0.5mm. The analysis of the DIC images reveals that the center portion of the beams experiences the highest deflection, with beams B1-5 exhibiting lower deflection readings due to their lower ultimate strength compared to the rest of the beams. A notable exception is beam D, which exhibits a high degree of variability in deflection readings, with a maximum deflection of 83.3mm recorded before failure.

10.4 Load vs. Strain Results

With the help of the DIC software, strain data for the entire beam surface was obtained, as opposed to strain gauges that can only provide local data at specific point. The Data obtained in graphs 10.8 - 10.11 is the strain data a long the length of the lower laminate of the glulam beam. Figure 10.22 shows the gauge line (Line 1) from the DIC software, along which the strain data presented here was obtained.

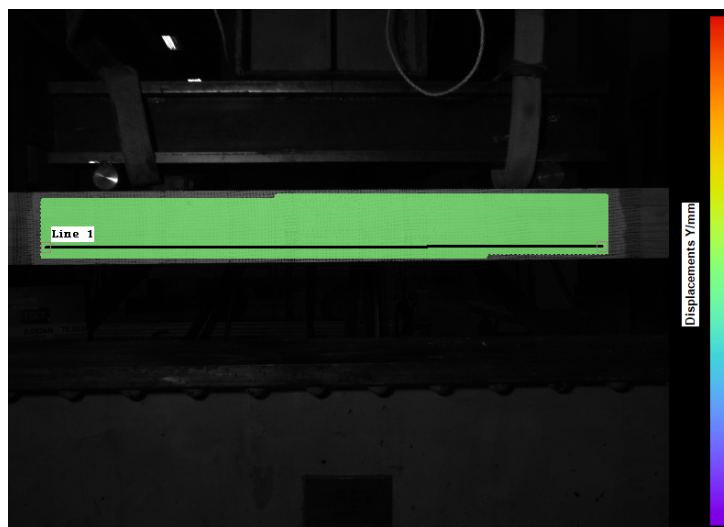
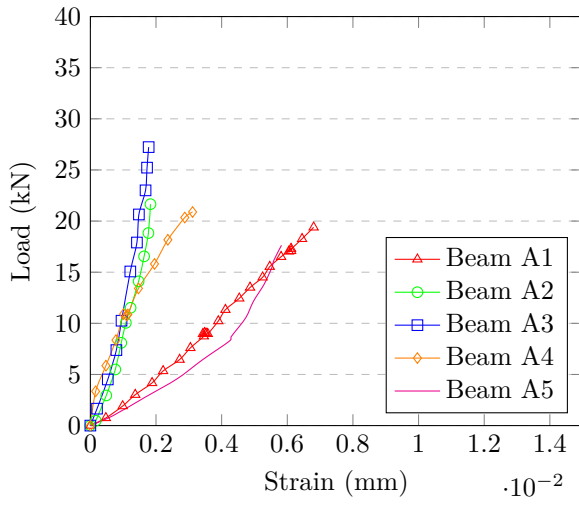
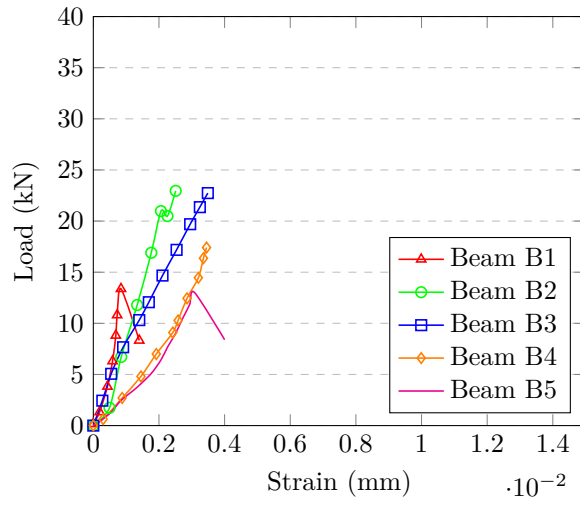


Figure 10.22: Line along which strain data was exported and graphed

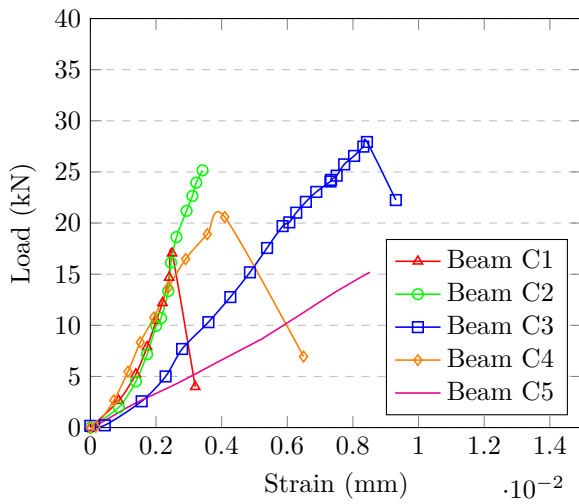
Graph 10.8: Load-Strain for unreinforced beams



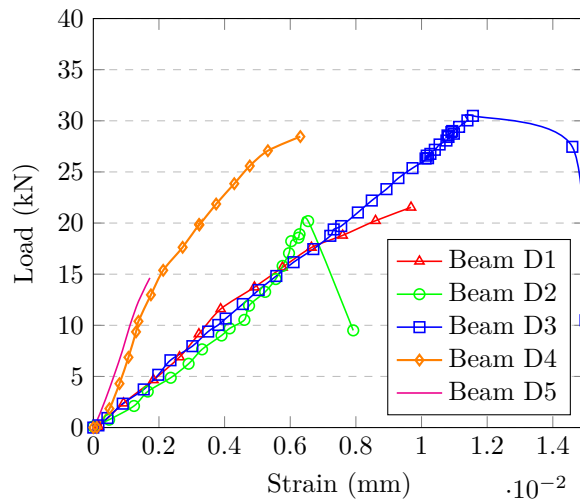
Graph 10.9: Load-Strain for 5 x 3mm cables



Graph 10.10: Load-Strain for 4 x 5mm cables



Graph 10.11: Load-Strain for 3 x 8mm cables



10.4.1 DIC Strain Images

The DIC global strain data for all the beams is presented in figures 10.23 - 10.42. The colour overlay shows strain contours for the centre 1 meter portion of the beam at the step 0.5s prior to breaking. A picture of the beam after failure is included, here the final fracture point is compared to the highest strain point data.

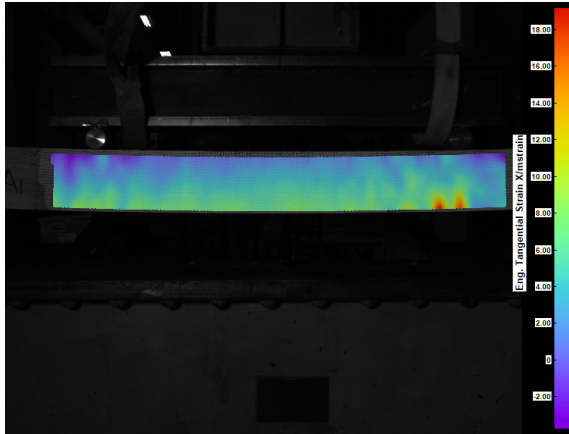


Figure 10.23: Beam A1 global strain vs failure point, 0.5s before failure

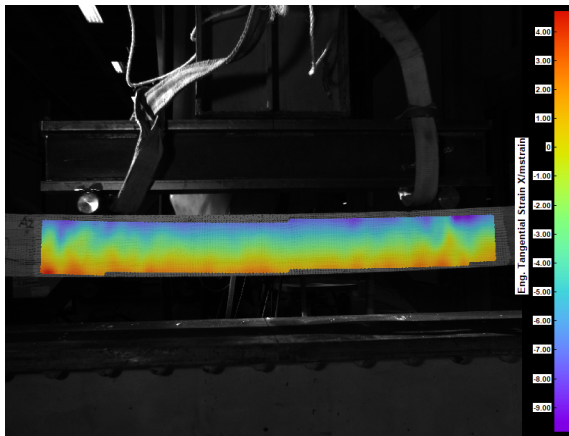


Figure 10.24: Beam A2 global strain vs failure point, 0.5s before failure

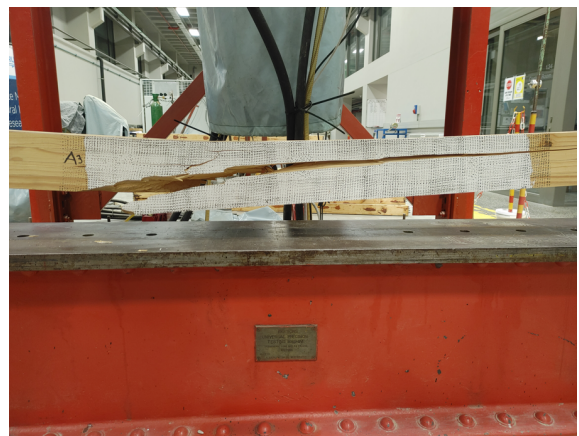
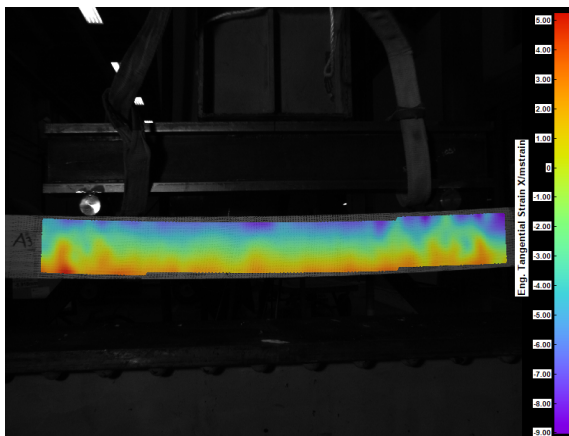


Figure 10.25: Beam A3 global strain vs failure point, 0.5s before failure

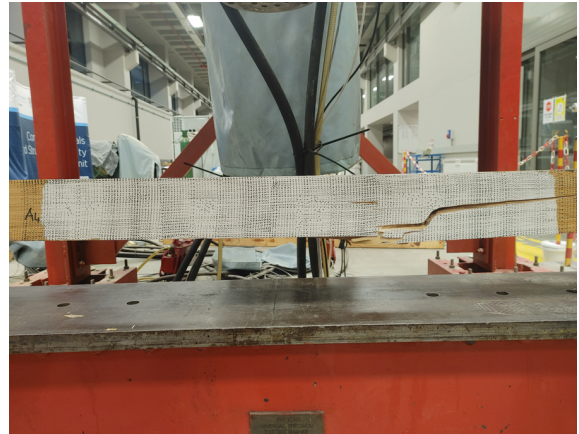
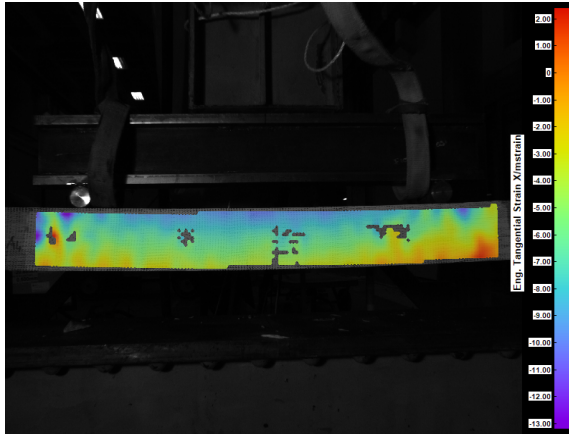


Figure 10.26: Beam A4 global strain vs failure point, 0.5s before failure

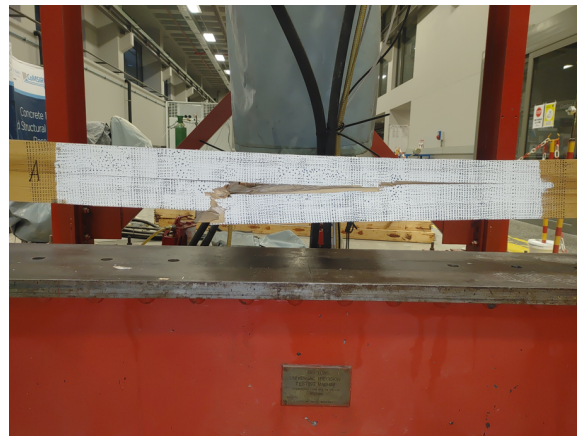
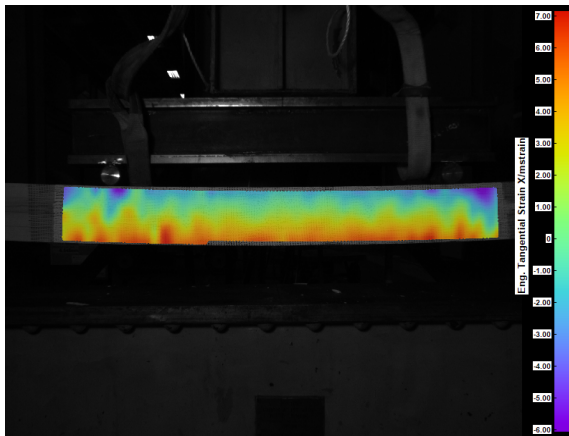


Figure 10.27: Beam A5 global strain vs failure point, 0.5s before failure

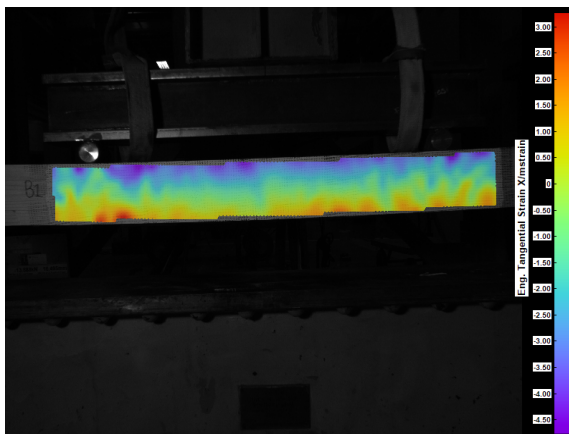


Figure 10.28: Beam B1 global strain vs failure point, 0.5s before failure

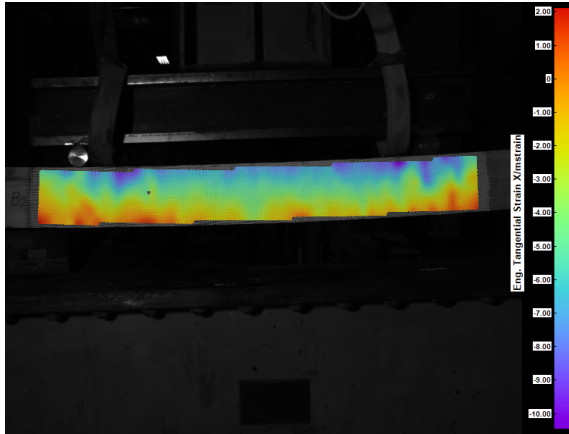


Figure 10.29: Beam B2 global strain vs failure point, 0.5s before failure

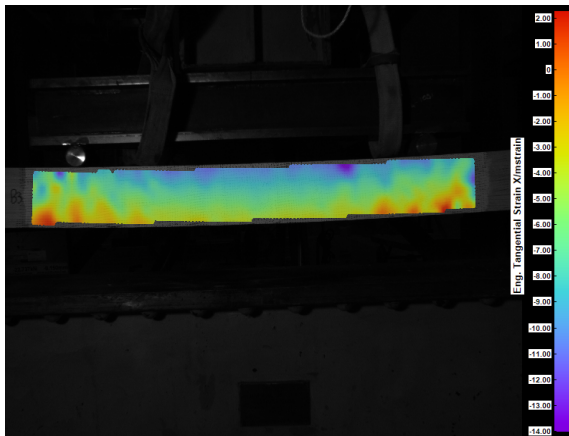


Figure 10.30: Beam B3 global strain vs failure point, 0.5s before failure

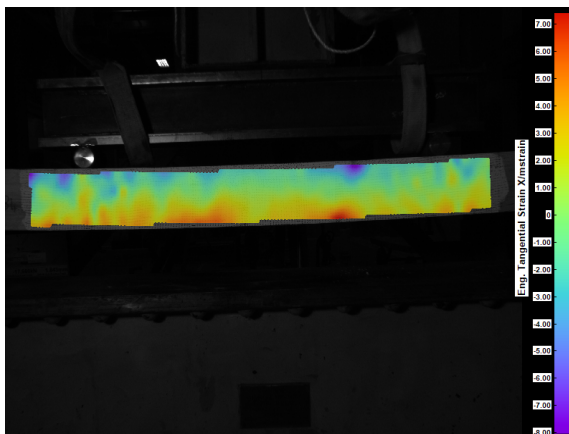


Figure 10.31: Beam B4 global strain vs failure point, 0.5s before failure

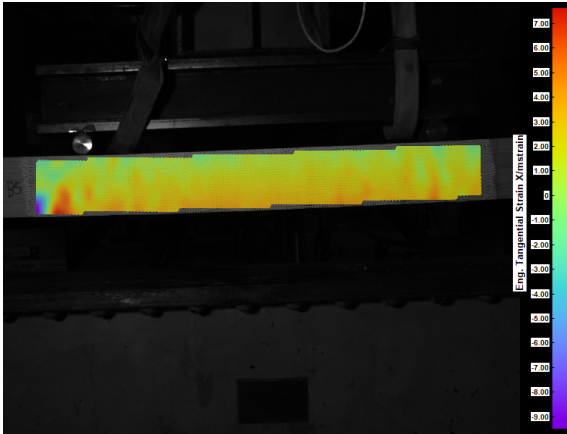


Figure 10.32: Beam B5 global strain vs failure point, 0.5s before failure

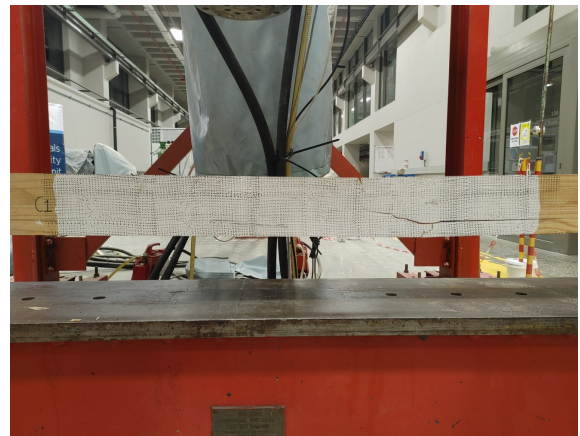
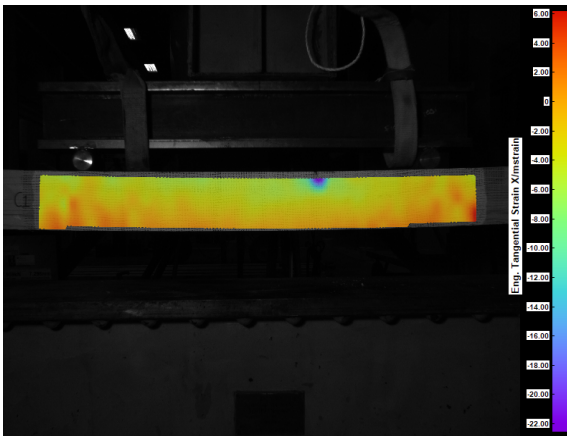


Figure 10.33: Beam C1 global strain vs failure point, 0.5s before failure

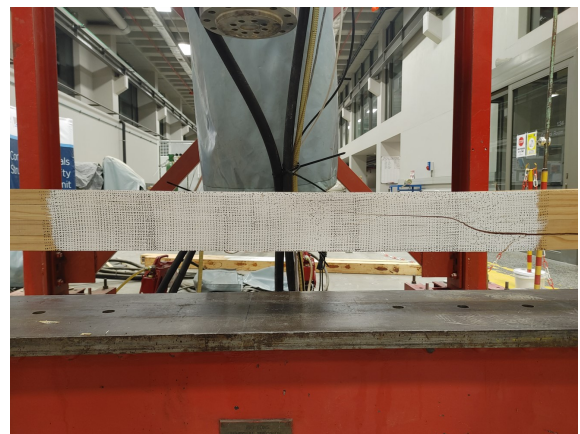
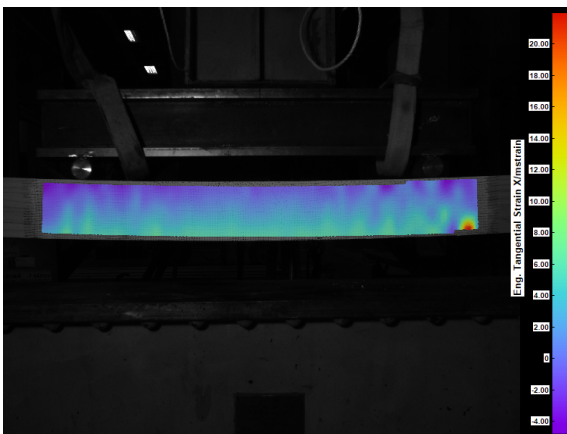


Figure 10.34: Beam C2 global strain vs failure point, 0.5s before failure

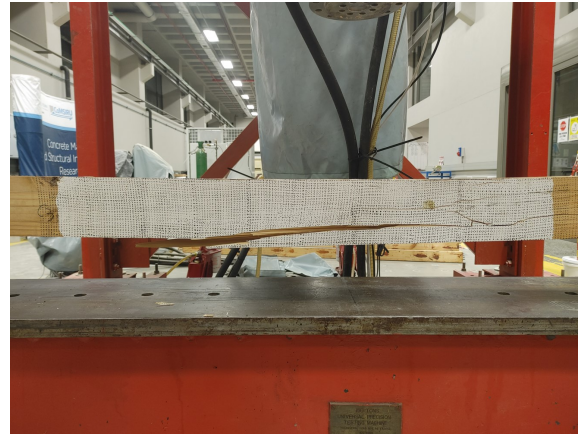
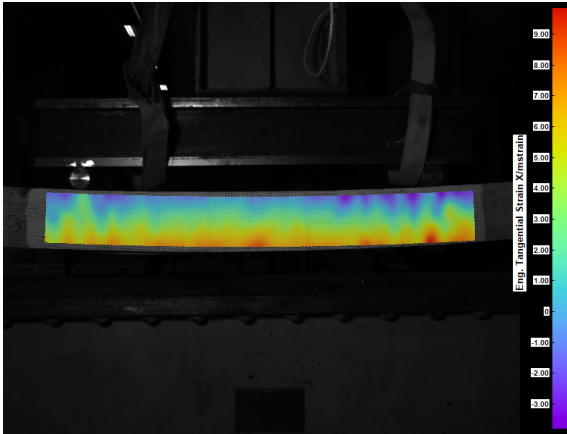


Figure 10.35: Beam C3 global strain vs failure point, 0.5s before failure

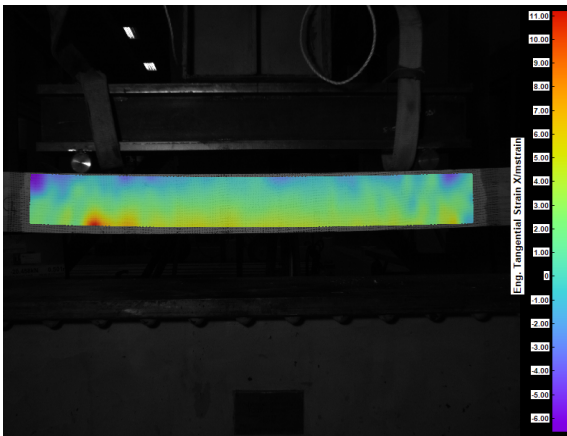


Figure 10.36: Beam C4 global strain vs failure point, 0.5s before failure

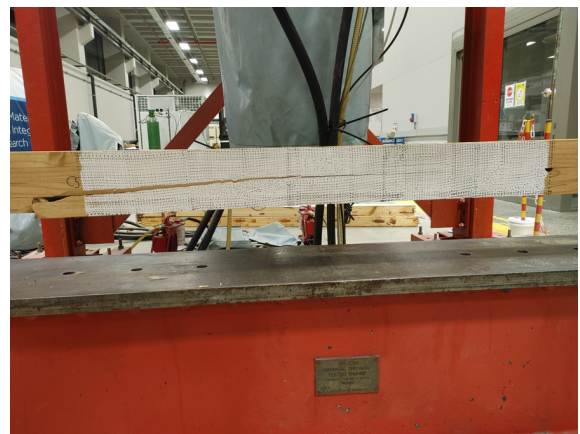
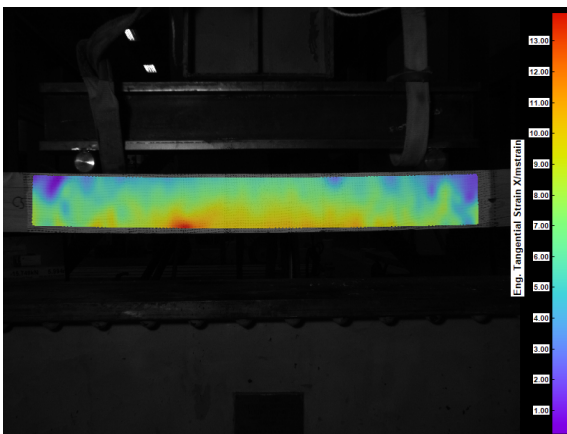


Figure 10.37: Beam C5 global strain vs failure point, 0.5s before failure

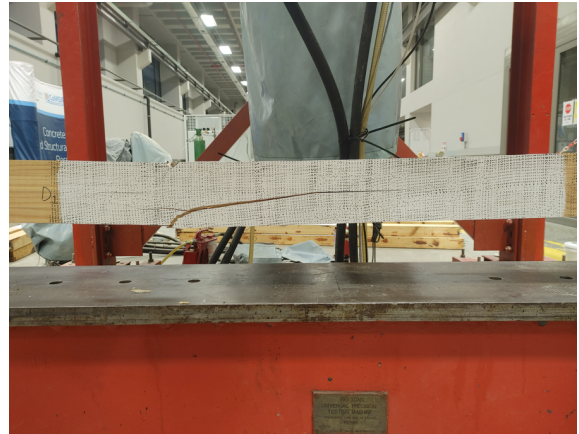
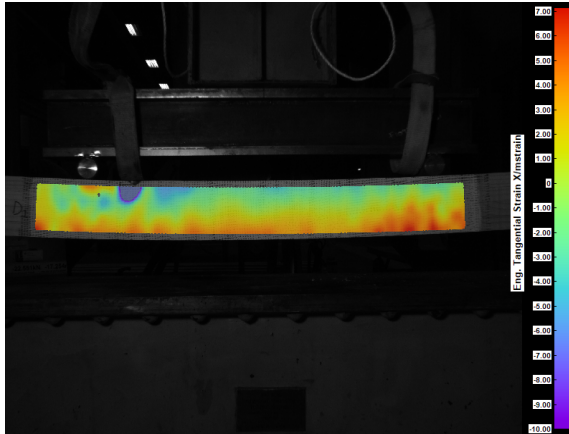


Figure 10.38: Beam D1 global strain vs failure point, 0.5s before failure

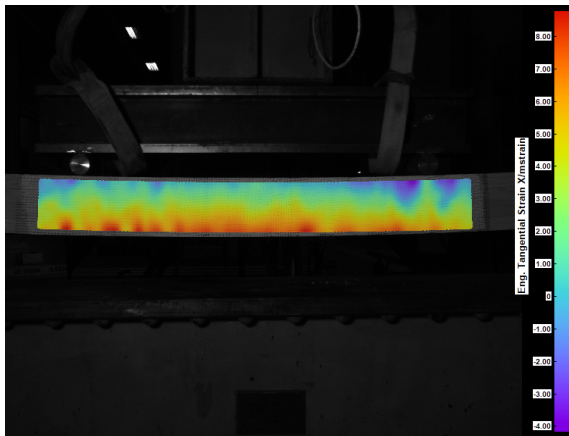


Figure 10.39: Beam D2 global strain vs failure point, 0.5s before failure

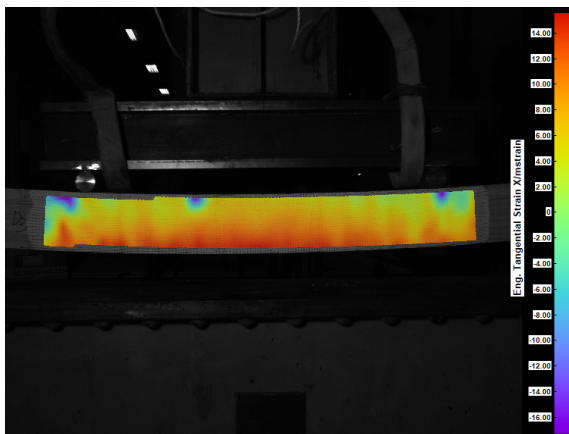


Figure 10.40: Beam D3 global strain vs failure point, 0.5s before failure

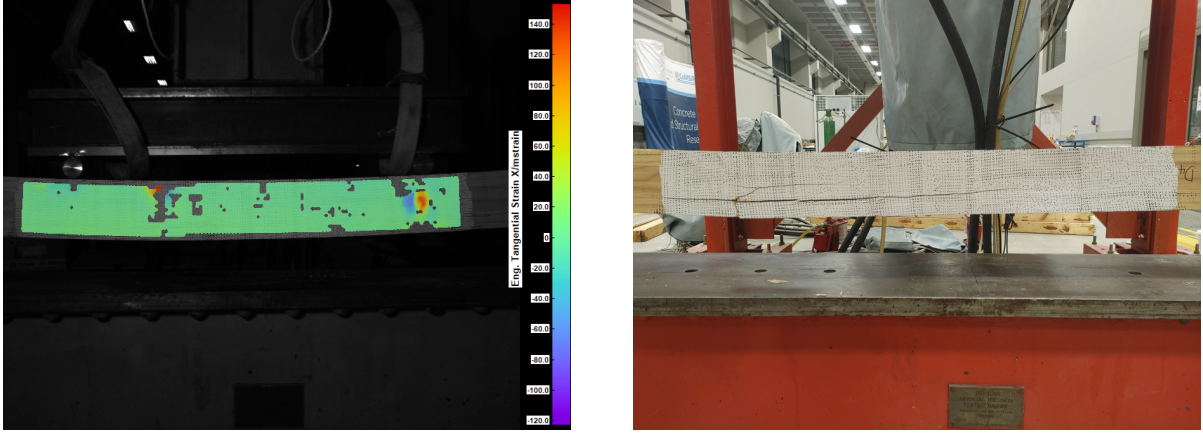


Figure 10.41: Beam D4 global strain vs failure point, 0.5s before failure

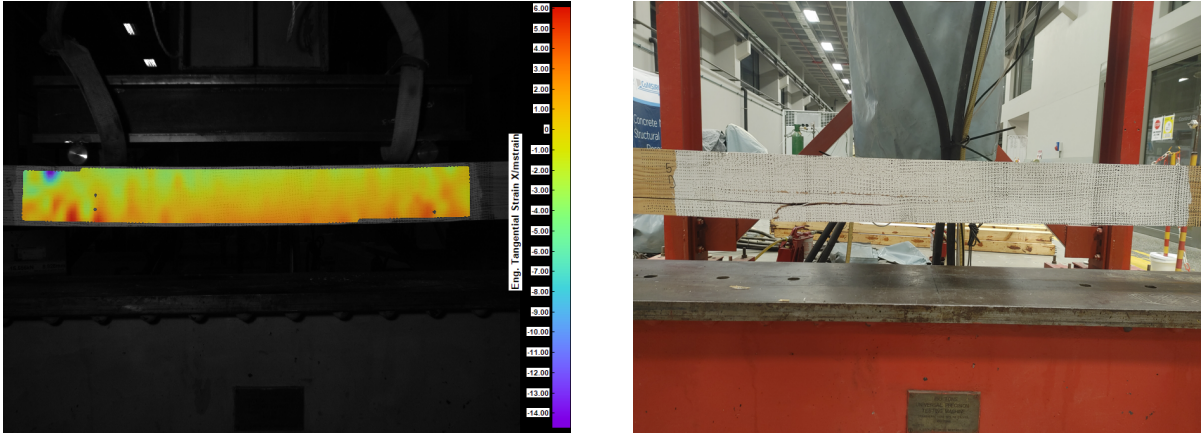


Figure 10.42: Beam D5 global strain vs failure point, 0.5s before failure

The effectiveness of DIC software in detecting the deformation behavior of timber beams was analyzed. The results showed that the DIC software was able to accurately predict the location of the initial crack formation at points of maximum strain. However, the failure location of 4 out of 5 beams in test set B was not correctly predicted by the DIC software. This was attributed to the high stresses being located within the glue-line, where the beams tend to fail, an area that was not analyzed by the DIC software. It should be noted that beam D4 has an invalid strain result due to out-of-plane sliding which is not captured accurately as a single DIC camera was used that analysis along a 2D plane only. To address these limitations, future research can improve the accuracy of the DIC software through the use of multi-camera setups and the integration of strain gauges between the glulam laminates.

10.5 Abaqus Results

The findings of the computational simulations performed using the Abaqus FEM software are presented in this section. The model was established in Abaqus, as detailed in section 9.3.1, with the implementation of the multi-linear damage criteria proposed by Gharib et al. (2017) and Nouri et al. (2019) as the material property for the timber beams in the form of a user-defined subroutine (UMAT). The UMAT was

Table 10.2: Beam names and reinforcing configurations

Beam	No. of Steel Bars	Bar Diameter (mm)	Reinforcing Cross Sectional Area (mm ²)
A	-	-	-
B	5	3	35.34
C	4	5	98.17
D	3	8	150.80

calibrated for both tension and parallel tests in section 9.3.3, and its efficacy for reinforced bending tests is subsequently evaluated in this section. Table 10.2 serves as a reminder for the beam naming convention with regards to its reinforcing cables diameter.

10.5.1 Load Deflection Analysis

The load-deflection results, up to the point of failure, for the experimental and numerical simulations of the four-point bending tests are presented in graphs 10.12 to 10.15. The upper and lower bound experimental results for the control beam, $\varnothing 3\text{mm}$, $\varnothing 5\text{mm}$ and $\varnothing 8\text{mm}$ steel cable reinforced beams are compared to the numerical results in each of these graphs. It can be observed that the Abaqus simulations accurately predict the experimental results for the control beam and 8mm steel cable reinforced beams. However, the numerical results for the 3mm and 5mm steel cable reinforced beams show a higher failure load as compared to the experimental results. The abaqus deflection output for Beam C is shown in figure 10.43.

For the control beam, the maximum deflection was found to be 52.9mm and the corresponding Abaqus simulation result was 52.2mm. This difference of 0.7mm is well within the margin of error. Similarly, the maximum deflection for the 8mm steel cable reinforced beam was found to be 81.1mm in the experimental tests, while the corresponding numerical output from Abaqus and the UMAT was 82.1mm, a difference of 1.0mm.

On the other hand, for the 3mm steel cable reinforced beam, the maximum deflection was found to be 42.3mm in the experimental tests, while the corresponding Abaqus result deflected 4.3mm further to 46.6mm, a difference of 10.2%. For the 5mm steel cable reinforced beam, the maximum deflection was found to be 48.7mm in the experimental tests, while the corresponding numerical result was 52.2mm, a difference of 7.2%.

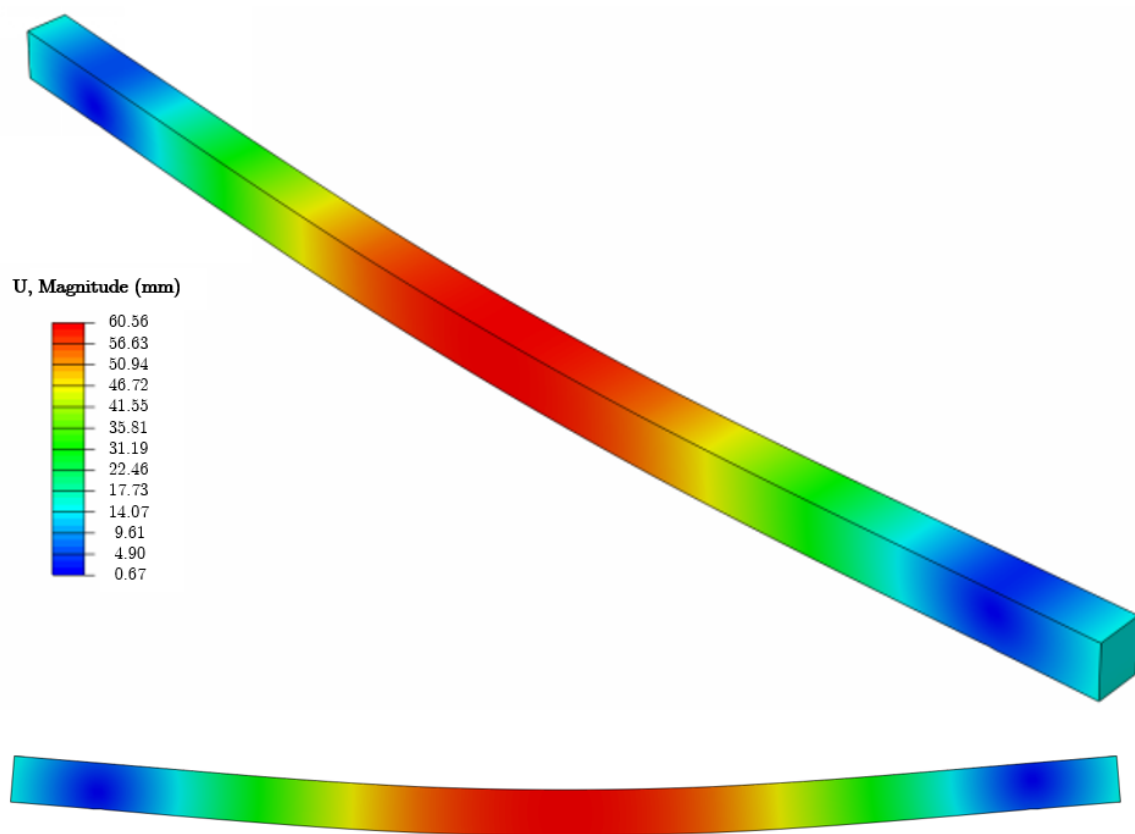
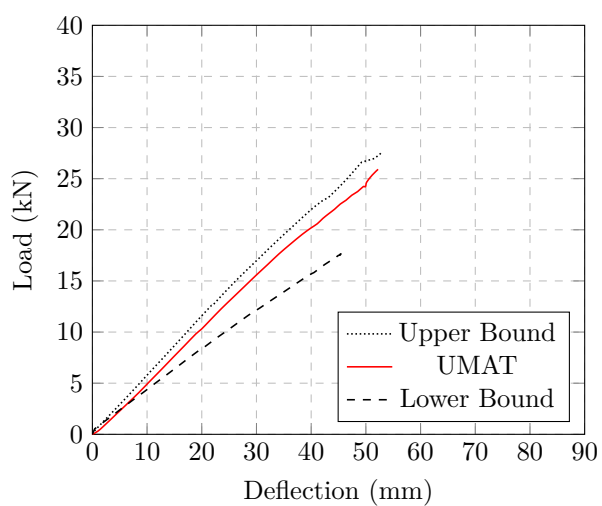
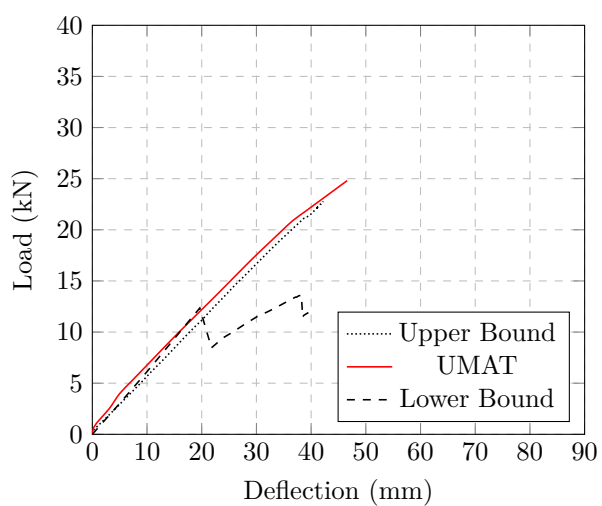


Figure 10.43: Abaqus deflection contours for Beam C

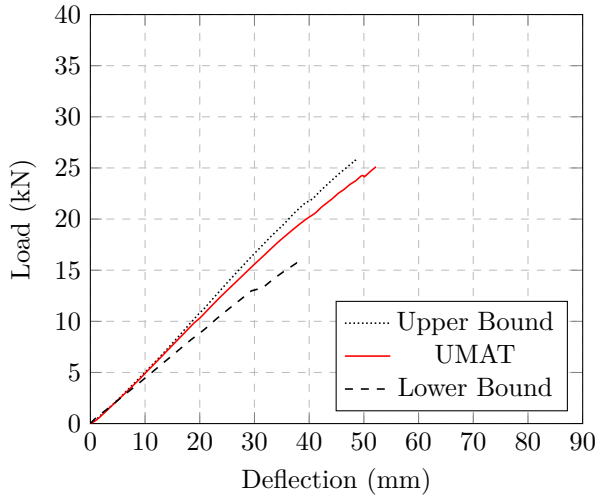
Graph 10.12: Abaqus Deflection - Beam A



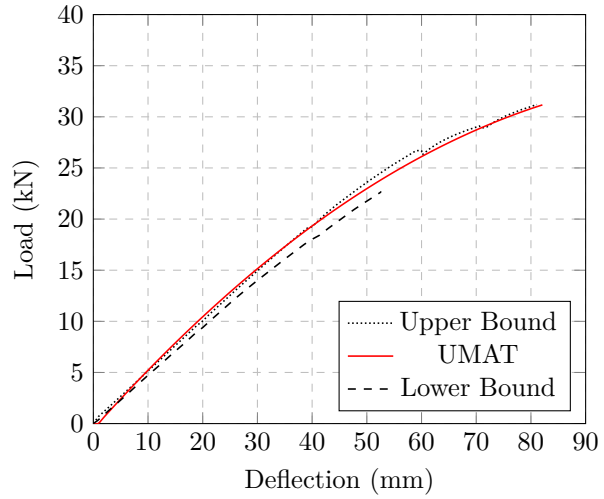
Graph 10.13: Abaqus Deflection - Beam B



Graph 10.14: Abaqus Deflection - Beam C



Graph 10.15: Abaqus Deflection - Beam D



Results of the load prediction from the UMAT and Abaqus follow a similar pattern of the deflection results. It should be noted that the stated loads are assumed to be split evenly between the 2 load points as they are connected by a spreader beam as shown in figure 9.6.

For the control beam, the experimental maximum load of 27.632kN was in close agreement with the simulation result of 25.91kN, with a difference of 1.7kN. The 8mm steel cable reinforced beam also showed a similar pattern, with a maximum load of 31.189kN in the experiments and 31.165kN in the FEA, a difference of only 0.024kN.

However, the 3mm and 5mm steel cable reinforced beams, once again, showed discrepancies between the experimental and numerical results. The experimental maximum load of 22.73kN for the 3mm reinforced beam was exceeded by the simulation result of 24.80kN, a difference of 9.12%. Similarly, the 5mm reinforced beam had a maximum load of 25.84kN in the experiments and 27.79kN in the FEA, with a difference of 7.5%.

The use of the UMAT in predicting load-deflection curves for timber beams is dependent on the accuracy requirements and the type of reinforcing used. When reinforced with steel cables, the UMAT can provide reasonable predictions of the load-deflection behavior, as demonstrated by the results of the 8mm steel cable reinforced beam. However, for the 3mm and 5mm reinforced beams, the differences between the experimental and simulation results suggest that the UMAT may not fully capture the complex interaction between the timber and steel cable.

It is also important to consider that the accuracy of the UMAT results will depend on the quality and accuracy of the input parameters, such as the material properties, boundary conditions, and loading conditions. Additionally, the quality and resolution of the finite element mesh can also impact the accuracy of the predictions.

10.5.2 Damage Evolution

The aim of this chapter is to present the results of the FEA simulations of the timber beam using the UMAT that was developed to incorporate the CM) model. The purpose of this analysis was to evaluate the accuracy and reliability of the CDM model in predicting the damage evolution in the timber beam.

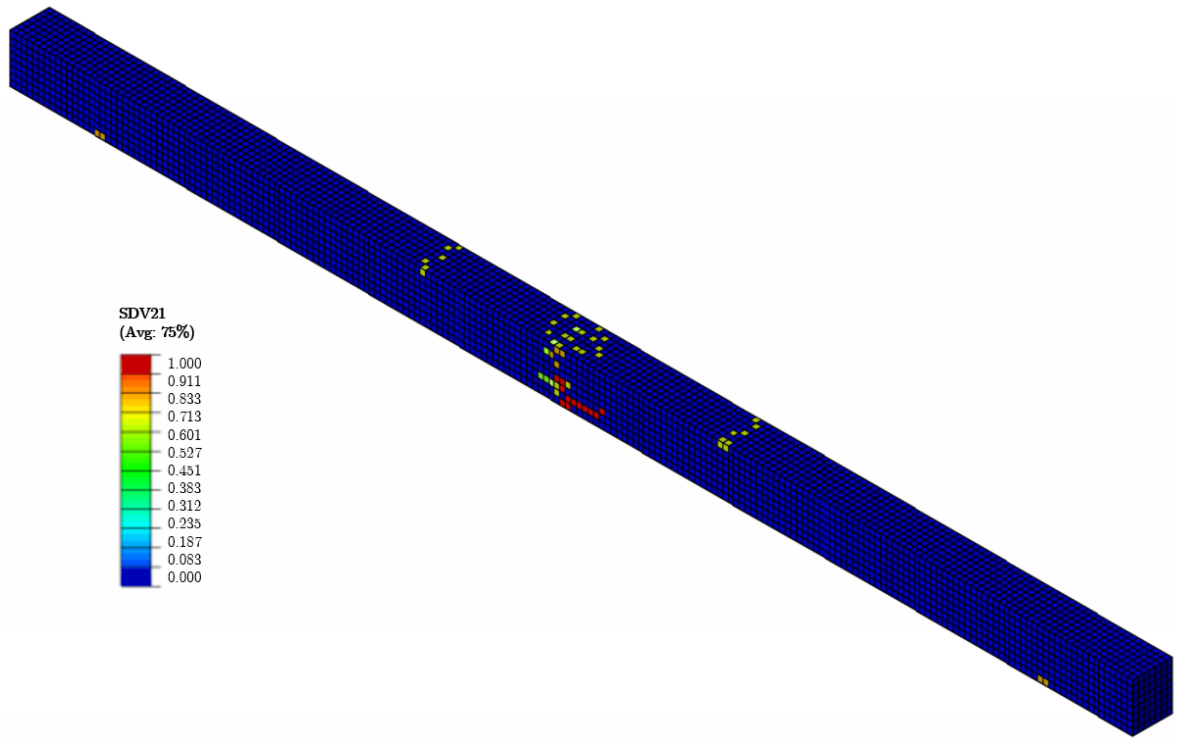


Figure 10.44: Damage Prediction for the Control Beam at Failure

The results of the FEA simulations show the damage evolution in the timber beam over time. Figure 10.44 and 10.45 shows the damaged state in the control beam. It is observed that the first damage occurred in the vicinity of the loading steels. As the load continued to increase, the damage became highly localized at a deflection of 51.112mm, which then quickly transformed into macroscopic cracks. This ultimately led to failure at a deflection of 52.918mm, resulting in a catastrophic brittle failure.

The predicted failure of the model was compared with the experimental results. Figure 10.46 shows the observed rupture in the experiment, and it is observed that the predicted failure of the model is in good agreement with the experimental results. A crack from the center of the tension zone can be seen propagating vertically, followed by a sharp horizontal splitting of the timber fibres.

The results of this study demonstrate the effectiveness of the UMAT in predicting the damage evolution in the timber beam. The UMAT was able to accurately capture the localized damage near the loading steels and predict the brittle failure at the mid-span deflection. This highlights the potential of the UMAT in accurately predicting the damage evolution in timber beams and can provide valuable insights into the behavior of the material and structure.

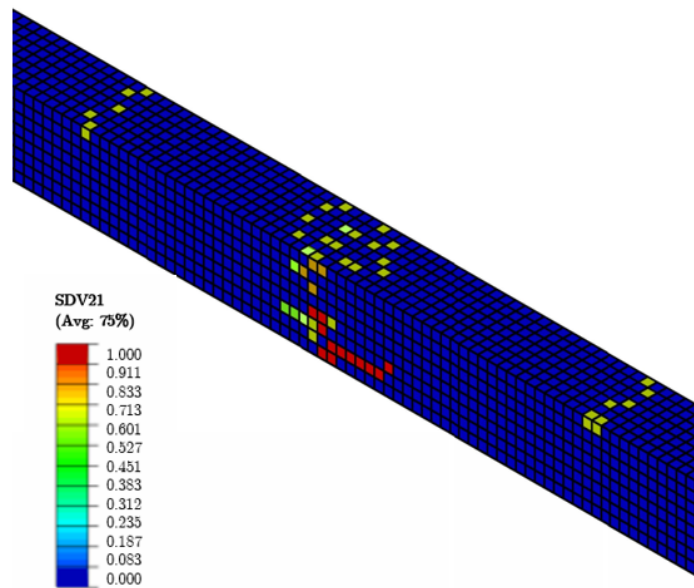


Figure 10.45: Damage Prediction for the Control Beam at Failure - Centre Portion

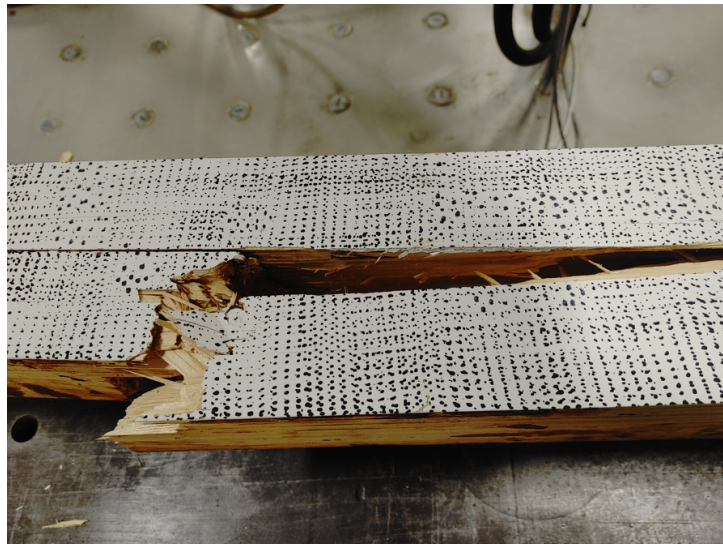


Figure 10.46: Damage pattern on a control beam

Chapter 11

Discussion

11.1 Introduction

This section presents the findings of an investigation into the use of a CDM to model glulam beams reinforced with glued-in steel cables. The research aimed to explore the feasibility of using CDM as a tool for predicting the mechanical behavior of these types of structures and to identify any limitations or challenges associated with its application. The study was conducted using a combination of experimental testing and FEM using the software Abaqus and a UMAT, the results of which can be found in section 10.

Overall, this research contributes to the growing body of knowledge on the use of CDM in the field of timber engineering and may have important implications for the design and analysis of these types of structures in the future.

11.2 Experimental Results

The experimental tests were conducted on full-scale, 3 meter glulam beams, with a control test being performed on 5 beams without reinforcing and three additional tests on beams reinforced with steel cables of increasing diameter. The test specimens were subjected to four-point bending and load was applied at a rate of 0.2mm/s. The control beam failed as expected, exhibiting the typical brittle tensile failure mode in bending. However, the results of the tests on the reinforced beams were more complex and revealed some interesting observations.

11.2.1 Failure Modes

In this section different types of failure modes are discussed. There was a distinct failure method for each of the 4 beam groups and they are presented below:

Beam A

The results of the control test performed on the glulam beam have revealed the typical mode of failure in a four-point bending scenario. The initiation of the crack was observed to occur in the lower laminate, followed by its progression into the adjacent layers of the timber. Subsequently, the crack propagated transversely along the length of the beam, in accordance with the orientation of the growth ring, resulting in a straight line crack. It is noteworthy to mention that the absence of any cracks along the glue-line signifies the robustness of the timber-timber connection and attests to its non-contributory role in the failure of the beam. The failure modes can be observed in figures 10.23 - 10.27.

Beam B

The failure mode of Beam B, which was reinforced with a 3mm diameter steel cable, proved to be the most surprising among all the beams tested. The steel cable caused internal shear within the beam, leading to a splitting of the glulam directly along the glue line in the tension layer. This result highlights the significance of carefully considering the appropriate diameter for the reinforcing steel cable in order to avoid such failure modes. The occurrence of internal shear failure, which was not anticipated, is a novel observation in this type of structure and represents a less desirable outcome compared to the tensile failure from bending. Although the deflection of the reinforced beams was less than what was predicted

by theoretical calculations, on average, these beams failed at a lower strength compared to the control beams. The failure along the glue line can be observed in Figure 11.1, where the steel cables have split from the steel-timber connection, exposing the cables.

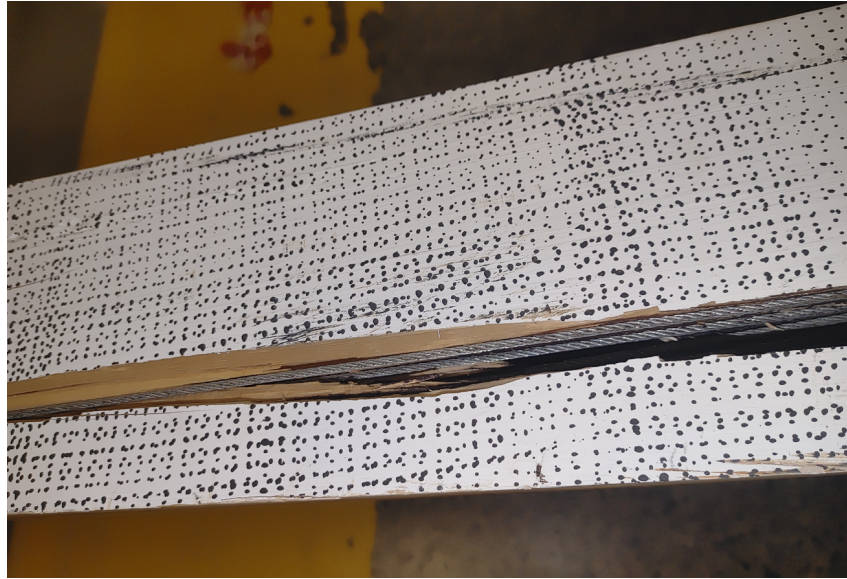


Figure 11.1: Failure Mode Beam B

Beam C

The failure mode of Beam C, reinforced with a 5mm diameter steel cable, was found to be comparable to that of the control test beams. The initiation and progression of cracks were observed to occur in the same region and spread throughout the beam in a similar manner. The similarity of the failure modes is further demonstrated by the close correspondence of the load-deflection values, seen in graph 10.6 and the Load-strain curves captured through the DIC software, as shown in graphs 10.8 and 10.10 .

Beam D

In the case of Beam D, which was reinforced with steel cables of the largest diameter (8mm), similar crack initiation and propagation patterns were observed as in Beams A and C. However, the cracks were observed to be narrower and the beams appeared to be slightly more intact post-failure. The results obtained from the DIC analysis (Figures 10.38 - 10.42) indicate a good distribution of high-strain points along the entire length of the tension portion of the beam, which suggests that the larger diameter steel cables effectively distribute the stresses uniformly across the beam, meaning that weak points such as knots and finger joints may become less critical. This could be the reason for the higher average load-deflection point observed in Graph 10.6. Furthermore, Graph 10.11 shows that the load-strain curves for Beam D are higher than those for the other beams.

11.2.2 Summary

The experimental results provided important insights in the behavior of glulam beams reinforced with steel cables under different loading conditions. It is clear that the use of steel cables with appropriate

diameter can improve the performance of glulam beams, increasing their strength and stiffness and may help distribute load more evenly along the length of the beam and removing the variability of knots and finger joints.

11.3 Computational Results

The finite element analysis (FEA) simulations were performed using the Abaqus software and a user material subroutine **UMAT** that was developed to incorporate the Continuum Damage Mechanics (CDM) model into the analysis. The FEA models were built to match the dimensions and boundary conditions of the experimental test specimens and the same loading conditions were applied to the models as in the experiments.

11.3.1 Outcome

The results of the FEA simulations were compared to the experimental results to evaluate the accuracy and reliability of the CDM model. The control test, where the beam was not reinforced, FEA results were found to be in good agreement with the experimental results. However, the FEA results for the reinforced beams struggled to accurately model the interaction between the timber and the steel cables. The discrepancies between the FEA results and the experimental results were mainly related to the prediction of the failure load. The CDM model tends to predict failure loads that are higher than the experimental results. Also, the CDM model may not fully capture all the non-linear behavior of the material and the structure, which can lead to some differences between the FEA results and the experimental results.

The FEA results also provided some additional insights into the behavior of the reinforced beams that were not visible in the experimental results. The FEA results showed the distribution of the damage in the beams, which helped to understand the failure modes and the failure mechanisms. However, the simulation of the interaction between the timber and the steel cables was not accurate, further research and development of the **UMAT** is needed to improve the accuracy of the simulation results.

11.3.2 Material Properties

The current **UMAT** utilized for predicting the failure of timber beams involves the use of 72 material properties. Although this approach results in accurate predictions for beams subjected to tension, compression, and bending, the challenge lies in obtaining the 72 material properties for each species of timber, which can be difficult. In the present research, several values for *Pinus Patula* were either assumed or drawn from other timber species due to the absence of available data. Other **UMATs** have been developed for modelling timber failure with a reduced number of material properties, such as the one proposed by Sandhaas (2012) that utilizes only 18 material properties while still producing accurate predictions. Hence, it would be beneficial to conduct a comparison between various **UMATs** to determine the optimal number of material properties that balance accuracy and feasibility.

11.4 Summary

The FEA simulations were conducted to evaluate the strength of reinforced timber beams using Abaqus and a UMAT model incorporating Continuum Damage Mechanics. Results showed good agreement for the control test but discrepancies for reinforced beams due to limitations of the CDM model. The current UMAT uses 72 material properties but reducing the number could improve feasibility.

Chapter 12

Conclusion and Recommendations

12.1 Conclusion

The research aimed to investigate the use of CDM to model glulam beams reinforced with steel cables, with the ultimate goal of encouraging the use of sustainable materials in construction by reducing the carbon footprint of the industry as well improving the overall strength of timber for its use in high rise-construction. The use of steel cables, with an appropriate diameter, as a reinforcement material for glulam beams not only improves their performance, but also offers a cost-effective alternative to other materials such as fiber reinforced polymers.

The study combined experimental testing and FEA using Abaqus and a UMAT to assess the feasibility and accuracy of CDM in predicting the mechanical behavior of these structures. The results of the experimental tests and FEA simulations were compared to evaluate the accuracy and reliability of the CDM model.

The experimental results showed that the use of steel cables with larger diameters can improve the performance of glulam beams, increasing their strength, as well as reduced the variability concern from knots and finger joints. However, thinner steel cables, produce an internal shear within the beam, causing the glulam to split directly along the glue line in the tension layer. This failure mode results in lower strength in the beam and should be avoided.

The results of the FEA simulations were found to be in good agreement with the experimental results for the control test, but was unable to accurately model the interaction between the timber and the steel cables for the reinforced beams. The discrepancies between the FEA results and the experimental results were mainly related to the prediction of the failure load. Abaqus tended to predict failure loads that were higher than that of the reinforced experimental results.

In summary, the research demonstrates the potential of using CDM to model glulam beams reinforced with steel cables and provides a step towards encouraging the use of sustainable materials in construction by reducing the carbon footprint of the industry.

12.2 Recommendations

1. Further research can focus on further improving the interaction modelling between the timber and the steel cables within the CDM model to increase its accuracy and reliability for use in industry as well as using larger diameter cables.
2. A comparative study should be conducted in order to determine an optimal number of material properties for the UMAT to remain feasible.
3. To further evaluate the cost-effectiveness and sustainability of using steel cables as a reinforcement material, a life-cycle analysis can be performed to compare the environmental impact and cost of steel cables to other reinforcement options such as FRPs.
4. Use a multi-camera DIC setup to more accurately track the strain and deflection readings in the experimental test. Strain gauges may also be attached directly to the internal steel cables to determine what is happening at the interface between the steel and timber.
5. To improve the performance of the reinforced beams, a tensile pre-stress force can be added to the steel cables, and anchorage can be used on the beams' ends to help the glue bond hold the pre-stress

force without letting the cable slip.

6. The **UMAT** code in Abaqus can be used to model a timber frame rather than just a beam, to verify the applicability of the CDM model on a larger scale.

Chapter 13

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Appendix A

Linking Abaqus 2019, Visual studio 2019 and Intel OneAPI

Introduction

The following is a procedure used to link Abaqus/CAE with Microsoft visual studio 2019 and Intel OneAPI. Recently Intel discontinued support and licensing of Intel Parallel Studio, so older methods to link fortran code with Abaqus are now outdated. The installation procedure provided will assume that you already have Abaqus/CAE 2019 installed on the working PC. The method detailed here is working as of October 13, 2023.

It should be noted that all previous versions of Microsoft visual studio and Intel Parallel Studio should be deleted prior to beginning the steps given here. This can be done using the removal tools provided by the respective companies.

Software Installation

In the following order install the software:

1. **Abaqus/CAE 2019** (Not the student version)
 - (a) Should already be installed if switching from parallel studio to OneAPI
 - (b) Otherwise, use the default installation
2. **Visual Studio community 2019**
 - (a) Available at:
https://visualstudio.microsoft.com/thank-you-downloading-visual-studio/?sku=Community&rel=16&src=myvs&utm_medium=microsoft&utm_source=my.visualstudio.com&utm_campaign=download&utm_content=vs+2019
 - (b) This should download the latest available version of the software. In the case where the link no longer works you will need to navigate through the <https://visualstudio.microsoft.com/> website to find your download.

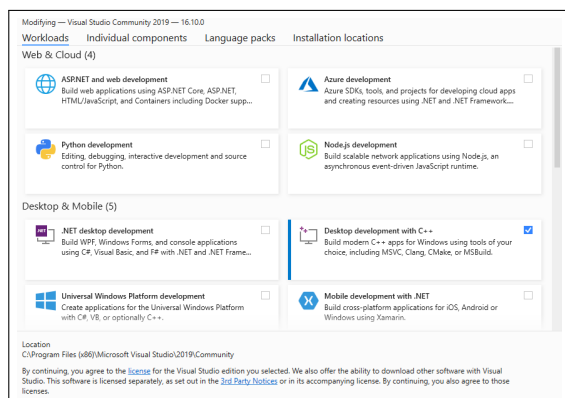


Figure A.1: Visual Studio Configuration

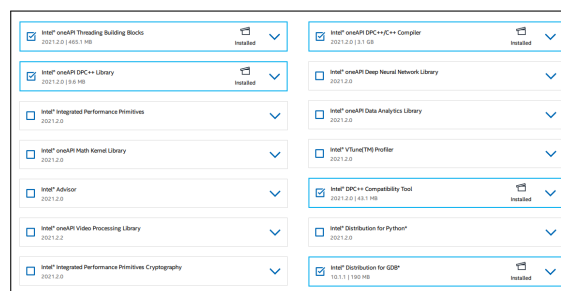


Figure A.2: OneAPI configuration

- (c) The default installation procedure is recommended however, if you prefer a custom installation to save disk space select the packages shown in figure A.1, namely 'Desktop development with C++'.

3. Intel OneAPI Base Toolkit

- (a) Available at:
<https://www.intel.com/content/www/us/en/developer/tools/oneapi/base-toolkit.html#gs.pu05qw>
- (b) Follow the prompts to download
- (c) Select the components shown in figure A.2

4. Intel OneAPI HPC Toolkit

- (a) Available at:
<https://software.intel.com/content/www/us/en/develop/tools/oneapi/hpc-toolkit/download.html?operatingsystem=window&distributions=webdownload>
- (b) Follow the default installation

Set the Environment Variable Path for the .bat files.

A new environment path needs to be set for the .bat files. More specifically this needs to be done for the files "*vcvarsall.bat*" and "*vars.bat*".

The paths for the two files should look something like this:

"vcvarsall.bat"

C:\Program Files (x86)\Microsoft Visual Studio\2019\Community\VC\Auxiliary\Build\vcvarsall.bat

"vars.bat"

C:\Program Files (x86)\Intel\oneAPI\compiler\2021.2.0\env\vars.bat

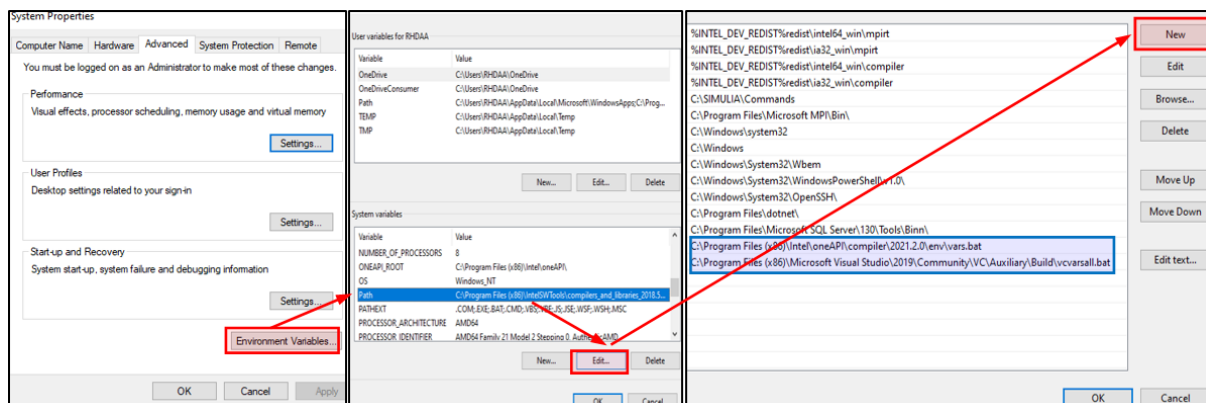


Figure A.3: How to change the environment paths

To do this follow the steps below and those shown in figure A.3.

1. Press the **windows key** and search **Path** in the search bar.
2. Open **Edit the system environment variables**
3. In the pop up window click **environment variables**
4. Search in the system variables section for **Path**
5. Click **Edit**
6. Then click **New** and paste the path for **vcvarsall.bat** and **vars.bat** separately. Make sure your path includes the names of the .bat files at the end (You may need to type these manually).

Setting up the Abaqus files

A new step that needs to be completed with the introduction of OneAPI is the configuration of the Abaqus files **abq2019.bat** and **abaqus__v6.env**.

First lets deal with the .bat file:

1. Locate the file at C:\SIMULIA\Commands
2. Right-click on the abq_2019.bat file
3. Select **edit** - Preferably with Notepad ++
4. Add the line: C:\Program Files (x86)\Intel\oneAPI\compiler\2021.2.0\env\vars.bat intel64 vs2019
5. Make sure you select the correct versions of your software (here 2021.2.0)
6. The edited file should look like figure A.4

```
1 @echo off
2 setlocal
3 set ABA_COMMAND=%~nx0
4 set ABA_COMMAND_FULL=%~f0
5 %call "C:\Program Files (x86)\Intel\oneAPI\compiler\2021.2.0\env\vars.bat" intel64 vs2019
6 "C:\SIMULIA\CAE\2019\win_b64\code\bin\ABQLauncher.exe" %*
7 endlocal
```

Figure A.4: Edited file for abq2019.bat

Now lets deal with the .env file:

1. **Locate** the file at:
C:\Program Files\Dassault Systemes\SimulationServices\V6R2019x\win_b64\SMA\site
2. **Right-click** on the abaqus__v6.env file
3. Select **edit** - Preferably with Notepad ++
4. At the bottom of the code add the line : compile_fortran +=['/names:lowercase',]
5. The edited portion of the file should look like figure A.5

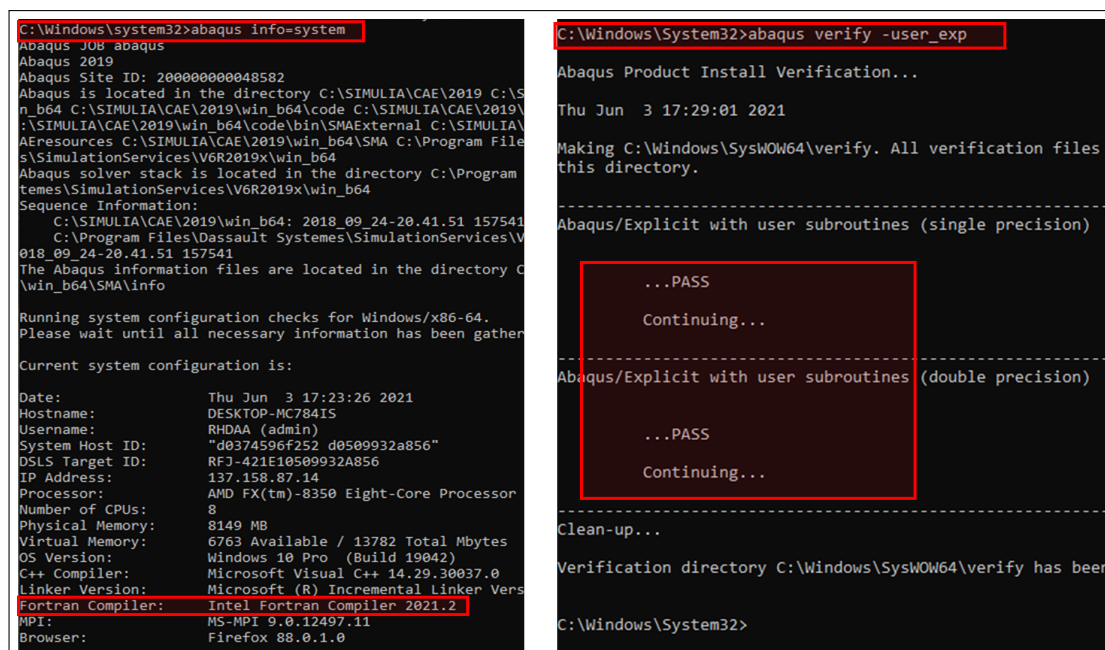
```
29 importEnv('graphicsConfig.env')
30
31 # Import site specific parameters such as licensing and doc parameters
32 importEnv('custom_v6.env')
33
34 del importEnv, driverUtils, platform
35
36 compile_fortran += ['/names:lowercase',]
```

Figure A.5: edited portion of the abaqus__v6.env file

Verification

Now to test whether the installation and linking procedures have all been done correctly there are some ways of verifying that everything is set up correctly before you begin working on a project. To do this you will need to:

1. Run Abaqus command with administrator rights
2. In the command prompt window that opens you can type the following:
 - (a) `abaqus info=system` (see figure A.6)
 - (b) `abaqus verify -user_std`
 - (c) `abaqus verify -user_exp` (see figure A.6)
 - (d) `abaqus verify -all`



The figure consists of two side-by-side screenshots of Windows command prompts. The left screenshot shows the command `C:\Windows\system32>abaqus info=system` and its output, which includes Abaqus version information, site ID, and a detailed system configuration table. The right screenshot shows the command `C:\Windows\System32>abaqus verify -user_exp` and its output, which includes the Abaqus Product Install Verification process, a timestamp, and verification results for single and double precision, both showing `...PASS`.

```
C:\Windows\system32>abaqus info=system
Abaqus JOB abaqus
Abaqus 2019
Abaqus Site ID: 200000000048582
Abaqus is located in the directory C:\SIMULIA\CAE\2019 C:\S
n_b64 C:\SIMULIA\CAE\2019\win_b64\code C:\SIMULIA\CAE\2019\
:\SIMULIA\CAE\2019\win_b64\code\bin\SMAExternal C:\SIMULIA\
AResources C:\SIMULIA\CAE\2019\win_b64\SMA C:\Program File
s\SimulationServices\V6R2019x\win_b64
Abaqus solver stack is located in the directory C:\Program
temes\SimulationServices\V6R2019x\win_b64
Sequence Information:
C:\SIMULIA\CAE\2019\win_b64: 2018_09_24-20.41.51 157541
C:\Program Files\Dassault Systemes\SimulationServices\V
018_09_24-20.41.51 157541
The Abaqus information files are located in the directory C
\win_b64\SMA\info

Running system configuration checks for Windows/x86-64.
Please wait until all necessary information has been gather

Current system configuration is:

Date: Thu Jun 3 17:23:26 2021
Hostname: DESKTOP-MC784IS
Username: RHDA (admin)
System Host ID: "d0374596f252 d0509932a856"
DSLS Target ID: RFJ-421E10509932A856
IP Address: 137.158.87.14
Processor: AMD FX(tm)-8350 Eight-Core Processor
Number of CPUs: 8
Physical Memory: 8149 MB
Virtual Memory: 6763 Available / 13782 Total Mbytes
OS Version: Windows 10 Pro (Build 19042)
C++ Compiler: Microsoft Visual C++ 14.29.30037.0
Linker Version: Microsoft (R) Incremental Linker Vers
Fortran Compiler: Intel Fortran Compiler 2021.2
MPI: MS-MPI 9.0.12497.11
Browser: Firefox 88.0.1.0

C:\Windows\System32>abaqus verify -user_exp
Abaqus Product Install Verification...
Thu Jun 3 17:29:01 2021

Making C:\Windows\SysWOW64\verify. All verification files
this directory.

-----
Abaqus/Explicit with user subroutines (single precision)

...PASS
Continuing...

-----
Abaqus/Explicit with user subroutines (double precision)

...PASS
Continuing...

-----
Clean-up...

Verification directory C:\Windows\SysWOW64\verify has been
C:\Windows\System32>
```

Figure A.6: Abaqus linking verification

If the command prompt window returns similar to that of figure A.6 then you have successfully linked the softwares of Abaqus/CAE 2019, Visual studio 2019 and Intel OneAPI in windows 10 x64

Appendix B

UMAT code

```
! Wood material subroutine (UMAT) based on continuum damage mechanics
! Code is partial, for the full code contact the authors
! Hamid Valipour, Gharib et al., 2017. and Nouri et al. (2019).
! -----
! Note: This subroutine is written for a 3D orthotropic problem
!       with orientations as: X = 1 = P, Y = 2 = R, Z = 3 = T
!       where: P = parallel, R = radial , T = tangential
! -----
! Index definition:
! x: t = tension, c = compression
! y: P = parallel, R = radial , T = tangential
! -----

SUBROUTINE UMAT( STRESS, STATEV, DDSDDDE, SSE, SPD, SCD, RPL,
1      DDSDDT, DRPLDE, DRPLDT, STRAN, DSTRAN, TIME, DTIME,
2      TEMP, DTEMP, PREDEF, DPRED, CMNAME, NDI, NSHR, NTENS,
3      NSTATV, PROPS, NPROPS, COORDS, DROT, PNEWDT, CELENT,
4      DFGRD0, DFGRD1, NOEL, NPT, LAYER, KSPT, KSTEP, KINC)

! -----
! DEFINING TYPES OF TO-BE-USED ARRAYS & VARIABLES
! -----

INCLUDE 'ABA_PARAM.INC'

!IMPLICIT NONE - does not work within ABAQUS

CHARACTER*80 CMNAME
```

```

DIMENSION STRESS(NTENS), STATEV(NSTATV), DDSDE(NTENS,NTENS),
1          DDSDDT(NTENS), DRPLDE(NTENS), STRAN(NTENS),
2          DSTRAN(NTENS), TIME(2), PREDEF(1), DPRED(1),
3          PROPS(NPROPS), COORDS(3), DROT(3,3), DFGRD0(3,3),
4          DFGRD1(3,3)

DIMENSION STRANT(NTENS), damMATRIX(NTENS,NTENS), STRESS_NO(NTENS),
1          elMATRIX(NTENS,NTENS), OLD_STRESS_NO(NTENS),
2          C(NTENS,NTENS), STREST(NTENS), OLD_STRESS(NTENS)

PARAMETER (ZERO=0.00, HALF=0.500, ONE=1.00, TWO=2.00, ERR=.0100)

DOUBLE PRECISION STRAN, DSTRAN, STRANT, STRESS, OLD_STRESS_NO,
1          DDSDE, damMATRIX, elMATRIX, STRESS_NO, C, STREST

INTEGER:: I, J

REAL (KIND=8):: CELENT, DTIME, LC1, LC2, DELTA, E11, E22, E33,
+      G12, G13, G23, nu12, nu13, nu23, nu21, nu31, nu32,
+      Sigtp, Sigcp, Sigtr, Sigcr, Sigtt, Sigct, Sigvr, Sigvt, Sigrt,
+      Sigu_tp, Epsu_tp, Gu_tp, Epsr_tp, Alf_tp, Bet_tp, Gf_tp,
+      EtP_els, EtP_hrd, EtP_sft, EtP_res, ktP_u, ktP_s, ETA_tp,
+      Sigu_tr, Epsu_tr, Gu_tr, Epsr_tr, Alf_tr, Bet_tr, Gf_tr,
+      EtR_els, EtR_hrd, EtR_sft, EtR_res, ktR_u, ktR_s, ETA_tr,
+      Sigu_tT, Epsu_tT, Gu_tT, Epsr_tT, Alf_tT, Bet_tT, Gf_tT,
+      EtT_els, EtT_hrd, EtT_sft, EtT_res, ktT_u, ktT_s, ETA_tT,
+      d_tp, d_tr, d_tT, d_tp_old, d_tr_old, d_tT_old,
+      d_tp_V, d_tr_V, d_tT_V, d_tp_V_old, d_tr_V_old, d_tT_V_old,
+      Sigu_cp, Epsu_cp, Gu_cp, Epsr_cp, Alf_cp, Bet_cp, Gf_cp,
+      EcP_els, EcP_hrd, EcP_sft, EcP_res, kcP_u, kcP_s, ETA_cp,
+      Sigu_cr, Epsu_cr, Gu_cr, Epsr_cr, Alf_cr, Bet_cr, Gf_cr,
+      EcR_els, EcR_hrd, EcR_sft, EcR_res, kcR_u, kcR_s, ETA_cr,
+      Sigu_cT, Epsu_cT, Gu_cT, Epsr_cT, Alf_cT, Bet_cT, Gf_cT,
+      EcT_els, EcT_hrd, EcT_sft, EcT_res, kcT_u, kcT_s, ETA_cT,
+      d_cp, d_cr, d_cT, d_cp_old, d_cr_old, d_cT_old,
+      d_cp_V, d_cr_V, d_cT_V, d_cp_V_old, d_cr_V_old, d_cT_V_old,
+      Sigu_vR, Epsu_vR, Gu_vR, Epsr_vR, Alf_vR, Bet_vR, Gf_vR,
+      EvR_els, EvR_hrd, EvR_sft, EvR_res, kvR_u, kvR_s, ETA_vR,
+      Sigu_vT, Epsu_vT, Gu_vT, Epsr_vT, Alf_vT, Bet_vT, Gf_vT,
+      EvT_els, EvT_hrd, EvT_sft, EvT_res, kvT_u, kvT_s, ETA_vT,
+      Sigu_RT, Epsu_RT, Gu_RT, Epsr_RT, Alf_RT, Bet_RT, Gf_RT,

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+      ERT_els, ERT_hrd, ERT_sft, ERT_res, kRT_u, kRT_s, ETA_RT,
+      d_vR, d_vT, d_vR_old, d_vT_old, d_RT, d_RT_old,
+      d_vR_V, d_vT_V, d_vR_V_old, d_vT_V_old, d_RT_V, d_RT_V_old,
+      F_tP, F_cP, F_tR, F_cR, F_tT, F_cT, F_vR, F_vT,
+      his_tP, his_tR, his_tT, his_tP_old, his_tR_old, his_tT_old,
+      his_cP, his_cR, his_cT, his_cP_old, his_cR_old, his_cT_old,
+      his_vR, his_vT, his_vR_old, his_vT_old

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! -----
! TAKING & CHECKING USER-DEFINED INPUT PARAMETERS
! -----

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! ELASTIC PROPERTIES

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E11 = PROPS( 1)      ! parallel to grain MoE (EPP)
E22 = PROPS( 2)      ! perp. to grain MoE - radial (ERR)
E33 = PROPS( 3)      ! perp. to grain MoE - tangential (ETT)

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G12 = PROPS( 4)      ! splitting (PR) shear modulus (GPR)
G13 = PROPS( 5)      ! splitting (PT) shear modulus (GPT)
G23 = PROPS( 6)      ! rolling (RT) shear modulus (GRT)

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nu12 = PROPS( 7)      ! Poisson's (PR) ratio (nPR)
nu13 = PROPS( 8)      ! Poisson's (PT) ratio (nPT)
nu23 = PROPS( 9)      ! Poisson's (RT) ratio (nRT)

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nu21 = nu12 * E22 / E11
nu31 = nu13 * E33 / E11
nu32 = nu23 * E33 / E22

```

! ULTIMATE STRENGTH (Sigu)

! Sigu HAS TO BE EQUAL TO YIELD STRESS (Sigy) TO DEACTIVATE HARDENING

```

Sigu_tP = PROPS( 10) ! tensile strength parallel to grain (ftP)
Sigu_tR = PROPS( 11) ! tensile strength perp. to grain - radial (ftR)
Sigu_tT = PROPS( 12) ! tensile strength perp. to grain - tangential (ftT)

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Sigu_cP = PROPS( 13) ! comp. strength parallel to grain (fcP)
Sigu_cR = PROPS( 14) ! comp. strength perp. to grain - radial (fcR)
Sigu_cT = PROPS( 15) ! comp. strength perp. to grain - tangential (fcT)

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Sigu_vR = PROPS( 16) ! splitting (PR) shear strength (fvR)
Sigu_vT = PROPS( 17) ! splitting (PT) shear strength (fvT)
Sigu_RT = PROPS( 18) ! rolling (RT) shear strength (frT)

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! STRAIN AT ULTIMATE STRENGTH (Epsu)
! Epsu HAS TO EQUAL YIELD STRAIN (Epsy=Sigy/E) TO DEACTIVATE HARDENING
! Epsu HAS TO BE DEFINED BIG ENOUGH TO AVOID SOFTENING
Epsu_tP = PROPS( 19)
Epsu_tR = PROPS( 20)
Epsu_tT = PROPS( 21)

Epsu_cP = PROPS( 22)
Epsu_cR = PROPS( 23)
Epsu_cT = PROPS( 24)

Epsu_vR = PROPS( 25)
Epsu_vT = PROPS( 26)
Epsu_RT = PROPS( 27)

! CONSUMED ENERGY UP TO ULTIMATE STRENGTH (Gu) FOR HARDENING CALCS
! Gu SHOULD NOT EXCEED A MAXIMUM LIMIT (GuMAX=Fu.Epsu-0.5.Fu^2/E)
! Gu HAS TO BE EQUAL TO GuMAX TO ACTIVATE PERFECT PLASTIC BEHAVIOR
Gu_tP = PROPS( 28)
EtP_els = E11
LC1 = EtP_els * Epsu_tP / Sigu_tP - ONE
LC2 = Sigu_tP * (Epsu_tP - HALF * Sigu_tP / EtP_els) / Gu_tP - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    Sig_tP = Sigu_tP
    EtP_hrd = EtP_els
    ktP_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    Sig_tP = Sigu_tP
    EtP_hrd = ZERO
    ktP_u = EtP_els * Epsu_tP / Sigu_tP
ELSE
    Sig_tP = ( TWO * Gu_tP - Epsu_tP * Sigu_tP)
1      / ( Epsu_tP - Sigu_tP / EtP_els)
    EtP_hrd = ( Sigu_tP - Sig_tP) / ( Epsu_tP - Sig_tP / EtP_els)
    ktP_u = EtP_els * Epsu_tP / Sig_tP
END IF

Gu_tR = PROPS( 29)
EtR_els = E22
LC1 = EtR_els * Epsu_tR / Sigu_tR - ONE
LC2 = Sigu_tR * (Epsu_tR - HALF * Sigu_tR / EtR_els) / Gu_tR - ONE

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IF ( DABS( LC1) .LE. ERR) THEN
    SigtR = Sigu_tR
    EtR_hrd = EtR_els
    ktR_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigtR = Sigu_tR
    EtR_hrd = ZERO
    ktR_u = EtR_els * Epsu_tR / Sigu_tR
ELSE
    SigtR = ( TWO * Gu_tR - Epsu_tR * Sigu_tR)
1      / ( Epsu_tR - Sigu_tR / EtR_els)
    EtR_hrd = ( Sigu_tR - SigtR) / ( Epsu_tR - SigtR / EtR_els)
    ktR_u = EtR_els * Epsu_tR / SigtR
END IF

Gu_tT = PROPS( 30)
EtT_els = E33
LC1 = EtT_els * Epsu_tT / Sigu_tT - ONE
LC2 = Sigu_tT * (Epsu_tT - HALF * Sigu_tT / EtT_els) / Gu_tT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigtT = Sigu_tT
    EtT_hrd = EtT_els
    ktT_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigtT = Sigu_tT
    EtT_hrd = ZERO
    ktT_u = EtT_els * Epsu_tT / Sigu_tT
ELSE
    SigtT = ( TWO * Gu_tT - Epsu_tT * Sigu_tT)
1      / ( Epsu_tT - Sigu_tT / EtT_els)
    EtT_hrd = ( Sigu_tT - SigtT) / ( Epsu_tT - SigtT / EtT_els)
    ktT_u = EtT_els * Epsu_tT / SigtT
END IF

Gu_cP = PROPS( 31)
EcP_els = E11
LC1 = EcP_els * Epsu_cP / Sigu_cP - ONE
LC2 = Sigu_cP * (Epsu_cP - HALF * Sigu_cP / EcP_els) / Gu_cP - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigcP = Sigu_cP
    EcP_hrd = EcP_els
    kcP_u = ONE

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ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigcP = Sigu_cP
    EcP_hrd = ZERO
    kcP_u = EcP_els * Epsu_cP / Sigu_cP
ELSE
    SigcP = ( TWO * Gu_cP - Epsu_cP * Sigu_cP)
1      / ( Epsu_cP - Sigu_cP / EcP_els)
    EcP_hrd = ( Sigu_cP - SigcP) / ( Epsu_cP - SigcP / EcP_els)
    kcP_u = EcP_els * Epsu_cP / SigcP
END IF

Gu_cR = PROPS( 32)
EcR_els = E22
LC1 = EcR_els * Epsu_cR / Sigu_cR - ONE
LC2 = Sigu_cR * (Epsu_cR - HALF * Sigu_cR / EcR_els) / Gu_cR - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigcR = Sigu_cR
    EcR_hrd = EcR_els
    kcR_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigcR = Sigu_cR
    EcR_hrd = ZERO
    kcR_u = EcR_els * Epsu_cR / Sigu_cR
ELSE
    SigcR = ( TWO * Gu_cR - Epsu_cR * Sigu_cR)
1      / ( Epsu_cR - Sigu_cR / EcR_els)
    EcR_hrd = ( Sigu_cR - SigcR) / ( Epsu_cR - SigcR / EcR_els)
    kcR_u = EcR_els * Epsu_cR / SigcR
END IF

Gu_cT = PROPS( 33)
EcT_els = E33
LC1 = EcT_els * Epsu_cT / Sigu_cT - ONE
LC2 = Sigu_cT * (Epsu_cT - HALF * Sigu_cT / EcT_els) / Gu_cT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigcT = Sigu_cT
    EcT_hrd = EcT_els
    kcT_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigcT = Sigu_cT
    EcT_hrd = ZERO
    kcT_u = EcT_els * Epsu_cT / Sigu_cT

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ELSE
    SigcT = ( TWO * Gu_cT - Epsu_cT * Sigu_cT)
1      / ( Epsu_cT - Sigu_cT / EcT_els)
    EcT_hrd = ( Sigu_cT - SigcT) / ( Epsu_cT - SigcT / EcT_els)
    kcT_u = EcT_els * Epsu_cT / SigcT
END IF

Gu_vR = PROPS( 34)
EvR_els = G12
LC1 = EvR_els * Epsu_vR / Sigu_vR - ONE
LC2 = Sigu_vR * (Epsu_vR - HALF * Sigu_vR / EvR_els) / Gu_vR - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigvR = Sigu_vR
    EvR_hrd = EvR_els
    kvR_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigvR = Sigu_vR
    EvR_hrd = ZERO
    kvR_u = EvR_els * Epsu_vR / Sigu_vR
ELSE
    SigvR = ( TWO * Gu_vR - Epsu_vR * Sigu_vR)
1      / ( Epsu_vR - Sigu_vR / EvR_els)
    EvR_hrd = ( Sigu_vR - SigvR) / ( Epsu_vR - SigvR / EvR_els)
    kvR_u = EvR_els * Epsu_vR / SigvR
END IF

Gu_vT = PROPS( 35)
EvT_els = G13
LC1 = EvT_els * Epsu_vT / Sigu_vT - ONE
LC2 = Sigu_vT * (Epsu_vT - HALF * Sigu_vT / EvT_els) / Gu_vT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigvT = Sigu_vT
    EvT_hrd = EvT_els
    kvT_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigvT = Sigu_vT
    EvT_hrd = ZERO
    kvT_u = EvT_els * Epsu_vT / Sigu_vT
ELSE
    SigvT = ( TWO * Gu_vT - Epsu_vT * Sigu_vT)
1      / ( Epsu_vT - Sigu_vT / EvT_els)
    EvT_hrd = ( Sigu_vT - SigvT) / ( Epsu_vT - SigvT / EvT_els)

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      kvT_u = EvT_els * Epsu_vT / SigvT
END IF

Gu_RT = PROPS( 36)
ERT_els = G23
LC1 = ERT_els * Epsu_RT / Sigu_RT - ONE
LC2 = Sigu_RT * (Epsu_RT - HALF * Sigu_RT / ERT_els) / Gu_RT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    SigRT = Sigu_RT
    ERT_hrd = ERT_els
    kRT_u = ONE
ELSE IF ( DABS( LC2) .LE. ERR) THEN
    SigRT = Sigu_RT
    ERT_hrd = ZERO
    kRT_u = ERT_els * Epsu_RT / Sigu_RT
ELSE
    SigRT = ( TWO * Gu_RT - Epsu_RT * Sigu_RT)
1      / ( Epsu_RT - Sigu_RT / ERT_els)
    ERT_hrd = ( Sigu_RT - SigRT) / ( Epsu_RT - SigRT / ERT_els)
    kRT_u = ERT_els * Epsu_RT / SigRT
END IF

! FRACTURE ENERGY FOR COMPLETE FAILURE (Gf)
! Gf HAS TO BE GREATER THAN Gu (Gf>Gu)
! Gf HAS TO BE BIG ENOUGH TO AVOID COMPLETE FAILURE
Gf_tP = PROPS( 37)
Gf_tR = PROPS( 38)
Gf_tT = PROPS( 39)

Gf_cP = PROPS( 40)
Gf_cR = PROPS( 41)
Gf_cT = PROPS( 42)

Gf_vR = PROPS( 43)
Gf_vT = PROPS( 44)
Gf_RT = PROPS( 45)

! RESIDUAL STRAIN ENERGY RATIO (TO POST-PEAK) AFTER KINK (0<Bet<1)
Bet_tP = PROPS( 46)
Bet_tR = PROPS( 47)
Bet_tT = PROPS( 48)

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Bet_cP = PROPS( 49)
Bet_cR = PROPS( 50)
Bet_cT = PROPS( 51)

Bet_vR = PROPS( 52)
Bet_vT = PROPS( 53)
Bet_RT = PROPS( 54)

! RESIDUAL STRENGTH RATIO (TO ULTIMATE) AT KINKING POINT (0<Alf<1)
! Alf HAS TO BE EQUAL TO Bet^0.5 TO ACTIVATE LINEAR SOFTENING
! Alf HAS TO BE LESS THAN Bet^0.5 TO ACTIVATE BI-LINEAR SOFTENING
Alf_tP = PROPS( 55)
LC1 = (Bet_tP ** HALF) / Alf_tP - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    EtP_sft = (Sigu_tP ** TWO) / (TWO * (Gf_tP - Gu_tP) / CELENT)
    EtP_res = EtP_sft
    Epsr_tP = Epsu_tP + TWO * ((Gf_tP - Gu_tP) / CELENT)
1      * (ONE - Alf_tP) / Sigu_tP
    ktP_s = EtP_els * Epsr_tP / SigtP
ELSE
    EtP_sft = (ONE - Alf_tP ** TWO) * (Sigu_tP ** TWO)
1      / (TWO * (ONE - Bet_tP) * (Gf_tP - Gu_tP) / CELENT)
    EtP_res = (Alf_tP ** TWO) * (Sigu_tP ** TWO)
1      / (TWO * Bet_tP * (Gf_tP - Gu_tP) / CELENT)
    Epsr_tP = Epsu_tP + TWO * ((Gf_tP - Gu_tP) / CELENT)
1      * (ONE - Bet_tP) / ((Alf_tP + ONE) * Sigu_tP)
    ktP_s = EtP_els * Epsr_tP / SigtP
END IF

Alf_tR = PROPS( 56)
LC1 = (Bet_tR ** HALF) / Alf_tR - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    EtR_sft = (Sigu_tR ** TWO) / (TWO * (Gf_tR - Gu_tR) / CELENT)
    EtR_res = EtR_sft
    Epsr_tR = Epsu_tR + TWO * ((Gf_tR - Gu_tR) / CELENT)
1      * (ONE - Alf_tR) / Sigu_tR
    ktr_s = EtR_els * Epsr_tR / SigtR
ELSE
    EtR_sft = (ONE - Alf_tR ** TWO) * (Sigu_tR ** TWO)
1      / (TWO * (ONE - Bet_tR) * (Gf_tR - Gu_tR) / CELENT)
    EtR_res = (Alf_tR ** TWO) * (Sigu_tR ** TWO)
1      / (TWO * Bet_tR * (Gf_tR - Gu_tR) / CELENT)

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      Epsr_tR = Epsu_tR + TWO * ((Gf_tR - Gu_tR) / CELENT)
1      * (ONE - Bet_tR) / ((Alf_tR + ONE) * Sigu_tR)
      ktR_s = EtR_els * Epsr_tR / SigtR
END IF

Alf_tT = PROPS( 57)
LC1 = (Bet_tT ** HALF) / Alf_tT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
      EtT_sft = (Sigu_tT ** TWO) / (TWO * (Gf_tT - Gu_tT) / CELENT)
      EtT_res = EtT_sft
      Epsr_tT = Epsu_tT + TWO * ((Gf_tT - Gu_tT) / CELENT)
1      * (ONE - Alf_tT) / Sigu_tT
      ktT_s = EtT_els * Epsr_tT / SigtT
ELSE
      EtT_sft = (ONE - Alf_tT ** TWO) * (Sigu_tT ** TWO)
1      / (TWO * (ONE - Bet_tT) * (Gf_tT - Gu_tT) / CELENT)
      EtT_res = (Alf_tT ** TWO) * (Sigu_tT ** TWO)
1      / (TWO * Bet_tT * (Gf_tT - Gu_tT) / CELENT)
      Epsr_tT = Epsu_tT + TWO * ((Gf_tT - Gu_tT) / CELENT)
1      * (ONE - Bet_tT) / ((Alf_tT + ONE) * Sigu_tT)
      ktT_s = EtT_els * Epsr_tT / SigtT
END IF

Alf_cP = PROPS( 58)
LC1 = (Bet_cP ** HALF) / Alf_cP - ONE
IF ( DABS( LC1) .LE. ERR) THEN
      EcP_sft = (Sigu_cP ** TWO) / (TWO * (Gf_cP - Gu_cP) / CELENT)
      EcP_res = EcP_sft
      Epsr_cP = Epsu_cP + TWO * ((Gf_cP - Gu_cP) / CELENT)
1      * (ONE - Alf_cP) / Sigu_cP
      kcP_s = EcP_els * Epsr_cP / SigcP
ELSE
      EcP_sft = (ONE - Alf_cP ** TWO) * (Sigu_cP ** TWO)
1      / (TWO * (ONE - Bet_cP) * (Gf_cP - Gu_cP) / CELENT)
      EcP_res = (Alf_cP ** TWO) * (Sigu_cP ** TWO)
1      / (TWO * Bet_cP * (Gf_cP - Gu_cP) / CELENT)
      Epsr_cP = Epsu_cP + TWO * ((Gf_cP - Gu_cP) / CELENT)
1      * (ONE - Bet_cP) / ((Alf_cP + ONE) * Sigu_cP)
      kcP_s = EcP_els * Epsr_cP / SigcP
END IF

Alf_cR = PROPS( 59)

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LC1 = (Bet_cR ** HALF) / Alf_cR - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    EcR_sft = (Sigu_cR ** TWO) / (TWO * (Gf_cR - Gu_cR) / CELENT)
    EcR_res = EcR_sft
    Epsr_cR = Epsu_cR + TWO * ((Gf_cR - Gu_cR) / CELENT)
1      * (ONE - Alf_cR) / Sigu_cR
    kcR_s = EcR_els * Epsr_cR / SigcR
ELSE
    EcR_sft = (ONE - Alf_cR ** TWO) * (Sigu_cR ** TWO)
1      / (TWO * (ONE - Bet_cR) * (Gf_cR - Gu_cR) / CELENT)
    EcR_res = (Alf_cR ** TWO) * (Sigu_cR ** TWO)
1      / (TWO * Bet_cR * (Gf_cR - Gu_cR) / CELENT)
    Epsr_cR = Epsu_cR + TWO * ((Gf_cR - Gu_cR) / CELENT)
1      * (ONE - Bet_cR) / ((Alf_cR + ONE) * Sigu_cR)
    kcR_s = EcR_els * Epsr_cR / SigcR
END IF

Alf_cT = PROPS( 60)
LC1 = (Bet_cT ** HALF) / Alf_cT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    EcT_sft = (Sigu_cT ** TWO) / (TWO * (Gf_cT - Gu_cT) / CELENT)
    EcT_res = EcT_sft
    Epsr_cT = Epsu_cT + TWO * ((Gf_cT - Gu_cT) / CELENT)
1      * (ONE - Alf_cT) / Sigu_cT
    kcT_s = EcT_els * Epsr_cT / SigcT
ELSE
    EcT_sft = (ONE - Alf_cT ** TWO) * (Sigu_cT ** TWO)
1      / (TWO * (ONE - Bet_cT) * (Gf_cT - Gu_cT) / CELENT)
    EcT_res = (Alf_cT ** TWO) * (Sigu_cT ** TWO)
1      / (TWO * Bet_cT * (Gf_cT - Gu_cT) / CELENT)
    Epsr_cT = Epsu_cT + TWO * ((Gf_cT - Gu_cT) / CELENT)
1      * (ONE - Bet_cT) / ((Alf_cT + ONE) * Sigu_cT)
    kcT_s = EcT_els * Epsr_cT / SigcT
END IF

Alf_vR = PROPS( 61)
LC1 = (Bet_vR ** HALF) / Alf_vR - ONE
IF ( DABS( LC1) .LE. ERR) THEN
    EvR_sft = (Sigu_vR ** TWO) / (TWO * (Gf_vR - Gu_vR) / CELENT)
    EvR_res = EvR_sft
    Epsr_vR = Epsu_vR + TWO * ((Gf_vR - Gu_vR) / CELENT)
1      * (ONE - Alf_vR) / Sigu_vR

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kvR_s = EvR_els * Epsr_vR / SigvR
ELSE
  EvR_sft = (ONE - Alf_vR ** TWO) * (Sigu_vR ** TWO)
1      / (TWO * (ONE - Bet_vR) * (Gf_vR - Gu_vR) / CELENT)
  EvR_res = (Alf_vR ** TWO) * (Sigu_vR ** TWO)
1      / (TWO * Bet_vR * (Gf_vR - Gu_vR) / CELENT)
  Epsr_vR = Epsu_vR + TWO * ((Gf_vR - Gu_vR) / CELENT)
1      * (ONE - Bet_vR) / ((Alf_vR + ONE) * Sigu_vR)
  kvR_s = EvR_els * Epsr_vR / SigvR
END IF

Alf_vT = PROPS( 62)
LC1 = (Bet_vT ** HALF) / Alf_vT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
  EvT_sft = (Sigu_vT ** TWO) / (TWO * (Gf_vT - Gu_vT) / CELENT)
  EvT_res = EvT_sft
  Epsr_vT = Epsu_vT + TWO * ((Gf_vT - Gu_vT) / CELENT)
1      * (ONE - Alf_vT) / Sigu_vT
  kvT_s = EvT_els * Epsr_vT / SigvT
ELSE
  EvT_sft = (ONE - Alf_vT ** TWO) * (Sigu_vT ** TWO)
1      / (TWO * (ONE - Bet_vT) * (Gf_vT - Gu_vT) / CELENT)
  EvT_res = (Alf_vT ** TWO) * (Sigu_vT ** TWO)
1      / (TWO * Bet_vT * (Gf_vT - Gu_vT) / CELENT)
  Epsr_vT = Epsu_vT + TWO * ((Gf_vT - Gu_vT) / CELENT)
1      * (ONE - Bet_vT) / ((Alf_vT + ONE) * Sigu_vT)
  kvT_s = EvT_els * Epsr_vT / SigvT
END IF

Alf_RT = PROPS( 63)
LC1 = (Bet_RT ** HALF) / Alf_RT - ONE
IF ( DABS( LC1) .LE. ERR) THEN
  ERT_sft = (Sigu_RT ** TWO) / (TWO * (Gf_RT - Gu_RT) / CELENT)
  ERT_res = ERT_sft
  Epsr_RT = Epsu_RT + TWO * ((Gf_RT - Gu_RT) / CELENT)
1      * (ONE - Alf_RT) / Sigu_RT
  kRT_s = ERT_els * Epsr_RT / SigRT
ELSE
  ERT_sft = (ONE - Alf_RT ** TWO) * (Sigu_RT ** TWO)
1      / (TWO * (ONE - Bet_RT) * (Gf_RT - Gu_RT) / CELENT)
  ERT_res = (Alf_RT ** TWO) * (Sigu_RT ** TWO)
1      / (TWO * Bet_RT * (Gf_RT - Gu_RT) / CELENT)

```

```

      Epsr_RT = Epsu_RT + TWO * ((Gf_RT - Gu_RT) / CELENT)
1      * (ONE - Bet_RT) / ((Alf_RT + ONE) * Sigu_RT)
      kRT_s = ERT_els * Epsr_RT / SigRT
END IF

! VISCOSITY FOR REGULARISATION (ETA)
! ETA HAS TO BE ADJUSTED DEPENDING ON THE OBSERVED MESH SENSITIVITY
ETA_tP = PROPS( 64)
ETA_tR = PROPS( 65)
ETA_tT = PROPS( 66)

ETA_cP = PROPS( 67)
ETA_cR = PROPS( 68)
ETA_cT = PROPS( 69)

ETA_vR = PROPS( 70)
ETA_vT = PROPS( 71)
ETA_RT = PROPS( 72)

!-----
! CREATE ELASTIC ORTHOTROPIC STIFFNESS MATRIX
!-----

DO I = 1, NTENS
  DO J = 1, NTENS
    elMATRIX( I, J) = ZERO
  END DO
END DO

DELTA = 1 / ( ONE - nu12 * nu21 - nu13 * nu31
1      - nu23 * nu32 - TWO * nu31 * nu12 * nu23)
elMATRIX( 1, 1) = ( ONE - nu23 * nu32) * DELTA * E11
elMATRIX( 2, 2) = ( ONE - nu13 * nu31) * DELTA * E22
elMATRIX( 3, 3) = ( ONE - nu12 * nu21) * DELTA * E33
elMATRIX( 1, 2) = ( nu12 + nu13 * nu32) * DELTA * E22
elMATRIX( 1, 3) = ( nu13 + nu12 * nu23) * DELTA * E33
elMATRIX( 2, 3) = ( nu23 + nu21 * nu13) * DELTA * E33
elMATRIX( 2, 1) = elMATRIX( 1, 2)
elMATRIX( 3, 1) = elMATRIX( 1, 3)
elMATRIX( 3, 2) = elMATRIX( 2, 3)
elMATRIX( 4, 4) = G12
elMATRIX( 5, 5) = G13

```

```
elMATRIX( 6, 6) = G23
```

```
!-----  
! CREATE SOLUTION-DEPENDENT STATE VARIABLES  
!-----
```

```
! Damage variables - not regularised
```

```
d_tP_old = STATEV( 1)    ! tension parallel to grain  
d_cP_old = STATEV( 2)    ! comp. parallel to grain  
d_tR_old = STATEV( 3)    ! tension perp. to grain - radial  
d_cR_old = STATEV( 4)    ! comp. perp. to grain - radial  
d_tT_old = STATEV( 5)    ! tension perp. to grain - tangential  
d_cT_old = STATEV( 6)    ! comp. perp. to grain - tangential  
d_vR_old = STATEV( 7)    ! splitting PR shear  
d_vT_old = STATEV( 8)    ! splitting PT shear  
d_RT_old = STATEV( 9)    ! rolling RT shear
```

```
! Damage variables - regularised
```

```
d_tP_V_old = STATEV( 10)  
d_cP_V_old = STATEV( 11)  
d_tR_V_old = STATEV( 12)  
d_cR_V_old = STATEV( 13)  
d_tT_V_old = STATEV( 14)  
d_cT_V_old = STATEV( 15)  
d_vR_V_old = STATEV( 16)  
d_vT_V_old = STATEV( 17)  
d_RT_V_old = STATEV( 18)
```

```
! Damaged matrix arrays for internal checks
```

```
damMATRIX( 1, 1) = STATEV( 19)  
damMATRIX( 2, 2) = STATEV( 20)  
damMATRIX( 3, 3) = STATEV( 21)  
damMATRIX( 4, 4) = STATEV( 22)  
damMATRIX( 5, 5) = STATEV( 23)  
damMATRIX( 6, 6) = STATEV( 24)  
damMATRIX( 1, 2) = STATEV( 25)  
damMATRIX( 1, 3) = STATEV( 26)  
damMATRIX( 2, 3) = STATEV( 27)
```

```
! History parameters
```

```
his_tP_old = STATEV( 28)  
his_cP_old = STATEV( 29)
```

```
his_tR_old = STATEV( 30)
his_cR_old = STATEV( 31)
his_tT_old = STATEV( 32)
his_cT_old = STATEV( 33)
his_vR_old = STATEV( 34)
his_vT_old = STATEV( 35)
```

```
! -----
```

```
RETURN
```

```
END
```

```
! -----
```

```
! -----
```

Appendix C

Job Safety Analysis

Table C.1: Production and Delivery JSA

Student name:	Aaron Rhodes	Date:
Student Number:	RHDAAR001	Approved by:
Supervisor	Kenny Mudenda	Approval date:
Title:	Continuum Damage Mechanics to Model Glulam Beams Reinforced With Glued-In Steel Cables	
Basic description of thesis: A number of glulam beams have been reinforced with multiple strands of steel lifting ropes. The steel is placed inbetween the glulam layers during production. The beams will then be tested in bending until failure to determine deflections, MOE and MOR.		
Beam production and Delivery		
Sequence of steps	Potential Hazards	Safe procedures/mitigation efforts
Beam production	N/A	The beams will be manufactured by an external company. Caution should still be taken when visiting the production facility to take photos and ensure good production practices.
	N/A	Beams will be delivered to UCT upper campus, civil engineering laboratory. The beams will be delivered by the production company and any safety procedures should be of their own undertaking.
Unloading	(a) General	The beams will be 3 meteres in length and have steel cables of 0.5 meter length protruding on either side.
	(b) Steel cables	Before being moved into the lab, the sharp ends of the protruding steel cables should be wrapped in red tape. This will prevent the wires fraying and make them easier to see.
	(c) Beams	The beams will weigh roughly 20kgs each and should be offloaded by a minimum of 2 people. Once the beams are inside the laboratory they can be moved downstairs using the lifting crane. The beams must be securely fastened before lifting.
Storage	(d) Tripping	Beams must be stored away from common walkways and stacked neatly. Red tape must placed/replaced around protruding wires
	(e) Clear space	20 beams at roughly 3 meters in length need to be stored indoors. The space should be clean and no heavy objects should rest on the beams.

Table C.2: Testing JSA

Student name:	Aaron Rhodes	Date:
Student Number:	RHDAAR001	Approved by:
Supervisor	Kenny Mudenda	Approval date:
Title:	Continuum Damage Mechanics to Model Glulam Beams Reinforced With Glued-In Steel Cables	
Basic description of thesis: A number of glulam beams have been reinforced with multiple strands of steel lifting ropes. The steel is placed inbetween the glulam layers during production. The beams will then be tested in bending until failure to determine deflections, MOE and MOR.		
Specimen testing		
Sequence of steps	Potential Hazards	Safe procedures/mitigation efforts
Loading the machine	(a) General	The beams will be tested in a 4 point bending set up
	(b) Projectile material	The area must be cleared of people and a safe standing distance should be outlined. A camera will be setup for specimen review at a later stage. A screen should be placed around the beam to prevent any sudden splintering of timber from hitting someone.
	(c) Obstructions	Ensure the machine is clear of any obstructions and that no unnecessary equipment or persons are next to the machine
Deflection test	(d) General	A deflection test is to be performed for the first 10mm of deflection using a dial gauge
	(e) Dial gauge placement	The dial gauge must be placed once the beam already has a small settlement load placed on the beam. To prevent the dial gauge from breaking and hands being placed under a loaded beam the dial guage should measure the deflection from the top of the beam. If possible the dial gauge should be placed to the side of the beam
Loading until failure	(f) General	After the final deflection reading has been taken the beams are loaded until failure
	(g) Failure	The beams are expected to fail either in timber or in the glue interface. If the glue interface fails the reinforcing may break free. If this happens safety screens must be in place to protect any persons nearby
Removal from machine	(h) Deloading	The load must be removed slowly
	(i) removal	The beam may only be removed from the machine once all loading hydraulics have come to a stop. Beams must then be placed back in their designated storage space.

Appendix D

Ethics

ETHICS APPLICATION FORM

Please Note:

Any person planning to undertake research in the Faculty of Engineering and the Built Environment (EBE) at the University of Cape Town is required to complete this form **before** collecting or analysing data. The objective of submitting this application *prior* to embarking on research is to ensure that the highest ethical standards in research, conducted under the auspices of the EBE Faculty, are met. Please ensure that you have read, and understood the **EBE Ethics in Research Handbook** (available from the UCT EBE, Research Ethics website) prior to completing this application form: <http://www.ebe.uct.ac.za/ebe/research/ethics1>

APPLICANT'S DETAILS		
Name of principal researcher, student or external applicant		Aaron Rhodes
Department		Civil Engineering
Preferred email address of applicant:		rhdaar001@myuct.ac.za
If Student	Your Degree: e.g., MSc, PhD, etc.	MSc. Civil Engineering
	Credit Value of Research: e.g., 60/120/180/360 etc.	180
	Name of Supervisor (if supervised):	Kenny Mudenda
If this is a research contract, indicate the source of funding/sponsorship		
Project Title		Using Continuum Damage Mechanics to Model Glulam Beams Reinforced With Glued-In Steel Rods

I hereby undertake to carry out my research in such a way that:

- there is no apparent legal objection to the nature or the method of research; and
- the research will not compromise staff or students or the other responsibilities of the University;
- the stated objective will be achieved, and the findings will have a high degree of validity;
- limitations and alternative interpretations will be considered;
- the findings could be subject to peer review and publicly available; and
- I will comply with the conventions of copyright and avoid any practice that would constitute plagiarism.

APPLICATION BY	Full name	Signature	Date
Principal Researcher/ Student/External applicant	Aaron Rhodes		23/06/2020
SUPPORTED BY	Full name	Signature	Date
Supervisor (where applicable)	Kenny Mudenda		23.06.2020

APPROVED BY	Full name	Signature	Date
HOD (or delegated nominee) Final authority for all applicants who have answered NO to all questions in Section 1; and for all Undergraduate research (Including Honours).	Dyllon Randall		1 July 2020
Chair: Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the questions in Section 1.			