

University of Southern Queensland  
Faculty of Engineering & Surveying

**Structural Performance of Cross Laminated Timber Panels as  
Walls**

A dissertation submitted by

Alan Turner

in fulfilment of the requirements of

**ENG4112 Research Project**

towards the degree of

**Bachelor of Civil Engineering**

Submitted: October, 2010

# Abstract

Cross Laminated Timber (CLT) is made from dimensional lumber, laminated in layers with the grain in each layer placed orthogonal to the layer before it. The resultant product is not dissimilar to thick plywood.

CLT is made from timber, which is an environmentally attractive option in comparison to the reinforced concrete or steel structures which dominate the multistorey building market sector. Timber is one of the very few sustainable carbon neutral building materials available.

New technology such as advanced Building Information Modeling (BIM) software and high accuracy CNC machinery are giving CLT the advantage over traditional on-site construction methods. Prefabrication is the key to CLT's improved construction time-frames, far greater quality control, than on-site construction can deliver.

There are no CLT manufacturers currently producing in Australia. Hyne and Son Pty Ltd. is a leading Australian manufacturer and distributor of engineered, structural and decorative softwood products to the building industry. This project forms part of an initial investigation into cross laminated timber wall panels, manufactured from non-structural low grade pine. Initially, the investigations will be directed solely at the low-rise multistorey residential market sector, which Hyne already services with other products.

The steps required to firstly select a suitable CLT panel using Finite Element Analysis through to fabrication of a sample CLT panel, experimental design and testing of structural properties, and finally comparing results with accepted formula from the Australian standards for the design of plywood. Considerations such as fire, and deflection under load are also made.

Results from initial investigations show that, even though fabricated from low grade structural timber, CLT has enormous axial capacity. Conclusions are drawn as to the application of certain parts of AS1720 Timber Structures to cross laminated timber design.

University of Southern Queensland  
Faculty of Engineering and Surveying

**ENG4111/2 *Research Project***

**Limitations of Use**

The Council of the University of Southern Queensland, its Faculty of Engineering and Surveying, and the staff of the University of Southern Queensland, do not accept any responsibility for the truth, accuracy or completeness of material contained within or associated with this dissertation.

Persons using all or any part of this material do so at their own risk, and not at the risk of the Council of the University of Southern Queensland, its Faculty of Engineering and Surveying or the staff of the University of Southern Queensland.

This dissertation reports an educational exercise and has no purpose or validity beyond this exercise. The sole purpose of the course pair entitled “Research Project” is to contribute to the overall education within the student’s chosen degree program. This document, the associated hardware, software, drawings, and other material set out in the associated appendices should not be used for any other purpose: if they are so used, it is entirely at the risk of the user.

**Prof F Bullen**

Dean

Faculty of Engineering and Surveying



# Certification of Dissertation

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

ALAN TURNER

0050063905

---

Signature

---

Date

# Acknowledgments

I would like to acknowledge Associate Professor Karu Karunasena for his technical support, patience, availability and direction throughout the project. Equally, I would like to acknowledge Geoff Stringer, Stephen Bolden and Tony Dakin from Hyne and Son Pty Ltd for the technical support, provision of test materials and valuable time. I am indebted to my family and especially my wife for the tolerance, support and assistance provided.

ALAN TURNER

*University of Southern Queensland*

*October 2010*

# Contents

<b>Abstract</b>	<b>i</b>
<b>Acknowledgments</b>	<b>v</b>
<b>List of Figures</b>	<b>x</b>
<b>List of Tables</b>	<b>xiii</b>
<b>Chapter 1 Introduction</b>	<b>1</b>
1.1 Outline of the Study . . . . .	1
1.2 Introduction . . . . .	2
1.3 Aim and Scope . . . . .	2
1.4 Research Objectives . . . . .	3
1.5 Thesis Overview . . . . .	3
<b>Chapter 2 Literature Review</b>	<b>5</b>
2.1 History of CLT Panels . . . . .	5
2.2 Australian Regulatory Requirements . . . . .	6

<b>CONTENTS</b>	<b>vii</b>
2.3 Timber Properties and Stress Grading . . . . .	7
2.3.1 Homogenisation and Engineered Wood Products . . . . .	8
2.3.2 Effects of Moisture . . . . .	10
2.3.3 Duration of Load (DOL) Effects . . . . .	11
2.4 Design for Fire . . . . .	11
2.5 Wall Panel Axial In-Plane Loading . . . . .	14
2.6 Wall Panel Raking Resistance (In-Plane Shear Capacity) . . . . .	16
2.7 Accepted Theory - Stability and Buckling . . . . .	16
<b>Chapter 3 Finite Element Analysis</b>	<b>21</b>
3.1 Introduction . . . . .	21
3.2 Properties of Dimensional Timber . . . . .	22
3.3 Calculation of Nominal Wall Loads . . . . .	27
3.4 Trial CLT Layups . . . . .	30
3.5 The Finite Element Model . . . . .	32
3.5.1 Model Geometry and Material Properties . . . . .	32
3.5.2 Forces and Restraints . . . . .	36
3.6 Model Sensitivity and Boundary Conditions Check . . . . .	39
3.7 Results of Finite Element Analysis . . . . .	40
3.7.1 Trial CLT Finite Element Analysis Results . . . . .	41
3.7.2 Sensitivity Analysis and Boundary Condition Check Results . . . . .	43

<b>CONTENTS</b>	<b>viii</b>
3.8 Summary of FEA and Selection of Trial Experimental CLT Layup . . . . .	45
<b>Chapter 4 Experimental Design</b>	<b>47</b>
4.1 Introduction . . . . .	47
4.1.1 Materials . . . . .	48
4.1.2 Sample Fabrication Procedure . . . . .	49
4.2 Axial Load Capacity of CLT Wall Panels . . . . .	51
4.2.1 Introduction . . . . .	51
4.2.2 Test Procedure . . . . .	52
4.3 Effective Modulus of Elasticity of CLT Wall Panels . . . . .	58
4.3.1 Introduction . . . . .	58
4.3.2 Test Equipment . . . . .	61
<b>Chapter 5 Results and Discussion</b>	<b>63</b>
5.1 Axial Load Test Results and Discussion . . . . .	63
5.2 Four Point Bending Test Results and Discussion . . . . .	69
5.3 AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms . . . . .	75
5.4 Development of Load vs Height Charts for the CLT Sample Panel . . . . .	79
<b>Chapter 6 Conclusions and Further Work</b>	<b>81</b>
6.1 Summary . . . . .	81
6.2 Achievement of Project Objectives . . . . .	81

<b>CONTENTS</b>	<b>ix</b>
6.3 Conclusion . . . . .	84
6.4 Recommendations and Further Work . . . . .	84
<b>References</b>	<b>85</b>
<b>Chapter 7 Project Specification</b>	<b>90</b>
<b>Chapter 8 Moisture Content and Acoustic Modulus of Elasticity</b>	<b>93</b>
<b>Chapter 9 MATLAB Source Code</b>	<b>95</b>
9.1 Load Vs Displacement Experimental Data Plotting Programs . . . . .	95
9.2 Secant Formula Buckling Linear Load Calculation Program . . . . .	98
<b>Chapter 10 Purbond Adhesive Datasheet</b>	<b>100</b>

# List of Figures

2.1	Stadhaus Building, London 2010 ( <i>Tall Timber Buildings The Stadhaus, Hoxton, London</i> 2010) . . . . .	6
2.2	Probability Density Functions for Glulam and Solid Timber (Thelandersson & Larsen 2003, p.19) . . . . .	9
2.3	Glulam Beam . . . . .	9
2.4	Shrinkage and Distortion Relative to Parent Log (U.S. Department of Agriculture, Forest Service, Forest Products Laboratory 2010, p.4-5) . . . . .	11
2.5	Timber Building of The Era (Robinson 2010) . . . . .	12
2.6	Stable and Unstable Systems . . . . .	17
2.7	Stability Factor VS Slenderness Factor (Standards Australia 2010 <i>b</i> , p.32) . . . . .	19
3.1	Nominal Residential Unit Building . . . . .	28
3.2	Trial CLT Cross Sections . . . . .	31
3.3	Transformed Material Properties for Vertical CLT Lamella . . . . .	33
3.4	Transformed Material Properties for Horizontal CLT Lamella . . . . .	34
3.5	Strand7 Orthotropic Block Element Material 1 . . . . .	34

**LIST OF FIGURES****xi**

---

3.6	Strand7 Orthotropic Block Element Material 2 . . . . .	36
3.7	I-Joist to CLT Wall Panel Connection(Pryda Australia 2009) . . . . .	37
3.8	Wall Top Edge Local Deflection . . . . .	37
3.9	Strand7 Rigid Link Example (Strand7 Pty Ltd 2010) . . . . .	38
3.10	Trial 1 Rigid Links, Node Restraints and Unit Load . . . . .	39
3.11	Critical Buckling Mode Shapes . . . . .	42
3.12	Trial 1 CLT Layup Model and Associated Subdivided Model . . . . .	44
3.13	Trial 1 CLT Layup Model and Associated Boundary Condition Check Model . . . . .	45
4.1	Locations of 25 mm × 90 mm Boards within Parent Log . . . . .	48
4.2	Wagner Moisture Meter . . . . .	49
4.3	Assembly of CLT Panel . . . . .	50
4.4	CLT Samples Fabricated by Hyne . . . . .	51
4.5	Axial Load Test, Loads of Interest . . . . .	52
4.6	Load Application and Vertical Deflection Measurement . . . . .	54
4.7	Safety Cage, load cell and Sample Panels . . . . .	55
4.8	Test Equipment Setup . . . . .	57
4.9	Four Point Bending Test (Standards Australia 2010 <i>a</i> ) . . . . .	58
4.10	CLT VS AS4063.1 Sample Cross Section Notation (Standards Australia 2010 <i>a</i> , p.5) . . . . .	59
4.11	Modified Four Point Bending Test Dimensions . . . . .	60



**LIST OF FIGURES**

---

4.12	Four Point Bending Test Equipment . . . . .	62
5.1	Sample 1, Post Ultimate Load Deflection . . . . .	64
5.2	Sample 2. Photo Sequence . . . . .	65
5.3	Axial Load VS Vertical Displacement . . . . .	67
5.4	Axial Load VS Mid-Height Horizontal Displacement . . . . .	68
5.5	Fracture Path for Samples Two and Three . . . . .	69
5.6	Sample Three Delaminated Glue Surface . . . . .	70
5.7	Four Point Bending Test:Elastic Region . . . . .	71
5.8	Four Point Bending Test: Post Failure . . . . .	72
5.9	Delamination of CLT Due To Flexural Shear . . . . .	72
5.10	Shear Force and Bending Moment Diagram (American Forest and Paper Association, Inc., American Wood Council 2007, p.8) . . . . .	73
5.11	Load VS Displacement Plot: Four Point Bending Test . . . . .	74
5.12	Plywood Cross Section and Notation(Standards Australia 2010 <i>b</i> , p.167) . . . . .	77
5.13	Hyne CLT Theoretical Height Vs Lineal Load Plot for Various Eccentricities . . . . .	80
8.1	Moisture Content and Acoustic Modulus of Elasticity Data Received From Hyne . . . . .	94

# List of Tables

3.1	Mechanical Properties of Timber used in Hyne CLT fabrication . . . . .	24
3.2	Orthotropic Elastic Properties of Slash Pine . . . . .	26
3.3	Roof Dead Loads . . . . .	27
3.4	Floor Dead Loads . . . . .	28
3.5	FEA Material Properties Summary . . . . .	35
3.6	Node Restraint Conditions (F=Fixed R=Released) . . . . .	40
3.7	Trial CLT Layup Linear Buckling Capacity Results . . . . .	41
3.8	Trial CLT Layup Design Buckling Load Capacities . . . . .	43
5.1	Service and Ultimate Axial Load Summary . . . . .	66

# Nomenclature

BCA	Building Code of Australia
CLT	Cross Laminated Timber
EWP	Engineered Wood Product
FEA	Finite Element Analysis
LVL	Laminated Veneer Lumber
USDA	United States Department of Agriculture

# Chapter 1

## Introduction

### 1.1 Outline of the Study

Cross Laminated Timber (CLT) panels are fabricated from strips of timber, laminated together in layers with the grain direction of each lamella orthogonal to that before.

According to B&K (*B & K Cross Laminated Timber* 2010), existing CLT wall panels currently on the market in Europe range from 57 mm to 300 mm in thickness, and consist of between three and nine laminates. The overall size of each panel is dependent on fabrication facilities and transportable size limitations.

Typically each CLT panel is prefabricated with penetrations for doors and windows already cut, ready for installation by way of simple screwed connections on site.

Prefabrication of wall panels reduces construction time and, therefore, costs in comparison with other commonly used construction materials. Costs may be further reduced by utilising non-structural grade timber in the manufacture of the CLT panels.

Hyne and Son Pty Ltd have identified Cross Laminated Timber panels (CLT) made from low grade Caribbean and Slash Pine as a product which may have significant market potential to replace existing concrete and masonry as wall systems for 'low rise' multi-storey buildings and commercial tilt-panel structures in Australia.

## 1.2 Introduction

This project investigates the structural performance of CLT panels manufactured from non-structural grade Hyne timber for use in both internal and external load bearing wall applications. The performance of the CLT panels for wall applications is measured in terms of ultimate strength and serviceability.

External and internal load bearing walls serve to transfer the load from the structure above to the structure below and to resist raking loads from earthquake and wind load actions. Serviceability of CLT panels encompasses the effects of moisture, allowable deflection and fire.

The challenge is to identify firstly an 'optimal' CLT lay-up for Hyne and Son Pty Ltd in terms of the specific said application, and then fabricate and test samples to ascertain if the selected layup performs as predicted.

Wall loading for strength includes in-plane axial compression, bracing capacity (in-plane shear) and load transfer around penetrations. Serviceability aspects investigated include deflection, fire resistance and the effects of moisture.

## 1.3 Aim and Scope

Broadly, the aim of this project is to analyse the structural performance of CLT panels manufactured from non-structural grade timber, for internal and external wall applications in terms of serviceability and strength.

The scope of the project is limited to the application of CLT wall panels for low-rise multi-storey buildings up to three storeys high, and to commercial and industrial buildings, including tilt-slab concrete buildings. Loads applied to the CLT panel will be those applied in the design of a typical three storey residential building in Australia.

Identification of the most suitable CLT cross section for the application was required such that further investigation could proceed into fabrication methods, cost of production and possible regulatory restraints on the introduction of the product to the Australian market.

As no Australian Standard existed for the fabrication or use of CLT wall panels, a comparison between the accepted existing standards for plywood in plane diaphragm loading with the results of the laboratory testing, is included for assessment of implementation using existing standards.

The method of manufacture of CLT wall panels, and fabrication of the sample CLT panels does not form part of the scope for this project, as it is mainly concerned with the structural performance of the CLT panels. Wall raking resistance is specifically excluded from the scope of this project.

## **1.4 Research Objectives**

The specific objectives of the project are to:

1. Research background information relating to the structural performance of cross laminated timber for wall applications. The research will include fire resistance, deflection and ultimate strength prediction of CLT panels.
2. Undertake Finite Element Analysis (FEA) to investigate the structural properties of a number of CLT cross sections which Hyne can feasibly produce, followed by a subsequent evaluation and selection of a CLT panel cross section for fabrication and strength testing.
3. Undertake strength testing and compare theoretical results with actual load test results, and also with accepted formulae from AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms. For serviceability, the aim is to provide theoretical internal and external CLT wall load charts for various common wall heights and report on the effects of fire.

## **1.5 Thesis Overview**

This dissertation is organized as follows:

**Chapter 2** is an in-depth literature review, which investigates all available literature on the design and serviceability aspects of CLT wall panels. This chapter provides background on existing regulations, timber serviceability with respect to moisture, design for fire, and accepted buckling and stability theory.

**Chapter 3** is involved with the selection of a trial CLT layup for experimental testing. Finite Element Analysis (FEA) is used to ascertain the buckling strength of a number of different CLT panel layups, and the results compared.

**Chapter 4** describes the experimental designs, how they were selected and developed for use in this project. Experiments were developed to determine the axial strength and section properties of the CLT sample selected in the previous chapter.

**Chapter 5** Covers the results and discussion of experiments conducted at the University of Southern Queensland. This chapter also reviews AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms to see if the accepted formula for the design of plywood diaphragms can be applied to CLT. A theoretical wall load vs height chart, for various load and material eccentricities is included.

**Chapter 6** Concludes the dissertation and suggests areas of further work.

**Chapters 7, 8 and 9** are appendices.

## Chapter 2

# Literature Review

### 2.1 History of CLT Panels

Cross Laminated Timber (CLT) panels provide many new possibilities for the efficient and environmentally sustainable construction of houses and buildings. The product has been available overseas since its development in the 1970's (*Building with Timber - Nine Storeys and Beyond* 2010), and is still considered an emerging technology as different construction methodologies are tested and new capabilities identified.

A recent swing away from prescriptive building codes has allowed the introduction of alternative building materials such as CLT. One example of CLT implementation in the UK is the Stadhaus building, Hoxton London. Standing at nine storeys, and completed in January 2009, it is the tallest pure timber building in the world. The completed Stadhaus building is shown in figure 2.1.

Current design practice for CLT, as employed by the engineering group, Techniker, is to pre-fabricate as large as is transportable panels and install on site directly with simple connection details.

According to Techniker, this style of CLT construction is economically viable up to fifteen storeys without the use of local strengthening to bearing points (*Tall Timber Buildings The Stadhaus, Hoxton, London* 2010). Simplicity of construction is key, and connections consist





Figure 2.1: Stadhaus Building, London 2010 (*Tall Timber Buildings The Stadhaus, Hoxton, London 2010*)

mostly of angle cleats and long screws. This type of connection facilitates rapid installation.

## 2.2 Australian Regulatory Requirements

For implementation in Australia, the material must be designed such that it complies with the Building Code of Australia (BCA), which is the governing regulatory Code to which buildings must be constructed.

The BCA is a not a prescriptive code and, therefore, it is possible that, assuming the material can be shown capable of achieving the performance levels stipulated, it may be used as a structural wall element. There is no Australian Standard for the manufacture or use of CLT products.

## 2.3 Timber Properties and Stress Grading

There are large variations between the structural properties of clear wood and the structural properties of timber. In this paper, timber is the term given to structural elements which includes the irregularities and defects such as knots, zones of compression wood, oblique fibre orientation etc.

For the characteristics of the timber to be determined in a reliable way, multiple samples must be tested in accordance with standardised test procedures, and statistical methods used to determine structural properties (Thelandersson & Larsen 2003, p.30). The properties can then be used in the calculation of the moment, tension, compression and shear capacity of the structural element by way of elastic theory.

The theory of elasticity, of course, is only representative if the strength properties used are accurate for the load type applied. This is of particular relevance to timber due to its anisotropic and non-homogenized nature, and explains, for example, why the characteristic bending tensile strength is used for bending, and characteristic tensile strength is used for pure axial tensile loading. The influence of the defects within the timber is included in the strength properties used.

Knots are the cause of the largest portion of failures, however other defects such as top rupture, compression wood, grain slope, decay, bark and gum pockets, wane, holes and splits or a combination of same also contribute. Determining the modulus of elasticity is the single most effective way of determining the strength characteristics of timber because it includes information regarding the clear wood properties, knots, slope of grain and other characteristics.

Additional grading information which can be incorporated with the modulus of elasticity to more accurately grade timber includes narrow and wide face knot size and locations, ring width and density among others (Thelandersson & Larsen 2003, p.30).

In timber production, non-destructive strength grading of the timber is required to define the strength properties used. Non-destructive strength grading can be either visual or by machine grading methods, with machine grading being the more accurate method, and hence offering a better yield of higher grade timber. Visual grading has limitations in terms of the thick-

ness of the timber which can be graded because faults or irregularities within the timber are undetectable (Thelandersson & Larsen 2003, p.30).

Typical machine grading usually incorporates a flat-wise bending test to determine the modulus of elasticity, and may include density measurement by radiation and/or knot detection by optical detection. The stiffness can then be accurately determined using the following formula:

$$EI = \frac{P \cdot l^3}{48 \cdot \delta}$$

Where  $P$  is the applied load,  $l$  is the span between supports and  $\delta$  is the deflection (Thelandersson & Larsen 2003, p.34).

Machine bend testing of CLT wall panels is unrealistic due to the size of prefabricated wall panels. Another method of determining the modulus of elasticity is to measure the natural frequency of the timber, and to this end, a fully automated procedure for determining the global elastic properties of full-scale CLT panels has been developed by Gsell et.al (Gsell, Feltrin, Schubert, Steiger & Motavalli 2007).

### **2.3.1 Homogenisation and Engineered Wood Products**

The term Engineered Wood Product (EWP) is a broad term that includes material typically made from smaller wood elements such as veneers and laminates. Commonly available EWP's include Laminated Veneer Lumber (LVL), plywood, glue laminated timber and I-beams.

CLT also falls into the category of EWP because it is made from laminations which have been glued together. The advantages of EWP over dimensioned timber are that they are typically dimensionally stable, and can be produced in required dimensions not limited by the individual element sizes.(Thelandersson & Larsen 2003, p.18)

The design strength of EWP is defined as the fifth percentile similar to dimensioned lumber, however due to the homogenisation of material, the fifth percentile characteristic strength value is generally higher than for timber. Figure 2.2 shows the probability functions for EWP and Timber (Thelandersson & Larsen 2003, p.19).

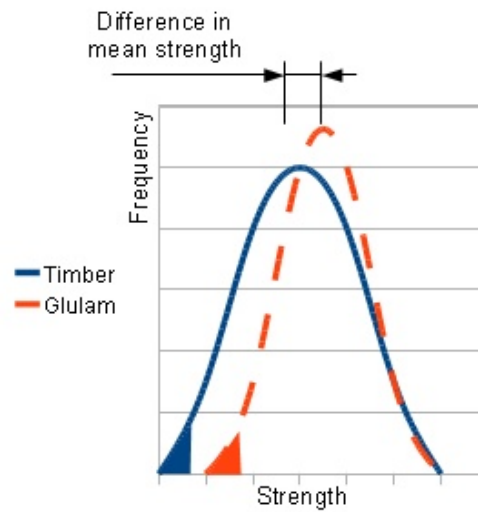


Figure 2.2: Probability Density Functions for Glulam and Solid Timber (Thelandersson & Larsen 2003, p.19)

Considering that CLT panels are made from dimensioned timber, it is not unreasonable to assume that there is a relationship between the strength of the individual elements and the effective strength of the CLT panel.

Two options available for predicting the strength properties of CLT panels of various thicknesses and lamella are firstly to produce sufficient samples such that the properties can be determined statistically, and secondly, to use a method which uses the strength properties of the individual elements to determine the strength characteristics.

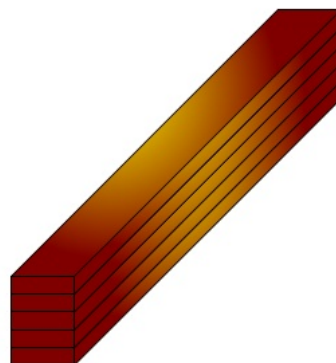


Figure 2.3: Glulam Beam

The second method is more difficult to accurately achieve for CLT panels than, for example, glue laminated (Glulam) beams indicated in figure 2.3, because of the composite nature of the orthogonally placed lamella.

For Glulam beams, the typical failure mode for bending is tensile failure of the outer laminate. Laminating increases the bending strength above the pure tensile strength of the laminates. The ratio of tensile bending strength to tensile strength is called the laminating factor (Thelandersson & Larsen 2003, p.69).

The Glulam strength increase is attributed to homogenisation of defects, and by reinforcement of defective areas by adjacent laminates. Some CLT panels are constructed in only three layers, and this presents a different case than for Glulam, which typically comprises many more layers.

To substantially increase the bending tensile strength by laminating would require outside lamella thickness to be reduced to less than 10mm (Thelandersson & Larsen 2003, p.71). CLT wall elements effectively load share along the length of the panel when loaded in plane and in bending, however unless more than three laminates are used, it is unlikely that the effective strength will be higher than that of the timber elements from which it is made.

In Australia, Glue Laminated Timber is fabricated to AS/NZS 1328, and establishment of characteristic properties of glulam is performed using AS/NZS 4063.

### **2.3.2 Effects of Moisture**

The dimension stability of the timber is relative to the moisture shrinkage. Shrinkage strains are much larger perpendicular to the grain than parallel to the grain, and also much larger tangentially than radially. Characteristic shrinkage and distortion of timber elements in relation to their location in the parent log from which they were sawn are indicated in figure 2.4

Drying of the timber to a moisture content less than 15 % results in the timber being classed as seasoned timber. The humidity of air inside a building is similar to the moisture content of seasoned timber and therefore equilibrium results in good dimensional stability for interior and dry applications. The dimensional stability is greatly enhanced for CLT panels due to the orthogonal lay-up of lamella.

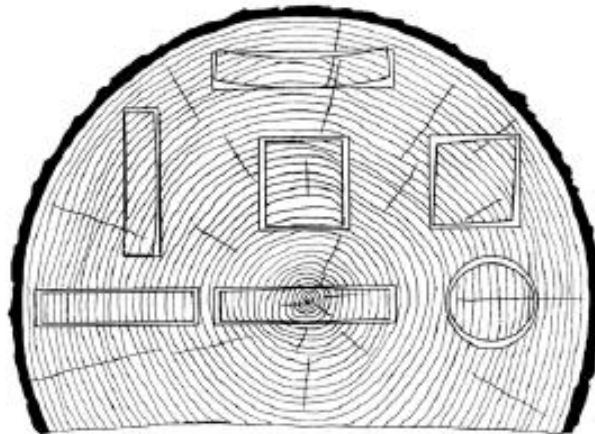


Figure 2.4: Shrinkage and Distortion Relative to Parent Log (U.S. Department of Agriculture, Forest Service, Forest Products Laboratory 2010, p.4-5)

The CLT samples provided by Hyne and Son Pty Ltd were backsawn, and therefore are subject to cupping due to the moisture gradient between the air and the timber, unless sufficiently sealed against moisture ingress. It is likely that moisture cracks will occur parallel with the grain on the exterior lamella of the CLT wall panels, and therefore the shear capacity should not be relied upon parallel to the grain.

### 2.3.3 Duration of Load (DOL) Effects

The strength of timber also depends on the duration and rate of loading. AS1720.1 1997 defines duration load factors for five seconds, five minutes, five weeks, five months and fifty years, with load factors ranging from 1.0 for five second wind gusts to 0.57 for permanent loads. The long term load factor is a result of the significant loss of strength timber undergoes when subject to load for long periods of time.

## 2.4 Design for Fire

The use of timber for multistorey residential buildings has not been fully embraced since the great fire of London in 1666, where 373 acres of London was burned to the ground, and 100,000

people were left homeless in a dreadful and uncontrollable fire (Robinson 2010). Prior to that time, most of the buildings in Europe were of timber construction.

After the fire, new legislation was enacted (The Rebuilding Act 1667), which prescribed that the outsides of all buildings should be made of bricks or stone (Museum of London 2010)



Figure 2.5: Timber Building of The Era (Robinson 2010)

It should be noted that there were a number of contributing factors to the disaster, apart from the use of timber in the building structure, which the Rebuilding Act 1667 recognises. Houses prior to the great fire encouraged the spread of fire by close placement of dwellings, having disorderly design with no partitions between dwellings, overhangs out over the small lane-ways and overhangs over other properties. A typical Low-rise multistorey dwelling which was claimed by the great fire is shown in figure 2.5.

The main points of the Act called for wider roads and lane-ways, fire separating brick walls around properties with minimum separation distances specified, no overhangs or balconies outside the boundary of the property unless protruding over a street, and construction of the boundary walls to be made from stone or masonry.

The Rebuilding Act 1667 is a prescriptive code, however if it were a non-prescriptive code then it could in part be interpreted that if a major fire could be contained to an individual dwelling,

then it has met the requirements of the Act. The Building Code of Australia (BCA) is a non-prescriptive code, which shares a remarkable resemblance to the Rebuilding Act 1667, in that it recognises classes of buildings and contains similar requirements which limit the spread of fire.

The BCA requires that compliance of the Fire Resistance Level (FRL) be provided in either a report from a Registered Testing Authority, stating that the material complies with the standard fire test, or alternatively is designed in accordance with AS1720.4 if it is a solid or glued-laminated timber structure (Australian Building Codes Board 2010, p.25).

Much published research is already available on the effects of fire to exposed CLT. In nature, large trees and logs can resist the effects of fire by charring. Pyrolysis is a chemical decomposition reaction timber undergoes when exposed to temperatures around 150° deg C to 200° deg C. The gases produced by the pyrolysis reaction are combustible. The char-layers which form once the timber is ignited have excellent insulating properties which protect the timber.

Designing for fire typically involves adding a redundant thickness to the CLT wall panel. The additional thickness of material required is dependant on the required FRL.

A charring rate is the rate at which char is formed and is proportional to the temperature. According to AS1720.4 (Standards Australia 2010c, p.8), the notional charring rate for untreated timber exposed to fire is:

$$c = 0.4 + \frac{280^2}{\delta} \quad (2.1)$$

and

$$d_c = ct + 7.5 \quad (2.2)$$

where:

$c$  = notional charring rate in millimeters per minute



$\delta$  = timber density at 12% moisture content

$d_c$  = effective depth of charring

$t$  = time period in minutes

It should be noted that AS1720.4 only applies to Glued Laminated Timber fabricated with phenol, resorcinol, phenol resorcinol or poly-phenolic glues.

Results from a recent study by Wilinder (Wilinder 2009, p.38) indicate that the charring rate of CLT is similar to that of solid wood as calculated by EN1995 1-2 for a solid beam burning on one side only.

It follows that for CLT constructed of Pine, with a density of  $600 \text{ kg/m}^3$ , the charring rate  $c$  is approximately 0.61 millimeters per minute. Further, for a nominal building constructed of CLT requiring a 30 min fire resistance level, a 26 mm additional thickness is required.

## **2.5 Wall Panel Axial In-Plane Loading**

Timber framed wall elements of buildings are typically between 2.7m and 3.0m tall for single and multistorey construction alike, and so depending on the CLT geometry, the panel may be considered slender in some circumstances.

For low-rise multistorey construction, the floor is typically placed directly over the timber walls beneath (platform construction), to ensure in-plane loading is achieved (*Tall Timber Buildings The Stradhaus, Hoxton, London* 2010). The top and bottom restraint conditions of the panel can be modeled as being restrained at each end in position only (i.e pin end top and bottom).

According to R.H. Leicester of the CSIRO, Victoria (Leicester 2009), two parameters are used in the calculation of the buckling strength of dimensioned timber structures when designing to Australian Standard - Timber Structures Part 1: Design methods (AS1720.1).

The parameters are the stability parameter, and the stiffness parameter. Steiger and Gulzow (Steiger & Gulzow n.d.) note that verification of CLT panel properties must be undertaken to account for the effects of homogenisation of the low grade timber with laminating, when considering elastic and strength properties of the panel.

One of the objectives of this project is to identify if the formula in Appendix I of AS1720.1 is applicable to CLT wall panels. Formulae are provided for diaphragms with lateral edges free and subjected to uniformly loaded edge forces. This is appropriate as the ends of the CLT wall sections will not necessarily have lateral support, as is the case for a wall section between two wall openings.

Ing, Blass and Fellmoser (Blass, Fellmoser & Ingenieurholzbau 2004) identify the composite theory as the current design method for calculating plywood properties. They describe the composite theory as only taking the modulus of elasticity of elements of the plies acting parallel to the load into consideration. This leads to inaccuracies because plies perpendicular to the load contribute to the bending rigidity of CLT panels.

Contrary to Ing, Blass and Fellmoser, AS1720.1 (Standards Australia 2010*b*, Appendix I) takes into consideration a small (3%) contribution for the plies acting perpendicular to the load, when determining the bending rigidity.

Unfortunately, however the equation for calculating the buckling strength of plywood is independent of the bending rigidity of the section. The slenderness coefficient is based on the ply lay-up and the stability parameter used is similar to that for solid timber.

Further, Ing, Blass and Fellmoser (Blass et al. 2004) note that when taking the plies in the X-axis direction into consideration, rolling shear of the laminate will detract from the panel stiffness, i.e. the thicker the laminate running in the X-direction, the higher will be the shear deformation.

To fabricate Hyne CLT panels with more than one laminate running in the X-direction would be expensive due to the additional fabrication complexity. One option to overcome rolling shear deformation of the central laminate, for a three ply CLT wall panel, would be to reduce the thickness of the central laminate.

## 2.6 Wall Panel Raking Resistance (In-Plane Shear Capacity)

Besides axial loading, walls are also used to transfer shear forces from wind loads to the ground. Vessby et al. (Vessby, Enquist, Petersson & Alsmarker 2009) recommend CLT panels as an alternative to timber framing for narrow tall houses. This recommendation is because narrow tall houses require a lot of bracing capacity, and often there is insufficient wall length for the placement of bracing plywood. CLT panels are similar to plywood panels in that they have lamella with grain direction in both axis, which allows them to function as a wall panel in compression as well as a bracing wall in shear.

## 2.7 Accepted Theory - Stability and Buckling

The theoretical buckling load capacity can be determined linearly using Eulers formula. Three defined states of loading for a system are when the system is stable, in neutral equilibrium and unstable. An axially loaded system is considered in neutral equilibrium when the applied load can be removed, and the system returns to it's original position. Euler's formula for the critical buckling load is...

$$P_{cr} = \frac{\pi^2 EI}{L^2}$$

where...

$P_{cr}$  = critical buckling load

$E$  = modulus of elasticity

$I$  = second moment of area

$L$  = height of column or wall

When the restoring force of a column or wall is less than the applied load  $F$  as indicated in figure 2.6, the system is termed unstable and will collapse.

The load at which the system changes from stable to unstable is called the critical load. When a system is stable, the stress in the column or wall can be determined directly using the following

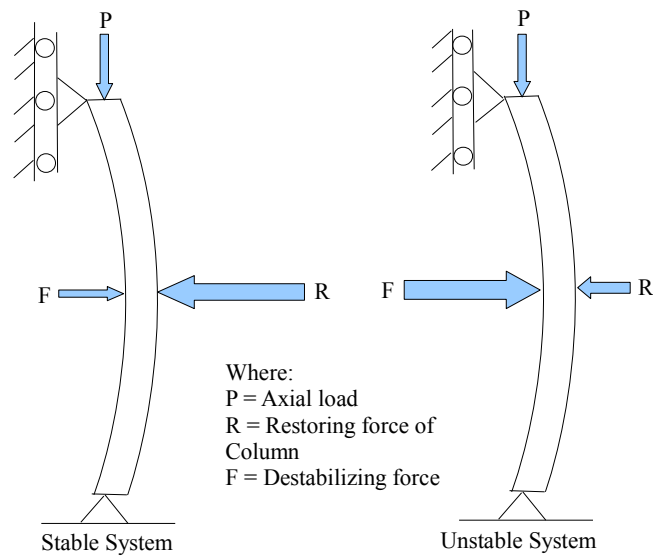


Figure 2.6: Stable and Unstable Systems

equation...

$$\sigma = \frac{P}{A}$$

where  $\sigma$  is the stress in the column or wall cross section,  $P$  is the applied load and  $A$  is the cross sectional area.

CLT load bearing panels can be described as slender columns with pin ends. Lateral support is only provided to the CLT wall panels at locations where a wall intersects. The trend in recent years is for large open span type living areas, and lateral support, therefore, cannot be assumed.

The important thing to note about the critical buckling load is that the material characteristic which alters the critical load is the stiffness, and not the strength of the material. Thus the modulus of elasticity and second moment of inertia are material variables which will determine the critical buckling load. Obviously the height of the wall is the most critical factor.

Localised buckling within the CLT material is not likely because each laminate is laterally restrained by adjacent laminates. As this project specifically deals with wall panels as opposed to columns failure, further calculations assume failure about the weak axis.

It must be assumed that the CLT wall is going to be subjected to eccentric axial loads, due to material eccentricities within the wall because timber is an inhomogeneous material.

Deflection of an eccentrically loaded wall increases proportionally to the applied load. Bending stresses or deflection limits for serviceability may define the allowable load, as opposed to the critical load. According to Gere and Timoshenko (Timoshenko 1991, p.596), the following formula is used to calculate the maximum compressive stress on the inside face of a column. . .

$$\sigma_{max} = \frac{P}{A} + \frac{M_{max}}{S} \quad (2.3)$$

Where

$\sigma_{max}$  = maximum stress on the compressive face

$p$  = axial compressive load

$M$  = moment induced by the load eccentricity

$e$  = load eccentricity

$S$  = section modulus.

The moment induced by the axial compressive load on an eccentric column is related to the induced curvature of the beam-column. The mid-height deflection of a beam-column of certain eccentricity can be calculated using the following formula. . .

$$\delta = e \left( \sec \frac{KL}{2} - 1 \right) \quad (2.4)$$

Combining equations (2.3) and (2.4), the following equation is formed, which is termed the secant equation and can be used to calculate the maximum compressive stress on the compressive face of a beam column. . .

$$\sigma_{max} = \frac{P}{A} \left[ 1 + \frac{ec}{r^2} \sec \left( \frac{L}{2r} \sqrt{\frac{P}{EA}} \right) \right]$$

Where  $r$  is the radius of gyration, and  $c$  is the distance from the centroid to the compressive face. It should be noted that the above secant equation only applies to small deflections and

when Hooke's Law holds.

Although Gere and Timoshenko specifically investigate compressive failure of the outermost fibre of the inner face, and derive the secant formula on this basis, it should be noted that other flexure failures need also be considered.

Australian Standards for design of Compressive Members - Design for strength AS1720.1 presents a formula similar to that presented by Timoshenko. The following formula is the design compressive strength capacity parallel to the grain. . .

$$N_{d,c} = \sigma k_1 k_4 k_6 k_{12} f'_c A_c$$

Where  $k_1$  is the Duration of Load (DOL) factor,  $k_4$  is a factor relating to moisture content and  $k_6$  relates to temperature. The remaining factor  $k_{12}$  is the stability factor.

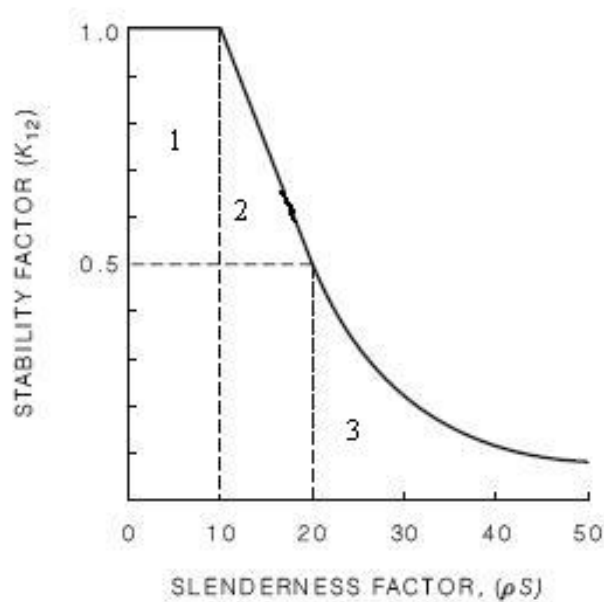


Figure 2.7: Stability Factor VS Slenderness Factor (Standards Australia 2010b, p.32)

The  $k_{12}$  stability factor is calculated on the basis of the three ranges similar to that described by Gere and Timoshenko (Timoshenko 1991, p.603), and as indicated in Plot 2.7. The slenderness factor  $\rho S$  indicated in the figure is a factored slenderness ratio, and takes into account column geometry and restraint.

Having a  $k_{12}$  value which falls within the first area of plot 2.7, indicates column fails will be due to pure axial compression as opposed to buckling. In this range the compressive strength capacity is not dependant on the material stiffness, or the slenderness ratio.

In the second area of plot 2.7, the strength of the outer compressive fibre on the inner face is dominant because the slenderness ratio is not high enough for the column to fail by buckling within the proportional limit.

Finally for the third area of plot 2.7, the formula used in calculation of  $k_{12}$  is similar to the euler buckling formula, and buckling of the column is due to high slenderness ratio of the column and the stiffness of the material. According to Gere and Timoshenko (Timoshenko 1991, p.603), failure which occurs in the third range is independent of the strength of the material.

## **Chapter 3**

# **Finite Element Analysis**

### **3.1 Introduction**

This chapter outlines the technical details and processes of Finite Element Analysis (FEA) to determine which CLT panel layup best suits the requirements of the low-rise multistorey residential building sector, and which can be economically produced by Hyne. A CLT panel layup refers to the number, thickness and direction of each lamella within the CLT panel.

Instead of fabricating and testing many different CLT panel layup samples, an FEA was conducted to predict a single suitable CLT Panel layup. The layup was selected based on a number of considerations including strength, available dimensional timber sizes which could be economically used in the CLT fabrication, and available test equipment capacities.

A nominal load case was developed for a low-rise multistorey residential building. The nominal building wall height is also identified. Further considerations such as available dimensional timber sizes are developed. This forms a criteria for selecting the CLT panel Layup for experimental analysis.

A total of six different CLT layups were modeled and analysed using the Strand7 Finite Element Analysis (FEA) package. Two additional models were also developed to test the sensitivity and boundary condition assumptions. Each wall panel modeled was 3m tall and 1m wide so that a load per unit meter could be directly calculated. The actual CLT panels for experimental



testing were limited to a width of 300 mm due to the machine axial load capacity.

Density and acoustic modulus of elasticity of the timber were provided by Hyne prior to fabrication of the sample panels, and the remainder were taken from published data. Orientation of the orthotropic elastic properties for each lamella was undertaken graphically.

Actual restraint conditions at the top and bottom of a building vary between construction methods. Restraint and loading conditions applied to the modeled wall sections are modeled as being a pin connection at the top and bottom of the wall. The walls were modeled using quad eight brick elements, and analysed using the linear buckling solver.

In summary, the following steps were required to select a suitable layup for experimental testing...

1. Determine the material properties of the dimensional timber used to fabricate the CLT panels
2. Define a nominal load case applicable to the low rise multistorey residential building sector
3. Identify a number of CLT layups which Hyne could economically produce
4. Undertake Finite Element Analysis (FEA) for all six of the identified CLT layups
5. Rationalise the results and select a suitable layup to fabricate samples

## **3.2 Properties of Dimensional Timber**

The CLT sample panels were fabricated from a mixture of non-structural grade Slash Pine (*Pinus Elliottii*) and Caribbean Pine (*Pinus Caribaea*) dimensional timber. Mechanical properties, including orthotropic elastic properties of the timber elements were required to fully describe the behavior of the CLT trial panels by FEA modeling.

In Australia, published timber properties are usually those properties used to design structures in accordance with Australian Standard AS1720 - Timber structures, and other associated Australian Standards. As there is such a large variation in mechanical properties of different tree

species, and also within species due to discontinuities such as knots and sloping grain etc., a grading system was developed.

For Slash and Caribbean pine, grading systems could be either by Machine Grade Pine (MGP) stress grading or grading under the general F grade system. The MGP grading system for structural pine is a result of the 'in-grade testing program of Australian Pine (radiata pine, pinaster pine, slash pine and Caribbean pine), undertaken by CSIRO and State Forests of NSW' in 1996 (*Education Resources Structural Grading* n.d.). The program aimed to achieve a better yield of pine timber with a higher stress grade by implementing more rigorous machine testing techniques.

The properties referred to in the Australian Standards for timber design are fifth percentile characteristic mechanical properties as opposed to mean mechanical properties, which may be significantly higher. If fifth percentile characteristic strengths were used, then the resulting FEA model would not be comparable with the laboratory test results.

Hyne provided mean density and acoustic modulus of elasticity properties for the dimensional timber as measured prior to fabrication refer to Appendix 8. Where published mean mechanical properties for Australian grown Slash and Caribbean Pine could not be found, properties for American Slash and Caribbean Pine were substituted where required, on the basis that the modulus of elasticity and density of the American variety was similar to those measured by Hyne.

Because the test data received from Hyne was not specific as to the species of pine, the mechanical properties of Slash Pine were used where no other data existed. According to the United States Department of Agriculture (USDA), 'the mechanical properties of Slash Pine and Caribbean pine are rather similar' (U.S. Department of Agriculture, Forest Service, Forest Products Laboratory 2010, p.2-39).

The mechanical properties for Slash Pine and Caribbean Pine from the USDA Forest Products Laboratory Wood Handbook, Hyne test data, and the MGP15 strength group are summarised on Table 3.1. The properties which were adopted for this project are also identified.

In addition to the timber properties listed in Table 3.1, FEA modeling requires a full description of the elastic properties of the timber to populate the compliance matrix. Timber is orthotropic

Table 3.1: Mechanical Properties of Timber used in Hyne CLT fabrication

Material Properties	Pine Timber Type	USDA Forest Service	Hyne Test Data (Mean Values)	MGP 15 Characteristic Strengths	Adopted Properties
Modulus of Elasticity (GPa)	Caribbean	15.4	17	15.2	17
	Slash	13.7			
Modulus of Rupture (MPa)	Caribbean	115	N/A	N/A	112
	Slash	112			
Density (kg/m <sup>3</sup> )	Caribbean	680	650	580	650
	Slash	590			
Compression Parallel (MPa)	Caribbean	58.9	N/A	35	56.1
	Slash	56.1			
Compression Perpendicular (Mpa)	Caribbean	Not Listed	N/A	N/A	7
	Slash	7			
Shear (Mpa)	Caribbean	14.4	N/A	9.1	11.6
	Slash	11.6			
Bending Tensile Stress (Mpa)	Caribbean	Not Listed	87	41	87
	Slash	Not Listed			

because it has different elastic properties in different orthogonal directions.

Twelve constants describe the elastic properties of an orthotropic material, including three elastic moduli,  $E$ , three moduli of rigidity  $G$ , and six Poisson's ratios  $\nu$  (U.S. Department of Agriculture, Forest Service, Forest Products Laboratory 2010, p.5-2). Three of the Poisson ratios are dependant, and related via the following equation...

$$\frac{\nu_{ij}}{E_i} = \frac{\nu_{ji}}{E_j}, \quad i \neq j \quad i, j = L, R, T \quad (3.1)$$

Orthotropic elastic properties of timber are approximately related to the longitudinal modulus of elasticity, and were therefore published as ratios to the longitudinal modulus of elasticity (U.S. Department of Agriculture, Forest Service, Forest Products Laboratory 2010, p.5-2).

The calculated orthotropic elastic properties for Slash Pine are provided in Table 3.2. No orthotropic data could be found for Caribbean Pine, however as previously stated, the material properties of Slash and Caribbean Pine are similar, and Slash Pine properties will therefore be substituted.

According to the Strand7 online manual (Strand7 Pty Ltd 2010), the compliance matrix for an orthotropic material is defined as ...

$$C = \begin{bmatrix} \frac{1}{E_1} & \frac{-\nu_{21}}{E_2} & \frac{-\nu_{31}}{E_3} & 0 & 0 & 0 \\ \frac{-\nu_{12}}{E_1} & \frac{1}{E_2} & \frac{-\nu_{32}}{E_3} & 0 & 0 & 0 \\ \frac{-\nu_{13}}{E_1} & \frac{-\nu_{23}}{E_2} & \frac{1}{E_3} & 0 & 0 & 0 \\ \frac{-\nu_{13}}{E_1} & \frac{-\nu_{23}}{E_2} & \frac{1}{E_3} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{12}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{23}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{31}} \end{bmatrix}$$

Where  $E$  is the modulus of elasticity,  $G$  is the modulus of rigidity or shear modulus, and  $\nu_{ij}$  are Poisson's Ratios. The first letter in the Poisson Ratio subscript refers to the direction of applied stress, and the second refers to the resulting direction of deformation.

Table 3.2: Orthotropic Elastic Properties of Slash Pine

USDA Material Property Notation	Material Property Value
Modulus of Elasticity (Mpa)	
$E_L$	17000
$E_T$	1258
$E_R$	935
Shear Modulus (Mpa)	
$G_{LT}$	901
$G_{TR}$	170
$G_{RL}$	935
Poisson's Ratio	
$\nu_{LT}$	0.444
$\nu_{TR}$	0.387
$\nu_{RL}$	0.02

A discrepancy between the relationship of the six Poisson's Ratios published in the Strand7 Online Manual (Strand7 Pty Ltd 2010) and Equation 3.1 was identified. The Strand7 Online Manual identifies one of the Poisson's Ratio relationships as being...

$$\nu_{23}E_3 = \nu_{23}E_2 \quad (3.2)$$

however, according to the USDA formula 3.1, the equation should read ...

$$\nu_{23}E_3 = \nu_{32}E_2 \quad (3.3)$$

The USDA published formula is adopted for this thesis, and according to the Strand7 Manual,  $\nu_{23}$  is interchangeable with  $\nu_{32}$  (Strand7 Pty Ltd 2010), so in any event the FEA model should still be accurate regardless.

Table 3.3: Roof Dead Loads

Roof Dead Loads	Mass (kN/m <sup>2</sup> )
Terracotta tiles (french pattern)	0.12
Timber trussed metal roof	0.12
Gypsum plaster (13 mm)	0.13
Total	0.37

### 3.3 Calculation of Nominal Wall Loads

Three storey low-rise apartments constructed in brick, rendered masonry and framed timber are popular in Australia. Hyne already services the low-rise residential market with other timber products and so, in the short term, this market presents a logical entry point for CLT wall products.

In the late 1970's, a standardised three storey brick block of flats emerged which was very economical to build and popular with investors. Three storey residential units are economical for developers because in general, they do not require the additional expense of services such as pressure boost water pumps and elevators, which are required by medium rise buildings.

In more recent times, architectural flare has seen a trend toward more complex and aesthetically pleasing buildings. As architectural flare is difficult to predict, a standardised rectangular building of similar proportions to the three storey brick block of flats will be used for the purposes of this project.

A cross section of the nominal unit block is indicated in figure 3.1. The CLT wall under consideration will be the external wall on the ground floor. This wall supports floor load widths and half of the roof span.

Standard construction systems producing the highest axial in-plane wall loading case are assumed. The floor is therefore a 300 mm thick tiled timber floor system, consisting of Hyne timber I-Beams, 15 mm thick cement fibre board and tiles over, which is the industry standard for bathrooms on timber framed floors.

The floors on all levels are assumed to be bathroom floors, which are often located similarly

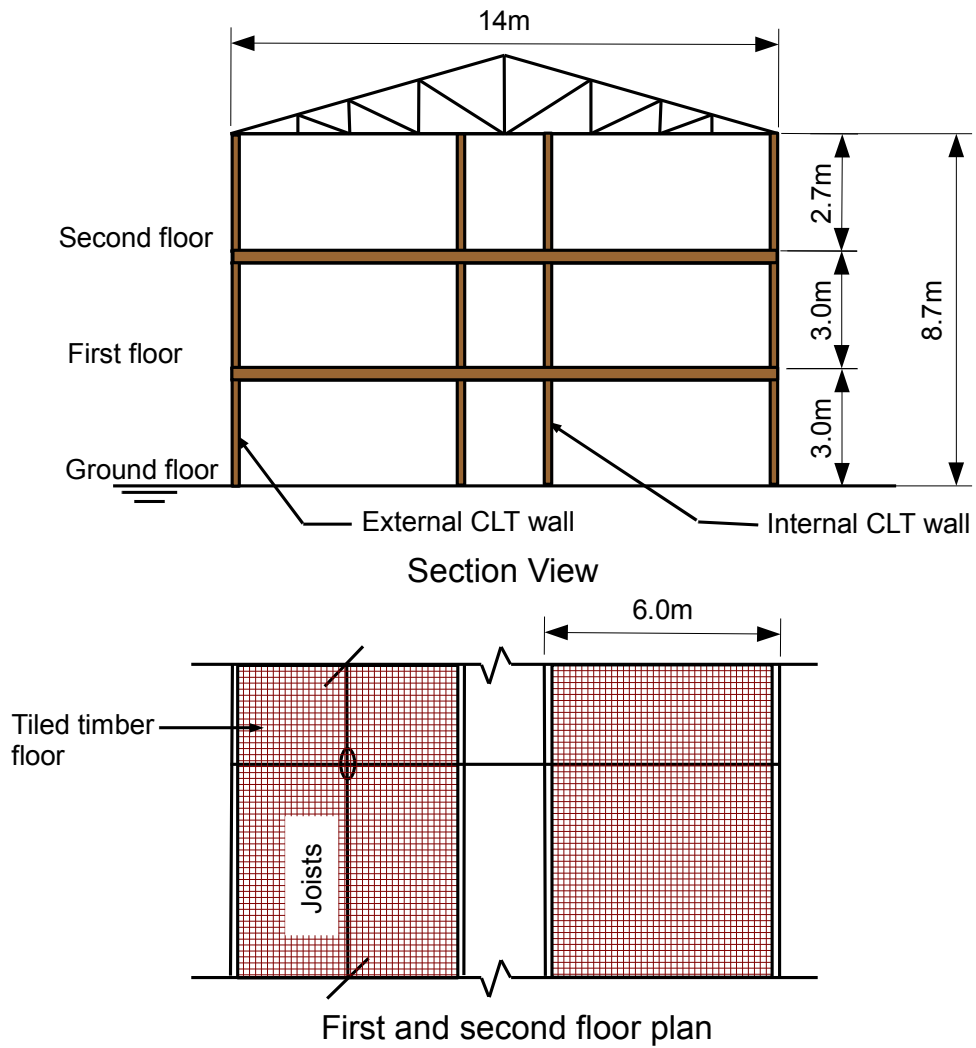


Figure 3.1: Nominal Residential Unit Building

Table 3.4: Floor Dead Loads

Floor Dead Loads	Mass (kN/m <sup>2</sup> )
Tiles	0.27
Fibre cement sheet (15 mm)	0.23
Hyne I-beam joists at 450 mm crs.	0.18
Total	0.68

on each level. The roof system is a timber trussed roof spanning between external walls, with terracotta tiles. Roof and floor dead loads used in the following calculation were taken from AS/NZS 1170.1 Structural Design Actions Part 1: permanent, imposed and other actions (Standards Australia 2002). Dead loads are summarised in tables 3.3 and 3.4.

As the load case considered is for a wall, a universally distributed load (UDL) is appropriate. The following equations 3.4 to 3.6 yield the required axial in plane ultimate load.

Dead Load...

$$\begin{aligned}
 G &= (M_{\text{Roof}} \times RLW) + (M_{\text{Floor}} \times FLW) + (M_{\text{Wall}} \times H_{\text{Wall}}) \\
 &= 0.37 \times 7.0 + 0.68 \times 6 + 0.63 \times 5.7 \\
 &= 10.26 \text{ kN/m}
 \end{aligned}
 \tag{3.4}$$

Where  $G$  is the dead load,  $RLW$  is the roof load width,  $FLW$  is the floor load width,  $H_{\text{Wall}}$  is the wall height and  $M$  is the mass of the associated element. The mass of the wall assumes a 100 mm thick CLT wall section.

Live Load...

$$\begin{aligned}
 Q &= (Q_{\text{Roof}} \times RLW) + (Q_{\text{Floor}} \times FLW) \\
 &= 0.25 \times 7 + 1.50 \times 6.0 \\
 &= 10.75 \text{ kN/m}
 \end{aligned}
 \tag{3.5}$$

Where  $Q$  is the uniformly distributed live load on the wall. Applied live loads  $Q_{\text{Roof}} = 0.25 \text{ kPa}$  and  $Q_{\text{Floor}} = 1.5 \text{ kPa}$  were taken from Table 3.2 of AS/NZS 1170.1 Structural Design Actions Part 1 (Standards Australia 2003, pg.506).

Ultimate Axial Load...

$$\begin{aligned}
 N^* &= 1.2G + 1.5Q \\
 &= 1.2 \times 10.26 + 1.5 \times 10.75 \\
 &= 28.44 \text{ kN/m}
 \end{aligned}
 \tag{3.6}$$



where  $N^*$  is the ultimate axial compressive wall load.

According to AS1720.1-1997 (Standards Australia 2003, pg307), the required design compressive capacity of timber parallel to the grain ( $N_c$ ) can be calculated from the following equation...

$$\begin{aligned} N_c &= \frac{N^*}{\phi} \\ &= \frac{28.44}{0.65} \\ &= 43.7 \text{ kN/m} \end{aligned} \tag{3.7}$$

Where  $\phi$  is the capacity factor for compressively loaded timber for the nominal building.

There is no suggested serviceability limit in AS/NZS 1170.2 for timber walls, however as the walls would likely be lined with plaster or gypsum board, Table C1 specifies an appropriate element response of wall height/300.

### 3.4 Trial CLT Layups

Apart from strength and serviceability, a number of considerations were taken into account when deciding the most suitable CLT layup including:

- Available timber sizes produced at the Hyne timber mill
- Complexity of CLT panel manufacture
- Capacity of the University of Southern Queensland axial load test rig

As the timber mill is optimised for production of timber with specific dimensions, it was decided on the basis of economics that only currently produced dimensional timber sizes would be considered for CLT manufacture. The preferred non-structural grade sizes provided by Hyne were 25x90 mm boards and 35x90 mm and 45x90mm utility grade studs. As resurfacing

is required prior to bonding to ensure an even surface, an allowance of between two and four millimeters was provided for all bonded surfaces.

The preferred layup is three layers, because this reduces manufacturing costs. The advantages of additional layers is increased dimensional stability and homogenisation. To what extent the disadvantages outweigh the advantages or vice-versa is difficult to determine due to the complexity of modeling timber discontinuities using FEA.

Due to the technical difficulties of gluing and applying bonding pressure for curing, Hyne specified that short edges would not be bonded. Applying the required 1 MPa of pressure to the short edge of the boards, as well as the long edge, requires a specialist CLT press, not available at the time of manufacture.

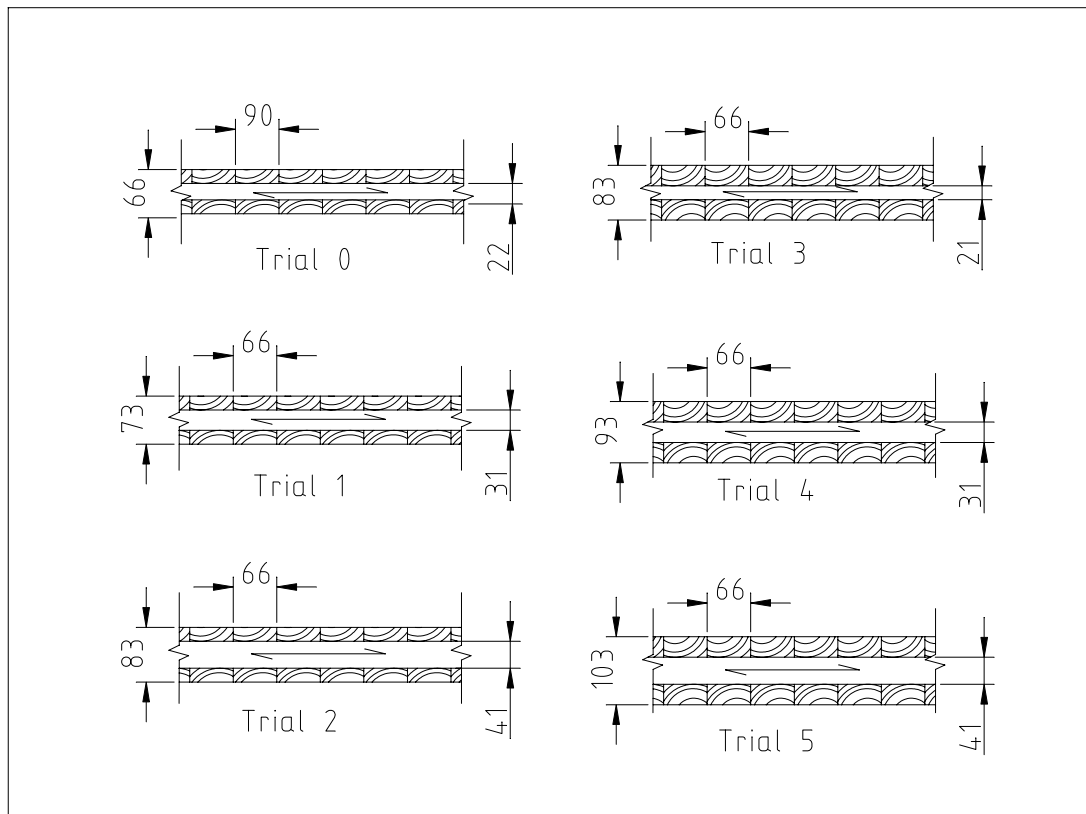


Figure 3.2: Trial CLT Cross Sections

Testing was carried out at the University of Southern Queensland Center of Excellence in Engineered Fibre Composites. The available test rig was limited to a wall height of 2.7 m and a maximum axial load of 200 tonnes. This is much less than the required building nominal load.

Six trial CLT layups were identified for FEA modeling, and a cross section of each is indicated in figure 3.2.

### **3.5 The Finite Element Model**

Strand7 is a general purpose finite element analysis package developed by G+D Computing, and offered by Strand7 Pty Ltd. The software has the capacity to analyse beam, plate and brick elements using a variety of solvers. The two solvers used for analysis in this project are linear static solver and the Linear buckling analysis solver.

The following steps were undertaken for each trial layup. . .

1. Create the geometric model
2. Apply material properties
3. Apply restraint conditions
4. Apply loads
5. Solve for linear static and linear buckling analysis
6. Check model quality

#### **3.5.1 Model Geometry and Material Properties**

Each geometric model was constructed of quad eight brick elements. If smaller than necessary element sizes are used, the model becomes inefficient to solve, and if the elements are too coarse, then the results may be inaccurate.

For simplicity, all quad-eight block elements were modeled with overall dimensions  $100 \text{ mm} \times 100 \text{ mm} \times$  the thickness of the respective lamella. A quality check was undertaken after each analysis to ensure that the brick elements were sufficiently sized to provide accurate results. More details of the quality check are provided later in this section.

The model was created by firstly using the grid command to create a quad eight plate element with the same overall dimensions as the wall panel. The plate was then extruded into brick elements. As there are three lamella, the brick element material properties were incremented such that the outside lamella had the same material property and the central lamella had a different material property.

A single material property could have been assigned to brick elements on a local axis basis, however this would have resulted in a complex FEA model, and differentiation between horizontal and vertical lamella in the Strand7 graphical environment would have been difficult.

For simplicity, two material properties were used, one for each lamella grain direction. The first material property (Material Property 1) was applied to lamella with a vertical (global Y-Axis) grain direction, and the second material property (Material Property 2) was applied to lamella with a horizontal (global X-Axis) grain direction. In Strand7, if no local axis are assigned, material property directions 1,2 and 3 correspond to global directions X,Y and Z respectively.

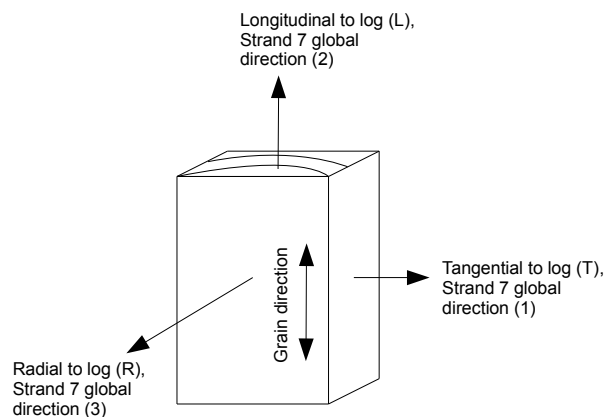


Figure 3.3: Transformed Material Properties for Vertical CLT Lamella

Instead of using a transformation tensor to re-orient the elastic material properties provided in section 3.2, the transformation of axis for material properties was done graphically as shown in figures 3.3 and 3.4.

The resulting Strand7 block element orthotropic material inputs are shown in Figures 3.5 and 3.6, and summarised in Table 3.5.

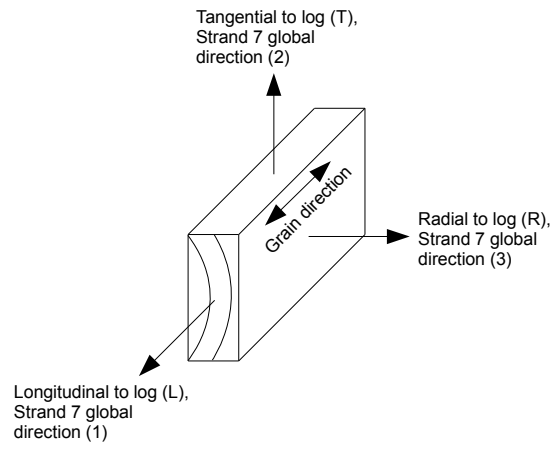


Figure 3.4: Transformed Material Properties for Horizontal CLT Lamella

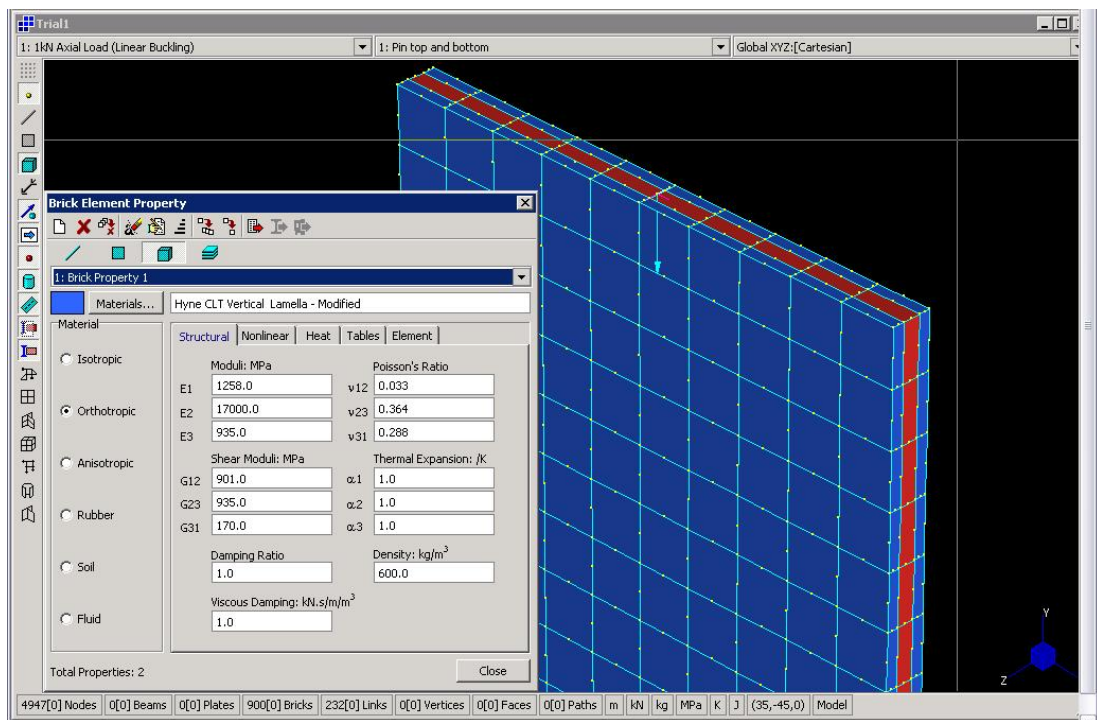


Figure 3.5: Strand7 Orthotropic Block Element Material 1

Table 3.5: FEA Material Properties Summary

USDA Material Property Notation	Strand7 Material Property 1	Strand7 Material Property 2	Material Property Value
Modulus of Elasticity (Mpa)			
$E_L$	$E_2$	$E_1$	17000
$E_T$	$E_1$	$E_2$	1258
$E_R$	$E_3$	$E_3$	935
Shear Moduli (Mpa)			
$G_{LT}$	$G_{21}$	$G_{12}$	901
$G_{TR}$	$G_{13}$	$G_{23}$	170
$G_{RL}$	$G_{32}$	$G_{31}$	935
Poissons Ratio			
$\nu_{LT}$	$\nu_{21}$	$\nu_{12}$	0.444
$\nu_{TR}$	$\nu_{13}$	$\nu_{23}$	0.387
$\nu_{RL}$	$\nu_{32}$	$\nu_{31}$	0.020
$\nu_{TL}$	$\nu_{12}$	$\nu_{21}$	0.033
$\nu_{RT}$	$\nu_{31}$	$\nu_{32}$	0.288
$\nu_{LR}$	$\nu_{23}$	$\nu_{13}$	0.364

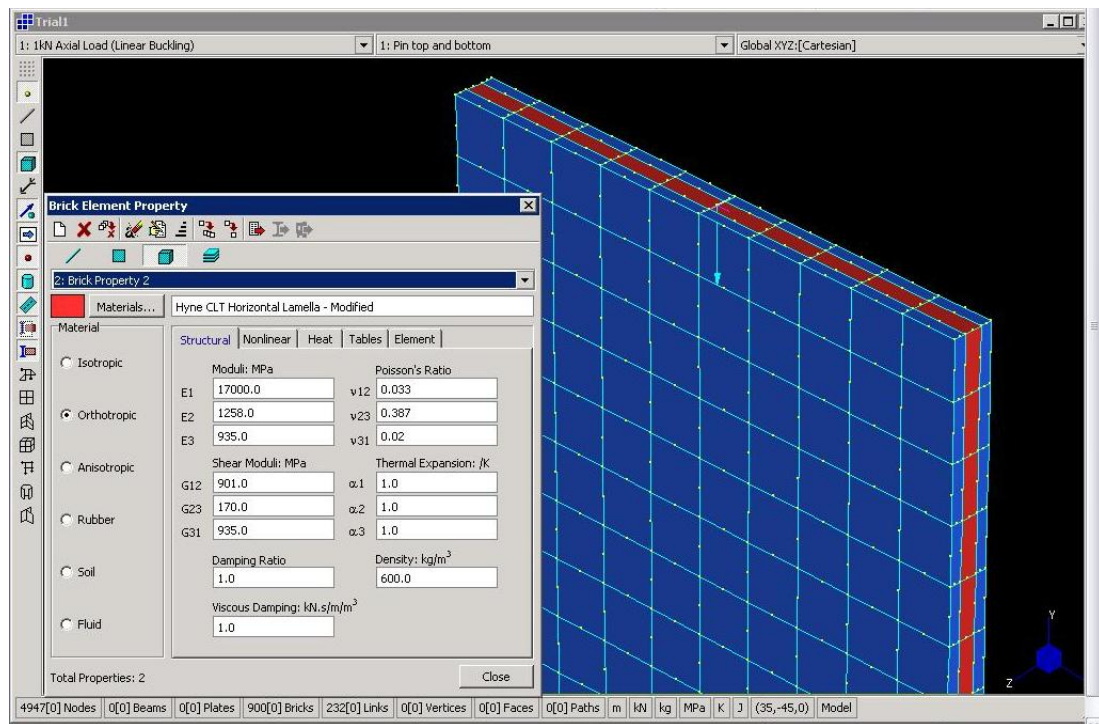


Figure 3.6: Strand7 Orthotropic Block Element Material 2

### 3.5.2 Forces and Restraints

Understanding the restraint conditions to apply to the wall requires knowledge of the construction method to be used. In general, low-rise multistorey CLT buildings are constructed using the platform method. By definition, walls of buildings constructed using the platform method are discontinuous between levels. Manufacturing, transporting and handling walls taller than three meters is not practical.

The flooring system adopted for the nominal building structure is the Hyne I-Beam floor joist product. I-beams are engineered beams consisting of a top and bottom chord connected with a shear web. Given the depth of the nominated floor joists (300 mm), it is not realistic that the floor joists bear directly on top of the walls. In this situation, the connection detail should have the floor joists supported on joist hangers to the side of the wall as shown in Figure 3.7.

The linear analysis solver in Strand7 does not account for load eccentricities, so modeling a load offset to the edge of the wall, even though it is a better representation of the physical connection detail, would not alter the calculated critical buckling load.

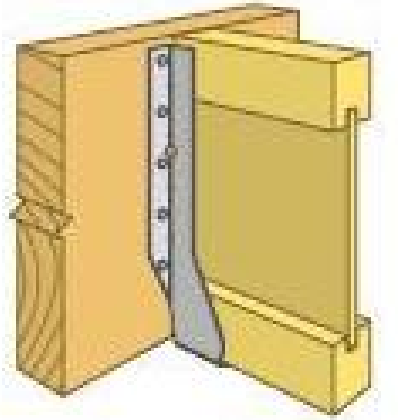


Figure 3.7: I-Joist to CLT Wall Panel Connection(Pryda Australia 2009)

The sides of the walls were modeled as being unrestrained, and no global restraints were applied. Obviously for a continuous wall, there would be some interaction with the continuing wall, and a check of the model quality is discussed in subsequent sections.

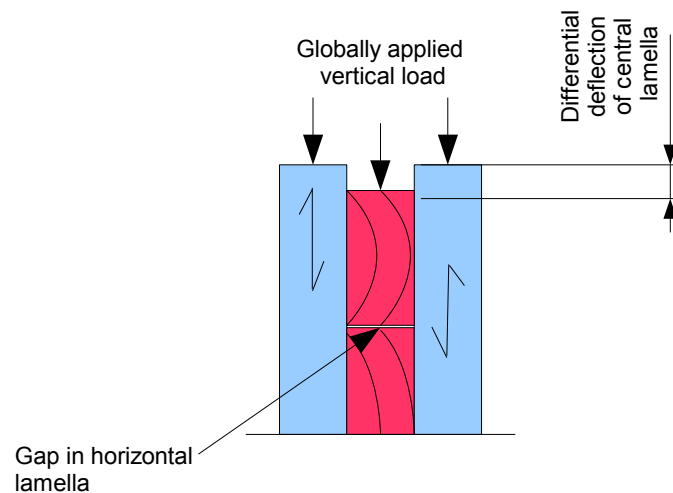


Figure 3.8: Wall Top Edge Local Deflection

Initially as the wall was modeled as block elements, the obvious choice was to apply a global vertical pressure. On closer inspection of the deformations to the top and bottom edges of the wall, the applied global pressure resulted in a pronounced vertical displacement of the central lamella, in relation to the exterior lamella as indicated in Figure 3.8. This was because the



modulus of elasticity of timber parallel to the grain is much lower than that perpendicular to the grain.

Applying a global pressure to the top of the wall is inaccurate for two reasons, firstly because the central lamella cannot displace further than the exterior lamella due to the joist attachment geometry, and secondly, the central horizontal lamella was not pressed along the short edge during fabrication and therefore, may not effectively transfer the applied load between horizontal dimensional timber elements.

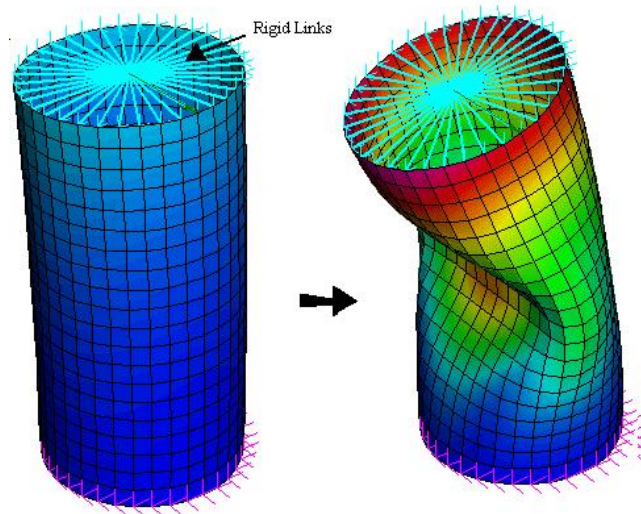


Figure 3.9: Strand7 Rigid Link Example (Strand7 Pty Ltd 2010)

A better method of applying the load to the top of the wall was to rigidly link all of the nodes to a master node in the center of the wall at the top. This is akin to placing an infinitely rigid top and bottom plate to the wall. The concept of applying rigid links is best demonstrated in figure 3.9. Applying rigid links to the top and bottom edges also simplifies the model because node restraints other than the two master nodes were not required.

The rigid links, master node, and applied loads for the top of Trial 1 CLT layup are shown in figure 3.10. The bottom of the wall is similar except that instead of a load being applied to the node, the node is restrained in the global Y-Axis. A summary of node restraints is provided in Table 3.6, where Node 1 refers to the Node located at the bottom of the wall, and Node 2 refers to the node at the top of the wall.

In order to undertake a nonlinear buckling analysis, the linear solver first had to be run to

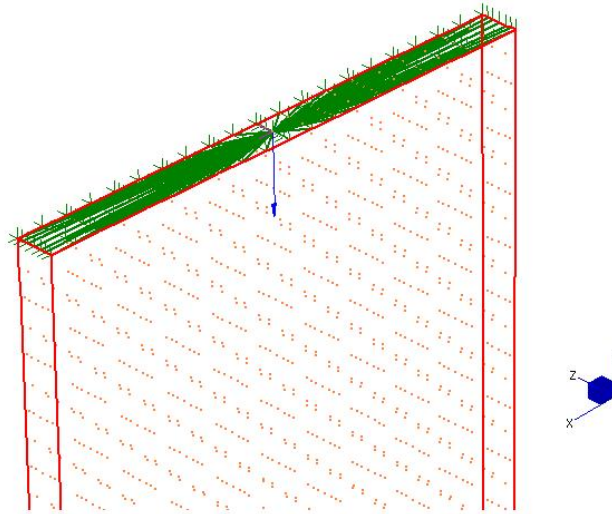


Figure 3.10: Trial 1 Rigid Links, Node Restraints and Unit Load

calculate the element geometric stiffness (Strand7 Pty Ltd 2010). The linear static analysis was undertaken with an applied load of -1 kN in the Y axis direction. This load was applied for the linear static analysis because the output from the linear buckling solver is a single buckling load factor for each buckling mode, which can then be multiplied by the applied load to determine the critical buckling load.

### 3.6 Model Sensitivity and Boundary Conditions Check

A simple check to identify if the element sizes are sufficiently small, is to subdivide all of the brick elements and re-run analysis. When subdivided, each brick element is divided into four new brick elements.

If the difference between the buckling capacities is insignificant, say less than 1% for example, then element size is assumed to be suitable. If the resulting difference in buckling capacities is large, then the wall needs to be broken down into additional elements.

The model quality check was conducted on only one trial CLT layup. This can be justified firstly because all of the CLT layups have similar overall dimensions and slenderness ratios, and secondly, the wall shape is not complex and does not include sharp corners or discontinuities,

Global Axis	Restraint	
	Lateral	Rotational
Node 1		
X	F	R
Y	F	F
Z	F	F
Node 2		
X	F	R
Y	R	F
Z	F	F

Table 3.6: Node Restraint Conditions (F=Fixed R=Released)

which require a higher resolution of elements.

One further quality check was undertaken regarding boundary conditions at the sides of the 1m wall section. The Strand7 model of the wall is 1m long, however in reality, the wall could be as long as the building.

To check the assumption that the linear buckling load of the modeled 1m wall is accurate to be used as a unit length buckling load, and that the boundary conditions on the side of the 1m wall have little effect, the wall was modeled as being two meters long, and the buckling load results compared.

Load comparison is only for the first buckling mode because modes above the first mode are not physically possible for the vertically discontinuous CLT wall section modeled with pin ends. The boundary condition check was only undertaken for the Trial 1 CLT layup.

### 3.7 Results of Finite Element Analysis

This section discusses the results of the linear buckling analysis for each of the CLT trial sections, as well as the sensitivity analysis and the boundary condition check.

### 3.7.1 Trial CLT Finite Element Analysis Results

The linear buckling solver calculated four modes of buckling failure. Though the graphical output indicates displacement and strain, only the critical buckling loads and associated modes of failure are obtained from the linear static buckling analysis.

Deflection and stress data is not attainable unless a non-linear analysis is conducted. A non-linear analysis involves applying loads at load intervals until convergence cannot be obtained. At this point the wall would be said to have buckled.

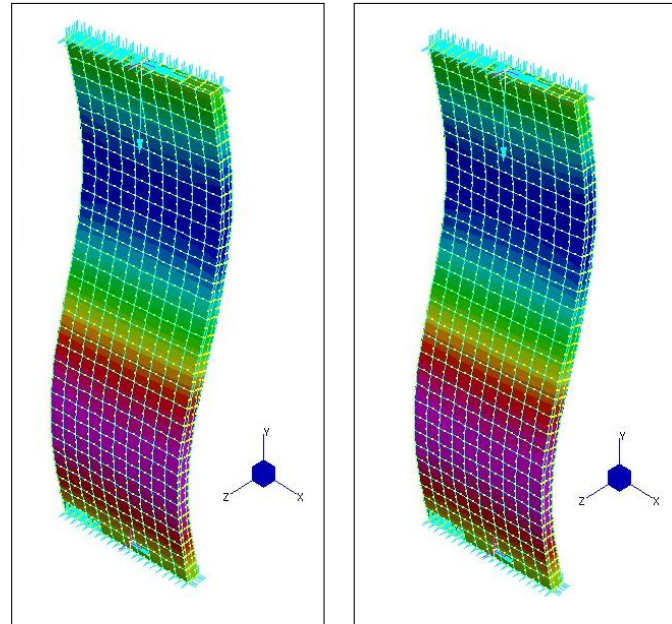
All of the CLT trial samples produced similar buckling mode shapes. The buckling mode shapes provide an insight into possible failure modes which would need to be considered if the walls were continuous over two or more levels. They also provide insight into the effects of the boundary conditions. The four modes of failure for Trial Layup 2 are shown in figure 4.10.

As an example of how the boundary conditions effect the buckling mode, if the wall was modeled as being longer, or the nodes on the sides of the wall were restrained from lateral movement in the Z-direction, the buckling mode shape of figure 3.13b, and the associated critical buckling load would be significantly different.

Table 3.7: Trial CLT Layup Linear Buckling Capacity Results

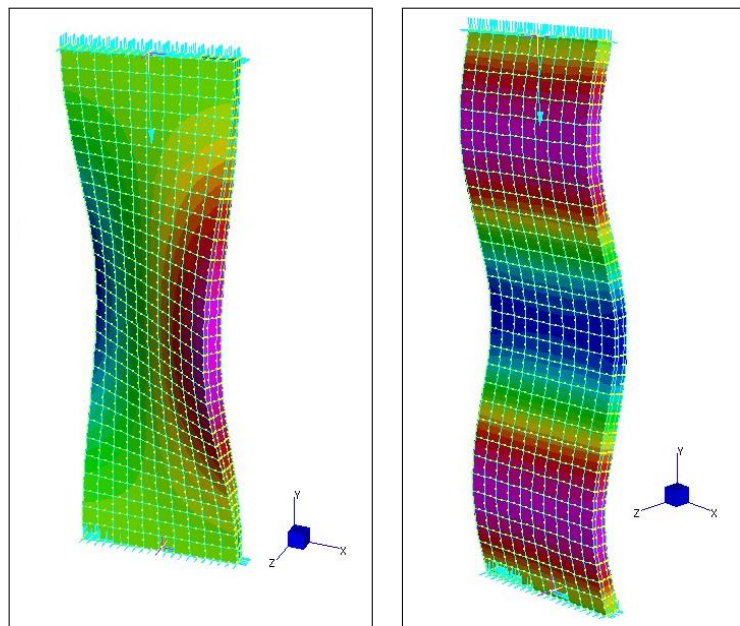
CLT Layup	Linear Static Buckling Loads (kN)			
	Buckling Mode			
	1	2	3	4
Trial 0	367.8	1373.01	2119.71	2777.58
Trial 1	545.28	1974.33	3056.93	3838.1
Trial 2	758.49	2666.64	4153.66	49998.97
Trial 3	845.78	3057.44	4638.57	5930.67
Trial 4	1150.12	3998.16	6037.28	7401.39
Trial 5	1501.5	5026.91	7576.37	8925.38

In this case though, being a platform construction type building, the first mode buckling load is the load of interest, and higher buckling modes are not physically possible. The results of the



(a) Mode 1

(b) Mode 2



(c) Mode 3

(d) Mode 4

Figure 3.11: Critical Buckling Mode Shapes

buckling analysis for each of the five trial Layups are provided in Table 3.7.

In addition to the capacity factor applied to the design action effect, modification factors apply, specifically  $k_1, k_4, k_6, k_{11}$ , which are for the effects of duration of load, moisture condition, temperature, and size respectively.

Table 3.8: Trial CLT Layup Design Buckling Load Capacities

CLT Layup	CLT Critical Buckling Load (kN)	CLT Design Buckling Capacity, ( $\phi N$ ) (kN)
Trial 0	367.8	209.65
Trial 1	545.28	310.81
Trial 2	758.49	432.34
Trial 3	845.78	482.09
Trial 4	1150.12	655.57
Trial 5	1501.5	855.86

Assuming that the building has a design life of 50 years, and is built in normal conditions of temperature and humidity, all of the modification factors are unity, apart from  $k_1$ .  $k_1$  is the duration of load factor and is equal to 0.57. The calculated design compressive buckling capacity of the CLT panels is summarised in table3.8.

### 3.7.2 Sensitivity Analysis and Boundary Condition Check Results

The first mode critical buckling load for the Trial 1 CLT layup is 545.28 kN. Subdividing the model resulted in an increase in the number of brick elements from 300 to 3600. The model was then re-solved, and the resulting critical buckling load decreased to 545.13 kN, which is a net percentage decrease of 0.03 %.

This result indicates that the model has a sufficient element resolution to provide consistent results regardless of increasing the number of brick elements. The Trial 1 CLT layup first mode buckling shape for the normal and subdivided models are shown in figure 3.12.

The boundary condition check is used to identify if a wall length of 1m is sufficient to determine a unit length load, or if the wall length needs to be increased so that effects of local conditions

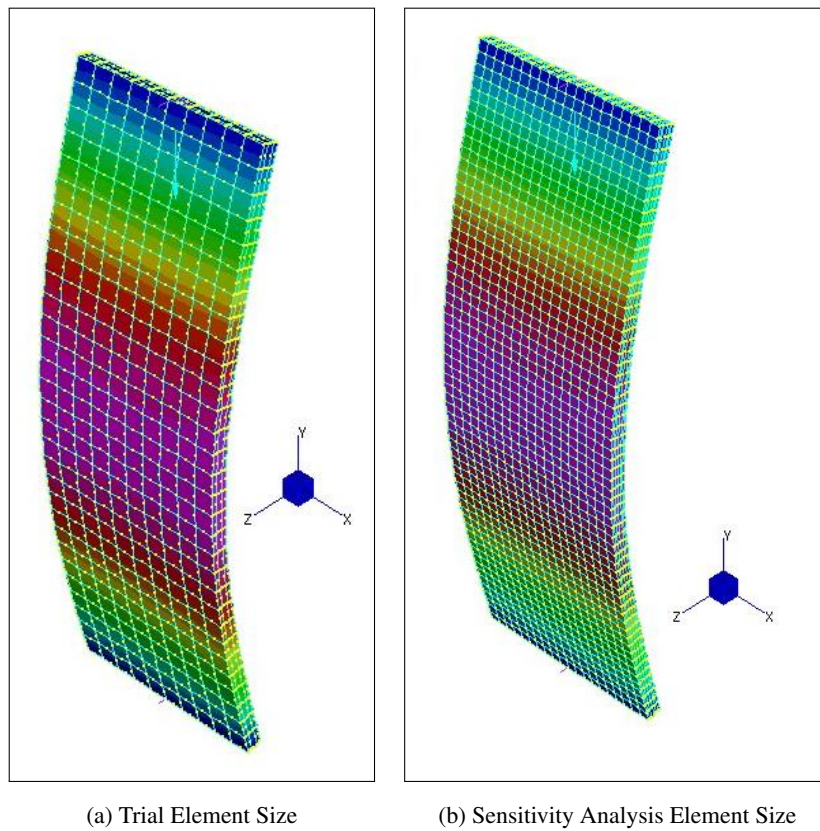


Figure 3.12: Trial 1 CLT Layup Model and Associated Subdivided Model



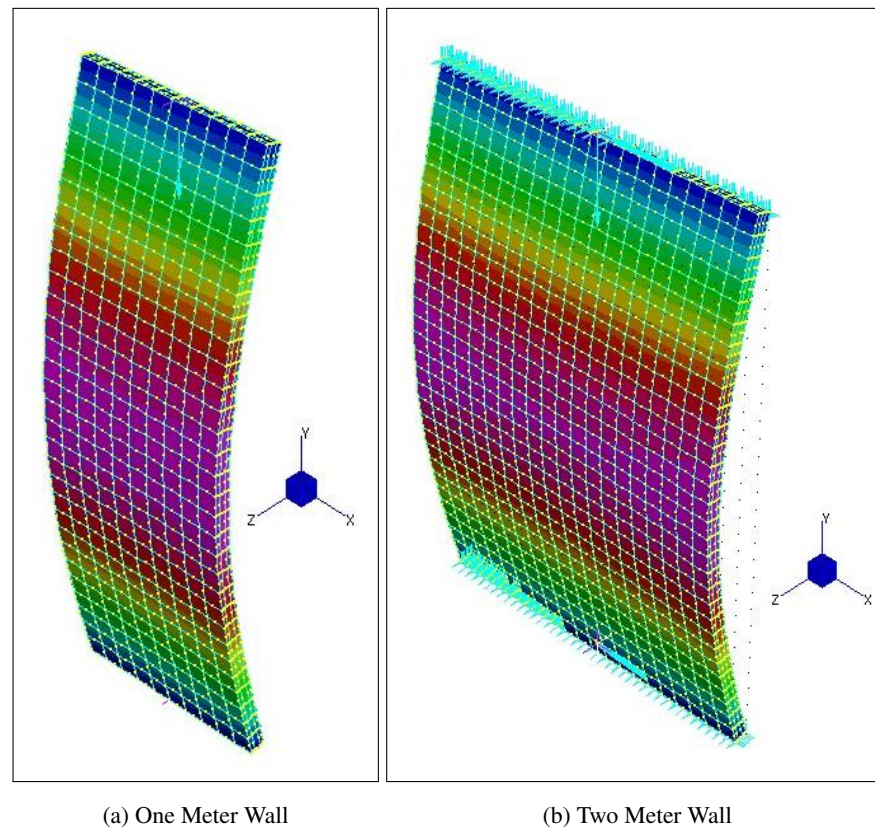


Figure 3.13: Trial 1 CLT Layup Model and Associated Boundary Condition Check Model

at the edge of the wall are made insignificant. To check the boundary condition, the length of the Trial 1 CLT wall model was increased from 1 m to 2 m, and the results compared.

Boundary condition check results indicate that increasing the length of the Trial 1 CLT wall from 1 m to 2 m increases the critical buckling load from 545.13 kN to 546.58 kN. This increase represents a net percentage increase of 0.26%, which is insignificant. The Trial 1 CLT wall layup 1m and 2m long wall models are shown in figure 3.13.

### 3.8 Summary of FEA and Selection of Trial Experimental CLT Layup

Six trial CLT wall panel layups or 'cross sections', which Hyne could feasibly produce, were modeled and analysed in the Strand7 Finite Element package to determine their linear buckling capacities.



Results of the FEA indicate that the trial CLT layups have a much higher design buckling capacity ( $\phi N$ ) than is required for the nominal load case ( $N^*$ ). The thinnest of the six CLT layups, which comprises of three lamella, all 21 mm thick, has a design buckling capacity of 209.65 kN/m, whilst the design action effect for the nominal load case requires a load bearing capacity of only 43.7 kN/m. This represents a safety factor of 4.78.

The design buckling capacity ( $\phi N$ ), has been modified to include the effects of moisture, temperature and duration of load in accordance with AS1720.1-1997, *Timber structures*, Part 1: Design Methods. It should be noted that the design buckling capacity derived from the FEA analysis does not take into account inhomogeneities in timber such as knots and sloping grain, and therefore is not conservative.

The CLT layup which best fits the criteria for experimental testing in terms of meeting the requirements of the intended market, being constructed of dimensional timber sizes currently being produced by Hyne and which can be tested on available test equipment, is the layup which consists of three lamella, each 22 mm thick (22/22/22).

Sensitivity analysis and boundary condition checks were undertaken on the finite element model to ensure its validity in terms of mesh resolution and wall length.

## Chapter 4

# Experimental Design

### 4.1 Introduction

In order to meet the requirements of the project specification, two experiments were required to be conducted, those being the in-plane load capacity and a bending test to determine the modulus of elasticity of the CLT panel. This chapter is intended to provide details of the two test procedures developed to determine the required properties.

Sample preparation for axial load testing at the University of Southern Queensland involved trimming three samples provided by Hyne to 2.7 m long so that they would fit between the load plates of the axial load test rig. No further preparation of the samples was required for either of the tests.

Though the bending test was not specifically included in the project specification, it was required to check the validity of the accepted formula for the buckling strength of plywood diaphragms. Also included in this chapter is an overview of the fabrication of the experimental test samples prepared by Hyne. Six test samples were produced to specification, each measuring 275 mm × 68 mm × 3000 mm.

### 4.1.1 Materials

Machine grading equipment at the Hyne & Son Tuan Mill in Maryborough, QLD, optimizes the yield of high grade timber by detecting and removing sections of lower grade timber, such as knots and timber with sloping grain, from otherwise high grade stock. The 25 mm × 90 mm boards used to fabricate the CLT samples were tested, but not graded, during milling.

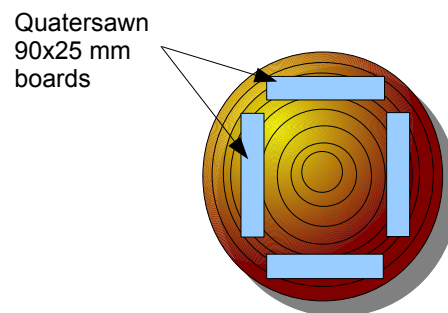


Figure 4.1: Locations of 25 mm × 90 mm Boards within Parent Log

Typical commercial application of the 25 mm × 90 mm timber boards produced by Hyne is for use as fence posts and other similar non-structural applications. The boards are quartersawn from the outside of the parent log as shown indicatively in figure 4.1. For Australian Slash and Caribbean Pine, boards cut from positions close to the outside of the log typically have a higher modulus of elasticity than timber from the rest of the log.

Prior to fabricating the sample panels, tests were conducted to determine the density and acoustic modulus of elasticity of the dimensional timber. The moisture content was tested using a Wagner Hand Held Moisture Meter similar to the one shown in figure 4.2, whilst the dimensions, mass and density were drawn from records taken by the proprietary testing equipment used at the Hyne mill.



Figure 4.2: Wagner Moisture Meter

#### 4.1.2 Sample Fabrication Procedure

The sample width was dictated by the requirement for handling during testing at the University, and also dimensions of the boards used in the sample fabrication. The dimensions of the boards were 22 mm × 90 mm. The samples were fabricated three boards wide (270 mm) for simplicity of fabrication, and to reduce boundary condition effects on the sides of the wall samples.

The process of fabricating the CLT samples panels is as follows...

- The boards for the first CLT lamella were assembled on an LVL platen press
- Glue was evenly applied to the exposed upper surface of the first lamella
- The second lamella was placed over the first lamella noting orthogonal grain orientation
- Glue was evenly applied to the exposed upper surface of the second lamella
- A final check that all of the boards were assembled as closely as possible was undertaken
- A 0.5 MPa pressure was applied using the press, and the assembly was left to cure

- After a curing time of 25 min, the sample was removed from the press

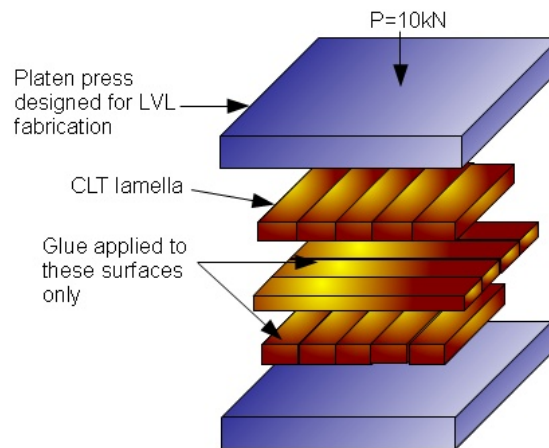


Figure 4.3: Assembly of CLT Panel

During production of the first sample CLT panel, a load was applied to the sides in order to compress the individual timber elements together on their short edge, however, this resulted in the upper lamella arching up in the center. Subsequent samples were produced without any side loads. An exploded view of the CLT sample in the LVL platen press is provided in figure 4.3.

The glue used in the production of the CLT panels was Purbond HB S109, which is a new generation single component isocyanate adhesive developed by Purbond. The adhesive has an open time (or working time) of 10 min, and a press time of 25 min before curing. A chemical reaction between moisture in the atmosphere and the glue causes the glue to cure (Purbond 2010).

Given the short time frame for assembly of the dimensional timber elements, it was possible that some areas of glue on the first laminate tacked-off prior to pressing. KLH Massivholz GmbH, a recognised CLT fabricator in Europe, recommends pressing laminates with a pressure of 60 kPa to achieve a high quality adhesion (KLH Massivholz GmbH 2010). The LVL press used to fabricate the samples could achieve a maximum pressure of 0.5 MPa.

To reduce the possibility of moisture content variation between measuring the moisture content between fabrication and testing, the samples were kept undercover at all times prior to testing at the University of Southern Queensland. The fabricated CLT samples are shown in figures 4.4.



Figure 4.4: CLT Samples Fabricated by Hyne

## 4.2 Axial Load Capacity of CLT Wall Panels

### 4.2.1 Introduction

This section provides an overview of the experimental design of the axial load test. A review of the technical aspects of the experiment followed by development of detailed experimental procedures is undertaken.

A number of samples fabricated by Hyne to the same specification were tested. Ideally the samples would have been tested to the full 3.0 m wall height as defined by the nominal residential building case, however the maximum height which could be tested in the axial load test rig at the University of Southern Queensland was 2.7 m.

When timber is loaded axially, typical failure modes are normal compressive failure, beam-column failure and buckling failure, depending on the geometry of the sample and applied load conditions. Other failure modes may result from flexure of the wall.

The slenderness ratio and stiffness of the wall determine the critical elastic buckling load. Due to assumptions made in the development of Euler's theory, the actual ultimate buckling load can be much less than the theoretical critical buckling load. The critical buckling load determined by Linear Static FEA is the theoretical upper limit of axial strength within the elastic limit of

the material.

The test setup was chosen to provide an indication of the wall's load capacity in terms of serviceability and ultimate strength. The serviceable limit for mid-height deflection as defined for the nominal residential multistorey building is  $\frac{h}{300}$ , where  $h$  is the height of the wall. Therefore, the mid height deflection of the CLT wall panel was measured and logged in relation to the applied load.

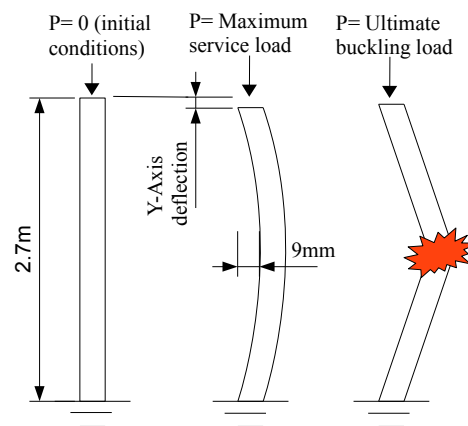


Figure 4.5: Axial Load Test, Loads of Interest

Axial deflection, though not specifically defined in AS1170.2 as a serviceability criterion for walls, was measured and logged in relation to the applied load. All three CLT wall samples were loaded to destruction to determine their ultimate axial buckling load capacity. The loads and displacements required to be recorded in the axial load test are shown diagrammatically in figure 4.5.

#### 4.2.2 Test Procedure

The following equipment was used to undertake the axial load testing:

1. 240V Circular Saw
2. 2000kN Axial load test rig fixed to a concrete strong floor

3. 300t Hydraulic ram and associated hydraulic power-pack (Enerpac models AC CLRG 30012 and Enerpac ZE5 class respectively)
4. 20t Load cell (Anyload Model 363YH)
5. Safety cage and associated fasteners for fixing to the test rig columns
6. Load top and bottom plates, and associated chain restraints
7. 2 × Cable extension position transducers (Celesco PT Series)
8. Screw to attach cable to sample for mid-height deflection measurement
9. Sturdy stand to mount the mid height deflection cable extension position transducer
10. Numerous G-Clamps
11. Signal processor and computer with proprietary laboratory calibration and data recording software

Steps involved in setting up for axial load test are as follows:

1. As the CLT wall samples requested from Hyne were 3.0m high, and the maximum height which would fit into the test rig was only 2.7m, the three samples which were to be used for Axial load testing were measured and docked to length using a circular saw.
2. The axial load test rig also needed to be setup to accept the 2.7m tall samples. This involved repositioning the cross beam of the test rig to the maximum height possible. Access to fasten the bolts for the test rig was achieved using a dog box on a forklift. The bolts retaining the cross beam were tensioned loosely with a hand wrench.
3. Next, the top load plate was installed. The top load plate distributed the load from the hydraulic ram evenly across the top of the CLT sample. Short chains were installed at either end of the load plate, so that when the sample failed, the load plate was restrained. The chains were then shackled to lugs mounted on the sides of the hydraulic ram, with sufficient additional length that the full travel of the ram was still achievable. The top plate and chains are shown in figure 4.6.



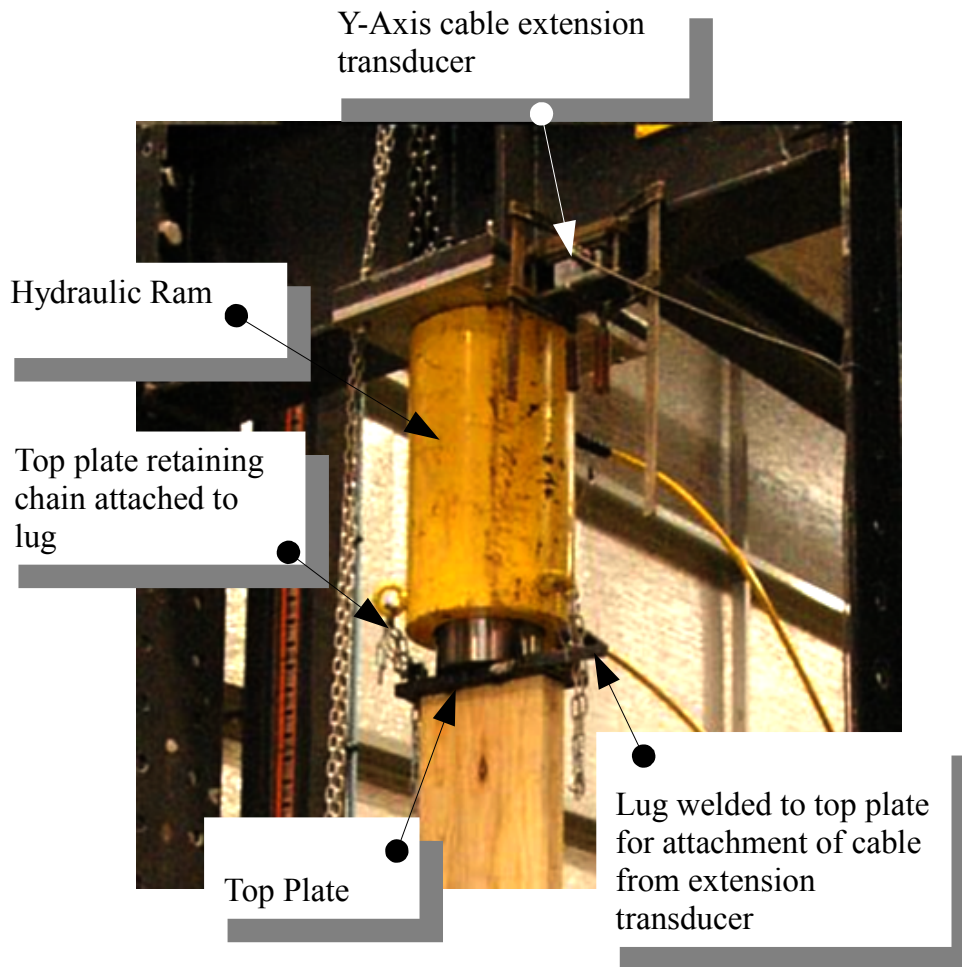
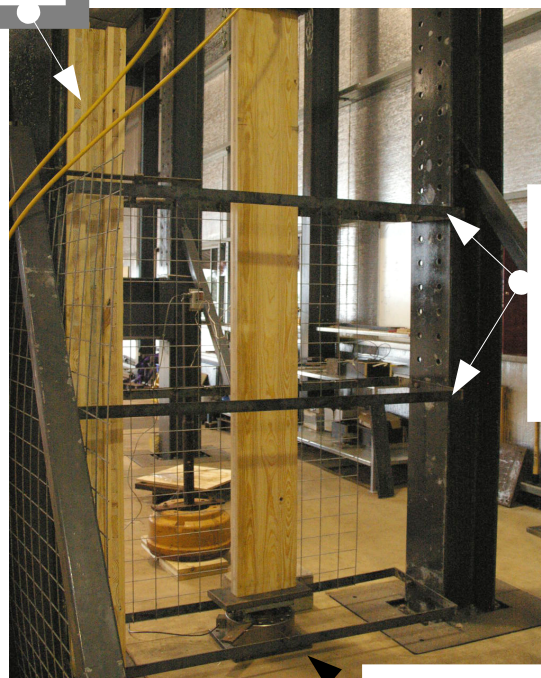


Figure 4.6: Load Application and Vertical Deflection Measurement

Samples awaiting testing positioned inside cage.



50x6mm Steel safety restraining rings, bolted to the test rig columns. The rings were also used to support the guard mesh.

Load cell and bottom plate

Figure 4.7: Safety Cage, load cell and Sample Panels

4. The Y-axis Cable Extension Position Transducer was then securely mounted to the underside of the test rig cross beam with two G-Clamps. The cable from the position transducer was then attached to a lug welded to the top load plate as shown in figure 4.6. Attaching the cable to the top load plate isolated the extension cable from the sample. Isolating the extension cable from the sample was necessary to limit damage to the instrument upon failure of the sample.
5. A mesh safety cage was then installed between the test rig columns. The mesh on one side of the safety cage was removable, allowing the samples to be positioned inside the cage. Rather than remove the cage each time a new sample was required, all of the samples were positioned inside the cage, and restrained against the test rig column as indicated in figure 4.7.
6. The bottom load plate and 200t load cell were then positioned at the bottom of the test rig.
7. The Z-axis Cable Extension Position Transducer was then bolted to a specially fabricated rigid steel stand as indicated in figure 4.8a. The stand was positioned in front of the wall and the base packed to ensure stability. The height of the position transducer was then adjusted to match the mid-height location of the wall sample.
8. After all of the instruments were connected to the signal processor/datalogger, which is shown in figure 4.8c, the first sample was positioned in place, and a small starting load applied to hold it.
9. A small self tapping screw was then installed at mid-height of the wall sample, and the Z-axis Extension Position Transducer attached to it.
10. The removable safety cage mesh was then installed as shown in figure 4.8b, and all of the instruments zeroed, ready for testing.
11. Load was applied over a couple of minutes to achieve a good data capture resolution, and to achieve failure within a five minute period. Once the sample had failed, the mid-height deflection anchor point (self tapping screw) was removed.
12. This procedure was repeated for three samples



(a) Z-Axis Transducer Stand



(b) Refitting of Mesh



(c) Signal Processor and Computer



(d) Hydraulic Power Pack

Figure 4.8: Test Equipment Setup

## 4.3 Effective Modulus of Elasticity of CLT Wall Panels

### 4.3.1 Introduction

For clarity, the term 'apparent modulus of elasticity' will be used in this report to describe the modulus of elasticity of solid timber including knots and other discontinuities, whereas the 'effective modulus of elasticity' will be used to describe the modulus of elasticity of the CLT panels.

The modulus of elasticity is the single most useful piece of information to describe the strength properties of dimensional timber. The effective modulus of elasticity of CLT panels includes additional information about the homogenising effects of laminating, laminate geometry and grain direction.

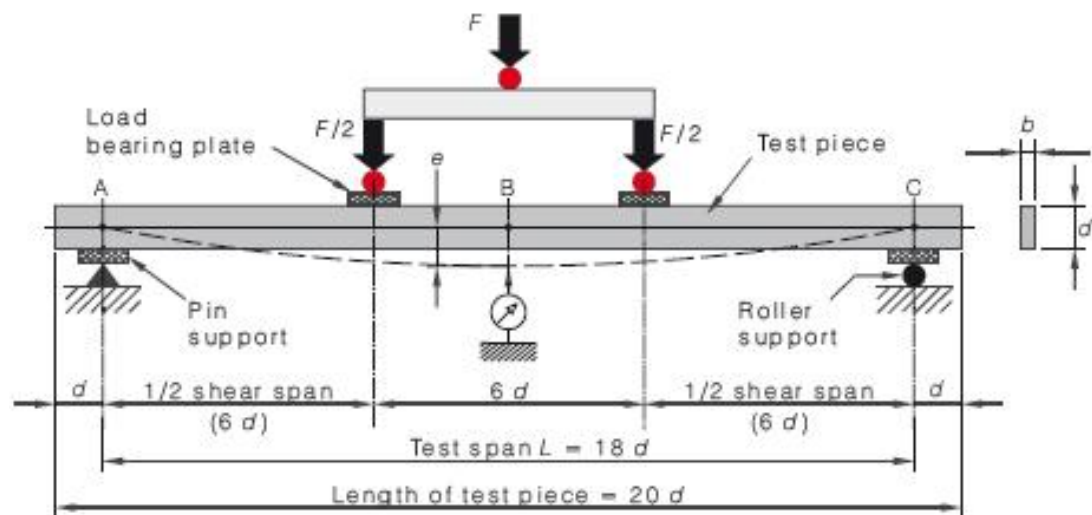
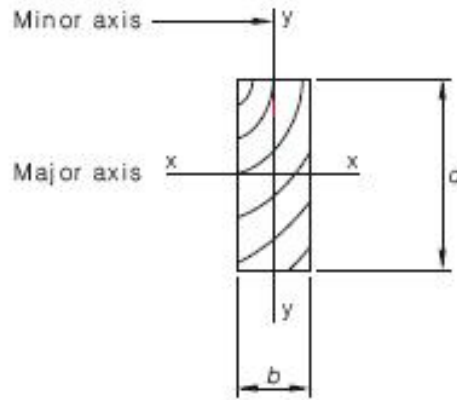


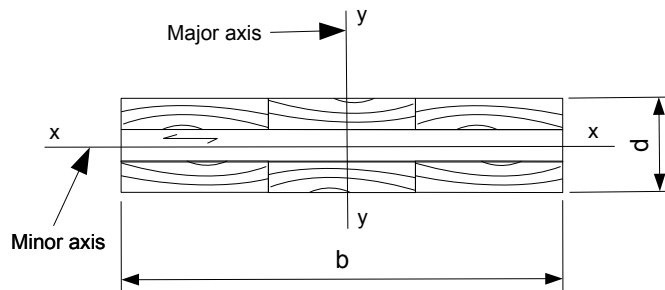
Figure 4.9: Four Point Bending Test (Standards Australia 2010a)

The method for determining the apparent modulus of elasticity of solid timber is the four point bending test, as described in chapter two of AS4063.1-2010, Characterization of structural timber. This method was adopted, with some modification, for testing the CLT wall panel effective modulus of elasticity. The four point bending moment test according to AS4063.1-2010 is shown in Figure 4.9.

The sample is to the same specification as that used for the axial load capacity test, which



(a) AS4063.1 Sample Cross Section



(b) CLT Cross Section Notation

Figure 4.10: CLT VS AS4063.1 Sample Cross Section Notation (Standards Australia 2010a, p.5)

is a layup of 22 mm thick x 90mm boards in three layers, with the central lamella placed orthogonally to the outside lamella (i.e. 22/22/22 layup). The breadth of the sample tested was 270 mm, and the length of the test piece was 3.0m, using the notation provided in figure 4.9.

Modification of the four point bending test was required to ensure a bending type failure of the test samples as opposed to shear failure. According to AS4063.1 (Standards Australia 2010a), the breadth of the test sample is the lesser cross-sectional dimension of the test piece, as indicated in Figure 4.10a.

The CLT wall panel sample has a very different cross section as shown in figure 4.10b, and when using three point test setup ratios indicated in figure 4.9, indicates a span between centres of approximately 1.2m.

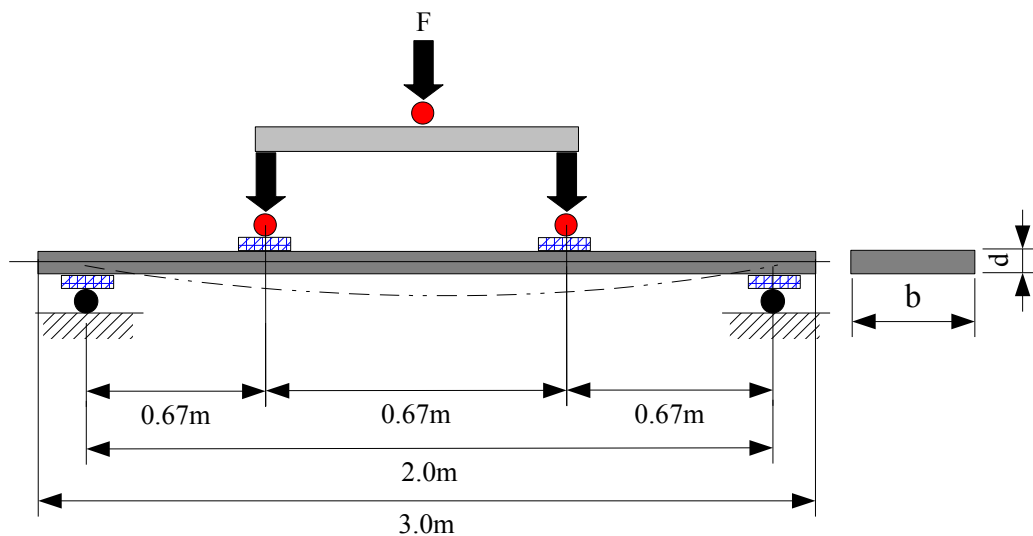


Figure 4.11: Modified Four Point Bending Test Dimensions

Given that the shear capacity of the glue was unknown, and considering the sample width to depth ratio, the test setup was rationalised by increasing the span between supports from 1.2 m to 2.0 m to induce a bending type. The rationalised dimensions of the modified four point bending test are shown in Figure 4.11.

Considering the application of the CLT panels will have a span between centres of 3.0 m



or greater in most cases, it follows that increasing the span to 2.0 m would provide more representative results.

In any case, the modulus of elasticity could still be determined even if the failure mode was shear failure, provided a clear linear elastic load range could be determined from the load vs displacement plot. According to AS4063.1 (Standards Australia 2010*a*, p.8), the equation for the modulus of elasticity is...

$$E = \frac{23}{108} \left( \frac{L}{d} \right)^3 \left( \frac{\Delta F}{\Delta e} \right) \frac{1}{b} \quad (4.1)$$

where  $\frac{\Delta F}{\Delta e}$  is the linear elastic slope of the load-displacement graph.

As the four point bending test is a standard test, no further theory or procedures are provided, and the reader is directed to chapter two of AS4063.1 (Standards Australia 2010*a*) for further details.

### **4.3.2 Test Equipment**

Specific equipment required for the Modified Bending Test at the University of Southern Queensland is shown in figure 4.12, and itemised as follows...

1. Universal beam test rig with concrete strong floor
2. 300t Hydraulic ram and associated hydraulic power-pack (Enerpac models AC CLRG 30012 and Enerpac ZE5 class respectively)
3. 20t Load cell (Anyload Model 363YH)
4. Load spreader beam with rollers and load plates to limit perpendicular to grain local crushing
5. Universal beam supports with load plates to limit perpendicular to grain local crushing
6. Cable Extension Position Transducer (Celesco PT Series)
7. PC with proprietary laboratory calibration and data recording software (not shown)



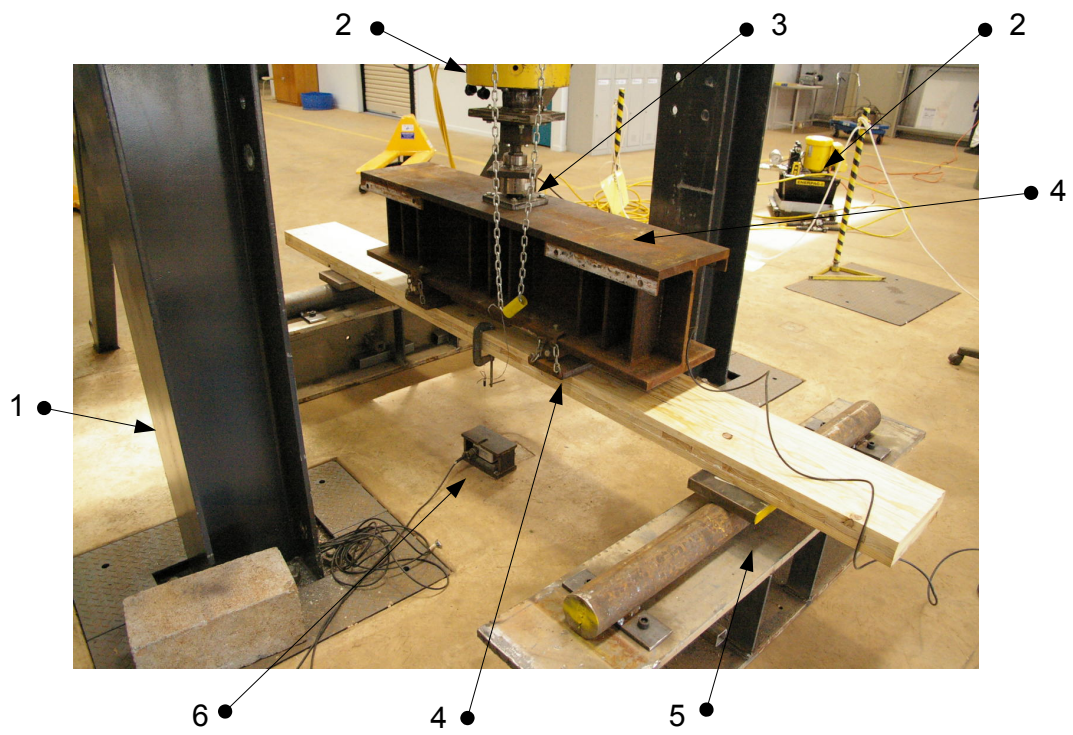


Figure 4.12: Four Point Bending Test Equipment

## **Chapter 5**

# **Results and Discussion**

### **5.1 Axial Load Test Results and Discussion**

Due to the high axial loads, and hence large stored potential energy during testing, it was initially decided that the deflection should be limited to around 150 mm. Exceeding this limit posed a safety risk.

The first sample tested reached its ultimate axial load capacity, then retained a degree of post failure strength right up until the 150 mm deflection safety limit was reached. Some loud cracking noises were heard as the load was applied.

It was not clear at exactly when the ultimate load was reached without monitoring the load during the testing because the failure was not rapid or violent. The first sample was then removed from the test rig and the testing continued with the second and third samples. A photo of the sample loaded showing post ultimate load deflection is provided in figure 5.1.

Apart from a small deflection, the second sample reached its ultimate axial load capacity without indication of impending failure. The failure was very rapid, and a loud cracking noise was emitted.

The second sample buckled in the opposite direction to the first and third test samples. The second sample was removed and replaced with the third test sample for the final axial load test.



Figure 5.1: Sample 1, Post Ultimate Load Deflection



(a) Just Prior to Failure



(b) Post Failure

Figure 5.2: Sample 2. Photo Sequence

Table 5.1: Service and Ultimate Axial Load Summary

Sample No.	Maximum Service Load (kN)	Ultimate Axial Load (kN)
1	350	370
2	200	270
3	340	345
Mean Average	297	328

The failure mode of the third test sample was similar to the second.

A two photo sequence of the load testing of sample two is provided in Figure 5.2. The first photo was taken just prior to failure.

A plot of axial load versus the displacement in the Y-direction (i.e. change in height of the wall) for all three samples is included in figure 5.3. The plot indicates that the ultimate axial load for the three samples varies between 275 kN and 370 kN.

All three samples had similar elasticity, however, the first sample showed signs of slipping and load redistribution resulting in a 1mm initial displacement. Axial deflections at ultimate axial loads ranged from approximately six to eight millimeters.

A plot of the axial load versus mid-height deflection is shown in figure 5.4. From the plot, it can be seen that the mid-height horizontal displacements for the samples at ultimate axial load ranged between 10 mm and 28 mm. All of the plot shapes were consistent.

The second and third sample post ultimate load strength terminated at approximately 35 mm deflection, as indicated by the straight line thereafter, whilst sample one retained some post ultimate failure load capacity at nearly 40mm deflection.

In meeting the aims of the axial load test, Table 5.1 provides a summary of the ultimate and service loads achieved by the three CLT samples.

Failed samples two and three were retained and disassembled with a pry bar to ascertain information on what may have caused the failure. Failure was in general along the glue line. It is not clear if the failure of samples two and three emanated from the top of the sample, or from



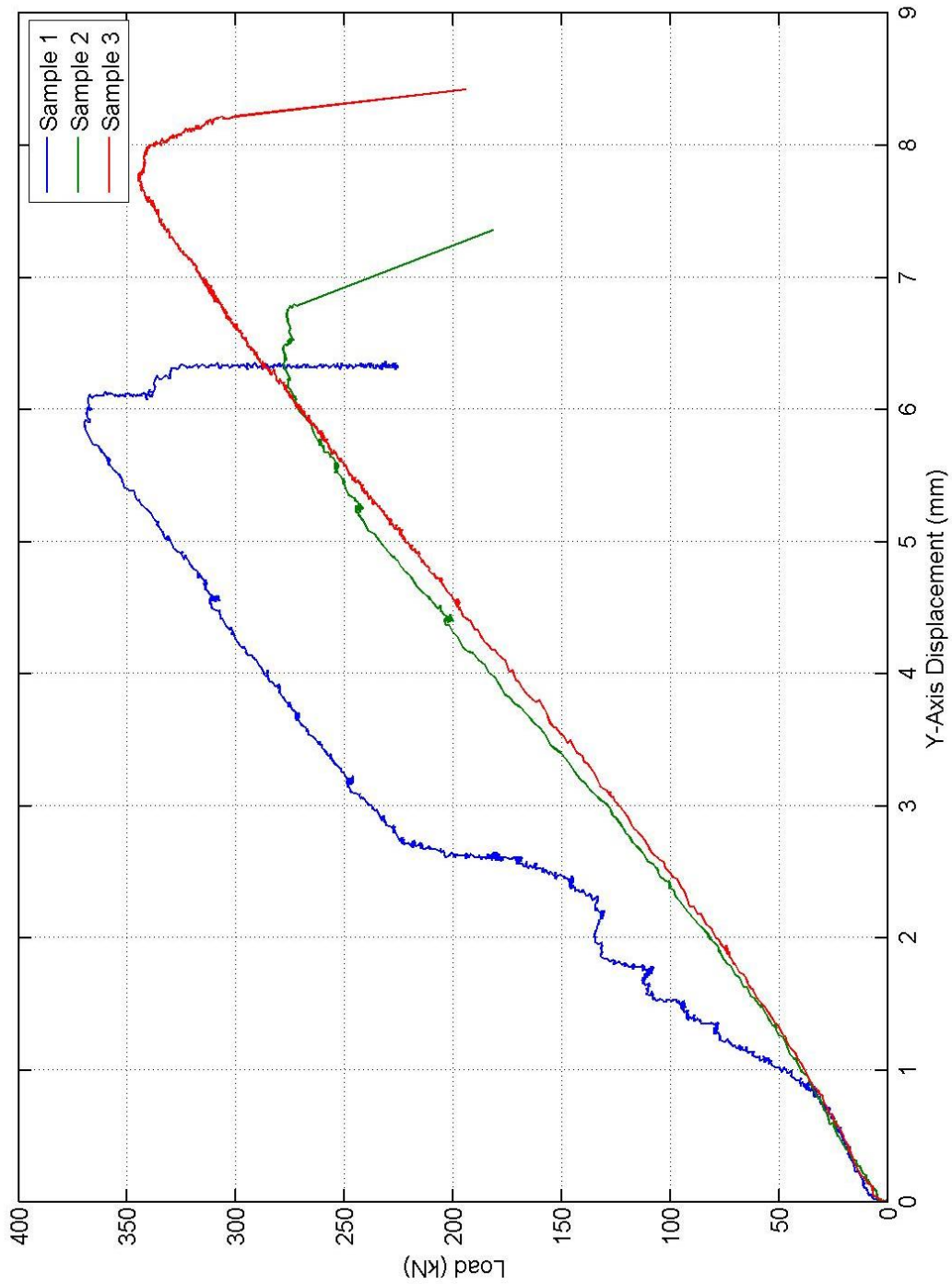


Figure 5.3: Axial Load VS Vertical Displacement

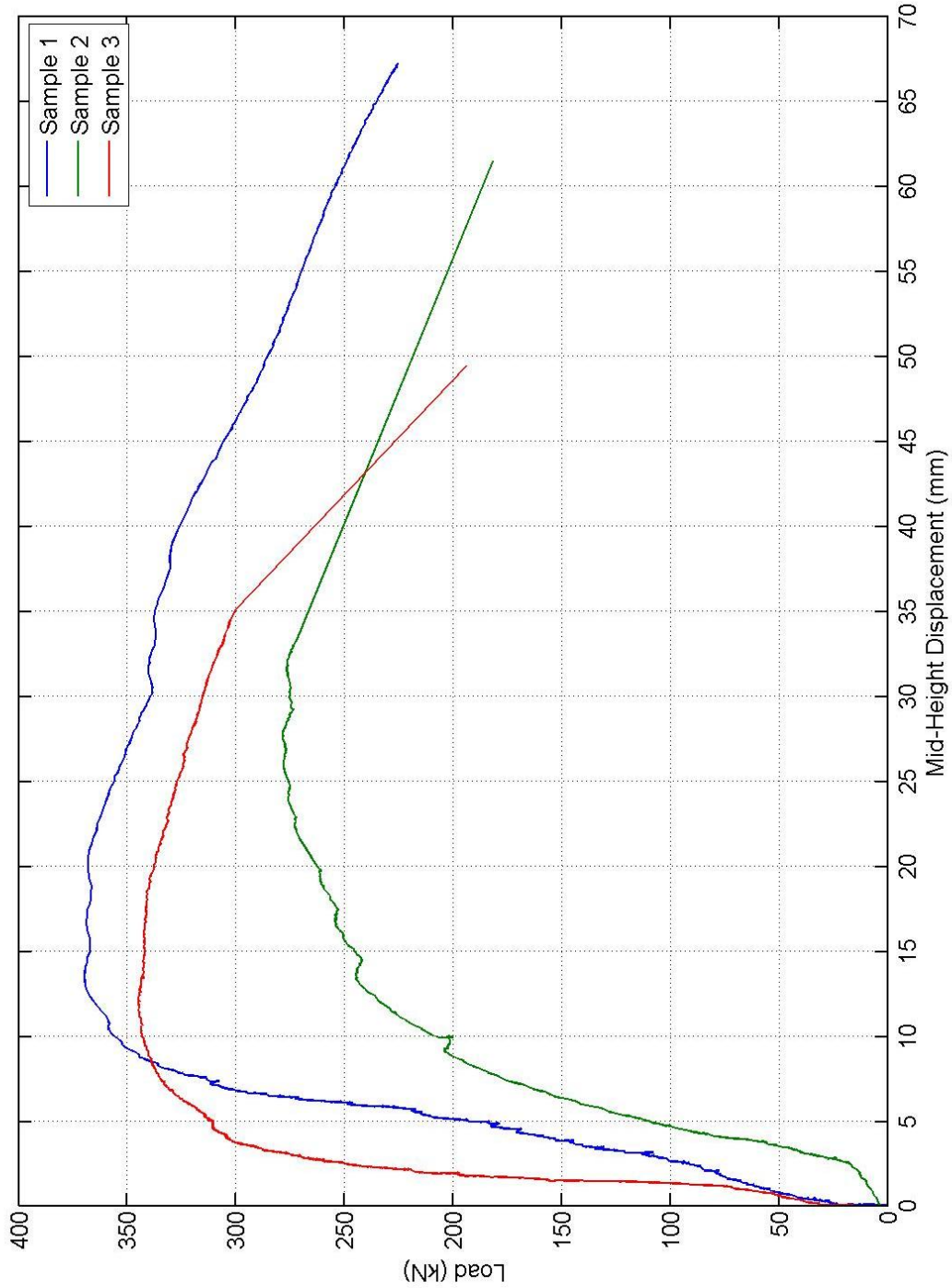


Figure 5.4: Axial Load VS Mid-Height Horizontal Displacement

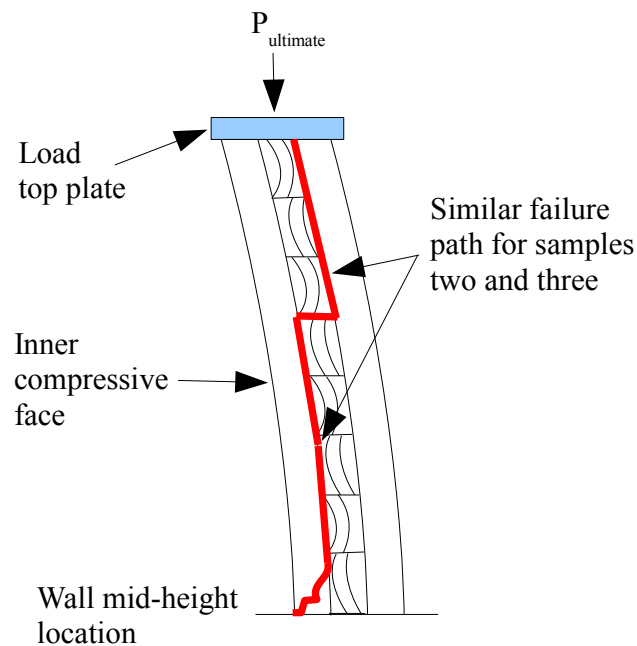


Figure 5.5: Fracture Path for Samples Two and Three

the inner compressive fibre, mid height.

In each case, the timber fibres were observed to be retained in over 30 % of the delaminated glue area, indicating insufficient glue adhesion to approximately 70 %. A diagram showing the general path of the fracture line for samples two and three is included in figure 5.5.

A photo of the the delaminated glue surface showing retained timber fibres is indicated in figure 5.6. It was also noted that the timber which was retained in the glue came from the central laminate.

## 5.2 Four Point Bending Test Results and Discussion

The four point bending test was undertaken in accordance with AS4063.1-2010, Characterization of structural timber, with modification to the span of the supports, as indicated in Figure 4.11 at the University of Southern Queensland Centre of Excellence in Engineered Fibre Composites (CEEFC) laboratory, Toowoomba.

The test was required so that an effective modulus of elasticity for the CLT panel could be de-





Figure 5.6: Sample Three Delaminated Glue Surface

terminated. The results will be used for comparison with the modulus of elasticity as determined from the plywood composite theory, which is discussed in more detail in Chapter 5.3.

On application of the load a noticeable mid-span deflection increased linearly, until failure. There was the occasional cracking sound at the sample flexed, followed by a clear cracking sound when the ultimate load was reached.

The CLT lamella had delaminated between the load application points and the supports at either end of the panel, as indicated in figure 5.7, and the convex shape of the sample during loading changed to include contra-flexure at the ends of the sample as shown in figure 5.8.

Though the test setup aimed to avoid a shear type failure, it was immediately evident that shear failure along the glue line was the failure mode as indicated in figure 5.9. This is verified by observing the locations of maximum shear in the shear force diagram for a four point bending test shown in figure 5.10.



Figure 5.7: Four Point Bending Test:Elastic Region

The linearity of the elastic region of the load versus displacement plot as indicated in figure 5.11, was surprisingly smooth apart from one small glitch at approximately 16 kN of load,





Figure 5.8: Four Point Bending Test: Post Failure



Figure 5.9: Delamination of CLT Due To Flexural Shear

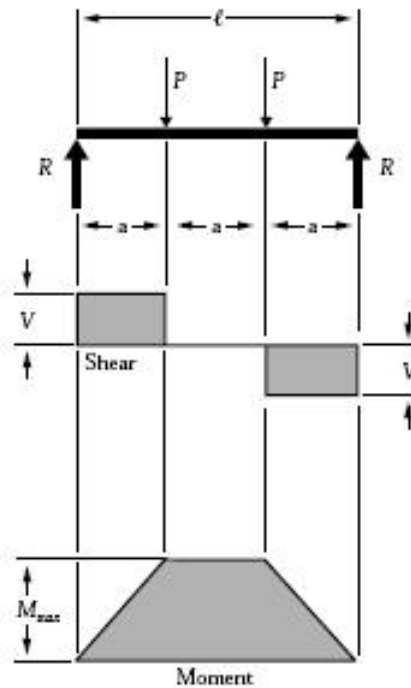


Figure 5.10: Shear Force and Bending Moment Diagram

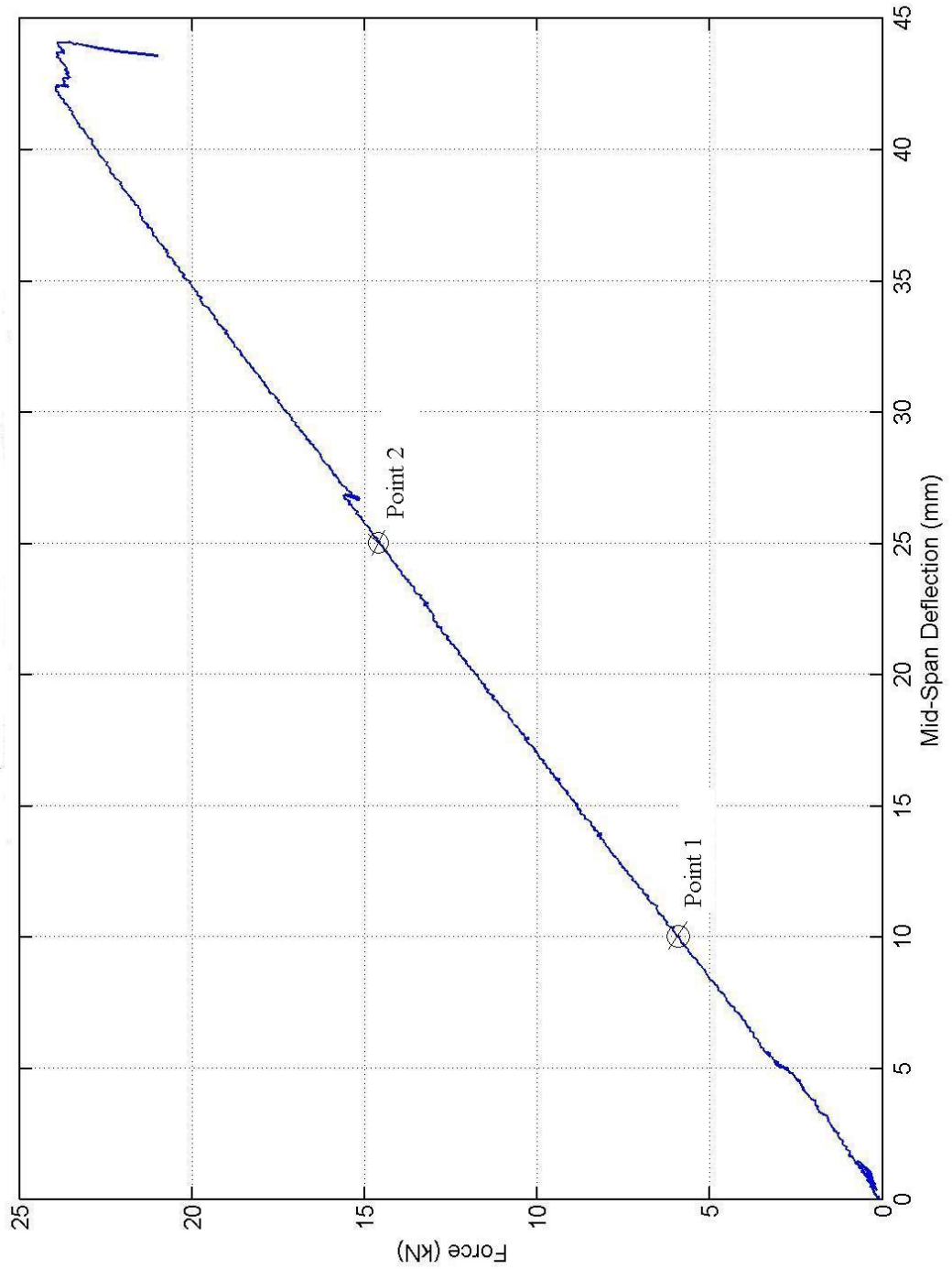
(American Forest and Paper Association, Inc., American Wood Council 2007, p.8)

possibly caused by load redistribution in the CLT panel. The effective modulus of elasticity was determined using equation 4.1, for points one and two indicated in 5.11, as follows...

$$\begin{aligned}
 E &= \frac{23}{108} \left( \frac{L}{d} \right)^3 \left( \frac{\Delta F}{\Delta e} \right) \frac{1}{b} \\
 &= \frac{23}{108} \left( \frac{2.0}{0.066} \right)^3 \left( \frac{(14.5 - 10) \times 10^3}{(25 - 6.5) \times 10^{-3}} \right) \frac{1}{0.27} \\
 &= 5.34 \text{ GPa}
 \end{aligned}$$

Therefore, the sample CLT effective modulus is much less than that of the timber elements from which it is composed. The timber elements tested by Hyne wee found to have a mean acoustic modulus of elasticity of 17 MPa.

Figure 5.11: Load VS Displacement Plot: Four Point Bending Test



### 5.3 AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms

Initially, it was thought that section five of AS1720.1, which deals with the design of plywood, would be the most likely formula to apply to the design of CLT panels, however, given the large panel area to thickness ratio of CLT wall panels, investigations moved toward using AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms. Appendix I takes into consideration the size of the diaphragm with a plywood assembly factor  $g_{61}$ .

According to AS1720.1, Appendix I - Buckling Strength of Plywood Diaphragms is applicable where large sheets of thin plywood are used in composite construction, and it is possible for buckling distortions to cause a reduction in the load capacity of the plywood membrane (Standards Australia 2010*b*, p.161). Typical examples are webs of I-beams and plywood box beams.

Section I2.3 of AS1720.1 Appendix I, refers to the calculation of buckling strengths of plywood 'diaphragms with lateral edges free and subjected to uniformly loaded edge forces' (Standards Australia 2010*b*, p.163).

In the case of a CLT wall panel, the load is typically considered uniformly distributed by evenly spaced joists and continuously from walls above. No consideration is given to concentrated edge loads, such as floor bearers and roof beams. Point load considerations are outside the scope of this project.

The buckling strength of plywood diaphragms is determined using the same method as that used for the design of timber compression members, with the exception being the calculation of the slenderness coefficient  $S$ . The slenderness coefficient is required for determination of the stability factor  $K_{12}$ . The reader is referred to section 2.7 of this document for background information on the design of timber compression members.

The buckling strength modification factor for diaphragms with lateral edges free and subjected to uniformly loaded edge forces is given by the following equation. . .

$$S = g_{61} \frac{d_w}{t_w} \quad (5.1)$$

Where  $g_{61}$  is a factor describing the layup of the plywood, in terms of the number of lamella, direction of grain, ratio of thickness of outer plies to that of the inner plies,  $d_w$  is the depth of the web, and  $t_w$  is the thickness of the web. For CLT wall sample panels, the depth of the web is actually the height of the sample which was 2.70 m, and the thickness of the web is the overall thickness of the CLT wall panel sample, which was .066 m.

The slenderness coefficient for the Hyne CLT sample panel, according to equation 5.1 is . . .

$$\begin{aligned} S &= g_{61} \frac{d_w}{t_w} \\ &= 2.2 \times \frac{2.7}{0.066} \\ &= 90 \end{aligned} \tag{5.2}$$

Where  $g_{61} = 2.2$  for a three ply CLT panel with equal lamella thickness, as per table I3 (Standards Australia 2010*b*, p.165)

The stability factor  $k_{12}$  is calculated from equation 3.3(11c) of AS1720 (Standards Australia 2010*b*, p.38) as follows. . .

$$\begin{aligned} k_{12} &= \frac{200}{(\rho_c S)^2} \\ &= \frac{200}{(1.0 \times 90)^2} \\ &= 2.47 \times 10^{-2} \end{aligned} \tag{5.3}$$

and putting this into equation 2.7 yields a design compressive strength of. . .

$$\begin{aligned}
 \frac{N_{d,c}}{\phi} &= k_1 k_4 k_6 k_{12} f'_c A_c \\
 &= 1.0 \times 1.0 \times 1.0 \times 0.025 \times 0.01782 \times 56.1 \times 10^6 \\
 &= 24.95 \text{ kN}
 \end{aligned} \tag{5.4}$$

It is interesting to note that in the calculation of the axial in-plane loading for plywood in accordance with Section Five of AS1720.1, the area used is the cross sectional area of plies with their grain parallel to the load, whereas for S1720.1 Appendix I - Buckling Strength of Plywood Diaphragms, the cross sectional area is that of the entire plywood cross section (Standards Australia 2010*b*, p.163).

Further, according to section I3.2 of Appendix I of AS1720.1, two second moments of area for plywood are required to describe plywood section properties. The first is used when determining the bending capacity, and the second for determining the bending rigidity.

Plies which run perpendicular to the direction of load are not considered when determining the bending capacity. When determining the bending rigidity of plywood, an additional amount of rigidity is included to account for the stiffness of plies which run perpendicular to the load direction. Non-inclusion of the additional second moment of area when calculating bending rigidity would lead to conservative design.

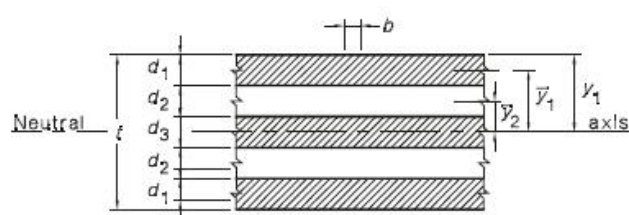


Figure 5.12: Plywood Cross Section and Notation(Standards Australia 2010*b*, p.167)

Though the section modulus for bending rigidity includes information about rigidity of plies running both parallel and perpendicular to the grain, neither the calculation of the compressive strength of plywood in section 5, nor buckling strength of plywood diaphragms in Appendix I, use the modulus of bending rigidity to determine the buckling capacity. The following equation is for the second moment of area for calculation of bending rigidity (Standards Australia 2010*b*,



p167), and should be read in conjunction with figure 5.12...

$$I_R = 2 \left( \frac{1}{12} b d_1^3 + A_1 \bar{y}_1^2 \right) + 2 \times 0.003 \left( \frac{1}{12} b d_2^3 + A_2 \bar{y}_2^2 \right) + \frac{1}{12} b d_3^3 \quad (5.5)$$

Where  $I_R$  = second moment of area about the neutral axis for bending rigidity calculations for a width  $b = 1$  mm, in  $\text{mm}^4$ .  $A_1 = d_1 b$ ,  $A_2 = d_2 b$  and 0.03 = factor applied for plies at right angles to the span.

Considering that the buckling capacity, especially for slender columns, is dominated by the stiffness of the section, and not the section capacity, the calculations for in-plane compressive strength as indicated in AS1170.2 are inconsistent with the requirements for determining the axial in-plane capacity of CLT wall panels of buildings, which are typically considered intermediate to slender.

The bending rigidity of the CLT sample is calculated according to equation 5.5, for comparison with the actual bending rigidity of the sample as follows...

$$\begin{aligned} I_R &= 2 \left( \frac{1}{12} b d_1^3 + A_1 \bar{y}_1^2 \right) + 0.003 \left( \frac{1}{12} b d_2^3 \right) \\ &= 2 \times \left( \frac{1}{12} \right) \times 270 \times 22^3 + 0.003 \times \left( \frac{1}{12} \right) \times 270 \times 22^3 \\ &= 6.23 \times 10^6 \text{mm}^4 \end{aligned}$$

whereas the actual effective second moment of area for the CLT test sample, as calculated from the results of the four point bending test using accepted deflection formula (American Forest and Paper Association, Inc., American Wood Council 2007, p.8), is as follows...

$$\begin{aligned}
 I &= \frac{Pa}{\Delta_{max} \times 24E} (3l^2 - 4a^2) \\
 &= \frac{20 \times 10^3 \times 0.67}{0.035 \times 24 \times 17 \times 10^9} \times (3 \times 2^2 - 4 \times 0.67^2) \\
 &= 9.575 \times 10^6 \text{ mm}^4
 \end{aligned}$$

The load of 20 kN and deflection of 35 mm were taken from the load deflection plot of the four point bending test in figure 5.11.

This result indicates that for a CLT wall panel, using the plywood composite theory to calculate section properties underestimates the contribution of plies with grain perpendicular to the load direction.

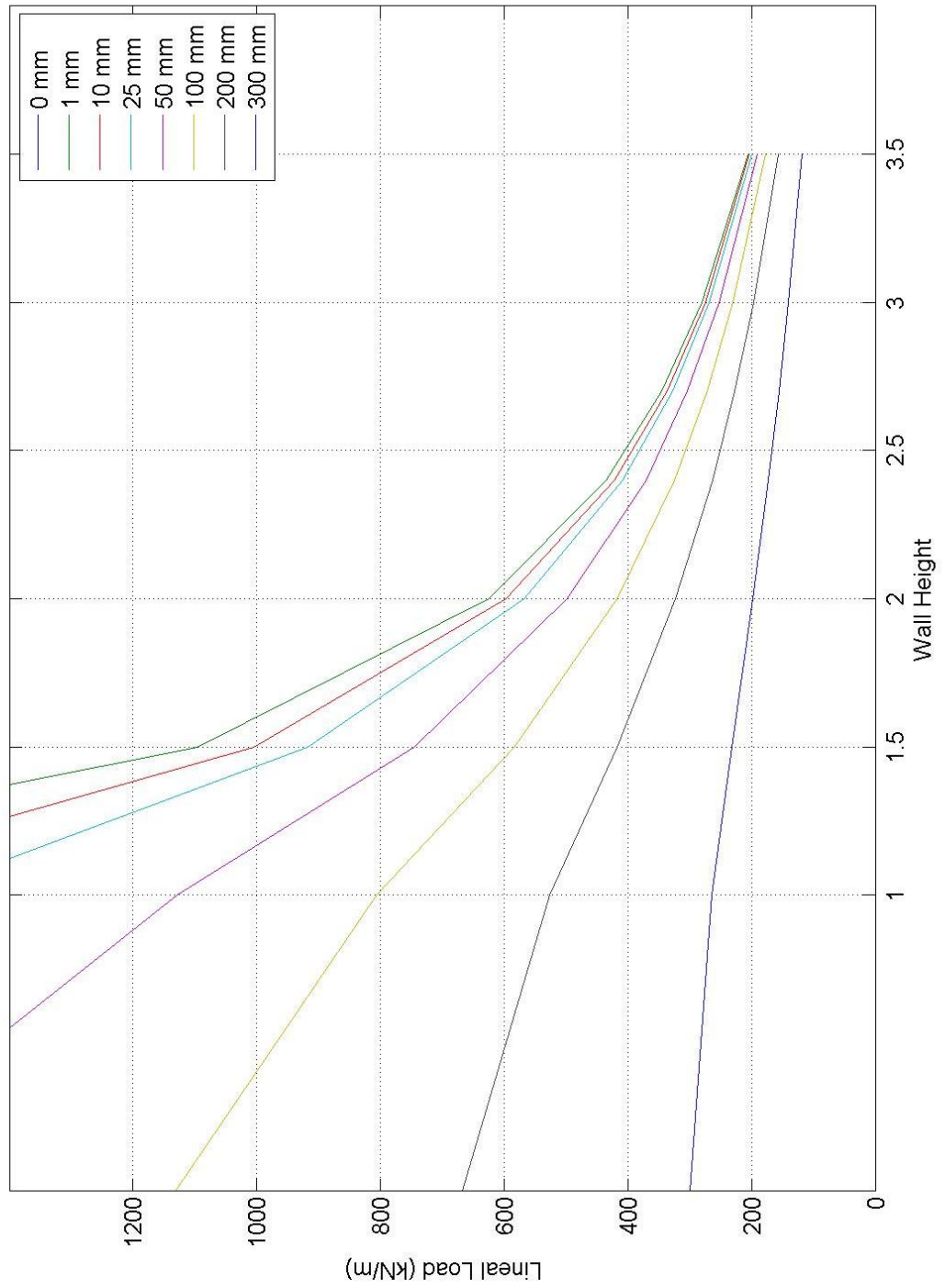
## 5.4 Development of Load vs Height Charts for the CLT Sample Panel

The secant formula (equation 2.7) was used to develop a theoretical load chart for the CLT sample layup (22/22/22), based on the effective modulus of elasticity of the CLT cross section as determined from the four point bending test, and the maximum compressive strength of the timber.

The load chart is shown in figure 5.13 is a theoretical load chart which assumes pin connection at the top and bottom of the wall. Various load eccentricities are indicated on the plot.

These loads are based on the mean modulus of elasticity of the CLT samples, and should not be used for design purposes. Fifth percentile characteristic strengths are significantly less. The matlab code, including variables used to calculate and plot the graph is included in Appendix 9.

Figure 5.13: Hyne CLT Theoretical Height Vs Lineal Load Plot for Various Eccentricities



## **Chapter 6**

# **Conclusions and Further Work**

### **6.1 Summary**

Investigation into serviceability aspects of CLT panels was undertaken, including preliminary calculations for required fire cover.

Finite element analysis (FEA) was used to specify a CLT panel layup to meet the requirements of the low rise multistorey residential market, as well as those of Hyne Pty Ltd. Samples of the proposed CLT panel were fabricated by Hyne and subsequently tested at the University of Southern Queensland.

Accepted formula from AS1720.1 for the buckling strength of plywood diaphragms were checked for application to CLT panels by comparing theoretical results with experimental results. Theoretical load charts were developed based on experimental results and accepted engineering theory.

### **6.2 Achievement of Project Objectives**

The following is a brief discussion on each of the project objectives outlined in the project specification...

*Research background information relating to the structural performance of cross laminated timber for wall applications. The research will include fire resistance, deflection and ultimate strength prediction of CLT panels.*

Research indicates that fire resistance of CLT panels is similar to that of normal timber, and charring factors can be calculated directly from AS1720.4-2006 Timber Structures Part 4: Fire resistance for structural adequacy of timber members. For external CLT walls, only one side of the panel is exposed to elevated temperatures.

Axial load testing of the Hyne CLT samples indicate that mid-height deflection at ultimate load is well inside the advised limits of AS1170.1 for walls.

*Use FEA to investigate structural properties of internal and external CLT wall panels for a number of CLT cross sections which Hyne can feasibly produce.*

Finite element linear static buckling analysis was undertaken using Strand7, as well as sensitivity analysis to ensure adequate mesh sizing. The analysis extended to multiple CLT cross sectional layups which could be feasibly fabricated by Hyne.

*Evaluate the results of the FEA modeling and select a CLT panel cross section for compression load testing.*

A criteria was developed for selection of a suitable CLT panel, which included available dimensional timber element sizes, the proposed market to which the CLT panel is aimed. A nominal load case was developed based on a low-rise multistory residential building representing the proposed market.

Evaluation of the results from FEA concluded that the smallest CLT panel of all the trial sections was, in terms of strength, more than adequate for the intended purpose.

Hyne then fabricated a total of six CLT sample panels to the selected single specification, for destructive testing at the University of Southern Queensland. Each CLT sample panel consisted of three orthogonally placed lamella, each 22 mm thick. The sample wall panel overall dimensions were 3.0 m tall, and 270 mm long.

*Undertake laboratory testing of the selected CLT panel cross section for buckling strength*

*when loaded in-plane. Compare theoretical results with actual load test results.*

The two laboratory tests were performed including an axial load test and four point bending test. A third test (shear test) was also undertaken so that, if time permitted, a theoretical analysis of the effects of hole openings in CLT panels could be investigated.

A comparison between theoretical results from finite element modeling and actual load test results revealed large disparities. These are believed to be related to the wall end connections.

The connections were modeled as pin end connections in the Finite Element software, which is loosely correct, as there is no rigid connection at the ends.

The test setup did not accurately represent the pin ends of the Finite Element model, and though not rigidly connected, the ends of the wall panels were restrained between the load plates, resulting in higher than expected load capacities.

*Compare theoretical and laboratory results with accepted formulae from AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms.*

To get a full understanding of how the accepted theory for plywood in-plane buckling load capacity reflects the material properties of CLT, a four point bending test was first undertaken.

An effective second moment of area for the experimental sample was then calculated for comparison with section properties calculated using accepted formulae from AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms.

The buckling strengths and section properties determined using Appendix I significantly underestimated the bending stiffness and axial load capacity of the sample CLT panel.

*Provide theoretical internal and external CLT wall load charts for various common wall heights and report on the effects of fire.*

A theoretical lineal wall load vs wall height chart has been developed using the secant formula, for a number of different material or load eccentricities.

*Undertake compression load testing of various common wall heights to verify results of theo-*

*retical analysis, as well as a theoretical analysis of the effects of hole openings in CLT wall panels*

Time was not permitting to undertake further work set out in the project specification.

### **6.3 Conclusion**

CLT wall panels are already being used overseas, to the point where standards for design and construction have been developed and implemented. The benefits of CLT as a building material have been proven by the many projects already completed overseas on a purely economic basis. Environmental benefits of CLT as compared to other construction materials are widely published, and as governments move towards carbon neutral development and use of sustainable resources, CLT construction will surely gain more popularity in Australia.

This project has identified that existing timber standards cannot be used directly for CLT design, as they do not accurately represent the strength of CLT panels. Though produced from low grade timber, the CLT sample panels have load capacities far in excess of what is typically required for residential low-rise buildings.

### **6.4 Recommendations and Further Work**

Further work is required to develop FEA models which more accurately represent the end connection details of the walls when they are installed in buildings. Many floor systems and connections details are possible for floors and walls, and so a number of models need to be developed.

Standards specific to the fabrication, testing and design of CLT building elements are required so that mainstream adoption of CLT panels as an option to existing building materials is possible. Connection details, shear capacity, vibration and transmission, and weather protection methods are all areas which need to be developed in relation to walls.

# References

- Aicher, S. & Dill-Langer, G. (2005), 'Effect of lamination anisotropy and lay-up in glued-laminated timbers.', *Journal of Structural Engineering* **131**(7), 1095 – 1103. viewed 19th of May 2010, EBSCOhost.
- American Forest and Paper Association, Inc., American Wood Council (2007), 'Design aid no.6, beam design formulas with shear and moment diagrams'. viewed 17th October 2010,  
<<http://www.awc.org/pdf/DA6-BeamFormulas.pdf>>.
- Australian Building Codes Board (2010), 'The building code of australia'. viewed 23th October 2010  
<<http://www.abcb.gov.au.ezproxy.usq.edu.au/abcbonline>>.
- Bejtka, I. & Lam, F. (2008), Cross laminated timber as innovative building material, Vol. 2, Quebec City, QC, Canada, pp. 1090 – 1099. viewed 15th May 2010, Compendex Engineering Village.
- B & K Cross Laminated Timber* (2010), website.  
<http://www.bkts.co.uk/cross-laminated-timber.html>.
- Blass, H., Fellmoser, P. & Ingenieurholzbau, L. (2004), Design of solid wood panels with cross layers, in 'Proceedings of the 8th World Conference on Timber Engineering, WCTE', pp. 14–17.
- Building with Timber - Nine Storeys and Beyond* (2010), Website.  
<http://www.timber.org.au>.



DUJIC, B., PhD, T., Ljubljana, S., KLOBCAR, S., Ltd, C., Celje, S. & ZARNIC, R. (n.d.), 'Shear Capacity of Cross-Laminated Wooden Walls'.

*Education Resources Structural Grading* (n.d.).

<http://www.timber.org.au/NTEP/menu.asp?id=89>.

FRANGI, A., BOCHICCHIO, G., CECCOTTI, A. & Lauriola, M. (2008), Natural Full-Scale Fire Test on a 3 Storey XLam Timber Building, *in* 'Proceedings of 10th World Conference on Timber Engineering (WCTE), Miyazaki, Japan', Vol. 2.

Gagnon, S., Hu, L. & Desjardins, R. (2008), Case study of 8 story buildings constructed in sweden, using massive timber systems, Vol. 3, p. 1415 to 1424. viewed 15th May 2010, Compendex Engineering Village.

Gonchar, J. (2010), 'Testing timbers limits', *Architectural Record* .

Gsell, D., Feltrin, G., Schubert, S., Steiger, R. & Motavalli, M. (2007), 'Cross-laminated timber plates: Evaluation and verification of homogenized elastic properties', *Journal of Structural Engineering* **133**(1), 132 – 138. viewed 15th May 2010, Compendex Engineering Village.

KLH Massivholz GmbH (2010), 'What is cross-laminated timber', manufacturers website.

Leicester, R. H. (2009), 'Buckling strength of timber structures', *Australian Journal of Structural Engineering* **9**(3), 249–256. viewed 20th May 2010, Compendex Engineering Village.

Mackerle, J. (2005), 'Finite element analyses in wood research: a bibliography', *Wood Science and Technology* **39**(7), 579–600. viewed 18th of May 2010, Google Scholar.

Mestek, P., Kreuzinger, H. & Winter, S. (n.d.), 'Design of Cross Laminated Timber (CLT)'.

Moody, R. & Hernandez, R. (1997), 'Glued-Laminated Timber', *Engineered Wood Products: A Guide for Specifiers, Designers and Users*, Smulski, S.(Ed.). PFS Research Foundation, Wisconsin, ISBN: 096567360X, pp pp. 1–39.

Museum of London (2010), 'The 1667 rebuilding act'. viewed 26th of August 2010,

<<http://www.museumoflondon.org.uk/English/>

[EventsExhibitions/Past/LondonsBurning/Themes/1405/1408/](http://www.museumoflondon.org.uk/English/EventsExhibitions/Past/LondonsBurning/Themes/1405/1408/)

[Page1.htm](http://www.museumoflondon.org.uk/English/EventsExhibitions/Past/LondonsBurning/Themes/1405/1408/Page1.htm)>.

- Park, H. & Fushitani, M. (2006), 'Effects of component ratio of the face and core laminae on static bending strength performance of three-ply cross-laminated wood panels made with sugi (*Cryptomeria japonica*)', *Wood and Fiber Science* **38**(2), 278–291. viewed 18th of May 2010, Google Scholar.
- Pryda Australia (2009), *Pryda Timber Connectors Hangars and Truss Connectors Guide*. viewed 18th October 2010,  
<<http://www.pryda.com.au/uploads/>>.
- Purbond (2010), 'Purbond hb s-line product overview', manufacturers website. viewed 19th October 2010  
<<http://www.purbond.com/purbond/Australia-New-Zealand>>.
- Rammer, D. (1996), Shear strength of glued-laminated timber beams and panels, in 'National Conference on Wood Transportation Structures: preliminary program and registration, October 23-25, 1996, Madison, Wisconsin', The Laboratory, p. 192. viewed 18th of May 2010  
<<http://www.woodcenter.org/docs/ramme96c.pdf>>.
- Robinson, B. (2010), 'London's burning: The great fire'. viewed 26th of August 2010,  
<[http://www.bbc.co.uk/history/british/civil\\_war\\_revolution/great\\_fire\\_01.shtml](http://www.bbc.co.uk/history/british/civil_war_revolution/great_fire_01.shtml)>.
- Standards Australia (2002), 'Structural design actions part 1: Permanent, imposed and other actions, as1170.1:2002'. viewed 20th October 2010  
<<http://www.saiglobal.com/online/autologin.asp>>.
- Standards Australia (2003), *HB2.2 Part 2. Structural Engineering*, Australian Standards for Civil Engineering Students, eighth edn, Standards Australia, Sydney.
- Standards Australia (2010a), 'Characterization of structural timber - test methods, as/nzs 4063.1'. viewed 17th October 2010  
<<http://www.saiglobal.com/online/autologin.asp>>.
- Standards Australia (2010b), 'Timber structures part 1: Design methods, as1720.1'. viewed 17th October 2010  
<<http://www.saiglobal.com/online/autologin.asp>>.

- Standards Australia (2010c), 'Timber structures part 4: Fire resistance for structural adequacy of timber members, as1720.4'. viewed 20th October 2010  
<<http://www.saiglobal.com/online/autologin.asp>>.
- Steiger, R. & Gulzow, A. (n.d.), Validity of bending tests on strip-shaped specimens to derive bending strength and stiffness properties of cross-laminated solid timber (x-lam), in 'The future of Quality Control for Wood and Wood Products'.
- Strand7 Pty Ltd (2010), 'Strand7 online help'. viewed 7th October 2010,  
<[http://www.strand7.com/html/docu\\_onlinehelp.htm](http://www.strand7.com/html/docu_onlinehelp.htm)>.
- Tall Timber Buildings The Stradhaus, Hoxton, London* (2010).  
<http://techniker.oi-dev.org>.
- Teibinger, M. (2008), 'Urban timber houses in vienna', *Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (IABSE)* **18**(2), 114 – 117. viewed 15th May 2010, Compendex Engineering Village.
- Thelandersson, S. & Larsen, H. (2003), *Timber Engineering*, John Wiley & Sons Ltd., West Sussex, England.
- Timoshenko, S.P. and Gere, J. (1991), *Mechanics of Materials*, third si edn, Chapman & Hall, London.
- U.S. Department of Agriculture, Forest Service, Forest Products Laboratory (2010), 'Wood handbook - wood as an engineering material centennial edition'. viewed 6th September 2010,  
<<http://www.fpl.fs.fed.us/products/publications>>.
- Vessby, J., Enquist, B., Petersson, H. & Alsmarker, T. (2009), 'Experimental study of cross-laminated timber wall panels', *European Journal of Wood and Wood Products* **67**(2), 211 – 218. viewed 2nd May 2010, Compendex Engineering Village.
- White, R. (2009), Fire resistance of exposed wood members, in 'Wood & fire safety: proceedings, 5th international scientific conference, April 18-22, 2004, Svolen, Slovakia', pp. 337–344. viewed 18th of May 2010  
<[http://www.fpl.fs.fed.us/documnts/pdf2004/fpl\\_2004\\_white001.pdf](http://www.fpl.fs.fed.us/documnts/pdf2004/fpl_2004_white001.pdf)>.

- Wilinder, P. (2009), Fire resistance in cross-laminated timber. Thesis, Tekniska Hgskolan, Sweden.
- Yu, W. & Hodges, D. H. (2008), 'Best strip-beam properties derivable from classical lamination theory', *AIAA Journal* **46**(7), 1719 – 1724.
- Zahn, J. J. & Rammer, D. R. (1995), 'Design of glued laminated timber columns.', *Journal of Structural Engineering* **121**(12), 1789. viewed 19th of May 2010, EBScohost item AN=6784142.

## **Chapter 7**

# **Project Specification**

## **Project Specification**

**For:** **Alan Turner**

**Topic:** STRUCTURAL PERFORMANCE OF CROSS LAMINATED TIMBER  
PANELS AS WALLS (INTERNAL AND EXTERNAL)

**Supervisors:** Associate Prof. Karu Karunasena  
Geoff Stringer, Hyne and Son Pty Ltd.

**Sponsorship:** HYNE & SON PTY. LIMITED

**Project Aim:** To analyse the structural performance of Cross Laminated Timber (CLT) panels manufactured from non-structural grade timber, for internal and external wall applications in terms of serviceability and strength.

**Program:**

1. Research background information relating to the structural performance of cross laminated timber for wall applications. The research will include fire resistance, deflection and ultimate strength prediction of CLT panels.
2. Use FEA analysis to investigate structural properties internal and external CLT wall panels for a number of CLT cross sections which Hyne can feasibly produce.
3. Evaluate the results of the FEA modeling and select a CLT panel cross section for compression load testing.
4. Undertake laboratory testing of the selected CLT panel cross section for buckling strength when loaded in-plane. Compare theoretical results with actual load test results.
5. Compare theoretical and laboratory results with accepted formulae from AS1720.1 Appendix I - Buckling Strength of Plywood Diaphragms.
6. Provide theoretical internal and external CLT wall load charts for various common wall heights and report on the effects of fire.
7. Submit an academic dissertation on the research

---

*As time and resources permit:*

1. Undertake compression load testing of various common wall heights to verify results of theoretical analysis
2. Undertake a theoretical analysis of the effects of hole openings in CLT wall panels

Agreed:

Student Name: Alan Turner

Date: 15/03/2010

Supervisor Name: A/Prof. Karu Karunasena

Date: 15/03/2010

Examiner/Co-Examiner: Approved

Date:

## **Chapter 8**

# **Moisture Content and Acoustic Modulus of Elasticity**



Figure 8.1: Moisture Content and Acoustic Modulus of Elasticity Data Received From Hyne

Board #	MoistCon (%)	Width (mm)	Thickness (mm)	Length (mm)	Mass (kg)	Frequency (Hz)	Density (kg/m <sup>3</sup> )	Acc E (GPa)
1	14.6	93	23	3024	4.637	855	717	19.170
2	12.3	93	23	3024	3.797	829	587	14.758
3	16.2	93	23	3019	4.510	879	698	19.672
4	13.9	93	23	3021	4.376	911	677	20.516
5	14.6	93	23	3025	4.475	831	692	17.481
6	17.3	93	23	3021	4.272	852	661	17.517
7	13.9	93	23	3024	4.033	814	623	15.111
8	13.7	93	23	3027	4.537	879	701	19.841
9	14.7	93	23	3008	4.407	856	685	18.163
10	15.4	93	23	3027	4.143	824	640	15.925
11	13.1	93	23	3023	3.819	788	591	13.407
12	10.9	93	23	3024	4.343	821	671	16.555
13	14.0	93	23	3024	3.682	784	569	12.796
14	15.2	93	23	3027	4.952	903	765	22.856
15	13.7	93	23	3022	3.178	809	492	11.754
<b>AVG</b>	<b>14.2</b>							
								<b>AVG</b>
								<b>17.035</b>
								<b>STDEV</b>
								<b>3.122</b>
								<b>COV</b>
								<b>0.183</b>

## Chapter 9

# MATLAB Source Code

### 9.1 Load Vs Displacement Experimental Data Plotting Programs

```
%function [ output_args ] = Untitled1( input_args )  
%This function plots the axial and mid-height  
%deformation of the sample under axial load  
  
clear;  
clc;  
  
%Load and post process raw data files  
load comp1.dat;  
xdispcomp1=comp1(:,4).*-1;  
ydispcomp1=comp1(:,3);  
loadcomp1=comp1(:,5)./1000;  
load comp2.dat;  
xdispcomp2=comp2(:,4);  
ydispcomp2=comp2(:,3);  
loadcomp2=comp2(:,5)./1000;  
load comp3.dat;  
xdispcomp3=comp3(:,4).*-1;  
ydispcomp3=comp3(:,3);  
loadcomp3=comp3(:,5)./1000;  
  
%Plotting - Axial Load VS Y-Axis Deflection  
figure('MenuBar','none');  
set(gcf, 'position', get(0, 'ScreenSize'));  
h=plot(ydispcomp1, loadcomp1, ydispcomp2, loadcomp2 ...  
        , ydispcomp3, loadcomp3);  
set(h, 'LineWidth', 1);  
set(gca, 'FontSize', 13, 'LineWidth', 1);  
xlabel('Y-Axis Displacement (mm)');  
ylabel('Load (kN)');  
%title('Axial Load VS Y-Axis Deflection', 'FontSize', 15);  
grid;  
set(gca, 'XTick', [0:1:10]);
```

```
label1='Sample_1';
label2='Sample_2';
label3='Sample_3';
legend(label1 ,label2 ,label3 );
orient landscape;
print ('-djpeg', 'axialplot1.jpg');
close;

%Plotting – Axial Load VS Mid–Height Deflection
figure ('MenuBar', 'none');
set (gcf, 'position', get(0, 'ScreenSize'));
h=plot(xdispcomp1 ,loadcomp1 ,xdispcomp2 ,loadcomp2 ,...
        xdispcomp3 ,loadcomp3 );
set (h, 'LineWidth', 1);
set (gca, 'FontSize', 13, 'LineWidth', 1);
xlabel ('Mid–Height_Displacement_(mm)');
ylabel ('Load_(kN)');
%title ('Axial Load VS Mid–Height Deflection', 'FontSize', 15);
grid;
set (gca, 'XTick', [0:5:70]);
label1='Sample_1';
label2='Sample_2';
label3='Sample_3';
legend(label1 ,label2 ,label3 );
orient landscape;
print ('-djpeg', 'axialplot2.jpg');
close;
%EOF
```

```
%function [ output_args ] = Untitled1( input_args )
%This function plots the mid-span deflection vs
%applied load for the flatwise bending test

clear;
clc;
load bend1.dat;
xb1=bend1(:,3).*-1;
yb1=bend1(:,4).*1e-3;

%Plotting - Mid-Span Deflection VS Applied Load
figure('MenuBar','none');
set(gcf, 'position', get(0, 'ScreenSize'));
h=plot(xb1,yb1);
set(h, 'LineWidth', 1);
set(gca, 'FontSize', 13, 'LineWidth', 1);
ylabel('Force (kN)');
xlabel('Mid-Span Deflection (mm)');
title('Load Displacement Graph for CLT Flatwise ...
      Bending Test', 'FontSize', 15);
grid;
set(gca, 'YTick', [0:5:25]);
orient landscape;
print('-djpeg', 'bending.jpg');
close;
%EOF
```

## 9.2 Secant Formula Buckling Linear Load Calculation Program

```

%function [ output_args ] = Untitled1( input_args )
%This function plots linear elastic column behavior
%using the Secant Formula the resultant plot can be
%used to assess the axial capacity of the CLT panel
%within the elastic region. The capacity is based on
%the maximum fibre stress when subjected to both axial
%and moment loading caused by eccentricities of the
%load application as well as and those caused
%by imperfections.

clear ;
clc ;

%External Lamella Thickness t1(mm)
t1=22;
%Central Lamella Thickness t2(mm)
t2=22;
%Wall length (m)
WLength=1.0;
%Compressive strength of outer fibre (Pa)
L1Fc=56.1e6;
%Effective Modulus of Elasticity Determined ...
%from Testing (Pa)
E=5.34e9;

%Calculate Section Properties
Width=((2*t1)+ t2)/1000;
Area=WLength*Width;
Height=[0,1.0,1.5,2.0,2.4,2.7,3.0,3.5];
I= 2*(((WLength*(Width)^3))/12)
RGyration=sqrt(I/Area);
SRatio=Height ./ RGyration
Centroid=Width/2
% Loading
LEccentricity=[0,0.001,0.005,0.01,0.025,0.05,0.1,0.25];
ERatio=(LEccentricity.*Centroid)./(RGyration.^2);
i=0;
j=0;
k=0;
for j=1:length(SRatio)
    k=0
    for k=1:length(ERatio)
        x=0
        while x<L1Fc
            i=i+1e4
            x=i*(1+(ERatio(k)*sec((0.5*SRatio(j))...
                *(sqrt(i/E)))));
        end
        Axial(k,j)=i
        i=0;
    end
end
end

%Plotting – Secant Formula Graph
Axial=Axial'
AxialMPa=Axial.*1e-6
figure('MenuBar','none');
set(gcf,'position',get(0,'ScreenSize'));

```

```

plot (SRatio , AxialMPa);
set (gca, 'FontSize', 13);
xlabel('Slenderness_Ratio_L/R');
ylabel('Axial_Stress_P/A_(MPa)');
%title('Secant Formula Graph For CLT Laminate
%(Timber Fc=56.1 MPa)for Various Load Eccentricities '
%, 'FontSize', 15);
grid;
set(gca, 'XTick', [0:10:140]);
set(gca, 'YTick', [0:5:22]);
axis([0 140 0 22]);
label1='0_mm';
label2='1_mm';
label3='10_mm';
label4='25_mm';
label5='50_mm';
label6='100_mm';
label7='200_mm';
label8='300_mm';
legend(label1, label2, label3, label4, label5, label6, ...
label7, label8);
pause;
orient landscape;
print('-djpeg', 'eccentric.jpg');
close;

%Plotting - Wall Height VS Axial Load
LinealLoad=( Axial.* Area)
LinealLoadkN=LinealLoad.*1e-3
figure('MenuBar', 'none');
set(gcf, 'position', get(0, 'ScreenSize'));
plot (Height, LinealLoadkN);
set (gca, 'FontSize', 13);
xlabel('Wall_Height');
ylabel('Lineal_Load_(kN/m)');
%title('Height VS Lineal Load of CLT Panel (Max.
%Timber Fc=56.1Mpa) For Various Load Eccentricities '
%, 'FontSize', 15);
grid;
set(gca, 'XTick', [1:0.5:3.5]);
set(gca, 'YTick', [0:200:1300]);
axis([0 4 0 1400]);
label1='0_mm';
label2='1_mm';
label3='10_mm';
label4='25_mm';
label5='50_mm';
label6='100_mm';
label7='200_mm';
label8='300_mm';
legend(label1, label2, label3, label4, label5, label6, ...
label7, label8);
pause;
orient landscape;
print('-djpeg', 'loadheight.jpg');
close;
%EOF

```

## **Chapter 10**

# **Purbond Adhesive Datasheet**



## \*\*\* SAFETY DATA SHEET \*\*\*

### 1. IDENTIFICATION OF THE SUBSTANCE/PREPARATION AND OF THE COMPANY/UNDERTAKING

---

PRODUCT NAME	PURBOND® HB S109
RECOMMENDED USE	Adhesive
SUPPLIER	Purbond AG CH-6203 Sempach-Station Switzerland
	EMERGENCY TELEPHONE: Tel. +41-(0)41-469-6863 (business hours)
	info@purbond.com

### 2. HAZARDS IDENTIFICATION

---

EMERGENCY OVERVIEW	SENSITIZER IRRITANT HARMFUL LIMITED EVIDENCE OF A CARCINOGENIC EFFECT
EYE	Will cause eye irritation.
SKIN CONTACT	Repeated and/or prolonged contact may cause irritation and skin sensitisation.
INHALATION	Vapors and/or aerosols may cause irritation. May cause allergic respiratory reaction. Danger of serious damage to health by prolonged exposure through inhalation.
INGESTION	Low oral toxicity.
PHYSICO-CHEMICAL	No unusual hazards are expected.
ENVIRONMENTAL	Not readily biodegradable.



---

### 3. COMPOSITION/INFORMATION ON INGREDIENTS

---

CHEMICAL FAMILY	Isocyanate Adhesive		
<u>COMPONENT</u>	<u>EC NUMBER</u>	<u>CONCENTRATION %</u>	<u>EC SYMBOL/ RISK PHRASES</u>
Diphenylmethanediisocyanate, isomers and homologues		10 - 20	XN XI R20 R36/37/38 R40 R42/43 R48/20
Diphenylmethane-4,4'- diisocyanate	247-714-0	5 - 10	XN XI R20 R36/37/38 R40 R42/43 R48/20

For full text of risk phrases, please refer to Section 16.

---

### 4. FIRST-AID MEASURES

---

EYE	Immediate medical attention is not required. Irrigate with eyewash solution or clean water until pain is relieved.
SKIN CONTACT	Immediate medical attention is not required. Wash skin with soap and water. Remove grossly contaminated clothing, including shoes, and launder before re-use. Discard shoes.
INHALATION	Immediate medical attention is not required. Remove to fresh air. If breathing is difficult, give oxygen. If breathing has stopped, give artificial respiration. Get medical attention.
INGESTION	Immediate medical attention is not required. Treat symptomatically and supportively.

---

### 5. FIRE-FIGHTING MEASURES

---

EXTINGUISHING MEDIA	CO <sub>2</sub> ; Dry Chemical; Foam
SPECIAL FIREFIGHTING PROCEDURES	Fire fighters should be equipped with self-contained breathing apparatus to protect against potentially toxic and irritating fumes.; Cool exposed equipment with water spray.
FIRE & EXPLOSION HAZARDS	Combustion will evolve toxic and irritant vapours.
HAZARDOUS COMBUSTION PRODUCTS	Decomposes upon heating to release toxic fumes of nitrogen oxides, carbon monoxide, carbon dioxide, and hydrogen cyanide.
LOWER EXPLOSION LIMIT (%)	Not applicable
UPPER EXPLOSION LIMIT (%)	Not applicable
AUTOFLAMMABILITY	480 °C
FLASH POINT	> 200 C (Pensky-Martens Closed Tester)

---

### 6. ACCIDENTAL RELEASE MEASURES

---

SPILL AND LEAK PROCEDURES	Adsorb spillages onto sand, earth or any suitable adsorbent material. Sweep up and shovel into waste drums.
---------------------------	---

For safety and environmental precautions, please review entire Safety Data Sheet for necessary information.

---

### 7. HANDLING AND STORAGE

---

STORAGE TEMPERATURE	Ambient.
HANDLING/STORAGE	Store at room temperature. Vapours can accumulate at potentially hazardous levels in the unvented headspace of drums or bulk storage vessels. Open drums in ventilated area. Avoid breathing vapours or mists.
SENSITIVE TO STATIC ELECTRICITY	No
SPECIAL SENSITIVITY	Keep away from moisture.

---

**8. EXPOSURE CONTROLS/PERSONAL PROTECTION**


---

COMPONENT	EXPOSURE LIMITS
Diphenylmethanediisocyanate, isomers and homologues	0.02 mg/m <sup>3</sup> TWA 0.07 mg/m <sup>3</sup> STEL
Diphenylmethane-4,4'-diisocyanate	0.02 mg/m <sup>3</sup> TWA 0.07 mg/m <sup>3</sup> STEL

**OCCUPATIONAL EXPOSURE CONTROLS**

VENTILATION REQUIREMENTS	Provide adequate ventilation to ensure that the defined occupational exposure limit is not exceeded.
EYE PROTECTION REQUIREMENTS	Safety glasses, goggles or face shield to protect against splashing. Personal eye protection should conform to EN 166.
GLOVE REQUIREMENTS	Gloves are recommended due to possible irritation. Gloves should conform to EN 374.
CLOTHING REQUIREMENTS	Appropriate protective clothing and equipment is recommended to minimize skin contact with this substance.
WASH REQUIREMENTS	Wash before eating, drinking, or using toilet facilities.
RESPIRATORY REQUIREMENTS	Respiratory protection required if the exposure level is unknown or has been measured and found to exceed the published exposure limits. Self-contained breathing apparatus with a full facepiece operated in pressure-demand or other positive pressure mode.

---

**9. PHYSICAL AND CHEMICAL PROPERTIES**


---

PURE SUBSTANCE OR PREPARATION	Preparation.
PHYSICAL FORM	Liquid
COLOUR	tan
ODOUR	Negligible
ODOUR THRESHOLD	Not available
OXIDIZING PROPERTIES	Not applicable
SOLUBILITY IN WATER	Insoluble
PARTITION COEFFICIENT (n-octanol/water)	Not applicable
VISCOSITY	20000 mPa.s
RELATIVE DENSITY	1.1
EVAPORATION RATE	Not applicable
VAPOUR PRESSURE (mmHg)	Not applicable
VAPOUR DENSITY (air=1)	Not applicable
VOLATILES	< 0.1 %
VOLATILE ORGANIC COMPOUNDS	< 1 g/litre
AUTOFLAMMABILITY	480 °C
FLASH POINT	> 200 C (Pensky-Martens Closed Tester)

---

**10. STABILITY AND REACTIVITY**


---

STABILITY	Stable
CONDITIONS TO AVOID	Avoid moisture contamination.
HAZARDOUS DECOMPOSITION PRODUCTS	Decomposes upon heating to release toxic fumes of nitrogen oxides, carbon monoxide, carbon dioxide, and hydrogen cyanide.

---

**11. TOXICOLOGICAL INFORMATION**


---

EYE	Will cause eye irritation.
SKIN	Repeated and/or prolonged contact may cause irritation and skin sensitisation.
INHALATION	Vapors and/or aerosols may cause irritation. May cause allergic respiratory reaction.
INGESTION	Low oral toxicity.
EFFECTS OF CHRONIC EXPOSURE	Although this product has not been tested for chronic effects it is judged as having a low order of toxicity based on component information. Use of good industrial hygiene practices is recommended.
TARGET ORGANS	Respiratory system, Skin
RESPIRATORY SENSITIZATION	Sensitizer.
SKIN SENSITIZATION	Sensitizer.
PRODUCT INFORMATION	Not established.

---

**12. ECOLOGICAL INFORMATION**


---

POTENTIAL EFFECT ON ENVIRONMENT	Not readily biodegradable.
AQUATIC TOXICITY	None Established
POTENTIAL TO BIOACCUMULATE	The product has low potential for bioaccumulation.

---

**13. DISPOSAL CONSIDERATIONS**


---

WASTE DISPOSAL METHODS	Waste disposal should be in accordance with existing Community, National and local regulations.
EMPTY CONTAINER WARNINGS	Empty containers may contain product residue; follow SDS and label warnings even after they have been emptied.


---

**14. TRANSPORT INFORMATION (See also section 9)**


---

IATA CLASSIFICATION	Not classified as dangerous.
IMDG CLASSIFICATION	Not classified as dangerous.
ADR/RID	Not classified as dangerous.

**15. REGULATORY INFORMATION**

EC INDICATION OF DANGER EC SYMBOL	HARMFUL 
RISK PHRASES	R20 - Harmful by inhalation. R36/37/38 - Irritating to eyes, respiratory system and skin. R40 - Limited evidence of a carcinogenic effect. R42/43 - May cause sensitization by inhalation and skin contact. R48/20 - Harmful: danger of serious damage to health by prolonged exposure through inhalation.
SAFETY PHRASES	S23 - Do not breathe gas/fumes/vapour/spray. S24/25 - Avoid contact with skin and eyes. S37 - Wear suitable gloves. S45 - In case of accident or if you feel unwell, seek medical advice immediately (show the label where possible).
CONTAINS	Diphenylmethanediisocyanate, isomers and homologues Diphenylmethane-4,4'-diisocyanate
SPECIAL PHRASES	Contains isocyanates. See information supplied by the manufacturer.
EINECS	All components of this product are listed in EINECS or ELINCS.

**16. OTHER INFORMATION**

REVISION	19/07/2010
REPLACES VERSION DATED	26/06/2009
ORIGINAL DOCUMENT DATED	05/03/2007
CHANGES SINCE PREVIOUS ISSUE	Section 1 Section 2 Section 3 Section 15 Section 16
FOR REGULATORY INFORMATION, CONTACT:	Joseph Gabriel Technical Director Purbond AG CH-6203 Sempach-Station Switzerland Tel. +41-(0)41-469-6863 Mail joseph.gabriel@purbond.com

**FULL TEXT OF RISK PHRASES FOR INGREDIENTS INDICATED IN SECTION 3:**

Diphenylmethanediisocyanate, isomers and homologues	R20 - Harmful by inhalation. R36/37/38 - Irritating to eyes, respiratory system and skin. R40 - Limited evidence of a carcinogenic effect. R42/43 - May cause sensitization by inhalation and skin contact. R48/20 - Harmful: danger of serious damage to health by prolonged exposure through inhalation.
Diphenylmethane-4,4'-diisocyanate	R20 - Harmful by inhalation. R36/37/38 - Irritating to eyes, respiratory system and skin. R40 - Limited evidence of a carcinogenic effect. R42/43 - May cause sensitization by inhalation and skin contact. R48/20 - Harmful: danger of serious damage to health by prolonged exposure through inhalation.

This information is provided for health and safety assessment by an industrial user. Reference should be made to any relevant local or national health, safety, and environmental legislation. This information does not constitute indication of suitability for specific uses.