DEPARTMENT OF ARCHITECTURE

A detailed investigation into the engineering properties and challenges affecting the potential introduction of a UK grown Dowel-laminated timber floor panel into the domestic construction market.

A thesis submitted in partial fulfilment of the requirements for the Degree of Doctor of Philosophy

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This thesis is the result of the author's original research. It has been composed by the author and has not been previously submitted for examination which has led to the award of a degree.

Signed:

Date:

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A Area of a cross section

a Shear span

a_i Distance from the centroid of the section being considered and the neutral axis of the entire section

a_m Height of a layer

a_1 Fastener Spacing parallel to the grain

a_2 Fastener Spacing perpendicular to the grain

$a_{3,t}$ Loaded end distance

$a_{3,c}$ Unloaded end distance

$a_{4,t}$ Loaded edge distance

$a_{4,c}$ Unloaded edge distance

b Width of timber

d Diameter

E_0 Modulus of Elasticity parallel to the grain

E_{90} Modulus of Elasticity perpendicular to the grain

E_d Dynamic Modulus of Elasticity

E_{static} Static Modulus of Elasticity

$E_{m,app}$ Apparent Modulus of Elasticity

$E_{m,g}$ Global modulus of elasticity

$E_{m,l}$ Local modulus of elasticity

F_{max} Maximum recorded load resistance

$F_{members}$ Member stiffness

F_{peg} Dowel flexibility

$F_{v,Rk}$ Characteristic load carrying capacity of fastener per shear plane per fastener

F_{vy} Yield shear stress

$F_{90,Rk}$ Characteristic splitting capacity

$f_{h,i,k}$ Characteristic embedment strength of the timber member

f_i Frequency at *ith* harmonic

$f_{m,k}$ Characteristic bending strength of timber

$f_{m,y,d}$ & $f_{m,z,d}$ Design bending strength about the two principal axes.

$f_{u,k}$ Characteristic tensile strength

G	Shear modulus
h	Member height
h_e	Loaded edge distance to the centre of the most distance fastener
h_i	Height of each layer
I_i	Second moment of area of a section
i	Harmonic number
K_i	Foundation modulus
K_{joint}	Joint stiffness
k_h	Size effect factor
k_i	Slip between the fasteners
k_m	Modification factor for redistribution of stress and the variation of the material
k_{mod}	Modification factor for load duration and moisture content
k_{ser}	Instantaneous slip modulus
k_{sys}	Modification factor for load distribution between closely spaced members
L	Span length
M	Moment
$M_{y,Rk}$	Characteristic fastener yield moment
M_y	Yield moment of dowel
P	Applied force
P_p	Plastic load resistance
R_d	NDS Reduction Factor
s_i	Spacing of the fasteners
t_i	Material thickness
T	Central member thickness
V	Stress wave velocity
w	Modification factor for connection type
w_c	Pre-camber
w_{creep}	Creep deflection
w_{fin}	Final deflection
w_{inst}	Instantaneous deflection
$w_{net,fin}$	Net final deflection
Z	Section modulus
z_i	Distance from the neutral axis to the centroid of a section

α	Angle of load to the grain
β	Ratio between embedment strength of timber members within a connection
κ_i	Initial foundation modulus
δ	Displacement
γ_i	Fastener efficiency factor.
γ_m	Partial factor for variation in material
$\sigma_{m,y,d}$	Bending stress about y-axis
$\sigma_{m,z,d}$	Bending stress about z-axis
ρ	Density
ρ_k	Characteristic density
ρ_m	Mean density
θ	Rotation at plastic hinge

List of Abbreviations

CLT	Cross Laminated Timber
COCIS	Centre for Offsite Construction and Innovative Structures
CSIC	Construction Scotland Innovation Centre
DLT	Dowel Laminated Timber
EWPs	Engineered Wood Products
EYM	European Yield Model
FC	Forestry Commission
Glulam	Glue-Laminated Timber
LVL	Laminated Veneer Lumber
MC	Moisture Content
MMC	Modern Methods of Construction
NA	National Annex
NDS	North American Design Specification
NFI	National Forest Inventory
OSB	Orientated Strand Board
SIRT	Strategic Integrated Research in Timber
SLS	Serviceability Limit State
SMEs	Small Medium Enterprises
T1	Tolerance Class 1 for sawn timber
T2	Tolerance Class 2 for sawn timber
ULS	Ultimate Limit State

Abstract

Satisfying the demand for more efficient and sustainable buildings presents a considerable challenge to the UK construction industry. Increasingly, the UK construction industry is looking towards prefabricated or offsite construction as a means of providing economic, sustainable and energy efficient buildings. Timber and engineered wood products (EWPs) are ideally suited to take advantage of the trend towards off site construction. However, in the UK, planted commercial tree species, are only used in a limited fashion by the UK construction industry as it is generally perceived to be of low quality. Forestry projections indicate that within the next 10 to 15 years there will be an abundance of UK timber as large amounts of standing stock reach maturity. Subsequently, there is a need to fully utilise the available and future resource, lest large amounts of commercial timber and capital will be underutilised. This has led to more research being conducted into new or novel methods of maximising the potential of the existing and predicted UK timber stock.

Whilst the production of a variety of EWPs have been embraced on the continent by small to medium enterprises working in the timber industry similar developments in the UK have been less forthcoming. One such product that has been developed on the continent is Brettstapel, Dowellam or Dowel laminated timber (DLT). These are solid timber panels that are formed without the use of adhesive and rely on hardwood dowels to join the single laminations together to form a panel. The creation of an engineered timber product without synthetic adhesives allows for a real reduction in the energy used to create a higher value EWP that can be produced in a variety of shapes and finishes.

Building upon the research of the Centre for Offsite Construction and Innovative Solutions (COCIS), this thesis explores the development of a DLT panel formed predominately of UK grown timber. A ground up approach to developing a DLT floor panel was undertaken; this included identifying market opportunities, locating avenues for its application and determining barriers to its implementation. One of these barriers highlighted within a market study was the lack of technical information and guidance.

The provision of technical information for the panel incorporated a staged analysis of materials and structural mechanisms within a panel that defined its structural performance. These included the determination of the pertinent mechanical properties of the softwood laminations and hardwood dowels including combined timber embedment properties to determine the most appropriate materials to be used in a panel. An experimental investigation into the use of hardwood beech dowels in conjunction with UK Sitka spruce and larch members in double and multiple shear plane connections is also undertaken. The findings for the study are integrated into a process for the design and verification for UK DLT panels. Through a series of experimental tests conducted on UK grown and produced DLT panels the thesis demonstrates that UK grown timber could be used to create DLT panels in the future. However, the position of the dowel and the overall stiffness of the all-timber connection cannot ensure a composite panel with improved performance is created.

Finally, the thesis demonstrates the potential for the UK production of DLT panels by presenting a case study for a new build property in the Scottish Highlands that utilises UK softwood and information obtained in this study.

1. Introduction

The United Kingdom is currently in the midst of a severe housing crisis. Increased demand for housing over the past 25 years in certain areas of the UK has far outstripped the supply, causing the price of housing to increase far beyond sustainable levels. Estimates based on projections of changing household occupancy levels and increasing population, suggest that, in the UK alone the number of additional households required by 2033 is approximately 6.8 million (DCLG, 2013; GRS, 2013; SFW, 2013; NISRA, 2013). The UK government alongside leading industry experts believe to meet this demand that at least a minimum of between 232,000 and 300,000 new housing units will need to be completed per year by 2016 and every subsequent year thereafter until 2033 (DCLG, 2010; RIBA, 2012). Latest figures produced by the Office of National Statistics (ONS, 2016) indicate that the number of new dwellings completed in 2012-13 was 133,010. Although in 2013-2014 there was a minimal increase in construction to 138,380 dwellings completed, these figures represent less than 60% of the UK's required yearly house production to meet the current housing shortage (RIBA, 2012). This disparity between supply and demand of housing stock has only been exacerbated in recent years by a worldwide recession causing the UK construction industry to stall significantly. The widening gap between the supply and demand of new houses has prompted the UK government to highlight the building sector as a potential leading actor in combating this crisis (Iddon and Firth, 2013).

Following the Latham (1994) and Egan (1998) reports in the mid to late 1990s greater emphasis has been placed on innovation in the construction industry and more recently the Construction 2025 strategy (DBIS, 2013a) has sought to expand and integrate the findings of the previous reports with an increased focus on green products and trade deficits. In combination with tackling the UK housing crisis, the UK government also has reiterated its desire to tackle the global issue of climate change at the 2015 United Nations Climate Conference of Parties (BBC, 2015). The UK government has indicated (DCLG, 2015) that there are two broad market opportunities for the transformation of the building sector into a low carbon-built environment, these can be outlined as:

- Reducing the energy and carbon emissions in new and existing buildings, through energy efficient, low carbon design and specifications.
- Exploiting innovation in sustainable building technologies in both domestic and global markets.

The largest single contributor to climate change is widely considered to be the increased level of Carbon Dioxide (CO_2) in the atmosphere and the greatest contributor to these CO_2 levels is the emissions created from the whole life cycle of contemporary buildings (Meggers *et al.,* 2012). To this end, the UK government has highlighted the requirements for new housing throughout the UK as a massive opportunity to achieve some of the mandatory reductions in CO_2 emissions (by 2050) stipulated by the Climate Change Act 2008 (Catto, 2008; Davies and Osmani, 2011; Iddon and Firth, 2013) through the use of more sustainable and efficient forms of construction.

A considerable portion of the emissions of a building product can lie within their 'embodied energy' prior to their arrival on site and their final operation within in a building (Coates, Gaterall & McManus, 2010; McManus, Gaterall & Coates, 2010; Monahan and Powell, 2011; Cuéllar-Franca and Azapagic, 2012). Embodied energy can simply be defined as the pre-burdened energy rating obtained from the extraction, manufacture, production and transportation of a product or material (Monahan and Powell, 2011). The embodied energy of a material has become more important and pivotal with the emergence of low energy houses and consideration of this burden should be integrated into preliminary design strategies to reduce overall energy demand (Guzowski, 2010; Iddon and Firth, 2013). In many instances, structural materials can represent more than 50% of the embodied energy in a building (Bribáin, Capilla & Usón, 2009) and normally low processed products have a lower embodied energy rating when comparatively assessed against traditional industrially produced building products, especially when they can be recycled (Cuéllar-Franca and Azapagic, 2012).

The issues for or against a products' successful potential adoption into the open market incorporate ideological, cultural, social, political, ethical, economic and technical aspects. Many of these considerations are inextricably linked to the requirement for the provision of more housing and the development of zero or low carbon homes in UK. At present, no regulations exist to regulate the embodied energy during the construction of a building (Iddon, and Firth, 2013), but there is an underlying strategic directionality towards sustainable construction as evidenced by recent government reports and directives such as Construction 2025 (DBIS, 2013a). The increased onus on the environmental responsibility for the sourcing of construction materials and overall reduction in waste processes (DBIS, 2013b) has reignited interest in timber building products within the UK. Timber has several environmental advantages over its industrially produced counterparts as it can normally be found locally and it can also provide holistic benefits such as low embodied energy, an ability to sequester carbon and can be recycled.

1.1. UK timber utilisation and production

In the UK, like in many parts of the world, the use of timber in construction has a large historic precedence and its use in construction is well understood (Herzog *et al.,* 2004). In one form, or another, timber construction was the principal method for constructing buildings in the UK up until the end of the 17th century (Pryce, 2005). In the 18th and 19th centuries, stone and brick became the materials of choice in the UK and then in the 20th century this preference evolved to incorporate steel and reinforced concrete. The use of timber in the UK has as a consequence, been increasingly marginalised by often cheaper industrial produced products and shifting views relating to the durability, stability and fire resistance of timber (Gold and Rubik, 2009).

Nowadays, the primary use of timber as a structural material within the UK built environment is in low cost timber frame housing within the domestic market. Within Scotland the use of timber framed housing for new build domestic dwellings has risen to approximately an 80% market share during the last decade (Tykkä *et al.,* 2010). Through the use of modern methods of construction (MMC) such as prefabrication of panelised systems, lean construction and increasingly sophisticated manufacturing processes, timber engineering can provide a realistic and viable solution to the energy and CO_2 emission burden that the present methodology of building construction throughout the UK currently uses (Monahan and Powell, 2011).

Unfortunately, the majority of structural timber currently being used within the UK construction industry is not supplied from locally grown materials. Approximately 62% of coniferous and non-coniferous sawn wood is imported from Scandinavia, the Baltic States or North America (FC, 2015) due to the accessibility, quality and cost of the timber materials being produced and imported. As demand for housing is increasing, the current export/import imbalance represents a missed opportunity for the sustainable economic growth of the UK timber industry and its associated industries and supply chain.

According to the volume figures produced by the Timber Trade Federation (2013) the most highly consumed wood-based product within the United Kingdom is sawn softwood accounting for 57% of all timber used in the UK, of which approximately 42 % is obtained from home grown sources. The last decade has seen a 21% reduction in the amount of primary wood processing plants within the UK whilst there has been an increase in imported softwood to a value of £1,084 million in 2012, making the UK the third largest net importer of forest products worldwide (FC, 2013; FC, 2015).

Recent figures suggest that the economic benefits of forest products are between £8.2 – 8.5billion (FC, 2015; Timber Trade Federation, 2013). Despite this, the industry is facing challenges in delivering the full potential of its standing resource. The demand for UK based softwood timber construction materials has remained fairly stagnant recently at approximately 7,000,000 green tonnes per annum for the five years up to 2013 (FC, 2013). At present, any increase in demand for the supply of home-grown timber resources has been shown to be predominant from the increased demand for wood fuel (FC, 2015) and not from an increased demand for wood products used in the construction industry. In March 2011, there was almost 46,000m^3 of overdue standing timber waiting to be felled (FC, 2012) and this figure could be compounded if current estimates of available softwood over the next 25 years are accurate. Present yield rates suggest an increase in the standing volume of 23% is expected (excluding overdue stocks) over the next 25 years (FC, 2012; FC, 2014a) leaving supply to peak while large amounts of sustainably harvestable resource potentially lie unused and awaiting felling.

The robust economic argument for the utilisation of the available forest resource is further strengthened through sustainability and the ideals of resource security and efficiency as well as social development (FC, 2003; Willis *et al*., 2003; FC 2015). Uncertainty created from the impending exit from for the EU threatens the current supply of natural resources into the UK. The security of the natural resources is always a priority of a sitting government, early evidence of this position can be seen by the UK's proposed withdrawal from international fishing arrangements (BBC, 2017). Whilst timber is a vital resource it is often overlooked as a largely imported material and it would be prudent to ensure that the domestic source of supply can provide the necessary security in the future.

The timber produced by the most abundant commercial coniferous species Sitka spruce (Picea sitchensis (Bong.) Carr) in the UK is likely to be suited for a wide range of applications. However, due to the typical size of the standing volume lying somewhere between 18-24 cm diameter at the base (FC, 2012) it is unlikely to produce the required quantities of highly valued large sawn sections, used in traditional timber frame construction. However, the increased use of poorer quality materials in value added processes has diversified the types of material being classified as high quality (Bawcombe *et al.,* 2010) and brings opportunities for the re-examination of Engineered Wood Products (EWPs).

The production and successful implementation of new home-grown EWPs into the UK construction market has the opportunity to provide a dramatic reassessment of the value of the current standing timber resource within the UK. If a variety of EWPs can be reliably produced and developed from UK grown timber a larger quantity of the annual timber yield will be able to be used in the production of EWPs, increasing overall consumption of the standing resource and providing larger economic benefits. The immediate and long-term benefits of which will be socio-economic as well as environmental.

The research covered in this thesis has been undertaken to further understand the feasibility of the production of one specific EWP, using UK grown timber, often known as Brettstapel in mainland Europe. In the UK, it is commonly referred to as Dowel Laminated Timber, Dowel-lam or simply DLT. Current information relating to DLT panels is lacking. There are significant gaps in knowledge relating to the strength, stiffness and overall structural performance of a formed panel. Within this context, this research project has been carried out to examine the strength and stiffness behaviour of DLT panels and full timber connections from UK timber, specially larch and Sitka spruce. To determine whether composite performance is created when timber is laminated using hardwood dowels and if the use of a DLT panel could be justified on the basis of improved structural performance.

1.2. Scope of research and research objectives

The research builds on previous investigations undertaken in collaboration between academics at the University of Glasgow (previously University of Strathclyde) and Edinburgh Napier University into the suitability of UK grown timber in other EWPs and has specifically coincided with studies carried out into the production of CLT products from the UK grown resource. The tests conducted during this research project, particularly into the properties of some British timber, have informed these other studies into innovative timber systems using home grown British timber.

A structured research programme of experimental and analytical work was pursued to obtain a number of core objectives, these are:

i. Understand how the introduction of UK produced DLT in the domestic construction market can be achieved. Isolate significant barriers to entry and determine incentives for introduction.
ii. The measurement of mechanical properties of UK Sitka spruce and larch, relevant for use in a DLT panel.
iii. The measurement of some physical properties from a selected group of hardwood dowels to determine their suitability for use within a DLT panel.
iv. The development of experimental tests to determine the interactions within a panel and quantify their behaviour.
v. Experimentally measure the strength and stiffness of a full-scale panel and analyse the panel based on the outcome of previous tests.
vi. Review and discuss the panel system in the context of real-world application in the Black Isle of Scotland.

1.3. Outline of thesis

The structure of this thesis follows the research objectives set out in section 0 and is divided into ten chapters. A brief outline of each is given below.

Chapter 2, 3, 4 & 5: Describes the current EWPs available and presents a context and an assessment of UK timber production. A contextual market study for the implementation of DLT within the construction industry is undertaken.

The study identifies the barriers and drivers for DLT within the industry and presents a potential route to market integration. The type and volume of UK grown timber is clarified and material requirements for a DLT panel are identified. Literature relating to the current method for design of individual timber beams and composite timber beams are reviewed. This review covers current research and design practices for laminated timber flooring and identifies gaps in the knowledge base that relate specifically to DLT panels. Literature relating to the connections within a DLT panel is also assessed including a review of the strength and stiffness of timber connections using timber dowels and metal dowel-type fasteners, and how these methods relate to the calculation of an all-timber connection is discussed.

Chapter 6: The identification, appraisal and selection of suitable softwood and hardwood materials for a DLT panel from the UK grown resources is conducted. Appraisal of the softwood elements include, strength testing, acoustic grading, dimensional stability and embedment strength. Appraisal of the hardwood elements include, dimensional stability and calculation of the effective yield moment of a timber hardwood dowel. An evaluation of the interaction between the different components within a DLT panel is also undertaken using empirical testing of a combined embedment arrangement.

Chapter 7: Describes an experimental program and sensitivity analysis relating to the investigation of the lateral load resistance of timber dowel joints and transverse stiffness of joints over multiple shear planes. The analysis follows a structured approach in which the effects of each factor on the joint behaviour within the panel are individually investigated.

Chapter 8: Utilising the research undertaken in this thesis, production of several DLT panels is carried out. Experimental tests are undertaken on 7 DLT panels and their strength and stiffness properties are quantified.

Chapter 9: Through collaboration with MAKAR Ltd, the information obtained throughout this thesis was used to aid the construction of a new build timber framed house. Discussion of the design and fabrication recommendations for the panel system is given.

Chapter 10: Conclusions of the study are stated and the applications and the impact of the study are discussed. Potential areas of future study are outlined.

2. Literature Review – Part 1: Market Study

Advances in our understanding of timber have enabled a large variety of EWPs to be developed during the last 100 years. EWPs attempt to exploit the inherent strength properties of timber whilst reducing any natural variations in strength and stiffness. By improving the consistency of the mechanical properties and adding value through manufacturing processes, EWPs allow for the commercial use of under used species and smaller diameter trees (Lam and Prion, 2003). EWPs can be used to enable the increased consumption of the future and existing UK timber stock while supporting more sustainable and economic commercial forestry practices in the UK. Furthermore, EWPs that utilise home grown timber into value added processes align seamlessly with the governmental agendas that centre around innovation and green product development.

EWPs come in many different guises, most commonly in combination with an adhesive but some products are also bonded mechanically. Historically many EWPs such as glue-laminated timber (Glulam), orientated strand board (OSB), Laminated Veneer Lumber (LVL), Fibreboard and Plywood have gained substantial market purchase in the UK and other new EWPs such as Cross Laminated Timber (CLT), are hoping to gain the same industry acceptance. Organisations like the Construction Scotland Innovation Centre (CSIC) have been created to tackle some of the research issues surrounding integrating new products such as CLT into the construction market namely through integration and collaboration with public and private bodies. However, there is greater scope for innovation and alternative EWPs are currently under explored and are often overlooked in the UK in preference for more traditional energy intensive building products.

A key issue in the delivery of any new innovative product is consumer and stakeholder awareness, and the differentiation of the various products and their uses within the market place (Rogers, 2003). Most of the research conducted to date on the implementation of innovation or low carbon products within the construction industry has focussed on an external overarching view of the construction industry provided from experts and academics (O'Reilly and Osmani, 2009; Coates, Gaterall & McManus, 2010; Davies and Osmani, 2011; McLeod, Hopfe & Rezgui, 2012; Iddon and Firth, 2013). Little research has been gathered on obtaining an internal perspective of product innovation and how best a seamless transition or implementation of a product can be made. This chapter investigates, through the use of a market study, attitudes of the key stakeholders within the construction industry toward EWPs. In this context, the barrier and drivers for the implementation of a new novel solid timber EWP, Dowel laminated timber (DLT) are defined and assessed. Research conducted in Part 2.1 highlights informational deficiencies and prevailing attitudes that need to be tackled prior to DLT being specified and procured.

2.1. Dowel Laminated Timber (DLT)

Many EWPs require the use of adhesive, to provide a uniform bond between the individual pieces of timber in order to create sections that negate the natural restrictions of section size, length and quality. Adhesives required to bond the timber elements together tend to utilise formaldehyde which are well known to damage the environment during production and affect indoor air quality during service through off gassing (Hughes, 2015; Ormondroyd and Stenfanowski, 2015; Moore and Cown, 2015). With the increased spotlight placed upon the responsible sourcing and recycling of materials there is a drive to find and develop EWPs that provide the increased performance over sawn timber sections without the lifecycle problems they potentially pose through the use of adhesives.

One EWP, which can avoid the use of adhesives, is Brettstapel or (as commonly referred to in the UK) Dowel Laminated Timber (DLT). DLT was originally developed by the German engineer Julius Natterer in the 1970's (Haller, 2008). Its earliest form consisted of planks (laminations) of sawn timber laid side by side, continuously nailed together to create solid structural elements around 600 mm wide. The width of the planks varied from 80-200 mm and long nails were used to join approximately 3-4 planks at a time. However, when laminations were fixed together using mild steel nails, difficulties were encountered during post fabrication modifications as randomly placed nails meant it was almost impossible to cut the timber without regularly damaging tooling.

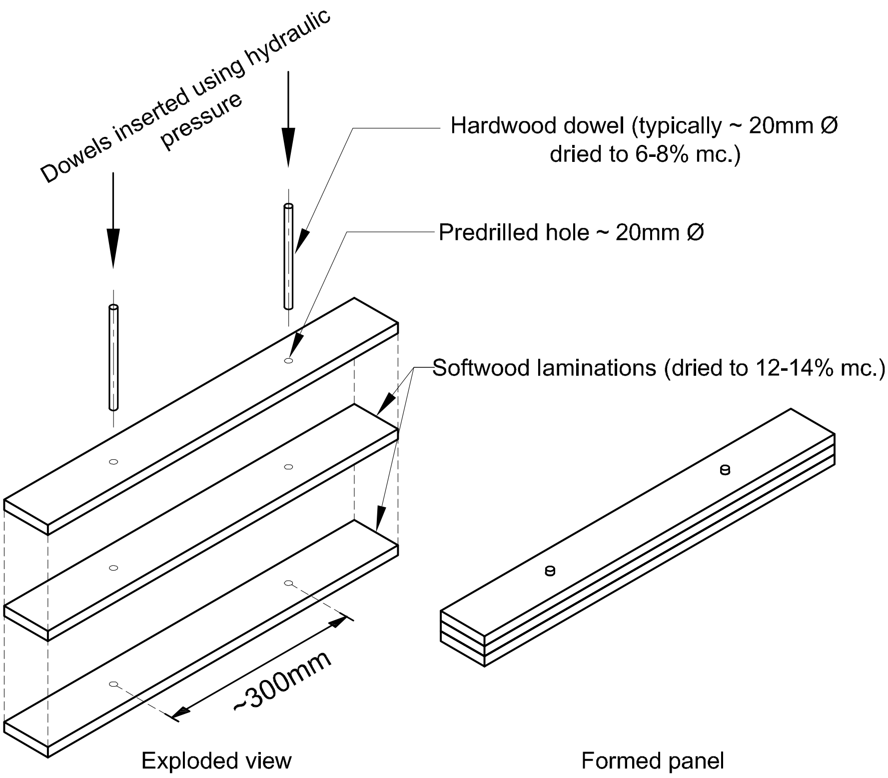

Figure 2-1: Formation of a DLT panel.

Nowadays, it is common practice to substitute the nails with hardwood dowels to bind the laminations together. This limits the use of adhesive in the manufacturing processes, whilst allowing for post processing to be easily achieved. The production procedure involves stacking a series of thin rectangular sawn timber pieces (laminations) typically between 20 – 45mm thick on their long face, aligned in the same grain orientation and joined together at regular centres using timber dowels passing perpendicularly through the centre of the laminations. The fixity between the planks is achieved by inserting hardwood timber dowels (typically between 12-24mm in diameter) into regularly spaced predrilled holes that run perpendicularly through the planks with substantially lower moisture content than the surrounding softwood laminations (see Figure 2-1, above). The differential between the moisture contents of the two differing wood species causes the dowels to swell as moisture equilibrium is achieved and this secures the planks together by friction and the normal force exerted by the dowel expansion.

2.1.1. DLT production

The manufacture of a DLT panel can be seen as rudimentary when compared to the facilities needed to create other forms of EWPs. A review conducted on the main European manufacturers shows that there are slight variations in the process of production depending on the end product use and the desired finish. In general, the manufacture of a DLT panel requires the cojoining of several key processes in a streamlined fashion, an overview of these key processes is detailed in Figure 2-2. Although the processes used to create the panel are technically straightforward, the tolerances required in the processes and in the materials themselves to achieve a consistent panel are equal to other forms of EWPs. An optional production stage that uses adhesives (shown in red in the Figure 2-2) is finger jointing which can be used to create larger lengths of continuous laminations. Finger jointing is not necessary for DLT panels, which are sized within the bounds of naturally occurring lengths. Typically, timber provided for domestic construction falls with naturally occurring lengths, as spans will not regularly be over 5 metres.

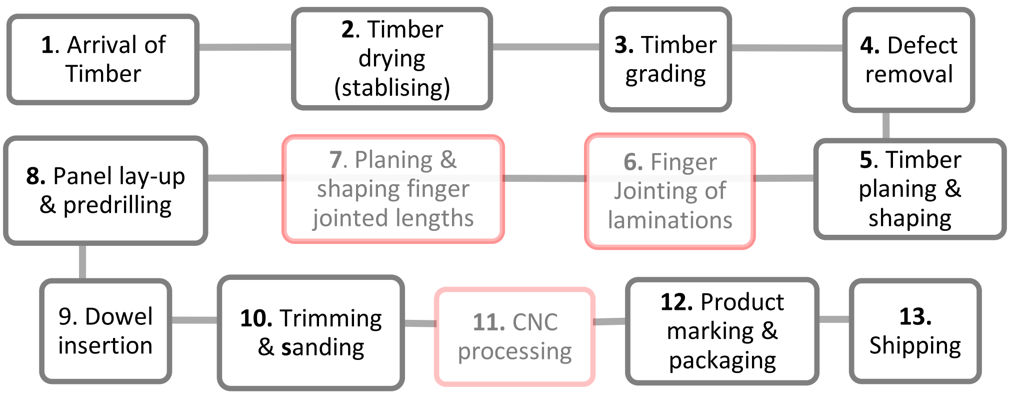

Figure 2-2: Outline of DLT production process.

2.1.2. DLT application

To assess any EWPs, it is necessary to isolate and investigate the most efficient and suitable use of the product at a particular time within a specific market. DLT offers a potential method of utilising massive amounts of low value timber for high value uses within a low technological framework without the aid of adhesive. Ranging from structural to non-structural, decorative and industrial, DLT can be used in a myriad of applications within the fabric of a building. The product is versatile, can be produced in a myriad of shapes, a variety of finishes and a potentially infinite amount of sizes (within the realms of transportability).

DLT requires a minimal capital initial investment for production, which opens up its potential to Small Medium Enterprises (SMEs) as well as larger manufacturers. Currently, there are around 20 manufacturers in Central Europe creating a range of products using the DLT. These range from composite flooring systems to structural wall panels all manufactured to high degrees of tolerance and performance. In almost all instances of production, there is an indication that the market value of the product is from the creation of a high-quality product that is manufactured off-site in factory conditions and used within low-rise and domestic buildings. Looking at the more mature and established markets for DLT there is an indication that the most common and effective application is within two main areas of usage, walling, flooring. The panels can be used in both visible and non-visible scenarios within buildings and can be created in a variety of arrangements and specifications suited to their end use, see Figure 2-3.

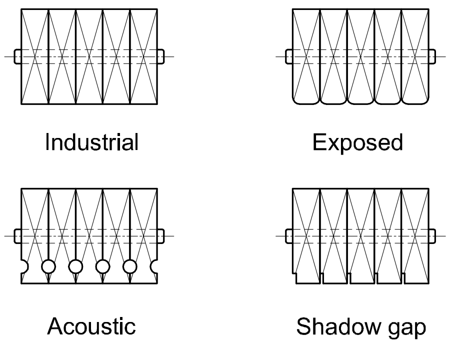

Figure 2-3: Some example finishes of a DLT panel.

2.1.2.1. DLT Wall Panels

Perhaps the most immediately apparent use of a DLT panel is its use within walling, in both structurally supporting and non-supporting applications. When used as a structurally supporting wall element, the panel is required to resist all internal and external actions placed upon the panel. Current standard timber frame practice is to sheath stud walling with a board material such as Plywood or OSB or the provision of a separate bracing frame to provide resistance to out of plane loads. Timber frame walls are designed on this basis in the UK in accordance with BS EN 1995-1-1 (2004) and more recently in line with further guidance given in PD 6693-1 (2012). Simplistically the DLT wall panel can be treated as a timber stud wall with studs placed at close spaced centres and it will resist any vertical actions in an identical manner to a counterpart stud wall at similar spaced centres, see Figure 2-4.

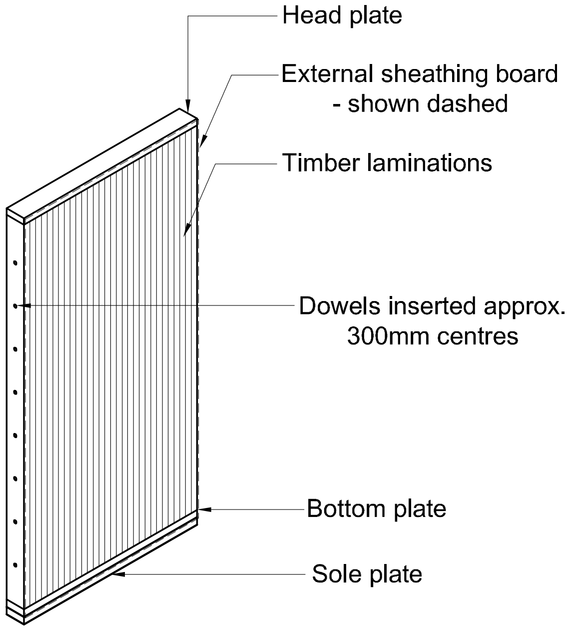

Figure 2-4: Isometric view of an isolated DLT wall panel.

2.1.2.2. DLT Floor Panels:

Many European DLT producing companies supply a wide range of floor panels that cater for a variety of differing uses depending on their levels of finish, and profile of the laminations. DLT floor panels can be exposed or hidden depending on the situation but are almost always a load supporting element, see Figure 2-5.

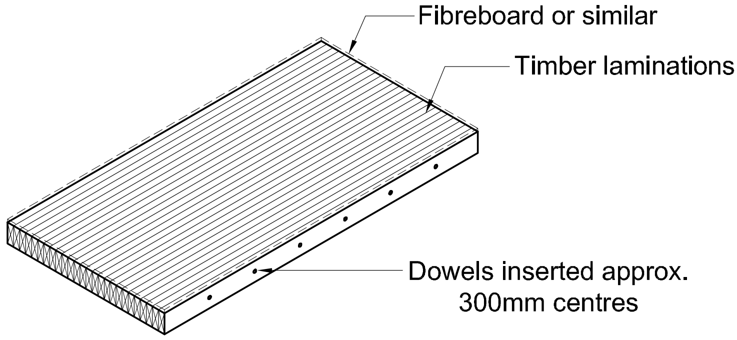

Figure 2-5: Isometric view of a DLT floor panel.

Timber floors are designed in accordance with BS EN 1995-1-1 (2004) the accompanying national annex and the complementary non-contradictory information supplied within PD 6693-1 (2012). DLT should be designed in accordance with the principles in BS EN 1995-1-1 (2004). Many of the DLT flooring products available are supplied with design guidance and technical information such as acoustic resistance and load span tables from the manufacturers, which have been obtained through laboratory testing.

2.2. Market study

A market study was undertaken to gauge the views and knowledge of DLT from current stakeholders operating within the UK construction industry. The market study was designed to identify current omissions in the existing strategy and determine how these fundamental knowledge gaps could be investigated further in this thesis. The market study addressed five key points or areas that were deemed to affect the successful integration of a new EWP into the UK construction market. These were: awareness; perception; suitability for purpose; relative advantage and consumer understanding.

2.2.1. Sampling

Several key stakeholder groups within the industry were identified that could provide a comprehensive selection of views from across the UK construction industry, with a particular emphasis on Scotland. These were based on sub-divisions on the most active stakeholder groups, which broadly represent the chain of supply within the construction industry, these were:

- Manufacturers
- Designers
- Contractors

The particular focus for the selection and identification of these stakeholder groups was to gauge the extent of variation or identify any similarities of opinion that could exist between different stages of the supply chain, in order to aid the development of a product. Obtaining a comprehensive snapshot of all stakeholder attitudes within the construction industry would have been impossible due to the fact that many stakeholders are temporary (i.e. clients) and have no affiliation with any agency or body (Walker, 2007; Fewings, 2005). However, a study conducted on the most active stakeholders who would have the greatest ability to support or conversely limit the integration of a DLT product into the construction market was deemed most appropriate for initial study.

2.2.2. Data collection

The survey design drew on the experience of previous studies conducted by the Centre for Offsite Construction and Innovative Structures (COCIS) into solid timber products, discussions with industry professionals and the review of literature. The survey comprised seventeen distinct questions relating to EWPs within the UK, with a primary focus on DLT and CLT products, see Appendix A. The questions corresponded broadly to five key thematic areas:

- Understanding the perception of UK grown timber within the construction industry.
- Determining how aware the professionals within the construction industry are of all forms of EWPs.
- Identifying the potential uses for EWPs and determining their particular location within the UK marketplace.
- Recognising the obstacles and incentives for the implementation of EWPs.

- Developing an understanding of potential EWP use in the future.

Data collection took place for a period of three weeks commencing 27th November 2013 and finishing on the 18th December 2013. In order to gain the most accurate and widespread response the methodology developed by Dillman (2007) was utilised. This involved initiating multiple separate points of contact including, a brief pre-notice email, an email with the questionnaire link a few days later and a reminder email approximately one week later. The Internet survey was distributed to a total of 323 people and in total, 115 usable responses were received by the closing date. This represented an overall response rate of 35%. Whilst the responders were asked to complete 17 questions related to EWPs only 11 of these questions related to a, the descriptive statistics and b, DLT. The remainder of the questions related to CLT and were collected to enable further research into CLT to be undertaken at a later date, the results and the analysis of these additional questions have been omitted from this chapter and from this thesis in general to avoid repetition and focus solely on DLT.

2.2.3. Interviews

To augment the findings of the on-line survey, a series of six semi-structured interviews were conducted during the survey period on a selection of professionals within the industry. The aim of the interviews was to provide context and build upon the quantitative data obtained. No specific details of the content of the interview were given prior to conducting the interview to allow for a true representation of the participants awareness of EWP's and any specific or specialist knowledge that they may already have about them. To create a standardised assessment an interview template was created prior to commencing the series of the interviews (see Appendix B), alongside the standard series of consent forms required from an interview of this nature.

2.3. Data analysis

Whilst every effort was made to ensure that the sample study was as wide ranging and comprehensive as possible, there is inevitably some coverage error. Of the three previously identified stakeholder groups, 41% of responders were designers, 25% were manufacturers and 14% were contractors. The final 20% were suppliers, local authorities or other consultant types that work in areas beyond the identified stakeholder groups (see Figure 2-6).

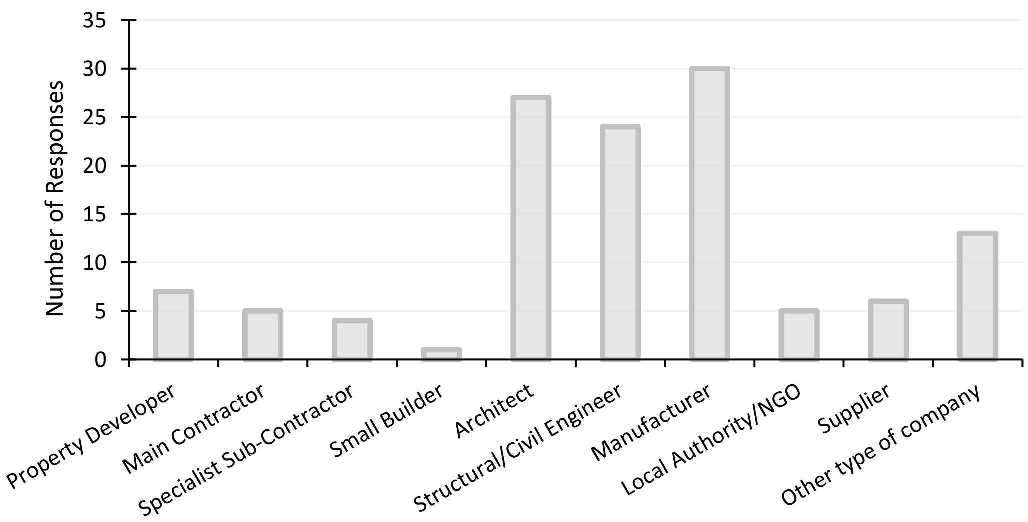

Figure 2-6: Occupation of responders.

Understanding the varying positions of employees within different sized companies was of interest, as their outlook may vary due to their individual perspective of the industry. Larger companies may tend to have more capital for R&D investment or a wider range of in-house expertise and therefore could have a differing outlook towards the design and manufacture of a product compared with a smaller company. The variation in the occupations of the respondents is illustrated in Figure 2-6 and was heavily skewed towards the design side, such as Architects and Engineers, Similarly the size of the company where respondents were based tended to favour smaller consultancy firms with fewer than ten employees (53%), see Figure 2-7.

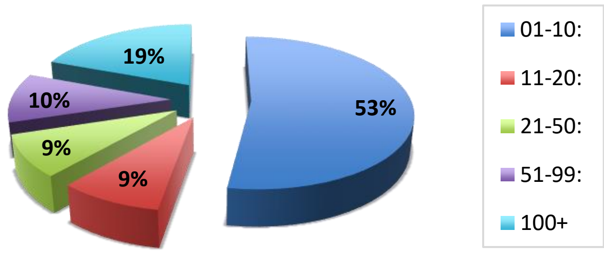

Figure 2-7: Number of employees in the company of survey responder.

The geographical areas that the samples were undertaken in were carefully selected to optimise the range of attitudes towards the products. This was due to the variable nature of construction techniques within the UK and also potential support that might exist from local planning authorities for particular construction systems. The survey was purposely slanted towards a more Scotland-centric base (63%) due to the greater likelihood of UK production of an EWP occurring in this locality due to the relatively significant forestry provision.

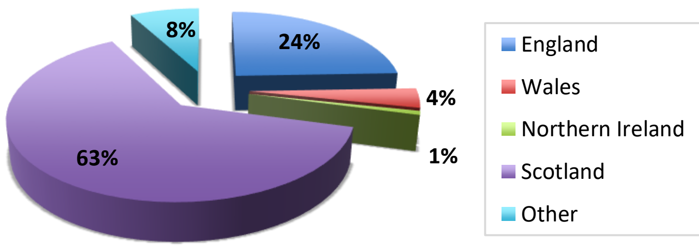

Figure 2-8: Location of survey responders.

2.3.1. The perception of UK grown structural timber

Concerns regarding the suitability of UK grown structural timber for high value products have been raised in the past by a number of researchers (MacDonald and Hubert, 2002; Malcolm, 1997; Moore *et al.,* 2009a; 2009b). Research by these authors suggested that there many factors within the supply chain that could affect the perception of UK timber to varying degrees. The market study firstly asked responders to rate their perceptions of UK home grown timber according to their own experiences. The results from responders indicate that the industry experience of UK grown timber was mainly ambivalent, with 43% of responders rating their experience of British timber as average and a further 40% rating the product good or better.

The fifth question in the survey asked responders to rate their perception of timber on a five-point Likert scale based upon a predefined set of criteria previously highlighted by researchers as factors that could affect the perception of UK grown timber, these were availability, cost, appearance, workability and structural performance. The results of the response are broken down into Table 2-1.

Perception rating	Perception criteria				
	Appearance	Availability	Cost	Structural Performance	Workability
No importance	3.6%	2.7%	2.8%	1.8%	3.7%
Little importance	9.0%	3.6%	4.6%	0.0%	2.8%
Neither important or important	29.7%	9.0%	8.3%	8.1%	19.4%
Important	45.0%	55.0%	56.9%	47.8%	60.2%
Great importance	12.6%	29.7%	27.5%	42.3%	13.9%

Table 2-1: Responder rated perception criteria for UK home grown timber.

Responders in the survey described that they felt that the fundamental rationale for their overall perception of home-grown timber was its structural performance. The selection of structural timber is highly dependent on design and therefore structural strength and stiffness criteria heavily influence the evaluation of the timber selection. Adequate grading of the material is necessary for the correct specification of any type of timber used in a structural application whether imported or sourced locally and this is a particularly pertinent with the use of UK home grown timber. One survey responder stated "despite being a part of the second largest timber producer in the UK, we have to buy C24 grade from Scandinavia for our product!"

Availability of the material also featured prominently in people's perception of UK timber and many of responders highlighted a lack of understanding about what timber is available at a typical timber merchant and suppliers' yard. The way that timber is treated through the supply chain in the UK was shown to be of additional concern. Particular instances of poor handling of the material through suppliers and finally on site were mentioned throughout the interviews, one interviewee stated that "We had timber on site condemned because it was too moist, it was sent back to the supplier" (Architect A, 2013). There is therefore an onus on the timber suppliers and merchants to provide a wide range of correctly graded and stored home-grown structural timber which competes not just on cost but also on quality of supply.

2.3.2. Determining awareness of differing forms of EWPs

In order to ascertain existing market permeation of existing EWPs, responders were then asked to indicate whether they were aware of the following engineered timber products; CLT; DLT; Glulam; LVL and composite I-Joists. Whilst this list did not encompass the whole variety of EWP's available in the UK market place it did provide a good barometer between the awareness of the more established EWPs that are commonly used in similar situations against the more recently developed or novel EWPs in the UK market such as CLT and DLT.

The results indicated a fairly strong connection with the hypothesis that the more market established EWPs were more widely recognised than the newer solid timber products (see Figure 2-9). Glulam was the most well-known EWP, with 98.21% of responders being aware of the product. Of the other mature EWPs, only 74.11% of responders were aware of LVL. This presents a somewhat lower figure than other established EWPs in the UK. An explanation of why this is the case may be provided by how the product is normally distinguished in the market place and how it is specified. LVL is usually specified through proprietary branded product ranges such as Kerto® LVL, which is produced in Finland by Metsä Wood and may not be as widely known through the generic term LVL.

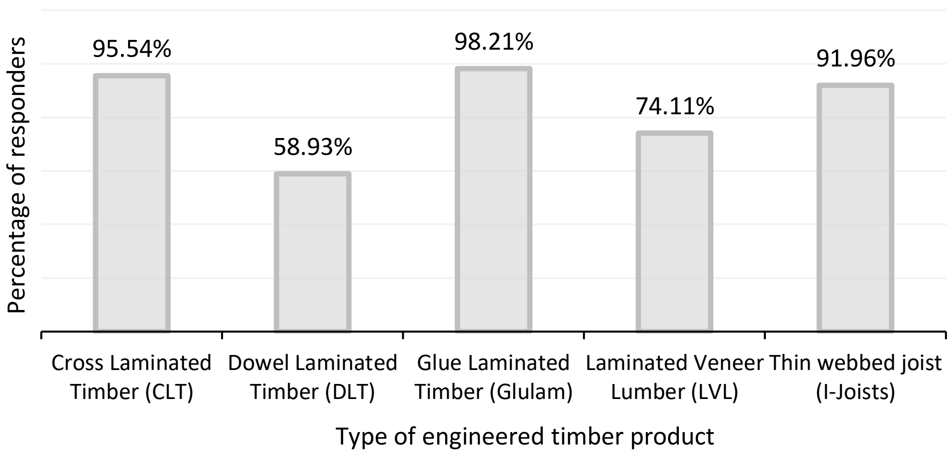

Figure 2-9: Responder awareness of differing forms of EWPs

Interestingly, 95.54% of responders were aware of CLT. This represented a level of awareness that was much greater than anticipated, as CLT is a comparatively immature EWP in the UK construction market. Although CLT is a relatively new form of EWP within the UK it has obtained a large amount of recognition through some innovative case studies such as Murray Grove (The Architects' Journal, 2008). The reputation and the publicity afforded to this building product has created a discourse within the industry that has increased awareness of the product and provoked discussion within the industry on the potential to provide large volumetric solid timber construction in the UK. Currently no large facilities exist in the UK for the production of CLT and the manufactured material is sourced from Europe, however pilot schemes have been implemented using UK grown timber (Crawford *et al.,* 2015). The lowest awareness metric by some considerable margin was DLT, with only 59% of respondents aware of the product. This concurred with the assumption that DLT would be a marginally more unknown product due to its juvenile nature in the UK construction market. DLT still remains a novelty material used in bespoke domestic housing in the UK and has not yet made the transition as a widely known construction technique. In fact, none of the responders had ever specified DLT prior to undertaking this survey, indicating that of the responders that were aware of the product none were in the position to integrate or justify the product selection within a project that they were undertaking.

2.3.3. Identifying the uses for DLT

The next stage in the study was to ascertain where the responders felt, based upon their experience, a DLT panel would be suited to be used. During the creation of the survey it was deemed necessary to provide an illustration of a typical DLT to enable the responder to answer the question of DLT use in a more informed manner as many of the responders would not be initially aware of DLT. The illustrated DLT panel was set in the vertical orientation, providing a visual context for the selection of DLT as a walling product. An overwhelming majority of responders (91%) felt that DLT could be used as a wall material. Considering that just under two thirds of responders were initially unaware of DLT, the initial presentation of the product could have affected their perception of the product and hitherto the selection of product towards its use as a walling product.

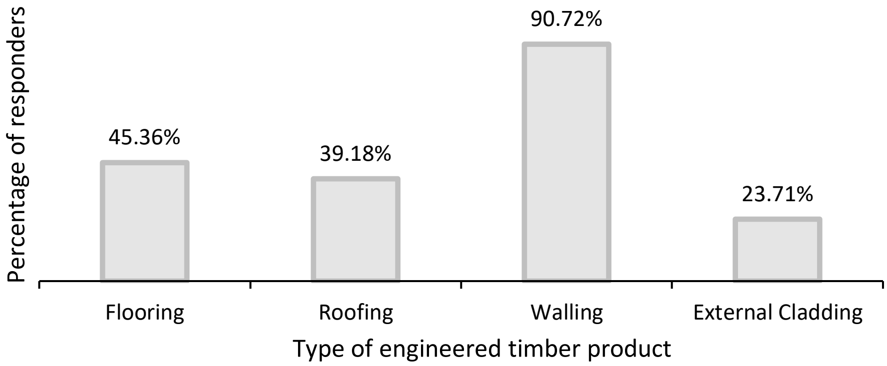

Figure 2-10: Application suitability of a DLT product according to survey responders.

Aside from walls, responder's felt that the other stated uses such as floors and roofs had similar potential (See Figure 2-10). Overall, the results indicated that people are not certain where DLT should be used unless a clearly defined element is presented to them.

2.3.4. Market location

The responders were asked to select the market areas that they felt were most appropriate for the implementation of DLT. The top three market areas indicated by responders related to the private housing and public buildings sectors (see Figure 2-11).

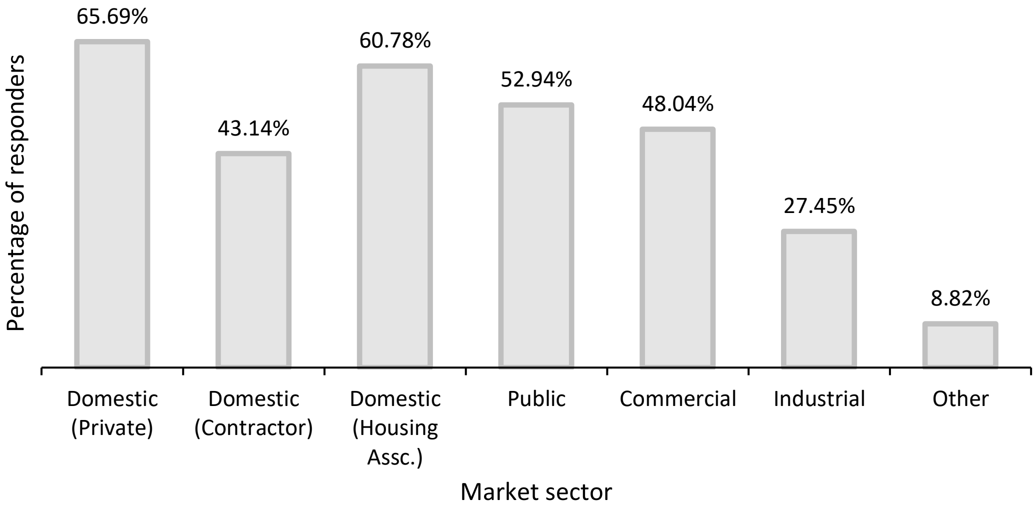

Figure 2-11: Potential market location for DLT according to survey responders.

There is a tendency within the construction industry to depend on established processes or products when budget security or certainty is required (Lovell & Smith, 2010) making a novel EWP a less viable solution in these instances. Correspondingly, the survey responders felt that where budget driven constraints were more likely to occur, there would be less chance for a novel EWP to gain traction in these areas. This is illustrated by a noticeable reduction in the percentage of responders believing commercial areas of construction were viable. Whilst the size of these projects in the markets indicated may not individually generate large volumes of DLT, the breadth of the market achievable has the opportunity to create scenarios for DLT's successful implementation. These market areas could be further refined to take advantage of modern methods of construction and specifically target a particular cluster of responders that are most likely to use the product in the future.

2.3.5. Market area and future use of DLT

A snapshot of the overarching perception toward DLT was determined through the final question in the questionnaire. This directly asked the likelihood of DLT inclusion or at least initial specification in a project in the future. The outcome of the poll was not immediately positive (Figure 2-12). Although approximately 51% of the poll suggested they were likely to use DLT in the future, they felt this

this would only come to fruition if the barriers towards implementation were overcome effectively.

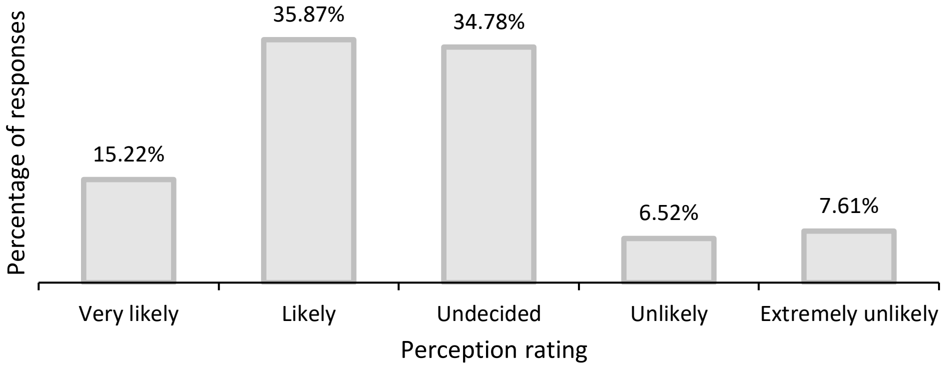

Figure 2-12: Likelihood of responders to specify DLT in the future.

2.3.6. Obstacles and incentives for the implementation of DLT

From the initial desk study, several factors were highlighted as being either possible barriers or drivers for the implementation of UK manufactured and sourced DLT. Respondents were then asked to rate these factors on a five point Likert scale to ascertain their relative importance to DLT manufacture, specification and installation, see Table 2-2 and Table 2-3.

Perception rating	Perception criteria						
	Building applications	Certification & guidance	Cost	Existing Products	Investment required	Lending, insurance & Risk	Market size
Negligible impact	2.2%	2.2%	1.1%	3.3%	2.2%	1.1%	3.4%
Small impact	6.5%	5.5%	2.2%	7.8%	3.2%	6.5%	2.3%
Neutral	20.7%	13.2%	9.7%	22.2%	24.7%	28.3%	20.2%
Significant impact	51.1%	51.7%	41.9%	42.2%	51.6%	38.0%	52.8%
Critical impact	19.6%	19.6%	45.2%	24.4%	18.3%	26.1%	21.4%

Table 2-2: Responder rated perception criteria for the barriers to home grown DLT.

Perception rating	Perception criteria							
	Acoustic properties	Aesthetic appeal	Legislation	Low embodied energy	Offsite Construction	Opportunity	Speed of construction	Thermal properties
Negligible impact	2.2%	1.1%	2.3%	2.2%	1.1%	3.3%	1.1%	1.1%
Small impact	3.3%	4.4%	9.1%	5.5%	2.2%	3.3%	0.0%	4.4%
Neutral	31.1%	22.0%	37.5%	23.1%	17.6%	42.9%	13.6%	16.5%
Significant impact	52.2%	57.1%	42.1%	50.6%	57.1%	45.1%	61.4%	55.0%
Critical impact	11.1%	15.4%	9.1%	18.7%	22.0%	5.5%	23.9%	23.1%

Table 2-3: Responder rated perception criteria for the drivers of home grown DLT.

The top two ranked positive responses, speed of construction and the potential of offsite construction (see Table 2-3), can be directly related to have an impact on cost and efficiency savings. Responders felt that justification of the product could be found through the time and ultimately cost saving achieved through this form of construction. The environmental performance of DLT rates highly in peoples' perception but it appears not to be the rationale about why people would select DLT but rather it would be seen as a secondary benefit. These secondary benefits would provide an additional motivation for the selection of the product but would fail to justify the product solely on those merits.

2.3.7. Additional themes not previously encountered

The internal view of the industry placed greater emphasis on two key themes that had not been considered. One the influence of variations in the procurement process on product selection, and two the dissemination of knowledge through the supply chain. Responders stated decisions taken in the procurement process are often unrelated to design ambition and can frequently inhibit the specification of products untried in the UK market place.

"People who make procurement decisions are not interested in expertise. They are interested in finance and speed and they are interested in clarity of contractual definition." (Architect B, 2013)

Some types of procurement processes traditionally seen in the UK may therefore limit specification of any novel or innovative product not just an EWP.

Consideration has to be placed on other actors within the construction procurement process that could affect and lead client decision processes. "Quantity surveyors are quite traditionally minded and have a lot to say to the client in terms of cost and program" (Architect C, 2013) and will inevitably err on the side of caution against newer building products. Once a ratified cost comparison between competing products is achieved and permeated through the industry, resistance against implementation purely on cost basis will be reduced as the risk of specifying a new innovative construction product will be tangible related to a comparative price cost. "Builders' have a massive swing over clients and the decisions they are going to make and the materials they are going to use" (Architect D, 2013). Ultimately, clients are likely to be steered towards products, which generate the greatest profits for the contractor.

Many in practice consultants "have no time on our timesheet for CPD (continuing professional development) or any additional learning" (Structural Engineer A, 2013). If no clear concise guidance for the product is provided or is easily available, the specification of DLT or any other innovative construction product will not occur. Many of the interviewees attributed the loss of traditional carpentry skills within Scotland and the UK as a key reason for the lack of development of timber construction in recent years. Subsequently, these lack of skills at the construction and installation phase have had a knock-on effect in the quality of timber structures being constructed within Scotland and the UK.

Due to the fragmented nature of stakeholders within industry and the ephemeral nature of design teams, the knowledge gained through collaboration and the use of new design products is frequently lost (Walker, 2007; Fewings, 2005). Without a central hub for research and knowledge exchange, which can be aggressively disseminated into industry, any new product uptake may stall at the first hurdle of the design desk. "It is a combination of trying to get costs down and having the technical literature available to give us the security to supply it" (Structural Engineer A, 2013). Since the completion of this questionnaire in 2013 steps have been taken towards creating focal hubs on websites for the dissemination of knowledge throughout the supply chains, such as the Construction Scotland Innovation Centre (CSIC).

2.4. Factor and cluster analysis of barrier and drivers:

To further understand the underlying motivations of the responders a factor analysis was conducted in the statistical analysis package, SPSS (IBM Corp, 2012). This was carried out in order to understand and explain any significant correlations that may have existed and reduce the data set to a series of summary variables that can explain the motivations of the responders. The responses given in relation to the barriers and the drivers were given an aggregated score based on the Likert scale, which was then used as the basis of the factor analysis. The factor analysis conducted had a Kaiser-Meyer-Olkin measure of sampling adequacy of over 0.8 indicating good factorability (Hair *et al.,* 2010) and was conducted using a factorability process based on an anti-image correlation matrix diagonal, with a principal component method of extraction assuming an orthogonal rotation between variables (i.e. independent variables). Although the factors were treated as independent, some minor interdependence will inevitably exist due to the interconnectivity of opinions that could exist in this topic. That is to say, if a responder had a poor perception of UK timber, the scores that a responder provided for all the isolated variables in the questionnaire would be skewed one way or another based on their overall perception of the central question, however for ease of analysis this was assumed not to be the case.

Five summary factors were defined through the analysis that encompassed 71.56% of the total variance that existed between peoples' perception of the barriers and drivers of EWP. Table 2-4 illustrates the five underlying factors chosen from the analysis to best explain the overall perceptions towards the barriers and drivers for DLT implementation. The factors shown in Table 2-4 are named appropriately based on the highest primary loadings of the key variables. The closer the factor loading is to 1, the higher the underlying strategic variables communality to the encompassing descriptive factor.

Underlying Strategic variable	Factor loading
Factor 1: Soft engineering	
Low embodied energy	0.777
Opportunity for new business	0.704
Aesthetic Appeal	0.699
Thermal properties	0.681
Factor 2: Competitive	
Existing Products	0.802
Factor 3: Cost orientated	
Investment	0.829
Market size	0.712
Cost	0.599
Factor 4: Modern Methods of Construction	
Offsite construction	0.869
Speed of construction	0.814
Factor 5: Technical guidance and liability	
Certification	0.873
Lending, risk, Insurance	0.592
Applications	0.546

Table 2-4: Factor analysis of barriers and drivers for DLT implementation.

To reduce the number of variables to those that provided the best explanations for the range of perceptions encountered. The second phase of the analysis elaborated on the isolated strategic variables by assigning factor scores to a hierarchic and non-hierarchic cluster analysis using the Ward Method as recommended by Hair *et al.* (2010). This enabled the further classification of stakeholders with similar objectives and behaviours into distinct groups or clusters to clarify more efficiently where a new product should be positioned and what areas of the product should be developed. From the cluster analysis, four groups were identified, named and profiled based on the factor scoring and the profiling of the associated independent variables according to their objectives, motivations and attitudes. These were identified as:

Cluster 1: ***Forward thinking; high potential selector***

Likely to use, cost driven, but see the benefits through modern methods of construction (32.2%).

Cluster 2: ***Cost concerned; undecided selector***

Undecided, cost driven, barriers of legislation and guidance currently overcome the benefits (13%).

Cluster 3: ***Design concerned; undecided selector***

Undecided, greatly concerned about the design aspects over the cost (19.1%).

Cluster 4: ***Unlikely selector***

Unlikely to use, see only limited relevance in the product (1.0%).

From these identified consumer segments, the '*high potential selector*' highlighted in the cluster analysis represents the best avenue to exploit for market implementation. However, the key variable throughout these entire clusters is the cost of the product. At this very earliest juncture of product research and development the cost certainty of the product is very difficult to achieve. Cost viability of the product is something that is clearly considered from the outset of product development but cannot be made within high levels of accuracy until the decision to commercialise the product is made. The second key factor to be isolated by the cluster analysis is the technical guidance necessary to ensure that specification can be undertaken seamlessly with the required amounts of confidence for the designer or installer of the DLT product.

2.5. Discussion

The internal view of the industry collected in this market study is fundamental in achieving a holistic appreciation of the potential of any product, but especially a product such as DLT. The market study identified that there are many divergent paths to DLT implementation depending on the volumes of manufacture, speed of implementation and the type of market targeted.

The market study has highlighted the key market areas that the product should focus on, at least initially. These are, bespoke domestic construction or public works. The panel type highlighted by the responders, as most suitable was walling. Whilst this response was tempered by initial preconceptions discussed in section 2.3.3, the technical information required will not be different from existing timber frame walls unless significant research is carried out into the in-plane racking strength of the panels without sheathing. Floor panels were also highlighted as a likely candidate by just under half of all the responders (45%) for inclusion into the construction market. The provision of flooring panels aligns with the manufacture requirements and can omit the extension of the laminations using adhesive as long as the spanning arrangements are not greater than the natural lengths of timber available in the supply chain. Floor panels provide a rich avenue to explore as the predominant type of flooring used in domestic dwellings throughout the UK is timber, whilst the type of walling used varies from masonry to timber depending whereabouts in the UK the construction is taking place. The development of a floor panel using DLT would achieve a far greater market share throughout the UK as a whole, due to its ability to integrate into the building rationales that are already in existence in the UK.

The results from the survey and the interviews support the idea that there is an unequal discourse in selection and procurement of material between those who pay the initial upfront costs of a construction project and the client who uses the building over the longer term (Rodrigues, Garatt & Ebbs, 2012). The supply chain in the UK is predominantly set up for ‘volume house builders’, that is, companies that build large housing developments based on an approach that is orientated towards the lowest material cost (Parag and Darby, 2009). Therefore, for these companies there is a greater emphasis applied to financial factors or constraints (O’Reilly and Osmani, 2009; Lovell and Smith, 2010;

Rodrigues, Garatt & Ebbs, 2012; Wang, Toppinen & Juslin, 2013) when specifying a building product over any other factor. Under this paradigm traditional building typologies are more likely to be used due to their initial cost certainty. Therefore, the justification of the product needs to be centred on a discourse emphasising the tangible aspects of the product, such as cost reduction, speed of construction and improvement of build quality.

The most obvious advantages of DLT are the 'soft' engineering factors that architects and designers are increasingly looking to specify for compliance with zero carbon and embodied energy criteria. These benefits can be broadly broken down as:

- Low embodied energy
- Improved indoor environmental quality
- Increased thermal mass
- Low thermal conductivity
- A breathable building envelope
- Acoustic resistance properties
- Low Volatile Organic Compounds (VOCs) emissions

However, these benefits are subservient to the safety and structural adequacy of the product in the situations they are used. When the long-term cost and the permanency of the product are considered DLT could provide significant savings through its life cycle by reduced foundation costs, lack of maintenance requirements, energy saving, VOC emissions, its ease of final demolition and recyclability. The survey highlights that research and development should relate to the materials available and the certification process needed to bring a construction product into the market in the EU. Technical guidelines and a sound knowledge base were clearly important to the responders and detailed research and development would be required to be undertaken on all aspects of DLT's performance before full market integration by relevant stakeholders could begin. At present this is not available for DLT produced from UK home grown resources.

Carrying out rigorous testing and analysis to allow the production of easy to understand technical information and extolling the product in relation to these identified drivers enables a focussed method for transitioning DLT into the construction market through specific stakeholders in key market areas. The emphasis of the research into a DLT panel was therefore focused on providing the information required for specification in accordance with the national design codes. Afterward, further research can be conducted into other technical characteristics for a variety of different uses and can also include the 'soft engineering' factors highlighted above.

2.6. Developmental framework

The transformation of this initial market survey into a methodological framework that acknowledges the motivations and dynamics of the various key stakeholders is vital to successful product implementation of DLT. The market study data has focussed the direction of the research in this thesis by providing the information sequentially needed in order to develop the product for market integration. An overview of the first 3 steps for the development of DLT and initial implementation into the UK construction industry are defined below. These steps will then be taken forward as the key areas of research within this thesis.

1. ***Initial panel specification from the home-grown UK resource***
    - Identification of the most suitable materials to be used in a DLT panel from the UK resource.
    - Determine the mechanical properties of the panel using home grown materials.

- Decide upon the combination of materials used within a panel including dimensions and type of species. Defined by availability, performance and cost.

2. ***Performance testing & feasibility analysis of preliminary panels:***
    - Evaluate different configurations of panel including lamination geometry, connection types, to understand and develop best practice scenarios.
    - Undertake studies on the mechanical properties of the panel.
    - Expand and develop existing methods for the analysis and design of existing laminate panels.
    - Provide methods for analysing the structural performance of a panel, in accordance with BS EN 1995-1-1 (2004).

3. ***Pilot manufacture of UK grown product:***
    - Create pilot panels based on the recommendations from the previous stages
    - Assess and evaluate the manufacturing process and analyse the mechanical properties of the pilot panels.
    - Critique (including, cost performance against manufacture time) and recommend changes for future development.

3. Literature Review - Part 2: Properties of British Timber

Following the developmental framework specified in the previous chapter, this chapter identifies the raw material requirements to produce DLT from and their availability within the UK. Determining the suitability and availability of British timber is more involved than simply counting the number of trees in the forests in the UK and requires an assessment of all levels of the supply chain. The supply chain of timber production can be simplified into two defined sections: upstream and downstream supply. The upstream supply chain deals with the availability, acquisition and primary processing of the raw resource prior to delivery to merchants or further processing plants. The downstream supply is concerned with manufacture, distribution and specification of the product. The formation of a DLT panel is dependent on the availability of two key material resources in the upstream supply chain:

- Softwood for the laminations
- Hardwood dowels for the connection between laminations

3.1.1. British softwood timber

Consideration needs to be given to the type and quality of the timber available and its suitability for use within a DLT product. For structural use, the three main quality criteria typically are, strength, stiffness and dimensional stability (Kliger *et al.,* 1995). Aside from these quality criteria one of the fundamental rationales for the selection of the lamination material is the accessibility of the timber material in the necessary quantities, sufficient quality and the correct dimensions.

Due mainly to a significant increase in private sector production, softwood production out of UK forests has been steadily increasing in the last 25 years (FC, 2015). 2014 saw the extraction of 11,431,000 green tonnes of softwood from UK forests. In the past 40 years the forest area in Scotland alone has risen from 12% to over 18% (FC, 2015) and the National Forestry Inventory (NFI) predicts that the availability of softwood will continue to rise past current levels by at least 10% and peak within the next 15 years due to the existing stock reaching maturity (FC, 2014a).

A breakdown of the current standing coniferous timbers that are commercially available in the UK is shown in Figure 3-1. Sitka spruce should be considered as the first softwood species to be used in any EWPs due to its prevalence in the UK as a commercial conifer. Currently it accounts for approximately 50% of the UK softwood resource and over 60% within Scotland (FC, 2015).

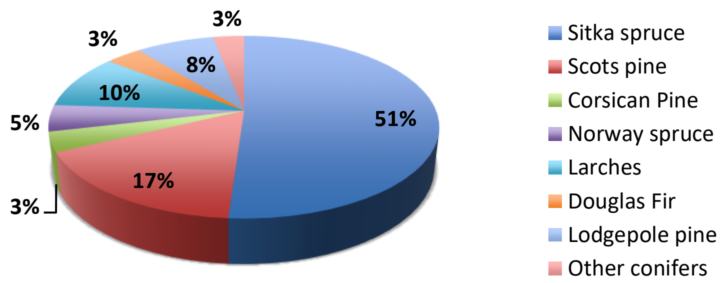

Figure 3-1: Proportion of coniferous timber species in the UK (adapted from FC, 2015)

Another species that needs to be strongly considered for any EWP produced from home grown sources is larch. Although only representing a mere 8% of all conifer timber volume, large amounts of larch are currently being felled to contain the spread of Phytophthora Ramorum (commonly referred to as Ramorum disease). Ramorum disease is a fungus-like pathogen, which has no known cure and can lead to extensive damage or mortality to a number of tree species, particularly oak and larch. Larch is highly susceptible to the disease and is known to act as an incubator and beacon for the spread of the disease. First found in South West England in 2009, it has quickly spread through the UK reaching Scotland in 2011 (FC, 2014b). Rough estimations suggest that 240,000m^3 of sawn larch will come into the market, representing almost 7% of all home-grown softwood used in the UK in 2015 (Moore & Cown, 2015). This creates pressure for the industry to develop additional scenarios for its use, such as inclusion in EWPs.

3.1.1.1. Softwood requirements

Despite the immediate availability of larch and ongoing availability of Sitka spruce it is not known whether timber produced from these species is adequate for the production of DLT. Vital to the development of a DLT panel or any other EWP from the UK timber stock is its ability to initially align its configuration with the available stock whilst maintaining high levels of performance. Three fundamental parameters of the upstream softwood timber supply need to be aligned in order for the resource to be acceptable for the construction of an EWP, such as DLT. These are:

- Available section sizes and lengths
- Dimensional stability during drying
- Strength and stiffness

3.1.2. British hardwood timber

The UK benefits from having a wide range of native hardwoods, many of which could potentially be used within the production of DLT panel. A breakdown of the principal species is shown in Figure 3-2.

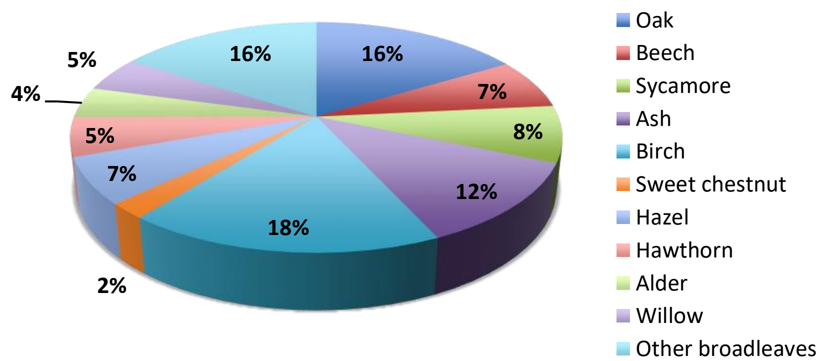

Figure 3-2: Proportion of deciduous timber species in the UK (adapted from FC, 2015)

Within the UK, deciduous forest covers a similar area to that of coniferous forests but is largely owned (92%) outside of the two main public bodies, the Forestry Commission and the Forest Service (FC, 2015). Hardwood is mainly preserved for fuel production (75%) and only a small percentage of the raw extractive (15%) is exposed to value added supply chains (FC, 2015). Latest estimates of UK hardwood availability suggest supply will double every 10 years for the next 30 years (FC, 2015). In contrast, the last 25 years has seen a significant decline in the amount of hardwood deliveries from UK forests despite the range and quality of species grown.

3.1.2.1. Hardwood requirements

Hardwood dowels are used to join the stacked softwood laminations in DLT together because of their higher ratio of moisture movement and typically greater strength properties when compared to the softwood laminations. The UK hardwood resource would need to be able to align with specific criteria for inclusion into a DLT panel. For the hardwood timber that will be formed into dowels the criteria that it must meet are:

- Manufacturing ability and tolerance
- Dimensional stability during drying
- Bending and shear strength and stiffness

Typically, in European production of DLT European beech is used to fuse the laminations together. Little is known about the suitability of other hardwood species within a DLT panel. In the UK, beech represents a mere 7% of the entire hardwood available in the UK and other hardwoods including oak, sycamore and ash represent a far greater proportion of the available hardwood (FC, 2015). The exemplar hardwood used in the manufacture of DLT panels in mainland Europe will be followed but other hardwood species will be considered due to their prevalence in the British Isles.

3.1.3. Available sizes of UK timber

Normally, during the process of converting felled logs into timber, the timber producers do not know the specific end-use intended. It is therefore common practice in the UK to provide a selection of stock sizes that can be readily specified to ensure availability and reduce cost. The stock sizes are referred to in the standard BS EN 336:2003 (2003) as 'target sizes' and are available in a number of differing cross sections and alternative finishes. Whilst the inclusion of target sizes has been removed from the current version of the standard, BS EN 336:2013 (2013), it is standard industry practice to still provide timber in stock sizes that align with the target sizes given in previous iterations of the standard, shown in Table 3-1.

Thickness (mm)	Width (mm)								
	72	97	120	145	170	195	220	245	295
35		✓		✓	✓	✓	✓		
44	✓	✓	✓	✓	✓	✓	✓	✓	
60				✓	✓	✓	✓		
72		✓		✓	✓	✓	✓	✓	✓
97		✓		✓		✓	✓	✓	✓
145				✓					✓

Table 3-1: Common target sizes for UK sawn timber machined on all four sides, tolerance class T2, adapted from Table NA.4 BS EN 336 (2003).

For EWP and DLT manufacture, dimensional tolerance is a pivotal consideration, this limits gaps occurring between laminations, eases adhesive bonding (Moore & Cown, 2015), provides a uniform bearing at points of support and where the panel is left exposed, delivers a pleasing aesthetic appearance. Timber is normally differentiated into two differing tolerance classes; tolerance class 1 (T1) is normally for rough sawn timber and tolerance class 2 (T2) for timber that is planed or machined. The tolerance classification ensures that the timber provided has only small amounts of distortion (Moore & Cown, 2015) and does not deviate from the published section size by a tolerance more than the values shown in Table 3-2.

Tolerance Class	Tolerance for thickness and width (mm)		
	≤100mm	>100mm and ≤300mm	>300mm
T1	-1, +3	-2, +4	-3, +5
T2	-1, +1	-1.5, +1.5	-2, +2

Table 3-2: Dimensional tolerances by tolerance class, adapted from Table 1 and 2 BS EN 336 (2013).

During initial investigations for the production of a home grown British DLT panel, specification for the length of the laminations and hence the panel is limited to the naturally available resource lengths. The justification for this selection process was to reduce the need for further manufacturing processes that could complicate production such as finger-jointing.

To enable consistent resource provision, cost certainty and dimensional accuracy of the timber resource, the standardisation of cross-sectional dimensions to be used throughout all EWP production in the UK should be an aspiration. Following studies undertaken into the viability of the UK resource for use in CLT (Crawford *et al.* 2015), it is proposed that the standard lamination thicknesses for DLT should be 20mm, 30mm and 40mm to align with European manufacturing capabilities and previous research studies undertaken by COCIS.

3.1.4. Availability and supply of hardwood dowels

Cylindrical hardwood dowels were traditionally formed from cleft sections of timber material that were set through a die or worked with a drawknife to form the required section shape. In the UK, there is a small precedence for using cleft oak dowels in traditional post and beam framing within the UK (Shanks and Walker, 2005) but there are no significant existing supply chains that deal with the processing of various types of hardwood into the size of dowel material required. Nowadays, it is common to manufacture dowels on a lathe from square 'blanks' that have been produced from sawmilling procedures. Using a lathe in this instance removes some of the quality control that cleft production provided and as such the dowels need close visual inspection to ensure quality is maintained (Shanks and Walker, 2005).

A preliminary study was conducted by COCIS in 2013 and 2015 to ascertain the availability of the hardwood material for dowels and their manufacturing capability in the UK. A selection of prismatic hardwood blanks was sourced from Scotland that included, ash (Fraxinus excelsior), beech (Fagus sylvatica), Silver birch (Betula pendula), sycamore (Acer pseudoplatanus) and scots pine (Pinus sylvestris). The prismatic blanks were dried to approximately 6% moisture content and then shaped using a jig and a spindle moulder (see Figure 3-3 and Figure 3-4) into 20.5mm diameter cylindrical sections.

Figure 3-3: Manufacture of UK hardwood dowel using spindle-moulder

Figure 3-4: UK hardwood dowel prior to trimming.

Geometric measurements taken from the dowels produced in this manner indicated that the tolerance required (i.e. ±1mm) was not achieved. On average, the dowels produced by this method had a tolerance of ±10% from the target diameter of 20.5mm (Turnbull, 2013). The prismatic blank was manually inserted into the spindle moulder in two stages, one half at a time. Each half did not always align correctly, creating a non-uniform cylindrical cross-section along its length. The report concluded that the variation witnessed between the properties of the dowels produced meant that this method of producing hardwood dowels was not viable (Turnbull, 2013).

The required quantity of hardwood dowels is much lower than the volumes required for the softwood elements of the panel therefore the rationale for the selection of the dowel material is less reliant on the location of the material and the existence of a supply chain. During the study, repeated attempts were made to acquire a volume of hardwood dowels from UK home grown sources but no such supplier was found.

3.1.5. Target moisture content

In the UK, there are concerns that the dimensional stability of UK grown Sitka spruce and larch (due to its relatively high density) during drying, could impede their use within higher value processes (Crawford *et al.,* 2015; Moore *et al.,* 2008). A key issue in the supply of UK timber is the ability to provide the timber for EWP production at the required moisture content and in significant quantities**.** Currently, timber in the UK is usually dried to a target moisture content value of 18% ± 2% (Crawford *et al.,* 2015). This limits decay occurring, reduces the amount of distortion when in service, reduces the weight during transportation and allows for easier machining to occur. EWPs are almost always produced from timber that is kiln dried to a moisture content of 12±2%. Maintaining a lower moisture content is important in EWPs. Firstly, it allows for the easier bonding of the constituent parts and it also provides greater stiffness and strength reliability and finally, it also limits distortion occurring after installation.

Unlike the softwood laminations of the panel there is a need to dry the hardwood dowels further past the in-service moisture equilibrium (to approximately 6-8% moisture content) so that the dowels will swell as they reach equilibrium with the surrounding atmosphere and effectively fuse the softwood laminations effectively. The shaping of the hardwood dowel blanks into the correct cross section shape will almost always occur when the timber is at a higher moisture content than the expected service class and hence the moisture content at the point of insertion into the panel. This not only facilitates easier working of the material but ensures a longer working life of the machining tools. To produce dowels at a lower moisture content than the expected in-service moisture content creates several foreseeable problems. Firstly, it is not always feasible to create, at an economical cost a workshop environment with the correct level of moisture and humidity to maintain the dowels at the required moisture content. Secondly, if further drying of the dowels is needed to achieve the insertion moisture content the dimensional tolerance of the dowel will be difficult to achieve as variations in the diameter will occur as the timber dries and shrinks in a non-uniform fashion. Assurances that the swelling of the dowels will fuse the panel together cannot be guaranteed. To counteract this, dowels are typically created and supplied at the same or slightly greater diameter than the predrilled holes, this ensures when the dowels are inserted a tight fit is achieved. Any swelling of the dowels that occurs subsequently is beneficial and not essential to fuse the panel together.

Calculations for the connection strength conducted in accordance with BS EN 1995-1-1 (2004), discussed later in section 5.1, conservatively negate the influence of friction at all times (using metallic dowels). In a DLT panel however, the friction force applied by the swelling of the dowel actively helps the formation of the panel and should not be initially neglected. The overall operation of a DLT panel may depend greatly on the initial adhesion provided by the dowels to form a panel with uniform characteristics. Whilst it can be reasonably assumed that the friction force applied (or a portion of this force) will be maintained whilst it is in service, this cannot be guaranteed with 100% confidence, for two principal reasons. During installation the product may experience changes in the moisture content and over time the fibres in the dowels and in the lamination surrounding the fixing may relax reducing the friction. Therefore, whilst the friction force is important during the formation of a panel and will provide a beneficial action for the creation of a composite section it cannot be relied upon throughout the entire service life of the panel, because of the moisture variations and relaxation such as creep that could occur throughout the lifespan.

3.1.6. Distortion of timber

Timber is hygroscopic and will strive to obtain a moisture equilibrium with the surrounding environment. The time it takes for a timber to reach equilibrium moisture content is dependent on the surface to volume ratio, temperature of the surrounding air, its movement and its relative humidity (Moore & Cown, 2015). The shrinkage and swelling of timber, much like its mechanical properties, is anisotropic across the three main planes of timber: radial, tangential and longitudinal to the grain and can distort a piece of timber as bound water is lost from its cells. With changing moisture content, the anisotropic properties of timber create the occurrence of differential shrinkage about the main timber axes. The distortion of wood becomes a significant issue when drying a piece of timber far beyond the fibre saturation point, as is required in DLT or EWP production. The key areas of concern when considering timber distortion relative to DLT production are; bow, spring, twist and cup, shown for reference in Figure 3-5.

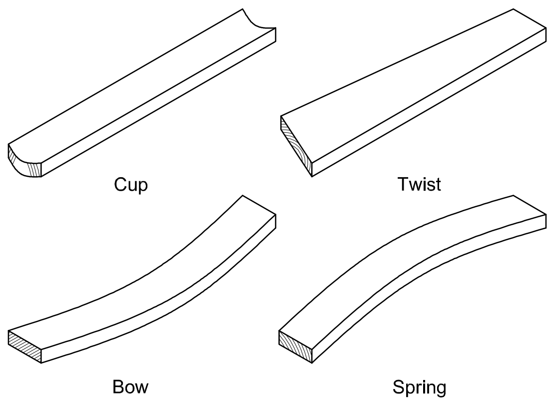

Figure 3-5: Common distortion types in dried timber

Due to the orientation and the composition of microfibrils within the timber, the transverse shrinkage is many times greater than what is witnessed longitudinally (i.e. along the grain). The shrinkage that occurs within the transverse plane is approximately two times greater in the tangential direction than the radial direction because ray cells within the timber restrain radial movement (Dinwoodie, 2000). The rate of movement of a piece of timber during drying is considered proportional to density and the coefficient of volumetric movement or hygrometric expansion (βv). Hoffmeyer (1995) states for practical purposes that shrinkage due to the reduction of moisture content below the fibre saturation point can be considered linear. A very simple approximation of the dimensional alterations that occur below fibre saturation point is given in BS EN 336 (2013) where a piece of timber's dimension changes linearly across the grain by 0.25% every 1% of moisture content lower than 20%.

Lengthwise distortion can often be caused by the presence of knots, juvenile wood or compression wood (Moore and Cown, 2015). Cup that occurs across the width of the section is often the result of different movement rates between the two transverse grain planes, radial and tangential. These deformations in the timber can and need to be limited as much as possible throughout the drying process as the requirements for timber laminations within a DLT and other EWP production processes are needed to be many factors smaller than the values given in BS EN 14081-1 (2011) presented here in Table 3-3.

<table>
<tr><th colspan="2">Strength class according to EN 338</th><th>C18 and below</th><th>Above C18</th></tr>
<tr><td rowspan="4">Maximum allowed warp in mm over 2 m of length</td><td>Bow</td><td>20mm</td><td>10mm</td></tr>
<tr><td>Spring</td><td>12mm</td><td>8mm</td></tr>
<tr><td>Twist</td><td>2mm/25mm width</td><td>1mm/25mm width</td></tr>
<tr><td>Cup</td><td>Unrestricted</td><td>Unrestricted</td></tr>
<tr><td colspan="2">Wane</td><td colspan="2">Wane shall not be greater than one third of the full edge and/or face dimensions of the piece</td></tr>
</table>

Table 3-3: Visual override requirements adapted from Table 1 BS EN 14081-1 (2011).

3.1.7. Characteristic properties of timber

The physical and mechanical properties of a piece of timber are dependent on a variety of parameters that include the type of species, silviculture, site location, moisture content, duration of load and the presence of defects within the timber (Moore *et al.,* 2008). To predict the mechanical properties of a piece of timber, the grading process non-destructively defines upper bounds of three properties strength, stiffness and density that determines the grade, through mechanical testing to BS EN 408 (2010). All other characteristic properties of a piece of timber can then be calculated from these three indicative properties, as they are sufficiently correlated or linked (Glos, 1995; Moore and Cown, 2015). To obtain a degree of certainty in the mechanical properties of a piece of timber each is assigned a strength classification. This classification is dependent on the characteristic material properties of the piece of timber in question. Within BS

EN 384 (2010), the characteristic values for timber are defined as a weighted lower 5th percentile value of a population.

Characteristic values of strength and stiffness for a particular softwood or hardwood grade are provided in BS EN 338 (2009). This is achieved through either visual or machine grading in accordance with the requirements set out in BS EN 14081-1 (2011). Visual grading is considered to be fairly subjective, as the grader has to firstly identify any strength reducing characteristics and measure them accordingly. Since there are numerous characteristics to be measured often within tight time constraints a lesser grade is often specified to a sample than what may be achieved by machine grading (Ong, 2015). Machine grading can be conducted using a bending grader, a dynamic (acoustic or vibration) grader, a radiation grader such as X-ray or combinations of each.

3.1.8. Strength and stiffness of UK timber

A good body of research was conducted several years ago into the mechanical properties of Sitka spruce grown within the UK. The results of these studies are packaged in a research report from the Forestry Commission entitled '*Wood properties and uses of Sitka Spruce in Britain'* (Moore, 2011). Most of the research compiled relates to small clear section properties and did not include the influence of macrostructure defects that can occur in a full-sized specimen. Furthermore, any full-sized results that are presented within the report are typically given as mean values and not the characteristic values needed to allow for design derived through the established testing procedures detailed in BS EN 408 (2010) and EN 384 (2010). UK Sitka spruce commonly does not meet the requirements for C24 timber due to its insufficient stiffness and barely meets the stiffness parameter for C16 timber (Moore *et al.,* 2009b).

UK larch had previously not been used to any great extent in structural uses and research into the structural use of UK larch is lacking. No compendium currently exists for the properties of UK larch beyond what was stated in Lavers (1967), and only visual and bending grading methods currently exist (Ridley-

Ellis *et al.,* 2015). Research has therefore been set in motion to address this imbalance partly due to the sudden need to harvest larch to stop the spread of Ramorum disease and therefore create new scenarios for its use.

3.1.9. Acoustic appraisal of the mechanical properties of timber.

Acoustic Grading has become more popular within the UK in recent years (Moore *et al*., 2009a; 2009b; Auty and Achim, 2008) as it provides a method of ascertaining an indicative property of the chosen material (most commonly the modulus of elasticity) without a lengthy training course needed for visual grading qualification (Moore & Cown, 2015). The use of portable acoustic graders is of interest in the development of EWPs using a UK home grown resource for a couple of reasons. Firstly, it allows SME's to source UK timber outside current supply chains and provide them with a structural grade classification prior to the manufacture of an EWP and secondly it removes the need of housing facilities for the larger mechanical graders. Moreover, acoustic structural grading can be conducted on section sizes and volumes of timber that are not normally suitable for automated or mechanical graded systems.

3.1.9.1. Dynamic Modulus of Elasticity

The acoustic grading instruments measure the speed of transmission of a disturbance through a material caused by an external excitation (commonly a hammer blow). The speed of the induced stress wave is related to the stiffness and density of the material via a one-dimensional wave relationship, expressed as:

$$E_d = \rho V^2 \tag{3.1}$$

E_d Dynamic Modulus of Elasticity, N/mm^2

ρ Density, kg/m^3

V Stress wave velocity, ms^{-1}

Here the dynamic stiffness or modulus of elasticity of the timber E_d is calculated. This stiffness is derived from a load that is rapidly changing and is not immediately compatible with the static modulus of elasticity obtained from a mechanical strength test. Nevertheless, it has been shown when an accurate assessment of the density is obtained reliable correlations can be drawn against the static modulus attained from a monotonic application of load and the dynamic load determined from resonance tests (Jones and Emms, 2010).

3.1.9.2. Stress wave velocity

Most acoustic graders that are used post timber conversion such as the Brookhuis Micro-Electronics MTG 960 (Figure 3-6, hereafter known as MTG 960) or the Fibre-Gen Hitman HM200 are resonance devices.

Figure 3-6: Brookhuis Micro-Electronics MTG 960 acoustic grader.

Resonance devices measure the fundamental frequency and its associated harmonics to determine the resonant velocity, through the equation:

$$V = \frac{2f_iL}{i} \tag{3.2}$$

Where:

V Stress wave velocity, ms^{-1}

f_i Frequency at *i*th harmonic, Hz

L Length of specimen, mm

i Harmonic number

Similar to the process for mechanical grading systems, grade settings must necessarily to be applied for all types of commercially available acoustic based instruments. It should be noted that different acoustic tools may have differing methods and algorithms for determining velocity and time of flight and each instrument will need to be ratified individually for the species in question (Wang, 2013).

4. Literature Review - Part 3: Properties of laminations

The two primary mechanisms that define the operational behaviour of a DLT panel are the strength and stiffness of the laminations in the longitudinal direction and the transverse stiffness of the fasteners between the laminations. In this section, the focus of an extensive literature review has been centred on the behaviour of a single lamination spanning in one direction and the behaviour of several laminations acting in unison with one another as a laminated composite EWP.

4.1.1. Limit states

Timber design has, with the aid of significant research, moved towards a theoretical analysis that has been calibrated with probability to ensure on going validation and reliability. By creating two limit states that the structure will no longer perform above, numerous structural issues can be investigated separately. These are known as the Ultimate Limit State and the Serviceability Limit State.

The Ultimate Limit State (ULS) deals with the safety of the structure under loading throughout its design life. The design conditions for a structure are defined by EC0 (BS EN 1990, 2002) and relate to four different design circumstances based on differing actions and environmental factors that act on a structure. The three main scenarios that must be considered in the UK are persistent, transient and accidental loading. Loads or actions are then considered in combination in favourable and unfavourable ways; partially factored in relation to the ULS that is being verified. Actions or loads on a structure are defined by representative values termed characteristic values; typically, permanent and/or variable actions. The value of the variable actions can by reduced by partial reduction factors ($\psi_{i,i}$), relating to their time dependent nature and the likelihood of acting unfavourably in unison. Typical combinations are given in the National Annex to BS EN 1990, Table NA. A1.1 (NA to BS EN 1990, 2002).

Whilst the safety of a structure is ensured through its compliance with the ULS, there are often times when the performance of a structure is deemed unacceptable for other reasons. Therefore, in addition to the aforementioned ULS the Serviceability Limit State (SLS) must be considered for all elements of a timber structure. The SLS is concerned with the correct functioning of the structure with regards to the comfort of the user, its visual appearance and it also provides a limit to avoid damage to non-structural elements. Failures of the serviceability criteria are far more widespread in timber structures than failure according to the ULS. Whilst these failures do not immediately pose a danger to life, they should not be underestimated and they are of key importance to the correct operation of a structure (Thelandersson, 1995). Many of the imposed limits of serviceability are subjective and should be clarified with the users/client before designing the structure. Whilst there is no formal agreement on the limits in EC5, guidance is given in the UK NA to BS EN 1995-1-1 (2004) for the limits of deflection for individual beams (Table NA.5) and vibrations in residential floors (Table NA.6).

4.1.2. Service Class

The operation of timber is affected by the conditions it is exposed to over the design life of the structure. In order to take account of these variable environmental effects, service classes are provided that detail boundaries for the environmental conditions that the timber will operate in over its design span. The assigned service class then defines the partial reduction factors that are necessary in its design and specification; the service class boundaries defined in clause 2.3.1.3, BS EN 1995-1-1 (2004) are as follows:

Service Class 1 (SC1): Average moisture content of the timber will not exceed 12% over its design life. Relates to a temperature of 20°c and a relative humidity of below 65% for all but a few weeks a year.

Service Class 2 (SC2): Average moisture content of the timber will not exceed 20% over its design life. Relates to a temperature of 20°c and a relative humidity of below 85% for all but a few weeks a year.

Service Class 3 (SC3): Relates to conditions that result in a higher moisture content than those shown in Service Class 2. This normally applies to timber that is exposed externally.

Table NA.2 of the UK National Annex to Eurocode 5 (NA to BS EN 1995-1-1, 2004), defines members that are fully internal as service class 1 and members that sit within construction layers that bridge between the internal and external environment as service class 2. In order to refine and simplify the analysis of the timber within a DLT floor panel, it is necessary to define its expected service class at an early stage. Although DLT panels have been used in a variety of service classes, this PhD simply considers the introduction of the panel within an internal environment. Therefore, in this instance throughout its design life the panel will function in Service Class 1 conditions and its operation in other service classes will therefore be omitted from the remainder of this research study.

4.1.3. Traditional timber floors design

Timber floors are utilised within the construction of all types of domestic properties in the UK. They comprise beams or joists spanning between at least two points of support at regularly spaced centres with at least an engineered timber board material, such as fibreboard, fastened to the top edge of the beams using metal dowel-type fasteners. Timber floor decks (as shown in Figure 4-1) act as a complex two-way spanning structural system, with partial composite action arising from the combination of the fasteners and sheathing continuities. For simplicity, the requirements set out in BS EN 1995-1-1 (2004) mean that the in-service stresses in a timber floor should be determined primarily using first order linear elastic theory based on a one-way spanning structure (Wheat, Vanderblitz & Goodman, 1983). The theoretical deflection of a section then can be derived from the differential equation of symmetrical bending based on the bending action of stresses and from shear stresses.

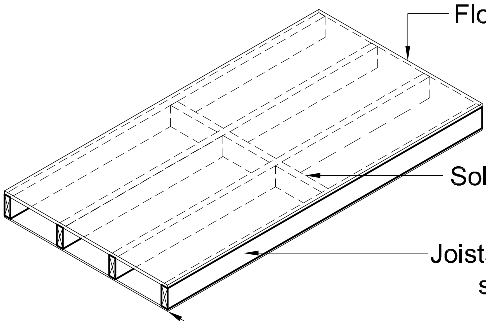

Figure 4-1: Typical suspended timber floor

4.1.4. Bending strength

Using the elastic theory of bending, the stress in a section can be calculated and then be equated to the strength of the section at that location on the basis of the design rules given in EC5 (BS EN 1995-1-1, 2004). Section 6 of BS EN 1995-1-1 (2004) provides the necessary expressions for validation at the ULS for a beam that is subjected to stresses in the direction of one of its principal axes. When a beam is subjected to flexure the design strength of the section will be based on a factored characteristic bending strength and any lateral torsional instability, that occurs during bending when the compression face of the member is not adequately restrained against lateral movement. The characteristic material strength is reduced by several partial factors that are dependent on the limit state being considered, the duration of load, the in-service conditions and the necessary reliability. According to BS EN 1995-1-1 (2004) the following bending conditions should be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.1}$$

Equation 6.11 (BS EN 1995-1-1, 2004)

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.2}$$

Equation 6.12 (BS EN 1995-1-1, 2004)

Where:

$\sigma_{m,y,d}$ & $\sigma_{m,z,d}$	Bending stresses about the two principal axes.
$f_{m,y,d}$ & $f_{m,z,d}$	Design bending strength about the two principal axes.
k_m	Modification factor that take into account the redistribution of stress and the variation of the material across the cross section. *For solid timber, Glulam and LVL the value of k_m can be taken as 0.7 for rectangular sections and 1.0 for all other sections.*

The design bending strength is defined by the factored characteristic strength of the section. For a DLT panel not subjected to lateral instability the design bending strength about the y-axis can be written as:

$$f_{m,y,d} = \frac{k_{mod} k_h k_{sys} f_{m,k}}{\gamma_M} \tag{4.3}$$

Where:

k_{mod}	Modification factor for the effect of moisture content and load duration. This is a composite factor of the load duration and moisture content effects on the strength of a connection due to the in-service operation based on the experience in practice of the interaction of both factors (Smith and Foliente, 2002) and is tabulated in Table 3.1, BS EN 1995-1-1 (2004).
k_h	Size effect factor, dependent on member size in relation to a reference section.
k_{sys}	Additional system strength factor allowed when a series of similarly spaced members sufficiently connected together allows the distribution of load between them, discussed further below (discussed in section 4.1.4.1).
Υ_M	Partial factor for the material, incorporating uncertainty in geometric, material properties and uncertainty in model assumptions. At the ULS the partial factor for material and resistance properties is given in the UK NA to BS EN 1995-1-1 in Table NA.3 (NA to BS EN 1995-1-1, 2004). For solid timber, this should be given as 1.3.
$f_{m,k}$	Characteristic bending strength of timber, N/mm^2. Normally it is depicted in BS EN 338 (2009) and is discussed in 3.1.7.

When a panel experiences combined compression and bending stresses such as in wall panels (constructed in the vertical plane) it would need to be verified for that combined condition. This is because there is a possibility that there will be a reduction of its bending strength due to lateral torsional instability. If DLT panels are to be used as structural walling panels then validation of the combined axial and bending condition given in section 6.3.3, BS EN 1995-1-1 (2004) is required. This check is omitted here because when the DLT panel is arranged as a floor panel it will be fully restrained along its length and will only potentially undergo combined bending and compression forces whilst transferring racking forces in the very short term. It can be assumed that any floor sheathing would transfer the load.

4.1.4.1. System strength Factor

When a continuous load distribution system is used to laterally fix a series of equally spaced supporting members BS EN 1995-1-1 (2004) allows for multiplication of member strength properties by a factor, k_{sys}. The assumption here is that the neighbouring members of greater stiffness will preferentially share the load from any weaker members and thereby increase the system strength beyond that of a single member. Typically, the value given for k_{sys} is 1.1, (i.e. a 10% increase in strength properties) but BS EN 1995-1-1 (2004) provides greater allowances for laminated timber decks or floors depending on the method of fixing, see Figure 4-2.

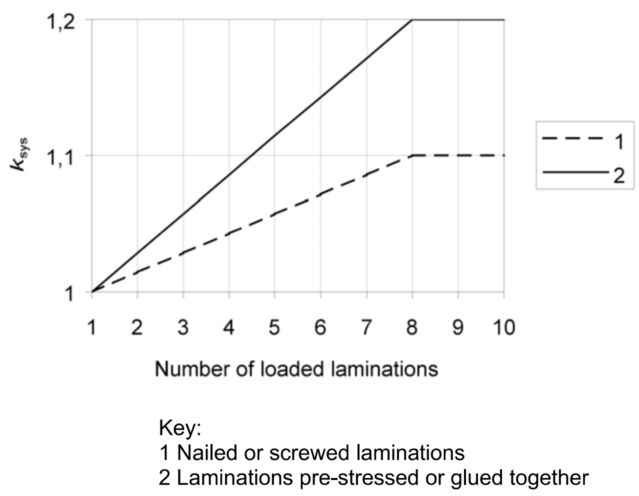

Figure 4-2: System strength factor, k_{sys} (Figure 6.12, BS EN 1995-1-1, 2004).

BS EN 1995-1-1 (2004) provides additional allowances for the system factor in floor plates comprising laminated members but no allowances are made for a panel such as DLT that is joined in a method that is not anticipated by BS EN 1995-1-1 (2004). Steel dowels are omitted as they are normally inserted with a defined tolerance of ± 1mm for dowels under 18mm. Where the strength verification of a new system is necessary there is a requirement within BS EN 1995-1-1 (2004) clause 6.6 for it to be confirmed using short-term durations of load and this will need to verified for DLT panels going forward as it is produced firstly, from dowels and secondly, from non-metallic fasteners.

4.1.5. Beams subjected to shear and bearing

To comply with the expressions given in BS EN 1995-1-1 (2004), a structural element made of timber must also comply with the design criteria given for the stress states in which shear and bearing will occur. Elastic bending theory assumes that shear stresses will be created throughout the cross-section. The magnitude and distribution will depend on the loads applied and the bending induced. Due to a piece of timber's relative lack of strength and stiffness in the perpendicular to grain orientation the stress in compression perpendicular to the grain will need to be verified by the designer at points of support. Whilst the analysis and verification of the shear and bearing stresses is necessary in all instances of timber design it will be excluded here due to the relatively small amounts of global shear and bearing stresses that will be induced in the DLT floor panel under load and up until failure. The assumption is that there will be a long continuous support condition in at least two ends of a panel and the amount of shear generated at the end of a panel under normal domestic loading is not normally considered critical, across the full width of a panel. Under abnormal loads or point loads this check will be required.

4.1.6. Deformation due to bending and shear

Compliance with the SLS is related to both the final deflection of the panel and any vibration that is induced by applied loads. From the principle of elastic bending theory, the maximum bending deflection of a beam under instantaneous loading has been developed for a multitude of differing load cases and can be found in any good textbook on the matter. In order to define the final net deformation, w_{fin} of a floor system a simplified process has been defined in BS EN 1995-1-1 (2004), whereby a characteristic combination of actions is surmised for instantaneous deflections and the final deformation, see Figure 4-3. Deflection is defined by a quasi-permanent combination of the characteristic actions using mean values of the appropriate moduli and materials with the same creep behaviour.

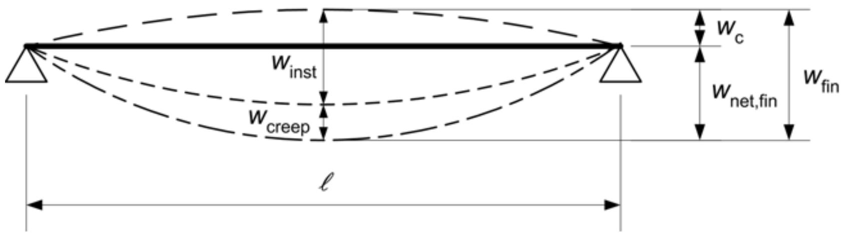

Figure 4-3: Components of deflection, Figure 7.1, BS EN 1995-1-1 (2004).

The final net deformation of a beam should be taken as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{4.4}$$

Equation 7.2 (BS EN 1995-1-1, 2004)

Where:

w_c	Precamber, mm
w_{inst}	Instantaneous deflection, mm
w_{creep}	Creep deflection, mm
w_{fin}	Final deflection, mm
$w_{net,fin}$	Net final deflection, mm

However, when bending is also induced by shear forces a beam will additionally deform by an amount related to the ratio between the Modulus of Elasticity parallel to grain ($E_{0,mean}$) and the Shear Modulus (G_{mean}). For industrial construction materials, the ratio between the two is normally low (i.e. in the region of 2 for steel) and will be such that it can be ignored for simplicity. For timber, the ratio between $E_{0,mean}$ and G_{mean} is thought to be in the region of 16 (EN 338, 2009; EN 384, 2010) and induced shear deflection can in some instances be significant (Thelandersson, 1995). In lower quality timber materials, the G_{mean} is enhanced by the presence of knots, the ratio of $E_{0,mean}$ to G_{mean}can be somewhere in the region of 25 to 30 (Chui, 1991) or no correlation can exist at all (Khokar, Zhang & Ridley-Ellis, 2010). The deformation arising from both bending and shear can be calculated from the value of deflection caused by bending amplified by a shear factor based on the load, cross-section dimension, ratio of Elastic and Shear Moduli and the support conditions of the beam and can be found in any good structural engineering textbook.

4.1.7. Vibration

The vibration of a structure or floor can have a great effect on its correct operation and can bring large levels of discomfort for the user over its design life if not properly considered. The compliance requirements for vibration given in BS EN 1995-1-1 (2004) only relate to residential flooring with a fundamental frequency greater than 8Hz. All other scenarios that are either below 8Hz or not for residential uses need to be investigated individually for compliance.

Traditional wooden floor structures comprising of solid joists at regular centres are a great deal stiffer in the longitudinal direction than in the transverse direction, due to the orientation of the grain. Therefore, their performance when subjected to human induced vibrations is relatively poor for long spans (Olsson, Jarnerö & Källsner, 2008). Traditional timber floor construction used in residential buildings does not suffer overly from uncomfortable levels of vibration, however more recent approaches to timber floor installation (such as using timber I Joists at regular centres) has aimed to minimise the weight of the floors for ease of installation and maximise structural efficiency. The benefits namely a reduction in material used has caused greater instances of user dissatisfaction relating to vibration of a floor structure due to the installation of lightweight floor decks (Smith, 2003).

To improve the vibration of a timber floor there are three main solutions: increase the mass of the floor; enhance the transverse stiffness and improve damping (Olsson, Jarnerö & Källsner, 2008). The provision of a solid engineered timber floor deck such as DLT increases the mass of the floor, it has the potential to improve the transverse stiffness and it also reduces the damping ratio as a result of its increased dead weight (Weckendorf *et al.,* 2008). This is therefore a significant advantage for DLT systems over timber joists at close centres, when spanning greater distances.

Whilst compliance with the vibration criteria given in the BS EN 1995-1-1 (2004) and its UK NA is mandatory for any timber floor deck in a residential setting, vibration as a research topic in this thesis has been omitted for two main reasons. Firstly, the existing verification criteria specified can be used directly on a DLT floor panel as the criteria relate to its stiffness and weight. In addition, any further values obtained through testing in this PhD can be incorporated back into the design formulae where applicable. Secondly, there have additionally been many disputes over the boundaries of these criteria over the years (Porteous and Kermani, 2013) and an in-depth analysis of the varying vibration conditions may not be pertinent to future designs.

4.2. Verification and design of DLT

A lack of design guidance and improved evaluation of DLT products was highlighted by responders of the questionnaire in section 2.3.5 and 2.3.6. Without a unified and formalised approach to design any specification will be based on possibly inefficient means of analysis and will not be easily specified by industry professionals. Despite the precedence of DLT in Europe (at least on a small scale) over the last 20 years, the amount of research undertaken on it specifically has been very limited. Research into timber composite panels constructed using mechanical fixings has not looked directly at the inclusion of hardwood timber dowels to create a plate structure or an enhanced EWP. Instead it has focused on metal dowel-type fasteners or adhesive connections being used in EWPs.

In order to understand how a DLT panel behaves it is necessary to ascertain the characteristics of each system within the panel and what behaviours the panel exhibits when loaded. It is not known whether or not the panel itself acts as a series of individual beams or as a fully composite plate structure. CLT is an EWP that is already widely used as entire structural subsystem, and whilst there are clear differences between CLT and DLT in the arrangement and the joining of laminations it can provide a benchmark for how analysis and design of a composite timber subsystem is engineered, designed, verified and specified. In the following section, the design considerations for CLT are discussed in relation to DLT. Furthermore, a brief description of the varying techniques for analysing an EWP such as CLT is carried out and their applicability to DLT is considered.

Verification of CLT panels is currently based upon a hybrid approach of engineering principle and mechanical testing based on published papers, published guidance documents such as the Canadian CLT handbook (Gagnon and Pirvu, 2011), manufacturers recommendations, European technical approvals and a recent harmonised standard (BS EN 16351, 2015). The mechanical properties of the panels can be derived in one of two ways. One by determining the properties of the single layers and combining them into a composite product or two determining and evaluating the properties of the entire panel through full-scale testing of the whole CLT element (Harris, 2015).

Often the simplest approach to everyday design situations is to negate a complex analysis of each individual layer and its fixing and assume compatibility in curvature and displacement to classical beam theory. A commonality between many of these methods developed for the analysis of mechanical jointed sections is the adherence to the already established method of strength and stiffness verification given in BS EN 1995-1-1 (2004). The aim throughout many of these methods is to reduce or transform the selected panels into a one-directional spanning system that has effective section properties depending on the main material used (i.e. timber). This can be achieved throughout the section by incorporating the fastener resistance at the interface as a series of linear springs into the model and applying classical beam theory to the other components of the jointed element (Ceccotti, 2003). The techniques also examine the fixation between the section elements, the support conditions and the loading arrangement. This effective section can then be directly substituted with the section properties determined from linear elastic mechanics and can be verified using the flexural analysis to the ULS and SLS depicted previously in sections 4.1.5 and 4.1.6.

For beams or panels with multi-layered slip characteristics, four prominent methods for modelling can be used; mechanically jointed beam theory, composite theory (also known as the ϒ-method), the shear analogy method and the finite element method. No general consensus has been reached on which method is the most suited for differing instances and it has been shown that the method of analysis selected can alter the estimated strength and stiffness by as much as 30% (Natterer and Weinand, 2008). Therefore, it is important in this study to define the methods of analysis undergoing research and currently available and being trialled in literature to decide which methods are most appropriate for the estimation of the strength and stiffness of a DLT panel.

4.2.1. Mechanically jointed beam theory: Annex B (EC5)

Developed in the 1950s (Natterer and Weinand, 2008), this method is only applicable when a section is created from pieces that are joined using mechanical fasteners spaced a constant distance 's' along the length of the beam (or can vary according to the shear force present), with a stiffness 'K' that is dependent on the limit state being considered but can be modified to assist with the analysis of EWPs products. By using the principle of linear elasticity an effective bending stiffness of a mechanically jointed section of differing cross-section can be created. The effective bending stiffness is given by the equation:

$$(EI)_{ef} = \sum_{i=1}^{n}(E_i I_i + \gamma_i E_i A_i a_i^2) \tag{4.5}$$

Equation B.1 (BS EN 1995-1-1, 2004)

Where:

A_i Area of section being considered, mm²

E_i Elastic modulus of the material being considered, N/mm²

I_i Second moment of area of the section being considered, mm⁴

a_i Distance from the centroid of the section being considered and the neutral axis of the entire section, mm

γ_i Fastener efficiency factor given by the equation:

$$\gamma_i = [1 + \pi^2 E_i A_i s_i / (K_i l^2)]^{-1} \quad (4.6)$$

Equation B.5 (BS EN 1995-1-1, 2004)

Where:

s_i Spacing of the fasteners, mm

K_i Slip modulus of the fastener, N/mm

This theory assumes that the loading applied creates a sinusoidal or parabolic moment profile along its length and integration can be used to determine the stiffness (Natterer and Weinand, 2008). The connection efficiency factor, γ_i is dependent on the slip characteristics of the fastener and can be shown to be equal to zero where no connection is present and equal to one where a fully rigid connection is achieved. The effective bending stiffness and the fastener efficiency factor (γ_i) is then used to determine the normal stresses in the compatible cross section that are illustrated in Annex B.1 BS EN 1995-1-1 (2004). An example of a section is repeated here for clarity in Figure 4-4. Through this method the shear deformation is taken into account along the section through the shear deformation of the cross layers.

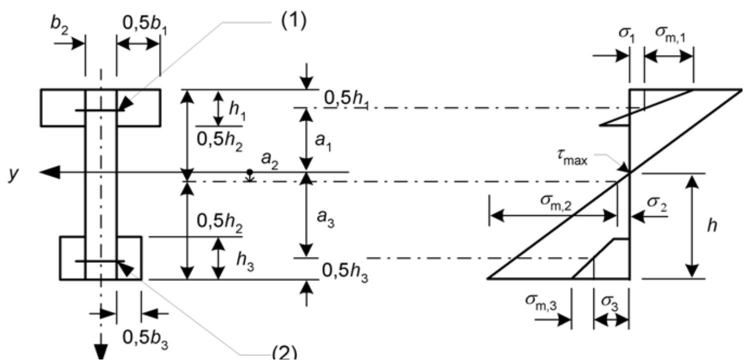

Figure 4-4: An example compatible cross section, Figure B.1, BS EN 1995-1-1 (2004).

Figure B.1 BS EN 1995-1-1 (2004) only illustrates sections that are asymmetrical about the z-axis and only act in unison when the section is bent about the y-axis. Although, if the section is symmetrical the effective bending stiffness of the section can be determined by the process described in section B.2 BS EN 1995-1-1 (2004). By considering two of the laminations of a DLT floor panel being bent around the y-axis and connected by a hardwood dowel the cross section can be postulated to be the same as Figure 4-5.

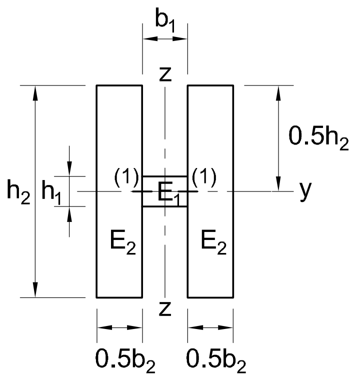

Figure 4-5: Idealised effective section based on a portion of a DLT panel.

An effective bending stiffness of the section about the y-axis $(EI)_{ef,y}$ will be:

$$(EI)_{ef,y} = \sum_{i=1}^{3}(E_i I_i + \gamma_i E_i A_i a_i^2) \tag{4.7}$$

$$(EI)_{ef,y} = \frac{E_1 b_1 h_1^3}{12} + \frac{E_2 b_2 h_2^3}{12} + \gamma_1 E_1 (b_1 h_1)(0^2) \tag{4.8}$$

Adapted from equation B.1, BS EN 1995-1-1 (2004)

Where:

$$\gamma_i = \left[1 + \pi^2 E_1 \frac{A_1 s_1}{2 k_1 l^2}\right]^{-1} \tag{4.9}$$

Adapted from equation B.5, BS EN 1995-1-1 (2004)

These equations assume that the fastener efficiency factor for a three-part section with a fixing through the centroid of the section will be effectively reduced to 0 and no additional composite behaviour can be assumed. However, where the panel itself comprises more than three sections with non-equal slip occurring between them under load it creates a scenario where the curvature created by the moment acting in the y-axis will not be constant and therefore the fastener will not always be in the plane of the neutral axis and some contribution from the fastener interaction however minimal can be expected, see Figure 4-6.

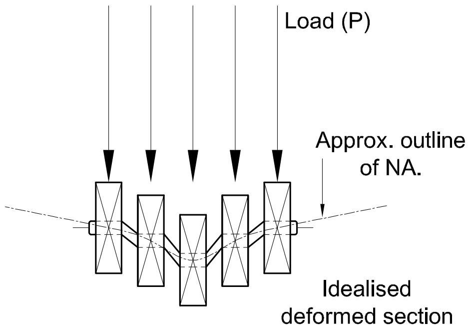

Figure 4-6: Deformed idealised section.

4.2.2. Composite beam theory method – K Method

Composite beam theory works on an equivalent single layer principle, whereby the behaviour of the composite panels is based upon the material and geometric characteristics of the individual layers adjusted by a composition factor. By assuming the material maintains a linear stress-strain relationship and the plane cross-section remains plane under deformation a composition factor can be created. This composition factor is based upon the ratio of the strength or the stiffness of the cross-section being analysed to the strength or stiffness of a fictitious cross-section with the layers orientated so that the grain is set parallel to the direction of the stress (Fellmoser and Blass, 2004), see Figure 4-7

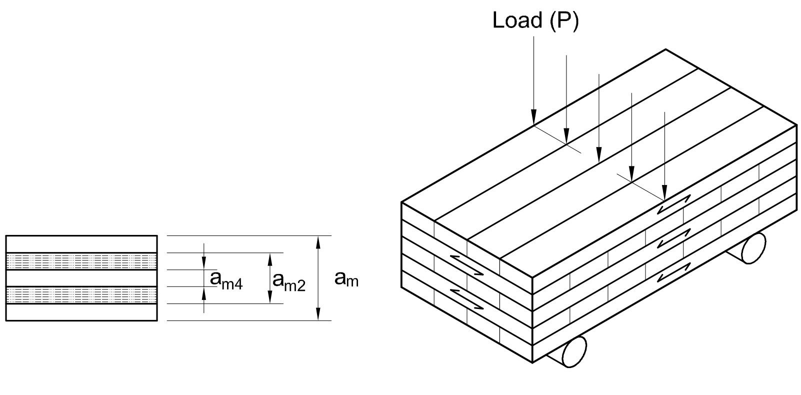

Figure 4-7: Derivation of composition factor perpendicular to the grain for a 5-layered panel, adapted from Fellmoser and Blass (2004).

Using the adjusted effective values of strength and stiffness the structural properties of the panel can be quantified and stress distribution and deformation that occurs across the cross-section can be calculated using linear elastic mechanics. For the normal floor support condition, perpendicular to plane loading the composition factor k_1 is determined using the formula below

$$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right)\frac{{a_{m-2}}^3 - {a_{m-4}}^3 + \cdots \pm {a_c}^3}{{a_m}^3} \qquad (4.10)$$

Where:

E_0 Elastic modulus parallel to the grain, N/mm²

E_{90} Elastic modulus perpendicular to the grain, N/mm²

a_m Height of panel being considered, mm

a_{m-i} Height of panel minus the number of i layers being considered, mm

a_c Height of the central layer (in the instance shown, a_1= a_{m4}), mm

(Fellmoser and Blass, 2004)

However, this method of calculation does not take into account shear deformation that occurs in bending members (Stürzenbecher, Hofstetter & Eberhardsteiner, 2010) and the laminating effect that is created, which could increase the strength and stiffness of a single lamination beyond its characteristic values. In DLT this method would not directly be applicable unless a fictitious cross layer element is placed in the centre of the panel, which is based on the stiffness characteristics of the dowel at that location. The formed panel will then comprise a three-layer arrangement with the cross layers represented by the dowels included at regular centres and the top and bottom layers being represented by the stiffness of the laminations as shown in Figure 4-8: Idealised cross-layered DLT panel. Unlike the CLT panel the cross layer is not continuous across the sample and the stiffness of the cross layer will need to factor in the reduced stiffness of this non-continuous layer.

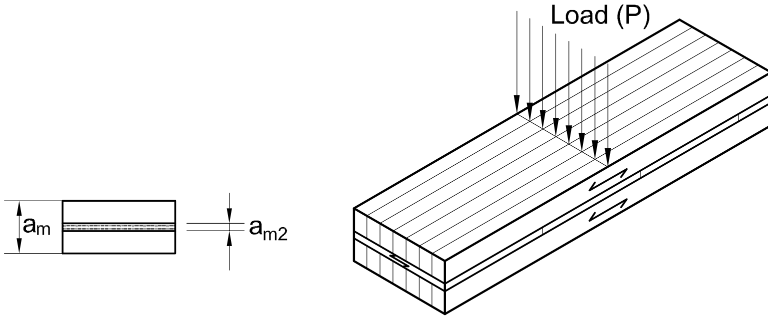

Figure 4-8: Idealised cross-layered DLT panel.

4.2.3. Shear analogy theory and method

Kreuzinger (2001), explored how a multi-layer cross section such as CLT can be analysed by breaking down the characteristics of the cross section into two distinct beams set parallel to one other and coupled with infinitely rigid members to ensure deflection continuity along their length, see Figure 4-9.

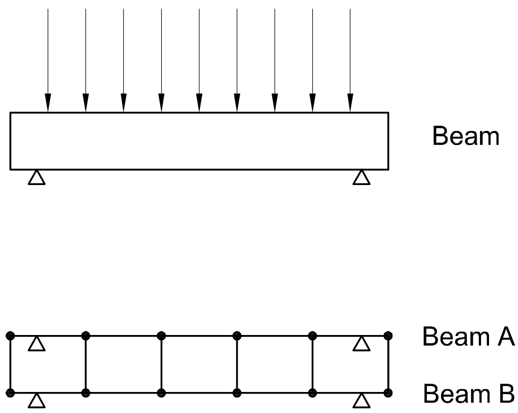

Figure 4-9: Shear analogy grid, adapted from Natterer and Weinand (2008).

The first beam represents the summation of the effective flexural stiffness of the individual layers of the cross section relative to their own neutral axes to form a virtual section from the bending moment and shear force in each layer. The effective bending stiffness can be calculated from the following:

$$(EI_a) = \sum_{i=1}^{n} E_i I_i = \sum_{i=1}^{n} E_i \, b_i \frac{{h_i}^3}{12} \tag{4.11}$$

Where:

EI_a Effective bending stiffness of beam A, N/mm²

b_i Width of each layer, mm

h_i Thickness of each layer, mm

(Gagnon and Pirvu, 2011)

The second beam represents the shear or translational stiffness of the whole panel and the flexibility of the connections between the constituent parts of the panel. The effective bending stiffness of beam B is given by the formula:

$$(EI_b) = \sum_{i=1}^{n} E_i A_i {z_i}^2 \qquad (4.12)$$

Where:

EI_b Effective bending stiffness of beam B, N/mm^2

A_i Area of each layer, mm^2

z_i Distance from the neutral axis to the centroid of each layer (in mm)

(Gagnon and Pirvu, 2011)

The shear stiffness can be represented by the formula:

$$\frac{1}{(GA)_{eff}} = \frac{1}{a^2}\left[\sum_{i=1}^{n-1} \frac{1}{k_i} + \frac{h_1}{2G_1 b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i b_i} + \frac{h_n}{2G_n b_n}\right] \qquad (4.13)$$

Where:

$k_i = \frac{K_i}{s_i}$ Slip between the fasteners (4.14)

G_i Shear modulus of the layer being considered, N/mm^2

(Gagnon and Pirvu, 2011)

By determining the bending and shear stresses in beam A and the normal and axial stresses in beam B. The final stress distribution can be calculated through a process of superposition and the maximum deflection can be surmised as the total contribution from bending and shear. Even though Bernoulli's hypothesis of plane section deformation is not always achieved immediately adjacent to point loads research has shown regardless that this method of analysis is accurate for uniform loads (Fellmoser and Blass, 2004) and point loads (Mestek, Kreuzinger & Winter, 2008). By modelling the fasteners between layers non-linearly the deformation of a screw laminated timber beam was shown to be most accurately modelled by using the shear analogy method and finite element analysis (Natterer and Weinand, 2008). This method could potentially be utilised for the calculation of a DLT provided the correct mechanical properties are used for the material and the correct slip for the connection is included.

4.2.4. Finite element

By creating a composite engineered timber floor system that acts in unison under load and is supported on more than two edges, the support arrangement alters from being a purely one-dimensional consideration to being a two-dimensional problem. Two types of theory, equivalent beam theory and plate theory can be used to calculate the performance of composite timber decks. Plate theories for laminated composites can be broken broadly into two categories, equivalent single layer methods and layer wise methods (Stürzenbecher, Hofstetter & Eberhardsteiner, 2010). Single layer methods focus on the variables, independent of the layers of the panels. The layer wise method takes into account the variables of each layer independently. Whilst the latter method increases accuracy it also increases analytical complexity.

Whilst it is often conservative to design a timber two-way spanning slab as a uni-directional support mechanism, it removes unnecessary complications that can exist with two and three-dimensional modelling and still provides valid results for design purposes (Schickhofer, 2011) but it does not include or exploit the improved performance benefits that two dimensionally supporting EWPs provide. For a product such as DLT where the identified use in this thesis is a domestic floor structure it would be prudent to attempt analysis of the panel based on the easily adaptable method already stipulated in BS EN 1995-1-1 (2004). Appropriate finite element analysis can be developed when the scenarios of its use are increased in complexity.

4.3. Comments

To understand which UK grown timber material is suitable for DLT production it is important to understand the performance requirements for a DLT panel, the complex mechanical interactions that take place within it and how they determine its behaviour under load. Literature on this topic is limited and is often referenced to already accepted methods for designing EWPs in one-dimensional support arrangements, that allow for analysis in accordance with the current timber design standards (BS EN 1995-1-1, 2004).

The type of load applied to the panel affects the verification method being employed. For the purposes of this thesis only out of plane bending is being considered (as discussed in section 4.1.4); axial loads and shear loads in plane more commonly occurs in wall construction and can be experienced in small amounts in the flooring when it acts as a diaphragm to transfer lateral load caused for wind or other forms of environmental loading. The assumptions here is that the out of plane environmental forces will be transferred through the floor deck or sheathing that is provided in typical UK construction and can be omitted in the initial stages of analysis. In plane strength and stiffness provided by the DLT floor panel and its associated boarding will affect the load distribution of the lateral loads in the surrounding walling system but this study aims to analyse solely DLT flooring as a sub-system of the overall structural system and will not look at the global behaviour of the structural system that the DLT floor panel will be a part of.

In future, studies more suited to analysing these characteristics independently (i.e. racking tests) will need to be conducted to ascertain the in-plane stiffness of a DLT diaphragm, and model it effectively. To be commensurate with the current timber design codes for the UK (BS EN 1995-1-1, 2004; PD6693-1, 2012), reduce the complexity of this research project and reduce the amount of testing required the assumption that a DLT floor panel acts as a flexible diaphragm designed to be simply supported between pin jointed walls, that provide the necessary in-plane strength and stability of the structure is deemed appropriate.

In determining the effective section properties of the built-up section or plate it is important to understand the stiffness of the connection between the differing layers or sections for analytical and modelling purposes (Natterer and Weinand, 2008). The four methods of analysis discussed in section 4.2 either create a fictitious layer that represents the stiffness of the cross layer or include a stiffness of the cross layer based on the stiffness of the connection joining the constituent elements.

5. Literature Review – Part 4: Composite Behaviour

Consideration of the strength and stiffness of a panel is often related to an effective section based upon the individual subsystems, acting independently within the panel rather than the product acting in complete unison. These approaches are seen as valid because the margin of error is small in comparison to the reduction in the analytical complexity provided (Schickhofer, 2011). Little, if any research has been undertaken previously on the effective section created from a panel connected that does not use metallic dowel-type fasteners or adhesive. At present, no research has been undertaken into the ability or effectiveness of non-metallic connections to create a connection that allows for composite behaviour to occur within a DLT panel. The research discussed in section 4.2indicates that the joints created between the laminations of a composite laminated EWP could affect its overall behaviour under load and its subsequent method of analysis.

The strength and stiffness characteristics of the connections within a timber structure will normally dictate the strength of the structure, influence the deformation behaviour, affect the internal force distribution and affect the overall durability of the structure and its fire resistance (Smith and Foliente, 2002; Bouchaïr*,* Racher & Bocquet*,* 2007; Chang *et al.,* 2009; Porteous and Kermani, 2013). For a DLT panel, the connections between the laminations are made from non-metallic, typically hardwood fasteners or dowels. The amount of research and literature available for the use of non-metallic fasteners within timber structures is comparatively very small. Analysis of connections made with non-metallic fasteners typically follows the accepted methods employed for metal fasteners. Whilst there is a vast amount of information and research undertaken into the behaviour of metallic connections many of the methods and research employed widely cannot be immediately assumed to correlate with non-metallic fasteners without careful consideration. To provide a background of the current practice for the analysis of non-metallic dowel-type fasteners, a review of the current practice for the analysis and design of timber connections made with metallic dowel type fasteners is undertaken. The most up to date research in the understanding of connections using all timber connections, including the calculation of strength and stiffness is presented.

5.1. Connection strength in accordance with BS EN 1995-1-1

Dowel-type fasteners are by far the most commonly used mechanical fasteners in general timber construction. When subjected to lateral loading dowel-type fasteners transfer the load through the connection via a combination of flexure and shear in the dowel itself and embedment in the timber. It has long been accepted that the strength and stiffness of a timber connection using metal dowel-type fasteners is affected by numerous physical and geometric factors, including but not limited to the species of timber, the number and thickness of members, the direction of load to the grain, the moisture content of the timber, the dowel diameter, its yield stress, the spacing of the fasteners and the tolerance of the fabrication (Soltis and Wilkinson, 1987).

Currently there is no standardised process of calculating the connection strength of an all timber connection. The current timber design suite addresses the strength and stiffness of a timber connection using metallic based fasteners on the basis of its failure mode. For metallic dowel fasteners, the failure strength of a connection is governed by the failure mode with the lowest estimated capacity. Two overall failure modes are considered in the design of a connection, ductile and brittle. Within each failure mode there are different categories of failure that can be witnessed and should be considered separately to obtain the estimated load capacity of a connection.

For ductile failure, an analytical model to determine the strength of a timber connection formed of metal dowel-type fasteners was originally developed by Johansen (1949). This model predicts the strength of two or three symmetrical member joints based on the static equilibrium of a connection and the associated material properties (this model can also be verified using the virtual work approach). Johansen (1949) apportioned the performance of a connection of a timber joint using a dowel-type fastener to two differing functions occurring within the connection, the dowel effect and the tensional effect of the bolt. The dowel effect relates to the resistance of the dowel in bending and the resistance of the timber to crushing. The tensional effect of the bolt depends on the friction between the bolt and the surrounding timber and the constraint at the end of a dowel fastener provided by washer assemblies on bolts or the head of nails (Hilson, 1995). The tension effect of the bolt is commonly called the 'rope effect' and is accounted for within the design codes where the failure mode will involve yielding of the fastener by including an addition of strength based upon a proportional increase of the yield model strength caused by friction and withdrawal effects. Larsen (1979) further modified and improved Johansen's yield model to include connections that comprise timber members with differing embedment strengths. This is achieved by considering a free body diagram of a bolt in a wood member and equating the applied load as equal to the embedment resistance of the member and the bending resistance of the dowel type fastener, see Figure 5-1.

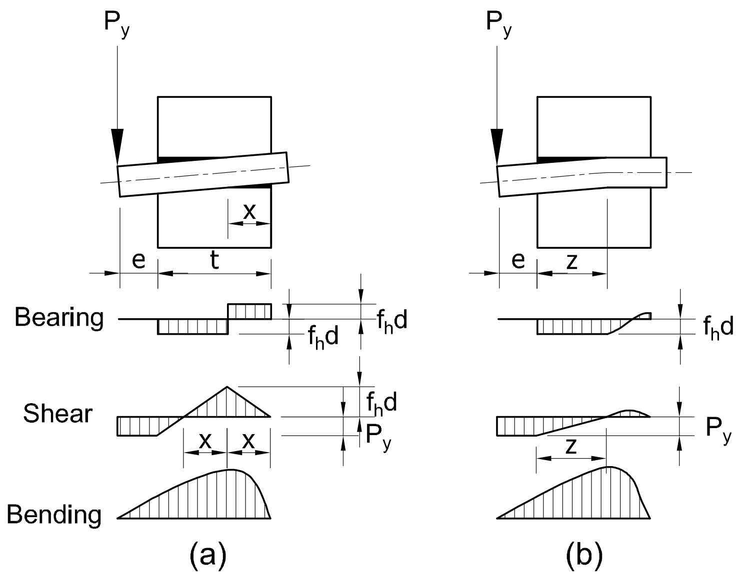

Figure 5-1: General modes of failure of metallic dowels (Larsen, 1979).

In order to formulate expressions for the load carrying capacity of the different failure modes that could occur in a two or three-member connection it was necessary to simplify the analysis. Johansen (1949) achieved this through the provision of material assumptions for the failure behaviour of the dowel and the timber being connected. By considering the embedment strength of the timber as a material property many of the material and geometric variables that occur can be simply included in the analysis through a simple test that determines the embedment strength of the arrangement. Another fundamental assumption is that both materials behave elasto-plastically, see Figure 5-2.

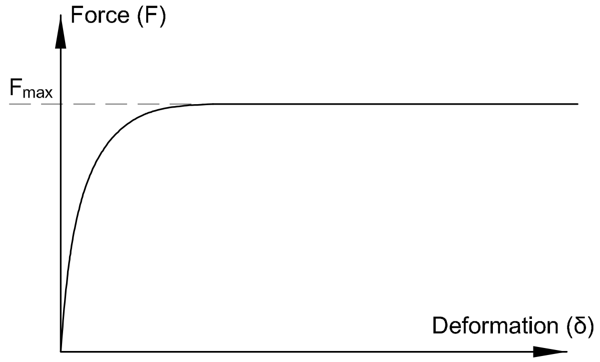

Figure 5-2: Idealised material behaviour.

This presupposes when the yield point of the connection is reached no more stress can be applied (Schmidt and MacKay, 1997). Here the fastener behaves in unison as a rigid plastic material or as a perfectly elastic-plastic material resulting from the deformation of fasteners and crushing deformation of the timber (Racher, 1995). The fastener rotates as a rigid body, Part A, Figure 5-1, or the fasteners rotate when a plastic hinge forms along its length (Smith and Foliente, 2002), Part B Figure 5-1.

From this methodology a countless variety of connection arrangements can be analysed upon the basis of static equilibrium or the principle of virtual work, as long as a certain number of simplifying assumptions can be maintained. These are; the fastener is homogenous, isotropic and elasto-plastic; the wood is homogenous, orthotropic, elasto-plastic in the parallel direction; no friction occurs; shear and tensile stresses in the fasteners do not affect the formation of a plastic hinge; the ends of fasteners are free to rotate and the bearing stress is uniformly distributed under the fastener (McLain and Thangjitham, 1983; Girhammer and Andersson, 1988).

The analytical methodology described by Johansen (1949) and further expanded upon by Larsen (1979) has been commonly accepted within timber design codes across the globe and is often referred to as the European Yield Model (EYM) as it describes how the yielding of a fastener contributes to connection strength (Patton-Mallory, Pellicane & Smith, 1997). The EYM states that the strength of a connection is dependent on the:

- Geometry of the connection
- Embedment strength of the base material
- Bending strength of the fastener

The EYM is only appropriate when the connection will fail in a ductile manner and all design codes that use this methodology have been developed to try to ensure that failure will be in a ductile manner.

5.1.1. European Yield Model double shear plane failure modes

By abiding by the material assumption that the fastener behaves in a rigid plastic or stiff manner there are three different types of failures for a double shear plane connection developed from the general failure modes shown in Figure 5-3 and these are commonly referred to as type 1, 2 and 3 failures. Type 1 is a direct embedment failure that assumes that negligible elastic deformations occur in the fastener itself and failure is in the laminations. Type 2 failure is a combination of embedment failure in the timber and the creation of a single yield point in the dowel. Type 3 failure has a double yield failure in the dowel with a combined embedment failure occurring in the timber. Type 2 and 3 failures are most often witnessed where slender fasteners are used creating a joint with greater ductility and greater potential for plastic hinges to form in the fasteners (Smith *et al.,* 2005). From these three modes, a range of strength expressions can be included for a variety of dowel type connections including double shear plane connections. However, these double shear plane connections are derived from their single shear plane counterparts and are therefore only limited to symmetrical assemblies of connections.

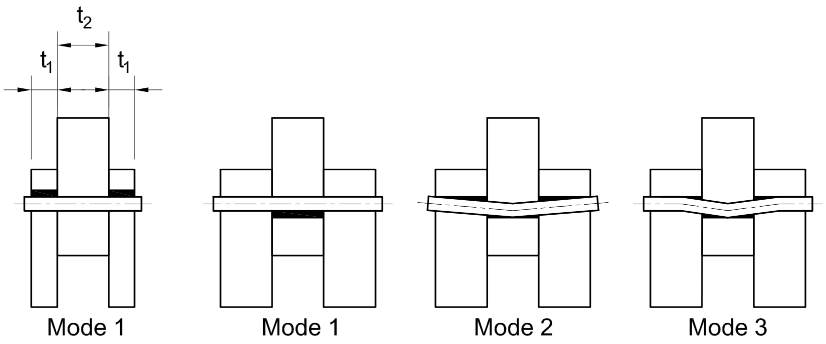

Figure 5-3: Failure modes for a dowel-type fastener in a double shear arrangement according to BS EN 1995-1-1 (2004).

Figure 5-3 illustrates the failure modes that a double shear plane connection would have according to the EYM. Listed below are strength expressions for the varying failure modes for double shear connections derived from the EYM and included in EC5. The characteristic load capacity of a fastener (per shear plane) is taken as the minimum value obtained from the formulas of the three modes.

$$F_{v,Rk} = f_{h,1,k} t_1 d \qquad \textit{[Mode 1]} \qquad (5.1)$$

$$F_{v,Rk} = f_{h,2,k} t_2 d \qquad \textit{[Mode 1]} \qquad (5.2)$$

$$F_{v,Rk} = \frac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} {t_1}^2 d}} - \beta\right] \qquad \textit{[Mode 2]} \qquad (5.3)$$

$$F_{v,Rk} = \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d} \qquad \textit{[Mode 3]} \qquad (5.4)$$

Adapted from Equation 8.7, BS EN 1995-1-1 (2004))

Where:

$F_{v,Rk}$ Characteristic load carrying capacity of fastener per shear plane per fastener, N

$f_{h,i,k}$ Characteristic embedment strength of the timber member, N/mm^2

t_i Thickness of timber side member, mm

d Dowel diameter, mm

$M_{y,Rk}$ Characteristic fastener yield moment, Nmm

β Ratio between embedment strength of the members, given by:

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \qquad (5.5)$$

For this the load carrying capacity of a single fastener (laterally loaded), $F_{v,Rd}$ is given by the formula:

$$F_{v,Rd} = \frac{k_{mod} F_{v,Rk}}{\gamma_M} \tag{5.6}$$

Where:

k_{mod} Modification factor for the effect of moisture content and load duration – See Table 3.1, BS EN 1995-1-1 (2004).

Υ_M Partial factor for the material, incorporating uncertainty in geometric, material properties and uncertainty in model assumptions. See UK NA to EN 1995-1-1 (2004) Table NA.3 - for connections this should be given as 1.3.

$F_{v,Rk}$ Characteristic load carrying capacity per shear plane per fastener (N)

Friction between the members in a connection will be induced by two actions, one during the assembly of the connection and two during the yielding of the fastener forcing the members together. Due to the hydroscopic characteristics of timber, friction is highly variable and cannot be guaranteed throughout the service life of a connection, this can give slightly conservative values for connection strength in laboratory tests but for design purposes should be ignored (McLain and Thangjitham, 1983). When a fastener yields it will cause a tensile force to occur in the fasteners creating a normal force between joint members producing increased friction. These withdrawal effects commonly referred to as the 'rope effect' were not included in the initial EYM but subsequently have been included in the BS EN 1995-1-1 (2004) design calculations (Porteous and Kermani, 2013). Here the value is based a proportion of the characteristic axial withdrawal capacity ($F_{ax,Rk}$) derived from expressions in BS EN 1995-1-1 (2004) based on the fastener type and the penetration depth. For this research only dowel fixings were being considered, therefore following the recommendations given in clause 8.2.2(2) (BS EN 1995-1-1, 2004) which state that the additional contribution to the strength of a connection by the rope effect should be taken as 0%.

5.1.2. Embedment strength

The bearing or embedment strength of a dowel-type fastener is a system property defined by the uniform dispersal of stress over the projected area of the dowel. This generalised property encompasses a variety of complex interactions at the contact surface as is namely defined by the density; fastener hole, diameter and shape; angle between load and grain direction; friction between dowel and surrounding timber; moisture content; grain reinforcement (Blass, 2003; Zhou and Guan, 2006; Xu, Bouchaïr & Racher, 2012) and can be simply calculated using the formula, based on Figure 5-4:

$$f_h = \frac{F_{max}}{d \times t} \tag{5.7}$$

Where:

f_h	Embedment strength, N/mm^2
F_{max}	Maximum load, N
d	Fastener diameter, mm
t	Material thickness, mm

Equation 1, BS EN 383, (2007)

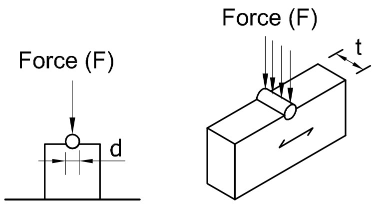

Figure 5-4: Dowel embedment.

Whale, Smith and Hilson (1989) conducted a series of tests that highlighted a strong inverse correlation between the diameter of a fastener and the embedment strength of a piece of timber. Through further investigation this correlation was refined and adopted through regression analysis into BS EN 1995-1-1 (2004) as an expression for embedment strength based on two independent parameters its mean density and the diameter of the fastener. Attempts were then made to relate the embedment strength to 5th percentile values that could be fully integrated into the probabilistic framework that underpins the current timber design codes (Leijten, Köhler & Jorissen, 2004) and subsequently now the embedment strength is derived from an interpolated formula based on the characteristic density of the timber and the diameter of the dowel.

Where larger fasteners are used (i.e. above 8mm) modification or interpolation of the embedment strength in relation to its angle to the grain in accordance with an approximate trigonometric model such as the Hankinson relationship, (Hankinson, 1921) was found to be necessary and subsequently included into BS EN 1995-1-1 (Sawata and Yasumara 2002). The focus of the research is such that only large diameter dowel sizes (above 8mm) are considered and the expressions given in BS EN 1995-1-1 (2004) are shown below.

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha} \tag{5.8}$$

$$f_{h,o,k} = 0.082(1 - 0.01d)\rho_k \tag{5.9}$$

$$k_{90} = \begin{cases} 1.35 + 0.015d\ (for\ softwoods) \\ 1.30 + 0.015d\ (for\ LVL) \\ 0.90 + 0.015d\ (for\ hardwoods) \end{cases} \tag{5.10}$$

Where:

$f_{h,o,k}$ Characteristic embedment strength parallel to grain N/mm²

ρ_k Characteristic timber density, kg/m³

α Angle of load to the grain, °

d Bolt diameter, mm

Equations, 8.31, 8.32 and 8.33, BS EN 1995-1-1(2004)

The expression for the embedment strength of a fastener in timber has been an area of much deliberation for researchers and many of the key assumptions and correlations have been challenged as discussed below. For example, the relationship between dowel bearing strength parallel to the grain and dowel diameter has been contradicted in research. Whale, Smith and Hilson (1989) and Hilson *et al.* (1987) suggested that the bearing strength parallel to the grain decreased as dowel diameter increased, whereas Church and Tew (1997), Harada *et al.* (2000), Sawata and Yasumara (2002) indicated that this was not witnessed in members bearing parallel to the grain in glulam and timber respectively. Kim, Oh & Lee. (2010) corroborated the findings shown in Whale, Smith & Hilson (1989) and expanded that the relationship between dowel diameter and bearing strength parallel to the grain becomes more pronounced when the moisture content drops below 19%. Similarly, Ehlbeck and Warner (1992), Harada *et al.* (2000) and Sawata and Yasumara (2002) showed that the embedding strength reduced when the dowel diameter increased in the perpendicular to grain direction.

The density of the timber will not only affect the ability to drive a dowel into a piece of timber but also its embedment strength. In the most part, when the density of a piece of timber is increased there is positive correlation with the increase of embedment strength (Jumaat, Razali & Rahim 2008; Schoenmakers, Jorissen & Leijten, 2010). Although, Jumaat, Razali & Rahim (2008) conducted tests on 225 specimens with three different diameter bolts and four different species parallel to the grain. They found that although the embedment strength was more or less constant for differing diameters below a density of 900kg/m^3 over this value a proportional reduction in the embedment strength occurred when the diameter of the dowel increased. Hassan *et al.* (2014) explored this avenue of research further to corroborate the findings that the denser the base material the greater the effect that dowel diameter has on the embedment strength of a piece of timber. Of the home-grown materials (Sitka spruce and larch) being considered for inclusion in the study none are expected to reach the density values to adversely affect the embedment strength as witnessed from the studies by Jumaat, Razali & Rahim. (2008) or Hassan *et al.* (2014).

The compilation of results discussed in Franke and Magnière (2014a), seems to suggest that any influence of diameter size on embedment strength could be limited to a species by species basis. As such any species that are to be considered for research will need to have the embedment strength investigated fully before any further correlations can be derived from tests or expressions that are based on the embedment strength as a system property.

5.1.3. Yield moment

The bending strength of a metallic fastener is characterised by its plastic moment or yield moment. The plastic moment is the value when the yield stress of the material is reached across its entire cross-section. The plastic capacity of a nail or dowel is estimated from the maximum moment resistant within a bending test, the apparent yield stress is then used as a fair approximation prior to any strain hardening that would occur under load (Smith, Craft & Quenneville, 2001). Where steel dowels are concerned the inclusion of the plastic moment capacity given in BS EN 1995-1-1 (2004) as the yield moment is easily achieved through testing to BS EN 409 (2009) or using an idealised stress strain curve the plastic moment capacity of a steel dowel can be easily calculated by its geometric properties and the characteristics of the material. BS EN 1995-1-1 (2004) gives concise calculations on each fastener type based on the theoretical derivation of the fastener bending angle at a permissible level of slip (nominally set as 15mm in BS EN 1995-1-1). Following research undertaken by Blass, Bienhaus & Krämer (2001) the effective bending capacity or characteristic yield moment for the dowel given in BS EN 1995-1-1 (2004) is:

$$M_{y,Rk} = 0.3 f_{u,k} d^{2.6} \tag{5.11}$$

Where:

$M_{y,Rk}$ Characteristic value for yield moment, Nmm

$f_{u,k}$ Characteristic tensile strength, N/mm^2

d Dowel diameter, mm

Equation 8.30, (BS EN 1995-1-1, 2004)

5.1.4. Material property parameters

The two material property parameters of a connection, the embedding strength of the timber and the plastic moment capacity of the fastener are invariably derived from tests that seek to obtain a short-term measure of the properties of timber. It has been well documented and understood that timber has viscoelastic properties that alter with time (Dinwoodie, 2000). The application of the EYM in the analysis of a connection creates difficulties in the study of the time dependent properties of a connection. The load carrying capacity of a connection will be dependent on the minimum value derived from a variety of different failure modes given in EC5. The challenge lies in the fact that each failure mode could potentially have differing time dependent behaviour (Marlor and Bulleit, 2005).

Congruently, the effect the loading rate has on the yield load and behaviour of a connection is of additional importance in understanding a connection in operation. To further complicate matters the direction of load to the grain will have an effect on its behaviour in operation and failure. The assumption within the BS EN 1995-1-1 (2004) is that the behaviour of a connection over time is identical to the load duration behaviour of the same timber acting in bending. At worst the modification factor of a connection formed of two or more different materials that have differing time-dependent behaviour is the square of the product of the differing modification factors described in Table 3.1 (BS EN 1995-1-1, 2004).

The increase of dynamic strength over static strength is pronounced within all types of timber. Marlor and Bulleit (2005) evaluated the behaviour of single shear connections over the short and long term. By arranging the connections in a series of different geometries they were able to isolate the behaviour of four differing types of yield failure and isolate whether or not the joint behaviour over time was related to the mode of failure or a single time dependent variable. The results for 160 samples indicate that the single time dependent factor based on the bending strength over time, i.e. the 'Madison Curve' (Wood, 1951) was sufficient as a close approximation to actual behaviour over the four differing yield modes. The results are assumed to apply to symmetrical double shear plane tests as theoretically the double shear failure modes are a mirror of their single shear counterparts and should not experience differing time dependent properties. Therefore, it can be assumed that the results found from short term tests are compatible with the long-term performance of a panel in operation.

5.1.5. Multiple shear plane calculation to EYM

EYM application is limited to connections with single or double shear planes. Where a connection comprises multiple shear planes such as the transverse connection of a DLT panel, the connection is idealised into a series of double shear planes and the connection strength based on the minimum combined strength value of compatible failure modes. The yield modes of a multiple shear connection will be dependent on the symmetry of a connection, the distances between shear planes, the assumptions of timber embedment and dowel bending behaviour (Sawata, Sasaki & Kanetaka, 2006). The shear strength of a connection is based on the lowest strength calculated from a corresponding compatible yield mode based on the EYM failure modes based upon the creation of symmetrical three-member connection assemblies, see Figure 5-5.

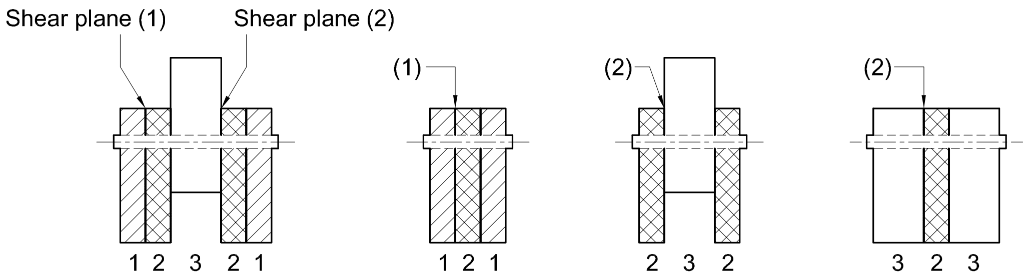

Figure 5-5: Formation of symmetrical three-member connection assemblies (adapted from Porteous and Kermani, 2013).

Where the number of shear planes is increased, the compatibility of the yield modes is called into question. Connections with multiple shear planes will have a greater number of differing yield modes than those described through the EYM (Sawata, Sasaki & Kanetaka, 2006). Although Murty, Smith & Asiz (2007), argue that the conclusions drawn from a simple double shear plane arrangement are valid for other connections (which consist of varying numbers of shear planes) and therefore can utilise the same mechanisms of failure and simplifies what would otherwise be a complicated problem (Porteous and Kermani, 2013).

5.1.6. Brittle failure of connections

The need to provide the reliability necessary for a timber structure has led to the emphasis being placed on the calibrating all the parameters of timber joints, including whether they need to be designed on an individual basis or as a system (Smith and Foliente, 2002). The EYM is calibrated for a ductile failure in a single fastener connection and does not consider fully the possibility of brittle failure occurring in many different instances but in particular when a connection comprises many fasteners or stocky dowels (Quenneville and Mohammad, 2000). The design capacity of a connection will need to be governed by the estimation of minimum failure load derived from both ductile and brittle modes. Using the fracture modes described in Franke and Quenneville (2011), splitting perpendicular to the grain can occur in three different modes; symmetric separation; in-plane shear; out of plane shear; that can be categorised further into different modes due to the anisotropic nature of timber. The ductile modes for metal fastener connections are shown in section 5.1.1 for double shear plane connections, however BS EN 1995-1-1 (2004) also requires the verification against certain brittle modes of failure as illustrated in Figure 5-6.

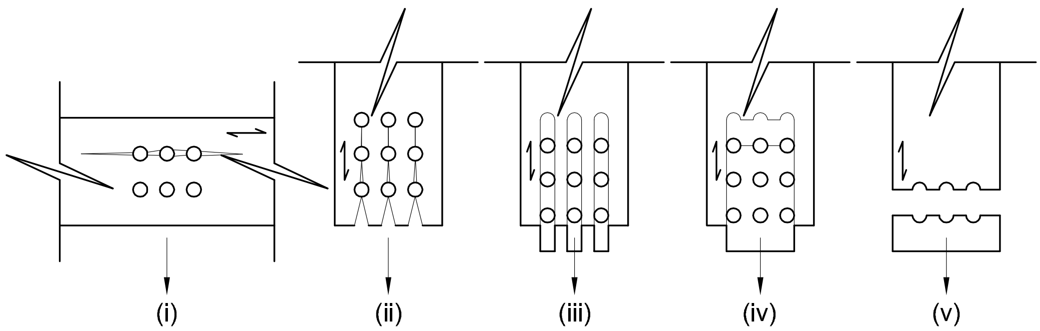

Figure 5-6: Brittle connection failure, i) perpendicular splitting, ii) in-line splitting, iii) plug shear, iv) group tear-out, v) net tension

In timber joints, particularly in the instances where large diameter fasteners are used, these systems experience a wider variety of failures than assumed in the EYM (Quenneville and Mohammad, 2000). Brittle failure arises when the tension or shear strength of a piece is overcome. How the tension or shear strength is determined varies greatly depending on the assumptions used for analysis. Whether the maximum or minimum calculated values for tension or shear strength are used or a summation of the two values is used creates a large range of theoretical strength values, which disagree with the experimental findings (Zarnani and Quenneville, 2014). In DLT construction, net tension, plug shear or group tear out are unlikely to be a problem, the other brittle failure modes could potentially pose a problem and should be verified. The most pertinent brittle failure mode for DLT would be perpendicular to the grain tension failure as the panel will be loaded in a perpendicular manner.

Where a longitudinal brittle failure occurs, it is believed that the shear strength of the timber is the most significant characteristic determining the strength of a connection (Quenneville and Mohammad, 2000). The shear strength of the connection will be dependent on the smallest spacing between the rows of bolts or the distance to the end of a piece of timber. Too small a distance and the shear stress cannot be redistributed to other bolts in the row and brittle failure will occur (Quenneville and Mohammad, 2000). Hindman *et al.* (2010) investigated the effect of loaded edge distance when a timber connection (using a metal bolt) was loaded perpendicular to the grain, through 102 tests he determined (in a single bolted connection) that three types of failure were witnessed. At a loaded edge spacing of 4 dowel diameters, splitting occurred in the outer members with non-deformation occurring in the timber surrounding the dowel (through embedment) or in the dowel itself. Franke and Quenneville (2014) called this failure a 'symmetrical separation'. When the loaded edge distance perpendicular to the grain was increased to the region of between 7-10 dowel diameters two types of mixed mode failure were seen. Both failures included eventual splitting failure of the outer members, whilst one saw large amounts of embedment occurring the other witnessed the formation of a plastic hinge in the dowel.

The difficultly in predicting brittle failure reliably is in part due to the fact it is not an independent action but a composite action that relies on the convergence of many different factors. These tend to be geometric considerations, including size of members, spacing of connectors, number of connectors, loaded edge and end distances (Ballerini and Rizzi, 2007; Sawata *et al.,* 2013). Many researchers challenged the assumption that there is a non-linear influence between parameters and have devised analytical models based on Linear Elastic Fracture Mechanics (LEFM), which the current European timber design suite utilises (Jensen, 2005; Ballerini and Rizzi, 2007) and is discussed further in 5.1.8.

5.1.7. Spacing rules

For a large variety of joint types using timber and metal dowel fixings brittle failure should be considered, many design codes attempt to circumvent this need by placing restrictions on their layout and spacing based on empirical rules (Daudeville, Davenne & Yasumura, 1999). By ensuring that a failure in a timber connection using metal fasteners occurs in the fastener itself by providing the required distance between fasteners, ductile behaviour can be assumed (Bouchaïr, Racher & Bocquet, 2007). The code sets out stringent guidelines for the arrangement of a joint based on minimum spacing's, edge and end distances to reduce the probability of brittle failure (Smith *et al.,* 2005), see Figure 5-7.

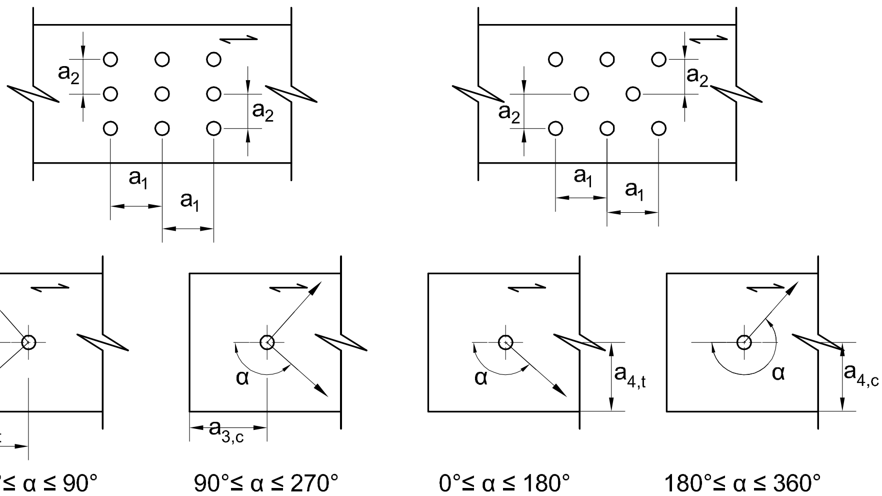

Figure 5-7: Spacing and end and edge distances from BS EN 1995-1-1 (2004).

The provision of minimum spacing rules for metallic connectors given in BS EN 1995-1-1 (2004) has been defined relative to the diameter of the fixing, the angle to the grain of the load that is applied, placement of a fixing relative to one another, and the end and edge distances to limit (or remove entirely) potential for a brittle failure to occur, see Table 5-1.

Distance	Angle to the grain	Minimum distance or spacing
Spacing parallel to the grain, a_1	$0° \leq \alpha \leq 360°$	$(3 + 2\lvert\cos\alpha\rvert)d$
Spacing perpendicular to the grain, a_2	$0° \leq \alpha \leq 360°$	$3d$
Loaded end distance, $a_{3,t}$	$-90° \leq \alpha \leq 90°$	$\max(7d; 80mm)$
Unloaded end distance, $a_{3,c}$	$90° \leq \alpha < 150°$	$(a_{3,t}\lvert\sin\alpha\rvert)$
	$150° \leq \alpha < 210°$	$\max(3.5d; 40mm)$
	$210° \leq \alpha \leq 270°$	$(a_{3,t}\lvert\sin\alpha\rvert)$
Loaded edge distance, $a_{4,t}$	$0° \leq \alpha \leq 180°$	$max[(2 + 2\sin\alpha)d; 3d]$
Unloaded edge distance, $a_{4,c}$	$180° \leq \alpha \leq 360°$	3d

Table 5-1: Minimum spacings and distances for dowels (BS EN 1995-1-1, 2004).

5.1.8. Brittle failure perpendicular to the grain

For connections comprising multiple fasteners, brittle modes of failure can dominate when stiff fasteners with a low slenderness ratio are used. When a connection applies a force at an angle to the grain in a piece of timber, there will be a tensile force component that could cause a brittle failure through splitting perpendicular to the grain. Modification factors for multiple fasteners loaded parallel to the grain have been verified, but these factors have not been validated for perpendicular to the grain (Quenneville and Mohammad, 2001). The design splitting capacity stated in BS EN 1995-1-1 (2004) is based upon linear elastic fracture mechanics (LEFM) that presupposes that the largest tension force will be induced in the fastener located at the furthest distance from the loaded edge causing splitting to occur along the grain (Porteous and Kermani, 2013). This aligns with the findings previously that brittle failure modes propagate from cracks occurring parallel to the grain, with (sometimes) some embedding occurring prior to this crack initiation (Daudeville, Davenne & Yasumura, 1999). BS EN 1995-1-1 (2004) gives the characteristic splitting resistance for one member based on LEFM research conducted by Leijten and Van der Put (2004) as:

$$F_{90,Rk} = 14bw\sqrt{\frac{h_e}{(1-\frac{h_e}{h})}} \tag{5.12}$$

Equation 8.4, BS EN 1995-1-1 (2004)

Where:

$F_{90,Rk}$ Characteristic Splitting Capacity, N

b Member thickness, mm

w Modification factor (=1 in all instances apart from punched metal plates)

h_e Loaded edge distance to the centre of the most distance fastener, mm

h Member height, mm

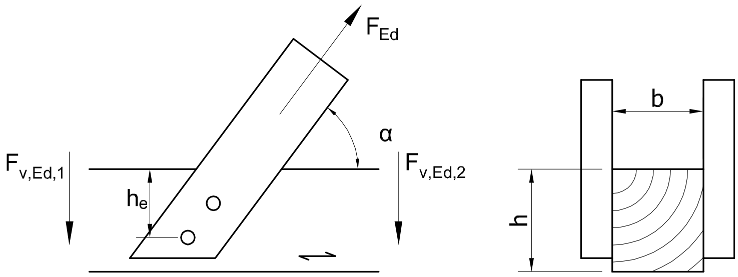

Figure 5-8: Perpendicular to grain geometry, BS EN 1995-1-1 (2004).

Ballerini and Rizzi (2007) concluded that there was a fundamental oversight in the understanding of the brittle capacity of a connection as it is influenced heavily by the configuration of connectors, including number, area and distances between fastener groups. Additionally, the LEFM model over simplified (for design purposes) the normal forces that are exerted in the split portion of a beam when the entire width of the beam has not failed, particularly when slender dowels are used (Jensen, 2005). By considering the variations in the stiffness of the timber that surrounds and resists the load from a fastener it is likely that a non-uniform load will be applied to each fastener. As a result, the shear and tension forces resisted by each individual fastener would be the proportional to the stiffness of the timber volume being loaded and how much of the load is transferred through to the planes of failure of the volume being loaded (which also will be a function of stiffness) (Zarnani and Quenneville, 2014). Where a species of timber has high levels of anisotropy it is expected to be more susceptible to brittle modes of failure, in particular cracking due to stresses in the perpendicular to grain direction (Santos *et al.,* 2013). Here the probability of such a failure will increase when lower strength timber is used, which will be the case from timber sourced from the home-grown British timber stock. In practice, there is an unequal amount of load shared between the fasteners which is particularly more evident in connections where limited ductility is witnessed prior to failure such as small ratios between fastener diameter and member thickness (Tan and Smith, 1999) which may occur in some standard DLT configurations.

5.2. All-timber connections

Design guidance given by the Institution of Structural Engineers for non-metallic dowel materials has been to simply substitute the moment capacity (M_y) of the chosen connector into the analytical method given in BS EN 1995-1-1 (2004) for calculating connection capacity (IStructE, 2007). Several researchers have argued that the stiffness and bending strength of a steel dowel is fundamentally different to that of a timber dowel and will cause different more complex failure modes to occur beyond the rules and expressions outlined in BS EN 1995-1-1 (Schmidt and Mackay, 1997; Schmidt and Daniels, 1999; Sandberg, Bulleit & Reid, 2000; Shanks and Walker, 2005; Thomson *et al.,* 2010).

The failure modes given in BS EN 1995-1-1 (2004) are based on equilibrium and compatibility of materials and the only difference between a connection utilising a metal fastener and one using a timber fastener is the material itself (Schmidt and MacKay, 1997). Where a material has been proven to yield as is the case with all-timber connections (Shanks and Walker, 2005) and GFRP connections (Thomson *et al.,* 2010) this is applicable. The application and compatibility of the new failure modes based on the failure modes for a double shear connection depicted by the EYM is not necessarily accurate because the assumption that a timber dowel yields plastically in bending is not always met due to the shear loads in the timber dowel (Shanks and Walker, 2005). Significant yielding is commonly witnessed in typical mortise and tenon connections using materials with the same bearing stiffness behaviour, tests have shown that it cannot be guaranteed with materials with different bearing stiffness (Sandberg, Bulleit & Reid, 2000). Nevertheless, current research has extrapolated the results to postulated additional failure modes for timber dowel connections based on the EYM created by Johansen (1949) and developed by Larsen (1979).

5.2.1. Failure modes of an all-timber connection

The connection strength of an all timber connection is dependent on many different factors namely, bending and strength of the dowel, bearing strength of the dowel and the shear strength of the timber material (Schmidt and MacKay, 1997). Due to the similarities in bearing stiffness throughout an all timber connection and the differing shear and bending behaviour of a timber dowel in comparison to a steel dowel many of the bearing failure modes given in BS EN 1995-1-1 (2004) cannot be followed directly. Where there is a greater difference between dowel bearing strength and the timber bearing strength perpendicular to the grain the results were more in line with the results for test conducted using steel dowels (Church and Tew, 1997). Where the ratio of stiffness between the dowel and the bearing material are similar to that given in BS EN 1995-1-1 (2004) the expected failure mechanism will be similar to that assumed in BS EN 1995-1-1 (2004).

Failure modes for all timber connections are depicted in Schmidt and McKay (1997) and were expounded further by several researchers (Schmidt and Daniels, 1999; Sandberg, Bulleit & Reid, 2000). Mostly these modes of failure are deemed inapplicable for traditional UK oak frame construction (Shanks and Walker, 2009) but in DLT production there is a greater disparity between the relative stiffness of the dowel and the connection material and this is similar to North American practices of post and beam framing and should behave more in line with BS EN 1995-1-1 (2004) predictions. The majority of the initial research conducted on all-timber connections was undertaken in the US so the definition of the failure modes follows the precedence set in the US National Design Specification (NDS, 2005) and is followed here but related to the failure mode type in accordance with BS EN 1995-1-1 (2004).

Based upon the four modes of failure for double shear plane connections outlined in the EYM, Schmidt and McKay (1997) found an additional failure mode that was applicable for an all-timber connection termed, mode V that included combined shear and bending in the dowel occurring. Figure 5-9 illustrates the modes of failure of an all-timber connection after Schmidt and McKay (1997). The occurrence of a failure mode will vary depending on connection geometry and dowel stiffness (Shanks, Chang & Komatsu, 2008).

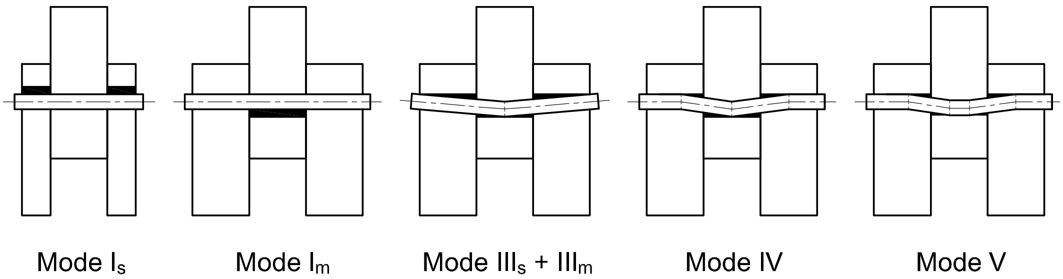

Figure 5-9: Failure modes of double shear plane connections using timber dowels.

5.2.1.1. Failure mode *I* (EYM type 1 failure):

- I_s– (EYM Mode Type 1): Bearing dominated failure of the side members. Occurs when the central member is *wider* than the sum of the side members and fastener relatively large diameter.
- I_m– (EYM Mode Type 1): Bearing dominated failure of the central member. Occurs when the central member is *narrower* than the sum of the side members and the fastener has a relatively large diameter.

Schmidt and Daniels (1999) also included a failure mode, termed $\boldsymbol{I_d}$ for when the bearing failure of a fastener occurs prior to the central and side members.

5.2.1.2. Failure mode *III* – (EYM type 2 failure):

Fastener yielding has been shown to develop in two ways:

- III_s – (EYM Mode Type 2): Fastener yielding with the creation of one plastic hinge per shear plane and bearing dominated yield of side member. Occurs if the central member's thickness to the side members is relatively large and the fastener diameter is fairly small. Shanks and Walker (2009) noted that this normally occurs when a shear span is between 1.5d – 2.5d.

The rotation that occurs within the dowel creates significant bearing occurring in the side members (Shanks, Chang & Komatsu, 2008).

- III_m – (EYM Mode Type 2): Flexible dowel with side wall failure, created by a two-hinge failure with tension perpendicular to the grain being induced on the inner portion of the member by the rotation of the dowel. Stiff dowels will cause perpendicular to the grain failure in the side members as it will load it them uniformly (Shanks, Chang & Komatsu, 2008).

5.2.1.3. Failure mode IV – (EYM type 2 failure)

Formation of two plastic hinges per shear plane in the fastener alongside localised bearing failure in centre and side members. Occurs when both the ratio between the centre member(t) or side members(t) and dowel diameter (D) are fairly small. Mode $\boldsymbol{IV}$ may not be directly applicable to timber dowels due to the formation of plastic hinges but it has been witnessed in some all-timber connection tests conducted by Schmidt and McKay (1997); Schmidt and Daniels (1999); Sandberg, Bulleit & Reid, (2000); Shanks, Chang & Komatsu (2008); Shanks and Walker (2009). Mode $\boldsymbol{IV}$ forms a ductile failure through the confinement of the dowel and is most commonly witnessed with a flexible dowel that is sited with sufficient edge and end distances. The shear span (i.e. distance between yield points) will be a function of the stiffness surrounding the dowel interface (Shanks, Chang & Komatsu, 2008). Shanks and Walker (2009) also noted that this normally occurs when the dowel has a shear span of between 0.5D – 1.5D. But where the span between the formations of the plastic hinges in the dowels is greater than the width of the central member the formation of failure modes other than Mode $\boldsymbol{IV}$ will occur (Schmidt and McKay, 1997).

5.2.1.4. Failure mode V_d – (EYM type 3 failure)

Occurs when there is a cross grain shear failure of the dowel brought about by the composite action of shear, bending and bearing of the dowel (Miller, Schmidt & Bulleit, 2010). This type of failure induced by the confinement of the dowel and the gap between the side and main members can be more akin to the brash failure witnessed by tension loads rather than the typical parallel to grain shear failure mode (Sandberg, Bulleit & Reid, 2000). A mode $\boldsymbol{V_d}$ three hinge failure occurs when the central member has a width that is small enough so that the shear span is at least half of the width causing the two expected hinges to form a single central hinge (Shanks, Chang & Komatsu, 2008).

5.2.2. All-timber connection strength

Quenneville (2009) and Thomson (2010) both raise an important point regarding the clarity of the definition of ductility in a timber connection and yielding in a connection using glass fibre reinforced polymer (GFRP) dowels. This definition is applicable to the description of an all-timber connection and distinctly different to the terminologies when applied to other materials such as steel. Here ductility is defined as the ability of the connection to resist sudden failure through increased deformation that results in an irreversible energy transferral (in this case crushing of timber) that cannot resist subsequent cycles of loading (Quenneville, 2009). Furthermore, the yielding of a non-metallic dowel often results in the initiation of inter-laminar shear failure that induces noticeable yield point but does not allow further absorption of energy to occur across further cycles (Thomson *et al.,* 2010).

The response of the all timber connection under load can be roughly broken down into four distinct phases, shown in Figure 5-10.

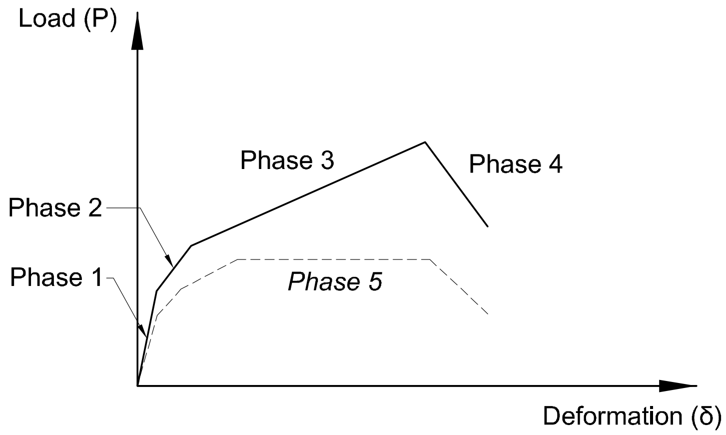

Figure 5-10: Typical load deformation of a mortice and tenon timber dowelled joint, adapted from Shanks, Walker & Harris (2006).

The first phase relates to the pre-stress in the connection, the second phase shows a stiffness reduction through bearing. After the initial linear region (phase 1 and 2) a brittle failure was often witnessed when a relish failure occurred due to short end distances or splitting of the side members occurred when a stiff dowel was used. However, a ductile failure was apparent when the dowel failed in shear and bending (Shanks, Chang & Komatsu, 2008), indeed joint failure associated with dowel failure i.e. $\boldsymbol{I_d}$, $\boldsymbol{V_d}$, $\boldsymbol{III_m}$ tends to occur in a non-linear ductile manner (Burnett *et al.,* 2003). The bearing stresses induce local crushing of the timber members, which in turn lengthens the shear span of the dowel creating bending and shear failure in the dowel to occur in phase 3. Phase 4 the connection yields and then loses load resistance as deformation increases, the timber fibres debundle and the remaining tensile capacity in the dowel can cause the connection to wedge where the friction forces will dominate for the final stages (Shanks & Walker, 2005; Shanks, Walker & Harris, 2006). In some connections a plastic plateau occurs (shown as phase 5 in Figure 5-10), before phase 4 behaviour initiates.

Shanks and Walker (2009) developed an energy-based model for analysing the connection strength based upon the assumption that a timber dowel connection behaves in an elasto-plastic manner with significant yielding occurring (see Figure 5-11). In simple terms, the energy-based model assumes a virtual displacement of a dowel in single shear by a unit load. From which an expression of the area, A crushed underneath the dowel can be defined in terms of connection geometry and the distance a, from point of rotation can be solved in relation to F_v (Parsons and Bender, 2004). The sum of these actions then defines the connection capacity, which can be given by the equation:

$$F_v = \sum(f_h A) + \sum\left(\frac{M_y}{a}\right) \tag{5.13}$$

Where:

F_v Connection capacity, N

A Area of material crushed, mm^2

a Distance from the point of dowel yielding in the side member (shear span), mm.

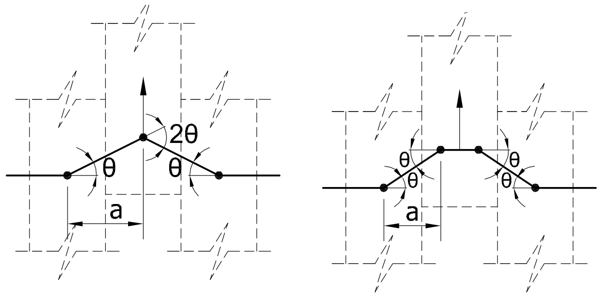

Figure 5-11: Development of 3 or 4 plastic hinges in a timber dowel connection.

In materials with different embedment behaviour the energy absorbed through bearing actions cannot be surmised to be negligible when comparatively assessed against dowel yielding. The method of inquiry used here is of interest, in the development of a reliable method of understanding the internal energy dissipation occurring in the connection when a dowel yields and forms hinges in four or three locations, see section 5.2.1.2 - 5.2.1.4 and Figure 5-11. The plastic load resistance of the dowel then can be summarised as:

$$P_p = \frac{4M_y}{a} \tag{5.14}$$

Where:

P_p Plastic load resistance, N

M_y Yield moment of dowel, N/mm

a Shear span, mm

(Shanks and Walker, 2009)

5.2.2.1. Dowel shear mode

Three-point bending tests conducted on timber dowels indicate that the failure of the timber dowel is caused by brittle tensile failure in the outermost fibres of the dowel, see Figure 5-12. However, this mode of failure is not normally witnessed in dowels of an all-timber connection due to a variety of factors such as confinement, shear span and friction. Several researchers (MacKay, 1997; Shanks and Walker, 2009) have, by using a simple three-point bending test and the principle of structural mechanics (including shear effects), determined the yield moment of a timber dowel and have termed it an effective bending strength.

Figure 5-12: Typical brittle tensile failure experienced by timber dowel undergoing three-point bending

Further refinement of the EYM has been conducted to include the effects of shear stresses in the timber dowel itself through the development of an effective peg shear strength (Miller, Schmidt & Bulleit, 2010). In all softwood connections, it was found for dowel diameters between 12 and 18mm the increase in strength was directly proportional to the increase in diameter of the dowel for perpendicular to grain double shear tests (Shanks, Chang & Komatsu, 2008). Whilst Eckelman and Haviarova (2007) discovered that the ultimate load capacity was proportion to $D^{1.5}$ for parallel to the grain connections using timber dowels. Often it is difficult to tell apart the failure mechanisms that experience the formation of a plastic hinge (Mode $\boldsymbol{III_m}$) or one that experiences complete cross-grain shear failure (Mode $\boldsymbol{V_d}$) because both modes induce concentrated shear discontinuities in the dowel itself which after failure are difficult to distinguish between clearly.

Further investigation of the 'effective peg shear mode' in double shear was undertaken by Miller, Schmidt & Bulleit (2010) who described the failure process as a simultaneous multiple shear failure, deducing that the connection capacity (Z_v) should be determined from the average stress value through the cross-sectional area of the dowel or peg. Given by the equation:

$$Z_v = \frac{\pi D^2 F_{vy}}{2R_d} \tag{5.15}$$

Where:

F_{vy} Average mode V yield shear stress (at 5% diameter offset) is given by the equation below, N/mm^2

D Dowel diameter, mm

R_d NDS Reduction Factor (intended to calibrate theoretical EYM analysis with empirical design), in this instance 3.44.

$$F_{vy} = 33440 G_{peg} G_{base}{}^{3/4} \tag{5.16}$$

Where:

G_{peg} Specific gravity of dowel

G_{base} Specific gravity of timber

The effective bending resistance of the timber dowel was compared to BS EN 1995-1-1 (2004) values by Shanks and Walker (2009) and they found that lower bound predicted values compared well for turned pegs but overestimated values for die driven or tapered pegs. Shanks and Walker (2005) conducted tests on three types of oak dowel manufacture, die driven, turned and tapered and they found that die driven dowels performed the greatest and unsurprisingly the reduction of the cross section in the tapered section greatly affected its performance along its length. In addition to these findings, radially loaded pegs proved to be stiffer and stronger than tangentially loaded pegs (Shanks and Walker, 2009; Church and Tew, 1997), and this was increasingly apparent when shear span decreases (Shanks and Walker, 2009).

5.2.2.2. Edge and end distances

Where end and edge distances are sufficient to provide the shear area to resist the applied load on a connection, plug failure or relish failure in the timber will be stopped and the formation of a hinge in the dowel is more likely. The minimum end distances required will be related to the relative strength of the dowel and the base material (Shanks, Chang & Komatsu, 2008). A steel dowel will require more distance than a hardwood dowel in a softwood base material (Burnett *et al.,* 2003) and even more than using a softwood dowel (Shanks Chang & Komatsu, 2008). Thomson *et al.* (2010) has demonstrated that lower individual connector capacity of a non-metallic fixing arrangement can be overcome by utilising the reduced spacing allowed to provide more connectors whilst avoiding potential brittle failure.

Fastener rigidity has been shown to affect the nature of contact between the fastener and the laminations, creating further chance for splitting to occur in the timber members of a connection (Kharouf, McClure & Smith, 2003). Where there is a contact between materials that is more deformable or relatively more deformable than a solid steel dowel and a piece of timber the propensity for crack propagation is reduced (Murty, Smith & Asiz, 2007). However, Eckelman and Haviarova (2007) postulated that under load the timber dowels deform to form an oval cross-section that creates stress concentrations that exert a force outward promoting splitting in the timber laminations.

Previous investigation into the typical geometry of a traditional mortice and tenon joint in the UK has shown that an unloaded edge distance of 1.5d can be used (Shanks, Walker & Harris, 2006). Here the stress distribution is based on the two materials within a connection having the same or similar bearing characteristics. Studies conducted in North America by Burnett *et al.* (2003) state that the minimum end distance for hardwood dowel in softwood member should be 2.5d to 4.5d within differing base materials and these constraints are taken forward throughout this study. Further investigations are necessary to confirm the edge and end distances where materials have different stiffness characteristics such as UK grown softwood and hardwood.

5.3. Stiffness of a timber connection using metallic fasteners

Due to manufacturing tolerances, yielding of fastener or timber under load (or any combination of all three), all joints formed in timber will slip (Porteous and Kermani, 2013). The amount of deformation or slip in a connection will alter depending on the type of fastener being used, its behaviour under load and the overall arrangement of the connection.

The graph shown in Figure 5-13 illustrates the typical load slip behaviour of a bolted connection. In almost all instances the initial load slip in a connection is non-linear due to immediate settlement of the joint and where there is tolerance provided for the insertion of a fastener an initial slip occurred as the fastener shifts to the point of bearing. From this relationship, the stiffness of a fastener can be defined as the ratio of the lateral load (per shear plane) divided by its slip. This stiffness property is referred to as the slip modulus in BS EN 1995-1-1 (2004).

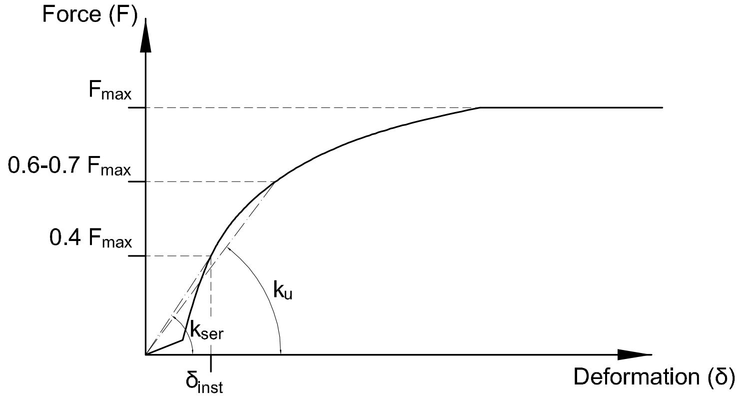

Figure 5-13: Typical load slip curve for a bolted connection.

At the SLS the instantaneous slip modulus (K_{ser}) gives an approximation of the initial behaviour of a fastener and is based on the secant modulus of the curve at approximately 40% the maximum load of the connection (Ehlbeck and Larsen, 1993). At the ULS the instantaneous slip modulus of a connection (K_u) can be taken as $2/3\ K_{ser}$ according in clause 2.2.2(2) of BS EN 1995-1-1 (2004), which equates roughly to a secant modulus of the load deformation curve between 60 to 75% the maximum load (Porteous & Kermani, 2013).

At SLS the slip modulus K_{ser} according to BS EN 1995-1-1 (2004) can be calculated from a series of empirical expressions that are based upon the connection type, its diameter and the density of the timber being connected under static loads (see Table 5-2). Derived from a regression analysis conducted on a large body of tests (Ehlbeck and Larsen, 1993) it only incorporates metallic based fasteners and does not include geometrical considerations (Reynolds, Chang & Harris, 2013). The expressions given in BS EN 1995-1-1 (2004) of the deformation behaviour of the fasteners can also be based on the slip modulus per shear plane obtained from configurations and test procedure outlined in BS EN 26891 (1991).

Fastener Type	K_{ser}
Dowels	$\rho_m^{1.5}d/23$
Bolts	
Screws	
Nails (with predrilling)	
Nails (without pre-drilling)	$\rho_m^{1.5}d^{0.8}/30$
Staples	$\rho_m^{1.5}d^{0.8}/80$
Split-ring connector type A according to EN912	$1.5\rho_m d_c/2$
Shear-plate connectors type B according to EN912	

Table 5-2: Formulae for slip modulus of fasteners according to BS EN 1995-1-1 (2004).

From these expressions, it is clear to see that increasing the mean density of the base material (ρ_m) and the diameter of a metallic fastener will increase the slip modulus and hence the stiffness of a connection. Where a connection is formed of members with differing densities, $\rho_{m,1}$ and $\rho_{m,2}$ the expression given in BS EN 1995-1-1 (2004) clause 7.1(2) can be used, which is:

$$\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}} \qquad (5.17)$$

Equation 7.1, BS EN 1995-1-1 (2004)

The underlying assumption is that the linear laws of analysis followed in the standardised methodology given in BS EN 26891 (1991) ensure compatibility with the existing theories; but there is in fact a bi-exponential load slip response (Natterer and Weinand, 2008). Meaning there is an initial stiffness prior to yielding and a final stiffness after yielding, see Figure 5-14. Whilst it is evident in numerous tests conducted on double shear plane connections, that hardening is witnessed after yielding occurs in a connection continuing compatibility with existing theory requires that a linear approach is taken.

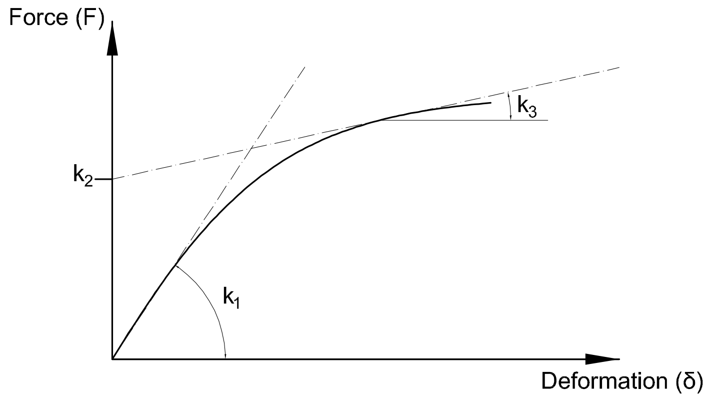

Figure 5-14: Bi-exponential load diagram, adapted from Natterer and Weinand (2008).

5.3.1. All timber connection stiffness

The stiffness of an all timber connection will be dependent on the non-linear anisotropic behaviour of the timber surrounding the fastener (Bouchaïr, Racher & Bocquet, 2007) and in the fastener itself. Sandberg *et al.* (2000) presented a method of analysing the stiffness of a timber joint based on the assumption that the dowel was simply supported beam by the side members and loaded at two points by the resultants of the tenon force.

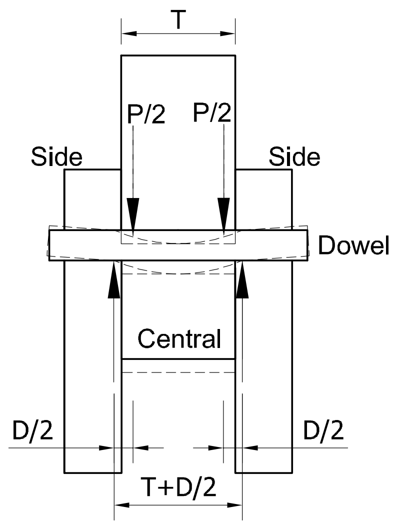

Figure 5-15: Dowel modelled as a simply supported beam, adapted from Sandberg Bulleit & Reid (2000).

The overall stiffness of the dowel is then defined through the summation of the bending and shear stiffness (assuming that the ratio of elastic and shear modulus is 1/16).

$$F_{peg} = \frac{2(6T - d_{peg})}{3\pi E_{peg} {d_{peg}}^2} + \frac{160}{9\pi E_{peg} d_{peg}} \qquad (5.18)$$

Where:

F_{peg} Flexibility of dowel, N/mm

E_{peg} Modulus of Elasticity of the dowel, N/mm^2

d_{peg} Diameter of the dowel, mm

T Central member thickness, mm

(Adapted from Sandberg, Bulleit & Reid, 2000)

By assuming that any additional flexibility of the members is independent of any dowel flexibility and as such can be simply expressed as an empirically based linear function of specific gravity; the stiffness of a single dowel joint could be ascertained by:

$$K_{joint} = (F_{peg} + F_{members})^{-1} \qquad (5.19)$$

Where:

K_{joint} Joint stiffness, N/mm

$F_{members}$ Member stiffness, N/mm

(Sandberg, Bulleit & Reid, 2000)

In BS EN 1995-1-1 (2004) connections formed with metallic dowels have the same stiffness independent of load direction. Sandberg, Bulleit and Reid (2000) assumed similarly that the stiffness of an all timber connection would mainly be dependent on the deformation of the fastener and only attributed a small influence on the stiffness of the other components in the joint (such as grain direction). The joint stiffness calculated relates to a single timber dowel loaded in a double shear test. Where additional dowels are used the authors suggest the stiffness of the connection can simply be calculated as the product of the number of dowels and the stiffness of a single dowel loaded in double shear. Although the authors of the paper found that stiffness values underestimated the actual tests by approximately 25-30%, Shanks and Walker (2009) found that for UK mortice and tenon connections the methodology predicted stiffness far higher (often triple) than seen throughout testing and the model was not compatible with European species that have a higher bearing stiffness.

Understanding the behaviour of the connection throughout the loading is crucial to the development of a suitable stiffness model. In initial stages, the dowel was often shown to rotate within its hole and act as if it is simply supported (Shanks and Walker, 2009; Shanks, Walker & Harris, 2006). Therefore, by considering Sandberg *et al.* (2000) four-point bending model, Shanks and Walker (2009) developed equations for the deflection of the connection and hence the stiffness based upon the continuing assumption that the dowel is simply supported with a constant moment between load points. The elastic deflection considering shear and bending deflection can be given as:

$$\delta = \frac{Pa}{EI}\left(\frac{L^2}{16} - \frac{a^2}{12}\right) + \frac{3Pa}{5GA} \tag{5.20}$$

Where:

δ Overall deflection of connection, mm

P Load applied, N

a Shear span of the dowel, mm

L Clear span of the dowel, mm

E Elastic Modulus of the dowel, N/mm^2

G Shear modulus of the dowel, N/mm^2

I Second moment of area of the dowel, mm^4

(Shanks and Walker, 2009)

This equation considers shear span of the dowel as a critical function in the overall stiffness of the connection and should be taken for traditional timber frame connections in the UK as 1.5d (Shanks and Walker, 2009) but this could vary due to construction tolerance, connection geometry and will not be suitable for other connection types and needs to be investigated fully. Researchers have shown that whilst the dowel orientation did not seem to have an effect on the strength of oak dowels its orientation greatly affected its stiffness (Church and Tew, 1997; Sandberg, Bulleit & Reid, 2000). Shanks and Walker (2005) similarly reported that the radial stiffness of the dowels was between 20-50% greater than the tangential direction, this was attributed to the presence of parenchyma cells in hardwood dowels restraining the fibres in the radial direction.

Often the initial research on dowel connections omits the present of contact friction for simplicity, this can be a fair conservative rationalisation, as friction cannot be assumed to exist throughout the lifespan of the structure. For an all-timber connection, the friction becomes a more important parameter because the rough surface of a piece of timber may increase the friction coefficient between the two materials, it also increases the potential for the reduction in joint capacity due to surface defects (Zhou and Guan, 2006). There have been attempts to model the dowel in the connection as a rigid beam with friction effects included without shear and bending deformation considered (Fukuyama *et al.,* 2008). Whilst good correlation was achieved between experiment and theoretical values, it is seen only to be applicable to single shear connections with stocky dowels. Further investigation is needed to understand the proportional improvement or reduction from the friction effects particularly in double (or a greater number of) shear planes and where the dowels are partially driven to form the connection. What is clear from the preceding research into all-timber connections is that the embedding stiffness of the fastener is a crucial characteristic in determining the overall stiffness of an all-timber connection.

5.4. Comments

Current guidance into the specification of non-metallic dowels is limited. This is primarily because these types of fasteners are not included in the current structural timber design code, BS EN 1995-1-1 and the complementary standards. Further research of all timber connections has reacted to its revival by providing research that has focussed predominantly on their inclusion in traditional carpentry connections. Analysis methods are therefore based on traditional connection types such as mortice and tenon connections and cannot be directly assumed to be applicable to different connection types without further research. Furthermore, as there are no independent comprehensive design guides, specification of these connections has to be undertaken from disjointed pieces of research that do not necessarily form a complete picture of the problem. This is in part a reflection of the research that has been undertaken and the different practices that exist for traditional all timber connection types throughout the globe. For instance, in South East Asia softwood framing members and softwood dowels are used to construct timber post and beam frames (Shanks, Chang & Komatsu, 2008). In the UK hardwood dowels and hardwood members are used to create timber post and beam frames whilst in the USA a combination of hardwood dowels and softwood members are used in the construction of timber post and beam frames.

Stiffness of connections affects the load transfer and distribution through any structure and, with regards to DLT panels this is particularly important. Stiffness of a connection is based on the slip modulus, which is derived from a linear appreciation of the load slip curves of differing metallic fasteners. For a timber fastener, there is an understanding in research that the rigidity of the fastener will play an important part it the overall stiffness of the connection and attempts have been made to model this using the exponential behaviour witnessed in the load slip curves or the shear span of the dowel as a critical function of stiffness. Research in this area has not advanced enough to provide clear answers for the efficiency expected for all-timber connection in the creation of composite sections. In this study research has been carried out to address the gaps in this research by analysing the connections that are in the panel individually and combining these results into the analysis of a full scale DLT panel.

6. Characterisation of Materials

Part 1: Evaluation of softwood

In order to define the strength and stiffness of a DLT panel it is necessary to gain a comprehensive understanding of the behaviour of the individual components of the panel and of the interactions between them. Within this chapter an appraisal of the required properties of both softwood and hardwood is undertaken. This includes; an assessment of the dimensional stability of UK Sitka spruce when it is dried below 18% moisture content; an evaluation of the strength and stiffness of UK Sitka spruce and larch

6.1. Mechanical appraisal of UK Sitka spruce and larch

During this PhD study, the SIRT network conducted an investigation into the mechanical properties of structural sections of Sitka spruce and larch for the creation of specific grade settings for a number of commercially available (or at the time near available) machine-controlled timber grading machines. Additional non-standard timber sections of UK larch and Sitka spruce were made available from the SIRT study for testing in this study in accordance with BS EN 408 (2010), to consider their suitability for use within an DLT panel and determine their mechanical properties. To represent the main areas of Sitka spruce and larch populations, samples were collected from different regions around the UK, Scotland, Wales and Northern Ireland.

6.1.1. Test procedure

To investigate the bending strength parallel to grain ($f_{m.k}$), the local modulus of elasticity in bending ($E_{m,l}$) and the global modulus of elasticity in bending ($E_{m,g}$) 70 Sitka spruce samples, shown in Table 6-1 were tested by the author in the summer of 2014 at Edinburgh Napier University. The determination of the three indicative mechanical properties was achieved in accordance with BS EN 408 (2010) by symmetrically loading a simply supported beam in bending about its edgewise orientation by two equally separated point loads (see Figure

6-1) on a Zwick Z050 universal testing machine (Zwick Roell, Germany) at a temperature of (20°C) within a humidity controlled laboratory (65% relative humidity). The beam had a minimum length of 19 times the height of the section and the span was 18 times the height in accordance with BS EN 408 (2010) (see Figure 6-2).

Sample set	Sample Size		Number of samples
	Width (mm)	Thickness (mm)	
South Scotland	100	40	10
Wales	100	40	10
Northern Ireland	100	40	50
		Total	70

Table 6-1: Sitka spruce bending samples

Figure 6-1: Four-point bending test on UK Sitka spruce.

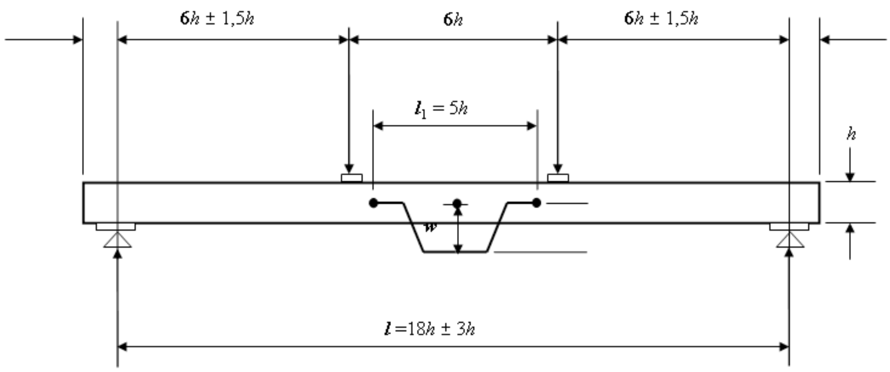

Figure 6-2: Testing arrangement for measuring the local modulus of elasticity in bending (Figure 1, BS EN 408, 2010, p.9).

Lateral restraint was provided at third points along the span to prevent lateral torsional buckling but still permitting the beam to deflect without significant frictional resistance occurring. In accordance with BS EN 384 (2010), a critical section where failure was expected to occur was selected and placed within the central loaded zone of the bending test show in Figure 6-2. Prior to conducting the experimental test, the mass of the test samples were recorded by a set of scales accurate to ±0.1g and the cross-sectional dimensions were measured with a set of digital callipers accurate to ±0.1mm. The dimensions of the sample were taken as the average of three measurements, one taken at the centre of the samples and the others 150mm from each end of the sample.

Load to the sample was applied at a constant rate of 0.4 mm/s, corresponding to 0.003h mm/s, until the beam reached 40% $F_{max,est}$ at which point the test was paused. The central external displacement measuring apparatus was removed and the sample was again loaded at 0.4 mm/s until failure occurred. The loading up until 40% $F_{max,est}$ was recorded to within an accuracy of 1% of the total load applied using a central transducer deflection measured using Linear Variable Differential Transducers (LVDT's) measuring devices to an accuracy of 1% to obtain a value of the Global Modulus of Elasticity. LVDT's set in a cradle either side of the sample recorded the deformation all the way to failure. The loading rate was adjusted such that failure occurred within 300±120 seconds.

After testing, a full cross section of the test specimen was cut close to the point of failure and the moisture content and density was determined in accordance with EN 13183-1 (2002). The measured values of global modulus of elasticity values and density were corrected to a 12% moisture content according to BS EN 384 (2010) and the bending strength was adjusted to the 150mm reference size using the k_h factor as stipulated in BS EN 384 (2010).

6.1.2. Results

The tests conducted on Sitka spruce, coincided with a large range of tests conducted by SIRT on the mechanical properties of UK larch for grade classification and machine grading calibration purposes. The methodology of the tests conducted on UK larch were conducted in accordance with BS EN 408 (2010) and BS EN 384 (2010) in exactly the same manner as the Sitka spruce samples, albeit on a much larger number (706) and range of samples. The results are shown in Table 6-2 for comparative and reference purposes.

Property	UK Sitka spruce	UK larch
Number, N	70	706
Strength, $f_{m,mean}$ (N/mm^2)	33.8	39.1
Strength, $f_{m,k}$ (N/mm^2)	16.8	21.2
Strength, COV (%)	19.2	30.7
Stiffness, $E_{0,12\%,mean}$ (kN/mm^2)	9.15	9.57
Stiffness, COV (%)	15.2	25.7
Density, $\rho_{12,mean}$ (kg/m^3)	410	494
Density, $\rho_{12,k}$ (kg/m^3)	336	406
Moisture, ω_{mean} (%)	10.8	13.9
Moisture, COV (%)	1.42	16.8
R^2 – Strength: Stiffness	0.43	0.56

Table 6-2: Comparison of the experimental mechanical properties for UK grown Sitka spruce and properties of UK larch published by Ridley-Ellis *et al*. (2015).

The results illustrated in Table 6-2 show that the UK Sitka spruce sampled should be classified as C16 grade in accordance with BS EN 338 (2009). Its determining property is its characteristic strength ($f_{m,k}$) mirroring the results from previous studies conducted on Sitka spruce (Moore *et al.,* 2009a; 2009b; Moore, 2011). Figure 6-3 illustrates the coefficient of determination (R^2) for the relation between strength and stiffness of UK grown Sitka spruce from this study. The coefficient of determination (R^2) value can normally be seen in the region of somewhere between 0.51-0.73 (Johansson, 2003) for the strength and stiffness of a piece of timber. Here the R^2 value for UK Sitka spruce is slightly lower than anticipated at 0.43 and 0.56 for UK. A reduction in the correlation is normally attributed to the presence of knots in the samples (Johansson, 2003) and the timber tested did exhibit a large number of knots (Figure 6-4).

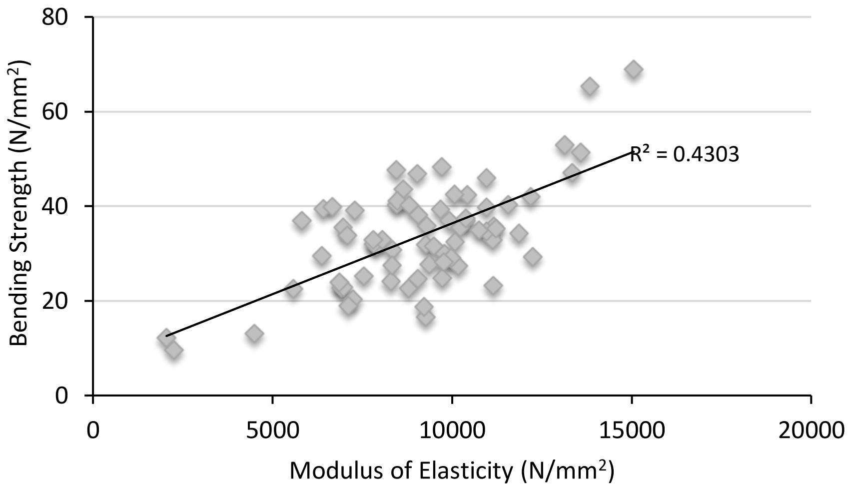

Figure 6-3: Strength vs. Stiffness of UK grown Sitka spruce.

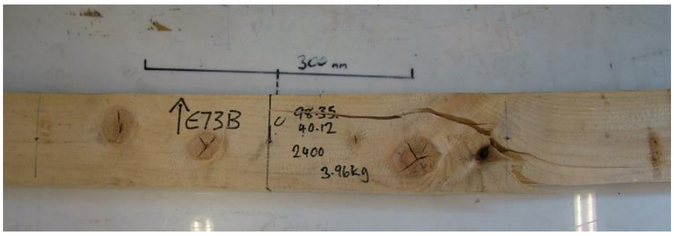

Figure 6-4: Example of a knotty specimen

6.1.3. Comments

UK grown Sitka spruce was widely expected to obtain the strength classification grade of C16, but some of the characteristic values are greater than the assigned grade classification. This is often the case with timber; the determining property is the lowest parameter relative to the grade classification; in this case for Sitka spruce the bending strength parallel to the grain ($f_{m,k}$). It should be noted that this was a relatively small sample (70 specimens) of structural timber and if more accurate grade classification was sought a much greater sample size and variety of section sizes would be required, as was witnessed for the UK larch samples.

The results illustrated in Table 6-2 show that UK larch can be expected to obtain the strength classification grade of C20 in accordance with BS EN 338 (2009). The characteristic density of the assigned strength grade, C20 is 330kg/m^3 in BS EN 338 (2009) whereas UK larch has a characteristic density of 406kg/m^3

approximately the same characteristic density as C30 (400kg/m^3). Judging by the appraisal of joint stiffness given in Table 5-2, UK larch has the potential to far outperform its grade class classification when used in connections, due to its much higher mean density (494kg/m^3) than timber typically assigned the same grade classification in BS EN 338 (2009), i.e. 370kg/m^3.

6.1.4. Acoustic grading study of UK Sitka spruce

The timber material supplied for study in this thesis was not pre-graded. In the future the variety of cross-sectional dimensions needed or the quantity required for DLT production means that there is no guarantee that the supplier or producers have access to automated machine graded processes or that timber will be produced in large enough quantities to make machine grading of the converted specimens economic. Previously discussed in section 3.1.9, acoustic grading provides a portable, low cost alternative for smaller timber suppliers to accurately grade timber material. Prior to commencing the destructive tests on the 70 Sitka spruce samples discussed in section 6.1 the specimens were weighed and their harmonic frequency was recorded using the MTG 960. The stress wave velocity of each sample was then calculated based on the formula given in section 3.1.9.2. The strength of a piece of timber can vary wildly due to local defects along the piece of timber and will not always correlate well with the stress wave velocity (Johansson, 2003; Wang, 2013). Therefore, the dynamic modulus (E_d) was calculated based on the relationship given in 3.1.9.1 for all 70 samples.

A linear regression was calculated between the static elastic modulus of the 70 specimens derived from the testing conducted in section 6.1 and the calculated dynamic modulus obtained from the stress velocity measurements, see Figure 6-5. The results in this PhD study produced an R^2 of approximately 0.69. Ivković *et al.* (2009) showed that in some instances the R^2 between MOE_{dyn} and MOE_{static} can be as low as 0.31 but good levels of correlation were witnessed between the static and dynamic modulus for UK grown Sitka spruce and this aligns with research conducted by Jones and Emms (2010). Moore *et al.* (2008)

explained that the strong radial distribution of stiffness within a log means that sawn processed pieces of timber (that are more likely cut close to the centre of the log) will have a greater correlation between MOE_{dyn} and MOE_{static} and the larger number of specimens used from centre cut materials tends to corroborate these findings.

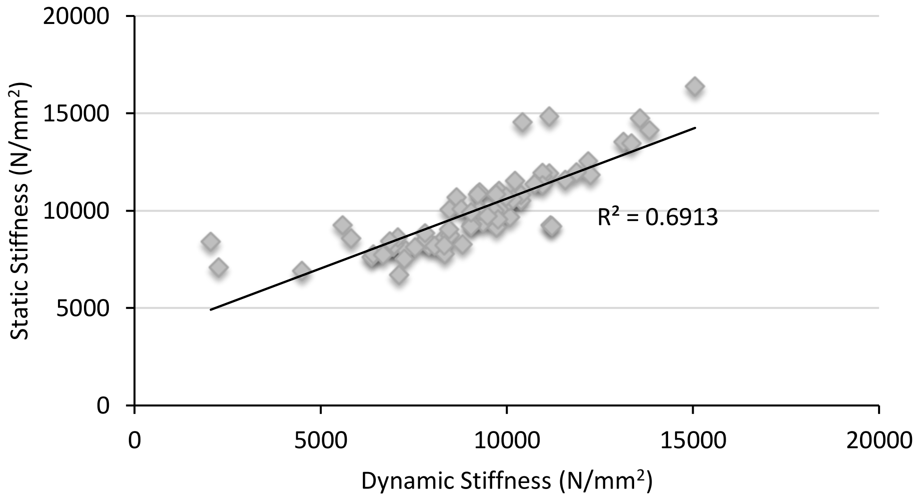

Figure 6-5: Dynamic Elastic Modulus Vs. Static Elastic Modulus of UK Sitka spruce.

The high level of correlation shown between static and dynamic stiffness assessments of the same samples in this study has shown that an accurate and reliable assessment of the stiffness of laminations can be achieved through the use of acoustic grading devices and specifically the MTG 960 is suitable for UK Sitka spruce. Subsequently each individual piece of UK Sitka spruce used from in this thesis for further study can be non-destructively graded and classified prior to testing. Furthermore, a study conducted into the use of UK larch by Ridley-Ellis *et al.* (2015) provided the proposed settings required for the use of the MTG 960 in grading UK larch and hence they can be classified using the same principle.

6.2. Drying stability assessment of UK Sitka spruce

A preliminary assessment of the dimensional stability of Sitka spruce was conducted by the author of this study alongside colleagues for COCIS. The investigation was undertaken to understand the amount of timber that could be expected to be rejected when the UK resource (in this instance Sitka spruce) was dried to the moisture content level expected for EWP production. This distortion study was conducted through the first half of 2013 at the Forestry Commission's Northern Research Centre (NRS) and expands upon a study conducted by Crawford *et al.* (2015) carried out in 2013 by including an additional 30 samples of British Sitka spruce that were supplied from the South of Scotland.

6.2.1. Test procedure

The sample set comprised five differing cross-sections (see Table 6-3) above the fibre saturation point (i.e. green) which were then later kiln dried to 10±2% moisture content within a small research kiln at the NRS. The five sample cross sections shown in Table 6-3 below were chosen as they were the closest common target sizes to the expected lamination thickness of 20, 30 and 40mm. The samples were dried based on a moisture controlled drying schedule specifically for Sitka spruce (SS305) using the average moisture content based on the electrical resistance of electrodes driven into three boards within the drying stack. The boards selected to have the electrodes inserted into them were chosen to be the largest pieces within the centre of the stack. This ensured that the all the samples were dried to a level below the expected in-service moisture content of 12%.

Work Stream	Position within log	Nominal Green Dimensions			Number of samples
		Depth (mm)	Width (mm)	Length (mm)	
Southern Scotland Sitka spruce	Sideboard	22	150	2400	5
		22	100	2400	5
	Centre-cut	32	100	2400	5
		38	100	2400	10
		38	150	2400	5
				Total	**30**

Table 6-3: Distortion study samples

Before and after the kilning process, thickness and width measurements were taken using digital callipers at three points along the length of the beam. A handheld (GANN Hydromette M 4050) moisture meter was used to record the moisture content of the pieces of timber at the same three locations that the dimensional measurements were taken and the samples were weighed before and after drying to calculate the density. The measurement of shrinkage along the length of the sample was omitted, because it is known that the shrinkage longitudinally to the grain of a piece of timber is minimal in comparison to the other grain orientations (Dinwoodie, 2000) and would not be the critical shrinkage parameter. The kilning process took 87 hours in total to complete. The average moisture content of the samples before drying was 42.1% and after drying was measured at 7.32%.

The distortion in the cross sections was recorded before and after drying using a laser distortion-scanning device built by Freiburg University called the Freiburg's Improved Timber Scanner or simply the FRITS frame (see Figure 6-6). This involved resting the piece of timber on each of its faces or orientations in turn, while a laser travelled along a 2m length of the timber, measuring at regular intervals the difference in height, and its corresponding location along the length to ± 1mm (see Figure 6-7).

Figure 6-6: FRITS frame

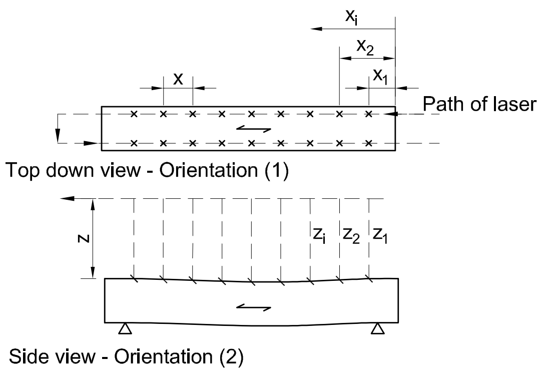

Figure 6-7: Process of measuring deformation using a FRITS frame.

From these measurements, a geometric model was created from the co-ordinates determined from the FRITS frame before and after drying. Using the procedure set out in BS EN 1310 (1997) for measuring the defects in a sample, the average bow, spring and twist for each sample was calculated before and after kilning. Measurements across the width of the sample were taken at regular intervals at two locations close to the edge of the sample see Figure 6-7. The procedure followed by the FRITS frame did not allow for verification of cup distortion. Additional measurements in the centre of the sample would have been necessary to gain an understanding of cup that occurs during drying but the alignment of the rig did not facilitate these additional measurements.

6.2.2. Shrinkage assessment of UK Sitka spruce

The first stage of the distortion analysis was to determine how much the dimensions of the timber altered when the moisture content of the timber was reduced beyond the usual levels of commercial drying (i.e. 18±2%). In order to give a full representation of the results the minimum, maximum and the average value of shrinkage (in percentage) for each sample are presented in Table 6-4.

Southern Scotland - Sitka spruce: Shrinkage										
Sample Set	Number of samples	Width (% change)			Thickness (% change)			MC (%)		
		Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean
100 x 22	5	3.32	4.61	4.00	0.00	7.13	2.43	6.90	7.37	7.11
100 x 32	5	1.78	3.79	2.71	1.70	4.64	3.03	6.87	7.93	7.44
100 x 38	10	1.56	3.95	3.08	0.93	5.46	3.31	7.07	8.97	8.09
150 x 22	5	4.10	6.09	5.02	0.06	7.76	3.58	5.40	7.40	6.61
150 x 38	5	2.66	4.97	3.49	0.46	6.69	2.63	6.97	8.10	7.37
Average (%)				3.50			3.00			7.34

Table 6-4: South Scotland shrinkage results

The overall average shrinkage witnessed across the width of the 30 samples was 3.50% and 3.00% across their thickness, when reduced to an average moisture content of 7.34%. Whilst the shrinkage across the thickness was fairly consistent irrespective of sample size (see Table 6-4). The shrinkage across the width varied depending on the original thickness of the sample. The largest proportional shrinkage through the thickness of a sample range was witnessed in narrower samples, indicating that the speed or amount of drying had an adverse effect on the amount of shrinkage that occurred in the section.

The results from this shrinkage study aligned with the study conducted on Sitka spruce samples from the North of Scotland by Crawford *et al.* (2015). In order to identify an influential property in pre-determining the expected amount of shrinkage the percentage change of the sample width was compared to the density of sample. No correlation between the values were witnessed (see Figure 6-8), but the results suggest that there is a vague positive correlation between the variables suggesting that the amount of expected shrinkage and density could be related. A further increase in the sample size however would be necessary to confirm this relationship.

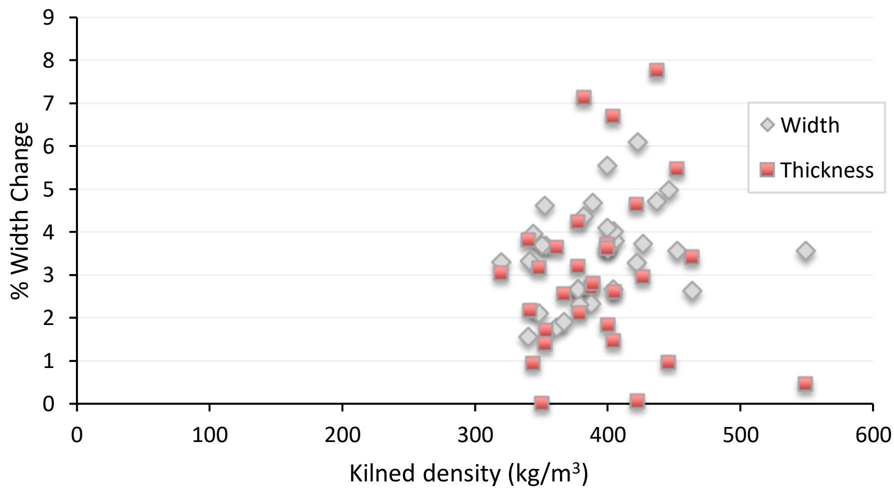

Figure 6-8: Relationship between density and % change of width of South Scotland Sitka spruce during kilning.

Following drying of the timber there is a pre-requisite that the timber used in DLT production is provided to the correct tolerance class, in this instance T2 (i.e. +1mm for the thickness and +2mm for the width, as shown in 3.1.3. It is therefore necessary that the cross-sectional dimensions of the dried samples are able to be machined further to achieve the correct tolerance. From the mean values of shrinkage obtained from the test conducted in this study an ideal green timber cross section can be calculated to ensure that the dried timber cross section can be machined post drying to below 18% moisture content.

The suggested starting green target sizes based on these shrinkage experiments are shown in Table 6-5. The suggested original green target sizes are slightly larger than the cross sections supplied to the UK construction industry (Table 3-1). This will present a challenge in procuring the necessary green timber cross section sizes unless the proposed lamination sizes for DLT are reduced to take into account the expected shrinkage or the upstream supply chain is modified to account for the expected dimensional reduction, that occurs when drying beyond 18% moisture content.

Sample set	Required size (mm)		Average shrinkage (mm)		Target size (mm)	
	Width	Thickness	Width	Thickness	Width	Thickness
100 x 22	100	20	5	1	107	22
100 x 32	100	30	3	1	105	32
100 x 38	100	40	4	2	106	43
150 x 22	150	20	9	1	161	22
150 x 38	150	40	6	2	158	43

Table 6-5: Green timber starting dimensions based on shrinkage data

6.2.3. Distortion assessment of UK Sitka Spruce

The boundaries given to the distortion criteria in BS EN 14081-1 (2011) are often the critical factor in determining the capability of a piece of timber to be incorporated into a DLT (or an EWP) when it is dried to the same level as the expected service class, SC1. Due to the correlation between the shrinkage results obtain from the test conducted in this thesis with the study conducted by Crawford *et al.* (2015) the raw data from the FRITS frame analysis of the 60 samples from the North of Scotland were added to the 30 samples measured for this thesis from the South of Scotland. The results were integrated and jointly analysed, to provide a larger sample set in which to evaluate distortion.

The average combined values of bow, spring and twist before and after kilning are shown in Figure 6-9 for 90 specimens from both Northern and Southern

Scottish Sitka. All the samples were then assessed against the three visual override criteria given in 3.1.6 to form an estimate of the reject percentage based on the distortion if material was kiln dried ready for use in a DLT panel. On average, the values for both spring (7.43mm per 2m) and bow (7.13mm per 2m) fell within the criteria for C18 and above. After kilning the average twist (2.41mm per 25mm) of the sample was greater than the minimum requirements for C18 given in BS EN 14081-1 (2011).

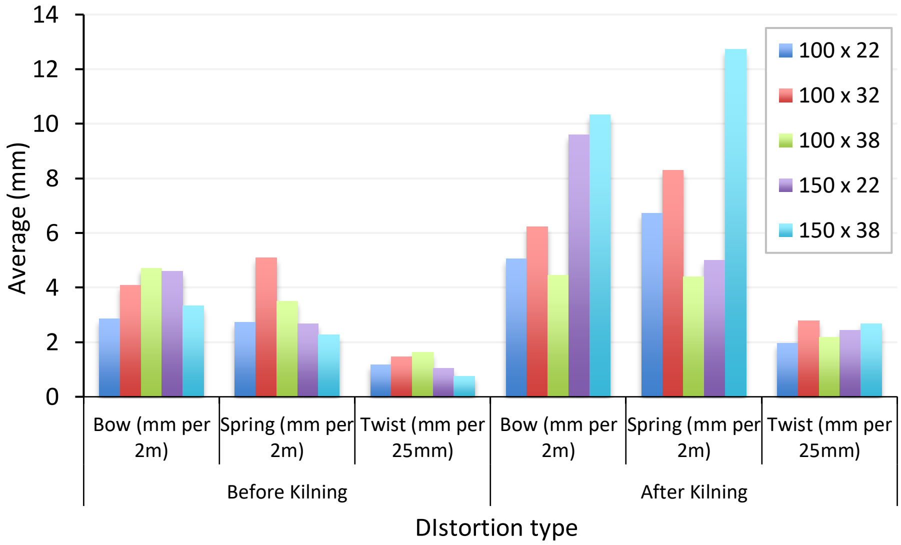

Figure 6-9: Mean distortion of Scottish Sitka spruce before and after kilning.

Twist is possibly the most stringent criterion given in BS EN 14081-1 (2011) and is commonly seen as the main distortion criterion for downgrading timber (Kliger, 2001). Indeed, twist measurements of Sitka spruce at a 12% moisture content undertaken by Searles (2012) varied from between 1.54mm to 2.37mm per 25mm. The results from this study averaged 2.52mm and ranged between 0.61mm and 6.06mm per 25mm for an average moisture content of 8.9% (including North Scotland samples). The increase in twist in this sample set can be explained by the findings from Canavan, Jarvis and De Borst (2013), who found that the amount of twist in Sitka spruce can increase greatly (almost double in centre cut logs) between 15% and 8% moisture content. An increase of 0.42mm in both the lower and upper bound values of twist was consistent with the variance in twist found by Searles (2012), suggesting that twist could occur at a linear rate until at least 8.9% moisture content.

Determining if there is a linear correlation between the amount of twist that occurs in a sample before and after kilning could confirm if twist increases in a linear proportion and therefore would provide a suitable method of predicting further twist in the sample. Determining accurately the expected amount of twist would allow for timber to be potentially rejected at an earlier stage in the conversion process for DLT or EWP production and diverted to other more appropriate production chains reducing waste and increasing productivity. Figure 6-10 illustrates that there is no correlation between the twist that is present in the green timber and the resultant twist that had occurred after kilning. As a result, predicting how the timber will twist, based on the amount of twist in the sample prior to kilning is not achievable. Further investigation is required into the macro and micro structure of the timber in order to inform timber processors how best to practicably, cut, dry and process the timber to limit these distortions.

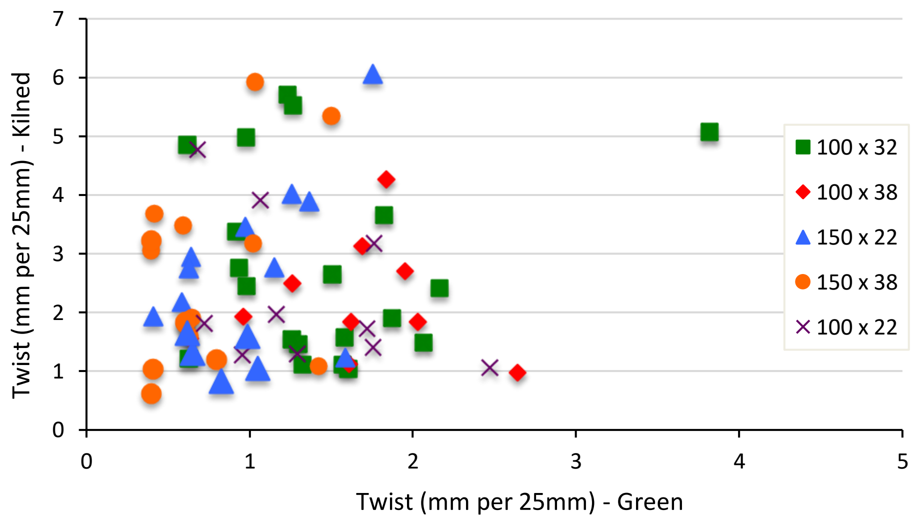

Figure 6-10: Comparison between twist of Scottish Sitka spruce pre and post kilning.

Canavan, Jarvis & De Borst, (2013) suggested an avenue to reduce the twist in a dried Sitka spruce is to avoid the juvenile centre of a tree by using side board or side cut material. The 150 x 22mm and 100 x 22mm side board section sizes used in this study did, on average, have a reduced amount of twist than the centre cut sections. However, the juvenile core of UK grown Sitka spruce is much larger than is normally witnessed in other conifers (Searles, 2012) and a sensitivity study will need to be conducted to ensure areas of the tree prone to large amounts of twist are avoided.

6.2.4. Comments

Research indicates an exponential increase of rejects is witnessed when the moisture content is reduced below 18% but the amount varies depending on the mode of distortion investigated (Canavan, Jarvis & De Borst, 2013). The outcome of the distortion study conducted in this thesis suggested that kiln drying UK Sitka spruce to the requirements for EWP production i.e. a 12±2% target moisture content (an average value of 8.9% was achieved for this study) will result between 40-90% of the material being classified as C18 or below. The current reject rate at UK sawmills when drying softwood to around a moisture content of 20% is approximately 6% (Crawford *et al.,* 2015). By considering the reject rate found from this study (45%) then the additional reject rate to dry UK Sitka spruce to ≈8% moisture content is in the region of 39%.

The study conducted in this thesis and the initial study by Crawford *et al*. (2015) had several limitations. To investigate the behaviour of a variety of different cross-sections, several different cross sections were included in a single batch during kilning. This variation in sample sizes in a single batch would not have occurred in a commercial oven. The rate of drying is dependent on the amount of evaporation that can occur for the surface of the timber, which will be dependent on surface area and temperature of the oven. Therefore, the rate at which drying would occur throughout the batch would not have been uniform.

The rate and intensity of drying was determined by the measurements taken from the in-situ electrodes. Installing the moisture sensors into the largest samples increased the speed and the amount of drying that took place in the smaller samples. The average moisture content in the smaller cross sections was therefore far lower than the intended target moisture content (10%±2%), causing additional distortion to take place. In a commercial situation, the settings of the drying cycle could be iteratively tweaked over time to improve yield and potentially quicken the drying time. Therefore, it is anticipated that the percentage yield would increase when considering commercial drying of UK Sitka spruce. Although, as much effort as possible was taken to replicate the drying practices undertaken in commercial settings, it was not feasible to apply a large degree of restraint to the drying stack or place a large amount of dead load on top of the drying stack, because facilities did not exist to do so. As a result, it is likely that the distortions recorded in this study were greater than would be expected on a large-scale commercial basis, where stack restraint and top loading would be applied.

The preliminary study undertaken in section 6.2 indicates that there are challenges surrounding the drying of UK grown Sitka spruce to moisture contents below 20%. Many of the issues surrounding the drying of timber could be iteratively dealt with through a greater understanding the timber species, the position of the cut cross section within a tree and the drying arrangements including top loading, drying position and drying rates. For UK larch it is unknown what the reject rate would be when if it was dried down to 12% moisture content. There are concerns that the comparatively high density in UK larch would adversely affect the distortion rate and therefore further research will be required to understand if this is indeed the case, and if so, how can it be mitigated. This comprehensive research into larch was not undertaken in this study but is to be included in further studies undertaken by SIRT.

6.3. Appraisal of UK softwood

Due to the immediately available quantities of UK Sitka spruce and larch (illustrated in section 3.1.1) both were considered for the production of DLT panels. Where the supply of this timber falls outside the normal streams of timber acquisition and averts the standard method of structural grading, more cost-effective methods of grading small batches of timber should be considered. In this chapter a validation of acoustic grading using the MTG 960 was undertaken, and the bending strength and elastic modulus of UK Sitka spruce was calculated for a selection of samples for areas within the UK. For the purposes of this study, a selection of unused timber from both the Sitka spruce study undertaken in this thesis and the larch study undertaken from Ridley-Ellis *et al.* (2015) was set aside to investigate the properties of UK produced DLT panels. All of which were then taken from a population where the indicative properties were known and would only require acoustic verification to determine its properties before inclusion within further experimental investigations undertaken in this thesis.

For DLT or EWP production the converted sections of timber are required at a tolerance and size that is not provided on a wide scale in the UK. Experiments undertaken in this chapter highlighted the challenges surrounding the provision of UK Sitka spruce at the necessary moisture content and dimensional tolerance. Both issues are intertwined. Drying the timber further beyond the UK industry standard causes a reduction in its target cross section whilst increasing the amount of distortion occurring in the sample. The study conducted shows that the sample size prior to kilning has to be enlarged or the cross-sectional sizes of the laminations used within a DLT panel need to be reduced accordingly. Following these findings, a 140mm deep piece of timber is to be specified for further use in a DLT panel to ensure that the remaining cross section (after drying) of 150mm deep section can be processed to the prerequisite tolerance.

Twist is the most prevalent distortion mechanism that occurs in UK Sitka spruce and can cause the samples to consistently warp beyond acceptable levels. The expected amount of twist in a sample cannot be determined by the extent of twist that had already occurred prior to kilning and no correlation exists between the density of the Sitka spruce sample and the amount of distortion occurring in the sample. At present the only ability to reduce the twist is to provide material that is not from the centre of the tree, but due to the enlarged area of juvenile material in the centre of a Sitka spruce tree this could be a tricky proposition. Further avenues for the reduction of distortion in kiln dried timber are to be explored through judicious cutting patterns in the timber and the iterative development of drying practices of the material. No significant body of research exists for the behaviour of UK larch during drying and further study is required, but was deemed beyond the scope of research for this thesis. To ensure that the least amount of distortion in further samples was limited all further samples to be used in this study were air dried in a SC1 environment until an equilibrium moisture content was reached at 12%±2% moisture content.

Part 2: Hardwood characteristics

Research carried out in 2014 by COCIS (Turnbull, 2014) into the production and properties of UK grown and produced hardwood dowels found no suitable method for producing cylindrical hardwood dowels using UK home grown hardwood. Given the level of tolerance needed, the volumes required and the relatively good availability of imported hardwood dowels, this meant in this research into the opportunities for a UK produced DLT panel, it was decided that it was not critical that dowels had to be sourced from UK grown hardwood timber.

The hardwood dowels supplied for the remainder of this project were therefore obtained in one large bulk order from one UK based supplier of hardwood dowels, called G & S Specialist Timber, Tools & Machinery based in Cumbria, England. The dowels were produced from European and North American hardwoods using a die cut process to ensure uniformity. In order to mirror the availability of UK hardwood species, a selection of ash (Fraxinus excelsior), beech (Fagus sylvatica), oak (Quercus robur) and sycamore (Acer pseudoplatanus) dowels were provided. In the UK, there is a precedence for the use of oak dowels in traditional post and beam timber frame so oak was added as an additional species for consideration. No exact provenance of the timber could be given but a series of material tests conducted in the following sections, provides the necessary material values for further investigation.

6.4. Moisture expansion of selected hardwood dowels

The requirement in DLT manufacture for dowels to swell is unusual in timber construction. Most research is directed towards understanding the shrinkage (and distortion) of timber as it dries in order to avoid damage that could occur either aesthetically or structurally. Publications such as Glass and Zelinka (2010) depict the shrinkage of a timber from green to oven–dry as a percentage of the green dimension. Whilst this is of interest for general timber construction use, it is not relevant for studying the behaviour of dowels in use within DLT panels.

For this PhD thesis a simple comparative analysis of the moisture expansion of a selection of 20.50mm diameter hardwood dowels was conducted by oven drying samples in accordance BS EN 13183-1 (2002) and BS 373 (1957) to a moisture content of 0% and assessing their dimensional properties as they reacquired moisture to a service environment of approximately 12% moisture content. The selections of samples are shown in Table 6-6.

Dowel species	Number of samples	Nominal dimensions (mm)	
		Diameter	Length
Ash	30	20.50	50
Beech	30	20.50	50
Oak	30	20.50	50
Sycamore	6	20.50	50

Table 6-6: Specimens used for the analysis of shrinkage and swelling.

The diameter of the dowel used was 20.50mm as this sized closely matched the diameter previously used on the continent to secure the laminations together and could be procured easily. The dowels were oven dried and then left in a service class 1 environment. To assess the volumetric expansion geometric dimensions were taken at weekly intervals at three locations along the dowel length in the two transverse planes (radial and tangential) and once longitudinally to the grain using a pair of digital callipers to an accuracy of 0.01mm. During these measurements, the dowels were also weighed to an accuracy of ±0.01g to calculate the moisture content and density according to the standardised process explained in BS EN 13183-1 (2002). The measurements were stopped when the dowels had been shown to have reached a moisture equilibrium with the environment over a consecutive set of weeks (in this instance approximately 10% moisture content over a duration of 4 weeks).

6.4.1. Results

From the recorded values, the average dimensional expansion change (i.e. swelling) could be quantified in both the radial and tangential directions for each dowel species, see Figure 6-11. The volumetric expansion expressed ignored the proportion of swelling that is attributed to the longitudinal direction due to the small amounts that would occur (Walker, 2006) and its irrelevance in securing the laminations of the panel together.

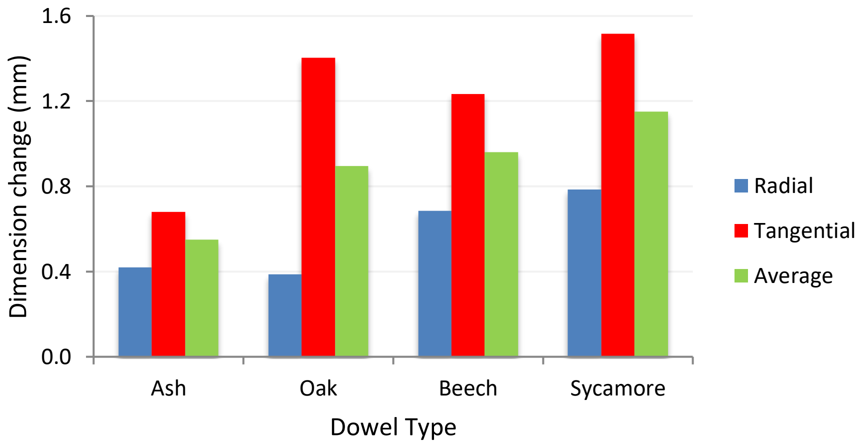

Figure 6-11: Dowel expansion from 0-10% M.C. of selected hardwood species.

Due to the anisotropic nature of timber, swelling or shrinkage does not occur uniformly across the cross-section. A key indicator of the dimensional stability of a piece of timber is the ratio of dimension change tangentially to radially across the grain, termed the T/R ratio. Table 6-7 illustrates the average amount of swelling that occurred in each of the dowel species and the T/R ratio.

Dowel species	Average swelling from 0% to 10% M.C. (mm)			T/R Ratio
	Radial (R)	Tangential (T)	Average	
Ash	0.42	0.68	0.55	1.62
Oak	0.39	1.40	0.90	3.62
Beech	0.69	1.23	0.96	1.80
Sycamore	0.79	1.52	1.15	1.93

Table 6-7: Expansion results for selected hardwood species from 0% to ~10% M.C.

6.4.2. Discussion

In DLT production the arrangement or orientation of the dowel is critically important. The creation of the necessary force to fuse laminations together in a DLT system will be dependent on the normal force caused by the dowel expansion, the friction coefficient at the point of interaction and the relative embedment strength of the conjoining materials. The friction coefficient will fluctuate due to the changing interaction that occurs between the softwood lamination and the hardwood dowel. A number of different factors including, grain orientation, moisture content, variation in dimensions and the roughness (finish) of both pieces of timber can affect the friction coefficient. For instance, the friction coefficient perpendicular to the grain of a piece of timber is expected to be higher than the friction coefficient parallel to the grain (Hirai *et al.,* 2008). Therefore, the friction force induced by the connection has the potential to be far lower if the dowel is orientated in such a manner that the largest amount of expansion occurs parallel to the grain of the laminations. Where a T/R ratio is closer to parity, the likelihood for smaller friction forces to be induced are reduced accordingly.

During panel production it is not expected that the insertion of the dowel will be undertaken with the control necessary to specify a specific grain orientation. This lack of certainty in the grain orientation of the dowels within the panel means that it is prudent to rely on the smallest expansion that occurs across the diameter to secure the laminations in place. A pragmatic approach is to ensure that the dowels with the largest and most uniform swelling in both directions (i.e. the lowest T/R ratio) are most suitable for DLT production and this will provide the most consistent connection. Based on this assumption the T/R ratio for ash beech and sycamore were consistent, however beech and sycamore expanded by 0.96mm and 1.15mm respectively, up to double the amount of ash (0.55mm). The small sample size of sycamore could affect the reliability of that result and would need to be investigated further to ensure that the greater expansion witnessed in this study was repeatable. Oak experienced much greater expansion in the tangential direction and could not be guaranteed to form a consistent connection due to the variability of the expansion occurring across the dowel indicated by the large T/R ratio (3.62).

6.5. Plastic moment capacity of hardwood timber dowels

The EYM described in section 5.1 defines the strength of a dowel type connection based on the geometry of the connection, embedment strength of the timber and the bending strength of the fastener based on its plastic moment. The plastic moment of a steel dowel is determined from a three-point bending test. When this test is applied to determine the effective bending strength or apparent yield moment of a timber dowel, the resulting defined properties would not be the yield capacity of a dowel but a value equal to its flexural strength that does not take into account the influence of shear deflection and the benefits of confinement in the connection (Sandberg, Bulleit & Reid, 2000). The determination of a relevant value for the plastic modulus of a timber dowel will need to include the effect that shear will have on a dowel over short spans.

Three-point bending tests have a tendency to induce a pure bending failure in the dowel, without incorporating inter-laminar shear failure caused by the confinement of the dowel within the connection that is often seen within an all-timber connection. Thomson *et al. (*2010) adopted an approach to characterise the moment capacity of GFRP dowels, through the use of a loading arm that mimicked the constraint of a dowel within a timber connection and thereby incorporated shear effects into the derivation of an effecting yield moment of a GFRP dowel. By applying these principles to a timber dowel, the dowel can be constrained from failure in bending by a central plate and the formation of hinges and the inter laminar shear failure that can occur in the dowel in an all timber connection can be mimicked. Based on the procedure outlined in Thomson *et al*. (2010) a bespoke test rig was created to allow the loading of the dowel over very small shear spans. Figure 6-12 shows the bespoke test rig.

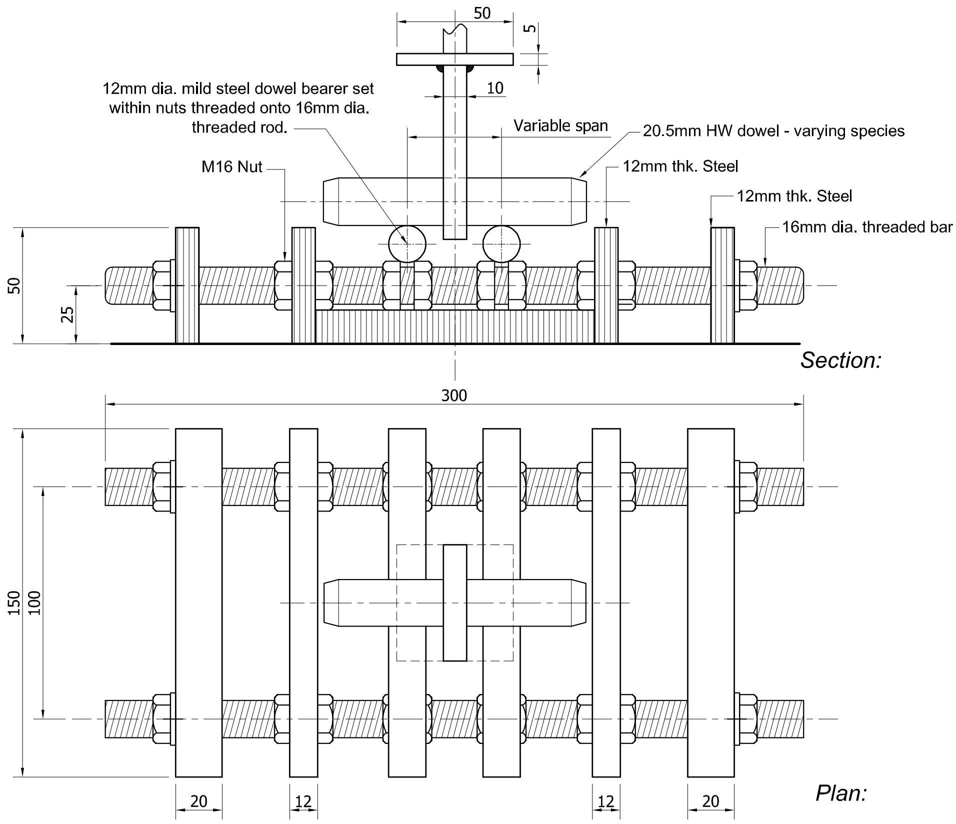

Figure 6-12: Test rig for testing effective bending stiffness of hardwood dowels.

The loading arm was created from a 10mm thick mild steel plate that was predrilled with a 21mm diameter hole. The timber dowel specimen was then placed through the hole and aligned correctly onto the support beneath. The steel loading arm was then used to uniformly load the centre of the dowel while providing the restraint to mimic the confinement of the dowel within a connection. The points of the base support could be altered precisely to achieve accurate spans over short distances. The shear span was simply derived from the point of support to the edge of the loading arm, illustrated in Figure 6-13.

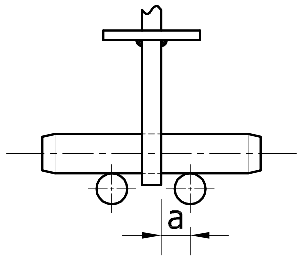

Figure 6-13: Shear span of dowel.

6.5.1. Test procedure

In March 2015, experimental three point bending tests were conducted on four hardwood species over a series of five different spans (see Table 6-8) using the bespoke testing rig, see Figure 6-12.

L/D ratio	Clear Span (mm)	Shear Span (mm)	Number of tests			
			Ash	Beech	Oak	Sycamore
1.5	30	10	5	5	5	5
2.0	40	15	5	5	5	5
2.5	50	20	5	5	5	5
3.0	60	25	5	5	5	5
4.0	80	35	5	5	5	-
		Total	**25**	**25**	**25**	**20**

Table 6-8: Test matrix for yield moment calculation.

All the dowels were orientated so that the load to the dowel was applied tangentially to the grain to the load to ensure the worst-case result. Each sample was loaded at a rate of (1.5mm/min) on a Zwick (Zoell, Germany) universal testing machine so that the failure of the dowels occurred in 300±120s from the start of the test, see Figure 6-14. Displacement of the loading arm and the load applied were logged continually throughout the test using platen displacement. Following the tests, the entire failed sample was used to calculate its density and the moisture content of each sample in accordance with BS EN 13183-1 (2002).

Figure 6-14: Adapted three-point bending test in operation

The mean values of plastic load resistance (P_p) were calculated from the recorded slip results and tabulated for each species and shear span. The approach utilised the 5% offset method defined in ASTM D5764-97a (2007) whereby the plastic load resistance (P_p) or yield load, was defined by the gradient of the line that passes through $0.2F_{max}$ and $0.4F_{max}$ with a regression of at least 0.99. The line was then offset by 5% of the diameter of the dowel and the intercept of the line with the load slip plot is taken as the yield or plastic resistance load (P_p). The proportional limit of a connection seen in Figure 6-15, was defined by the point in the load deformation graph where the curve becomes non-linear (Soltis, Hubbard & Wilkinson, 1986), see Figure 6-15.

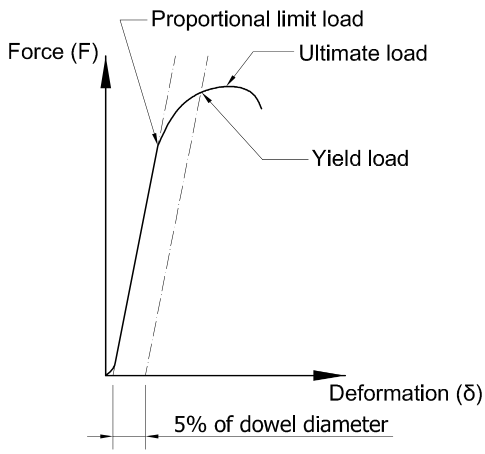

Figure 6-15: Determination of the yield load using the 5% dowel diameter offset method.

6.5.2. Results

The tabulated results for yield load, ultimate load and density are presented as mean values (due to the sample size of five for each test series) and are shown in Table 6-9.

Dowel species	Total clear Span (mm)	Yield load (kN)	Mean Ultimate load (kN)	Mean Proportional limit load (kN)	Mean density, ρ (kg/m³)
Beech	30	7.79	9.20	3.33	753
	40	6.57	7.69	2.46	737
	50	8.64	9.02	3.39	755
	60	6.36	6.92	2.74	730
	80	5.69	6.08	2.12	709
Ash	30	7.72	10.11	4.07	703
	40	7.89	8.30	3.73	703
	50	6.53	6.82	3.85	687
	60	5.69	6.08	2.12	709
	80	6.06	6.34	3.16	720
Oak	30	5.59	5.78	2.38	656
	40	5.09	5.56	2.19	656
	50	5.19	5.33	1.95	689
	60	4.31	4.52	2.09	661
	80	3.21	3.27	1.69	632
Sycamore	30	4.57	5.30	1.91	547
	40	4.27	4.56	1.68	546
	50	4.33	4.69	1.74	560
	60	3.15	3.48	1.33	536

Table 6-9: Three point bending test results

The typical load displacement curves for each species is shown in Figure 6-16. Ash, beech and oak experienced a linear load deformation response up until approximately 1.5-2mm deflection, followed by a non-linear response with a small plateau of nearly plastic load deformation occurring immediately following an initial failure of the dowel. Two distinct drops in strength occurred, first when slippage between the fibres occurred or when brittle bending failure was constrained by the loading arm and second when an abrupt brittle shear failure occurred directly after maximum load was reached.

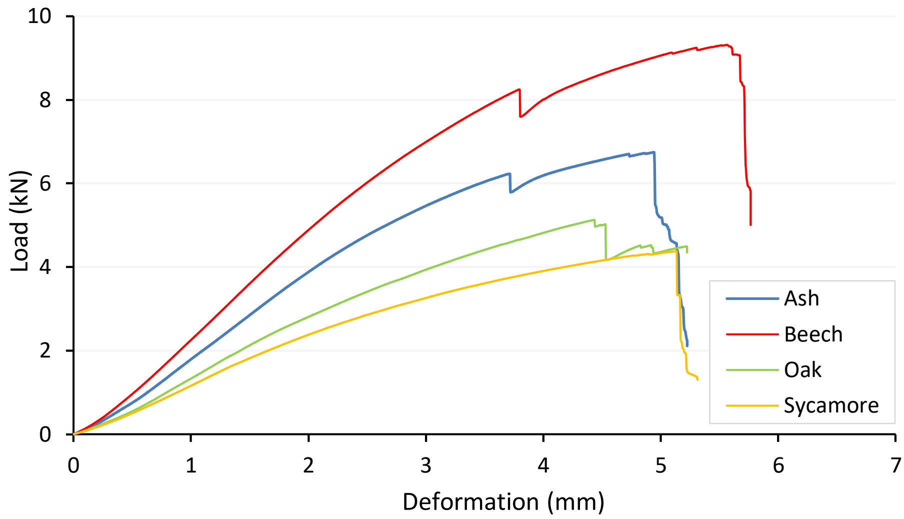

Figure 6-16: Typical load deformation plots for select hardwood dowels at 2.5D span under three-point bending

In the three-dowel species, ash, beech and oak there is a suggestion that a small amount of plasticity is produced by the bending resistance of the dowel and the constraining action between the dowel and the central loading plate that is created after the initial strength slips. Small amounts of extended post yielding were witnessed predominantly in ash and beech over shear spans between 0.5 and 2 times the diameter but they occurred less frequently in oak and yielding was almost non-existent in sycamore. The small amount of post yielding witnessed was not believed to be caused by the tension edge of the dowel being fully constrained by the bottom edge of the loading arm, but rather a combination of crushing and embedment in the dowels at the point of load application and at points of support, similar to a type V_d failure outlined in section 5.2.1.4

Research by Shanks and Walker (2009) has shown that there is a quadratic relationship between bending strength and shear span. Based on this relationship, the yield load can be expressed as a function of shear span (a) and the energy dissipated by the dowel as it yields in a three-hinge failure can be derived from its yield load capacity. Each quadratic relationship (shown in Figure 6-17) can then be incorporated into the expression for the internal energy dissipation given in section 5.2.2. For example, peak load for a die driven beech dowel from this study can be expressed by the quadratic equation, given in Figure 6-17 as:

$$P_p = \frac{4Z}{a}(-0.0039a^2 + 0.0663a - 7.2829) \qquad (6.1)$$

Where:

Z Elastic section modulus of the dowel, mm^3

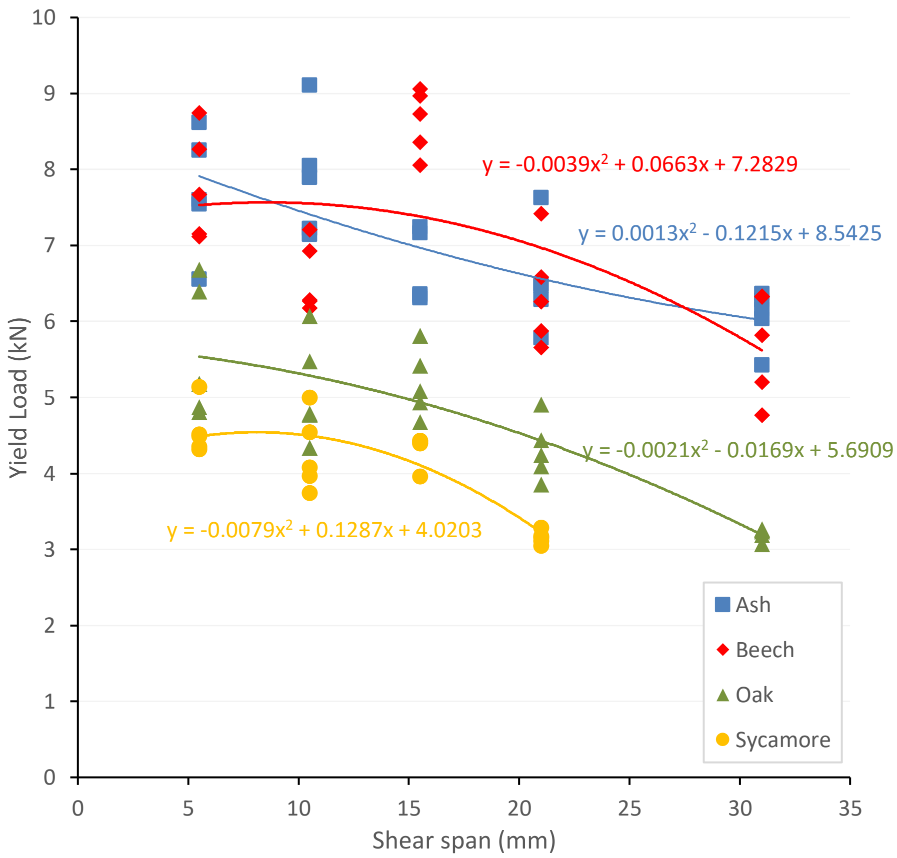

Figure 6-17: Graph illustrating the ultimate bending resistance of a 20.50mm dowel as a function of the shear span.

Figure 6-18 shows the load response of an ash dowel loaded over a shear span of 0.5 times the dowel diameter. Following the initial slip and settlement of the arrangement a linear response was seen. At approximately half the ultimate load, crushing of the timber immediately below the loading arm and embedment at supports caused a non-linear response with a notable reduction of stiffness to occur at lower levels. Final failure at lower spans occurred in a more gradual manner, due to the resistance provided by the base of the loading arm as the dowel rotated further on its supports. Over longer clear spans a brittle failure occurred without the elongated portion of non-linear response, that was seen over lower spans.

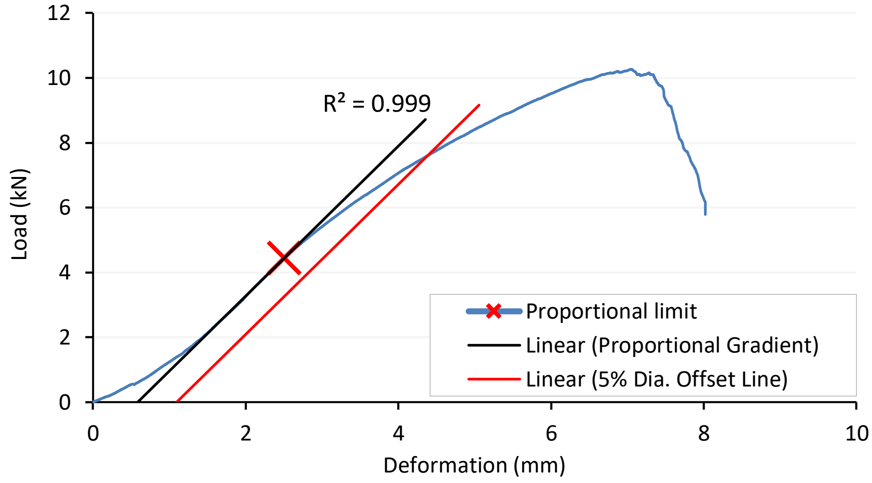

Figure 6-18: Load deformation curve of an ash dowel, with a shear span (a) of 0.5D illustrating early signs of crushing and embedment at supports.

Figure 6-19 illustrates the abrupt failure immediately after the ultimate load is reached followed by a small increase in strength as the sample was confined or wedged by the loading arm. During the wedging it is postulated that crushing and embedment continued allowing the sample to rotate further and lose the confinement that is provided by the base of the loading arm (see Figure 6-20) and cause the eventual abrupt brittle failure of the dowel.

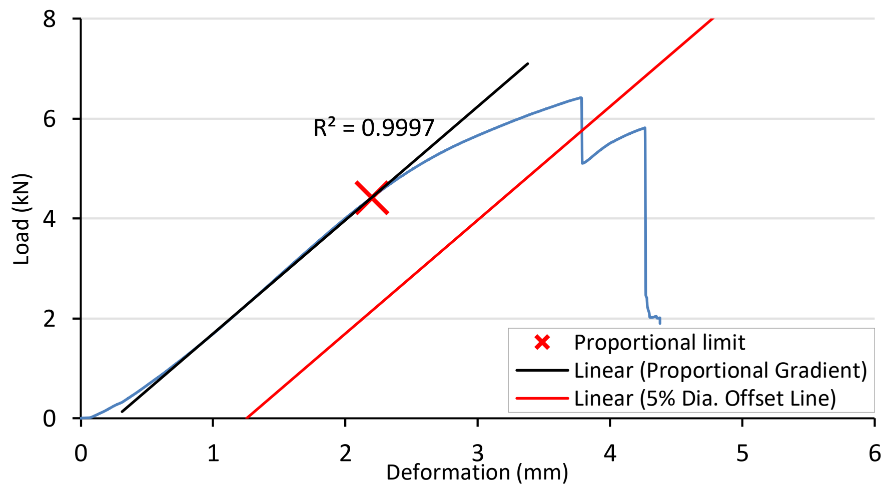

Figure 6-19: Load deformation curve of an ash dowel, with a shear span (a) of 1.25D, illustrating wedging occurring after the ultimate load is reached.

The diameter of the hole drilled through the steel plate was approximately 0.5mm greater than the nominal diameter of the hardwood dowels. This difference allowed for a small tolerance of approximately between 0.25-0.5mm, so the dowel would be positioned in the loading arm. The small slip that occurred at the beginning of the test is accounted by the loading head coming to meet the sample and so varied depending on the diameter of the dowel. However, this created a small gap between the base of the hole in the loading head and the bottom edge of the dowel being loaded. The gap did not fully allow the fibres on the dowel bottom face of the sample to be fully restrained when loaded. Instead a partial restraint to dowel rotation was provided that was dependent on the initial tolerance of the sample.

6.5.3. Comments

The timber dowels in this arrangement show the propensity of the timber to embed at point of supports and crush directly beneath the loading arm. Localised crushing in the top edge of the dowel would locally constrain the top edge through strain actions allowing the bottom edge of the sample to rotate further as an increasing gap between the beam edge of the hole in the loading arm and the base of the dowel was created, see Figure 6-20.

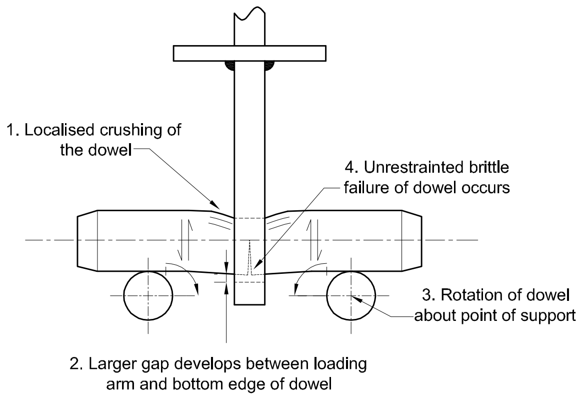

Figure 6-20: Dowel constraint during loading using bespoke loading arm.

The greater the difference between the dowel diameter and the predrilled hole in the steel plate the smaller the restraint to the tensile fibres in the bottom edge of the dowel. Where larger rotations in the dowels were allowed to occur the width of the loading arm played a pivotal part in restraining the dowel rotation. The closer the gap the higher the likelihood that the post yield phase would be extended because of the frictional resistance between the members and the dowel.

The results presented in Table 6-9 are the stiffness characteristics of the dowel under bending, shear and embedment. High levels of bearing at the support and crushing at the top of the load plate (see Figure 6-21) removed the potential for stiffness to be calculated. Where the materials are formed of two materials with similar bearing strength and stiffness, embedment will occur in both materials and this cannot be replicated while the dowel itself is supported on a mild steel test apparatus that aims to constrain the dowel during loading.

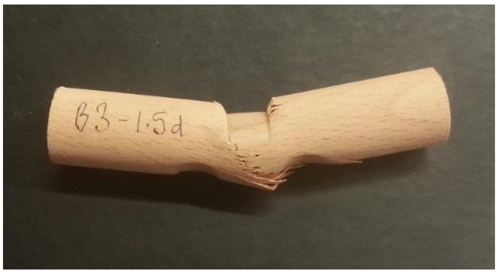

Figure 6-21: Crushing and bearing failure of a dowel accompanied with brittle tensile failure.

During this study the width of the loading arm was set at 0.5 times the diameter of the dowel being tested to allow a three-hinge formation to be induced in the dowel and the internal energy dissipated quantified. To mimic with visual certainty the formation of hinges in the dowel, the width of the loaded arm will need to be increased to ensure that the formation of four hinges along the length of the dowel. Otherwise the formation of the hinges will take place in too close a proximity to one another and would conjoin in the centre of the sample. The results for plastic moment resistance of the dowel were seen as valid, as the value was derived from a 5% diameter offset of the gradient of the line that was defined from the proportional load deformation of the load slip response. At no point did the intersection of the 5% offset line yield a smaller result than the proportional limit or occur at a point where failure had already occurred in the sample. Here the yield load can be seen as a combination of flexural strength, embedment strength and a small portion of confinement of the tensile fibres in the dowel.

Part 3: Connection characteristics

6.6. Embedment

The EYM considers embedment strength of the timber side members as a system property that encompasses material and geometric parameters. The embedment characteristics of the dowels and the surrounding lamination materials need to be fully quantified before an appreciation of the global embedment behaviour of a connection can be corroborated. The deformation of timber during embedment is distinctly different depending on the grain alignment of the timber while it is being loaded. Figure 6-22 illustrates the deformation characteristics of timber under embedment in both grain directions.

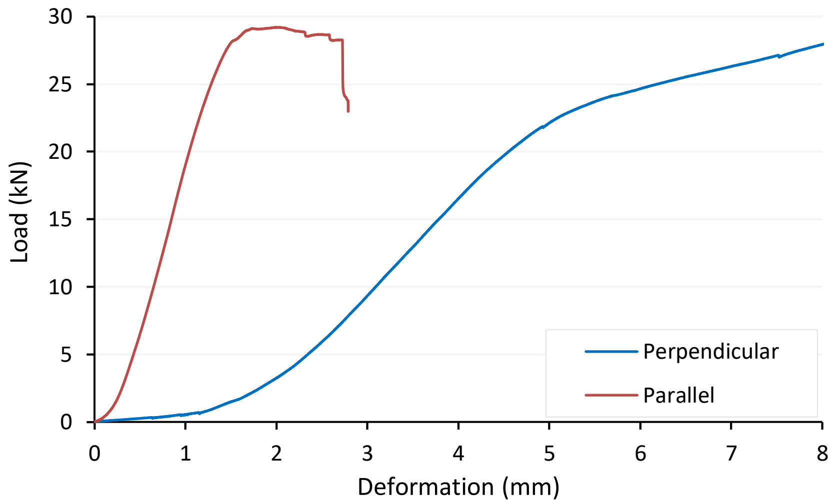

Figure 6-22: Typical load deformation curve for timber embedment.

During embedment parallel to the grain the load deformation curve exhibits a quasi-ideal plastic behaviour with a high slip modulus (Franke and Magnière, 2014b). When loaded in the perpendicular to grain orientation embedment deformations exhibit a bi-linear response with an increased stiffness occurring when the fibres in the timber expand laterally and are restrained from expanding (Gattesco, 1998). Leijten and Köhler (2004) described the observed hardening as a chord or cable affect, the amount of which is dependent on fastener diameter and the ability of the timber to deform in the embedment zone (Schoenmakers, Jorissen & Leijten*.* 2010).

6.6.1. Embedment strength

The two prominent standardised test methods for ascertaining the embedment properties of a piece of timber described in BS EN 383 (2007) and ASTM D 5764-97a (2007) differ in their application of load to the dowel and the derivation of the embedment strength (Rohana, Azmi & Zakiah, 2011; Xu *et al.,* 2014). The most recent standards agree that an arbitrary offset to the linear portion (i.e. up to the proportional limit) of the load deformation curve should be used to classify the yield strength and embedding strength of a fastener (BS EN 383, 2007; ASTM D5764-97a, 2007). The embedment strength for both parallel and perpendicular to the grain samples is defined as the maximum stress within a 5mm displacement limit in BS EN 383 (Leijten, Köhler & Jorissen, 2004), whereas in ASTM D5764-97a the yield load is evaluated as the point where the load deformation curve meets the extrapolated linear line of initial stiffness (i.e. the linear portion of the load deformation curve) offset by 5% of the dowel diameter, see Figure 6-23.

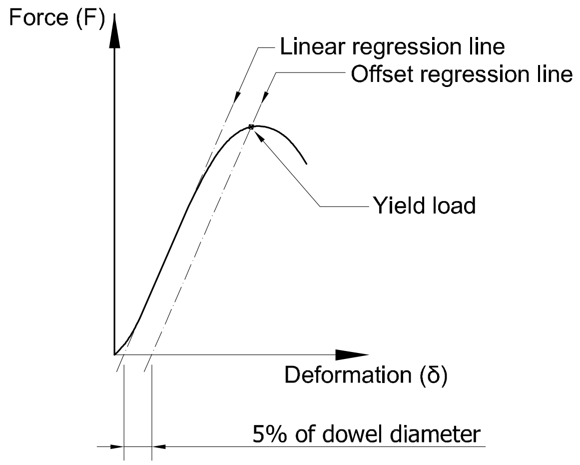

Figure 6-23: Derivation of yield load from load deformation plot.

Adoption of the 5% offset method has some advantages for the determination of yield load as it provides a more accurate and repeatable method of obtaining joint capacity (Schmidt and Daniels, 1999) and has less variability (Wilkinson, 1992; Sawata and Yasmura, 2002). Another main difference between the two standards lies in their application of load to the fastener. The ASTM D5764-97a (2007) applies a uniform load (in the half hole setup), in the European standard, BS EN 383 (2007) the dowel is loaded on either side of the timber sample. Whale and Smith (1989) hinted that the half hole arrangement does not give an accurate assessment of the stresses around the fastener but does give a result free of an influence of fastener bending (Franke and Magnière, 2014b) and allows the ends of the fastener to rotate (Hassan *et al.,* 2014) unlike BS EN 383 (2007), which causes additional bending to be induced in the fastener being loaded and is not beneficial when considering timber dowels.

6.6.2. Embedment stiffness

BS EN 383 (2007) illustrates a method for obtaining the initial foundation modulus of a connection, but ASTM D5764-97a (2007) does not provide a formal method for defining the stiffness of the arrangement. The BS EN 383 (2007) calculates the embedment stiffness also called the initial foundation modulus, κ_i from the secant modulus of the load deformation curve at approximately 40% the maximum load. The arbitrary value of 40% of gives a reliable approximation of the stiffness whilst limiting the amount of embedment occurring within reasonable limits. However, there is no set limit on the amount of embedment that is allowed to occur and further consideration can be given for the joint slip at the different limit states. For comparative purposes the initial foundation stiffness is determined from the secant modulus of the load deformation curve at 40% maximum load.

The initial foundation modulus can then be normalised to include the geometric properties of the test piece to provide a comparative value through the equation.

$$K_i = \frac{\kappa_i}{d \times t} \tag{6.2}$$

Where:

K_i Foundation modulus, N/mm^2

κ_i Initial foundation modulus, N/mm

d Diameter of dowel, mm

t Thickness of test piece, mm

6.6.3. Test procedure

The tests undertaken in this PhD study were in accordance with the ASTM D5764-97a (2007) half hole procedure which uses a streamlined method for obtaining the strength in both grain directions. Tests were conducted with the load applied to softwood timber samples in both grain orientations, parallel and perpendicular to the grain. 10 samples were conducted in parallel to the grain embedment and 20 samples in perpendicular to the grain embedment for both UK grown Sitka spruce and larch.

The samples were prepared by using a mitre saw. 10 test samples of nominal dimensions 82mm wide x 120mm long x 32mm thick were cut from planed clear sections of UK grown larch and Sitka spruce for both the perpendicular and parallel orientation to the grain. The sample was then positioned in a jig and predrilled with a 21mm diameter spade drill bit and the top portion trimmed according to ASTM D5764-97a (2007), see Figure 6-24.

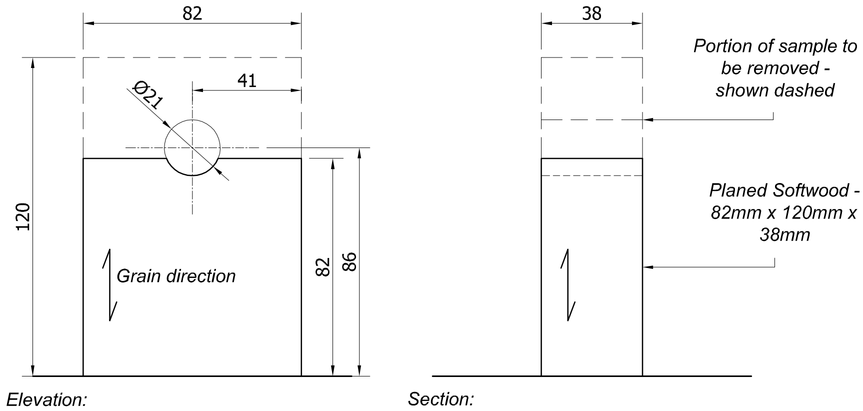

Figure 6-24: Schematic view of sample preparation

According to Jumaat, Razali & Rahim. (2008) embedment strength will remain constant independent of the dowel diameter when the average density of the softwood samples is below approximately $900kg/m^3$, therefore as the density of the softwood samples of both UK Sitka spruce and arch were shown to be under this value in previous tests (see section 6.1) only a single dowel diameter was tested. For these tests a 20.50mm mild steel dowel matching the proposed size of the timber dowels for DLT panels was used to determine the embedment strength of the timber foundation, see Figure 6-25. The use of a mild steel dowel is referenced in the standards as it is comparatively stiff and will not deform at the levels the timber is expected to fail, furthermore using the same diameter steel dowel as the proposed timber dowels allows the close comparison of the distribution of stresses that will occur around the hole.

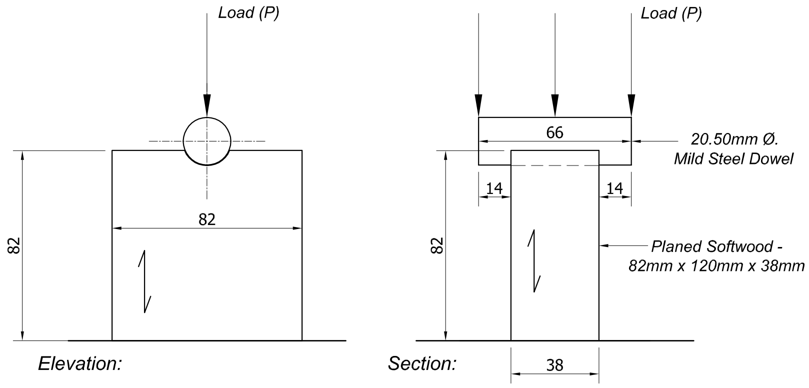

Figure 6-25: Embedment load application.

In some instances where dowels are protruding from the side of the connection, this could add to the axial withdrawal of the connection and has been witnessed in tests undertaken previously on traditional mortice and tenon connections (Shanks and Walker, 2005) and the same dowel protrusion is maintained throughout all tests undertaken going forward in this study. The steel dowel was manufactured to ensure that the same length of protrusion from each side of the dowel will be maintained throughout all tests. Each test sample was given a unique identification number, written upon the sample with an indelible marker. Prior to testing each sample was conditioned in an environment having a relative humidity of (65 ± 5%) and a temperature of (20 ± 2)°C. The width, the overall height and the height to the underside of the predrilled hole for each sample was measured immediately prior to commencing the test.

All specimens were tested using a Zwick (Zoell, Germany) universal testing machine, deformation was recorded as the crosshead displacement of the testing rig and the load applied was taken directly from the load cell. All values of load and displacement were recorded continually throughout the tests. Load was applied at a constant rate of 0.3mm/min and adjusted such that failure occurred within 300 ± 120 seconds. The test was stopped when failure occurred or when at least a 7mm displacement had occurred in the sample. Density and moisture content of each sample was then calculated immediately after testing in accordance with BS EN 13183-1 (2002).

Figure 6-26 Perpendicular to grain embedment testing

6.6.4. Results

The embedment yield strength was calculated in accordance with the 5% offset method illustrated in ASTM 5764-97a (2007) and the foundation modulus was determined from the initial foundation modulus determined from the secant modulus of 40% the maximum load. The mean and 5th percentile characteristic values in accordance with BS EN 14358 (2006) for the embedment yield strength, foundation modulus and density for both UK grown larch and Sitka spruce are shown in Table 6-10, overleaf. The results followed the classic load deformation of both parallel and perpendicularly loaded timber, previously depicted in Figure 6-22. Samples loaded parallel to the grain exhibited quasi-ideal plastic behaviour, with a maximum load being achieved between 2-3mm deformation followed by a plastic response and final abrupt failure, see Figure 6-27 (overleaf). A greater variation in the embedment strength of UK larch samples (COV 22.58%) caused the characteristic embedment strength of the UK larch to be lower than Sitka spruce, but in all other aspects the embedment strength and stiffness of UK larch was superior to UK Sitka spruce.

Property	UK larch		UK Sitka spruce	
	Parallel, 0°	Perpendicular, 90°	Parallel, 0°	Perpendicular, 90°
Number, N	10	20	10	20
Mean embedment strength, $f_{h,mean,\propto}$ (N/mm²)	32.35	11.69	27.31	7.37
Characteristic embedment strength, $f_{h,k,\propto}$ (N/mm²)	**21.85**	**8.29**	**23.45**	**6.11**
Strength, COV (%)	22.58	9.70	6.94	12.43
Initial Modulus, $k_{i,mean,\alpha}$ (kN/mm)	**20.81**	**8.75**	**15.92**	**8.15**
Foundation Modulus, $K_{i,mean,\alpha}$ (kN/mm²)	**26.24**	**9.65**	**20.50**	**7.44**
Foundation Modulus, COV (%)	21.61	20.55	13.02	18.15
Yield load, $f_{y,mean}$ (kN)	25.94	17.14	21.45	16.08
Yield, COV (%)	17.78	11.03	7.47	12.18
Density, ρ_{mean} (kg/m³)	519.14	499.25	382.33	497.40
Density, ρ_k (kg/m³)	375.25	433.09	338.33	363.69
Moisture, ω_{mean} (%)	13.59	13.14	10.93	11.05
Moisture, COV (%)	5.31	3.29	1.64	3.99

Table 6-10: Results from conducted embedment tests

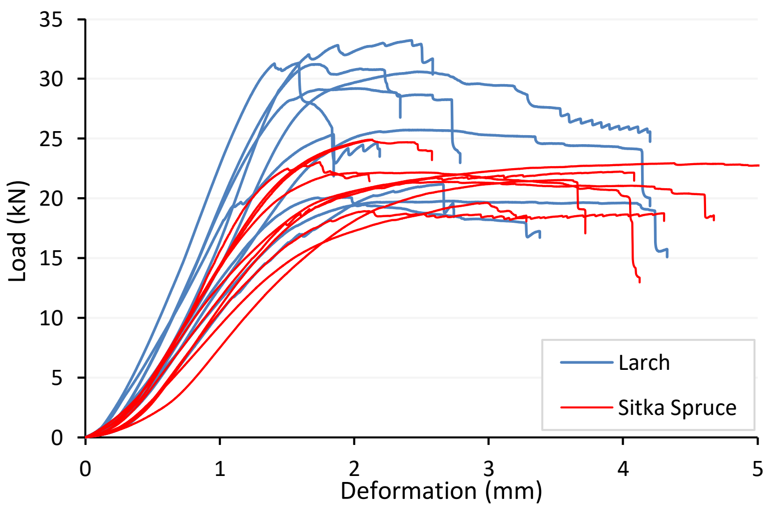

Figure 6-27: Load deformation graph for parallel to the grain embedment using a 20.50mm steel dowel.

Perpendicular to the grain samples deformed between 5-7mm prior to failure, in all instances the a form of hardening was witnessed after the yield load was reached and increased hardening was caused by densification of the timber under the dowel, see Figure 6-28.

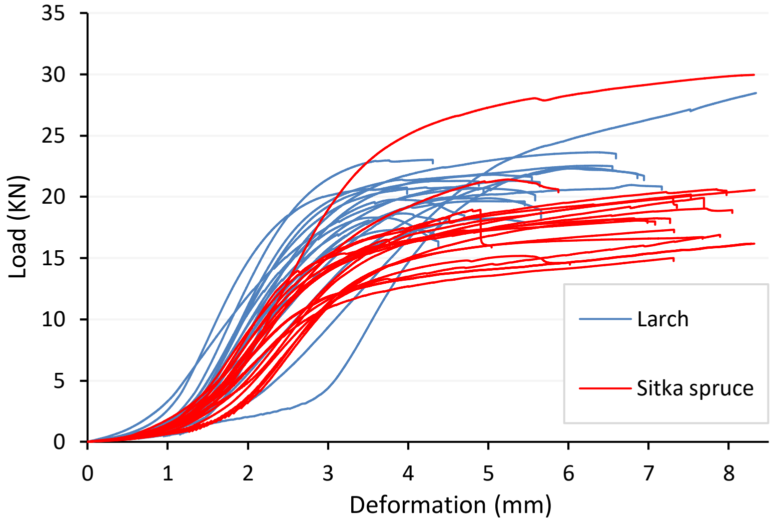

Figure 6-28: Load deformation curves for perpendicular to the grain embedment using a 20.50mm steel dowel.

6.6.5. Comments

The embedment strength of a timber member is derived in BS EN 1995-1-1 (2004) from its characteristic density which is adjusted depending on the orientation to the grain using a factored Hankinson relationship, see section 5.1.2. The density and embedment strength of each sample were plotted against each other in Figure 6-29 to see if there was a positive correlation between the two values as surmised in BS EN 1995-1-1(2004). In the parallel to grain orientation there is a positive correlation between density and embedment strength in particular with UK Larch (R^2 = 0.64). In the perpendicular to the grain direction the density of the sample does not appear to affect the embedment strength of the sample and it appears more or less constant irrespective of density.

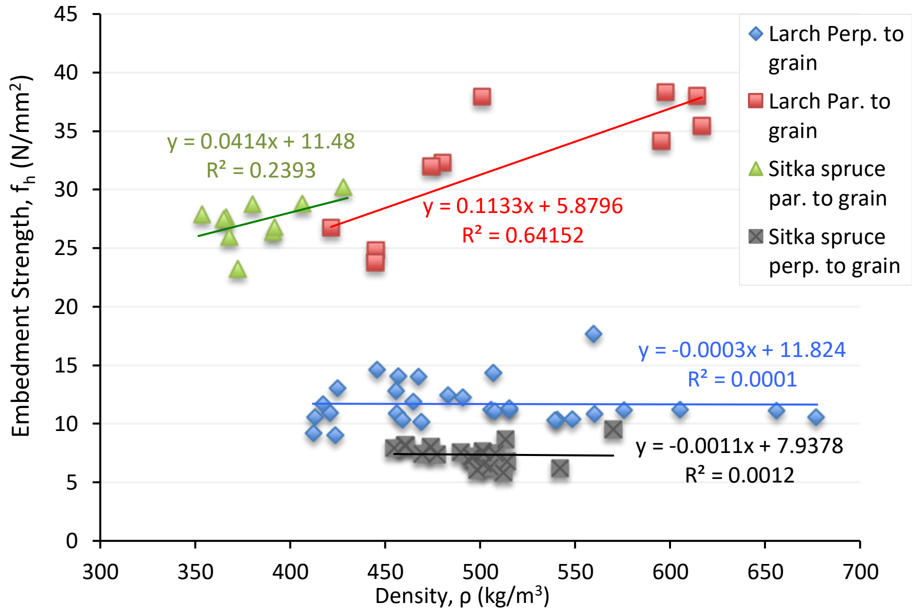

Figure 6-29: Embedment strength vs. density of all tested samples

The results for the embedment strength carried out in this thesis were then compared against the estimated embedment strength provided by the relationship to density given in BS EN 1995-1-1 (2004) for steel bolts up to 30mm (see section 5.1.2). The mean and characteristic densities (adjusted to a 12% moisture content) of both UK Sitka spruce and larch obtained were then used to calculate the predicted embedment strength and the slip modulus in accordance with BS EN 1995-1-1 (2004). These values are shown in Table 6-11.

Density, ρ	Embedment Strength, $f_{h,\propto}$ (N/mm²)			
	Larch		Sitka spruce	
	0°	90°	0°	90°
Mean, ρ_{mean}	33.73	20.01	24.85	19.46
Characteristic, ρ_k	24.39	17.35	21.99	15.86
	Slip Modulus, K_{ser} (kN/mm)			
Mean, ρ_{mean}	10.54	9.94	6.66	9.89
Characteristic, ρ_k	6.48	8.03	5.55	6.18

Table 6-11: Estimated embedment properties based on density of test samples according to BS EN 1995-1-1 (2004).

In the parallel to grain samples the characteristic values for embedment strength obtained through BS EN 1995-1-1 (2004) methods are within 10% of the value obtained through the experimental tests. In the perpendicular to grain orientation the embedment strengths are less than half the corresponding calculated values from BS EN 1995-1-1 (2004). BS EN 383 (2007) determines the maximum load as the peak load at failure or at a maximum deformation of 5mm. For the parallel to grain samples the deformation did not surpass 5mm and no hardening occurred in samples post the yield load being reached, similarly the yield values for perpendicular to grain arrangement did not obtain a value for beyond 5mm deformation. However, Zhou and Guan (2006) determined where the deformation curve follows near perfect elasto-plastic behaviour (i.e. the parallel to grain orientation) the differences in the derivation of the embedment strength will not be wildly different between the two standards but can cause wide variation in other setups and this is apparent in these tests.

Franke and Magnière, (2014b) suggest that inaccuracies exist in the current expression given in the code for determining embedment strength, particular for the perpendicular orientation (sometimes in the region of a 20% overestimation) and the results conducted in this chapter concur with these results. However, the overestimation in perpendicular to grain strength was far greater in these samples. The difference in the sample size for embedment loaded perpendicular to the grain in both the EN 383 (2007) and ASTM 5764-97a (2007) could account for a variation in the strength witnessed.

The indication is that embedment strength perpendicular to the grain is relatively constant irrespective of the density of the sample and can be attributed to the deformation directly below the dowel and the conduction of stresses to the surrounding fibres, provided sufficient edge distances are provided. Assuming the dispersion of stresses under the dowel follows a linear ratio (Schoenmakers and Svensson, 2011) the difference in sample sizes used in the test procedures creates a discrepancy in the dispersion of stresses under the sample at higher loads. The BS EN 383 (2007) will allow for a full effective length of resistance whilst the effective resistive length will be limited in the ASTM5764-97a (2007) to the sample width. At lower levels of load the deformation characteristics will be dependent on local deformation around the fastener and will not vary considerably between test arrangements. In this regard the embedment strength of the timber will be proportional to the square root of the ratio between the loaded area and the effective supporting area (Schoenmakers, Jorissen & Leijten, 2011). Reynolds, Chang and Harris (2013), simplified the stress function for Norway spruce loaded parallel to grain by assuming a 20% reduction in the stress field (in the direction of load) a distance of 7 times the diameter when loaded parallel to the grain (in this instance 143.5mm) and two times the diameter perpendicular to grain (41mm). However, dispersion of stresses in the perpendicular direction from a circular fastener have been confirmed optically and is generally agreed at a ratio of between 1:1.25 and 1:1.5 from the edge of the applied load (Schoenmakers and Svensson, 2011; Leijten and Köhler, 2004; Leijten, Köhler, & Jorissen, 2004). The point of load spread will be dependent on the friction between the materials i.e. the extent of the sliding friction zone, the stiffness of the dowel and the stiffness of the foundation material, see Figure 6-30.

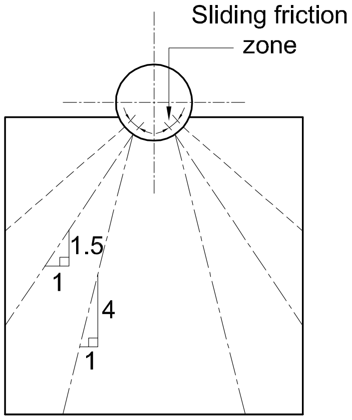

Figure 6-30: Potential embedment stress dispersal.

When the angle of load spread is shallow a large amount of timber is activated in resisting the load. When there is a high friction coefficient between the materials and the dowel embeds into the timber sample the gradient of load spread will be greater and the point of load spread will be brought closer to the centre of the sample. The material underneath the dowel that is resisting the load will be effectively pinned and restrained. In these instances, the overall width of the sample will be relevant. Load transmitted from the dowel through the samples will be spread to the unrestrained outer edges of the sample and will not allow chord or cable effects to be activated fully in the loaded area.

6.7. Combined all-timber embedment tests

When considering an all timber connection the bearing strength of a timber arrangement is affected by deformation occurring in the dowel as well as the base material. Efforts have been previously made by Schmidt and Daniels (1999), using semi-empirical observations to determine the embedment strength of a White oak peg based on its specific gravity at 12% moisture content, but no complete collection of the embedment strength of dowels has been published due to the difficultly in determining the embedment strength of a timber dowel. Church and Tew (1997) developed a method for determining the embedment strength (and stiffness) of both the timber and the dowel based upon the standardised half hole method for derivation of the embedment strength presented in ASTM 5764-97a (2007). The method depicted in Figure 6-31 overcomes the difficulty of determining the combined resistance by placing a second piece of timber of equal embedment strength above the dowel to limit localised crushing that would occur if the fastener were to be loaded directly by a steel plate and mimics the actual connection arrangement.

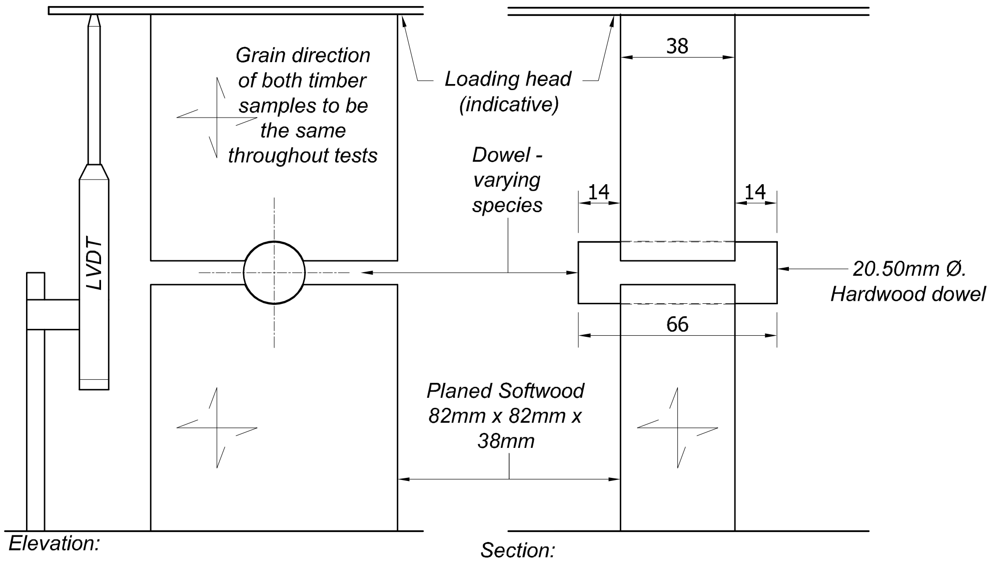

Figure 6-31: Combined embedment test setup (adapted from Church and Tew, 1997).

6.7.1. Test procedure

Four different species of 20.75mm diameter hardwood dowels set tangential to the grain were tested against two different species of softwood timber set in either parallel and perpendicular grain orientation. Five samples of each test arrangement were conducted, see Table 6-12.

Dowel Species	Lamination species			
	Larch		Sitka spruce	
	Parallel	Perpendicular	Parallel	Perpendicular
Ash	5	5	5	5
Beech	5	5	5	5
Oak	5	5	5	5
Sycamore	5	5	5	5

Table 6-12: Number and arrangement of combined embedment tests.

To match the previous embedment tests conducted in section 6.6.3 each softwood timber piece measured 82mm x 120mm x 32mm thick. The test pieces were formed in the same manner as the single embedment tests that were conducted previously in section 6.6. The top and bottom piece were created from the same planed and regularised piece of timber. The full sample was set in a jig and a 21mm diameter hole was drilled through the sample using a 21mm diameter spade drill bit. Each sample was then cut into two pieces

5mm from the centreline. This allowed for an 8mm gap between the pieces to be created when the whole test sample was located in the test rig, see Figure 6-32.

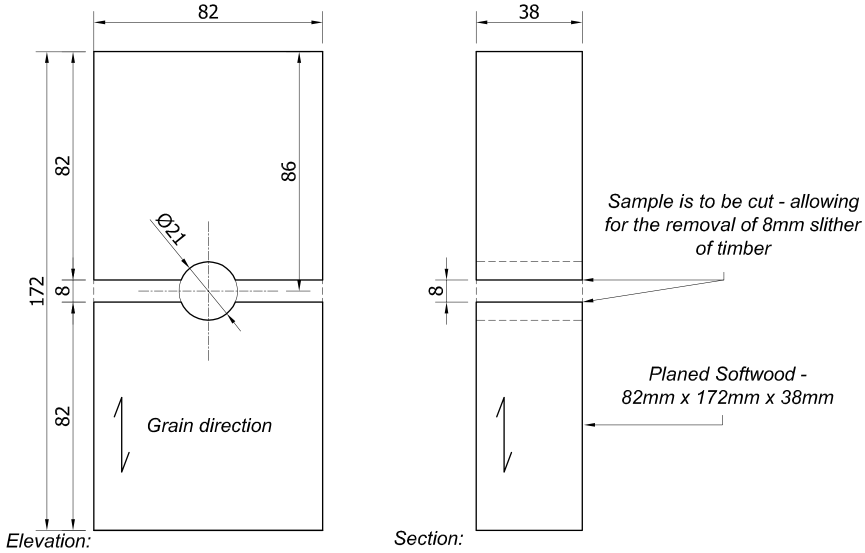

Figure 6-32: Creation of combined embedment samples.

The hardwood dowels used nominally matched the diameter of the steel dowel used for single embedment tests and were cut to 66mm long to match the length of protrusion of the steel dowel either side of the timber foundation material. Prior to testing each element of the sample was conditioned in a SC1 environment until a constant mass was reported for three separate days. Each sample was then carefully placed into the testing rig and loaded at a rate of 0.2mm/minute, until failure occurred or the maximum time allotted was reached (i.e. 420 seconds), see Figure 6-33. The deformation of the sample was measured continuously using platen displacement and the load was measured using the integrated load cell. After testing the moisture content and density of all three elements of the test were carried out in accordance with the procedure outlined in BS EN 13183-1 (2002).

Figure 6-33: Combined embedment test.

6.7.2. Results for combined embedment tests

Embedment strength was calculated from the equation given in 5.1.2 with the thickness of the based material being defined as the average width of the two timber blocks above and below the dowel. The combined stiffness of the arrangement was derived from the load displacement curve. The experimental initial stiffness modulus was defined as double the gradient according to the method previously undertaken by Church and Tew (1997). For each sample the gradient was based on a line of linear regression between load and deformation with a coefficient of determination (R^2) of above 0.99 between $0.2F_{max}$ and $0.4F_{max}$ for parallel to grain orientation and between $0.25F_{max}$ and $0.45F_{max}$ for the perpendicular orientation of the load deformation graph. These points were chosen to avoid the higher levels of initial settlement that occurred and the non-linearity that was witnessed at higher levels. Density is presented as a weighted density based upon the density of the three different timber elements and their respective volumes. The determination of characteristic values was not undertaken due to the small sample sizes and the variation witnessed in some of the sample sets. The typical load displacement curves for each test arrangement are shown in Figure 6-34 to Figure 6-41. A summary of the experimental results is shown for each species and grain orientation in Table 6-13.

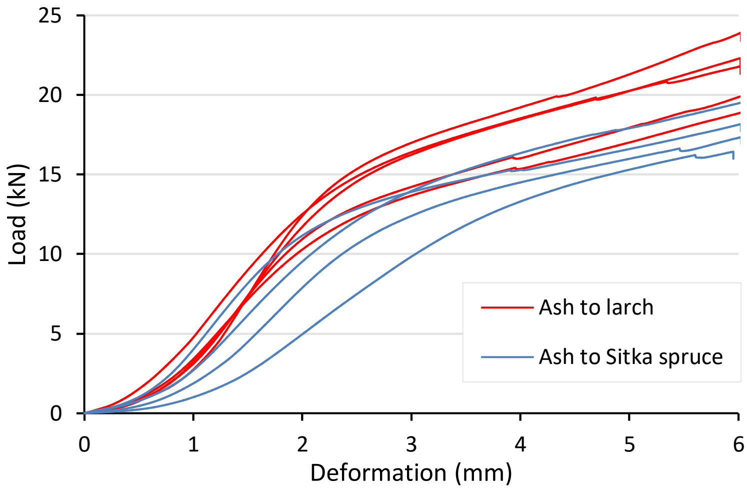

Figure 6-34: Combined parallel embedment– Ash

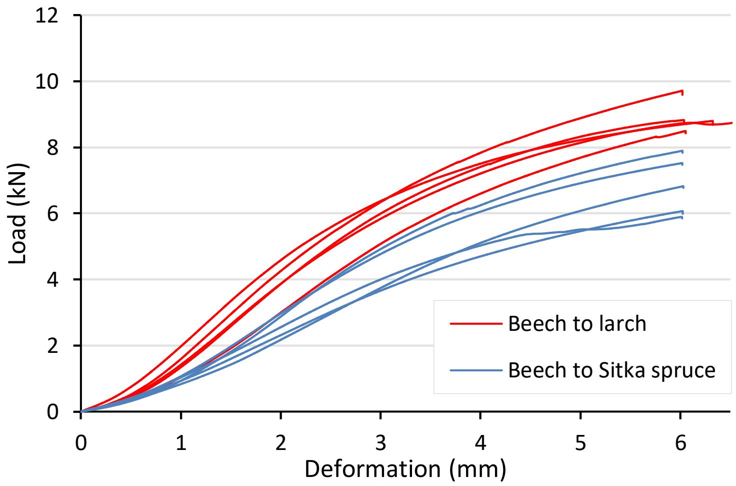

Figure 6-35: Combined parallel embedment - Beech

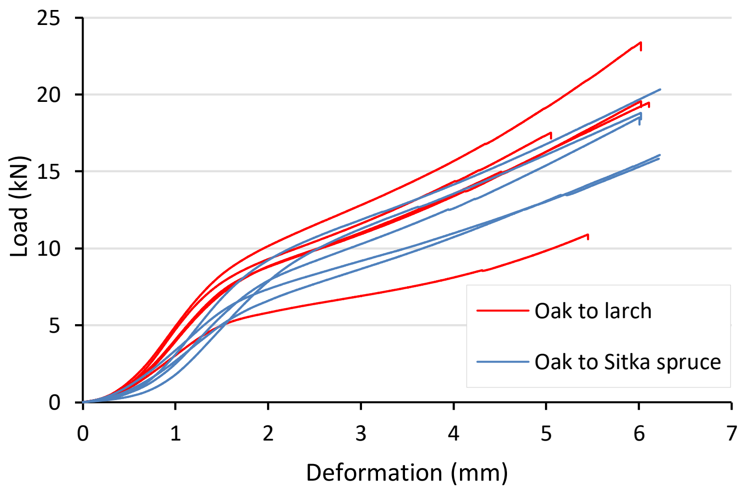

Figure 6-36: Combined parallel embedment – Oak

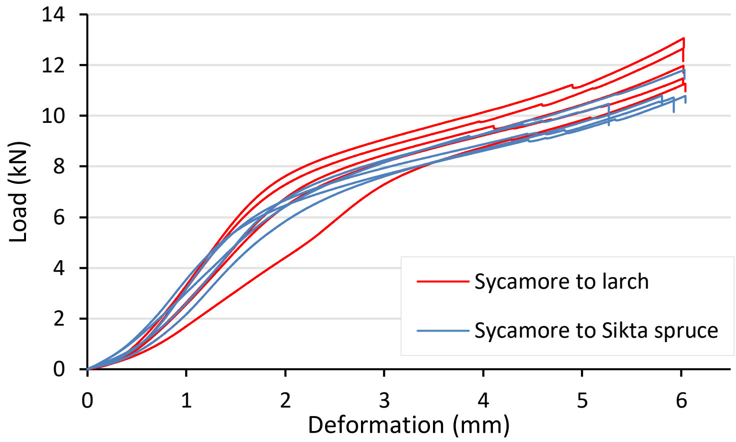

Figure 6-37: Combined parallel embedment – Sycamore

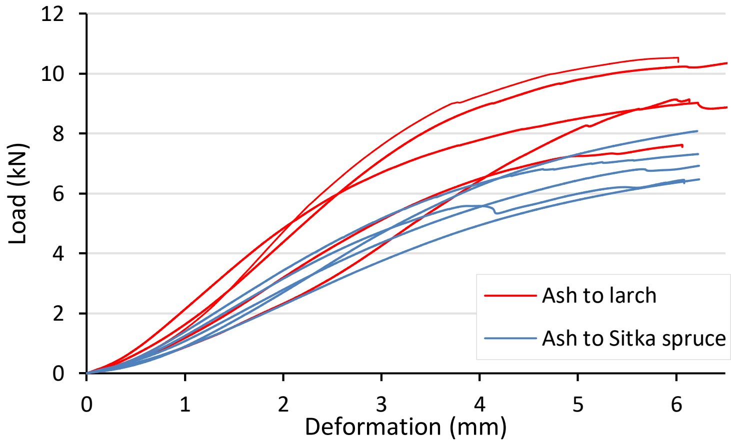

Figure 6-38 Combined perpendicular embedment – Ash

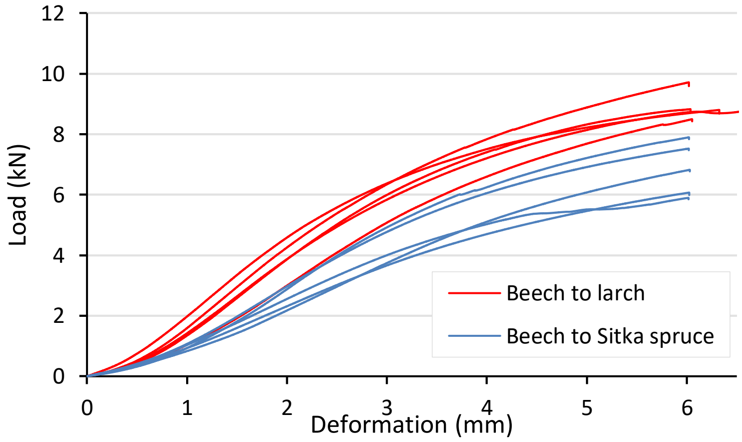

Figure 6-39: Combined perpendicular embedment – Beech

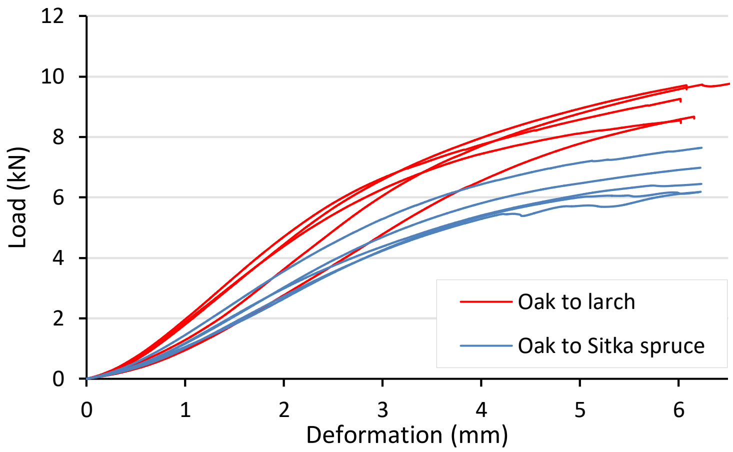

Figure 6-40: Combined perpendicular embedment – Oak

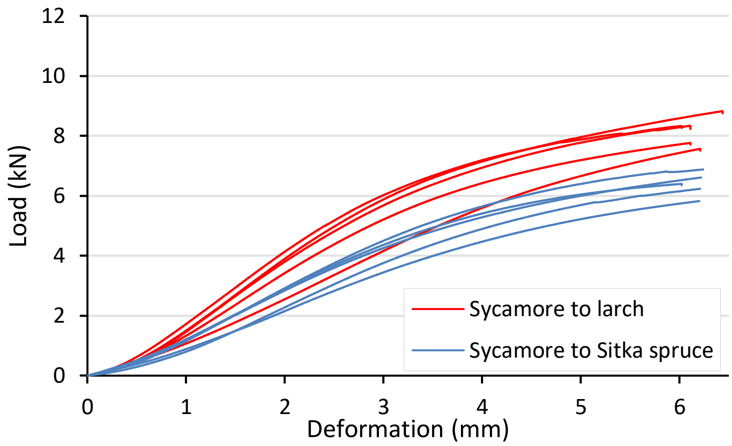

Figure 6-41: Combined perpendicular embedment – Sycamore

Property	UK larch								UK Sitka spruce							
	Ash		Beech		Oak		Sycamore		Ash		Beech		Oak		Sycamore	
	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°	Par. 0°	Perp. 90°
Number, N	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
$f_{h,mean}$ (N/mm^2)	21.23	11.20	15.72	10.26	13.78	10.54	11.72	9.58	18.99	8.39	14.91	8.41	14.75	8.05	10.84	7.74
Strength, COV (%)	8.07	11.83	6.79	4.40	20.85	6.08	5.16	3.22	5.97	12.82	6.94	11.19	14.20	6.98	6.73	4.23
Initial modulus, $k_{i,mean}$ (kN/mm)	**18.55**	**5.43**	**11.60**	**5.20**	**13.53**	**5.13**	**9.04**	**4.44**	**13.30**	**3.62**	**9.84**	**3.47**	**11.51**	**3.62**	**8.88**	**3.18**
Foundation modulus, $k_{i,mean}$ (kN/mm^2)	**23.59**	**6.90**	**14.64**	**6.68**	**17.26**	**6.51**	**11.57**	**5.71**	**17.00**	**4.66**	**12.98**	**4.54**	**14.86**	**4.74**	**11.69**	**4.13**
Stiffness, COV (%)	12.46	24.91	13.59	9.92	18.21	13.54	23.14	16.25	14.45	12.49	11.24	17.62	18.47	8.00	11.89	9.95
Yield load, $f_{y,mean}$ (kN)	**16.69**	**8.82**	**12.43**	**7.99**	**10.81**	**8.31**	**9.16**	**7.44**	**14.87**	**6.53**	**11.29**	**6.42**	**11.42**	**6.15**	**8.22**	**5.95**
Yield, COV (%)	9.79	11.69	6.19	4.14	21.78	6.17	5.06	3.24	7.51	13.32	4.10	11.86	11.60	8.16	6.88	5.05
Density, ρ_{mean} (kg/m^3)	541.3	488.7	527.0	489.7	513.0	492.6	480.0	507.4	391.8	431.0	389.8	453.1	395.3	421.9	385.4	416.9
Density, ρ_k (kg/m^3)	410.5	432.0	357.8	432.8	406.4	435.3	348.4	396.8	346.1	376.9	332.5	400.3	349.2	295.5	336.9	368.6
Moisture, ω_{mean} (%)	14.10	13.20	14.42	13.52	14.53	13.21	14.00	13.02	12.31	11.59	12.17	11.75	11.78	11.35	12.69	11.81
Moisture, COV (%)	2.19	3.03	4.80	2.83	3.80	3.41	4.93	1.99	2.96	3.57	2.56	4.17	7.30	5.18	1.73	2.38

Table 6-13: Results: Combined embedment tests using 20.50mm hardwood dowels.

Deformation of the timber is largely dependent on the stress interaction (Racher and Bocquet, 2005) at the junction between the dowel and the timber and the adherence that is created when the dowel is embedded in the timber (Quenneville and Mohammad, 2000). Research has shown when the embedment strength test included crushing of both the dowel and foundation material there was approximately a 50% reduction in the strength in the same test compared to using a steel dowel (Church and Tew. 1997). This primarily occurs because the ratio of stiffness between the fastener and foundation material is close to parity, therefore any embedment may occur predominantly in one element or in both timber elements simultaneously unlike in a steel-timber connection where deformation solely occurs in the timber foundation.

The tests conducted in this chapter provide evidence that the type of dowel species will affect the strength and stiffness of the connection. The higher the density of the hardwood dowel species the better performance was. In the best performing samples, the average strength of the combined embedment sample loaded parallel to grain saw only a 30-35% reduction from its counterpart steel test in single embedment using a steel dowel. In the perpendicular to grain combined embedment tests the average strength of the connection was at parity with the single embedment samples but the average combined initial stiffness ranged between 35-60% less than the single embedment counterpart. In the parallel to grain arrangement the reduction in stiffness was far less pronounced ranging between 8 -45%.

The combined embedment strength and stiffness follows the same pattern of results as the direct embedment tests (see section 6.6). UK larch had greater embedment resistance properties than UK Sitka spruce. The orientation of the base timber had a significant effect on the embedment properties. The perpendicular to grain arrangement had a yield strength somewhere between 75- 43% of the parallel to grain yield strength, see Figure 6-42. There was a greater variation between the species in the parallel orientation, the perpendicular to grain strength was consistent between the differing hardwood species. This indicated that the strength of the perpendicular to grain sample was more dependent on the strength of the timber foundation than on the hardwood dowel.

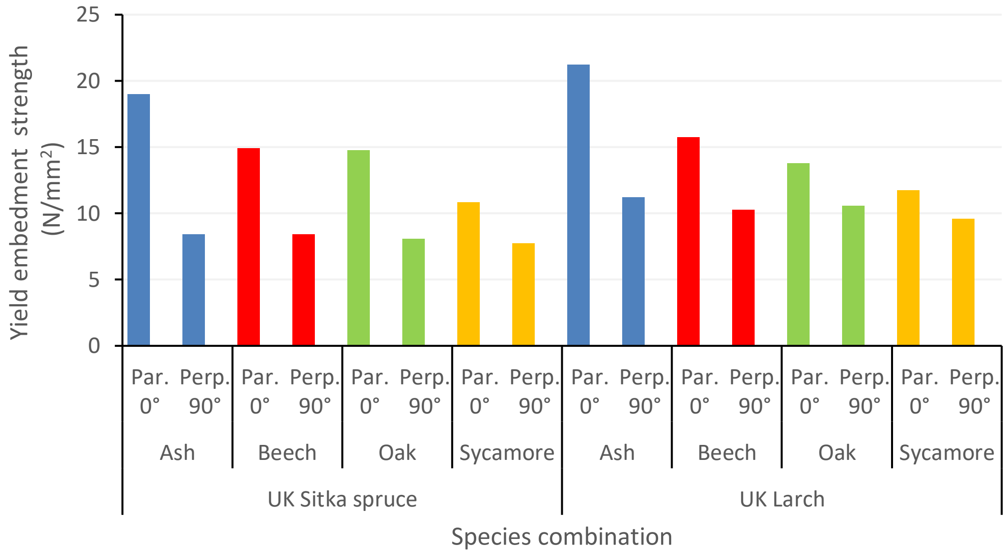

Figure 6-42: Comparison of the mean yield embedment strength of combined embedment test.

The embedment strength perpendicular to the grain was fairly consistent throughout the differing softwood samples, regardless of the dowel species used. This indicates that the response of the perpendicular to the grain arrangement is more dependent on the embedment stiffness of the timber laminations, which is shown from the tests conducted in section 6.6 and is constant irrespective of the density of the samples.

The results indicate that the perpendicular yield stiffness could be in the region of between 52-36% of the parallel to grain stiffness depending on the hardwood dowel used and softwood orientation. The hardwood dowel species used also had an effect on the embedment properties of the combined test arrangement. Ash had the greatest embedment strength and stiffness and sycamore the least. Interestingly for both oak and sycamore the difference between the embedment strength in the two different orientation was less marked than ash or beech.

Where the foundation material had a greater embedment strength and stiffness (i.e. parallel to the grain) there was a greater variance of the results between the dowel species, see Figure 6-43. Here the stiffness of the dowel became an influential factor as embedment stiffness of the timber lamination was at parity or greater than the tangential stiffness of the dowel and therefore the amount of embedment occurring in the dowel would be greater.

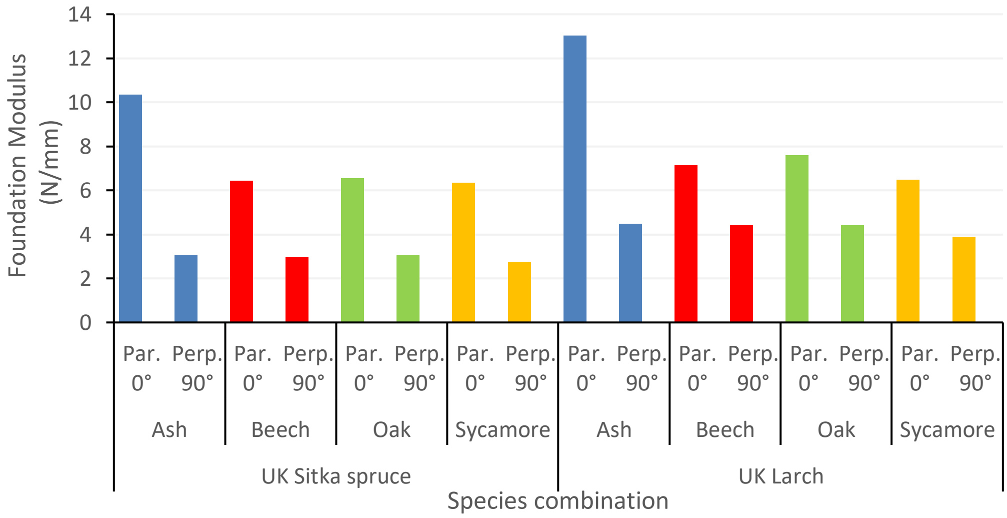

Figure 6-43: Comparison of the mean foundation moduli of combined embedment tests.

7. Experimental study of all-timber connections within a DLT panel

Experimental results obtained in Chapter 6 indicate that different timber species have notable differences in their embedment characteristics. In this chapter a series of experiments are undertaken to investigate the performance of all-timber connections using UK larch or Sitka spruce. The experiments conducted in this chapter explore the interaction that occurs in an all-timber connection formed of different hardwood dowel species and softwood laminations through a series of double shear connections tests. The test were conducted using a modified method based on the procedure outlined in BS EN 1380 (2009) and BS EN 26891 (1991). The objective of the experiments in this chapter were to identify the different load responses of connections form from different base and dowel materials, in order to inform the selection of the most appropriate materials in a DLT panel.

7.1.1. Species investigation

Previously in this thesis, UK grown Sitka spruce and larch were selected as probable lamination materials for a DLT panel. The density of the members in relation to the dowel has been shown to have a large effect on the behaviour of an all-timber connection under load (Schmidt and McKay, 1997; Schmidt and Daniels, 1999; Sandberg, Bulleit & Reid, 2000). Both species showed significant differences in their properties when they were experimentally tested in Chapter 6. The performance of both within an all-timber connection would need to be jointly analysed to understand their relative performance in a DLT panel. In an all-timber connection the high characteristic density witnessed in UK larch could cause the ratio between the density of a hardwood dowel and the connection members to be brought closer to parity causing modes of failure more commonly witnessed in traditional UK carpentry frame joints.

During Chapter 6, four hardwood dowel species were investigated in a series of properties appraisals. Whilst beech was seen as the most likely choice, a small series of comparative connection tests were conducted with ash, beech, oak and sycamore within a larch parent material to get a greater understanding of the performance of each material. Each dowel type was tested in accordance with BS EN 26891 (1991) to include two different bearing arrangements, one set with the side members' grain orientated parallel to the load direction and the other so that the side members' grain was orientated perpendicular to the load direction.

7.1.2. Member thickness

Explicitly stated in the relationships shown in the EYM is the correlation between the thickness of the side and central members and the strength and stiffness of a connection. Importance was placed on the differing thickness of side and central members and variations in the dowel diameter. It is known that the ratio between the connection members and the dowel diameter alters the shear span and creates differing locations for the formation of plastic hinges within the dowels. To further understand the behaviour of all timber connections manufactured in a DLT panel, a variety of thicknesses (20, 30, 35 and 40mm) for the lamination were manufactured to investigate and compare the strength and stiffness of the connections. The objective of this series of tests was to confirm the effect the thickness of the members had on the number, type and location of plastic hinges that form along the length of the dowel. Additionally, the thickness of the members could be used to confirm if brittle failures occurred due to a variation in thickness during the testing.

7.1.3. Dowel diameter

Beech dowels with 10.50mm and 20.50mm diameter were used to investigate the effect fastener diameter had on the performance of the connection. Tests were conducted to understand how the behaviour of the connection up to the ultimate load was affected by the changing ratio of dowel diameter to member thickness. This included establishing the mode of failure and performance up to and beyond yielding of the connection. Tests conducted by Daudeville, Davenne & Yasumura (1999) used a relatively thick dowel diameter when compared to the thickness of the laminations (i.e. $t_2/D \sim 2\text{-}4$) and this ratio was also included within the test series to ensure that the bending of the dowel was limited and different potential modes of failure were incorporated.

Two types of steel dowel with differing diameters were also included in the test. These dowels were selected for having similar proportional strength and stiffness to the dowel diameters under consideration. Through calculating the equivalent steel diameter based on the relative relationship between the printed values of the Elastic Modulus of hardwood from Lavers (1967) and steel, 8mm and 10mm diameter steel dowels were also tested alongside the hardwood dowels to verify the performance of these hardwood dowel connections. This enabled a comparison to the established methods of calculation given in BS EN 1995-1-1 (2004). The steel dowels were set in predrilled holes with the same dimensional tolerance as the beech dowels, additionally the steel dowels were prefabricated in such a manner that the length of the dowel protruding from the face of either side of the connection was the same throughout all the tests conducted.

7.1.4. Limitations and assumptions

Prior to forming the connection, a decision was made to remove the increased friction force and the increased stresses that would be induced by the insertion of the dowel in a smaller hole. The friction coefficient between the dowel and the connection members is highly variable and could create large amounts of variability in the tests beyond what is normally assumed for timber testing. Furthermore, insertion of the dowel at the correct constant force and at the correct angle could not be guaranteed using the equipment available at the time. To ensure the utmost achievable compatibility between the tests a small tolerance between the predrilled hole and the target dowel size was provided of +0.5mm. By attempting to reduce the variability between the tests the requirements for the dowel to swell during service to create a solid connection were not required. Any force induced by the insertion of the dowel into the connection could reduce the connection strength and stiffness by locally damaging the fibres in the immediate vicinity of the hole. The dowel could therefore be conditioned in an internal environment to the same moisture content (approximately 12%) of the surrounding laminations and ensure compatibility with the other elements of the connection.

Parallel spacing of the hardwood dowels was not considered in the DLT arrangement because the spacing expected between the dowels (300mm and above) are beyond the minimum spacings given in Table 8.4, BS EN 1995-1-1 (2004) based on the required spacing given in EC5 for an equivalent steel dowel. Reduced spacing of timber dowels could be allowed due to the reduced individual fastener capacity and a more compact distribution of stresses within a connection (Thomson, 2010). A study into this phenomenon was omitted as the spacings to be used in the final DLT panel construction would limit the propensity of brittle failure to occur.

Additional factors that contribute to the design of a connection such as swelling, shrinkage, combinations of fasteners, eccentricity and group effects (Racher, 1995) have not been considered within the matrix of test configurations conducted here for compatibility purposes. Expanded test regimes would be necessary to investigate these behaviours and were not seen as likely within a DLT panel and were omitted for brevity.

Perpendicular to the grain loading presents a problem for the sudden brittle failure of the connection prior to or immediately after the yield point of the connection. Often loading induces tensile forces to be induced perpendicular to the grain that promote splitting prematurely. Described in detail in section 5.1.6 the brittle splitting mode of failure is thought to be instigated when the shear or tension strength is overcome in the particular orientation to the grain that is being loaded. The LEFM model given in BS EN 1995-1-1 (2004) presents a simplified lower bound approach for a fastener located in softwood, based on uniform load across the fastener and a consistent stiffness in the timber surrounding the timber. Whilst the inadequacy of this model is more of a concern in groups or blocks of fasteners it is more relevant for a single timber dowel in a DLT panel connection because of its stockiness in relation to the members thickness and the potential for non-uniform embedment to occur across the dowel bearing.

To counter this occurrence the ratio of the three members within the connection were set at parity to induce greater embedment stresses in the central member that was orientated parallel the grain. How this design consideration affects the performance of an all-timber dowel connection with a comparatively large dowel diameter but with potentially lower stiffness performance is unknown and this was investigated using a combination of 7 different connection configurations that altered the ratio between the connection members and the dowel diameter. The aim was to investigate how the changing stiffness parameter could potentially affect the ductility of the connection overall.

7.2. Testing

7.2.1. Sample inspection:

Connection tests involving timber connections need to mitigate the risks of splitting (Murty, Smith & Asiz, 2007) and the natural of variance in timber (Franke and Quenneville 2014) and therefore EWPs such as LVL, LSL or Glulam are commonly used in connection tests. The main body of the experiments in this PhD investigate the influence of homegrown British timber on the overall characteristics of DLT panels and their effect on the individual interactions within a DLT panel and a timber material with low variance could not be included in the study.

Due to the complication of sourcing UK softwood laminations and the amount of man hours required to process the raw material to the required quality and consistency, a high level of precision was required in the manufacture of the specimens. The raw lamination material supplied was UK grown Sitka spruce provided from the same experimental set utilised in Chapter 6. The remainder of the green Sitka spruce was left to dry in neat stacks with separating spacers in an internal SC1 environment (a large warehouse facility in Portobello, Edinburgh) for approximately 12 months to allow samples to air dry to a moisture content of approximately 12%. Following the distortion study conducted in Chapter 6 the timber was allowed to reach a moisture equilibrium gradually in order to limit any distortion that could be equated to the speed of the kiln drying process. A quantity of larch obtained from the SIRT study was left in the same environment as the Sitka Spruce, albeit only for a period of 9 months.

After the seasoning process each air-dried plank was cut to the required length using a slide mitre saw and planed to the correct thickness and depth using a planer-thicknesser. This limited the amount of distortion of the sample and provided a tolerance of ± 1mm for each dimension. The thickness and the depth of the specimens was checked locally using a set of 600mm deep jaw digital callipers and a set of 300mm digital callipers to ensure that the tolerance of ±5% to the target size was achieved for each dimension of the sample. The dowel holes in each lamination were predrilled using a vertical pillar drill. A pre-built jig and a 21mm diameter spade drill was used to correctly align and drill the specimens for each sample arrangement. Each regularised lamination was then inspected for knots and assigned a number.

Figure 7-1: Inspection, marking and arrangement of samples.

One of the main issues with UK timber that was highlighted in the desk study and the experimental study conducted in section 6.1 was the high propensity for knots. The material provided for this study was from normal supply chains and no effort was therefore made to limit the knottiness of the material prior to delivery. Whilst it is common practice to test clear wood samples to determine the material properties of the timber, arguments can be made that it is not a true representation of the structural products that would be created from a population of timber because of the probability of defects to occur in these larger volumes of timber. Allowances had to be made for the inclusion of knots in the test regime, otherwise the number of tests conducted would have been minimal as the amount of pure clear wood specimens that would have been achievable from the material provided would have been negligible. Findings from Thomson *et al.* (2010) stated that no significant difference in strength was found using connections with knots, but an increase of stiffness was witnessed in areas where high density of annual growth rings was apparent. Therefore, within the realms of pragmatism and efficacy some limited allowance of knots was included provided that they were within reasonable boundaries, tolerance and would be considered to not detrimentally influence variability between the tests, as described next.

During each stage of the processing of the material any areas with high concentrations of knots were removed. Any remaining samples with knots were judiciously cut so that knots were located in the samples in areas deemed not to affect the overall behaviour. The areas which knots were allowed were set by geometric boundaries based on the direction of load, the expected loaded area and design rules for connections given in BS EN 1995-1-1 (2004), see Figure 7-2 and Figure 7-3. The rules for unloaded edge distances for bolts given in Table 8.4 BS EN 1995-1-1 (2004) and illustrated in Figure 5-7 were adapted based on an equivalent steel dowel stiffness of a hardwood dowel. The equivalent steel diameter was calculated by transforming the geometric dimensions of the hardwood dowel based on their comparative elastic moduli obtained from literature, in this instance Lavers (1967) for the hardwood dowels. The edge and end distances were then calculated based on the dowel diameter. Any such timber sample that had a knot within the demarcated zones was removed prior to testing. Any timber sections with resin pockets or other kinds of defects were also excluded from the specimen group to reduce the amount of scatter in the results and maintain a clear ideal for model comparison.

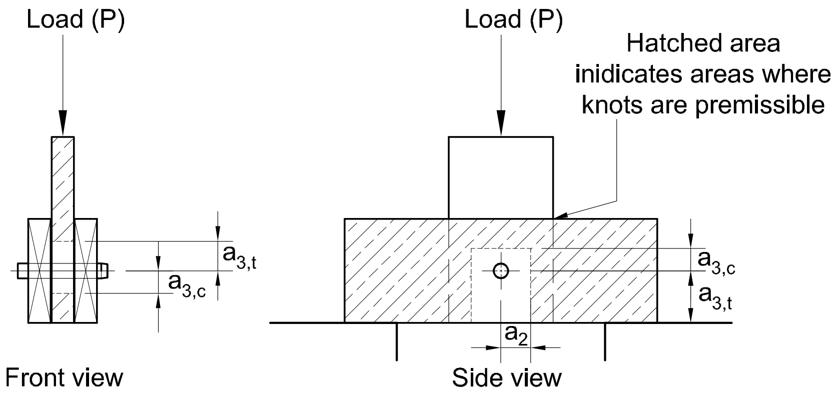

Figure 7-2: Allowance of knots in perpendicular to grain tests.

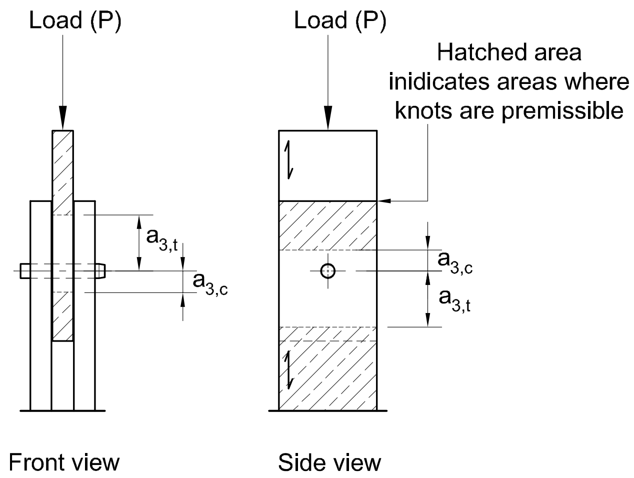

Figure 7-3: Allowance of knots in the parallel to grain tests.

Each hardwood dowel was inspected to ensure that no defects such as knots or grain deviation were present in the sample, any such samples were removed from the study. Each hardwood dowel was then trimmed using a band saw and a guide to ensure that the amount of the dowel that protruded (14mm) from either side face of the connection was consistent throughout all test configurations.

7.2.2. Factored edge and end distances

Preliminary tests conducted for this study on connections formed using European beech dowels set in Sitka spruce connection members, highlighted the propensity for the timber to fail by splitting along the grain, see Figure 7-4. This brittle failure was caused by tensile stresses perpendicular to the grain when a loaded edge distance of 2.5 times the dowel diameter perpendicular to the grain was used. This indicated that the lower range specified by Burnett *et al.* (2003), referenced in section 5.2.2.2 would not be applicable to home grown Sitka spruce.

Figure 7-4: Examples of brittle failure occurring in preliminary testing.

Following the recommendations set out in Burnett *et al.* (2003) and verifying the connection in accordance with the LEFM procedure given in BS EN 1995-1-1 (2004) the connection arrangement was enlarged to avoid brittle failure particularly with regard to the anisotropy witnessed in UK grown timber and increased susceptibility to that manner of failure (Santos *et al.,* 2013). It was therefore decided to increase the depth of the material to a larger target size to ensure compatibility throughout. Based on the expected final dried sizes obtained from the existing supply as discussed in section 6.2.2, an increased section depth of 140mm was utilised (i.e. a stock size of 147mm planed to the required tolerances). This gave an edge distance of approximately 3.5 times the dowel diameter to limit the occurrence of brittle failure in the sample prior to dowel yielding or embedment occurring.

7.2.3. Test Setup – Parallel to the grain

The test arrangements for the parallel to the grain tests are shown in Figure 7-5, the arrangement of the sample is based on the requirements stipulated in BS EN 1380 (2009). To minimise local indentation occurring, steel plates measuring

100 x 300 x 12mm were inserted between the members of the connection and the loading head and supports.

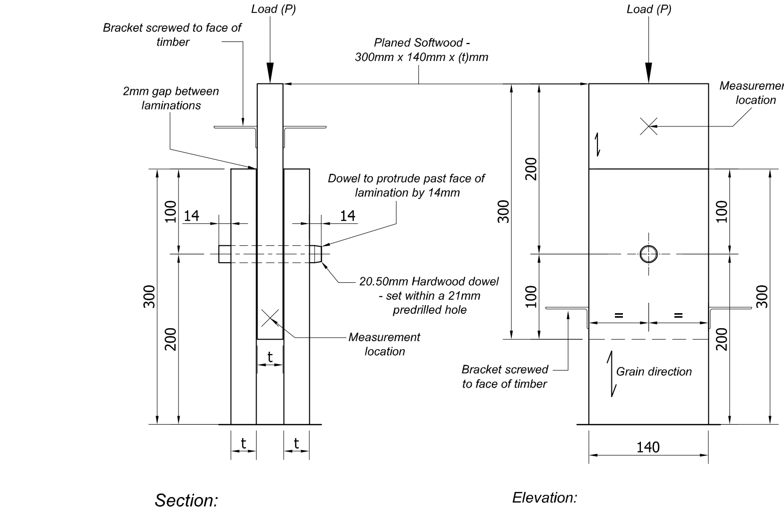

Figure 7-5: Parallel to grain double shear plane tests.

7.2.4. Test Setup – Perpendicular to the grain

The test arrangements for the perpendicular to the grain tests are shown in Figure 7-6, the arrangement of the sample is based on the requirements stipulated in BS EN 1380 (2009) and the edge and end distances discussed in section 7.2.2. Load was applied in compression at a variable load rate through the top of the central member with a steel bearing plate to remove the potential for localised crushing at the head of the connection. The side members were supported on 50mm wide steel plates set on rollers at either end.

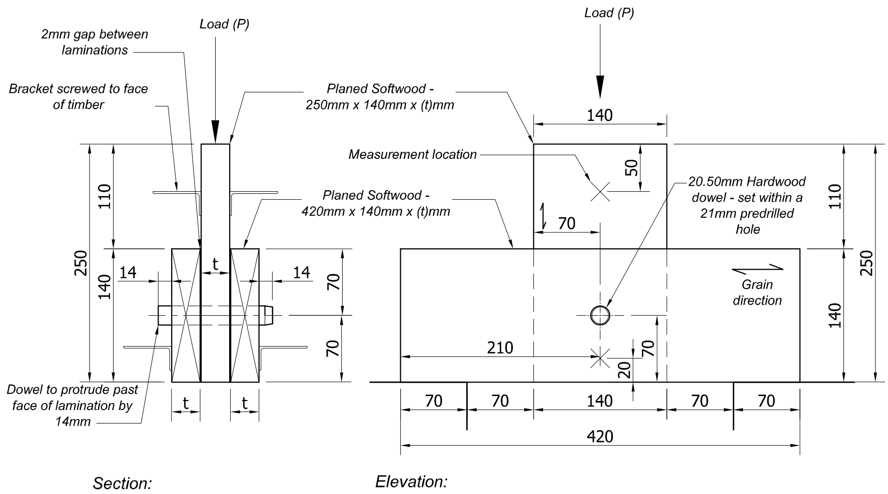

Figure 7-6: Perpendicular to the grain double shear plane testing.

7.2.5. Test samples

Before testing was undertaken all the dried softwood members and hardwood dowels were stored in a climatically controlled store at 20±3 °C and 65±2% relative humidity for two weeks prior to the start of testing. The width and depth of each lamination was measured at 3 positions along its length using digital callipers, once at each end and once at the centre, and the mean value of dimensions determined. The length of each timber specimen was measured once using a measuring tape immediately prior to assembly. Hardwood dowels (previously cut to length and seasoned) were left in an internal environment until a constant weight was achieved over three consecutive days, they were then measured radially, tangentially across the grain and longitudinally along the grain using digital callipers. All samples (both softwood and hardwood) were weighed immediately prior to sample formation and testing to an accuracy of 0.01g.

Each series was marked in the same manner, with the type of dowel, its diameter, shear plane orientation and the type and the width of the lamination, i.e. $S_L08_{par}SS20$ is an 8mm steel dowel, orientated parallel to the grain with 20mm thick Sitka spruce laminations. A key to the terms used in sample identification are shown in Table 7-1. The samples were marked numerically within each series designation. Each lamination within the specimen was additionally labelled A, B and C to signify its location in the sample. B being the central sample and A and C being the left-hand side and right-hand side of the sample respectively.

Test signifier	Species
A	Ash
B	Beech
O	Oak
S	Sycamore
S_L	Steel
L	Larch
SS	Sitka spruce

Table 7-1: Key for test series

7.2.5.1. Parallel to grain testing

A total of 40 tests were conducted on parallel to the grain double shear plane arrangements in larch. The test series conducted for double shear plane tests parallel are shown in Table 7-2.

Test series	Dowel type	Dowel diameter (mm)	t/D ratio	No. of samples
$A20_{par}L30$	Ash	20	1.5	4
$B20_{par}L30$	Beech	20	1.5	4
$B10_{par}L30$		10	3	4
$B20_{par}L30$		20	1.5	4
$B10_{par}L40$		10	4	4
$B20_{par}L40$		20	2	4
$O20_{par}L30$	Oak	20	1.5	4
$S20_{par}L30$	Sycamore	20	1.5	4
$S_L8_{par}L30$	Steel	8	3.75	4
$S_L10_{par}L30$		10	3	4

Table 7-2: Parallel to grain double shear plane test arrangements using larch and differing dowel types.

A total of 68 tests were conducted on parallel to the grain double shear plane arrangements in Sitka spruce. The test series conducted for double shear plane tests parallel are shown in Table 7-3. A minimum of 5 tests were conducted for each test arrangement that utilised a timber dowels and a minimum of 3 tests were conducted for a connection using a steel dowel.

Test Series	Dowel type/species	Dowel Diameter (mm)	t/D ratio	No. of samples
$S_L08_{par}SS20$	Steel	8	2.5	3
$S_L08_{par}SS30$		8	3.75	4
$S_L10_{par}SS20$		10	2	3
$S_L10_{par}SS30$		10	3	4
$B10_{par}SS20$	Beech	10	2	7
$B10_{par}SS30$		10	3	6
$B10_{par}SS35$		10	3.5	5
$B20_{par}SS20$		20	1	10
$B20_{par}SS30$		20	1.5	10
$B20_{par}SS35$		20	2	10
$B20_{par}SS40$		20	1.75	6

Table 7-3: Parallel to grain double shear plane test arrangements using Sitka spruce.

7.2.5.2. Perpendicular to grain testing

An exact replica of the test series defined in the parallel to grain tests were conducted for perpendicular to grain tests in accordance with the test arrangements depicted in BS EN 1380 (2009). A total of 40 tests were conducted on perpendicular to the grain double shear plane arrangements in larch. The test series conducted for double shear plane tests perpendicular are shown in Table 7-4.

Test series	Dowel species	Dowel diameter (mm)	t/D ratio	No. of samples
$A20_{perp}L30$	Ash	20	1.5	4
$B20_{perp}L30$	Beech	20	1.5	4
$B10_{perp}L30$		10	3	4
$B20_{perp}L30$		20	1.5	4
$B10_{perp}L40$		10	4	4
$B20_{perp}L40$		20	2	4
$O20_{perp}L30$	Oak	20	1.5	4
$S20_{perp}L30$	Sycamore	20	1.5	4
$S_L8_{perp}L30$	Steel	8	3.75	3
$S_L10_{perp}L30$		10	3	3

Table 7-4: Test arrangements for double shear plane connection tests using larch and differing dowel types.

In total, a further 68 tests were conducted on perpendicular to the grain double shear plane arrangements in Sitka spruce and are summarised in Table 7-5.

Test Series	Dowel type/species	Dowel diameter (mm)	t/D ratio	No. of samples
$S08_{perp}SS20$	Steel	8	2.5	3
$S08_{perp}SS30$		8	3.75	4
$S10_{perp}SS20$		10	2	3
$S10_{perp}SS30$		10	3	4
$B10_{perp}SS20$	Beech	10	2	7
$B10_{perp}SS30$		10	3	7
$B10_{perp}SS35$		10	3.5	5
$B20_{perp}SS20$		20	1	10
$B20_{perp}SS30$		20	1.5	10
$B20_{perp}SS35$		20	2	10
$B20_{perp}SS40$		20	1.75	5

Table 7-5: Test arrangements for double shear plane connection tests using Sitka spruce and differing dowel types.

7.2.6. Test procedure

After the seasoning and conditioning process described in section 7.2.1, the members of the connections were sorted and formed immediately prior to the test commencing. The timber dowels orientated in such a manner that they were loaded tangentially to the grain to ensure the worst performance from them. Mack (1966) stated that when there was a gap of 0.71mm between the members there was a 35% strength increase arising from friction but no friction contact occurred when the gap between the members was greater than 1.42mm. In a similar manner to the tests undertaken by Shanks *et al.* (2008) and Smith *et al.* (2005), a small series of steel shims were used to separate at the top and bottom of the sample to ensure a gap of 2mm between the members was provided to limit friction. The connection pieces were clamped together and the dowels were inserted through the predrilled holes in the samples, until they protruded 14mm either side of the sample the steel shims were then removed prior to loading leaving the connected sample.

Shanks and Walker (2005) reported that between 30 seconds and 30 minutes, the effects of load rate were negligible on the strength of an all-timber connection. The loading rate was therefore selected to ensure failure occurred between 180-720 seconds in accordance with BS EN 26891 (1991). Testing was conducted on a Schenck-Treblek RM testing machine with a 50kN load cell. Load was applied to the sample at a variable rate proportional to its estimated maximum load (F_{est}). The value of F_{est} was derived from inspection of the load slip curves given in literature for hardwoods and previous experience of hardwood dowel connection testing. Load was placed on the head of the sample until 40% F_{est} was achieved after which point the load on the test piece was reduced to 10% F_{est}. The load was then applied at a constant rate of 2.5kN/min until 70% F_{est} when the loading rate dropped 1.25kN/min and continued at this rate until failure occurred. Reducing the load and then reloading the sample was conducted because Thomson (2010) stated that better correlation occurred between the beam on foundation model and actual results when unloading and then reloading of the sample was undertaken.

For each specimen, continuous deformation readings were taken at an accuracy of 1% using LVDT's bearing on small steel brackets fixed to either side of the central member at an unstressed location. The average of the two values determined the relative slip between the members. Additionally, to take into account any deflection that would occur due to vertical load being placed centrally in the span of the side members in the perpendicular to grain test samples, two further LVDT's were used to record the deflection at the bottom outer edge of the side members. The relative slip between the central and side members was then calculated from the average deflection of each member combination subtracted from one another. The test was conducted using live deflection readings and was terminated when the relative value of slip of the central member reached approximately 15mm or a significant failure occurred compromising the integrity of the connection. This was stated as the F_{max} in accordance with the guidelines set forth in BS EN 26891 (1991).

After testing, a full cross section of the softwood test specimens was cut as close as possible to the point of failure and the moisture content and density was determined in accordance with EN 13183-1 (2002). This included all three pieces that formed the connection. The moisture content and density of each dowel was determined using the process described in 6.5.1. Furthermore, the measured values of modulus of elasticity and density were corrected to a 12% moisture content as stipulated in EN 384 (2010). The density of the connection reported is based on the expression given in BS EN 1995-1-1, equation 7.1, (see section 5.3) to determine the overall density of a connection based on different densities.

7.3. Results

The ultimate load (F_{max}) was defined as the point of maximum load prior to failure or at a 15mm connection slip, unless any decrease in the load capacity was not immediately followed by a rapid increase in load carrying capacity equal to or greater than the slope of the initial stiffness modulus (Burnett *et al.,* 2003). Using these principles for obtaining the ultimate load was seen as pragmatic as often during testing an initial crack was heard and some relaxation of the joint occurred that was followed immediately by a rebound of strength which did not compromise the overall integrity of the connection in a similar manner to testing undertaken by Burnett *et al.* (2003) and Eckelman and Haviarova (2007).

The initial connection stiffness was calculated from the gradient of the line that corresponds with the initial points of 20% F_{max} and 40% F_{max} on the load deformation curve as shown in Figure 7-7. The initial stiffness is reported at a 12% moisture content, achieved by using a weighted moisture content from the reported moisture of the constitute elements using the oven dry method stipulated in BS EN 13183-1 (2002).

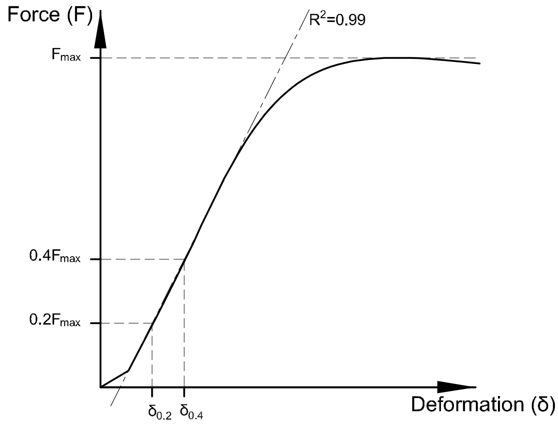

Figure 7-7: Derivation of initial stiffness

For compatibility purposes the yield load of the connections was derived in the same manner as the other derivations of material parameters conducted in this thesis (Wilkinson, 1993). Determination of the yield load is based on a 5% offset method described from ASTM 5764-97a (2007) previously illustrated in Figure 6-23. Following a discussion instigated by Branco, Cruz and Piazza (2009) into the viability of BS EN 26891-1 (1991) referring to the actual behaviour of composite joints with pronounced non-linear behaviour in the initial phase, the yield stiffness was also obtained through testing. The yield stiffness was taken as the secant modulus to the intercept of the 5% offset line and the load deformation curve from the test.

7.3.1. Parallel to the grain - Larch connections

The larch connections tests were conducted in May 2015, at Edinburgh Napier University. The results for the ten different connection configurations are shown in Table 7-6, including ultimate load (F_{max}) and initial stiffness (k_i) using the method described in section 7.3.

Test series	Dowel species	Ultimate load (kN)		Yield Load (kN)		Initial stiffness (kN/mm)		Yield Stiffness (kN/mm)	
		Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
$A20_{par}L30$	Ash	12.67	14.96	9.00	38.88	5.62	23.64	2.70	19.70
$B10_{par}L30$	Beech	4.61	18.79	2.64	13.87	1.96	9.80	1.40	10.73
$B20_{par}L30$		14.84	5.43	11.93	3.63	5.31	9.44	2.15	8.85
$B10_{par}L40$		4.88	14.19	2.76	9.13	1.92	7.51	1.50	22.37
$B20_{par}L40$		14.86	10.14	10.78	10.48	6.18	6.65	2.58	9.42
$O20_{par}L30$	Oak	10.96	24.27	7.74	1.29	4.40	46.88	2.49	10.65
$S20_{par}L30$	Sycamore	11.12	16.25	8.63	10.34	4.89	34.55	2.05	34.51
$S_L8_{par}L30$	Steel	10.32	18.48	6.95	9.24	4.65	31.33	3.68	16.54
$S_L10_{par}L30$	Steel	11.55	18.57	9.33	16.08	4.07	13.34	3.31	11.77

Table 7-6: Summary of double shear plane connection test using UK larch.

The typical load slip response of each connection type was tested in the parallel to the grain orientation using 30mm laminations and 20mm dowels and this is shown in Figure 7-8, shown overleaf. When loaded parallel to the grain, all of the connections exhibited linear behaviour up until the yield point, after which ash, oak and sycamore experienced an abrupt drop in strength. Although these species all regained their strength up to a level close to or beyond their initial yield point, their stiffness was severely reduced. The beech and the steel dowels maintained the strength and performed plastically after the yield load was reached. All dowels witnessed hardening after yielding of the dowel was reached, however only the beech dowel did not show an immediate reduction in strength following yielding of the connection.

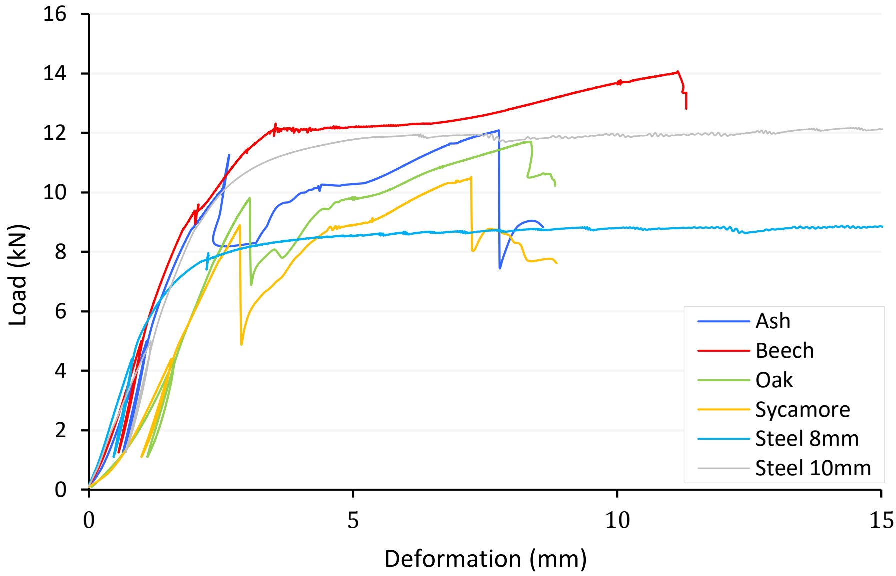

Figure 7-8: Typical load slip response of selected 20.50mm diameter hardwood dowel and mild steel dowel fasteners in parallel to the grain double shear connection tests using UK larch.

The results showed that 20.50mm Beech dowels had the greatest ultimate and yield load capacity of all the tested dowel types. When used in 30mm laminations, the connection had a mean ultimate load capacity of 14.84kN and yield load of 11.93kN; a comparison of the mean ultimate and yield loads for all tested species is shown in Figure 7-9.

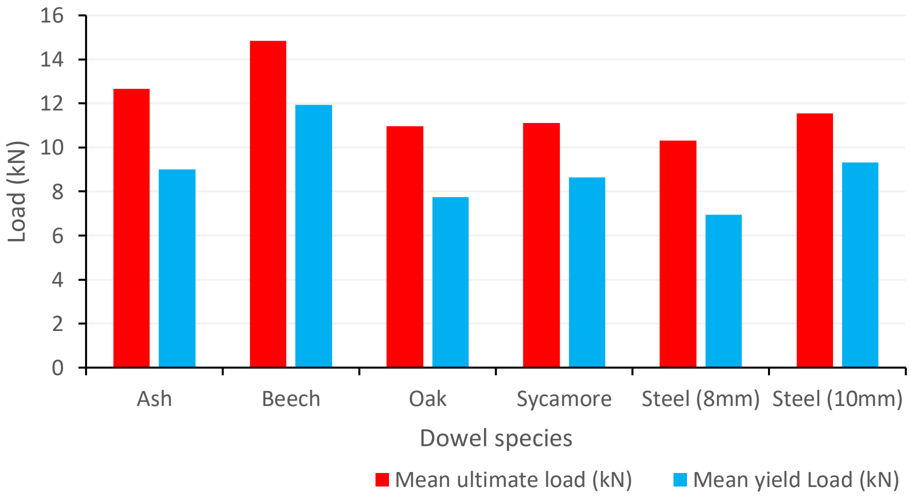

Figure 7-9: Comparison of the mean yield and ultimate loads of 20.50mm diameter hardwood dowels and 8-10mm mild steel dowels in 30mm diameter larch laminations

Due to the fluctuation in the modulus of elasticity of the various hardwood species, two differing steel diameter dowels (8mm and 10mm) were included in the test series to best encompass the potential variation between the dowels. The results indicated that the maximum load capacity of the 8mm steel in 30mm laminations is a close approximation to the performance of both a 20.50mm oak and sycamore dowel. Whilst the 10mm steel dowel could be used as a strength estimation of an ash dowel, it would be slightly conservative for a beech dowel in larch. Care should be taken in directly comparing the yield loads of the timber dowel connections and steel dowels. A plastic response from the dowels was not guaranteed as an immediate drop in strength and stiffness at the yield point indicates that yielding of the dowel did not occur. Any further increase in resistance was a product of the confinement of the dowel and other effects surrounding its confinement.

The yield stiffness of the timber dowels was far lower than the steel dowels, which is expected when using the 5% diameter offset and the secant modulus to determine stiffness. The offset of the proportional gradient line is smaller when using smaller diameter dowels, meaning the position of the yield load will occur at lower levels of deformation when using steel dowel. A direct comparison of the yield stiffness between timber and steel dowels of differing diameter was unachievable using different dowel diameters.

The hypothesis that the greater the width of the laminations the greater the ultimate load of a connection of same diameter was confirmed on this small series of tests conducted using larch laminations and beech dowel. There is evidence that when the ratio between the lamination thickness, t and the dowel diameter, d is larger there is a more noticeable increase in maximum load capacity between connections formed of the same diameter dowel, i.e. t/D = 3 (4.61kN) to t/D =4 (4.88kN) than t/D =1.5 (14.84kN) to t/D = 2 (14.86kN), see Figure 7-10.

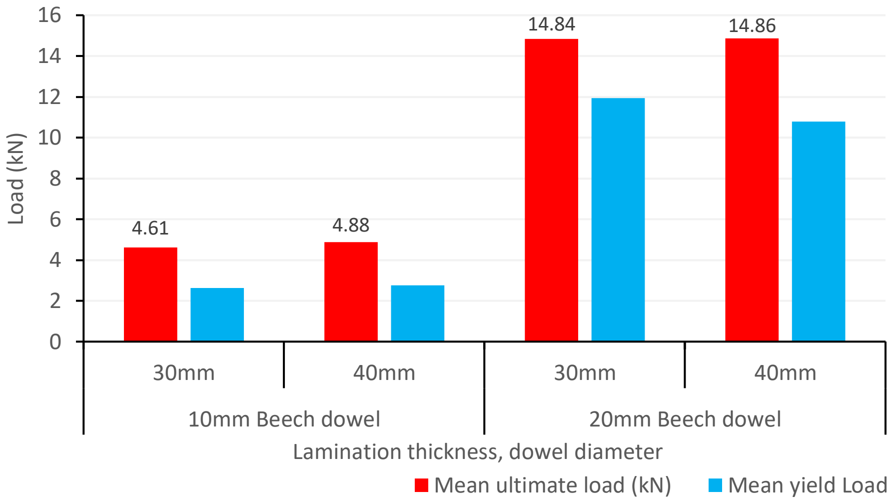

Figure 7-10: Comparison of the mean yield and ultimate load of beech dowels in UK larch with varying diameter and thickness.

Further experimental investigations into the sensitivity of larch with differing thicknesses and beech dowels of two differing dimensions illustrate that plastic yielding did occur at t/D ratios of at least 2. Above this ratio plastic yielding was consistent for beech dowels, see Figure 7-11.

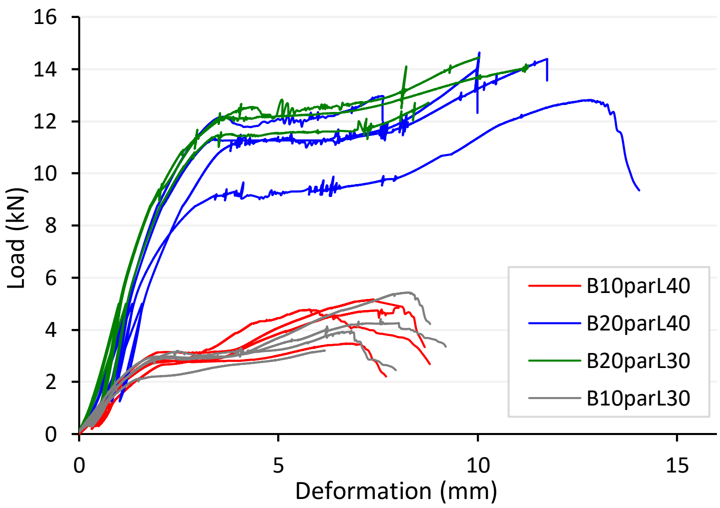

Figure 7-11: Load deformation curves for parallel to grain double shear plane connection tests using beech dowels and UK larch.

7.3.2. Parallel to grain – Sitka Spruce Connections

The Sitka spruce connections tests were conducted in June 2015, at Edinburgh Napier University. Table 7-7 shows a summary of the mean ultimate load, the mean yield load, the mean initial stiffness and the yield stiffness. The mean values recorded from all the test groups showed a definitive increase in strength and stiffness when the central member thickness was increased. The increase in ultimate load witnessed between dowels of the same diameter over differing laminations thickness was proportional with both dowel diameters, see Figure 7-12.

Test Series	Ultimate Load		Yield Load		Initial Stiffness		Yield Stiffness	
	Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
$S08_{par}SS20$	5.37	1.44	3.70	10.23	2.89	37.91	2.15	29.31
$S08_{par}SS30$	8.90	6.95	6.12	4.56	6.12	14.64	4.15	15.54
$S10_{par}SS20$	6.32	11.22	5.12	8.49	4.38	33.24	2.64	25.38
$S10_{par}SS30$	10.01	17.65	7.91	18.33	5.00	34.28	3.59	18.33
$B10_{par}SS20$	4.03	16.59	3.26	15.45	1.43	37.95	1.43	37.95
$B10_{par}SS30$	4.47	22.14	3.43	14.04	2.42	29.31	1.52	18.57
$B10_{par}SS35$	4.68	23.12	3.19	7.45	2.26	36.00	1.52	21.85
$B20_{par}SS20$	10.35	12.54	9.40	10.32	4.00	20.56	2.50	10.32
$B20_{par}SS30$	13.16	15.56	11.57	12.62	5.51	21.59	3.32	15.07
$B20_{par}SS35$	14.98	18.79	11.73	11.97	5.54	17.54	3.45	14.67
$B20_{par}SS40$	16.77	19.28	12.65	12.95	5.94	12.06	3.87	7.93

Table 7-7: Test results from parallel to grain connection tests.

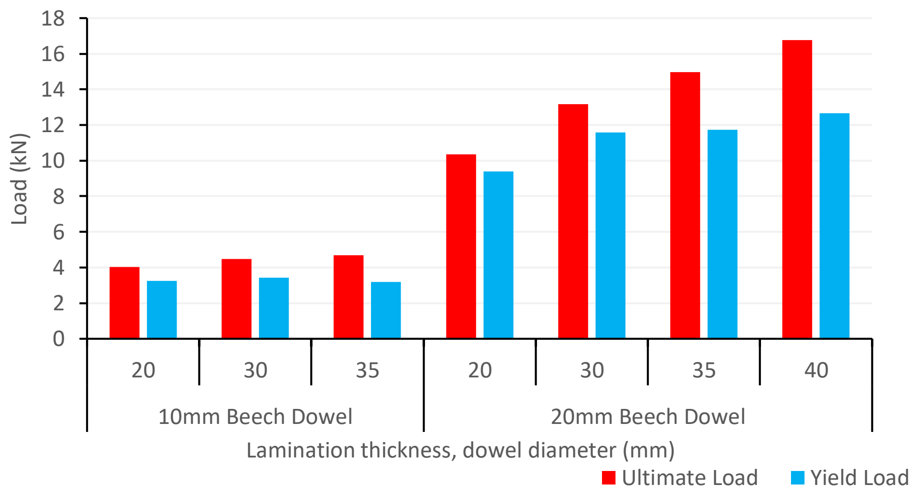

Figure 7-12: Comparison of the mean ultimate and yield load of beech dowels in UK Sitka spruce.

The average yield load followed the same pattern, apart from the 10mm beech in 35mm thick laminations that had a yield load less than the 20mm and 30mm thick laminations, see Figure 7-12. This can potentially be explained by the smaller sample size (see Table 7-3), and the effect one or two weaker samples can have on the mean values of the whole sample group.

The increase in the stiffness of the connection was apparent but far less pronounced than the ultimate or yield load, see Figure 7-13. Similar to larch connections; the strength achieved from the connections formed using 8mm and 10mm diameter mild dowels was below the strength of 20.50mm beech dowels. However, the values obtained for the stiffness of the connection was comparable.

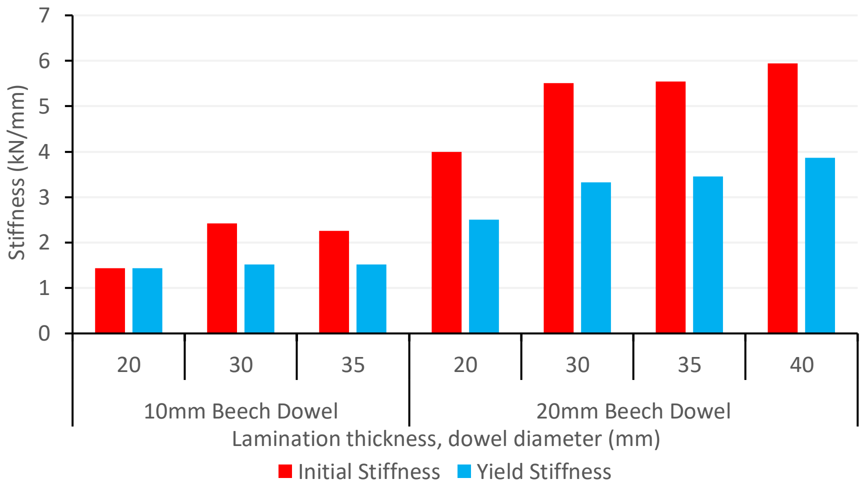

Figure 7-13: Comparison between the mean yield stiffness and the initial stiffness of beech dowels in Sitka spruce.

For the test specimens with both diameter beech dowels, the load slip response varied with regard to the failure mechanism that occurred within the connection. The next section gives a brief overview of the mode failures of all timber connections outlined in section 5.2.1 that were witnessed in the testing and when they predominately occurred.

7.3.2.1. Mode III_m - failure

Single yielding per shear plane (i.e. Mode III_m in the NDS) occurred alongside bearing in the central member at the smallest ratios of dowel diameter to central member thickness (i.e. t/D ≤1). Bearing failure in the side members in these instances was typically avoided by the provision of a central member with a thickness equal to half the combined width of the side members. The proximity of the hinges formed in the dowel from each shear plane created one single hinge forming in the centre of the dowel span. The load slip response in these instances was at first linear elastic, followed by a non-linear elastic range up to the yield point. Post yield the specimens suffered a large decrease in load capacity, see Figure 7-14. The confinement induced by the central member was not sufficient to halt the progressive failure of the dowel in bearing, bending and shear. After the yield point is reached a brash brittle failure occur in the dowel occurs.

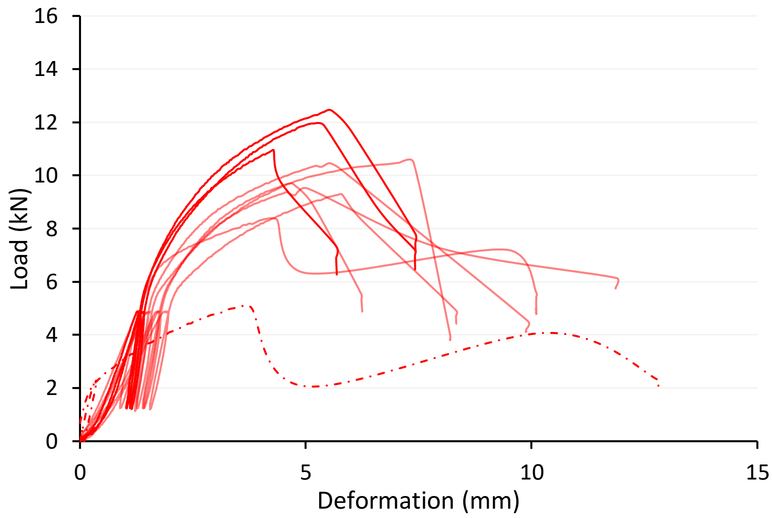

Figure 7-14: Load deformation curve from testing of $B20_{par}SS20$, illustrating III_m failure response.

Significant variation could be seen in the strength and stiffness behaviour of one sample group ($B20_{par}SS20$) because of one single anomalous result (B41) caused by the presence of a defect in the dowel that caused a premature shear failure through the centre of the dowel, shown dashed in Figure 7-14. This sample was therefore removed from the derivation of the mean values illustrated in Table 7-7.

7.3.2.2. Mode V_d - failure

Failures caused by the formation of two plastic hinges per shear plane alongside localised bearing failure were sometimes difficult to discern from visual inspection of the failed specimens but through examination of the load-slip response the failure mechanism was often clear. This mode of failure (V_d) first discussed by Schmidt and McKay (1997) is specific for all-timber connections and predominantly occurs in samples with a ratio of between 1-1.75 of dowel diameter to central member thickness. The behaviour pre-yield is initially linear elastic followed by a region of non-linear elastic behaviour at approximately $0.5F_{max}$. Post yield the connection experienced an almost perfectly plastic response similar to the connection behaviour using metallic dowel type fasteners although some hardening was witnessed. After peak hardening, there was a linear reduction in strength followed by a brittle failure often at a load below the yield point, see Figure 7-15. Here, the occurrence of a plastic plateau is due to the confinement of the dowel within the connection, any hardening in the connection strength can be seen as a by-product of the formation of hinges and is a function of the stiffness of the timber in the dowel and the material surrounding the dowel.

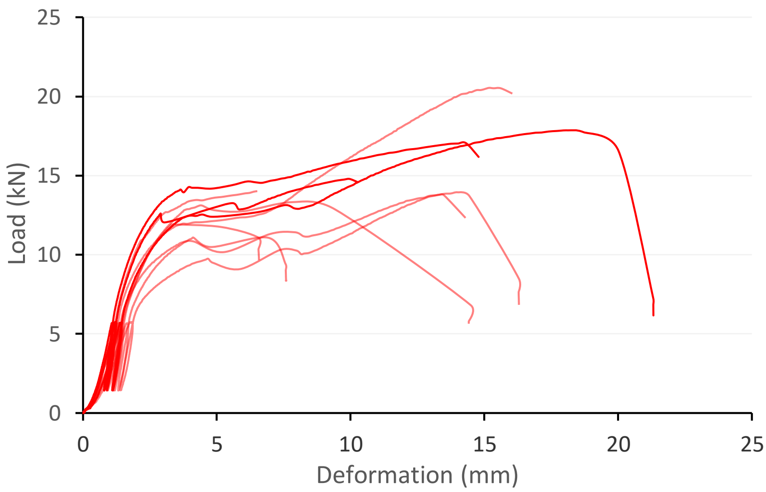

Figure 7-15: Load deformation curve from testing of $B20_{par}SS35$ illustrating V_d failure response.

The stiffness variation across the dowel and the side members means that a simultaneous formation of the hinges across the dowel cannot be guaranteed. Photographs taken of the final deformed hardwood dowels removed from failed connections test illustrate plainly the inability of some dowels to form hinges due to the variation in stiffness and resistance in the dowels and the side members, see Figure 7-16.

Figure 7-16: Irregular formation of hinges within in Beech dowel during V_d failure.

7.3.2.3. Mode IV - failure

Above the t/D ratio of 1.75 the shear span of the dowel is great enough to allow two distinctly different hinges to form in the dowel across the width of the central member. This is most commonly seen in a proportionally flexible peg. The load slip response correlates closely with the behaviour of the connection that forms a single central hinge, however post yield after a period of plasticity the connection exhibits an approximately bi-linear response with an increase in strength at a much higher level than the yield point. After which, the load strength rapidly decreases in a linear fashion until brittle failure of the dowel occurred. Where the formation of two plastic hinges per shear plane (see Figure 7-18) was prevalent a long plastic plateau response with some hardening was witnessed in all instances prior to this brittle failure, see Figure 7-17. This ductile response can be assured if the relative proportions between dowel diameter and the central member thickness are maintained.

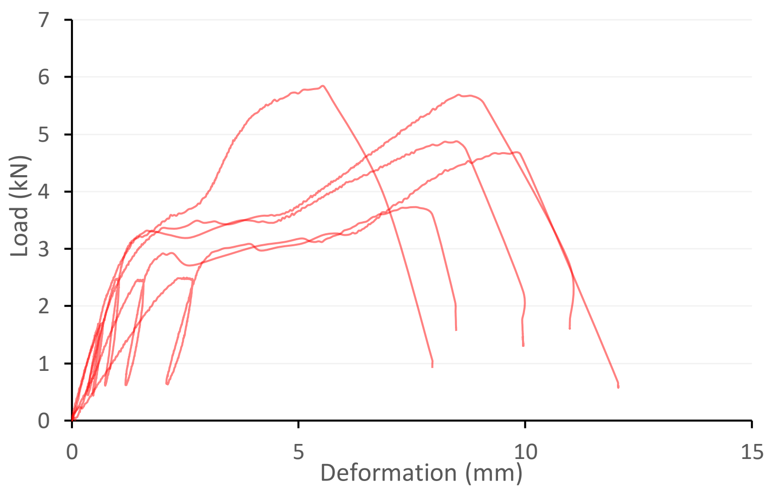

Figure 7-17: Load deformation curve from testing of $B10_{par}SS35$ illustrating IV failure.

Figure 7-18: Formation of two hinges per shear plane within a dowel during mode IV failure.

7.3.2.4. Steel dowels - failure

All specimens that incorporated steel dowels exhibited a linear load-slip response up until the dowel or the timber began to yield. After which, an almost perfectly plastic response followed with little or no hardening until the connection failed at high levels of deformation either through direct embedment in the central member (EYM mode 1 failure) or in combination with dowel yielding (EYM mode 2 failure). Dowel yielding accompanied embedment failure when the ratio of dowel diameter to member thickness ratio was $^1/_3$ or below. Where yielding of the dowel was witnessed an increased level of post yield hardening was encountered, Figure 7-19.

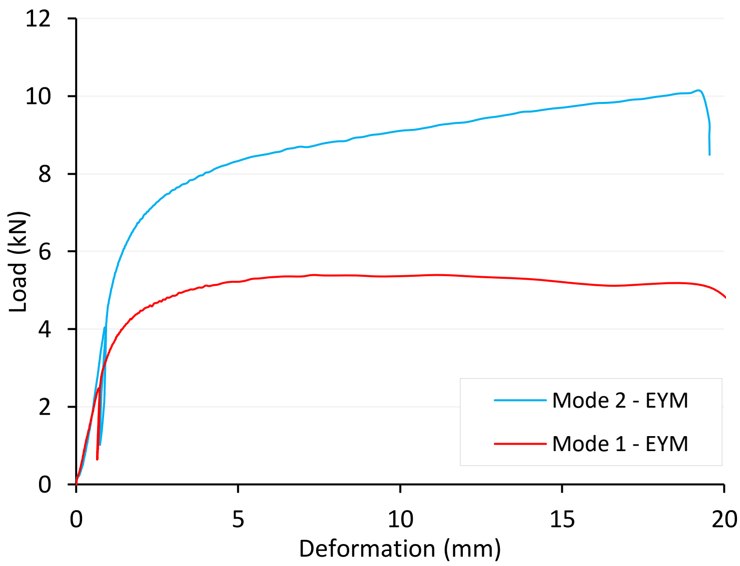

Figure 7-19: Typical load deformation curves from parallel to the grain connection tests using steel dowels - depicting EYM mode 1 & 2 failures.

7.3.3. Perpendicular to grain - Larch connections

The mean results for the twelve different connection configurations for perpendicular to the grain are shown in Table 7-8, including ultimate load

(F_{max}), yield load, yield stiffness and initial stiffness (k_i) using the method described in BS EN 26891 (1991).

Test series	Dowel species	Ultimate load		Yield Load		Initial stiffness		Yield Stiffness	
		Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
A20$_{perp}$L30	Ash	11.27	9.12	10.74	11.34	4.54	2.72	2.43	5.57
B10$_{perp}$L30	Beech	3.19	9.05	2.32	9.26	1.41	32.13	0.99	15.68
B20$_{perp}$L30		10.97	13.14	9.99	14.64	4.61	8.56	2.88	21.05
B10$_{perp}$L40		4.41	23.65	2.22	16.20	1.26	16.10	0.89	6.67
B20$_{perp}$L40		13.32	5.77	10.14	14.50	4.50	13.85	1.97	21.79
O20$_{perp}$L30	Oak	8.44	7.84	6.16	12.22	2.29	21.19	1.76	13.21
S20$_{perp}$L30	Sycamore	7.81	14.22	7.02	15.89	2.84	34.68	1.96	14.10
S$_L$8$_{perp}$L30	Steel	9.63	11.49	5.37	9.27	2.09	24.33	1.93	14.13
S$_L$10$_{perp}$L30	Steel	12.35	7.37	8.32	7.29	5.00	5.56	2.41	5.02

Table 7-8: Summary of double shear plane connection test using UK larch.

The typical load slip response of each connection type tested in the perpendicular to the grain orientation using 30mm laminations and 20mm dowels are shown in Figure 7-20.

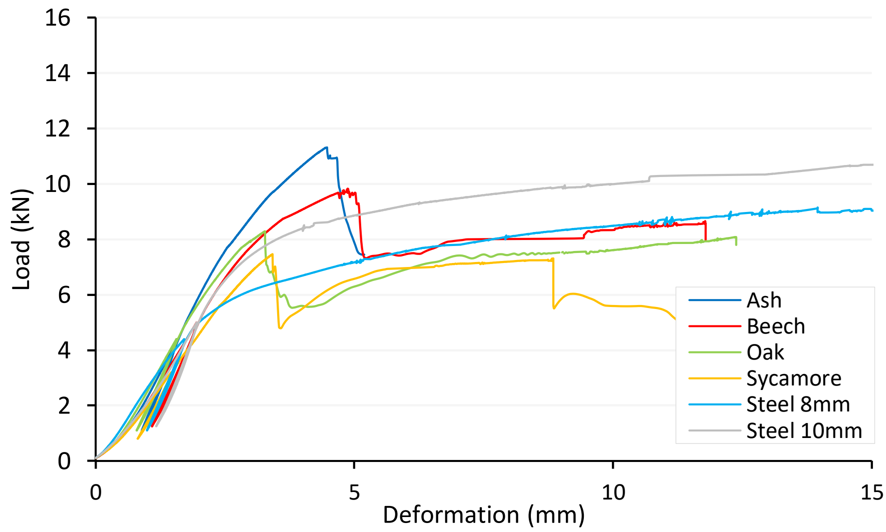

Figure 7-20:Typical load slip response of selected 20.50mm diameter hardwood dowel and mild steel dowel fasteners in perpendicular to the grain double shear connection tests using UK larch.

When the connections were tested in the perpendicular to grain orientation all hardwood timber dowels experienced a significant drop in strength capacity after the yield point was reached. Prior to the yield point a period of non-linear behaviour was experienced up until the initial yield failure. From this initial failure only a marginal increase in strength was seen and the increase in strength up until final failure was far below the yield point level. The steel dowels did not experience a precise yield point but rather they exhibit a non-linear response during yielding and continue to harden after the limit of proportionality was passed up until ultimate failure. Throughout all the load slip responses of both steel and hardwood the initial stiffness and response was similar.

The results showed that 20.50mm Ash dowels had the greatest ultimate and yield load capacity of all the tested hardwood dowel types. When used in 30mm laminations, the connection had a mean ultimate load capacity of 11.27kN and yield load of 10.74kN, a comparison of the mean ultimate and yield loads is shown in Figure 7-21. Unlike the parallel to grain connections the maximum mean ultimate load of the 10mm diameter steel dowels was greater than both the 20mm diameter ash and beech dowels.

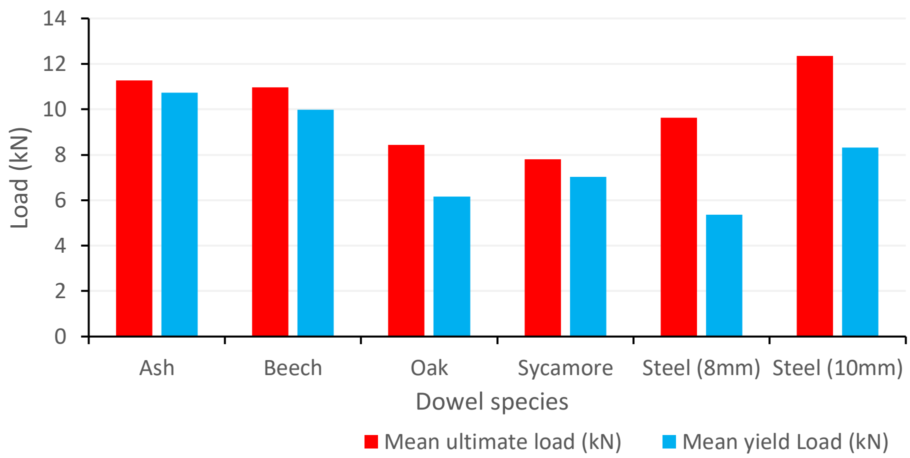

Figure 7-21: Comparison of the mean yield and ultimate loads of 20.50mm diameter hardwood dowels and 8-10mm mild steel dowels in 30mm diameter larch laminations

Overall the connection capacity of the perpendicular to grain orientation was lower than that of the parallel to grain connection test conducted using UK larch, see Figure 7-22.

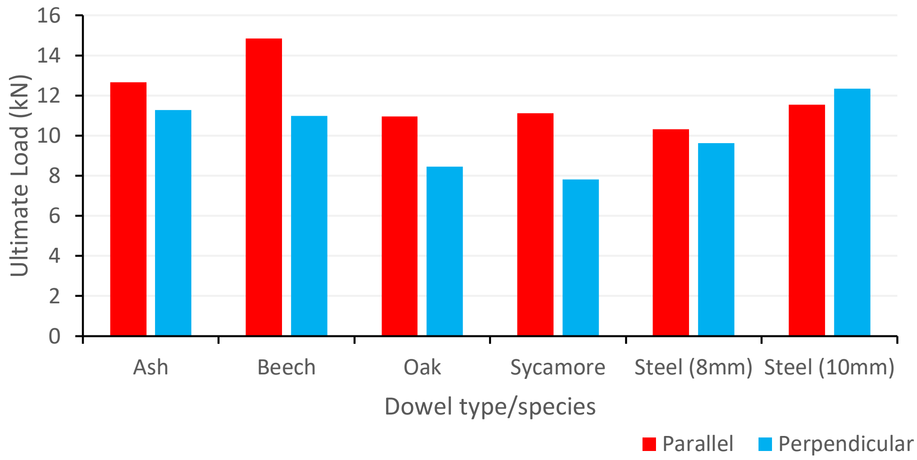

Figure 7-22: Comparison between the ultimate load capacity of 20.50mm diameter hardwood dowels and mild steel dowels in both the parallel and perpendicular to grain orientation.

The ultimate load values of the connections formed using timber dowels were more affected by the grain orientation of the outer laminations than the mild steel dowels. There was the greatest disparity between the strengths of the two orientations when beech dowels were used. The initial stiffness of both ash and beech dowels was almost double the stiffness of oak and sycamore and only slightly less than the comparative 10mm steel dowel, see Figure 7-23.

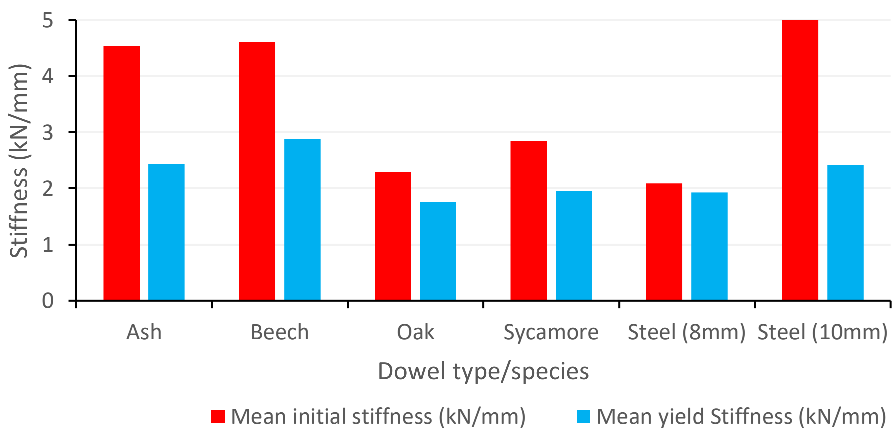

Figure 7-23: Comparison between the initial and yield stiffness of UK larch in perpendicular to the grain connections.

The results indicate that neither an 8mm or 10mm steel dowel will give an accurate representation of a hardwood dowel in an all timber larch connection orientated perpendicular to the grain. The values of an 8mm mild steel dowel did not correlate with the strength and stiffness of either an oak or sycamore dowel, likewise an 10mm mild steel dowel overestimated the connection strength properties of a connection formed with an ash or beech dowel.

From these results there is evidence that the increase of the lamination thickness has a greater effect on the load capacity of a connection than is experienced in the parallel to grain orientation, (see Figure 7-24). When the ratio between the lamination thickness, t and the dowel diameter, D is larger there is a more noticeable increase in maximum load capacity between connections formed of the same diameter dowel, i.e. t/D = 3 (3.19kN) to t/D =4 (4.41kN) than t/D =1.5 (10.97kN) to t/D = 2 (13.32kN). This indicates that the embedment strength in the perpendicular to grain orientation increases by a greater proportion when the widths of the members are increased.

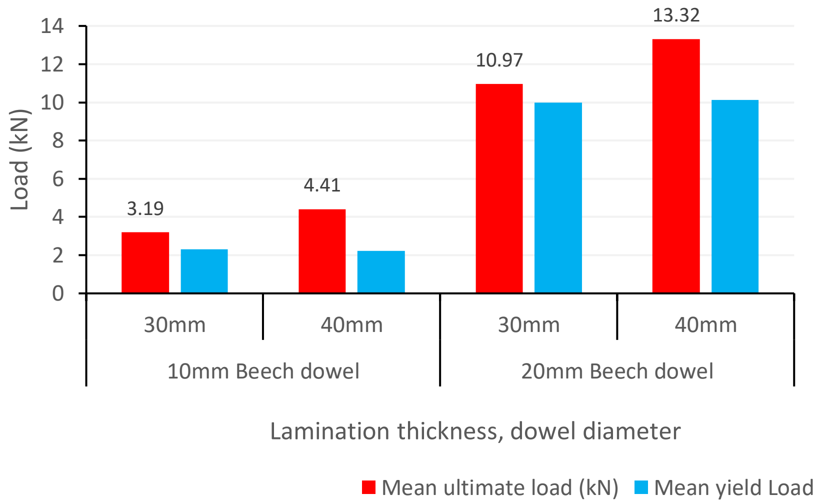

Figure 7-24: Comparison of the mean yield and ultimate load of beech dowels in UK larch with varying diameter and thickness.

7.3.4. Perpendicular to the grain – Sitka Spruce connections

The results for the perpendicular to the grain tests using Sitka spruce are shown below in Table 7-9, including ultimate load (F_{max}), yield load, yield stiffness and initial stiffness (k_i).

Test Series	Ultimate Load		Yield Load		Initial Stiffness		Yield Stiffness	
	Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
$S08_{perp}SS20$	5.93	11.85	3.91	6.52	1.96	51.04	1.66	30.64
$S08_{perp}SS30$	7.46	15.72	4.36	26.82	2.47	14.59	1.83	11.17
$S10_{perp}SS20$	8.36	9.83	5.14	6.45	2.02	35.69	1.60	21.15
$S10_{perp}SS30$	8.97	6.03	7.26	17.93	2.40	54.88	1.97	41.95
$B10_{perp}SS20$	2.92	23.55	2.41	22.24	1.51	33.70	1.17	27.25
$B10_{perp}SS30$	4.12	20.75	2.98	15.16	1.75	26.58	1.33	18.33
$B10_{perp}SS35$	4.96	17.96	3.33	6.62	1.69	29.66	1.30	26.77
$B20_{perp}SS20$	10.96	15.90	8.78	17.41	2.97	23.39	1.94	18.19
$B20_{perp}SS30$	10.71	9.45	10.24	7.90	3.14	13.92	2.19	7.90
$B20_{perp}SS35$	11.71	10.06	11.06	7.79	3.52	15.64	2.41	21.23
$B20_{perp}SS40$	12.19	10.06	11.27	6.02	3.86	12.27	2.63	8.50

Table 7-9: Test results for perpendicular to grain connection tests.

The mean values recorded from all the test groups showed a definite increase in strength and stiffness when the member thickness was increased. The increase in ultimate load witnessed between dowels of the same diameter over differing lamination thickness was proportional when using the two different dowel diameters. The increase in load capacity between lamination thicknesses was more pronounced with the smaller dowel diameter, see Figure 7-25. The 20.50mm dowels in 20mm thick laminations proved to be a slight anomaly and the average ultimate load was higher than the corresponding connection using 30mm laminations. The average yield load followed the pattern of increasing with a greater increased dowel diameter and wider laminations.

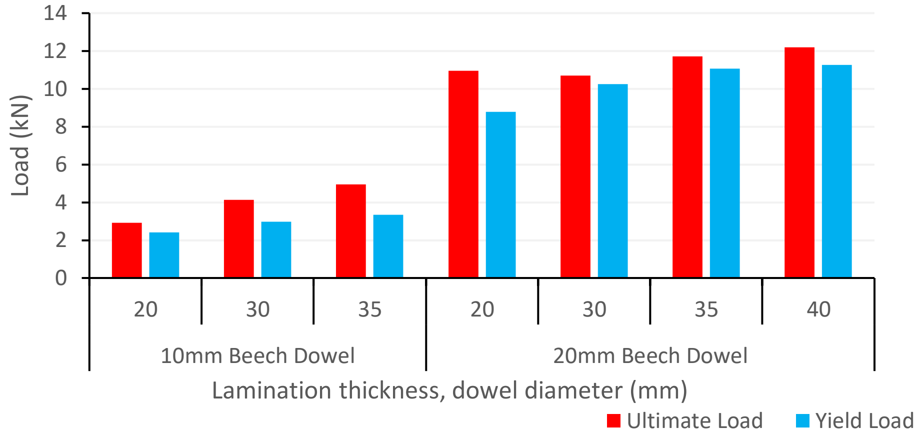

Figure 7-25: Comparison of the mean ultimate and yield load of beech dowels in UK Sitka spruce perpendicular to the grain.

The increase in the stiffness of the connection was apparent when the lamination thickness increased, see Figure 7-26. Comparable to the larch connections, the strength achieved with 8mm and 10mm diameter mild steel dowels was not consistent with 20.50mm beech dowels, but the values obtained for the stiffness of the connections were comparable.

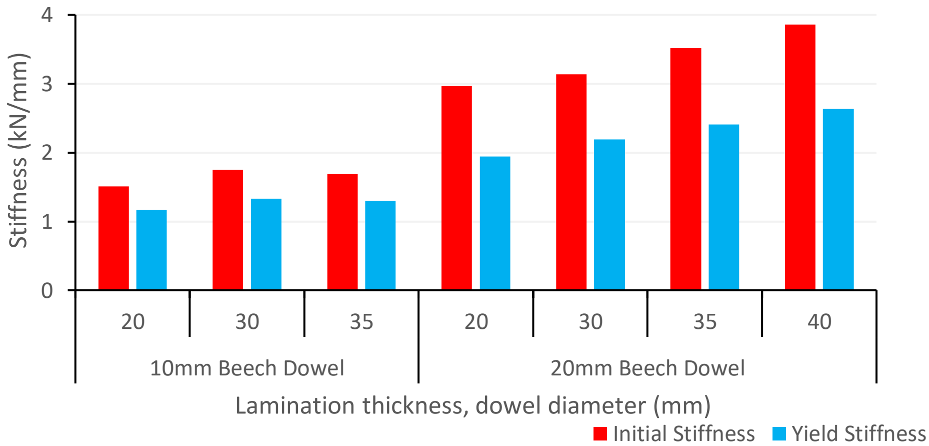

Figure 7-26: Comparison between the mean yield stiffness and the initial stiffness of beech dowels in Sitka spruce perpendicular to the grain.

On the whole, the strength and stiffness of the perpendicular to grain connection arrangements were lower than their parallel to grain counterparts. However, this was not always the case where the dowel diameter was smaller or the t/D ratio was closer to parity. Here there was a smaller difference in the maximum capacity in the parallel to perpendicular orientation, see Figure 7-27.

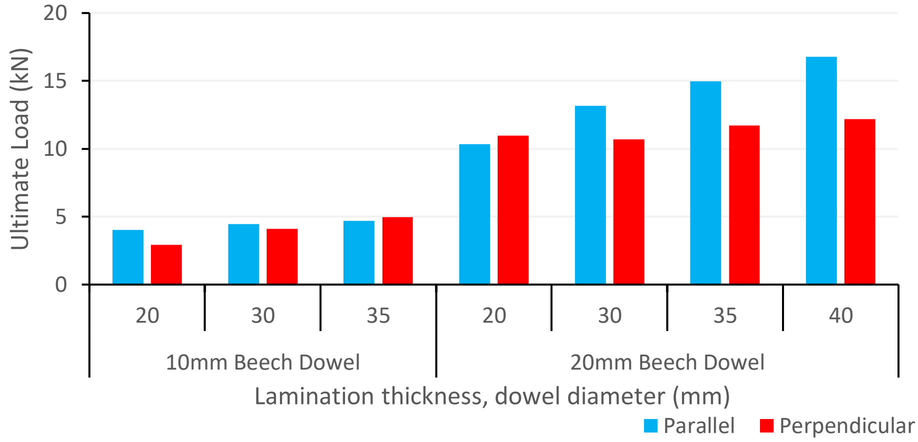

Figure 7-27: Comparison between parallel to grain connections and perpendicular to grain connections using UK Sitka spruce.

The load slip response of the connection varied depending on the failure mechanism that occurred within the connection. The next section gives a brief overview of the mode failures of all timber connections outlined in section 5.2.1 that were witnessed in the perpendicular to grain testing and when they predominantly occurred.

7.3.4.1. Mode I_m failure

At low shear spans of between 1 – 1.5 times the dowel diameter, failure occurred in a progressive manner and the dowel failed in bending following embedment failure of the loaded edge of the dowel (Mode I_m). The response was typically linear up until approximately 40% of the ultimate load, followed by a non-linear elastic portion until the yield point. Following yielding of the connection there was a linear reduction in strength due to progressive bending failure occurring in the dowel. Confinement by the central member and friction effects occurred in line with phase 4 characteristics illustrated in Figure 5-10 and shown experimentally in Figure 7-28. The steepness of the gradient depended on the extent of confinement of the dowel. This was exacerbated at lower ratios of dowel diameter to central member thickness as an earlier more brittle failure would occur. Bearing dominated failure in the side members was mitigated throughout these tests by the ratio of central member to side member included in these tests.

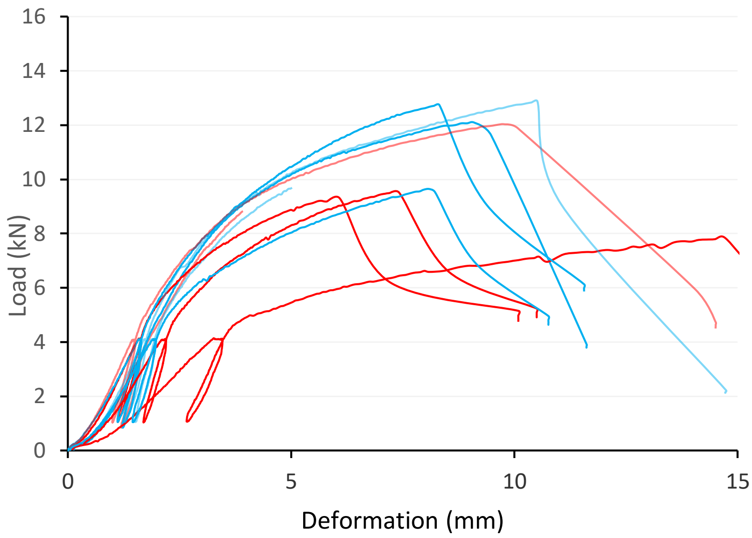

Figure 7-28: Load slip response curves from perpendicular to the grain testing of series $B20_{perp}SS20$ illustrating failure mode I_m.

7.3.4.2. Mode III_s and III_m failure

Between shear spans of 1.5-2 times the diameter there was a marked drop in the connection capacity immediately following the yield point, Figure 7-29.

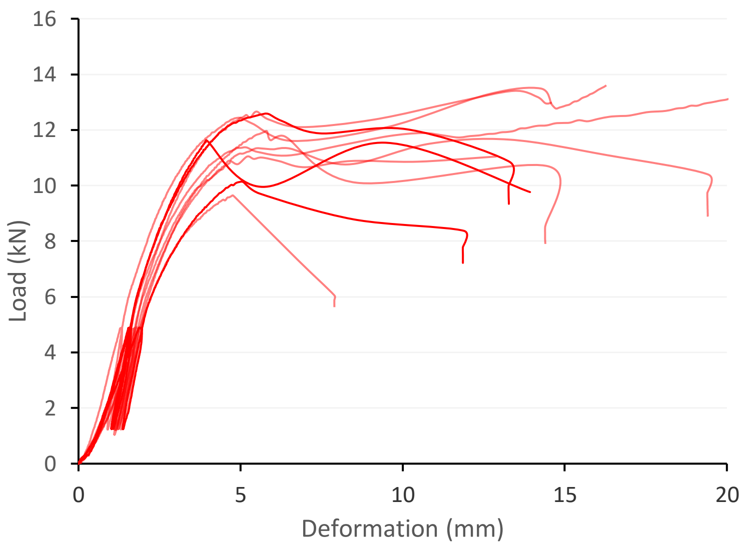

Figure 7-29: Load deformation curves from perpendicular to the grain testing of $B20_{perp}SS35$ illustrating III_m failure.

The capacity of the connection was maintained following the slight abrupt drop in strength (to approximately 80% F_{max}) and plastic behaviour proceeded to occur until embedment failure or splitting of the side members occurred at high levels of deformation. Thomson (2010) attributed the stepped response of the load deformation plots conducted using GFRP dowel to progressive plug shear and splitting of the side members. Where a stepped response is seen here during these tests it is more than likely due to the progressive confined shear of the dowel. Both failure modes III_s and III_m previously discussed in section 5.2.1.2 were witnessed in the timber dowel connections. The variation between the occurrences of the two failure modes III_s and III_m was dependent on numerous factors. The location where the plastic hinge formed in the dowel was fundamental to the point of rotation of the dowel about the centre of the connection. Where the formation of a hinge or bending of the dowel occurred principally about the centre of the sample, other hinges could not form due to separation of the members. Lack of confinement in the central member could cause high levels of non-uniform load across the width of the side members causing increased probability of embedment failure in the outer edges of the side member. These localised embedment failures did not affect the overall integrity of the connection due to the dispersal of localised embedment stresses and a perfectly plastic response of the load deformation curve that immediately followed the yield point. Embedment failure tended to occur when the shear span of the dowel was between 1.5-1.75 times the diameter and the width of the side members was approximately 1.5 times the diameter. Here the embedment stresses applied to the outer edge of the side members were not great enough to cause splitting of the side members after large amounts of deformation occurred and did not provide enough load resistance in the side members for a further hinge to form in the dowel. An III_s failure mode was more common in these connection geometries because the embedment strength and stiffness of the parallel to grain central member were proportionally higher than the combined strength and stiffness of the two side members undergoing embedment perpendicular to the grain, see Figure 7-30.

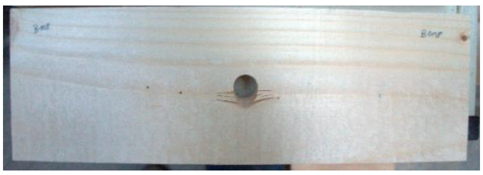

Figure 7-30: Localised embedment failure in a side lamination.

7.3.4.3. Mode IV failure:

A Mode IV failure mechanism allows the formation of two hinges in the dowel across the width of the central member thickness with final failure occurring due to a combination of shear, bending and bearing of the dowel. Mode IV failures occur as expected in flexible dowels that have larger shear spans located in connections that have sufficient edge and end distances to resist the shear and tensile forces induced by the dowel. The behaviour pre-yield was initially linear elastic followed by a region of non-linear elastic behaviour. Post yield the connection experienced an almost perfectly plastic response similar to the connection behaviour using metallic dowel type fasteners with hardening witnessed. It is postulated that the formation of the two central hinges precedes the formation of the hinges in the side members. The formation of which creates the bi-linear response that is witnessed in the load deflection curve, see Figure 7-31. After peak hardening, there is a sudden steep reduction in the strength of the connection as the hinges created fail in shear in a brash manner.

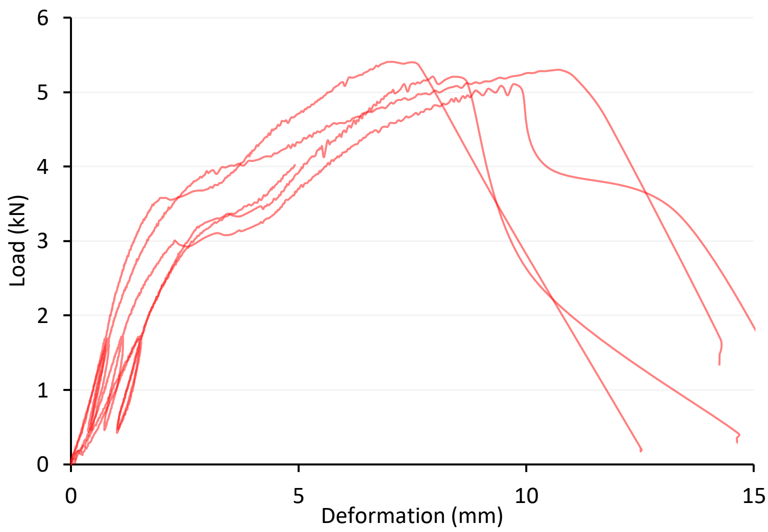

Figure 7-31: Load deformation curve for perpendicular to the grain connection test series B10$_{perp}$SS35 - depicting failure mode IV.

7.3.4.4. Steel dowel failure

All specimens that incorporated steel dowels in the connection exhibited a linear load-slip response up until approximately 40% F_{max} after which non-linear elastic behaviour dominated up until the yield point. Following the yield point an almost perfectly plastic response was observed with little or no hardening. Any hardening that did occur was in line with the previous parallel to grain results, whereby the higher the ratio between central member thickness and dowel diameter the greater the amount of hardening occurred.

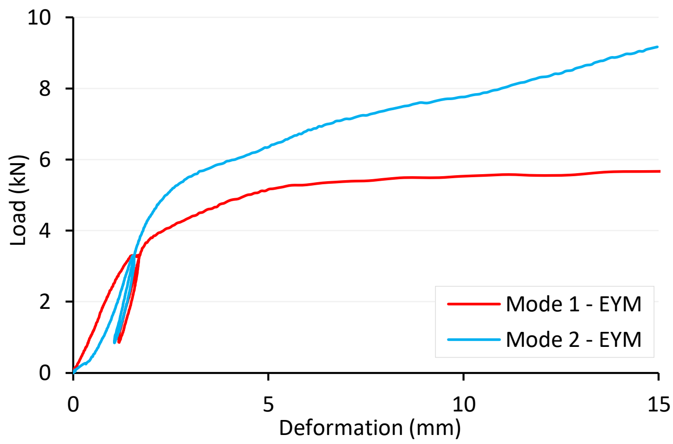

Figure 7-32: Typical load deformation curves for perpendicular to the grain connection tests using steel dowels - depicting EYM mode 1 & 2 failures.

7.4. Additional shear plane connection tests

The standardised test on connections is conducted on an isolated joint arranged so that two shear planes are induced. Within a DLT panel, the connections created between the laminations by the hardwood dowel can be sheared in a far greater number of shear planes than is typically represented in connection testing depicted in BS EN 26891 (1991). Section 5.1.5 describes the current method for analysis according to BS EN 1995-1-1 (2004), for a greater number of shear planes. This method extrapolates the isolated compatibility of each shear plane in accordance with the analysis of a double shear plane connection given in BS EN 1995-1-1 (2004) and provides a strength value based on the minimum connection strength across a shear plane. Whilst this is deemed as a satisfactory compromise for connections formed with metallic based fasteners, the potential variation of the yield modes created from the modes already depicted through EYM theory and the additional yield modes for timber dowels independent of BS EN 1995-1-1 (2004) (described in section 5.2.1) creates an uncertainty in the existing analysis procedure that will need to be investigated prior to DLT manufacture.

To ensure compatibility of the timber connections within a DLT panel to the theory discussed in section 5.2, the test schedule was expanded to include dowel connections that were loaded across four shear planes. These tests included parallel to the grain as well as perpendicular to the grain test arrangements using the same diameter beech dowels over a variety of differing lamination thicknesses. Whilst the test setup for these connections differed from the double shear plane tests (and will be illustrated in detail later) the production of the raw material for the laminations and the dowels was identical to the processes described in section 7.2 . The final fabrication of the connection followed the same principle as the double shear plane, albeit with more laminations to arrange.

7.4.1. Test setup

7.4.1.1. Parallel to the grain

The test setup was conducted in a similar manner to the previous parallel to grain connection tests conducted using in section 7.2.3, with the addition of two further laminations, Figure 7-33.

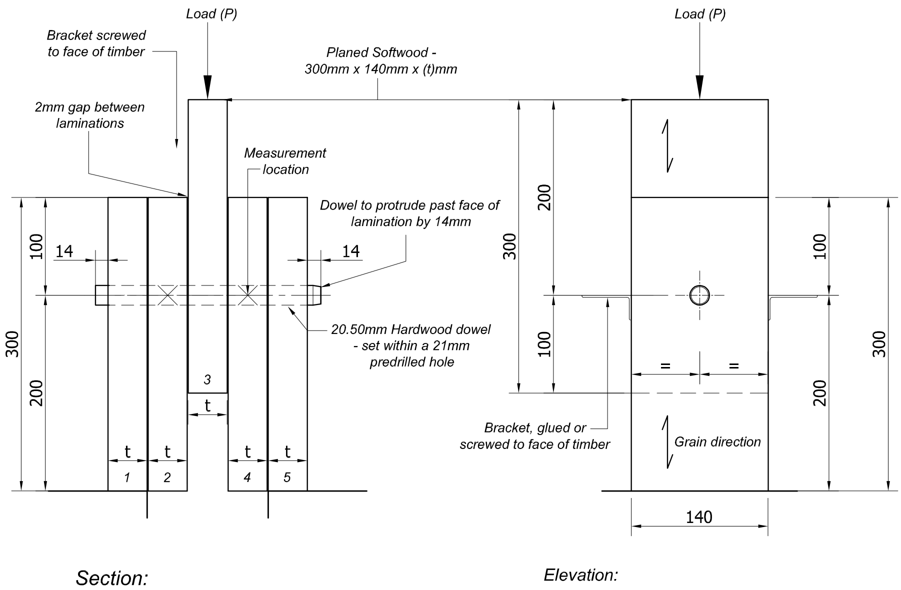

Figure 7-33: Additional shear plane tests parallel to the grain.

For each specimen, continuous deformation readings were taken to an accuracy of 1% using LVDT's fixed to small steel brackets fixed to either side members 2 and 4 (Figure 7-33). An unstressed location was selected where the average of the two values front and back determined the relative slip of member 2 and 4 and the overall deformation was taken as the average displacement of members 2 and 4. Load was applied by a universal testing machine and both load and slip were recorded using a data acquisition system. The test was conducted using live deflection reading. The ultimate load of the connection was defined when the overall value of slip reached was approximately 15mm or a significant failure occurred compromising the integrity of the connection.

7.4.1.2. Perpendicular to the grain

The test arrangements for the perpendicular to the grain tests are shown in Figure 7-34. Load was applied in compression at a constant load rate through the top of the central member with a steel bearing plate to remove the potential for localised crushing at the head of the connection. The outer side members were supported on 50mm wide steel plates set on rollers at either end. Deformation was recorded by placing LVDT's on the top edge of the outer laminations (as close as possible to their centre point).

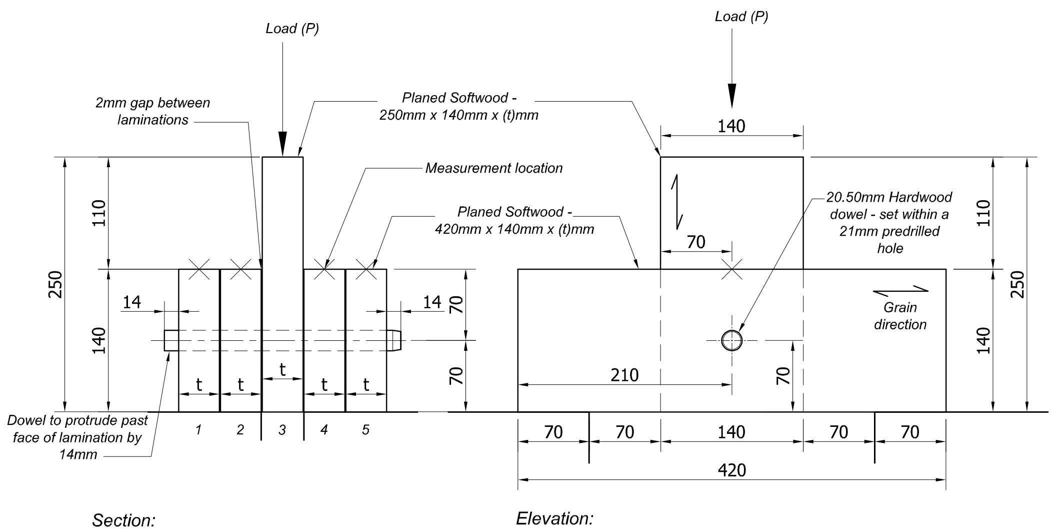

Figure 7-34: Additional shear plane tests perpendicular to the grain.

Displacement was recorded in the loaded central member through cross head displacement. Average deformation of the connection was calculated at each shear plane as the average displacement between the symmetrically placed LVDT's about the central member (i.e. the average of transducer 1 and 5 and the average of transducer 2 and 4); see Figure 7-34. The overall deformation of the connection was the average displacement across all four transducers. The compatibility of the central displacement recorded by the load cell could not be used to negate the bending of the sample over the loaded span so the results incorporate the additional bending of the members induced by a point load in the centre of the span.

7.4.2. Four plane shear plane testing

In total, 25 parallel to grain tests were conducted and 20 perpendicular to the grain double shear plane arrangements in Sitka spruce were tested. Three different lamination thicknesses were tested and the results are summarised in Table 7-10. Each specimen was then loaded along the width of the central member via a 100 x 300 x 12mm steel bearing plate to minimise local indentation occurring at the loading head and additional steel bearing plates were used at the points of support. The loading profile and rate was based on the previous tests conducted on double shear plane tests using beech dowels and Sitka spruce members in accordance with BS EN 26891 (1991). Load was then applied at a constant rate proportional to its estimated maximum load (F_{est}). Load was placed on the beam until 40% F_{est} was achieved from which point the load on the test piece was reduced to 10% F_{est}. After which, load was applied at a constant rate of 2.5kN/min until 70% F_{est} when the loading rate dropped to 1.25kN/min and continued at this rate until failure occurred.

Test series	Lamination thickness (mm)	Dowel diameter (mm)	Side member orientation	t/D ratio	No. of samples
$5B20_{par}SS20$	20	20	Parallel	1	13
$5B20_{perp}$ SS20		20	Perpendicular	1	9
$5B20_{par}SS30$	30	20	Parallel	1.5	4
$5B20_{perp}$ SS30		20	Perpendicular	1.5	5
$5B20_{par}SS35$	35	20	Parallel	1.75	8
$5B20_{perp}$ SS35		20	Perpendicular	1.75	6

Table 7-10: Test series for four shear plane connection tests.

The ultimate load (F_{max}) and the yield load was defined as previously stated in section 7.3.2. The initial connection stiffness was calculated from the gradient of the line that corresponds with the initial points of 10% F_{max} and 40% F_{max} on the load deformation curve as shown in Figure 7-7. The yield stiffness was taken as the secant modulus at the yield load, assuming a linear elastic response up until that point.

7.4.3. Results: Parallel to the grain - 4 shear plane testing

Table 7-11 shows a summary of the mean ultimate load, the mean yield load, the mean initial stiffness and the final stiffness of each test series for parallel to grain connections over four shear planes.

Test Series	Ultimate Load		Yield Load		Initial Stiffness		Yield Stiffness	
	Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
$5B20_{par}SS20$	9.59	17.20	7.47	27.43	5.88	25.21	3.07	32.98
$5B20_{par}SS30$	12.14	14.19	8.57	42.68	3.80	14.08	2.11	35.82
$5B20_{par}SS35$	15.41	11.91	10.38	18.29	4.71	18.24	2.76	14.70

Table 7-11: Results from 4 plane shear planes parallel to the grain.

Similar to the double shear plane tests there was a distinct increase in strength with an increasing ratio of t/D, however unlike the double shear planes no consistent increase in stiffness was achieved over an increasing ratio. Final failure often occurred with the connection separating with a brash shear failure of the dowel. Often the shear integrity of the dowel was compromised at the yield point - highlighted by the slight reduction in strength immediately after yield point was reached. Deformation continues until the confinement of the dowel within the laminations effectively causes embedment to occur in the side members. Embedment deformation continues until the rotation of the joint or the dowel was enough to allow the dowel to separate along the initial line of shear failure, see Figure 7-35.

Figure 7-35: Shear failure of dowel following large rotation.

Where the outer laminations of the connection were not of a sufficient width to resist the embedment caused by the rotation of the dowel within the connection, little or no hardening of the connection was witnessed, see Figure 7-36. The inclusion of additional shear planes limited the occurrence of immediate brittle failure after yield point and allowed for the plastic response caused by embedment to dominate the yield characteristics.

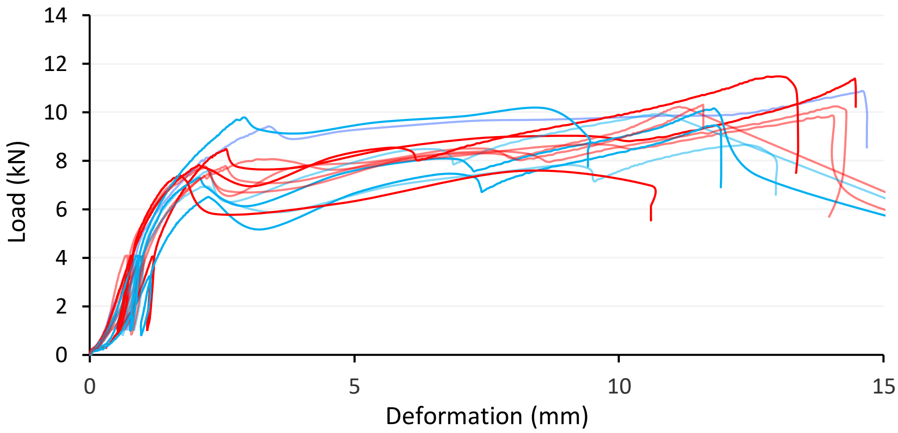

Figure 7-36: Load deformation curve for 4 plane shear test using 20mm laminations parallel to the grain.

Where the connection was formed at greater t/D ratios the final brash shear failure of the dowel was evident, however its effects were less pronounced until final failure. Both the dowel along its length (see Figure 7-37) and laminations in the immediate surrounding of the dowel experienced embedment. Here, no sudden drop of strength was noted directly after the yield point, rather consistent yield hardening was witnessed.

Figure 7-37: Shear failure of beech dowel with accompanying embedment.

At a t/D ratio of 1.5 there appeared to be a small plateau after the yield load that may suggest rotation occurred in the connection because the dowel is already effectively held in position and yield hardening is allowed to commence, see Figure 7-38.

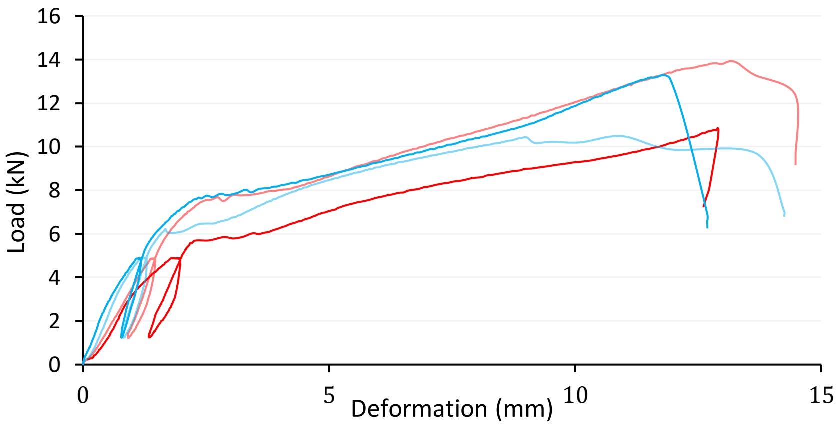

Figure 7-38: Load deformation curve for 4 plane shear test using 30mm laminations parallel to the grain.

Increasing the ratio of t/D caused a more pronounced bi-linear response of the connection under load, i.e. no rotation of the dowel is allowed, as it is effectively restrained by the laminations, at all times (see Figure 7-39). Due to the nature of final failure in all the four shear plane tests conducted in the parallel to the grain orientation, the final stiffness can be attributed to the combined embedment stiffness of the timber foundation.

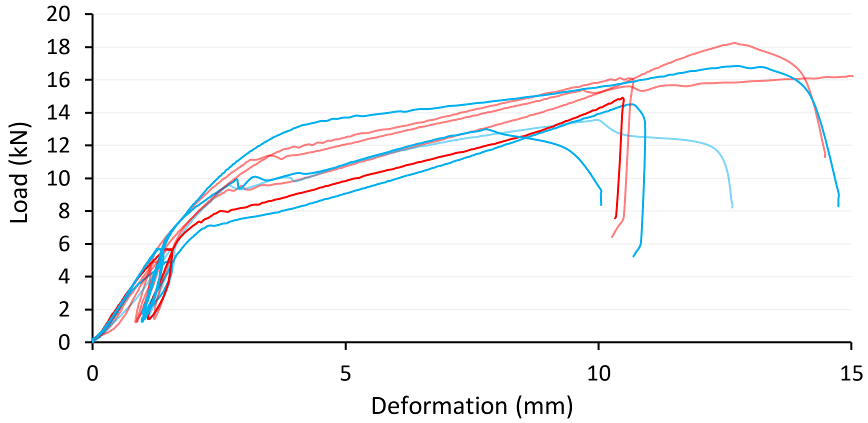

Figure 7-39: Load deformation curve for 4 plane shear test using 35mm laminations parallel to the grain.

7.4.4. Results: Perpendicular to the grain - 4 shear plane testing

Table 7-12 shows a summary of the mean ultimate load, the mean yield load, the mean initial stiffness and the mean yield stiffness of each test series for perpendicular to grain connections over four shear planes. The results presented followed the same methodology of the parallel to grain tests discussed previously in section 7.4.1.1.

Test Series	Ultimate Load		Yield Load		Initial Stiffness		Yield Stiffness	
	Mean (kN)	COV (%)	Mean (kN)	COV (%)	Mean (kN/mm)	COV (%)	Mean (kN/mm)	COV (%)
$5B20_{perp}SS20$	5.98	11.15	4.46	34.87	5.32	26.20	4.46	34.87
$5B20_{perp}SS30$	9.05	14.58	6.47	3.95	5.27	16.49	3.32	26.37
$5B20_{perp}SS35$	11.52	4.55	8.02	20.25	5.53	23.08	2.99	22.77

Table 7-12: Results from 4 plane shear planes perpendicular to the grain.

Similar to the double shear plane tests there was a distinct increase in the ultimate load and yield load with an increasing ratio of t/D, however unlike the double shear planes no consistent increase in stiffness was achieved over an increasing ratio. In four shear plane connections the load responses were at first, linear elastic, followed by a non-linear elastic range up to the yield point. Unlike the parallel to grain samples, at smaller ratios of t/D the post yield response of perpendicular to grain specimens suffered a large decrease in load capacity (see Figure 7-40), at low levels of deformation. The stiffness of the 20mm dowel and 20mm lamination thickness did not allow large deformations to occur in the connection before brittle failure. Failure occurred before 4mm deformation in all instances. The addition of another lamination did not provide the confinement required to resist the failure of the dowel in bearing, bending and shear after the yield point is reached i.e. a brittle bending failure occurred in the dowel immediately after the yield load.

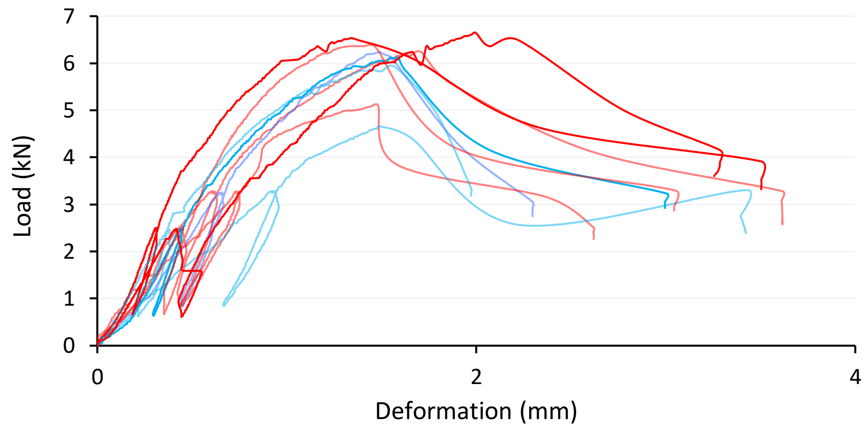

Figure 7-40: Load deformation curve for 4 plane shear test using 20mm laminations perpendicular to the grain.

At greater ratios of t/D there was not an immediate reduction of strength following the yield load. Similar to the parallel to grain samples but slightly more pronounced in the perpendicular to grain samples a plateau occurred immediately after the yield load, see Figure 7-41. Here the rotation that was allowed to occur was greater due to the reduction in stiffness that was apparent from embedment in differing angles to the grain as discussed in section 6.6.4.

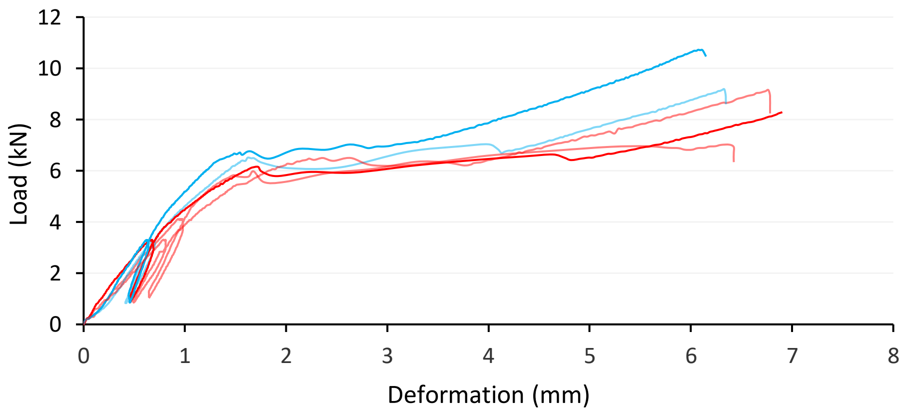

Figure 7-41: Load deformation curve for 4 plane shear tests using 30m laminations perpendicular to the grain.

Mirroring the parallel to grain samples; at ratios of t/D above 1.5 there is a bi-linear response under load in the perpendicular to grain arrangement (see Figure 7-42). Here the hardening of the final gradient can be related to the combined embedment of both the dowel and the lamination.

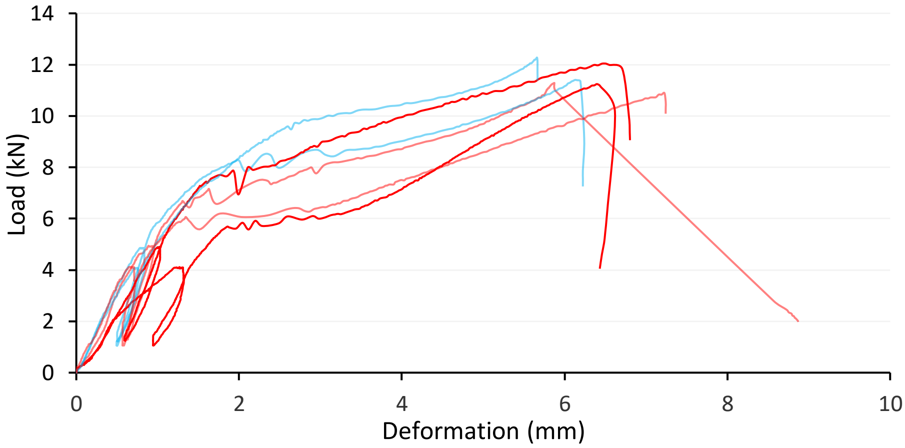

Figure 7-42: Load deformation curve for 4 plane shear tests using 35mm laminations perpendicular to the grain.

7.5. Discussion

Eighty larch samples were tested and 136 Sitka spruce samples in total were tested in double shear connections. The results indicate that ash and beech provided the best yield capacity of all the dowel types tested. The strength and stiffness recorded for the oak and sycamore dowels was 20% lower in some instances. The large values of COV witnessed can be attributed to the limited number of tests and the variations that occur naturally in the softwood laminations and the hardwood dowels. Dinwoodie (2000) states that it is not uncommon for the design parameters such as modulus of elasticity and bending strength to have a COV value between 15-30% and below 15% for other biological properties. The diameter of the dowels means any variation in their properties could have a significant influence on the variability witnessed between tests, this will include slight changes in the grain and differences in density. All dowels and laminations were inspected prior to processing and manufacture for defects but the larch lamination material had a high propensity for knots and although every effort was made to remove these knots prior to testing unseen defects cannot be ruled out in their entirety.

Looking at the failed specimens the hardening that was witnessed in some of the dowels was attributed to a combination of embedment in the laminations and crushing of the dowel, see Figure 7-43. Whereas in the samples where a stepped response occurred in the load slip curve there was clear indication that the failure in the dowel was commonly witnessed often in four different locations. The formation of a hinge in the dowel at these locations would ultimately end in brittle cross grain shear failure across the entire cross section at the point of failure, see Figure 7-44.

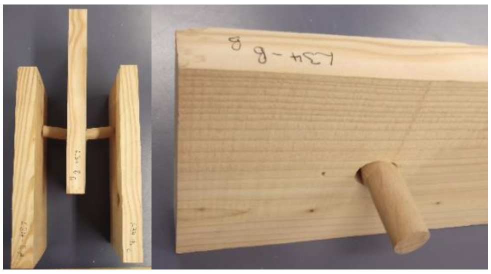

Figure 7-43: Embedment of laminations and crushing of hardwood dowel.

Figure 7-44: Cross grain shear failure of hardwood dowels.

The behaviour of the dowel within the connection varied greatly. On one hand the dowels formed hinges and ultimate failure was caused through cross grain shear (see Figure 7-45) and on the other hand the dowel failed through bending in the central part of the dowel (see Figure 7-46). Often these differences occurred in the same test group and the difference in behaviour could be apportioned to local differences in relative strength and stiffness of the dowel and the laminations.

Figure 7-45: Three hinge failure.

Figure 7-46: Brittle bending failure.

The influence of shear stiffness is witnessed in almost all of the dowels, with either a central hinge or four points of failure being seen throughout the testing samples. In timber this ratio between the elastic modulus and shear modulus is much higher and shear deflection will contribute a much larger proportion of the overall deflection in the connection (small shear spans occur within a connection of these types). When the dowel and the laminations have a greater disparity between their respective characteristic densities different shear spans are created that could adversely affect the performance of a connection overall. Figure 7-47 shows the fibres in the dowels deformed due to shear, accompanied by the beginning of bending failure in the top edge of the dowel due to hinge formation.

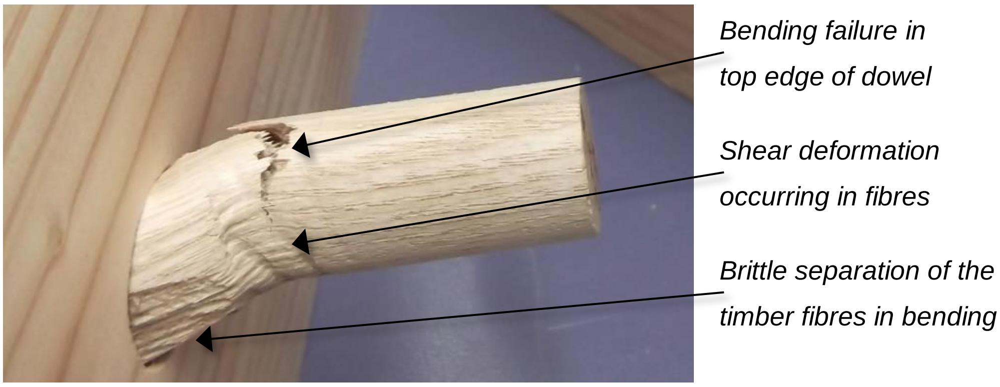

Figure 7-47: Failure of a timber dowel.

Once the yield point is reached the strength of the connection continues up to the yield load but is accompanied by large deformations due to embedment of the outer laminations. Every effort was made to ensure that the individual pieces of the connection did not have contact at the start of the test. During the tests the pieces tended to bear together as the fixing in the sample deformed. Contact between the members affects the strength of the connection but only by a marginal proportion and would occur normally in almost every type of dowel-type connection test, timber or steel. Where the direct bearing of the members would have the most effect on the results of the test is on the stiffness parameters. It would be expected that the stiffness of the connection would be increased once the members began to touch and friction forces were being developed, but this was not evident in the tests conducted.

When using a timber dowel, embedment will occur in the dowel as well as within the members of the connection. The amount of embedment will depend on the relative stiffness' of the dowel and parent connection material. The abrupt failure response witnessed in the perpendicular to the grain arrangement is an instance of this as the central member was loaded parallel to the grain and therefore would have a higher embedment stiffness relative to the side members.

Hinges formed in the beech dowels due to a combination of confinement in the central and side members and the moment resistance in the dowel. The formation of plastic hinges in the dowel through the thickness of the central and side member did not occur simultaneously. In several instances, and in particular, where the formation of two plastic hinges per shear plane in the dowel was prevalent, significant post yield ductility with some hardening was experienced. The amount of ductility post yield varied depending on the relative proportions between dowel diameter and the central member thickness. The laminations wedged together and could effectively pin the dowel at a point of support, this may not occur when the members are held in place during testing. How effectively the dowel is held in place will be a function of the embedment strength of the lamination and the bending resistance of the dowel. The material properties of an individual timber element can be hard to discern using conventional methods used on other materials. Therefore, the point at which plasticity is assumed to commence cannot be verified accurately and will change depending on the localised properties of a piece of timber at that location.

Stages of failure in an all timber connection can be broken down as follows:

- Dowel rotation and flex
- Partial shear failure
- Continual dowel rotation until confinement
- Embedment
- Total shear failure of dowel section

The load slip response from connections with a t/D ratio of 2 or above indicate that some form of plasticity is achieved after yielding but prior to brittle failure of the dowel, see Figure 7-48. It was previously surmised that this is caused by the wedging and the increased frictional resistance of the connection as the dowel rotates causing the side members to wedge against the central member. Similar to the findings of Shanks, Chang and Komatsu (2008) plasticity was witnessed inconsistently at large deformations. Due to the small quantity of samples tested this cannot be relied upon for any additional strength capacity in the connection type.

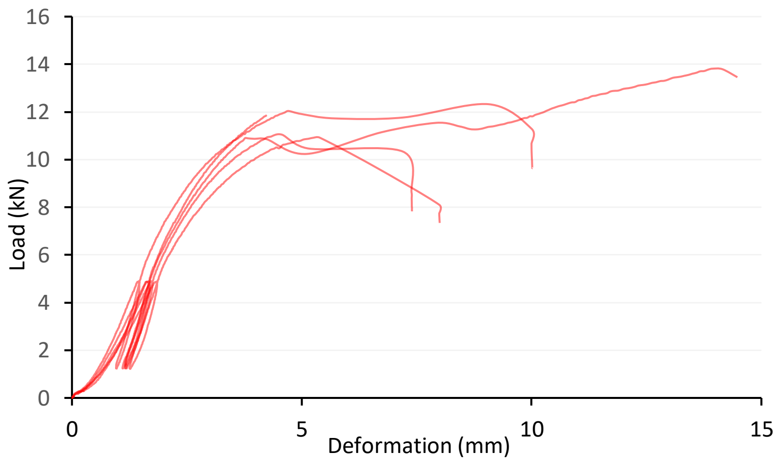

Figure 7-48: Load deformation curve from perpendicular to the grain connection tests series B20$_{perp}$SS40.

The load deformation curves of the perpendicular to grain test show that a ductile failure was witnessed when the ratio of dowel diameter to central member thickness was increased beyond 1.5. Below this ratio brittle failure occurred through embedment of the dowel in the central member immediately after the yield point was reached. In both strength and stiffness parameters the COV for the 10mm beech dowels is greater than the 20mm beech dowel. This is unsurprising because any slight variation in the material quality will have a far greater effect on the smaller volume dowels.

Sandberg *et al.* (2000) noted only a small, non-consistent variation between parallel to the grain and perpendicular to the grain double shear plane tests using Eastern White pine and Sugar maple and a similar comparison can be made between the double shear plane tests conducted here. The COV for the ultimate load and the yield load provided from these tests indicate that the levels of variance were within expected bounds for timber testing (i.e. within a range of 0-25%) as indicated by Dinwoodie (2000). Generally, the yield load was more consistent than the ultimate load and any anomalous levels of variance can be explained by either a lower number of test specimens in the test series (i.e. 3 or below) or sets using smaller elements or volumes that would be more affected by the presence of defects across the cross section. Interestingly the yield stiffness and the initial stiffness showed compatible amount of variance across the test series. This phenomenon can be explained by the initial linear behaviour of these connections (after the initial slip) and the non-linear behaviour that occurs at a level of between 40-60% that creates a gradual yielding behaviour to occur across the connection. The load-slip response from the all-timber double shear plane connections complemented the findings from Bulleit *et al.* (1999). Namely, where all parts of the connection had similar bearing properties due to equal densities there was a linear response up until approximately 40% of the ultimate load and any increase of this linear response was due to the embedment behaviour of the timber involved and the moment resistance of the dowel.

The ultimate load had greater amount of variation than calculated for the yield load and this was not unexpected as after yielding the behaviour and performance of the connection can vary greatly due to the failure mode recorded. The number of influencing factors that can cause a variation in the type of failure are numerous, including dowel and timber member material variance and the exact proportional sizes of the members. It should be noted that when the material consistency of the dowel is assured (i.e. a homogenous material) the variation of ultimate load is reduced and the failure mode can be more assured. However, the variation for the yield load witnessed for both the steel and timber dowels are very similar indicating that behaviour up to the yield point for both timber and metal dowel-type fasteners have a similar amount of variation and it is only the post yield behaviour of the timber dowels that varies greatly.

Overall, stiffness has a greater variation than the strength because of the embedment characteristics of timber. Greater variation was witnessed in the initial stiffness than the secant stiffness to the yield load. Significant variation in connection stiffness can generally be accredited to natural variations in the material properties (Thomson *et al.* 2010). Again, this is expected because the initial stiffness will incorporate some of the initial slip and the beginning stages of embedment that could be dependent on the variation of finish from the predrilled hole in the timber. The secant modulus however will ignore these initial variations. Interestingly, the dowel diameter did not have a significant effect on the initial stiffness. This can be explained by the fact that any relative increase in embedment stiffness is not proportional to diameter increase (Brandon *et al.* 2014a).

A comparison between the two and four shear plane tests in both the parallel and perpendicular orientation indicates there are subtle differences between their performance when additional shear planes are incorporated into the connection arrangement. At lower ratios t/D in the parallel to grain arrangement, the double shear plane arrangement achieved a higher ultimate load, when this ratio increased, the ultimate load of the four-plane shear test was greater, see Figure 7-49. The stiffness values in the parallel arrangement of four shear planes showed no correlation with increasing lamination thickness, throughout the arrangements.

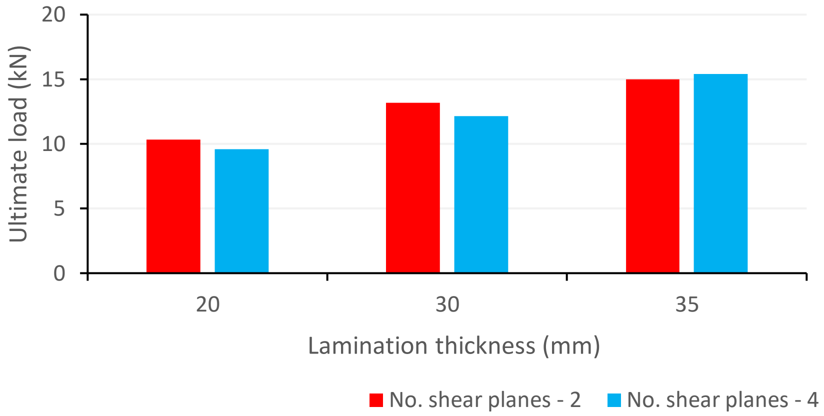

Figure 7-49: Comparison between the ultimate load of two and four shear plane connections loaded parallel to the grain using 20.50mm diameter beech dowels and UK Sitka spruce.

The results indicate that there are large variations in the stiffness values for large shear planes due to the disparities in embedment stiffness in the laminations. In the perpendicular arrangement the ultimate load capacity of the connection followed the same comparison profile of the parallel arrangement. Whereby at lower ratios of lamination thickness and dowel diameter the two-shear plane arrangement had a greater strength capacity but at higher ratios the four-shear plane arrangement had a more comparable strength. In the four-shear plane perpendicular to grain connection arrangement the stiffness across the different lamination thicknesses tested was comparable. The stiffness in the four-plane arrangement was almost double the two-plane arrangement at 20mm lamination thickness. Whilst the difference in stiffness values between the two arrangements (two and four shear planes) reduced as the lamination thickness became greater there was still a pronounced stiffness variation between the arrangements, see Figure 7-50.

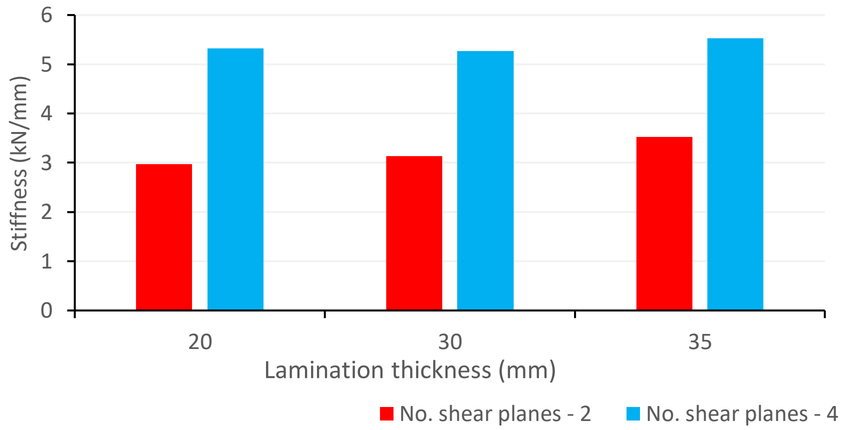

Figure 7-50: Comparison between the ultimate load of two and four shear plane connections loaded perpendicular to the grain using 20.50mm diameter beech dowels and UK Sitka spruce

Between the two and four shear plane test series the shear planes were kept identical so it would stand to reason that the lowest performing shear plane (strength wise) would determine the overall strength of the connection. The indication from the results is that the four-shear plane connection could be assigned a strength classification based on the lowest performing shear plane when the ratio between the dowel diameter, d and the lamination thickness, t is beyond 1.5. The overall strength of the connection in both parallel and perpendicular to grain orientation was similar although the behaviour after yielding of a connection formed of more shear planes was more pronounced, and a bi-linear response was witnessed prior to failure.

7.5.1.1. Brittle failure perpendicular to the grain

Where the ratio of the dowel diameter to side member thickness became greater than 1.5 but less than 2, a central hinge formed in the dowel and the large amount of embedment failure occurred in the side members. In these instances, the lever arm of the dowel from the central point of rotation was increased creating additional embedment stresses to be created across the width of the connection. Often, the continuing rotation of the dowel overcame the shear and tensile perpendicular to the grain resistance of the side members and caused splitting across the grain to occur in the side members. Burnett *et al.* (2003) reported immediately prior to the brittle failure, a small crack was witnessed at the top of the peg hole due to perpendicular to the grain tension or shear stresses that accompanied a subsequent relaxation of the joint followed by the cleavage of the material. In some instances, this phenomenon was witnessed before a catastrophic failure of the side members. High levels of deformation were seen in all tests that experienced a brittle failure outlined in section 5.1.6. It is highly likely that the formation of stress cracks may have been imperceptible to the human eye or located within the thicknesses of the material prior to cleavage of the members.

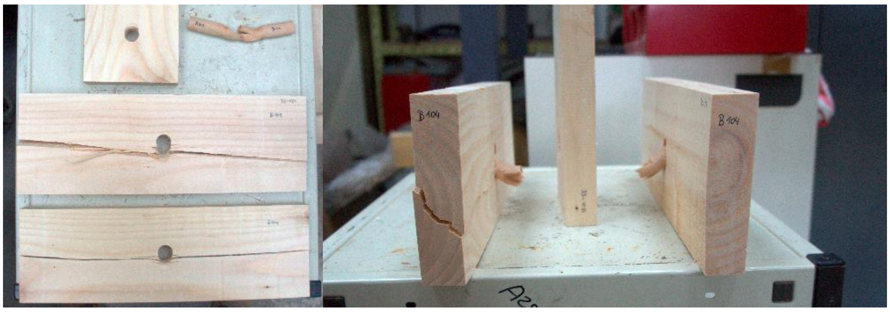

Figure 7-51: Perpendicular to the grain splitting of the side members.

Splitting failure of the outside members perpendicular to the grain tended to occur between failure mode III_m and V_d depending on the level of confinement and the overall stiffness of the peg. Often failures caused by tension splitting perpendicular to the grain occurred in combination with the brash failure of the dowel. Where the shear span of the dowel and the ratio between the dowel diameter and the side members increased beyond 2 and provided that the timber dowel was relatively stiff, the embedment resistance provided by the side members and central members is such that large levels of embedment failure did not occur. The dowel is allowed to rotate large amounts and fail in a manner due to cross-grain shear (failure mode V_d), see Figure 7-52. The increased rotation of the dowel causes a non-uniform embedment stress to be induced across the width of the side member creating a stress concentration that promotes splitting caused by the tension stress perpendicular to the grain.

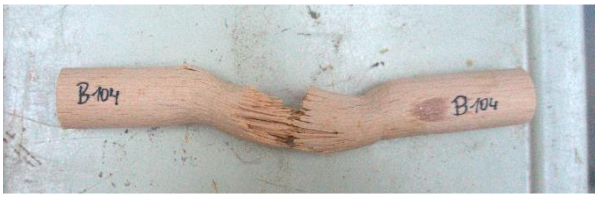

Figure 7-52: Cross grain shear failure of beech dowel.

7.6. Conclusion

The conundrum of the correct selection of the timber components for optimal panel construction has been analysed in detail through the comparative analysis of the connection tests. These tests carried out encompass the majority of known interactions occurring within the connections and these have been tested in controlled experimental conditions. For the purposes of this study the selection of a single hardwood dowel species to continue to full testing was chosen to be beech due to its comparatively similar behaviour in failure to a steel dowel.

Often the ultimate load failure led to a rapid loss in the strength of the connection due to brittle splitting occurring in the laminations. Brittle modes of failure were shown to occur in the perpendicular to grain orientation when the ends of the dowel were allowed to rotate. The tensile failure perpendicular to the grain was not explicitly addressed in this preliminary study, but may be a limiting factor in the configuration of a connection within a DLT panel.

8. Evaluation of full-scale panels

The potential uses for UK grown and produced DLT products are numerous and have been discussed in detail in section 2.1.2. For the purposes of this study a DLT floor panel was identified as the best possible avenue for entry into the UK marketplace. And therefore, this chapter experimentally appraises seven DLT panels created using the UK home grown resource. The evaluation of the panel was based on standardised testing regime in accordance with BS EN 408 (2010) for structural timber to obtain and assess the strength and stiffness properties of the produced DLT panel. The objective of these tests was to understand the enhanced performance, if any, of a DLT panel over a traditional timber joist floor and gather information that could form the basis for UK grown DLT. Originally UK homegrown Sitka spruce laminations were expected to be supplied this but due to lack of supply and the increased amount of UK grown larch that was available during the SIRT study, a proportion of the larch that was supplied for was used to form DLT panels.

8.1. Panel production

Prior to production of the seven DLT panels, larch pieces with an initial target dimension of 150 x 32mm were dried in a portable dehumidifier kiln to approximately 10% ±2% moisture content to achieve a final processed target dimension of 140 x 30mm, as outlined previously in section 6.2. After the drying process the operative confirmed approximately 25% of the dried batch was deemed unsuitable for further use due to excessive amount of distortions occurring within the timber. The remaining samples of UK Larch were measured, weighed and acoustically graded at MAKAR Ltd in March 2015, using a handheld resonance grading machine (MTG 960) and visually checked against the override criteria as described by EN 14081-1 (2011) by the researcher. Twelve additional pieces were removed using this method, eight from excessive twist, three due to bow and one from cupping of the sample.

100 pieces of UK larch with a nominal cross-sectional dimension of 30 x 140mm were selected and measured. The moisture content for each specimen was taken using a portable reader (GANN Hydromette M 4050) and the average moisture content of the samples after drying was found to be 9.46%. Mean average density of the sample set was calculated and found to be 469.43kg/m^3. The frequency of the piece was measured using the MTG 960 (see Figure 8-1) and the Stress Wave Velocity, V and Dynamic Modulus of Elasticity E_d were then calculated in accordance with the equations given in 3.1.9. The mean Dynamic Modulus of Elasticity, E_d at 12% Moisture Content was then transformed to a Static Modulus of Elasticity E, by the grade setting of the device. And in this instance a factor 0.91 was applied, to give a mean E of 9.70kN/mm^2.

Figure 8-1: Acoustic grading in progress.

The calculated E for each sample then was treated as the indicative property to determine its grade classification as stipulated in BS EN 338 (2009). Each sample was then assigned a grade, see Figure 8-2. All the samples within the study were graded at C16 or above and over 95% were graded at C22 and above as they had an E greater than 10kN/mm^2.

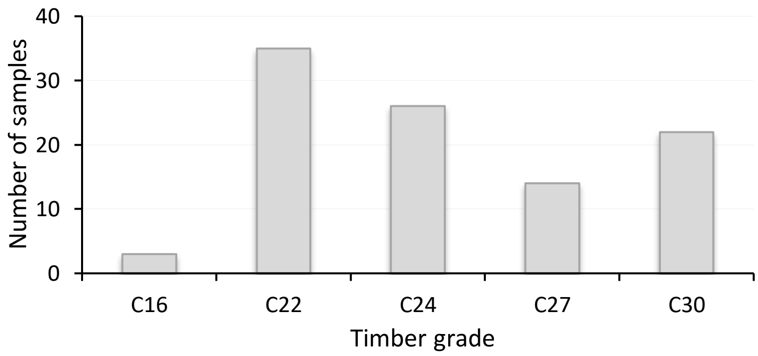

Figure 8-2: Results of acoustic grading on subset of the UK larch provided.

The larch samples were then used to produce a series of 300mm wide x 140mm deep x 3000mm long panels in a process previously described in 2.1.1. The pieces were stacked on their long face, clamped together and predrilled with 21mm diameter holes at 300mm centres, see Figure 8-3. Beech dowels of 20.75mm diameter (pre-dried to approximately 6% moisture content) were then inserted through the predrilled hole using a pneumatic press to bind the laminations together.

Figure 8-3: Clamping and drilling of laminations.

The beech dowels for the panels were supplied from the same subset of samples obtained for this study and tested as in section 6.5. The dowel ends were trimmed flush to the face and the panels were then planed on all sides to ensure the correct dimensional tolerance and a suitable aesthetic appearance for when the panel is exposed, Figure 8-4.

Figure 8-4: Larch DLT panels pre and post planing.

The panels were constructed in 300mm widths for number of practical reasons. Beyond 300mm the spoil produced from the drilling process was too great and could not be removed easily causing the machinery and the timber to overheat. Drill bits greater than 300mm in length also had a tendency to veer off the vertical when a large amount of force was exerted on an unrestrained drill piece. Finally, it was found that the modified frame press that was used to construct the panel could not achieve the necessary pressure to drill through a greater width than 300mm. To ensure quality assurance in the panel production and allow for easier on site handling it was decided that the width of the individual panels should be limited to a maximum of 300mm wide.

Seven of the finished panels were shipped to a warehouse in Edinburgh and the remaining panels were used in a new build in the Scottish Highlands, discussed further in section 9.1. The panels transported to Edinburgh were stored and conditioned in an internal SC1 environment for a period of six months. The internal conditioning of the specimens allowed a more consistent moisture profile through the whole panel to be achieved and ensured a more consistent bond was achieved through the laminations.

8.2. Experimental tests

To investigate the strength and stiffness of a UK produced DLT panel, two stages of static bending tests were undertaken on the seven different UK larch DLT panels in conjunction with Edinburgh Napier University in the winter of 2016.

8.2.1. Three-point load bending tests

The first stage of testing involved loading the panels within their proportional limit using a single central point load, to ascertain the initial stiffness of the panel and understand the load distribution that occurred across the laminations. The arrangement for this test is illustrated in Figure 8-5

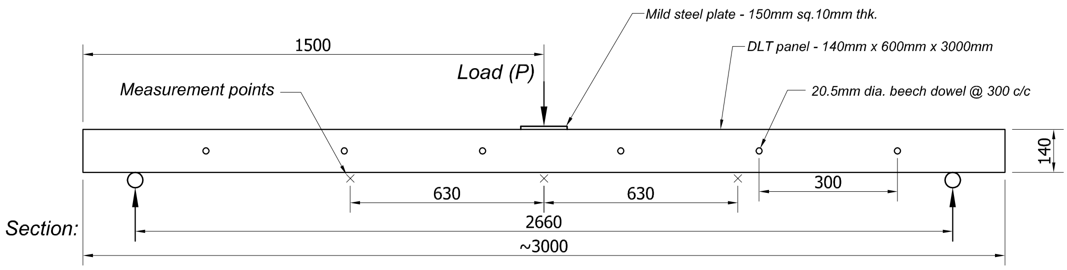

Figure 8-5: Full-scale DLT panel three point bending test arrangement.

The DLT panel was simply supported by roller supports at either end of the sample over a clear span of 2520mm, i.e. 18 x depth of the panel in accordance with BS EN 408 (2010). The roller supports provided a continuous bearing across the full width of the sample, with steel spreader plates located at both support positions and points of load application. Care was taken to ensure that the position of the dowels in the panel were at the same location relative to the load in all instances, i.e. one dowel located in the centre of the span.

The central load was transferred to the specimen using a square 150mm x 10mm thick steel plate located over the central four laminations only. For each specimen, continuous deformation readings were taken at an accuracy of 1% using LVDT's. LVDT's were placed at several locations on the underside of the panel to evaluate deflection. Three LVDT's were placed to align with the centre and outer laminations. Furthermore, the deflection of the outermost laminations at two diagonal locations 630mm away from the centre of the sample were recorded, see Figure 8-5 and Figure 8-6.

Figure 8-6: Positioning transducers for three-point bending test.

Load was applied at a constant rate of 0.08 mm/s until approximately 40% of the estimated maximum load, at this point the load was removed to ensure limits of proportionality were not exceeded and that the section was not damaged. The estimated 40% maximum load was based on the bending strength of the timber determined by the grade allocation (conducted in section 8.1), the geometric section properties of the timber directly underneath the load and the moment induced by the method of loading.

8.2.2. Four-point load bending tests

The second stage of testing was undertaken to evaluate the bending strength parallel to grain ($f_{m.k}$), local modulus of elasticity in bending ($E_{m,l}$) and global modulus of elasticity in bending ($E_{m,g}$) in accordance with BS EN 408 (2010). The DLT panels were symmetrically loaded in their flatwise orientation by two equally separated line loads acting across the entire width of the sample up until failure. The geometric arrangement for this test is illustrated in Figure 8-7.

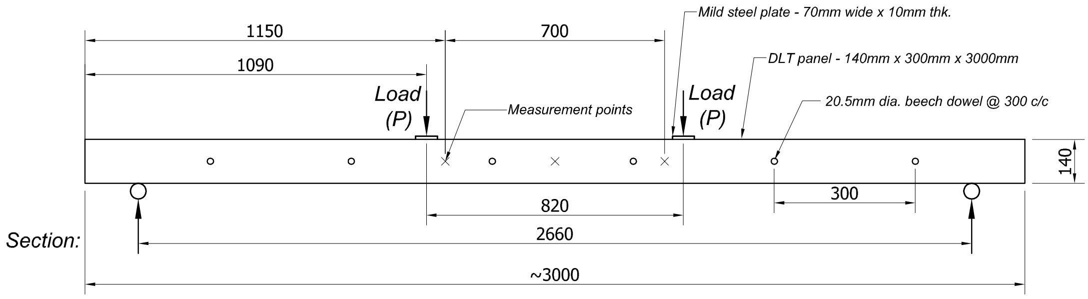

Figure 8-7: Full scale DLT panel four point bending test.

The DLT panel was supported and aligned in exactly the same manner as the three-point bending load, test described in section 8.2.1. No lateral restraint was provided at 1/3 span as the beam was deemed to have a relative slenderness for bending, $\lambda_{rel,m}$ below 0.75 (approximately 0.13) according to BS EN 1995-1-1 (2004). Deviations from the three-point bending arrangement were in the application of load and the positioning of the LVDT's to monitor deformation. Load was applied through line loads situated at two equally spaced points along the length of the beam.

Figure 8-8: Four point bending set-up.

The deformation up until 40% $F_{max,est}$ was recorded to within an accuracy of 1% using 6 LVDTs positioned on either side of the test sample (Figure 8-7). In accordance with BS EN 408 (2010), two LVDT's were located at centre on the underside of the panel and a further 4 were positioned either side of the mid-span at 700mm centres (each 350mm from mid-span). Load was applied at a constant rate of 0.4 mm/s, corresponding to 0.003h mm/s, until the beam reached 40% $F_{max,est}$ at which point the test was paused. During this pause in the displacement the LVDTs placed under the sample were removed (to prevent them from incurring any damage) before the sample was again loaded to 0.4mm/s until failure occurred within 300 seconds. Load head displacement was used to record the ultimate load failure.

8.2.3. Procedure

Prior to conducting the experimental tests, the cross-sectional dimensions of the entire panels were measured with a set of digital callipers accurate to ±0.1mm and the length measured using a tape measure. The dimensions of the sample were taken as the average of three measurements, one taken at the centre of the samples and the others 150mm from each end of the sample.

After the four point bending tests, the test specimen was cut as close to the point of failure as possible. The moisture content and density of the central and outer laminations were determined in accordance with EN 13183-1 (2002). Moreover, 3 dowels from each specimen were removed, the one closest to either point of support and the most centrally located dowel and the values of moisture content and density were found using the applicable harmonised standard, BS EN 13183-1 (2002).

8.2.4. Results

8.2.4.1. Three point bending tests

The three point bending tests were conducted to understand the consistency of the load transfer across a panel. In this instance LVDT's were placed on the outer most laminations in the centre of the span at two location 630mm away from the centre of the panel to align with a line of dowel positions. Ideally more LVDT's should have been placed but the experiment was limited by the number of transducers available and the data processing unit. A compromise was achieved by placing the LVDT's at opposite corners to understand how the transferral of load was conducted by the panel and to verify if uneven deflection was occurring across the panel, see Figure 8-9 for placement of LVDT's.

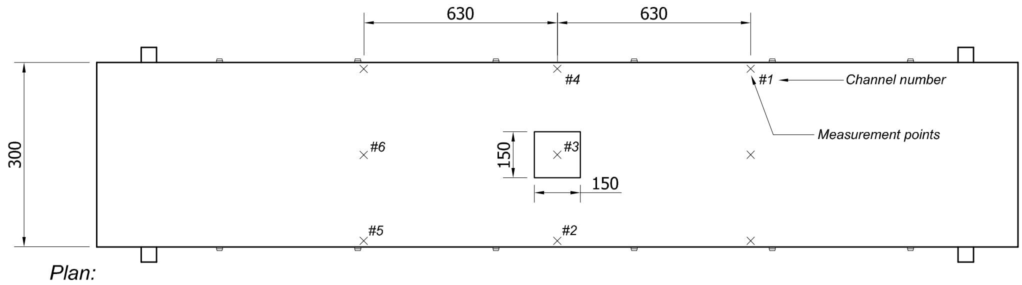

Figure 8-9: LVDT location on three-point load test.

The apparent modulus of elasticity, $E_{m,app}$ was calculated in this arrangement using the method given in BS EN 408 (2003), furthermore the shear modulus was derived from the single span method given in BS EN 408 (2010), utilising the local modulus of elasticity in bending $E_{m,l}$ derived from the tests in the following section 8.2.4.2.

Sample number	Apparent MOE, $E_{m,app}$ (kN/mm²)	Shear Modulus, G (kN/mm²)
Panel 1	8.16	0.192
Panel 2	8.95	0.225
Panel 3	6.90	0.085
Panel 4	7.31	0.129
Panel 5	8.45	0.210
Panel 6	7.90	0.098
Panel 7	7.32	0.068
Mean	7.85	0.143
COV %	6.70	35.55

The derivation of the shear modulus using the single span method given in BS EN 408 (2003) showed that there is a ratio between the elastic bending modulus ($E_{m,l}$) and the shear modulus (G), of approximately 73. This value represents a far higher ratio than is purported to exist between the two mechanical properties as given in BS EN 384 (2010). The presence of such a high ratio between the two properties is not unheard of, Ravenshorst, de Vries and van de Kuilen (2014) found that 31 samples of Austrian Spruce (Picae

Abies) had a mean static G equal to 190N/mm^2 and a ratio of 69.4 between the $E_{m,l}$ and G that was later additionally verified using dynamic torsional stiffness methods.

8.2.4.2. Four point bending tests

The measured values of local ($E_{m,l}$) and global ($E_{m,g}$) modulus of elasticity in bending and average density across the panel were corrected to 12% moisture content and the bending strength parallel to the grain ($f_{m.k}$) was adjusted to the 150mm reference size using the k_h factor as stipulated in EN 384 (2010). Mean values were calculated from the four-point bending tests and are summarised in Table 8-1. Alongside these calculated values the load deformation graphs of all the specimens up until failure are shown in Figure 8-10

Sample number	Ultimate load, F_{max} (kN)	Global MOE, $E_{m,g}$ (kN/mm²)	Local MOE, $E_{m,l}$ (N/mm²)	Bending strength, f_m (N/mm²)
Panel 1	85.34	10.39	9.68	35.98
Panel 2	86.17	10.52	10.83	36.28
Panel 3	73.10	8.58	10.08	29.73
Panel 4	81.10	8.52	9.25	34.30
Panel 5	98.94	10.19	9.93	41.43
Panel 6	59.15	9.47	11.45	23.90
Panel 7	79.40	10.36	12.16	33.58
Mean	80.46	9.72	10.48	33.60
COV %	15.28	8.90	9.92	16.47

Table 8-1: Mechanical properties of panel derived from four-point bending tests.

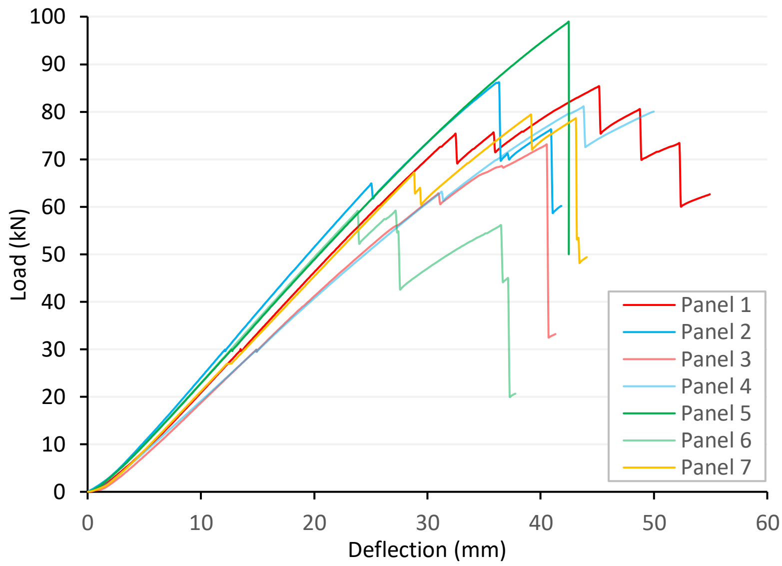

Figure 8-10: Load deformation plot of UK larch DLT panels under four-point bending.

All specimens exhibited a linear load slip response immediately following an initial non-linear response, indicating a period of settlement in the panel before full load resistance was achieved. Proportionality of the load deformation curve was maintained up until an abrupt reduction of strength was experienced, indicated by several steps occurring in the load deformation curves.

Often the steps recorded in the load slip response mirrored the amount of lamination failures, so a correlation between strength loss and lamination failure can be made. Table 8-2 below, presents the failure characteristics of each DLT panel that was tested. Each lamination within the panel was investigated individually and its failure location was recorded, see Figure 8-11. The presence of an asterisk indicates that the failure was caused by a local defect in the timber such as a knot.

Figure 8-11: Measuring failure positions in laminations

Sample	Failure location in individual laminations from LHS (mm)							
	L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8
Panel 1	1030	1470*	N/A	1690	1425*	N/A	1400	N/A
Panel 2	1160	N/A	1170*	850	N/A	1470	1510*	1710*
Panel 3	N/A	900*	N/A	1150	960*	N/A	390	1280*
Panel 4	N/A	N/A	N/A	1140*	1470*	1290*	1170*	N/A
Panel 5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1758
Panel 6	N/A	980*	770	1680*	1275*	2425	1170	1238
Panel 7	900	N/A	1810*	1400*	1760*	N/A	1640*	1040*
* Denotes failure at a local defect, such as a knot.								

Table 8-2: Location of failures in the lamination.

The irregularity of the load deformation curves above the proportional limit suggests that some laminations resist higher proportions of load due to their high levels of stiffness. The step response of the load deformation curve is indicative of the sequential failure of the laminations of the panel, with the failure in each lamination occurring separately. In most instances there was a slight recovery of the strength of the panel before another failure occurred in the panel. This pattern followed until the test was stopped or when a reduction of 20% of the maximum load occurred. The stepped load response highlights the redundancy in the panel but also illustrates the lack of composite action occurring at high levels of load. The sequential failure of the laminations occurred mostly due to tension failure in the lower edge of the panel, initiated at locations with noticeable macro defects such as knots or areas where large slope of grain were apparent.

8.2.5. Comments

A comparison between the characteristic bending strength of a DLT panel comprising mainly C22 timber achieved a characteristic bending strength similar to that of Glulam Grade GL26h to BS EN 1194 (1999) timber or C27 timber as given in BS EN 338 (2009). This indicates that the system factor discussed in 4.1.4.1 can be included for DLT panels based on the assumption that the laminations are aligned closely together. Any increases in system strength beyond the 10% stipulated in BS EN 1995-1-1 (2004) due to the method of conjoining the panels cannot be guaranteed without further testing of many more samples. Considering the characteristic and mean bending strength for UK larch are 21.55N/mm^2 and 33.6N/mm^2 (as shown in Table 6-2) respectively, the increase in values is not outside the realms of possibility and a significant increase in strength would be expected when using multiple sections together in close proximity.

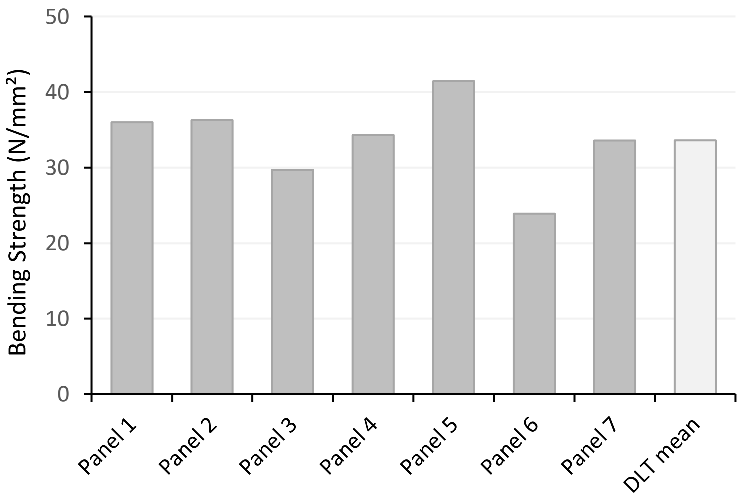

Figure 8-12: Comparison between bending strength of DLT.

Often the determining design consideration in a timber floor deck (particularly over larger spans) is the SLS criteria. Of interest in this study is the expected deflection of a DLT panel, determined by its Modulus of Elasticity. The calculated mean value for the panel was 9.72kN/mm^2, which is equivalent to a C20 material and far lower than the values of the Elastic Modulus of EWPs with a comparable bending strength, Figure 8-13.

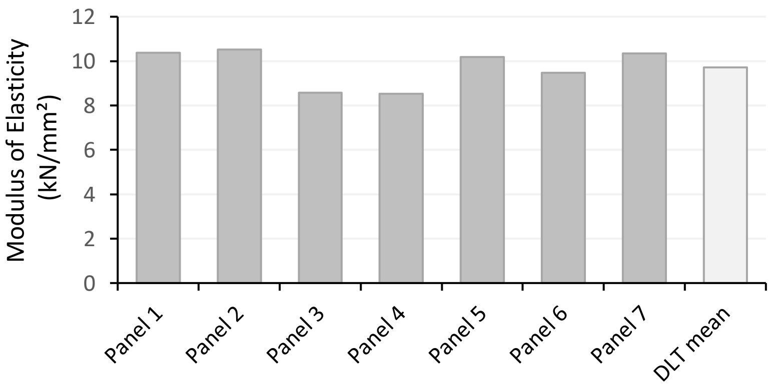

Figure 8-13: Comparison between stiffness of DLT and similar products.

It is clear the formation of the panel did not increase the overall stiffness of the constituent timber members. The mean Modulus of Elasticity of the sample set when acoustically graded prior to manufacture was 9.70kN/mm^2 and after manufacture the Modulus of Elasticity of the tested panels was 9.72kN/mm^2. Whilst it stands to reason that the stiffness of the panel would be representative of the population it was taken from. The arrangement of the panel did not provide any reinforcement of defects that would be provided in a panel joined rigidly together using adhesives. This creates a problem when the material that may be provided for production into panels may contain significant amounts of defects.

The variations in the deflection across the centre of the panel in the three-point tests indicated that full composite action of the panel was not enacted. The panel did not act uniformly at low or high level of loads. Greater separation between the deflection in the centre and those across the sample were noticeable at upper levels of load. Figure 8-14 shows the load deflection response of the panel at various locations across the cross section and along the length as depicted in Figure 8-9. The centrally located LVDT is number #3 and is shown dashed, the LVDT's either side of the panel are #1, #2, #4 and #5

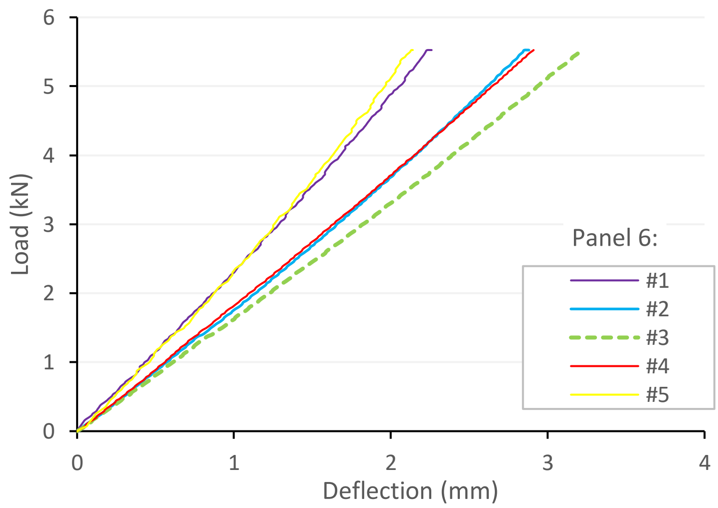

Figure 8-14: Load deflection curve at differing points of a DLT panel in the elastic range.

There was an expectation that due to the connection stiffness reducing at higher levels of load because of the non-linear response and the yielding of dowels that a higher deviation in deflection would be expected between the central transducer and the outer transducers. However noticeable separation was also occurring at low levels of loads. The result of transverse prestressing caused by the insertion of the dowels, creates a friction force between the laminations. In DLT panels the pre-stressing force needs to generate a compressive strength between the laminations that is greater than the transverse tension force induced by transverse bending of the panel, otherwise separation of the laminations could occur, in a manner that is witnessed in stress laminated timber decks (Ekholm, Kliger & Crocetti, 2012; Ekholm, Crocetti & Kliger, 2013). Through these tests the probability of inter-laminar slip occurring under in-service loads was not generally seen and the behaviour of the panel shifted from linear to non-linear behaviour at higher loads, when lamination failure occurred.

In the three-point bending test the maximum shear in the sample occurred at the centre of the sample directly under the point of load. Decreased dowel spacing at locations of high shear could be utilised to increase composite action at higher levels of shear force. However, the positioning of the dowels through the neutral axis and their relative stiffness to the timber means the spacing of the dowels could be required at very close centre and could fail the spacing criterion given in Table 5-1. . It is acknowledged however that the standard operating parameters identified in 2.1.2.2 such as domestic construction that the locations of high shear forces will be limited.

On the continent the placement of a concrete screed placed on the top face of the panel, may not only provide better acoustic properties and improved vibration characteristics but may enable increased composite performance between the laminations of the timber deck. Often the concrete screed is keyed into the timber deck by means of mechanical fixings or recessed or channelled areas within the deck to promote further composite action not only between the timber but between the concrete and timber deck.

8.2.6. Brittle failure

Brittle splitting failure perpendicular to the grain often occurred in the outer laminations, emanating from the locations where the dowels were inserted into the panel, see Figure 8-15.

Figure 8-15: Perpendicular to the grain splitting occurring in the centre of the sample.

The propensity for the splitting failure to occur in these outer laminations and close to points of supports occurred in two instances and in both occasions only occurred in the laminations on one of the outside edges.

Figure 8-16: Examples of splitting perpendicular to the grain.

The uniform application of load and the width of the overall sample could have played a key role in the propensity for the panel to fail in a brittle mode. The dowel laminated panels created here for reasons already detailed in section 8.1 were 300mm wide and more akin to beam sections. In the three point bending tests, a curvature of the panel is expected, causing the centre of the dowel to deflect at a greater ratio than at the edges of the panel. This causes greater embedment stresses perpendicular to the grain to occur across the width of the outer laminations. This in turn creates larger stress concentrations acting perpendicular to the grain increasing the propensity for brittle failure to occur in the outer laminations.

Where bending failure occurred it often began from the extreme fibres of the specimens but followed a path through the line of the dowels suggesting a line of weakness in the timber being induced by the insertion of the dowel in the panels - Figure 8-15. The mechanism of failure could be linked to the method of dowel insertion into the timber. Similar to driven nails, the strength and stiffness of the timber is reduced in the immediate vicinity of the hole due to the additional stresses induced by the crushing of the timber fibres as the dowel is driven into the panel.

The construction of the panels may have adversely affected the results. In the connection tests undertaken previously brittle failure perpendicular to the grain did occur in several instances. However, in the production of the panel the edge distances of the dowels were not as meticulously observed as they had been during the construction and formation of the connection tests, see Figure 8-17. This could have led to a higher propensity for spitting perpendicular to the grain than was expected from the previous connection tests. Additionally, no specification was given for the grain direction of the dowels prior to their insertion in the panel. In allowing this variation to occur a more typical representation of the expected panel construction could be achieved and analysed.

Figure 8-17: Non-alignment of dowel insertion during production.

8.3. Effective stiffness calculation

Where a composite panel is built up of many sections joined by some form of mechanical fastener between each member or layer, when the overall section is flexed there will be a slip. Following the discussion in 4.2, several methods of calculating an effective stiffness of the panel were presented, mechanically jointed beam theory, the K Method and the shear analogy methods. Of the three methods indicated, only two were deemed potentially suitable for the analysis of a DLT panel, the mechanically jointed beam and the shear analogy method.

8.3.1. Mechanically jointed beam method

The mechanically jointed beam theory allows for the direct substitution of the connection stiffness in the form of a fastener efficiency rating that is calculated depending on the profile of the cross section. At present as the fixing in a DLT panel is through the centroid of a section it will not provide additional stiffness to the panel. However, when the beam bends the transverse cross section does not bend to the assumed curvature so there is an additional stiffness provided by the fastener. By using the fundamentals of the testing standard BS EN 26891 (1991) the maximum deformation that a connection is allowed before the test is terminated is 15mm and by the previous timber standard BS 5268-2 (2002) the deflection of a floor was limited to 14mm to maintain vibration serviceability. Therefore, it would be prudent to say that when the slip of the connection has reached 14mm, the floor is at the limit of serviceability or the connection has yielded excessively. Then the maximum distance that the fictional element is inserted into the effective section model (Figure 4-8) can be assumed to be 14mm away from the centroid of the section. The effective stiffness of the section is then calculated on this basis. The formation of the connection efficiency factor is based on the instantaneous mean stiffness property of the connection that does not incorporate quasi permanent variables for loading or modification factors based on the duration of load or the service class of the timber being tested. The calculation of the effective bending stiffness of a panel based on the geometric properties of the DLT panel as tested in 8.2 is shown in Table 8-3. The value of foundation modulus is derived from tests conducted in chapter 7, and the Modulus of elasticity is based from the acoustic grading undertaken previously on the sample set.

Property	Element			
	Panel		Fictitious Element	
Area, mm^2	A_2	42000	A_1	330
Modulus of elasticity, N/mm^2	E_2	9570	E_1	12600
Second moment of Area, x 10^6 mm^4	I_2	68.6		
Distance from Neutral Axis, mm			a_1	14
Fastener Efficiency			γ_1	0.841
Stiffness of connection, N/mm			k_i	4616
Spacing of connections, mm			s_1	300
Length of panel, mm			l	2660
Effective bending stiffness, $(EI)_{eff}$, N/mm^2			$(EI)_{ef,y}$	657.19 x10^9
Bending stiffness, EI, N/mm^2			EI	656.50 x10^9
Percentage difference				0.105 %

Table 8-3: Calculated properties of DLT panel using mechanically jointed beam method.

In Table 8-3 the bending stiffness, EI of the panel found empirically from the tests conducted in Section 8.2 is shown immediately beneath the effective bending stiffness, $EI_{, eff}$. Here the percentage difference between the empirical results and the derived effective stiffness is 0.105%. The small difference between actual and theoretical bending stiffness makes the mechanically jointed beam method a potential analysis option for reliably determining the overall stiffness of the panel.

8.3.2. Shear analogy Method

The shear analogy method derives the effective stiffness from two fictional elements, the second of which, beam B, characterises the translational stiffness of the panel and the flexibility of the connections. Consider a DLT panel comprising three layers as shown in Figure 8-18 .

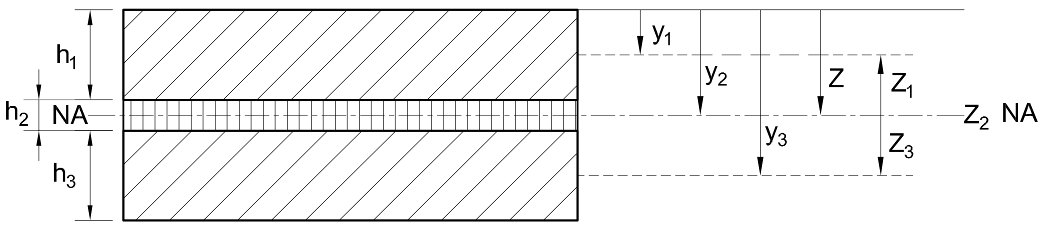

Figure 8-18: Shear Analogy layers

The middle continuous cross layer comprises the hardwood dowel and the outer layers of the softwood material. The calculation and summation of Beam A and Beam B in accordance with the shear analogy principle can be carried out and an effective stiffness calculated, see Table 8-4.

Calculation of B_A			Calculation of B_B		
$E_1I_{a,1}$	51.03	N/mm^2	$E_1I_{b,1}$	276.19	N/mm^2
$E_2I_{a,2}$	2.71	N/mm^2	$E_2I_{b,2}$	0.00	N/mm^2
$E_3I_{a,3}$	51.03	N/mm^2	$E_3,I_{b,3}$	276.19	N/mm^2
B_A	104.78	N/mm^2	B_B	552.37	N/mm^2
Stiffness					
Effective bending stiffness, $(EI)_{eff}$			657.15	x10^9 N/mm^2	
Bending stiffness, EI,			656.50	x10^9 N/mm^2	
Percentage difference				0.099%	

Table 8-4: Effective bending stiffness using shear analogy method

The bending stiffness (EI), of the panel found empirically from the tests is compared to the effective bending stiffness ($EI_{,eff}$), derived from the shear analogy method and is shown in Table 8-4. In a similar manner to the mechanically jointed beam method, the variation between the theoretical value and the experimental results is very minimal (0.099%) and could also potentially be used as a viable method of analysis.

A fundamental aspect of this derivation is the inclusion of the shear stiffness of the layers and the stiffness of the flexible connection. These values are dependent on an accurate depiction of the shear modulus of the layer being considered. During this study no definite appreciation of the shear modulus of the dowel was found due to difficulties in mirroring the exact conditions of the dowel in a connection. For ease of replication the generally accepted ratio between elastic modulus and shear modulus of 1/16 was used to determine the effective shear stiffness of the beam. The result of which is shown in Table 8-5.

Calculation of S_B			
a	80.25		mm
(GA_1)	166.49	x10^{-6}	N
(GA_2)	43.39	x10^{-6}	N
(GA_3)	166.49	x10^{-6}	N
	376.37	x10^{-6}	N
Effective Shear Stiffness			
S_B	17.11	x10^{6}	N

Table 8-5: Effective shear stiffness calculation using shear analogy method

8.4. Comments:

The creation of DLT panels with large amounts of knots or other defects is not advisable. Often the presence of defects hinders the installation of the dowels through the panel. Furthermore, the installation of oversized dowels creates inherent weaknesses in the panel in the immediate vicinity of the dowel. Whilst the prestressing induced by the oversized dowels aids panel cohesion at the point of fabrication, it reduces the embedment strength at their location and can cause brittle failure to be induced at relatively low levels of load.

There is an expectation that higher levels of strength and stiffness will be achieved for a panel product when compared to the mechanical properties of an individual member. The formation of a DLT panel allows for a great benefit in the strength but unfortunately does not benefit the stiffness of the system. Often in flooring the stiffness of the deck is the main design consideration and the use of DLT flooring when it is spanning in one direction does not increase the stiffness of the parent material. Further investigation will be necessary, to see if the fixing of a floor deck (such as fibreboard) at regular centres provides additional stiffness or the support of the panel on all four sides allows for a plate structure to be created as seen in stress laminated decks.

If the stiffness and the position of the dowel connection between laminations does not impact the overall performance of the panel by any noticeable margin.

Then the inclusion of timber dowels within the panel will need to be justified for reasons other than an increased strength and stiffness.

Derivation of the stiffness from the two methods employed, mechanical jointed beam theory and the shear analogy method, confirm the experimental test findings. The influence of the connections between the lamination are very minimal. This is because the stiffness of the formed connection is low and also because the location of the dowel is through the neutral axis of the cross section. The DLT panel formed does not perform as a plate material and should not be treated as an orthotropic plate material but rather considered as a series of one-way spanning beams and therefore simplified design methods as shown in section 4.1.3 should be utilised.

The strength to weight ratio of the panel creates issues surrounding the serviceability criteria, vibration and deflection. Vibration analysis and testing of DLT panels (as discussed in 4.1.7) has been omitted from the study as it was felt that the assumptions pertaining to vibration calculations relevant for floor construction are still valid for a DLT panel. However, the benefits of improved stiffness (if any) of a DLT panel with respect to the serviceability performance have yet to be quantified.

9. Case study

This chapter presents a description of stage three of the developmental framework outlined in 2.6; a pilot project. COCIS alongside MAKAR Construction Ltd undertook a pilot project near Fortrose, Northern Scotland that incorporated a suspended DLT floor panel produced using home grown UK larch within its construction. During the experimental processes conducted throughout this thesis large amounts of pertinent data were collected that related to the production and performance of the DLT panel.

9.1. Pilot project

During the preliminary discussions and initial analysis for the possible implementation of a homegrown DLT, it became apparent that the inclusion of pilot projects using DLT would be needed to overcome technical as well as cultural barriers. It was thought the provision of a pilot project to act as a benchmark for information would;

- Demonstrate the benefits of producing a solid timber panel from locally sourced timber and create information that could be used for future certification approvals.
- Implement a quality assured fabrication process with acoustic grading technology to ensure it was of the standard necessary for satisfying the engineering specification.
- Allow the design to be structurally certified using available knowledge from test work undertaken.

The building selected for the trial product was a small private domestic dwelling in Northern Scotland, consisting of three bedrooms, two washrooms and an open plan living area comprising approximately two-thirds of the ground floor area. The open living area consisted of a double height space at the western end with the above intermediate floor deck terminating two thirds along the length of the building. Set in a remote part of Northern Scotland, the proposed building was to be entirely clad in vertical larch timber with internally exposed timber elements making it an ideal candidate for the use of an intermediate DLT floor deck. Figure 9-1 shows the near finished building in the summer of 2015.

Figure 9-1: Finished building

By selecting DLT as an intermediate suspended floor deck some of the initial design complications could be neglected. The provision of an intermediate floor deck provided an opportunity for a fairly straightforward loading arrangement and a stable service class 1 environment.

9.1.1. Manufacture and tolerance

The installed DLT suspended floor covered an area of approximately 20 m^2. The DLT floor panels were set to have a clear span of 4.2m and were manufactured using 30mm laminations to form an individual panel with the dimensions of 140mm deep by 300mm wide and were joined together with 20mm diameter Beech dowels at 300mm centers. Whilst at the time of construction of the new build the analysis of a DLT panel was not formalized the arrangement was based upon preliminary findings of the research. The depth and thickness of laminations was based on the observations from the connection and full-scale tests conducted in this thesis.

The panels were constructed using exactly the same process and material that was used to create the DLT panels tested in Chapter 8. A rough approximation of the expected loads on the floor panel, using the requirements for loading at the ULS given in BS EN 1990 (2002) and the dead and imposed loading given in the relevant parts of BS EN 1991-1-1 (2002), indicated that the expected design moment in the center of the span at the ULS for a 300mm wide DLT beam in isolation over the design span is approximately 2.70kNm. This would induce a maximum bending stress in the DLT beam at the ULS of 2.71N/mm^2, a value far below the characteristic bending strength given in Chapter 10 as 21.55N/mm^2 and the expected medium term duration design bending strength of 13.26N/mm^2 (assuming the factors k_{sys} and k_h were implicitly included in the experiment and analysis process) in accordance with BS EN 1995-1-1 (2004) (see section 4.1.4). The expected instantaneous deflection due to bending and shear of the DLT panel at the SLS was calculated to be 4.87mm from the values of the characteristic elastic modulus taken from the experimental tests conducted in chapter 8, a value well within the permissible boundary of instantaneous deflection given in Table 7.2, BS EN 1995-1-1 (2004) as 14.21mm. The final deflection due to the permanent and quasi-permanent actions calculated in accordance with BS EN 1995-1-1 was shown to be 6.64mm, which is well within the permissible bounds of 28.42mm provided by the expression given in Table NA.5 of the NA to BS EN 1995-1-1 (2004) for net final deflection.

To avoid excessive shrinkage or warping the panels were manufactured with laminations at a moisture content as close as possible to the expected in-service moisture equilibrium. Storage of the product after manufacture, during delivery and installation of the panels was conducted with the utmost care to ensure no excessive uptake of moisture was allowed. The panels were brought on site immediately prior to installation and any panels left on site unattended were stored in such a manner that no additional moisture could affect the panel adversely.

Figure 9-2: Installation of panels.

Prior to inhabitation the moisture content of the panels were measured at 9 different locations on the underside of the floor using a handheld moisture content reader (see Figure 9-3) The average moisture content readings were taken to be 15%, indicating that the panels were carefully handled prior to installation and would not suffer from undue distortion occurring due to drying.

Figure 9-3: Handheld moisture content reading of DLT panel in-situ.

9.1.2. Details and connections

In the internal environment the moisture content of the timber panels will fluctuate throughout the year and the dimensions of the panel will subtly alter seasonally in response to the changing moisture content and temperature of the panel throughout the year. Due to the solid configuration of the panel the swelling across the grain of the individual laminations will be exacerbated by their confinement in the panel formation. The joining together of the individual panels and their support therefore had to be given careful consideration, otherwise stresses could be applied internally within the panel or at the point of fixings that could hamper the correct operation of the panel during its lifespan.

From the outset, the building envelope was envisaged to be formed using balloon framing techniques where the external walls were provided as full height prefabricated continuous members that provided high levels of insulation, airtightness and speed of construction. The primary decision on the method of timber frame construction simplified the support arrangement of the DLT panel. By necessity the panel would be supported in direct bearing from a ledger that was face fixed back to the balloon frame. A timber dwang was inserted into the prefabricated panel at the correct height and location to allow fixing of ledger to the prefabricated timber panel using metallic dowel type fixings (see Figure 9-4).

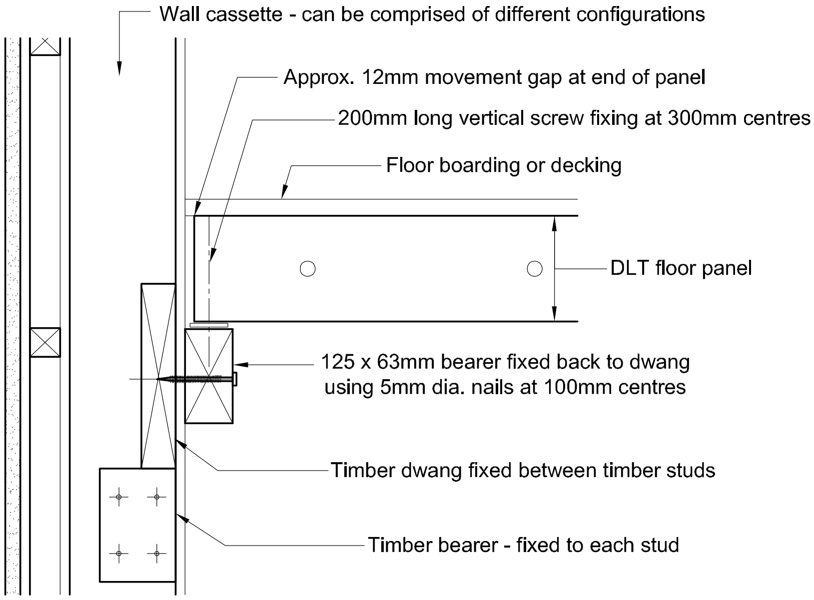

Figure 9-4: DLT floor panel to wall panel, example detail.

By providing a ledger plate some tolerance in the placement of the panel could be incorporated and any stresses induced by the swelling of the panel longitudinal could be alleviated by allowing a small tolerance at either end to allowing for swelling to occur without inducing any further stresses in the surrounding timber frame. The suspended floor was not fully restrained around all four edges of the panel and was provided with a free edge to allow any expansion across the grain to occur on the unrestrained edge, the arrangement is shown in Figure 9-5.

Figure 9-5: Edge fixings around DLT panel.

One of the by-products of creating smaller panel widths was the reduced weight of the panel. The average weight of the prefabricated DLT panels were approximately 80kg in total and required at least four people to move into position unless mechanical lifting aids were used. For larger panels the weight of an individual panels could be in excess of 100kg and it would be expected lifting points would need to be incorporated into the panel to ease handling of the panels

The connections required between the panel becomes less of a structural issue to allow the load share and achieve enhanced composite action but becomes more of a serviceability issue to ensure that the panels are aligned correctly with no gaps between the members. For this reason, connections between individual DLT floor panels were made with self-drilling screws installed in pairs at an angle of 30-45° to draw the panels together in pairs. The panels themselves weighed approximately 80kg, so the process of tying the panels together was not a trivial matter and can only be achieved if some mechanical action was included in the installation.

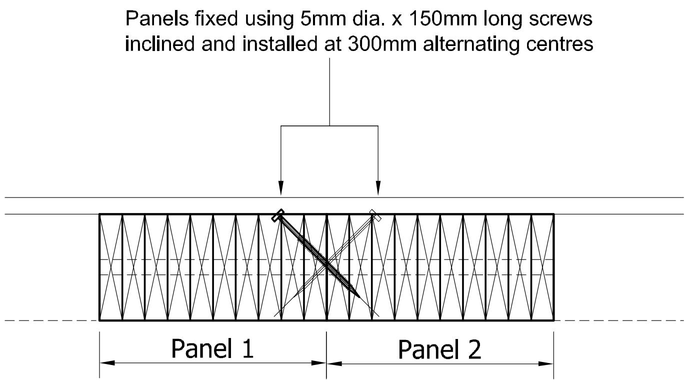

Figure 9-6: Example of mechanical joint between DLT panel sections.

9.1.3. Finished product

The successful adoption of UK home grown larch was achieved in a domestic new build in Northern Scotland. Whilst there were challenges regarding successful drying of the material to the required moisture content as well as creation of larger sized panels the inclusion of UK home grown DLT panel is an innovative solution in the building and shows the clear potential for future use.

Figure 9-7: Intermediate DLT floor deck 1.

Figure 9-8: Intermediate DLT floor deck 2.

10. Conclusions and Future Work

The main aim of this thesis was to study the suitability of British home-grown timber in the fabrication of a DLT panel and develop an understanding of the performance of the panel. The aim has been satisfied but requires further research if the use of home grown DLT panels is to increase. This thesis provides an in-depth introduction into the suitability of the home-grown material and the research and design of a DLT panel. The study aimed to provide a foundation for future work to be undertaken in this area. The potential for the system was shown through a case study conducted on a domestic new build in northern Scotland. The main conclusions from this study are presented below, alongside recommendations for further research.

Whilst there is an increasing supply of home-grown timber, adopting this material into value added processes has been limited. Previous research completed by COCIS studied the use of home-grown timber in solid engineered products. These studies and the wider literature review conducted in Chapter 2 emphasised the lack of knowledge of using home grown timber products in solid engineered products and in DLT in particular. Building on this research, investigations were conducted into the material properties of the timber, and a DLT panel configuration was defined, developed and investigated.

Often the properties of timber ensure that the design of a timber structure according to the relevant standards is more onerous when considering serviceability limit states than the ultimate limit states and, in particular, stiffness. Therefore, the focus of this work has been on the experimental investigation of the all-timber connections within a panel under load and how they affect the stiffness of the formed panel.

10.1.1. UK grown softwood:

DLT is a product that can be developed within the UK using the local resource. Currently the softwood supply chain can supply the necessary sizes of timber required for the production of DLT but will have to alter their drying practices to provide the timber at the moisture content required for DLT production or other solid EWPs.

Where the timber is to be used in the production of a solid EWP, further processing will be required and some of the distortions could be mitigated, provided that they are within the limits for the processing and regularising equipment. A large amount of twist that was witnessed in the samples could be mitigated by providing higher levels of restraint, selecting timber with a lower propensity for twist and finally specifying section sizes that could be later processed into the necessary regularised sections. Further research will be necessary to understand how much and what type of distortion is expected, where it can be reduced and lastly how it can be mitigated through further processing of the material.

10.1.2. UK grown hardwood:

Whilst the hardwood resource within the UK could easily provide the necessary volumes for initial and increased DLT production, it is hampered by the small amounts of hardwood that is currently being felled and earmarked for value added processes. An evaluation of the dimensional stability of dowels was undertaken on four selected hardwood species. Each species tested exhibited differing levels of shrinkage and expansion based on the orientation to the grain. How the swelling of the dowels affects the suitability for its use within a DLT panel is currently unknown. The use of beech in mainland Europe can be seen as the pragmatic harvesting of a hardwood that is locally available in the areas that currently produce DLT. A small selection of other hardwood dowels commonly available in the UK has indicated that the level of expansion witnessed is comparable to that experienced from beech and should be given due consideration for inclusion in a DLT panel.

Following observations from all-timber connections, an amount of plasticity in the dowels was witnessed in-situ in a connection, but due to the anisotropic nature of timber this plasticity could not be reproduced and indeed was proven not to be representative when determining the yield moment of a dowel from a standard three-point dowel test. The effective yield moment or plastic load resistance of a series of hardwood dowels was calculated from an adapted three point bending test. The results show a non-linear increase in dowel strength after an initial failure or slip and the effective yield moment was shown to be a combination of flexural strength, embedment and confinement. However, in the test arrangement large amounts of localised crushing and embedment were seen at the loading arm and at points of support. Furthermore, the width of the loading arm promoted a single central hinge to occur that was not fully restrained on its tension edge promoting brash brittle failure of the timber dowel to occur.

10.1.3. Material selection:

Embedment tests were conducted on both UK Sitka spruce and larch to quantify their embedment strength and stiffness in both the parallel and perpendicular direction. The results indicate that both species followed the acceptable load deformation characteristics for parallel and perpendicular embedment. The value for perpendicular embedment was far lower than its parallel to grain counterpart and a figure well below the estimated embedment strength given by the expression in BS EN 1995-1-1 (2004). Other research conducted (Franke and Magnière, 2014b) corroborates the underestimation of embedment strength. However, the extent of the reduction witnessed in this study was exacerbated by the initial specimen sizing.

A series of combined embedment tests were then conducted on a variety of types of hardwood dowels set in Sitka spruce and larch firstly, to understand their combined behaviour and secondly, to ascertain their strength and stiffness. Tests were conducted in both perpendicular and parallel to the grain using a modified embedment test setup. The results found that both the strength and the stiffness of the combined arrangement were affected by the comparative density between the samples and could create a reduction in the strength and stiffness of between 8-60% depending on the orientation to the grain and the dowel species used.

10.1.4. Connection testing

Timber to timber connection tests were conducted in both the parallel and perpendicular grain orientations based on the methodology reported in the harmonised standards BS EN 26891 (1991) and BS EN 1380 (2009) to investigate the behaviour under load and the type of failure that occurred. Double shear plane connection tests conducted on UK larch and varying dowel species indicated that the performance of beech surpassed the alternative hardwood dowels tested. Unlike other dowel species the beech dowel response was plastic at lower ratios of dowel diameter (D) to lamination thickness (t), making the derivation of a yield strength more applicable to this connection arrangement.

Further investigation of the connection behaviour of an all-timber connection constructed of both UK grown Sitka spruce and beech included a sensitivity analysis of two varying parameters, dowel diameter and lamination thickness. Failure modes depicted in previous studies conducted on all-timber connection studies were shown to occur with beech dowels situated in UK grown Sitka spruce and UK larch. In every all-timber parallel to the grain specimen, final failure always occurred in a brittle manner within the dowels either through cross grain shear failure (a failure mode specific to timber dowels), or in tensile bending failure in the fastener themselves.

Ash, beech, oak and sycamore dowels were experimentally investigated for their suitability to create connections within UK grown softwood members. Significant post yield ductility can be achieved by ash, beech and oak dowels when loaded parallel to the grain, provided that the thickness of the central member is above 1.75 times the dowel diameter. Brittle failure can occur at high levels of yield as a result of brash shear failure caused by yielding of the dowel in both the tension and compression edge. Sycamore dowels are not recommended for their structural application due to the common brittle mode of failure. Brittle failure can occur as a result of perpendicular to the grain splitting failure of the outer timber members. These modes can be mitigated through appropriate measures.

10.1.5. Design and Fabrication

The fabrication of DLT panels provides a scalable process that can be created as a fully automated process or simply manufactured by hand. At the lower end of the technological scale lengths of the panels are limited by natural lengths of timber and the width is limited by the ability to remove spoil from the drilling process and press the dowel through the laminations. The inclusion of pre-driven dowels into smaller depth pieces of timber creates issues with the splitting of timber at relatively low levels of load. The depth of the panel or the diameter of the dowel will need to be considered holistically to limit splitting occurring.

10.1.6. DLT Performance

The strength performance of DLT panels is greater than expected from a single lamination, however they do not experience a proportional increase in stiffness when spanning in one direction. This indicates that the location of the dowels and their stiffness do not provide enhanced properties and DLT cannot be treated as a composite panel without further supplemental fixing being used.

10.2. Further work:

This thesis presents the strategy to integrate DLT panel production using UK grown natural materials into the UK construction industry. Information from the manufacture and the product testing stages should be used to refine the cost model and provide overall product characteristics. Panels produced have been shown to have high strength capacity but low stiffness when spanning in one direction. Further research is required to understand the stiffness of the panels using different support conditions including two way spanning situations. Additional research will be required on the location of the dowels in the cross section, the spacing of the dowels and the provision of floor sheathing on the deck.

Research into the effect of variable actions and long-term effects of creep in combinations of both the laminations and the dowels together should be undertaken. It is alluded to in 4.1.7 that the vibration characteristics of the panels themselves should be straightforward to quantify according to the principles stipulated in BS EN 1995-1-1 (2004). However, the low stiffness values recorded for DLT panels reported in 8.2 indicate that the vibration response of a DLT panel should be investigated further.

Further research is necessary on the individual mechanisms that form the panel. In particular a consensus is needed on the method of determining embedment strength and stiffness of the laminations individually and the combined embedment stiffness that can occur. The derivation of semi-empirical expressions that define both the Elastic Modulus and Shear Modulus of the hardwood dowels over low spans is necessary to facilitate easier calculation of the strength and stiffness of an all-timber connection for general engineering purposes.

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Appendix A: Market questionnaire

Understanding the feasibility of UK sourced and manufactured engineered solid wood products

Company Overview:

1. What type of company/business are you? (Please select appropriate box)

- Property Developer
- Builder – Main Contractor
- Builder – Specialist Sub Contractor
- Builder – Small Builder
- Architect
- Civil/Structural Engineer
- Manufacturer
- Local Authority
- Other (please specify):

2. What is the size of the company? How many employees are there? (Please select appropriate box)

- 1-10
- 11-20
- 21-50
- 51-99
- 100+

3. Where are you based? (Please select appropriate box)

- England

- Wales
- Northern Ireland
- Scotland
- Other (please specify):

Understanding the perceptions towards UK grown timber:

4. What is your perception of UK & Irish grown timber? (Please select appropriate box)

- Excellent
- Good
- Average
- Poor
- Very poor

5. Why is this your perception of UK and Irish sourced timber? Please rate the below factors according to their relative importance to your perception.

	No importance	Little importance	Neither important or unimportant	Important	Great importance
Availability					
Cost					
Appearance					
Workability					
Structural Performance					

Other factor (please specify)

Understanding the awareness of engineered wood products (EWPs):

6. Are you aware of the following engineered timber products? (Please select all that are appropriate)

- Dowel Laminated Timber (**DLT**)
- Laminated Veneer Lumber (**LVL**)
- Glue Laminated Timber (Glulam)
- Thin webbed joist (I-Joists)
- Cross Laminated Timber (**CLT**)

If you have used/specified any of these products, please indicate which ones.

Demonstration of DLT - potential application and market:

7. Shown below is a picture of a **DLT** panel:

More information about the product can be found here.

Where do you see the application for a **DLT** product lies within the UK construction industry? (Please select all that you think are appropriate)

- Flooring
- Roofing
- Walling
- External Cladding/Finish

Any other suggestions:

8. Where do you see the market for a **DLT** product lies within the UK construction industry? (Please select all that you think are appropriate)

- Domestic (Private Client)
- Domestic (Contractor)
- Domestic (Housing association)
- Public
- Commercial
- Industrial
- Other (please specify):

Understanding the barriers and drivers for DLT product implementation:

9. Specifically looking at a **DLT**. Please rate the issues below with respect to their effect on the manufacturing, supplying or specifying homegrown **DLT** products?

	Negligible impact	Small impact	Neutral	Significant impact	Critical impact
Investment required					
Market size					

	Negligible impact	Small impact	Neutral	Significant impact	Critical impact
Building applications					
Certification and guidance					
Lending, insurance and risk issues					
Already available construction products					
Cost of product					

Any other issues (please specify)

10. Specifically looking at **DLT**. How do you perceive the factors below as drivers in specifying or supplying homegrown **DLT** products? (Please rate according to your perceived value)

	Not relevant	Limited relevance	Neutral	Very relevant	Extremely relevant
Offsite construction					
Speed of Construction					
Opportunity for new business					
Legislation					
Low embodied energy					
Acoustic performance					
Thermal properties					
Aesthetic appeal					

Any other issues (please specify)

11. If a homegrown new **DLT** product were available today at a similar price point and performance to other similar products, how likely would you be to use it instead of the competing products currently available? (Please select as appropriate)

- ☐ Very likely
- ☐ Likely
- ☐ Undecided
- ☐ Unlikely
- ☐ Extremely unlikely

Demonstration of CLT - potential application and market:

12. Shown below is a picture of a **CLT** panel:

!

More information about the product can be found here .

Where do you see the application for a **CLT** product lies within the UK construction industry? (Please select all that you think are appropriate)

- ☐ Flooring
- ☐ Roofing
- ☐ Walling

External Cladding/Finish

Any other suggestions:

13. Where do you see the market for a **CLT** product lies within the UK construction industry? (Please select all that you think are appropriate)

- Domestic (Private Client)
- Domestic (Contractor)
- Domestic (Housing association)
- Public
- Commercial
- Industrial
- Other (please specify):

Understanding the barriers and drivers for CLT product implementation:

14. Specifically looking at a **CLT**. Please rate the issues below with respect to their effect on the manufacturing, supplying or specifying homegrown **CLT** products?

	Negligible impact	Small impact	Neutral	Significant impact	Critical impact
Investment required					
Market size					
Building applications					
Certification and guidance					
Lending, insurance and risk issues					

	Negligible impact	Small impact	Neutral	Significant impact	Critical impact
Already available construction products					
Cost of product					

Any other issues (please specify)

15. Specifically looking at **CLT**. How do you perceive the factors below as drivers in specifying or supplying homegrown **CLT** products? (Please rate according to your perceived value)

	Not relevant	Limited relevance	Neutral	Very relevant	Extremely relevant
Offsite construction					
Speed of Construction					
Opportunity for new business					
Legislation					
Low embodied energy					
Acoustic performance					
Thermal properties					
Aesthetic appeal					

Any other issues (please specify)

16. If a homegrown new **CLT** product were available today at a similar price point and performance to other similar products, how likely would you be to

use it instead of the competing products currently available? (Please select as appropriate)

- Very likely
- Likely
- Undecided
- Unlikely
- Extremely unlikely

17. Within the next 5 years do you see **CLT** and **DLT** products becoming relevant to your business?

- Strongly agree
- Agree
- Neither agree or disagree
- Disagree
- Strongly disagree

Appendix B: Interview Format Sheet

Department of Architecture

UK Dowel Laminated Timber: Marketing & Feasibility Study

The questions below are a series of questions relating to DLT production and implementation into the UK construction marketplace. All these questions may or may not be relevant to the interviewee and will be omitting by the interviewer on a case-by-case basis. In this example format all the main topics have been broken down into sections, of which approximately 5 minutes of discussion will be directed on each of these topics.

Section 1: Overview

1. What is your professional occupation?
2. What type of company do you work for?
3. What is the size of this company?
4. Are you aware of solid timber products? If so, which ones?
5. If yes, how did you become aware of solid timber products?

Descriptions and pictures of the various different products to be shown to the user.

6. Do you believe that these products could be relevant to your company in the future?

Looking more closely at DLT, show some of the various products, which are currently in the marketplace from the EU and US (where applicable).

Section 2: Manufacturing & supply

Imagining the scenario that DLT production was to take place in the UK within the next 5 years.

In terms of establishing DLT production in the UK and based on your experience with the construction industry, what do you perceive as the largest challenges in terms of:

1. Raw materials
    a. What issues, if any do you think exist in the available lamella (raw) material?
    b. What issues, if any do you think exist in the available dowel material?
2. Supply chain

a. Do you feel confident that the existing supply chain will be able to provide the necessary material for production?

3. Manufacturing process
4. Knowledge transfer for the success in the above categories
   a. What transfer of knowledge do you think will be required to establish DLT production? (Design? Manufacture? Procurement and assembly? Logistics? Sales?)
   b. Can this knowledge be transferred remotely (through email or distribution of technical manuals) or is an in-house specialist required?
5. Intellectual property
   a. Do you believe there will be any intellectual property issues related to DLT manufacture?
   b. To your knowledge, is there any technological, resource, or intellectual property barrier to the transfer of this dowel laminated timber production to the UK?

Section 3: Specification & technical product information

1. Have you ever been involved in the specification of a DLT product? If yes, could you please give examples and any problems therein?
2. What in terms of knowledge on DLT products do you require the most immediately, to begin specifying the product?
   a. Structural capability (Stiffness and Strength Properties)
   b. Connection detailing
   c. Shrinkage and dimensional stability
   d. Durability
   e. Life cycle analysis (if conducted, i.e. environmental impact)
   f. Ease of modification (e.g. for installation of services, etc.)
   g. Hydro-thermal performance
   h. Acoustic performance
   i. Fire retardation rating
3. In terms of establishing DLT production in the UK and based on your experience with the construction industry, what do you perceive as the largest challenges in terms of:
   a. Uptake and acceptance by the industry
   b. Competition from existing products (Concrete and Steel)
   c. Product promotion
      i. Would you be more likely to specify the product if you aware there is monitoring the field performance of DLT products post installation (for the initial implementation)?
      ii. Instilling the value of the product

4. Design
    a. Many companies in Europe have created their own CLT design software, which is available as a free download from their websites. Do you feel that this something which could be replicated for DLT and provide great benefit?
5. Regulatory acceptance
    a. Do you believe there are any issues with the specification of DLT in terms of lending and warranties to house builders, private clients?
    b. Do you feel warranties from organisations such as NHBC pivotal to the specification of DLT?

Section 4: Dowel Laminated Timber (DLT) market

1. In your experience, what do you believe is the primary market for Dowel laminated timber (DLT) products (e.g. single-family housing, multifamily housing, medical, retail, commercial, single story, etc)?
2. Do you believe there are any secondary markets for DLT, besides which you stated above?
3. Who do you believe will be the primary specifiers and consumers of DLT products? (Architects, consultants, engineers, builders, end-users? Who is asking for your product to be used?)
4. Where do you believe the most effective promotion of DLT should be carried out?
5. What do you believe to be the clients' key motivational drivers and performance indicators for the selection of DLT? (For example; Cost? Sustainability? Speed of construction? Quality assurance?)
6. Do you believe these key drivers are regulatory or end-user driven?
7. Do you foresee (or have recently seen) any regulatory or consumer changes that will increase demand for DLT products?
8. Are you aware of any other next-generation products coming online, which will represent threats to the DLT products?
9. What do you perceive as the primary advantages of competitor products (steel, concrete, lightweight timber) over DLT products?
10. What are the primary disadvantages of these competitor products compared to DLT products?
11. What do you believe to be the unique selling points or added-value aspects of DLT products versus competitor products such as concrete, steel or other engineered timber?
12. Do you believe these unique selling points are clear and have the power to generate interest in the product?
13. In your opinion, what are the detrimental factors or primary barriers to DLT use within the marketplace? Are there factors causing a barrier to

DLT use? Please indicate whether you feel this is a real or simply perceived factor?

14. Do you feel that DLT, would be better served as a composite construction component, i.e. concrete and timber floor decking?
15. Do you feel that DLT should be marketed as a separate product or something that is essential component to delivery?
16. In your opinion, do you believe there is sufficient market (by sector: residential, public, commercial, industrial) for DLT in UK to become establish as a building product?

Appendix C: Larch: Parallel

Double shear plane: Results

LARCH PARALLEL ASH							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L9P(A)	10.90	465.75	8295.16	12818.10	4440.46	5106.20	2762.06
L10P(A)	11.10	448.63	6370.09	10310.00	2863.03	5628.86	2020.71
L11P(A)	10.84	443.11	14127.62	14948.50	3879.97	7448.23	3319.26
L12P(A)	11.36	509.11	7242.13	12608.10	4475.71	4316.34	2697.84
Min	10.84	443.11	6370.09	10310.00	2863.03	4316.34	2020.71
Max	11.36	509.11	14127.62	14948.50	4475.71	7448.23	3319.26
Average	11.05	466.65	9008.75	12671.18	3914.79	5624.91	2699.97
St. Dev	0.23	29.91	3502.17	1896.22	752.41	1329.91	531.91
COV %	2.10	6.41	38.88	14.96	19.22	23.64	19.70

LARCH 30mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L32P(B)	10.96	469.04	12123.26	14710.10	4926.93	5614.01	2075.57
L33P(B)	10.80	483.34	12406.06	14607.10	5757.35	5619.31	2719.93
L34P(B)	11.15	486.47	11545.56	14069.00	4826.99	4744.83	1734.93
L35P(B)	10.99	493.73	11540.44	15973.80	4933.91	4834.18	1889.52
Min	10.80	469.04	11540.44	14069.00	4826.99	4744.83	1734.93
Max	11.15	493.73	12406.06	15973.80	5757.35	5619.31	2719.93
Average	10.97	483.15	11903.83	14840.00	5111.30	5203.08	2104.99
St. Dev	0.14	10.37	432.35	806.44	433.46	478.95	432.97
COV %	1.30	2.15	3.63	5.43	8.48	9.21	20.57

LARCH 30mm PARALLEL BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L28P(B)	11.24	435.80	2828.30	3962.92	2218.21	2184.29	1524.17
L29P(B)	11.06	508.19	2863.65	5280.09	1998.44	1961.07	1447.42
L30P(B)	11.07	547.00	2783.47	5431.06	1761.80	1961.07	1370.95
L31P(B)	11.68	483.27	2094.81	3767.31	1198.14	1728.94	1190.43
Min	11.06	435.80	2094.81	3767.31	1198.14	1728.94	1190.43
Max	11.68	547.00	2863.65	5431.06	2218.21	2184.29	1524.17
Average	11.26	493.57	2642.56	4610.35	1794.15	1958.84	1383.24
St. Dev	0.29	46.59	366.63	866.41	438.88	185.91	142.95
COV %	2.57	9.44	13.87	18.79	24.46	9.49	10.33

LARCH 40mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L20P(B)	11.61	503.44	11174.57	14639.40	3105.67	5676.72	2400.29
L21P(B)	10.06	505.09	11773.46	15807.00	4113.84	6514.70	2857.89
L23P(B)	11.64	453.95	10997.68	16167.60	3966.08	6001.40	2698.65
L24P(B)	11.36	488.10	9155.63	12820.40	3645.56	6510.67	2349.55
Min	10.06	453.95	9155.63	12820.40	3105.67	5676.72	2349.55
Max	11.64	505.09	11773.46	16167.60	4113.84	6514.70	2857.89
Average	11.17	487.64	10775.33	14858.60	3707.79	6175.87	2576.59
St. Dev	0.75	23.73	1129.68	1507.23	446.47	410.89	242.67
COV %	6.71	4.87	10.48	10.14	12.04	6.65	9.42

LARCH 40mm PARALLEL BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L24P(B)	11.44	464.93	2744.23	4883.63	568.39	2136.29	1556.74
L25P(B)	11.37	450.14	3145.99	5511.30	741.37	1844.17	1937.32
L26P(B)	11.29	418.03	2550.94	3913.01	649.36	1848.35	1172.43
L27P(B)	11.88	480.14	2699.08	5201.23	623.94	1851.03	1315.54
Min	11.29	418.03	2550.94	3913.01	568.39	1844.17	1172.43
Max	11.88	480.14	3145.99	5511.30	741.37	2136.29	1937.32
Average	11.50	453.31	2785.06	4877.29	645.77	1919.96	1495.51
St. Dev	0.27	26.52	254.39	692.05	72.15	144.25	334.52
COV %	2.31	5.85	9.13	14.19	11.17	7.51	22.37

LARCH PARALLEL OAK							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus , $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L16P(O)	11.21	524.73	7835.25	12560.90	2841.78	5746.00	2150.09
L17P(O)	11.34	430.37	7823.55	13265.00	3219.24	6135.14	2402.68
L18P(O)	11.47	531.45	7657.00	10708.80	3646.33	3825.03	2698.68
L19P(O)	11.48	509.78	5897.40	7314.66	2383.98	1692.63	1645.46
Min	11.21	430.37	5897.40	7314.66	2383.98	1692.63	1645.46
Max	11.48	531.45	7835.25	13265.00	3646.33	6135.14	2698.68
Average	11.37	499.08	7303.30	10962.34	3022.84	4349.70	2224.23
St. Dev	0.13	46.69	940.80	2660.04	537.97	2039.01	446.25
COV %	1.12	9.36	12.88	24.27	17.80	46.88	20.06

LARCH PARALLEL SYCAMORE							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus , $K_{i,12}$ (N/mm^2)**
L33P(S)	10.97	422.54	9940.38	13622.30	3933.66	6934.15	2746.77
L34P(S)	11.18	450.21	8071.52	11279.60	2898.69	5575.15	1929.09
L35P(S)	10.95	468.13	8470.53	9903.83	3409.13	3572.98	1112.58
L36P(S)	11.59	461.18	8049.32	9691.26	3297.70	3427.97	2403.75
Min	10.95	422.54	8049.32	9691.26	2898.69	3427.97	1112.58
Max	11.59	468.13	9940.38	13622.30	3933.66	6934.15	2746.77
Average	11.17	450.51	8632.94	11124.25	3384.79	4877.56	2048.05
St. Dev	0.30	20.05	892.85	1808.06	426.51	1685.17	708.05
COV %	2.66	4.45	10.34	16.25	12.60	34.55	34.57

Appendix D: Sitka spruce: Parallel

Double shear plane: Results

SITKA SPRUCE 20mm PARALLEL STEEL 8mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
M1	13.66	367.35	3404	5430	1602.30	1689.86	1506.88
M2	14.12	379.17	3566	5392	3013.37	3138.64	2411.21
M3	13.62	355.78	4125	5281	3630.31	3836.54	2777.96
Min	13.62	355.78	3404	5281	1602.30	1689.86	1506.88
Max	14.12	379.17	4125	5430	3630.31	3836.54	2777.96
Average	13.80	367.43	3698	5368	2748.66	2888.35	2232.01
St. Dev	0.28	11.70	378.28	77.42	1039.60	1095.01	654.21
COV %	2.00	3.18	10.23	1.44	37.82	37.91	29.31

SITKA SPRUCE 30mm PARALLEL STEEL 8mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
M7	14.57	436.17	6200	8474	6349.94	6925.09	5245.22
M8	14.86	432.87	5753	8337	4676.17	4857.91	3837.17
M9	14.67	425.18	6424	9679	4599.55	5538.12	4010.93
M10	14.03	427.89	6113	9107	5437.37	5975.50	4357.21
Min	14.03	425.18	5753.00	8337.00	4599.55	4857.91	3837.17
Max	14.86	436.17	6424.00	9679.00	6349.94	6925.09	5245.22
Average	14.53	430.53	6122.50	8899.25	5265.76	5824.15	4362.63
St. Dev	0.36	4.93	279.00	618.64	815.75	866.10	626.84
COV %	2.45	1.14	4.56	6.95	15.49	14.87	14.37

SITKA SPRUCE 20mm PARALLEL STEEL 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
M15	14.33	391.14	5566	7119	3862.99	6018.87	3571.14
M16	14.10	370.03	5107	6076	2603.95	3235.08	2275.82
M17	13.61	351.07	4697	5765	2928.12	3886.63	2406.79
Min	13.61	351.07	4697	5765	2603.95	3235.08	2275.82
Max	14.33	391.14	5566	7119	3862.99	6018.87	3571.14
Average	14.01	370.75	5123	6320	3131.69	4380.19	2751.25
St. Dev	0.37	20.04	434.73	709.21	653.73	1456.05	713.06
COV %	2.61	5.41	8.49	11.22	20.87	33.24	25.92

SITKA SPRUCE 30mm PARALLEL STEEL 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
M21	14.30	389.24	7244	9356	2610.31	3037.55	2747.27
M22	13.76	403.56	6175	7840	4497.41	5770.54	3527.87
M23	13.22	372.59	9058	11083	3525.94	4233.26	3124.74
M24	13.95	421.87	9145	11766	5087.44	6941.01	4442.77
Min	13.22	372.59	6175.00	7840.00	2610.31	3037.55	2747.27
Max	14.30	421.87	9145.00	11766.00	5087.44	6941.01	4442.77
Average	13.81	396.82	7905.50	10011.25	3930.28	4995.59	3460.66
St. Dev	0.45	20.96	1448.77	1767.43	1090.33	1712.72	728.20
COV %	3.28	5.28	18.33	17.65	27.74	34.28	21.04

SITKA SPRUCE 20mm PARALLEL BEECH 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B1	13.34	375.75	2883	3255	1437.01	1479.42	1430.84
B2	13.50	417.30	3317	4374	3425.02	3367.72	2151.87
B3	13.66	394.57	3007	3690	2029.81	2275.01	1591.68
B4	13.17	474.70	3404	5231	1831.61	1905.29	1835.83
B5	13.44	368.04	2970	3491	2274.02	2968.84	1017.77
B6	13.47	395.56	2907	3839	1347.94	1344.65	1096.82
B7	12.55	355.62	3640	4299	2068.52	2210.81	1890.96
Min	12.55	355.62	2883	3255	1347.94	1344.65	1017.77
Max	13.66	474.70	3640	5231	3425.02	3367.72	2151.87
Average	13.30	397.36	3161	4026	2059.13	2221.68	1573.68
St. Dev	0.36	39.70	293	668	690.10	741.04	420.49
COV %	2.74	9.99	9.27	16.59	33.51	33.35	26.72

SITKA SPRUCE 30mm PARALLEL BEECH 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B15	13.05	426.29	4336	3255	1515.82	1549.78	1377.40
B16	13.88	415.75	3429	4374	2706.82	3341.89	2020.33
B17	13.92	385.48	3355	3690	2059.07	2404.23	1591.63
B18	13.59	397.74	3404	5231	3128.67	3180.19	1210.57
B19	13.55	362.94	3230	3491	1892.04	2187.87	1539.93
B20	13.34	426.66	2883	3839	1770.89	1885.35	1362.12
Min	13.05	362.94	2883	3255	1515.82	1549.78	1210.57
Max	13.92	426.66	4336	5231	3128.67	3341.89	2020.33
Average	13.55	402.48	3440	3980	2178.88	2424.89	1517.00
St. Dev	0.33	25.31	483	720	613.77	710.62	281.71
COV %	2.43	6.29	14.04	18.08	28.17	29.31	18.57

SITKA SPRUCE 35mm PARALLEL BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B28	12.21	463.31	3367	5840	2501.34	2984.71	1835.16
B29	12.37	402.98	2970	4672	1083.89	1093.05	1129.14
B30	12.05	404.15	2895	3727	1729.73	1775.91	1386.84
B31	12.63	384.37	3404	5691	2687.49	2504.84	1354.49
B32	11.98	398.73	3305	3491	2875.63	2943.76	1900.44
Min	11.98	384.37	2895	3491	1083.89	1093.05	1129.14
Max	12.63	463.31	3404	5840	2875.63	2984.71	1900.44
Average	12.25	410.71	3188	4684	2175.62	2260.45	1521.22
St. Dev	0.26	30.44	238	1083	749.80	813.76	332.40
COV %	2.15	7.41	7.45	23.12	34.46	36.00	21.85

SITKA SPRUCE 20mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B38	12.95	358.94	8362	9517	2301.29	2671.01	2633.68
B39	13.98	341.20	9443	10486	2830.77	3532.34	2325.18
B40	13.88	333.08	9381	10462	3341.92	3709.10	2613.15
B41	13.60	361.77	5119	5119	14433.92	20333.51	1434.04
B42	13.91	381.83	8002	9269	2652.67	3488.04	2167.39
B43	12.22	361.88	8387	8387	3860.38	4082.12	1993.85
B44	11.30	411.34	9679	9679	3316.52	3660.02	2073.12
B45	11.33	377.65	10710	12437	3706.71	5327.32	3073.78
B46	11.76	664.80	10151	10946	3857.59	4749.64	2936.70
B47	11.58	758.56	10449	11953	3604.42	4798.67	2934.05
Min	11.30	333.08	8002	8387	2301.29	2671.01	1993.85
Max	13.98	758.56	10710	12437	3860.38	5327.32	3073.78
Average	12.55	443.25	9396	10348	3274.70	4002.03	2527.88
St. Dev	1.15	155.68	969.43	1297.95	560.79	822.77	404.91
COV %	9.14	35.12	10.32	12.54	17.13	20.56	16.02

SITKA SPRUCE 30mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B58	13.93	415.76	10586	11605	3410.35	4169.17	3209.63
B59	13.74	398.83	11903	12499	3354.70	4595.51	2994.93
B60	13.32	363.93	9356	10337	3745.82	4776.47	2932.00
B61	13.76	445.17	11257	14599	4319.15	5175.49	3394.50
B62	13.27	414.78	13891	14611	4435.47	6274.95	3909.16
B63	13.84	368.23	12686	13481	4372.27	6203.65	3717.57
B64	14.48	392.99	12139	13282	3780.72	4898.28	3232.97
B65	14.22	456.42	12847	16500	6081.01	8067.14	4561.10
B66	14.28	398.40	11580	14611	4196.01	6353.86	3621.81
B67	13.69	390.31	9443	10101	3560.98	4627.65	2854.89
Min	13.27	363.93	9356.00	10101.00	3354.70	4169.17	2854.89
Max	14.48	456.42	13891.00	16500.00	6081.01	8067.14	4561.10
Average	13.85	404.48	11568.80	13162.60	4125.65	5514.22	3442.86
St. Dev	0.39	29.71	1460.22	2047.67	796.57	1190.42	525.30
COV %	2.84	7.35	12.62	15.56	19.31	21.59	15.26

SITKA SPRUCE 35mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B89	12.13	434.36	10325	13717	4189.84	5168.31	3209.92
B90	12.39	409.29	12934	13319	3849.06	4667.87	3222.32
B91	12.09	389.71	11915	11915	3951.59	5157.73	3606.33
B92	12.25	425.37	12735	16550	4860.48	6936.64	3957.89
B93	12.75	399.23	10710	11083	3492.47	4570.46	2920.67
B94	12.12	420.05	8946	13791	3268.26	4287.78	2545.46
B95	11.76	466.22	11779	20401	4500.70	5487.92	3421.65
B96	12.43	455.13	13791	17047	5300.27	7056.57	4184.26
B97	12.09	411.81	12114	17159	4096.41	5674.78	3422.97
B98	12.29	389.28	12040	14785	4753.01	6356.09	3967.41
Min	11.76	389.28	8946.00	11083.00	3268.26	4287.78	2545.46
Max	12.75	466.22	13791.00	20401.00	5300.27	7056.57	4184.26
Average	12.23	420.05	11728.90	14976.70	4226.21	5536.42	3445.89
St. Dev	0.26	25.99	1404.46	2813.84	632.51	971.27	505.36
COV %	2.15	6.19	11.97	18.79	14.97	17.54	14.67

SITKA SPRUCE 40mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B78	14.17	441.38	12673	17656	5149.77	5160.30	3559.38
B79	14.46	453.47	14562	19196	4797.24	6727.85	4208.17
B80	13.68	420.96	12189	13506	4268.52	5957.15	3596.68
B81	14.20	457.44	11841	17233	4650.99	5525.63	4081.98
B82	14.06	435.09	10226	12338	5077.75	5397.07	3613.27
B83	14.14	442.89	14388	20650	4487.76	6872.17	4141.02
Min	13.68	420.96	10226.00	12338.00	4268.52	5160.30	3559.38
Max	14.46	457.44	14562.00	20650.00	5149.77	6872.17	4208.17
Average	14.12	441.87	12646.50	16763.17	4738.67	5940.03	3866.75
St. Dev	0.25	13.13	1638.27	3232.19	340.32	716.15	306.52
COV %	1.80	2.97	12.95	19.28	7.18	12.06	7.93

Appendix E: Larch: Perpendicular

Double shear plane: Results

LARCH PERPENDICULAR ASH							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulu s, $K_{i,12}$ (N/mm^2)
L12Pe(A)	11.15	496.40	8943.39	10032.20	3006.29	4520.90	2342.61
L13Pe(A)	11.31	477.79	11203.74	11313.50	2868.27	4611.19	2470.38
L14Pe(A)	11.50	469.72	11192.90	11192.90	3036.36	4658.04	2608.61
L15Pe(A)	11.16	442.90	11638.83	12546.50	2725.73	4377.90	2312.06
Min	11.15	442.90	8943.39	10032.20	2725.73	4377.90	2312.06
Max	11.50	496.40	11638.83	12546.50	3036.36	4658.04	2608.61
Average	11.28	471.70	10744.71	11271.28	2909.16	4542.01	2433.41
St. Dev	0.16	22.22	1218.71	1027.85	142.51	123.32	135.44
COV %	1.46	4.71	11.34	9.12	4.90	2.72	5.57

LARCH 30mm PERPENDICULAR BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulu s, $K_{i,12}$ (N/mm^2)
L5Pe(B)	11.39	472.20	9581.75	9686.00	3057.99	4711.37	2475.05
L6Pe(B)	12.11	577.23	11716.33	12594.20	2881.06	5078.97	3746.92
L7Pe(B)	11.67	473.97	8236.52	9821.03	2512.99	4126.08	2443.22
L8Pe(B)	11.42	479.34	10428.74	11761.40	2727.93	4547.72	2874.46
Min	11.39	472.20	8236.52	9686.00	2512.99	4126.08	2443.22
Max	12.11	577.23	11716.33	12594.20	3057.99	5078.97	3746.92
Average	11.65	500.69	9990.83	10965.66	2795.00	4616.03	2884.91
St. Dev	0.33	51.12	1462.20	1441.42	231.37	395.02	607.24
COV %	2.85	10.21	14.64	13.14	8.28	8.56	21.05

LARCH 30mm PERPENDICULAR BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L17Pe(B)	12.21	474.84	2350.37	3515.67	552.13	1927.05	1152.13
L18Pe(B)	11.36	486.41	2522.28	3104.58	513.81	1246.84	976.76
L19Pe(B)	11.25	446.04	2094.96	2958.41	520.13	1065.82	842.43
Min	11.25	446.04	2094.96	2958.41	513.81	1065.82	842.43
Max	12.21	486.41	2522.28	3515.67	552.13	1927.05	1152.13
Average	11.61	469.10	2322.54	3192.89	528.69	1413.24	990.44
St. Dev	0.53	20.79	215.01	288.93	20.54	454.09	155.30
COV %	4.54	4.43	9.26	9.05	3.89	32.13	15.68

LARCH 40mm PERPENDICULAR BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L9Pe(B)	11.84	488.55	11259.71	14037.80	2851.57	4765.18	2304.67
L10Pe(B)	11.70	430.32	11362.72	12902.70	2612.44	4650.12	2164.48
L11Pe(B)	12.23	450.43	8271.73	13892.90	1641.15	3587.65	1342.40
L12Pe(B)	12.23	494.95	9646.17	12452.20	2764.98	4989.34	2052.62
Min	11.70	430.32	8271.73	12452.20	1641.15	3587.65	1342.40
Max	12.23	494.95	11362.72	14037.80	2851.57	4989.34	2304.67
Average	12.00	466.06	10135.08	13321.40	2467.53	4498.07	1966.04
St. Dev	0.27	30.88	1470.03	768.26	559.72	623.08	428.36
COV %	2.25	6.63	14.50	5.77	22.68	13.85	21.79

LARCH 40mm PERPENDICULAR BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L13Pe(B)	12.80	542.56	1947.41	3562.34	430.91	1125.29	808.31
L14Pe(B)	12.38	460.36	2560.10	5791.43	631.00	1062.72	904.79
L15Pe(B)	12.21	557.06	1871.66	3643.02	409.28	1291.59	919.59
L16Pe(B)	12.11	496.03	2496.48	4625.12	436.70	1526.52	944.52
Min	12.21	460.36	1871.66	3562.34	409.28	1062.72	808.31
Max	12.80	557.06	2560.10	5791.43	631.00	1291.59	919.59
Average	12.46	519.99	2126.39	4332.26	490.40	1159.87	877.56
St. Dev	0.30	52.15	377.51	1264.32	122.25	118.29	60.43
COV %	2.42	10.03	17.75	29.18	24.93	10.20	6.89

LARCH PERPENDICULAR OAK							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L21Pe(O)	10.77	488.69	5177.26	8513.97	2284.42	2032.09	1540.48
L22Pe(O)	11.52	500.26	6150.32	9110.45	2699.29	2553.11	1720.92
L23Pe(O)	11.52	499.38	7004.68	8608.42	2847.23	2874.29	2104.41
L24Pe(O)	11.51	549.29	6298.24	7529.10	2253.89	1814.20	1685.04
Min	10.77	488.69	5177.26	7529.10	2253.89	1814.20	1540.48
Max	11.52	549.29	7004.68	9110.45	2847.23	2874.29	2104.41
Average	11.33	509.40	6157.62	8440.49	2521.21	2318.42	1762.71
St. Dev	0.37	27.10	752.43	661.58	297.51	483.14	240.78
COV %	3.28	5.32	12.22	7.84	11.80	20.84	13.66

LARCH PERPENDICULAR SYCAMORE							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
L29Pe(S)	10.60	453.08	8176.14	9057.25	2595.89	4107.18	2003.82
L30Pe(S)	11.10	506.85	5892.84	7992.76	2317.02	2470.59	1650.00
L31Pe(S)	11.28	504.81	6247.60	6355.78	1971.69	1775.02	1869.18
L32Pe(S)	10.85	458.56	7743.67	7847.51	2719.18	2976.72	2309.89
Min	10.60	453.08	5892.84	6355.78	1971.69	1775.02	1650.00
Max	11.28	506.85	8176.14	9057.25	2719.18	4107.18	2309.89
Average	10.96	480.83	7015.06	7813.33	2400.95	2832.37	1958.22
St. Dev	0.30	28.97	1114.65	1111.33	331.95	982.32	276.09
COV %	2.72	6.03	15.89	14.22	13.83	34.68	14.10

Appendix F: Sitka Spruce: Perpendicular

Double shear plane: Results

SITKA SPRUCE 20mm PARALLEL STEEL 8mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
M4	13.27	336.62	3653	5430	5392.00	824.04	1119.17
M5	13.92	439.06	4162	5392	6722.00	2708.16	2151.05
M6	13.64	388.46	3901	5281	5666.00	2359.28	1879.74
Min	13.27	336.62	3653	5281	5392.00	824.04	1119.17
Max	13.92	439.06	4162	5430	6722.00	2708.16	2151.05
Average	13.61	388.05	3905	5368	5926.67	1963.83	1716.65
St. Dev	0.33	51.22	254.53	77.42	702.27	1002.38	534.93
COV %	2.41	13.20	6.52	1.44	11.85	51.04	31.16

SITKA SPRUCE 30mm PARALLEL STEEL 8mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
M11	14.57	436.17	4846	6970	6970.00	2066.26	2083.14
M12	14.86	432.87	2646	6523	6523.00	1884.40	1650.22
M13	14.67	425.18	4684	7169	7169.00	2106.44	1879.79
M14	14.03	427.89	5268	9169	9169.00	2119.28	2078.44
Min	14.03	425.18	2646.00	6523.00	6523.00	1884.40	1650.22
Max	14.86	436.17	5268.00	9169.00	9169.00	2119.28	2083.14
Average	14.53	430.53	4361.00	7457.75	7457.75	2044.09	1922.90
St. Dev	0.36	4.93	1169.53	1172.38	1172.38	108.83	205.00
COV %	2.45	1.14	26.82	15.72	15.72	5.32	10.66

SITKA SPRUCE 20mm PARALLEL BEECH 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B8	13.45	341.60	2174	2212	1220.75	1451.25	993.16
B9	13.42	399.70	1578	1764	521.95	528.31	604.84
B10	13.88	384.19	3305	3591	2215.45	2128.96	1601.28
B11	13.50	428.70	2299	2982	1420.45	1711.85	1194.39
B12	13.42	396.69	2522	3516	953.74	1320.77	1083.94
B13	13.23	401.37	2249	3342	1311.48	1550.74	1360.63
B14	13.32	364.80	2771	3056	1914.56	1853.95	1329.65
Min	13.23	341.60	1578	1764	521.95	528.31	604.84
Max	13.88	428.70	3305	3591	2215.45	2128.96	1601.28
Average	13.46	388.15	2414	2923	1365.48	1506.55	1166.84
St. Dev	0.20	28.15	537	688	567.43	507.72	317.92
COV %	1.52	7.25	22.24	23.55	41.56	33.70	27.25

SITKA SPRUCE 30mm PARALLEL BEECH 10mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B21	13.23	400.25	2485	3553	1060.40	1146.02	1028.49
B22	13.84	452.04	3367	5715	1640.35	1849.73	1391.65
B23	13.73	424.60	2435	3715	1311.58	1352.91	1072.15
B24	13.50	388.95	2646	3280	1534.04	1405.67	1179.01
B25	14.70	420.12	3044	4796	1624.94	1904.80	1378.06
B26	14.22	412.02	3417	3690	1807.19	2128.00	1597.21
B27	14.15	461.38	3454	4075	2033.68	2439.17	1681.77
Min	13.23	388.95	2435.00	3280.00	1060.40	1146.02	1028.49
Max	14.70	461.38	3454.00	5715.00	2033.68	2439.17	1681.77
Average	13.91	422.77	2978.29	4117.71	1573.17	1746.61	1332.62
St. Dev	0.49	26.22	451.50	854.30	318.05	464.17	252.12
COV %	3.52	6.20	15.16	20.75	20.22	26.58	18.92

SITKA SPRUCE 35mm PARALLEL BEECH 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B33	11.15	404.61	3007	3380	1290.14	1449.80	1128.97
B34	10.54	412.01	3578	5405	2260.69	2362.89	1774.02
B35	11.87	371.86	3491	5293	2077.11	2074.30	1560.33
B36	11.95	391.17	3280	5517	1144.37	1224.93	1003.23
B37	11.82	392.85	3305	5206	1138.51	1331.13	1028.24
Min	10.54	371.86	3007	3380	1138.51	1224.93	1003.23
Max	11.95	412.01	3578	5517	2260.69	2362.89	1774.02
Average	11.47	394.50	3332	4960	1582.16	1688.61	1298.96
St. Dev	0.61	15.29	221	891	542.94	500.81	347.72
COV %	5.32	3.88	6.62	17.96	34.32	29.66	26.77

SITKA SPRUCE 20mm PARALLEL STEEL 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
M18	13.52	394.64	5181	7915	1251.90	1505.66	1468.81
M19	13.25	393.97	5442	9306	1493.65	1701.95	1351.44
M20	12.77	399.76	4784	7852	2386.37	2837.97	1988.52
Min	12.77	393.97	4784	7852	1251.90	1505.66	1351.44
Max	13.52	399.76	5442	9306	2386.37	2837.97	1988.52
Average	13.18	396.13	5136	8358	1710.64	2015.19	1602.93
St. Dev	0.38	3.17	331.33	821.88	597.55	719.27	339.05
COV %	2.87	0.80	6.45	9.83	34.93	35.69	21.15

SITKA SPRUCE 30mm PARALLEL STEEL 10mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
M24	13.67	393.44	7343	8971	1178.26	1322.67	1451.68
M25	14.44	428.95	8511	9505	3309.34	3865.96	3055.16
M26	13.36	362.74	5914	8424	1783.88	2006.12	1623.10
Min	13.36	362.74	5914.00	8424.00	1178.26	1322.67	1451.68
Max	14.44	428.95	8511.00	9505.00	3309.34	3865.96	3055.16
Average	13.83	395.04	7256.00	8966.67	2090.50	2398.25	2043.31
St. Dev	0.56	33.13	1300.68	540.51	1098.13	1316.21	880.47
COV %	4.03	8.39	17.93	6.03	52.53	54.88	43.09

SITKA SPRUCE 20mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B48	12.15	430.74	10300	12313	3205.68	4297.38	2007.48
B49	12.32	420.88	10114	12089	2079.75	2922.82	2072.29
B50	12.49	303.75	5541	7865	1255.49	1599.54	1086.95
B51	13.18	434.36	9530	11220	2161.34	2811.71	2022.61
B52	13.26	425.16	7952	9294	2572.26	3299.69	2125.40
B53	13.30	428.78	10002	12735	2575.16	3043.66	2231.27
B54	12.19	411.47	9132	12027	2818.19	3446.91	2316.50
B55	10.50	422.29	9791	12872	2385.78	3125.67	2094.96
B56	11.81	424.22	8076	9505	1941.75	2485.99	1698.68
B57	12.24	416.56	7343	9642	2144.54	2684.76	1780.88
Min	10.50	303.75	5541	7865	1255.49	1599.54	1086.95
Max	13.30	430.74	10300	12872	3205.68	4297.38	2316.50
Average	12.25	409.32	8695	10927	2330.96	2989.60	1934.93
St. Dev	0.83	40.02	1596.91	1845.57	562.29	734.87	373.84
COV %	6.75	9.78	18.37	16.89	24.12	24.58	19.32

SITKA SPRUCE 30mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B68	13.84	404.21	10424	11133	2852.51	3379.96	2281.96
B69	13.60	373.73	9530	9915	2068.47	2538.57	2165.48
B70	13.54	389.31	11803	12139	2312.34	3279.16	2332.87
B71	13.18	417.57	9716	9766	2464.80	3289.30	2288.90
B72	13.37	430.23	11070	12214	2808.99	3443.72	2459.17
B73	13.83	399.15	10549	11642	2484.70	3338.79	2343.23
B74	13.46	461.00	10350	10350	2829.35	3934.55	2219.61
B75	11.41	379.00	10052	10052	2187.71	2631.76	1894.16
B76	12.75	379.75	8933	9393	2114.33	2753.04	1945.48
B77	12.60	362.06	9940	10486	2168.26	2839.75	2002.78
Min	11.41	362.06	8933.00	9393.00	2068.47	2538.57	1894.16
Max	13.84	461.00	11803.00	12214.00	2852.51	3934.55	2459.17
Average	13.16	399.60	10236.70	10709.00	2429.15	3142.86	2193.37
St. Dev	0.74	29.92	808.67	1012.26	308.62	437.43	188.06
COV %	5.63	7.49	7.90	9.45	12.71	13.92	8.57

SITKA SPRUCE 35mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B99	11.52	368.71	9579	9642	2335.54	2770.48	1202.24
B100	11.55	451.73	11605	12847	3789.07	4423.89	2999.98
B101	11.31	363.59	11381	11953	2629.99	3201.88	2281.96
B102	11.20	401.41	10710	11046	2896.19	3267.26	2319.75
B103	11.48	376.68	10971	11654	2696.17	3160.81	2298.73
B104	11.91	390.44	10623	12251	3093.68	4055.93	2680.21
B105	11.81	455.26	12226	13406	2812.53	3539.11	2593.72
B106	10.19	405.58	11953	12574	3160.91	3809.14	2609.37
B107	11.50	402.69	9927	10126	2606.57	2883.78	2136.74
B108	11.98	414.39	11630	11630	3095.28	4040.07	3967.41
Min	10.19	363.59	9579.00	9642.00	2335.54	2770.48	1202.24
Max	11.98	455.26	12226.00	13406.00	3789.07	4423.89	3967.41
Average	11.45	403.05	11060.50	11712.90	2911.59	3515.23	2509.01
St. Dev	0.51	31.31	861.55	1178.67	402.72	549.66	698.60
COV %	4.43	7.77	7.79	10.06	13.83	15.64	27.84

SITKA SPRUCE 40mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m^3)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm^2)	Slip Modulus, $K_{i,12}$ (N/mm^2)	Yield Modulus, $K_{i,12}$ (N/mm^2)
B84	13.80	413.41	11046	11046	2627.58	3409.84	2504.23
B85	13.67	438.52	10636	10921	2701.33	3447.86	2367.68
B86	14.20	459.01	11431	12325	3417.59	4448.37	2964.87
B87	13.54	467.02	12363	12847	3000.00	4268.78	2687.09
B88	13.28	464.14	10859	13816	3000.84	3745.45	2632.54
Min	13.28	413.41	10636.00	10921.00	2627.58	3409.84	2367.68
Max	14.20	467.02	12363.00	13816.00	3417.59	4448.37	2964.87
Average	13.70	448.42	11267.00	12191.00	2949.47	3864.06	2631.28
St. Dev	0.34	22.52	678.52	1226.05	312.05	474.03	223.62
COV %	2.48	5.02	6.02	10.06	10.58	12.27	8.50

Appendix G: Sitka Spruce – Parallel & Perpendicular

Four shear plane: Results

SITKA SPRUCE 20mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m³)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm²)	Slip Modulus, $K_{i,12}$ (N/mm²)	Yield Modulus, $K_{i,12}$ (N/mm²)
B109	11.52	410.30	7157	10176	6056.48	8558.75	4365.29
B110	11.84	390.97	9418	10834	5033.66	5841.51	2779.31
B111	12.08	421.67	8474	11381	3420.05	4887.02	3284.43
B112	11.85	407.48	7293	7368	4353.39	6681.77	3528.73
B113	12.12	390.11	7815	6511	4133.94	5195.83	2891.76
B114	11.51	394.58	1354	9753	4158.33	4838.71	1532.08
B115	11.49	428.76	7828	11480	5717.55	8697.60	3756.40
B116	11.71	376.49	6511	9455	2871.76	4204.58	2891.93
B117	12.24	404.52	7778	10300	3521.51	4046.65	3036.79
B118	12.54	371.47	8660	8660	3840.38	4729.72	2560.23
B119	12.16	368.23	7380	7579	5235.77	6429.93	4296.06
B120	11.19	410.22	9778	10163	4639.70	5548.01	3275.61
B121	13.79	438.21	7703	10238	5199.19	6766.62	3594.85
Min	11.19	368.23	1354.00	6511.00	2871.76	4046.65	1532.08
Max	13.79	438.21	9778.00	11480.00	6056.48	8697.60	4365.29
Average	12.00	401.00	7473.00	9530.62	4475.52	5878.98	3214.88
St. Dev	0.65	21.63	2049.48	1561.21	944.90	1499.65	748.75
COV %	5.44	5.39	27.43	16.38	21.11	25.51	23.29

SITKA SPRUCE 30mm PARALLEL BEECH 20mm							
Sample number	Moisture Content, %	Density, P (kg/m³)	Yield 5% offset load (F_{max})	F_{max} (kN)	Initial slip Modulus, $k_{i,12}$ (N/mm²)	Slip Modulus, $K_{i,12}$ (N/mm²)	Yield Modulus, $K_{i,12}$ (N/mm²)
B131	14.87	353.52	6026	10859	2560.97	3042.80	1911.81
B132	13.96	427.61	13928	7505	3590.06	4171.25	2775.60
B133	13.62	401.98	7840	13294	4590.45	4181.77	2923.36
B134	13.62	378.76	6473	10486	3719.13	3790.03	2453.81
Min	13.62	353.52	6026.00	7505.00	2560.97	3042.80	1911.81
Max	14.87	427.61	13928.00	13294.00	4590.45	4181.77	2923.36
Average	14.02	390.47	8566.75	10536.00	3615.15	3796.46	2516.15
St. Dev	0.59	31.70	3656.52	2373.49	831.45	534.47	448.05
COV %	4.21	8.12	42.68	22.53	23.00	14.08	17.81

SITKA SPRUCE 35mm PARALLEL BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B140	13.01	401.61	9579	9567	4568.26	5884.60	2894.01
B141	11.87	428.16	11605	7803	4034.95	4202.87	2273.64
B142	12.25	429.54	11381	11369	5248.85	6090.36	3243.81
B143	12.07	383.68	10710	10288	3623.13	3821.51	2143.89
B144	12.05	423.97	10971	8337	3667.20	3828.39	2507.47
B145	11.81	400.97	10623	10076	4271.87	4473.84	3150.60
B146	11.99	408.24	12226	12176	4128.27	4644.28	2964.16
B147	11.92	421.92	11953	13456	4488.26	4755.35	2933.85
Min	11.81	383.68	9579.00	7803.00	3623.13	3821.51	2143.89
Max	13.01	429.54	12226.00	13456.00	5248.85	6090.36	3243.81
Average	12.12	412.26	11131.00	10384.00	4253.85	4712.65	2763.93
St. Dev	0.38	16.28	847.81	1898.95	527.77	859.49	406.43
COV %	3.17	3.95	7.62	18.29	12.41	18.24	14.70

SITKA SPRUCE 20mm PERPENDICULAR BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B122	12.27	401.97	4125	6237	6940.68	8093.63	1607.58
B123	12.34	414.11	4274	6237	5163.58	5284.46	2173.27
B124	11.91	313.95	3839	5107	4978.80	4639.99	1934.47
B125	11.92	401.49	4411	4659	3243.35	3090.88	3069.13
B126	11.76	379.92	6088	6535	5616.66	6431.37	5089.66
B127	12.25	378.93	6113	6113	4035.14	5647.88	3790.95
B128	11.60	395.42	4200	6361	5264.89	5448.17	2311.81
B129	11.76	359.08	5927	5927	4067.45	4401.91	2915.56
B130	11.53	417.31	5020	6647	4353.15	4852.66	1909.55
Min	11.53	313.95	3839.00	4659.00	3243.35	3090.88	1607.58
Max	12.34	417.31	6113.00	6647.00	6940.68	8093.63	5089.66
Average	11.93	384.69	4888.56	5980.33	4851.52	5321.22	2755.77
St. Dev	0.30	32.29	921.82	666.88	1081.10	1394.01	1111.74
COV %	2.50	8.39	18.86	11.15	22.28	26.20	40.34

SITKA SPRUCE 30mm PERPENDICULAR BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B135	13.48	404.87	6349	9157	4193.75	4287.36	1933.79
B136	12.98	398.55	6511	9182	4969.52	5907.82	4239.51
B137	13.62	385.55	6150	9157	4987.34	4877.57	3860.64
B138	13.71	420.93	6846	10723	5192.70	6413.93	3355.58
B139	12.67	374.65	6498	7020	4914.00	4844.48	3224.00
Min	12.67	374.65	6150.00	7020.00	4193.75	4287.36	1933.79
Max	13.71	420.93	6846.00	10723.00	5192.70	6413.93	4239.51
Average	13.29	396.91	6470.80	9047.80	4851.46	5266.23	3322.70
St. Dev	0.45	17.80	255.38	1319.10	382.54	868.33	876.04
COV %	3.36	4.49	3.95	14.58	7.88	16.49	26.37

SITKA SPRUCE 35mm PERPENDICULAR BEECH 20mm							
Sample number	**Moisture Content, %**	**Density, P (kg/m^3)**	**Yield 5% offset load (F_{max})**	**F_{max} (kN)**	**Initial slip Modulus, $k_{i,12}$ (N/mm^2)**	**Slip Modulus, $K_{i,12}$ (N/mm^2)**	**Yield Modulus, $K_{i,12}$ (N/mm^2)**
B148	11.76	436.51	7492	11282	5866.18	6821.19	3265.75
B149	11.60	422.47	8474	11406	6792.97	7218.77	3715.94
B150	11.90	364.39	9393	12040	4261.98	4713.76	2586.61
B151	11.48	445.65	6386	11244	4012.34	4013.54	2125.21
B152	11.42	418.43	10213	12276	4586.76	4735.91	2505.99
B153	11.47	411.97	6188	10896	5997.23	5655.71	3746.05
Min	11.42	364.39	6188.00	10896.00	4012.34	4013.54	2125.21
Max	11.90	445.65	10213.00	12276.00	6792.97	7218.77	3746.05
Average	11.60	416.57	8024.33	11524.00	5252.91	5526.48	2990.93
St. Dev	0.19	28.37	1625.14	524.79	1119.52	1275.25	681.10
COV %	1.62	6.81	20.25	4.55	21.31	23.08	22.77