Variations in low-grade wood modification and stress lamination:
small-scale models and full-scale column construction and testing

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This dissertation is submitted for the degree of Doctor of Philosophy

DECLARATION

This dissertation is the result of my own work and includes nothing which is the outcome of work done in collaboration except as specified in the acknowledgements and text.

This work is not substantially the same as any that I have submitted, or, is being concurrently submitted for a degree or diploma or other qualification at the University of Cambridge or any other University or similar institution. I further state that no substantial part of my dissertation has already been submitted, or, is being concurrently submitted for any such degree, diploma or other qualification at the University of Cambridge or any other University or similar institution.

This work does not exceed the prescribed 80,000 word limit by the Architecture Degree Committee.

SUMMARY

Sitka spruce (*Picea sitchensis*) trees in UK forests are expected to yield nearly 30% more softwood than current levels in the coming decades. These are relatively fast-growing trees, yielding low-grade wood that is incompatible with standards for glue-based lamination. Alongside this forecasted increase, there is a major worldwide shift towards building taller and faster with massive laminated and engineered wood products, mainly as substitutes for steel and concrete. For making the best use of the expected increase in UK softwood and also to expand the scope of tall wood construction, alternative ways of working with low-grade wood are needed, along with developing new variations of existing techniques in processing and construction. This thesis examines two strategies: wood modification by impregnation and stress lamination. The former involves treating wood under pressure in a liquid solution. Once impregnated, the liquid is then solidified in-situ, grafting onto the wood to enhance its properties through direct molecular interactions in the cell wall. While wood treatments are usually done to increase durability, literature and preliminary small-scale testing suggest that impregnation can also lead to increases in stiffness and strength. Compared to wood modification, stress lamination is a relatively simple yet effective technique, and is useful for laminating wood without glue. Although widely used in timber bridges, stress lamination has found little to no application in buildings, primarily due to concerns regarding losses in prestress levels from creep and the moisture-related movement and shrinkage of wood. Scale models and testing emphasise the technique's potential for both standard and bespoke structural elements for buildings. Full-scale detailing, construction, and testing of straight columns further establishes structural performance and feasibility. Test results from shearing five full-scale stress-laminated connections show performance beyond that of conventional mechanical fasteners. Twenty-five columns were also tested at full-scale, showing comparable buckling performance to Eurocode estimates for solid timber. Examples from literature and a new detail with overdried hardwood plates, tested during a six-month period, demonstrate that prestress losses can be mitigated to ensure long-term reliability in buildings. The full-scale testing performed in this thesis therefore highlights the usefulness, performance, and reliability of stress lamination with low-grade wood for multi-storey construction.

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Chapter 1 INTRODUCTION

The challenges of building with low-grade wood

Building with low-grade wood from fast-growing coniferous trees comes with formidable challenges. Trees like Sitka spruce (*Picea sitchensis*) yield wood that is characterised by relatively low density and widely spaced annual growth rings as compared to other species such as Douglas fir (*Pseudotsuga menziesii*). Low-grade wood therefore has noticeably lower stiffness and strength in structural applications, and can also be difficult to kiln dry without significant twisting and warping. These material characteristics directly influence structural design: seasonal variations in temperature and humidity can result in large amounts of movement in joints between panels and in connections between beams and columns. Limitations in density can also affect the load-carrying capacity of common types of dowelled connections due to the direct dependence of bearing strength on density. The overall stability of columns and other structural elements made with low-grade wood needs to be carefully considered due to the lower modulus of elasticity (MoE) of the material. In light of these challenges, the UK produces and uses the majority of its sawn wood in low-value applications like fencing and packaging, while importing higher-grade wood for most of its wood construction.[1] This thesis proposes and examines two alternative techniques, one chemical and the other mechanical, for working with low-grade wood in the UK, or even further afield, for the purpose of multi-storey construction.

UK context

Forestry, climate, and forecasted growth

Within the UK, two of the main influences on the quality of softwood are forestry management and climate. Compared to other European countries like Germany or Finland, the UK has fewer forests overall, but also fewer managed forests for properly growing and harvesting higher-quality wood.[2] The relationship between climate, tree growth, and mechanical properties, however, is also complex. The milder and shorter UK winters compared to winters in areas in central Europe, Scandinavia, and North America contribute to UK trees growing faster, allowing them to be harvested after a shorter growing rotation, in turn yielding lower density and more juvenille wood. For example, both Douglas fir and Sitka spruce are species native to North America that were introduced to the UK through deliberate planting efforts. Extensive testing by Lavers[3] shows that Sitka spruce from

Canada is on average 10% more dense with a 20% increase in the MoE in bending compared to UK Sitka spruce; comparisons of the properties of Douglas fir show very similar trends, with Canadian Douglas fir being on average 8% more dense with a similar 20% increase in the MoE in bending than Douglas fir from the UK. In the long term, increasing UK forestry management activities can help to produce wood with improved properties. Longer growing rotations, with practices like proper site selection, tree breeding and selection, and thinning and pruning can lead to older and straighter growing trees with fewer knots[4], thereby resulting in stronger wood from UK forests. The context of the UK's milder climate and its effects on lowering stiffness and density, however, is more difficult to overcome. Instead of importing large amounts of high-grade wood, industry and researchers can consider established techniques for adding value to wood, and develop new ways of working with low-grade wood for construction.

A major incentive to innovate with low-grade wood comes from the future growth predictions for the UK's forests. In the coming decades, the amount of UK wood available for harvest is expected to increase by an estimate of roughly 30%.[5] This increase is due to planting efforts in the latter half of the twentieth century. Current available wood volumes of 14 million m^3 annually are expected to increase to just over 18 million m^3 in 2027-2031. Figure 1.1 shows that the majority of this increase in wood originates from private forests in Scotland rather than those owned by the Forestry Commission (FC). More importantly, this increase in UK wood is almost exclusively expected to be from Sitka spruce trees, as illustrated in Figure 1.2. Estimates from the Scottish Forestry Commission in 2011 also indicate that the use of sawn UK Sitka spruce is almost equally divided between three areas: fencing, pallet and packaging, and construction.[6] In the last category, approximately 95% of the Sitka spruce for construction is dried, and planed into dimensioned structural timber, graded at the C16 strength class. The C16 strength class refers to softwood with a characteristic (or 5^{th}-percentile) bending strength of $16N/mm^2$.[7] Wood imported into the UK from central and northern Europe, however, is typically graded at the C24 strength class, with a characteristic bending strength of $24N/mm^2$. While the C16 strength class is still indeed a structural grade, it is the second lowest strength class, next to only C14, whereas the highest strength class quoted in standards is C50, for material with a characteristic bending strength of $50N/mm^2$.[8] The general use of the term 'low-grade' throughout this thesis therefore will be used to represent UK wood graded almost exclusively at the C16 strength class.

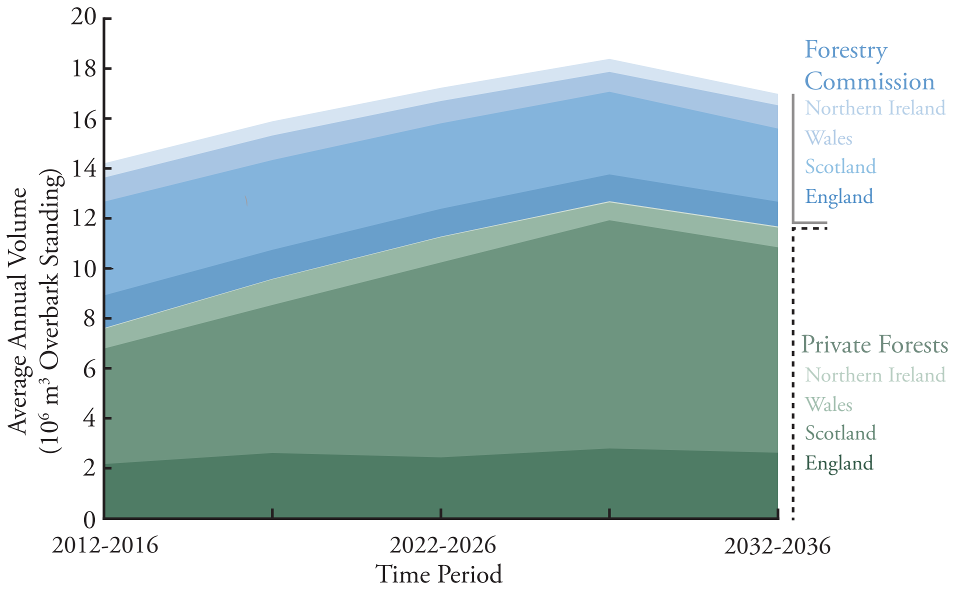


Figure 1.1: Predicted growth of UK forests based on updated estimates from the Forestry Commission in 2012.[5]

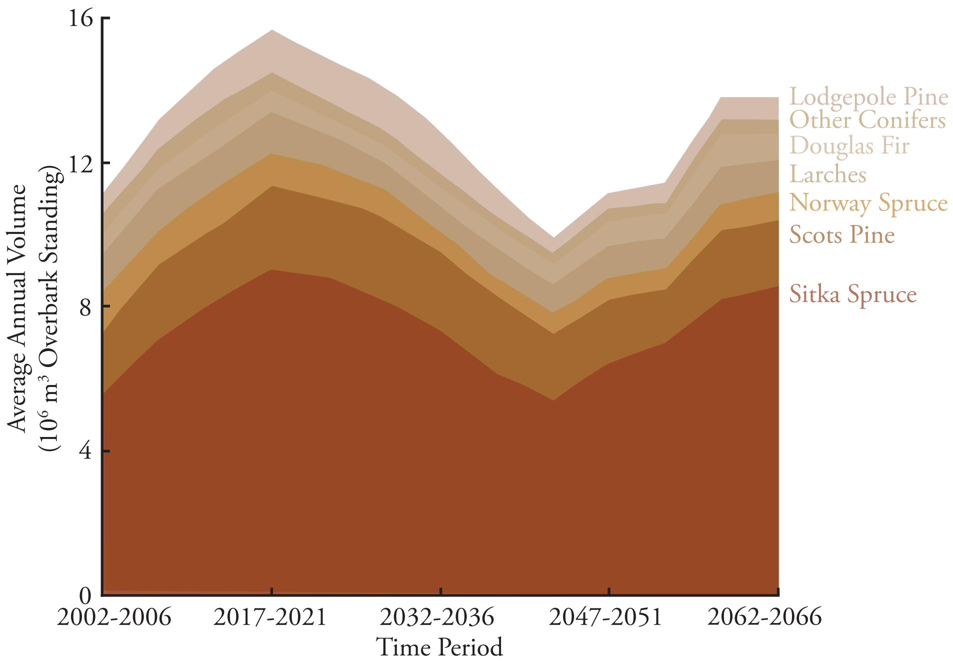


Figure 1.2: Predicted growth of UK forests by species based on estimates from the Forestry Commission in 2011.[4]

Sitka spruce as a 'giant' tree species

Despite yielding relatively low-grade wood for construction, Sitka spruce is known as a 'giant' tree species in old growth forests in North America. Instead of considering absolute values for the MoE or bending strength, the wood species' specific stiffness and specific strength are relatively high, as these 'specific' properties take into account density. The specific stiffness and specific strength are simply the ratio of the MoE or bending strength to density, respectively. The efficient properties Sitka spruce were also the main reason for its common usage in early aircraft construction, where weight needed to be kept to a minimum while maximising strength and stiffness.[9] Given basic wind protection and enough time in sufficiently rainy growing conditions, the world's tallest Sitka spruce trees, as shown in Figure 1.3, have also reached impressive sizes and heights due to the mechanical efficiency of their material. For example, the 400-year-old Carmanah Giant tree, only discovered in Canada in 1988, is approximately 95m tall with a breast-height diameter of 3m.[10] Other ‘giant’ species like Coastal Redwood (*Sequoia sempervirens*) can easily surpass the 100m mark to record-setting heights of around 115m, but Sitka spruce trees still have the ability to naturally reach heights of a 25-30 storey steel or reinforced concrete building. Compared to these ‘giant’ old-growth trees, which are now closely protected for heritage purposes, managed UK Sitka spruce trees are planted for the purpose of harvesting. These managed trees only reach heights of about 20-30m during a typical growing rotation of about 40 years, with a breast-height diameter of 25-40cm.[11] Height records and the reputation of Sitka spruce as a ‘giant’ tree species can nonetheless provide inspiration for designers and suggest an immense potential for multi-storey wood construction. The world’s tallest Sitka spruce trees not only surpass the height of many steel and concrete buildings, but also easily dwarf the tallest wood buildings today of 10-14 storeys that are 35-49m in height. Figure 1.4 illustrates this comparison, and shows how the world’s tallest timber buildings are actually quite similar in height to the UK’s managed Sitka spruce trees that are planted for harvest.

Figure 1.3: Giant Sitka spruce trees in Carmanah Walbran Provincial Park, British Columbia, Canada. (Photographer: Adrian Dorst)

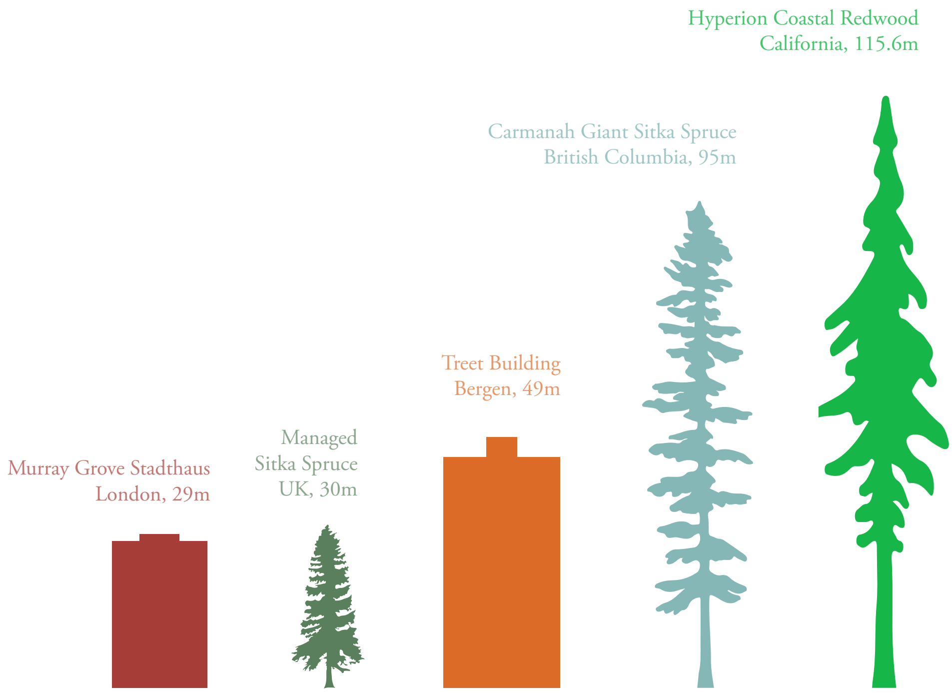


Figure 1.4: Height comparison of tall and managed trees with recent tall timber buildings.

UK wood imports and industry

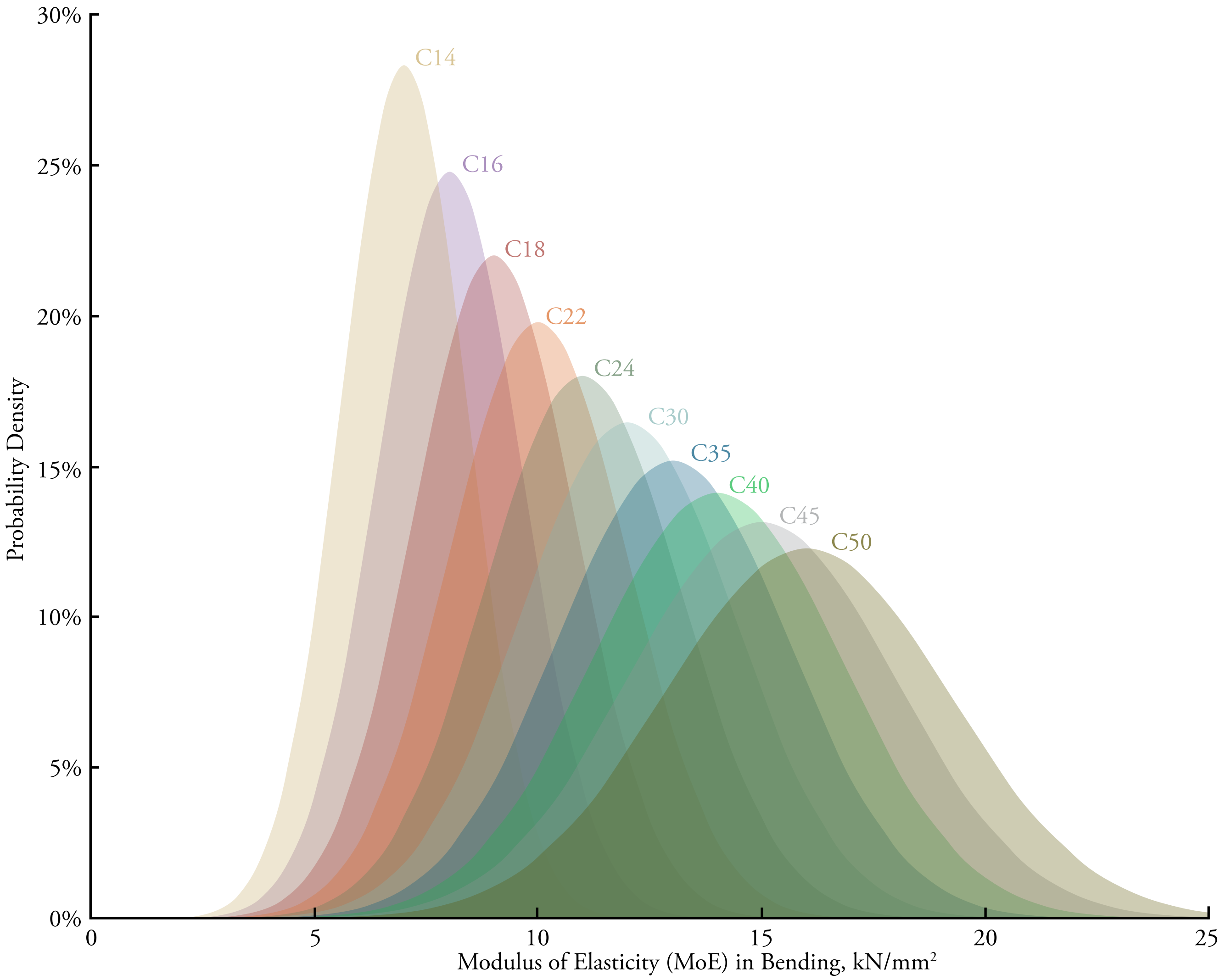
By general reputation, the UK is far more often associated with masonry, concrete, and steel construction than building with wood. Yet UK forests currently yield almost 11 million tonnes of wood annually, with approximately 1.2 million tonnes, or roughly 1 million m^3 used for construction.[12] Although this initial production estimate is for green wood in the form of logs, and includes the mass of water before sawing and kiln drying, the UK's overall wood production is comparable to other major structural materials in terms of raw tonnage: annual UK production of crude steel is approximately 10 million tonnes,[13] and 9 million tonnes for cement.[14] These annual production levels of UK steel, cement, and wood are somewhat similar, but imported amounts vary significantly: annual steel and cement imports are far less than domestic production, and come to roughly 6 and 1 million tonnes, respectively.[15] Annual UK wood imports for construction, however, are over four times that of domestic production, with 4.2 million m^3 imported each year.[16] Many types of value-added and engineered products for large-scale timber construction like glulam beams and columns, cross-laminated timber (CLT) panels, or laminated-veneer lumber (LVL) are only sparsely produced in the UK, if at all. The UK depends on imports for meeting its need for engineered wood products.[17] Reducing imports of value-added products for construction is therefore another strong incentive to innovate and make the best use of UK-grown Sitka spruce in the future.

One of the key barriers in the UK wood industry to add value to basic sawn timber for construction are current kiln drying practices. There is an apparent incompatibility between UK-grown wood and standards for gluing and lamination. While most European sawmills kiln dry softwood to about 12% moisture content or less to closely match the mean moisture content expected in service, the UK's largest sawmills only dry Sitka spruce to about 20% moisture content.[18] The UK mills follow this procedure because drying Sitka spruce to 12% moisture content results in significant increases in rejection rates in subsequent grading processes, especially due to problems with twisting.[19] Furthermore, some also claim that the UK market has little demand for wood dried to levels below 20% moisture content.[20] Such claims, however, should be seriously questioned in light of the UK being a major importer of wood dried to 12% moisture content. The higher moisture content of 20% of UK-grown wood, while representing the minimum amount of drying needed for grading and preventing fungal growth, precludes any further value-added processes involving gluing. Minimum standards for producing value-added products with glue, such as glulam or CLT, require material at a moisture content between 8-15%, with no two laminations having a difference in moisture content greater than 4%.[21] Current European glulam production therefore typically uses at least C24 graded wood, dried to 12% moisture content to produce common grades of GL24 glulam.

If UK Sitka spruce could be successfully dried to 12% moisture content, without major increases in rejection rates, the material could also be graded at a higher strength class. Even though the relatively low C16 strength class of UK Sitka spruce prescribes a characteristic bending strength of 16N/mm^2 for designers, the mean bending strength of the material from full-scale testing is actually around 31-36N/mm^2.[22] UK Sitka spruce is graded at C16 instead of a higher strength class of C18 or even C20 because of its limitations in stiffness or MoE: the wood's mean MoE value is about 8.2kN/mm^2, corresponding closely to the 8kN/mm^2 MoE value of the C16 strength class.[23] As the MoE of wood increases with decreasing moisture content, however, a reduction from 20% to 12% moisture content would result in an estimated 8% increase in MoE in bending.[24] After being dried to 12% moisture content, a simple estimate for UK Sitka spruce predicts that the wood could nearly meet the 9kN/mm^2 requirement for the C18 strength class; the wood's mean MoE value would correspond to about 8.9kN/mm^2.

The previous estimate illustrates how strength classes and grading processes performed in industry are not meant to characterise a material, but are actually a process of classification. Characterising the numerous material properties relevant for design of each individual piece of dimensioned timber produced in industry is infeasible, so grading provides a simple and effective means to classify and group similar pieces of timber together based on the estimates of a limited number of properties such as density and stiffness. Ridley-Ellis[25] has discussed this distinction in grading in more detail, showing that if one assumes a reasonable coefficient of variation (CoV) for wood of 0.2, the numerous C-series strength classes overlap one another significantly.[26] The overlap shown in Figure 1.5 means that while a large collection or bundle of graded timber may closely follow a strength class distribution, a significant number of individual pieces in that bundle could also be included in higher strength classes. Finally, as designers use characteristic or 5th-percentile values rather than mean values from strength classes for strength and ultimate limit state (ULS) design calculations, the inherent variability of wood introduces additional redundancy in a design for safety purposes.

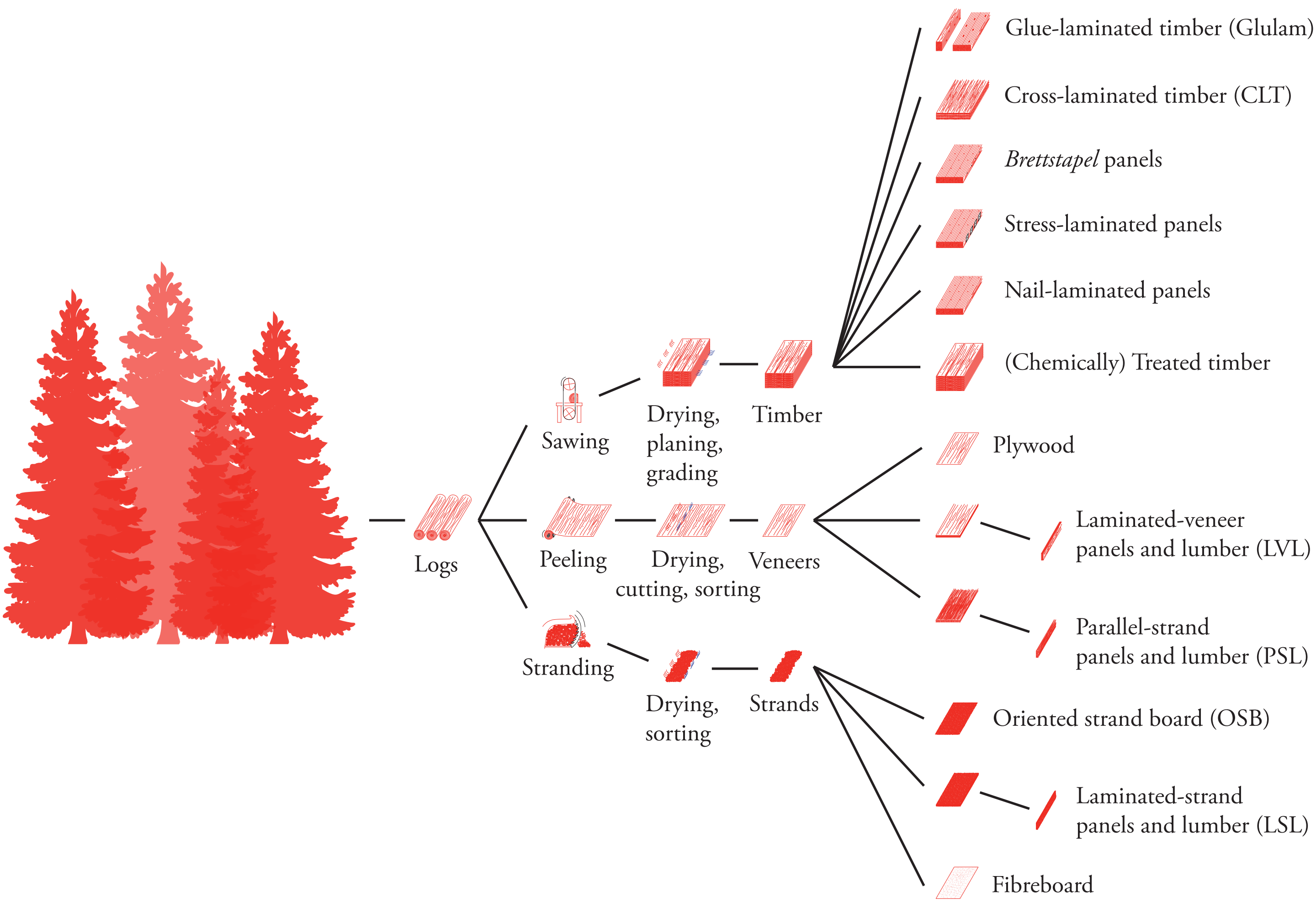
Figure 1.5: Approximate distributions for the C-series strength classes.
(Adapted from Ridley-Ellis)



Value-added wood products

Figure 1.6 (opposite): Value-added processes for engineered wood products used in construction.

After the primary processes of sawing, drying, and grading timber, various secondary processes for adding value and forming massive, solid wood elements for construction are illustrated in Figure 1.6. In addition to secondary processes for timber, alternative processes with veneers from peeling and strands from stranding are also shown. All wood products used in construction, with the exception of whole-log construction, pass through one of these three kinds of primary separation processes. Compared to sawing logs, however, peeling and stranding have a key advantage in that drying thin veneers and small strands is far more straightforward than timber. Peeling and stranding also allow more wood to be processed from logs, thereby reducing waste. In the UK, these two alternative primary processes are performed in some mills but lead to panel products like orientated strand board (OSB) for I-joists and walls in domestic construction. Large-scale engineered wood products like LVL or parallel-strand lumber (PSL), or laminated-strand lumber (LSL) still need to be imported into the UK, just like glulam or CLT produced from dried and graded timber. All of these lamination processes add value, however, by allowing large-scale elements to be formed from smaller pieces of timber, veneers, or strands. Furthermore, the natural defects of wood can be more evenly distributed, allowing increases in stiffness and strength, and reductions in variability due to the lamination process itself. Figure 1.7 illustrates how the typical strength distribution of timber can be shifted to a higher mean, but also narrowed with less variability with a higher characteristic value by using that timber in glulam. Just as drying is the common feature following and completing the three primary processes of sawing, peeling, or stranding, the general feature common to all secondary processes is lamination, either with or without glue, and sometimes in combination with chemical treatments but usually just with natural, untreated wood.



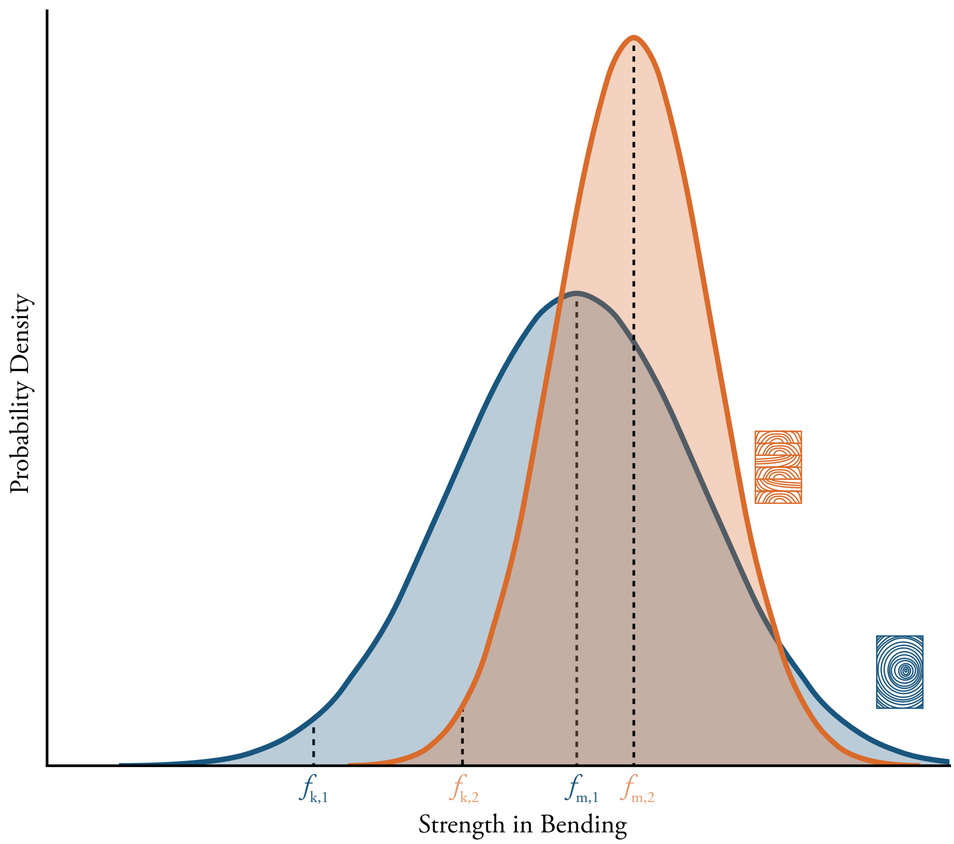


Figure 1.7: Typical strength distributions for solid timber and glulam. (Source: Nordic Glulam Handbook)

Two ways of making better use of low-grade UK wood is to establish alternative methods and techniques for drying Sitka spruce to 12% or even 15% moisture content without significant warping, or to investigate alternatives to glue-based lamination. In the former case, providing UK Sitka spruce could be successfully dried to 12% moisture content, a number of secondary processes for gluing and laminating massive engineered wood products like glulam and CLT would then be feasible for UK industry. Concurrent research in the UK is trying to address this issue of drying Sitka spruce to 12% moisture content to show the feasibility of CLT and *Brettstapel* (dowel-laminated panel) production. Researchers in Edinburgh Napier University[27,28] have claimed that simply readjusting kiln schedules with more suitable temperature and humidity levels can allow UK-grown Sitka spruce to be successfully dried to 12% moisture content without significant warping or rejection rates. They have also conducted extensive structural testing of prototype CLT and *Brettstapel* panels made with UK-grown material. In the case of CLT, their results show only minor differences in overall structural performance from using C16 instead of C24 material in the panels.[29] When speaking with representatives from industry on different occasions, these claims regarding kiln drying have been explicitly refuted, with explanations that kiln schedules are already optimised to minimise rejection rates.[30] In the case of the BSW Newbridge sawmill in Wales, which supplied the majority of the material used later in this work, the operation produces approximately 150,000m^3 of sawn UK timber annually. With such large volumes of wood and the potential economic losses involved, it is very unlikely that kiln schedules are not already carefully optimised. One possible source for this discrepancy may be that the Napier research has dried UK wood in a small humidifier type kiln.[31] The difference in scale between the larger kilns used by industry and the small kiln used in the Napier research program may be in part the cause for the difference of opinion between researchers and industry representatives. These two areas of kiln drying low-grade wood and finding alternatives to glue-based lamination are important topics for future research, where the latter will be discussed in greater detail later on.

Summary

There are significant challenges in designing and building with low-grade wood, but in the particular context of the UK, there are also strong incentives and opportunities to meet these challenges and overcome them. Most existing types of secondary processes for making large-scale building elements like glulam, CLT, or LVL are either not well established or entirely absent in the UK wood industry. Additional research on drying fast-growing, low-grade wood without causing significant dimensional changes or warping is needed. Furthermore, secondary processes for wood lamination without glue can also be developed or adapted from existing techniques, and would be useful in the UK and in many other contexts and countries. Compared to other European countries like Germany, Austria, Finland, or Switzerland, however, the UK has fewer forests that yield lower-grade wood, but the lack of diversity in the UK wood industry and UK wood culture in general is far more problematic. Establishing a UK CLT production with UK-grown Sitka spruce is an important goal in the future, but consideration of additional techniques and strategies for adding value to low-grade wood is also necessary.

Designing and building taller with wood

In parallel to UK forestry developments and the predicted increase in available UK Sitka spruce, there is also global shift to designing and building taller with wood. Steel and reinforced concrete are the conventional structural materials used almost exclusively in large scale, multi-storey commercial construction. Engineered wood products like glulam, LVL, and CLT have only recently been considered as viable options. Compared to conventional timber frame construction associated with domestic construction, engineered wood products can now be specified in massive dimensions for resisting considerable loads, taking full advantage of the general benefits of lamination already mentioned. From a design perspective, their large and solid character also provides a surprising degree of fire resistance due to the slow and predictable charring rate of softwoods subject to fire. Furthermore, the limited number of 'tall' timber buildings ranging from 7-10 storeys that have been built during the past six years have aggressively exploited off-site prefabrication, resulting in noticeable savings in construction time and cost.[32] Architects and engineers in the UK are playing an active role in this ongoing development. From 2009 to 2012, the nine-storey Murray Grove Stadthaus residential building in Hackney, London, was claimed to be the world's tallest residential timber building. Figure 1.8 shows an exterior view of the building. The project features a reinforced concrete ground floor structure, but the upper eight storeys are build exclusively from imported softwood in the form of CLT panels.[33] Along with the expected increase in the UK's forestry output, there is also a genuine and growing interest in building taller and faster with wood in the UK. Without a means to use UK-grown wood, tall wood construction in the UK will remain dependent on imported products.

Figure 1.8: Murray Grove Stadthaus by Waugh Thistleton Architects, 2009, London, UK. (Photographer: Will Price)

There are a number of familiar and traditional examples of constructing tall buildings with wood. Images of Norwegian Stave churches and Japanese temples and castles, like those in Figures 1.9 and 1.10, are the typical traditional building types that come to mind. Langenbach[34] has discussed and reviewed the fire protection strategies of various tall traditional wood buildings, especially 'slow-burning' heavy timber construction in nineteenth-century mills in America. These examples provide important lessons on fire resistance and protection for future developments, but also a warning for those wanting to build tall wood buildings beyond the height and reach of modern fire fighting ladders and equipment. At the same time as providing valuable lessons, these examples also highlight how the current developing trend in using engineered wood products for modern multi-storey construction is unprecedented. A brief review of current developments in tall wood buildings is therefore needed before proceeding to more detailed discussions of focused research. Accordingly, the following section summarises and expands upon two published works on tall wood buildings: the first by Fleming, Ramage, and Smith[35] from early 2014, focusing on design issues; and the second, by Perkins+Will[36] who completed a survey of tall wood buildings in 2014. The latter report focuses on lessons for delivering projects with input from different stakeholders, including architects, developers, planning authorities, and builders. In the relatively short time since these works were published, several new projects have either been announced in competitions or have already started construction, forming an apparent race to design and build the tallest timber building. A brief review and discussion of the current situation is merited, first to show the short sightedness of such a race, and secondly, to shift attention to more relevant architectural and structural issues that can help inform future research and design work.

Figure 1.9: Borgund Stave Church, ca. 1150-1180, Laerdal, Norway. (Source: Figure 267 in Lübke, Wilhelm, *Outlines of the History of Art*, New York: Dodd, Mead, and Company, Vol. 1, 1881)

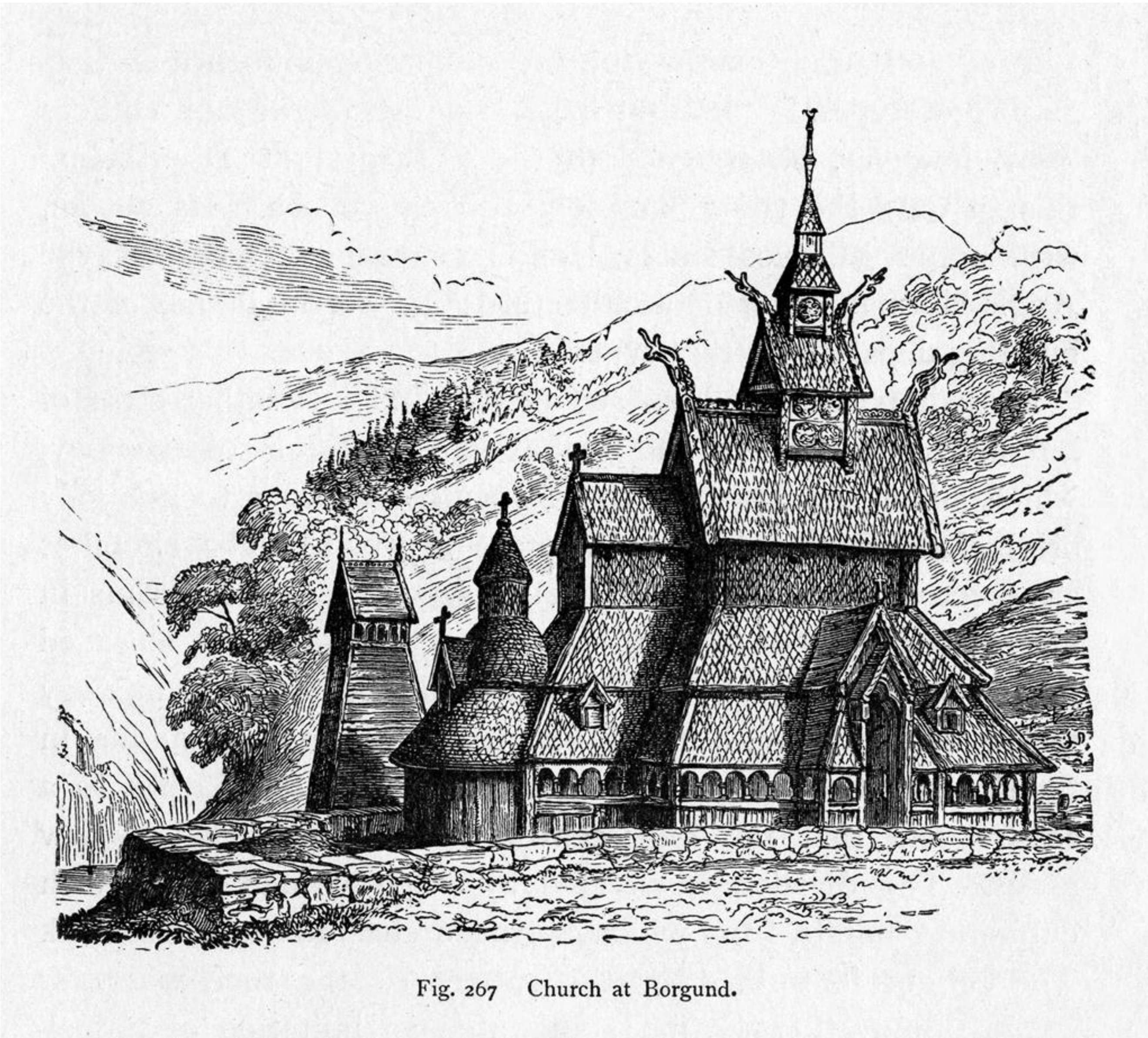


Figure 1.10 (opposite): Kiyomizu Temple in the Snow, woodblock print by Hasui Kawase, 1932. (Source: Artelino Archive)

Measuring up in timber

Fleming, Smith, and Ramage[37] have already reviewed and constructively criticised the design of a number tall timber buildings and proposals. Projects designed by Waugh Thistleton Architects, Fleming, Ramage, and Smith, Architekten Hermann Kaufmann ZT GmbH, Michael Green, Skidmore, Owings, and Merrill LLP (SOM), and Shigeru Ban Architects were included in the review. At the time of writing the article in 2013, in both cases of practice and research, most projects demonstrated a straightforward way of thinking about engineered wood products simply as a replacement for steel and concrete. The common argument for building with wood based on climate change and the carbon sequestration effect of growing trees was also briefly questioned as being oversimplified.[38] The authors argued that wood has far more potential in multi-storey buildings than simply mirroring the design of regular steel and concrete buildings with engineered wood products. Massive engineered wood products and future developments in timber engineering can provide a new material and new elements for designers to engage with in tall buildings, alongside issues of structure, space, and tectonics, or even broader architectural and urban issues such as anonymity, monumentality, and flexibility. Instead of simply measuring up in height, architects and engineers can engage with new ways and variations of working with wood for mid- and high-rise buildings, without neglecting architectural and urban issues.

One early project that was not included in Fleming, Smith, and Ramage's discussion was the E3 building in Berlin, completed in 2008 by Kaden Klingbeil Architekten. Upon visiting the building in person, the author quickly realised how important qualities of the project are difficult to understand in published images such as Figure 1.11. Like many other earlier tall timber buildings, the E3 building does not outwardly convey its wood structure or unique qualities; the architects felt they could not give any significant presence to the building's timber structure within its older urban setting in Berlin.[39] The project's separate concrete core satisfied the strict if not excessive fire protection requirements of authorities, but also encouraged an opposition between concrete and wood construction. The concrete is exhibited as safe and durable, separate and distinct from the main hidden wood structure of solid glulam beams and columns and *Brettstapel* floor panels. Seeing this project in person, the separation and its benefits, beyond fire issues, is easier to understand in relation to the immediate urban context of Berlin's perimeter blocks and interior courtyards. The separation establishes a unique connection between the street and the normally hidden courtyard behind the building. The open concrete core is also a mediating space between the private interiors of the apartments and the public urban street outside. As an alternative to typical enclosed and blank corridors and stairways outside of apartments, the architects intended for occupants to be strongly connected with the outer city in the open concrete core as they ascend and arrive to their own apartment door. This architectural intention and experience is strongly influenced by the building's fundamental structural design. This kind of engagement with fundamental architectural and urban issues is not outside the area of structural design, and can serve to enrich a design project and structure with purpose and meaning. More engagement is needed with these kinds of fundamental architectural and urban issues, in parallel with thinking about new ways of building taller and working with wood.

Figure 1.11: E3 building by Kaden Klingbeil Architekten, 2008, Berlin, Germany. (Photographer: Bernd Boracht)

Survey of international tall wood buildings

Perkins+Will's recent survey on tall wood buildings does not focus on these kinds of design issues in detail. The survey is nonetheless an important document, summarising valuable lessons on delivering tall timber buildings from design to completion. A number of ways of meeting codes and especially fire regulations are highlighted in the survey, either through combining different fire strategies, or even in a more straightforward way by conducting fire tests with research partners.[40] The report also emphasises how multi-storey wood buildings have been successfully completed in a range of contexts, such as Scandinavia, central and southern Europe, including earthquake prone areas in Italy. The UK and North America are also included and discussed. The administration side of getting tall wood buildings through planning and actually built has advanced even in the short timeframe of five years, from 2009 to 2014. Gaining experience from this first wave of tall wood building projects is therefore an important step forward for supporting and encouraging more challenging designs in the future. The long-term success of this first generation of tall wood buildings will also encourage and promote timber buildings in the future.

There are a few surprising omissions in Perkins+Will's survey, perhaps due to publication deadlines and time constraints. The Wood Innovation and Design Centre (WIDC) by Michael Green Architecture, with its construction started in the fall of 2013 and completed by spring 2014 in Prince George, Canada, was the first project notably missing from the survey. The building, shown in Figure 1.12(a), claims to be the most innovative and tallest contemporary wood building in North America.[41] The building is comprised of a basic glulam structure with a CLT core and a unique floor section also made of CLT panels. The CLT floor panels are lapped or staggered, as shown in Figure 1.12(b), to provide space for services, but still function as basic one-way spanning elements over the glulam beams. The use of CLT, which is a two-way floor spanning element, for basic one-way spans is simple and common practice today but also inefficient. In the early twentieth century, concrete construction also displayed the same tendencies before more innovative and efficient two-way spanning plate systems with point supports were developed. Here the WIDC's floor depth is effectively made up of an eight-layer CLT section, in addition to the depth of the glulam beams. Such a bulky structural design would be undesirable for taller buildings constructed in any material, where floor depth is usually strictly controlled and minimised for economy. Langenbach would also challenge the WIDC's promotion as the tallest timber building in North America, as the WDIC stands only 27m tall, noticeably lower than the 36m ten-storey timber structure of the Claremont Hotel in Oakland, California, built in 1906.[42] With a focus on floor construction innovation with CLT rather than overall design innovation, the WIDC would have been well suited for Perkins+Will's survey.

Figure 1.12(a) left and (b) right: Exterior and floor assembly diagram of the Wood Innovation and Design Centre by Michael Green Architecture, 2014, Prince George, Canada. (Source: BC Forestry Innovation Investment, and Michael Green Architecture)

The other surprising omission in Perkins+Will's survey is the Illwerke Zentrum Montafon (IZM) five-storey building in Vandans, Austria by Architekten Hermann Kaufmann ZT GmbH, also completed in 2014.[43] The IZM building uses the same modular structural system developed by CREE Rhomberg and demonstrated in its eight-storey LifeCycle Tower One (LCT One) building. The modular structural system used in both projects is made up of a limited number of basic structural components of timber columns and prefabricated wall sections, and a prefabricated timber-concrete hybrid floor panel, shown in Figure 1.13. Unlike the IZM building, the LCT One is included and discussed in Perkins+Will's survey. Apart from more careful and superior detailing on the IZM building's façade, the newer project offers little else in either design or construction when compared to the LCT One. Due to the simplicity of the structural system and clever floor and wall connection detailing, incredibly fast construction and erection speeds can be achieved, up to around one storey per day in the case of the LCT One. This modular structural system offers speed and cost savings during construction, but also limits the possible results. These are buildings that are well made and quickly constructed, but designed primarily to meet the demands of construction. The modular structural system is poised as a universal technology that can be applied to any site or surroundings, but with limited and generic results.

Figure 1.13(a) above and (b) below: Timber-concrete floor elements during assembly of the LifeCycle Tower One and Illwerke Zentrum Montafon buildings by Architekten Hermann Kaufmann ZT GmbH, 2012 and 2014, Dornbirn and Vandans, Austria. (Photographer: Darko Todorovic)

Structural and architectural challenges and potential

Another tall timber project currently under construction is the Treet building in Bergens, Norway. Upon completion, as envisioned in Figure 1.14, the fourteen-storey project is set to become the world's tallest timber building. The project was most likely designed and built for capturing this title and holding it, at least for a while. Compared to earlier tall timber precedents, however, the Treet building is perhaps the most structurally ambitious project to date, and not just in terms of height, but also in its design. Two other projects that display a simultaneous engagement with structural precedents, theory, and spatial issues are competition entries by C.F. Møller and Dinnell Johansson, and CEI Architecture, shown in Figures 1.15 and 1.16, respectively. These two projects are rather speculative proposals, with the former focusing on interior and transparency issues alongside the construction and ground condition of tall timber buildings. The latter project was submitted for the Office of the Future competition in 2012, and features alternating floors with diagonal bracing, giving structural action over the whole floor-to-ceiling height. This latter structural design with prominent diagonal bracing allows every other storey to be almost structure-free and open, applying the principles outlined in Myron Goldsmith's 1953 Master's thesis written under the supervision of Mies van der Rohe.[44] The Treet building follows a different structural strategy, and attempts to reinterpret Fazlur Khan's well-known trussed tube high-rise structural design in wood. Following on from developments and principles for high-rise design in steel and concrete is important, but much more work is needed. The key issue here is taking precedents from Goldsmith or Khan or others as the background for engaging with new ways of working with wood, rather than taking them as the design goal or point of completion.

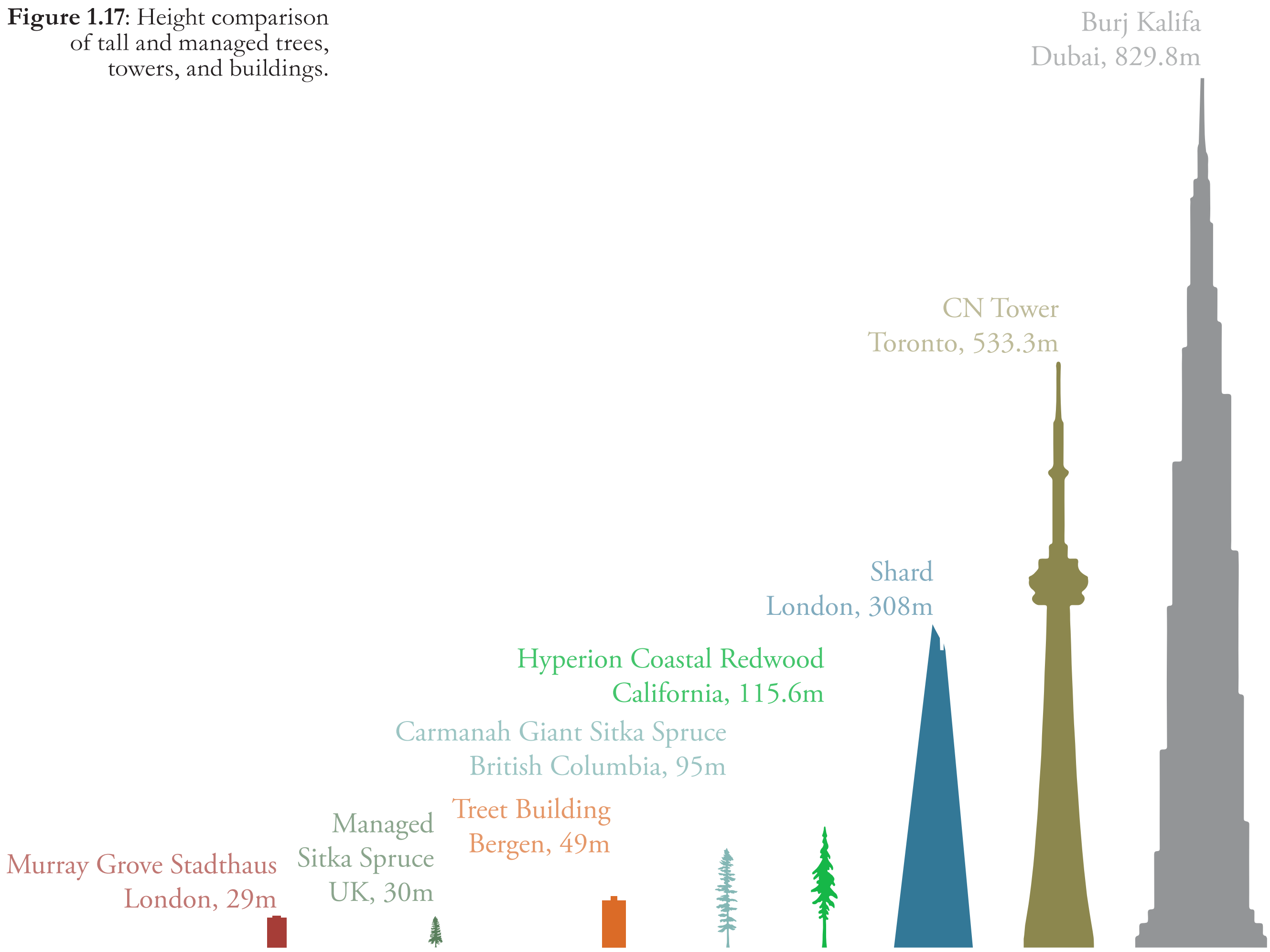
Figure 1.14:Rendering of the Treet building by Sweco AB, 2015, Bergen, Norway. (Source: Council on Tall Buildings and Urban Habitat)

Figure 1.15: Wooden skyscraper design proposal by C.F. Møller and Dinnell Johansson for the HSB Stockholm architectural competition, Stockholm, 2013. (Source: C.F. Møller and DinnellJohansson)

Figure 1.16: 40-storey wood office tower competition entry for An Office Building of the Future by CEI Architecture, 2012. (Source: CEI Architecture)

There is already at least one project currently being planned to quickly take over the title of the world's tallest timber building. At the time of writing, an 84m twenty-four storey timber tower in Austria has just been announced for Vienna, though with little detail or information.[45] Such announcements that emphasise the possible height of new timber construction provides more evidence as to how designers and the general public alike are preoccupied with the novelty of building tall with wood. In North America, the University of British Columbia's (UBC) eighteen-storey Brock Commons timber building is already in its design phase now for a site in Vancouver, Canada.[46] This project is only at an early design stage, but the various preliminary options emphasise how designers and clients are focused on the issue of height. The basic massing of these UBC proposals bears a curious resemblance to the WIDC in Prince George; the projects are similar in that the driving force behind their design and realisation is simply height and demonstrating construction leadership rather than design leadership. While the Treet or Brock Commons buildings may become the tallest timber structures in the world for a time, their 'tall' height of about 50-60m will always be dwarfed by previous and especially future achievements in steel and concrete. Figure 1.17 shows the tall trees and timber building illustrated earlier in Figure 1.4, alongside the dominant heights of current tall steel and concrete structures. During a presentation at the Alvar Aalto International Meeting on Modern Architecture in 2011, however, architect Claude Armstrong proposed an interesting way to think about this challenge of height in building tall with alternative materials, saying "it's not how tall you make it, but how you make it tall."[47] Chasing and even achieving the title of the tallest timber building is therefore a somewhat marginal position compared to the achievements in height already made in steel and concrete. If height is truly a genuine architectural concern and challenge, then architects and engineers can work to develop designs that convey a quality of height or 'tallness', without necessarily being the world's tallest timber structure.

Figure 1.17: Height comparison of tall and managed trees, towers, and buildings.



The basic adjustment of a building's massing is one of many ways for designers to articulate height and especially slenderness in tall timber buildings. Another more open architectural issue and challenge related to height is monumentality, or the quality of being monumental. There are countless examples of monumental skyscrapers already completed in steel and concrete, but apparently little engagement with this notion in tall wood buildings so far. Although monumentality is most often associated with size and scale, a clearer understanding of the term comes when considering monumental designs as memorable, or being related to memory and time. The 300 and 400-year-old 'giant' Sitka spruce trees, mentioned earlier in this section, arguably exemplify a quality of monumentality due to their age and not necessarily their size. When considering this aspect of time, monumentality also becomes an important design issue for tall wood buildings because of an implied association with durability. Monumental buildings, like monuments, are often durable, valued, and ultimately last over time, and are not simply tall or large investment commodities or instruments.[48,49] This issue of monumentality is particularly relevant in the context of the UK, where perhaps some of the most monumental spaces and structures have already been built with wood, as illustrated in Figure 1.18. Unlike steel, concrete, and masonry buildings, tall wood buildings have yet to engage with monumentality. There is immense potential for designers to do so, thereby engaging and grounding a new future in tall wood construction with a deeper tradition of building with wood. In the first instance, designers can move beyond standardised engineered wood products or basic modular construction. Research with a focus on the fundamental material properties of wood and alternative forms of lamination is relevant in this regard, and can directly expand upon the previous work already accomplished in standard engineered wood products like glulam and CLT.

Figure 1.18 (opposite):
Westminster Hall, colour etching and aquatint by Thomas Rowlandson.
(Source: Ackland Art Museum, The University of North Carolina at Chapel Hill Ackland Fund)


WESTMINSTER HALL.

Summary

There is a global shift towards building taller and bigger with wood, and some pioneering projects have been already completed in the UK. These buildings along with others worldwide confirm that engineered wood can easily compete with steel and concrete in multi-storey commercial construction. Those involved in this first wave of tall timber buildings and design proposals have taken a narrow and perhaps limited approach to architectural and structural design. Valuable lessons have still been learnt through the process of constructing projects and these lessons have been documented through international surveys. Preliminary buildings and the experience gained from their realisation will play a useful role as precedents for supporting more ambitious designs in the future. Rather than focusing on the novelty of height, or mirroring the design of steel and concrete buildings, designers of tall timber buildings need to engage with their material, alongside broader architectural and urban issues and in parallel with basic concerns of structure, fire, and construction. Wood and engineered wood products are fundamentally different than steel and concrete, not only in mechanical and structural ways, but also in their potential to articulate architectural ideas and concepts. Taking wood outside of and beyond the long shadow of tall steel and concrete buildings is a viable and promising future.

Thesis overview

Introduction to wood modification

Against this general background of developments in UK forestry and tall wood buildings, this thesis focuses on two strategies for working with low-grade wood for multi-storey construction: first, wood modification through impregnation is considered as a direct way of modifying and improving the properties of wood for enhanced applications and adding value to the material. This work on wood modification originally began as part of a larger research group at the University of Cambridge, considering new ways of treating wood for improving its mechanical properties. The group of chemists, material scientists, engineers, and architects formed around a common vision for developing new treatments and processing techniques that could modify and enhance the properties of low-grade wood directly at the molecular level. Like the development of CLT or the modular structural system of CREE Rhomberg, creating a new treatment process that can be applied universally to low-grade wood represents an avant-garde position. Developing such techniques requires close collaboration, and a wide array of testing methods at various scales, from molecules to full-scale structures.

The broad research interests in the University of Cambridge are well suited to such a project. Compared to other research groups focused on only one aspect of wood modification, such as chemical interactions, fluid or composite mechanics, a group interested in and working across all the scales of wood can develop unique collaborations and offer new perspectives.

As a secondary processing technique, wood modification and treatments in a broad sense can take advantage of the natural and efficient mechanical properties of wood, but also adjust and tune these properties by various means. For example, the heat treatment of wood, usually in combination with controlled pressure and compression, can enhance dimensional stability and resistance to moisture and decay. Heat treatments usually result in a densified wood product with a dark colour, but some degradation of the wood structure also arises, in turn causing a decrease in strength.[50] A number of existing heat treatments and heat-treated wood products like ThermoWood and PlatoWood, have also been developed to a commercial scale.[51] Alternatively, chemical wood modification involves immersing timber in a chemical solution that reacts with the molecules in the wood's cell wall and directly modifies its material properties. Like heat treatments, chemical treatments are generally used to improve the dimensional stability of wood and especially fungal, insect, and decay resistance. Unlike heat treatments, however, chemical modification in many cases does not reduce strength, but in some cases can actually yield minor increases in strength and MoE.[52] If toxic chemical agents are used in the treatment process, as was commonly done in the twentieth century, the service conditions and disposal of the resultant chemically modified wood product become vital considerations in the design process. Today, many commercial chemical modification techniques and processes have been adapted to use chemically benign agents, therefore avoiding these critical service life and disposal issues. Accoya, based on acetylation or chemical modification with a mild acetyl acid, is produced commercially and provides an example of a safe, non-toxic, but durable chemically modified wood product.[53]

Wood modification by polymer impregnation combines the beneficial results of heat and chemical treatment techniques, and is also relatively unexplored and much less developed commercially.[54] Impregnation is similar to chemical modification, as wood needs be immersed under pressure in a liquid solution usually involving a swelling agent, monomers, a catalyst, and a cross-linking agent. Impregnation differs from basic chemical modification, in that once the liquid solution has been taken up in the wood, the monomers can be polymerised with the application of heat, microwaves, or x-rays, to form chains of solid polymers throughout the wood.[55] Instead of simply filling the voids in the microstructure of wood, an interaction is desired between the new polymers from the treatment solution and the naturally occurring polymers in the wood cell wall.[56] The monomers and cross-linking agents in the solution are important for this interaction, while the swelling agents help increase the porosity of the wood's cell wall, thereby allowing the liquid monomers to diffuse directly into the cell wall.

The work on wood modification and treatment in this thesis focuses on polymer impregnation due to the technique's numerous potential benefits: enhanced dimensional stability, decay resistance, and potential for increases in mechanical properties like MoE and compression strength. These gains come at the price of increasing mass and density of the resultant wood-polymer composite. Furthermore, the polymer impregnation of wood is also considerably more complex than heat or chemical treatments, and requires specialised skills and knowledge outside the areas of architecture and structural engineering. The author has therefore performed mechanical testing of natural and polymerised wood samples according to wood testing standards. The preparation of the monomers and liquid solution, the vacuum-based impregnation process, and the final polymerisation stages have been performed by Drs. Emma-Rose Janeček and Zarah Walsh, under the supervision of Dr. Oren Scherman in the Melville Laboratory, in the Department of Chemistry at the University of Cambridge.

Introduction to stress lamination

In contrast to wood modification, the second strategy examined in this thesis for adding value to low-grade wood for large-scale construction is stress lamination. If polymer impregnation is an avant-garde technique, requiring specialised knowledge and a variety of equipment and advanced technology, stress lamination is definitely a '*derrière-garde*' position. Instead of relying on glue for lamination, stress lamination uses simple tools and construction methods for prestressing wood. Prestressing results in large friction forces and composite action between adjacent laminate layers or lamellas, so the different layers can effectively work together and transfer shear forces while resisting actions or loads. The technique is also highly relevant for UK Sitka spruce as stress lamination was originally developed for use with wood at maximum moisture content of 19%, although like glulam, matching the moisture content at the time of construction to that expected in service is still recommended.[57] An early stress lamination project, however, reports on the successful use of wood at even 23-30% moisture content.[58] Current design standards for timber bridges are more conservative and recommend wood with a maximum moisture content of 16% for stress lamination.[59] Compared to glue-based lamination techniques, stress lamination provides more flexibility in terms of moisture content because of its mechanical and adjustable nature.

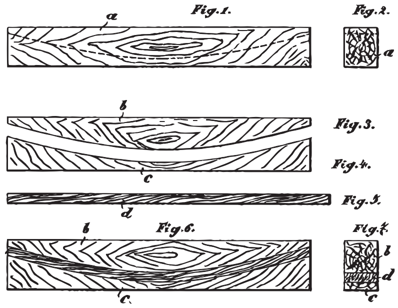
Rather than emerging from the wood industry, stress lamination was developed in the field by engineers and tradesmen in 1976 to repair older types of nail-laminated bridge decks in Ontario, Canada. Engineers quickly realised, however, that the method could be highly effective for making new panels and structures in addition to repairing older ones. Stress lamination then developed rapidly, and by the early 1980s, the Ontario Highway Bridge Design Code had been updated with provisions and design guidelines for stress-laminated bridges.[60] By the late 1980s, with the need to either repair or replace approximately 70,000 'functionally obsolete' highway bridges in the United States, American engineers soon adopted the technique along with developing codes of their own.[61] By the 1990s, the stress-lamination technique had subsequently spread to Switzerland and then other European countries, and was later included with design guidelines and procedures outlined in Eurocode 5 (EC5), in British standard (BS) and European National (EN) standard 1995-2:2004.[62] Due to its simplicity and effectiveness, the stress-lamination technique has now been adopted throughout the world and is still the subject of recent research based in Norway,[63] Sweden,[64] and Brazil.[65]

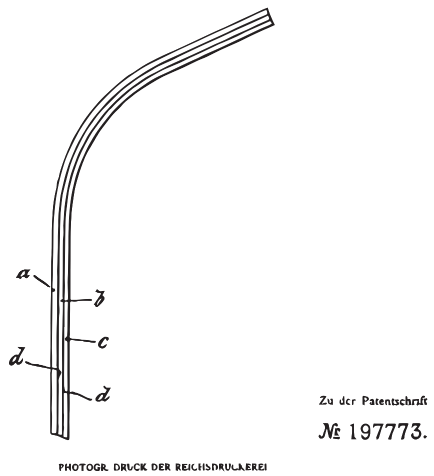
Despite the success and widespread use of stress lamination for timber bridge design, relatively little research effort has been made in extending the technique's basic method and principles. A short study and limited number of built projects by Freedman and Kermani[66] has started to address this issue by showing how stress lamination can be used for designing arched bridges instead of basic flat decks. Experiments by Awaludin *et al.*[67,68] and Quenneville and van Dalen[69] have also confirmed that prestressed connections, just like those in a stress-laminated bridge deck, can increase the performance of bolted moment-connections and split-ring connectors, respectively. Johansen's original 1949 seminal study, 'Theory of Timber Connections' also included several tests with prestressed bolts to examine the effects of friction alongside his now well-known yield models.[70] Johansen showed that simply prestressing a bolted joint can easily improve its load capacity by about 30%, but he concluded that such gains from friction effects should be ignored due to the possibility of the wood shrinking with changes in moisture content. In parallel to additional research efforts to investigate how stress lamination and prestressing can be applied in timber building design and connections, the understanding and control of prestress losses with time is an important issue and open to further development.

Otto Hetzer and the development of glulam

A dual approach of examining polymer impregnation and lamination techniques together is not commonly done in wood research today but may offer notable benefits. These two techniques and general areas of treatment and lamination are not mutually exclusive, but are complementary and can be combined in a straightforward manner. For example, Otto Hetzer's classic German patent (DRP 197773) from 1906 for a curved glue-laminated roof element, shown in Figure 1.19(b), is often regarded as the catalyst for today's global timber glulam industry. Hetzer's patent describes a basic glulam frame structure, while the straight elements in Hetzer's frame, in essence, are the standard types of solid glulam beams and glulam columns still widely used in current construction. Less known though is Hetzer's earlier 1903 patent (DRP 163144), for what he intially considered to be a true glue-laminated beam. Figure 1.19(a) shows how in this earlier beam configuration, rather than using straight horizontal lamellas, a parabolic camber was cut along the length of a solid rectangular section. A chemically-impregnated strip of timber was then inserted in between the two cut sections before gluing them back together. According to research by Müller[71], when Hetzer first proposed and patented this idea, testing showed noticeable improvements in structural performance, with reductions in deflections up to 60%. The basic testing, however, was unable to discern if these improvements were due to Hetzer's secretive processing and chemical treatment of the inserted strip, or the simple cutting and lamination process alone. While many current wood research programs tend to focus on either one or the other of these areas, Hetzer's dual approach of combining wood modification and novel mechanical lamination together is still one that is relevant for research today.

Following the enormous success and widespread use of the 1906 patent for curved glulam roof elements, Hetzer actually continued to investigate alternatives to his glulam technique. For example, in 1907 he filed another patent (DRP 225687) for designs of various trusses, lattice arches, and their connections in roofs, as seen in Figure 1.19(c). Searching for alternatives to the now well-established ways of producing large-scale timber elements, including glulam, CLT, or LVL, remains an important issue. Timber engineers and researchers may sometimes even take for granted that finger-jointed glulam beams and columns are not produced or even readily available in many places in the world. The present thesis therefore aims to explore alternatives to standard modern engineered wood products like glulam and CLT. This work focuses solely on low-grade wood, assessing the performance and building potential of new alternatives. Rather than seeking to replace today's well-established and thoroughly researched engineered wood products, the thesis examines new techniques for modifying, joining and working with low-grade wood, while broadening the scope of multi-storey wood construction.





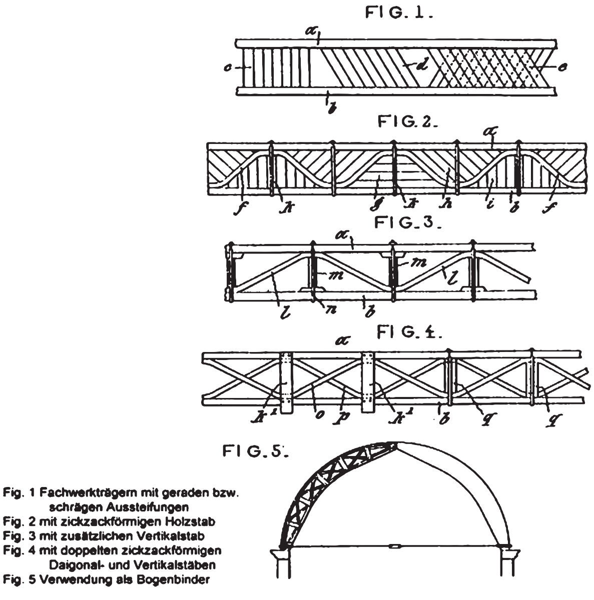


Figure 1.19(a) above, (b) middle, and (c) below: Patent illustrations by Otto Hetzer for (a) a parabolic composite beam in 1903, (b) a bent wood component with laminated construction in 1906, and (c) trusses made of wood in 1907.

Scope, themes, and structure

In practice, building with wood is influenced by structural concerns, but also specific fire and acoustical considerations. In this thesis only mechanical and structural issues will be addressed, as special fire testing facilities are not only limited in availability, but also are only needed after a structure's performance and behaviour have been tested and confirmed. A structure's minimum fire resistance also needs to be taken into account with other design issues: interior finishes and construction tolerances, centralised fire safety systems like alarms and sprinklers, and open-air escape routes on the outside of the building can all be means to limit the risk of fire and mitigate the loss of life due to fire. Similarly, in addition to structural and fire considerations, acoustical performance and testing is also dependent on interior finishes and especially construction quality and care taken on-site. The issue of making better use of low-grade wood in large-scale buildings, in the first instance, is a structural issue rather than a fire or acoustical concern.

Four general themes have emerged in this introductory chapter and can be seen as threads running through this thesis: first, the growth and associated material characteristics of wood; second, the harvesting and processing of wood, including primary processes like sawing and kiln drying and secondary processing for making products for specific uses in buildings; third, how the properties and characteristics of these elements are considered in a design process; and finally, the act of building with wood. Each of these themes will be expanded upon in greater detail in the following chapters, and especially the relationships and dependencies between the four themes. Throughout the course of this study, the author has encountered on more than one occasion practitioners and designers working with wood but lacking a firm grasp of its physical structure and properties. Working with wood without an understanding of its physical structure at different scales can also lead to a misunderstanding of wood's behaviour in buildings and especially structural details. For example, without a firm understanding of the microstructure of wood, is it difficult to explain even the material's most fundamental mechanical behaviour, such as its orthotropic properties with respect to the angle of the grain or cells, or the influence of moisture on such properties. Chapter two of this thesis therefore provides background information on the basic physical structure of wood, along with a discussion on general reasons for its use in buildings due to its embodied qualities. The term 'embodied qualities' is intentionally used to contrast against current discussions and environmental debates centred on the embodied quantities of wood, especially embodied energy and equivalent carbon dioxide emissions and sequestration effects. A review of literature from research, practice, and policy are used to highlight the shortcomings of these embodied quantities and those who rely on them for support.

Chapter three discusses the common methods and testing used throughout the thesis, and then focuses on more specific issues in wood modification by polymer impregnation and stress lamination. Following a brief review of current literature relating to polymer impregnation, the preliminary results of small-scale testing of natural and polymer-impregnated wood samples are presented and discussed. The established practice of stress lamination is then discussed in detail, with small-scale models and testing of stress-laminated models to gauge structural performance and feasibility before moving up in scale.

Due to limitations in laboratory equipment and general scale effects in the impregnation process, testing of polymer-impregnated wood in full-scale structures could not be performed and is left for future work. Full-scale detailing, construction, and structural testing of stress-laminated columns are presented in chapter four. With the remaining focus of the thesis on stress lamination, discussion is given to how to adapt the bulky prestressing elements used in conventional stress-laminated bridge design to buildings. Full-scale test results of connections and stress-laminated columns are discussed and compared to estimates and current design codes. The development of codes and theory for timber column buckling are reviewed to ensure rigour in the previous comparisons and to establish confidence in structural performance.

Based on the experience gained from full-scale construction and testing, an updated stress-lamination detail with UK-grown hardwood is proposed and tested in chapter five. Additional full-scale construction and testing of columns with different proportions provide more experimental data and demonstrate improvements in structural performance. Different strategies for controlling prestress losses in stress-laminated elements are also considered. Using overdried hardwood in the new connection detail offers a novel method to mitigate prestress losses, thereby ensuring long-term structural reliability and robustness. Logging the prestress levels in separate connection details over a period of six months provides physical evidence to support these claims.

The sixth and final chapter concludes the thesis with a summary of the major findings and original contributions, and also offers recommendations and suggestions for future work.

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Chapter 2 BACKGROUND

Growth characteristics and physical structure of wood

Wood as a natural material

Wood is a natural material with characteristics and features related to its growth in a landscape. Like the formation of stone, the natural features and character of a particular piece of wood are unique: dense knots where branches were originally formed on a tree, curved grain around these knots, and even small internal splits and cracks due to growth and exposure to windy conditions. As a natural material formed through a basic accumulation of cells, wood is therefore inherently variable, especially with regard to the material's mechanical properties and structural performance. Designers working with wood need to take special care and command a firm understanding of the material's features and characteristics. Compared to much more uniform and homogenous materials that are manufactured or synthesised to predetermined specifications, wood is far more complex. The basic properties and features of wood at different scales, along with relevant standards and techniques, are therefore reviewed herein.

An introductory literature review reveals that extensive study has already been done to develop an understanding and model of the basic physical structure of wood. Even in its natural state, however, many important features and characteristics are still poorly understood due to the complexity involved in the material's physical structure, especially at smaller scales than that of the typical wood cell. Discussions and reviews of the physical structure of wood include those by Dinwoodie[1], Wangaard[2], Bodig and Jayne[3], the United States Department of Agriculture (USDA)[4], and Gibson and Ashby.[5] The importance of understanding the structure of wood at various scales cannot be understated by anyone aspiring to work with the material, either as an architect, engineer, material scientist, or chemist. The following discussion reviews and summarises key details from the main references noted above, as each of the previous references tends to discuss certain scales more than others, depending on an author's interest and background. Relevant studies from the wood literature are also discussed, especially those dealing with new techniques for studying the structure and behaviour of wood. As most wood used in construction is softwood from coniferous trees, a detailed review of the more complex microstructure of hardwood from deciduous trees is omitted here. Emphasis is placed on common softwood species and the relationship between scales, especially between the micro and macroscale levels that are most familiar to designers.

Macrostructure

At the macroscale of everyday experience, many softwood coniferous trees such as Sitka spruce exhibit a slenderness, or height-to-diameter ratio, of about 100.[6] This natural slenderness ratio is far greater than the slenderness ratio of 5-10 for typical high-rise buildings and skyscrapers. The different regions in the cross-section of a tree, together with the micro and molecular structure of wood, all play an important role in the ability to grow in such an efficient and slender manner. The cross-section shown in Figure 2.1 is composed of heartwood, sapwood, cambium, and bark regions: the heartwood is the centre region of the cross-section and supports the tree, while the outer rings form the lighter-coloured sapwood region and also provide support and mineral and nutrient flow; the cambium is a thin layer of cells, located outside of the sapwood, and accounts for the only living cells in a tree trunk that initiates cell division and growth, with the bark providing an outer protective layer. Compared to the living cambium cells, the heartwood and sapwood cells have already died. The annual rings of the cross-section correspond to a tree's growth and age, where each year of growth involves the production of light-coloured earlywood in the spring, followed by darker and denser latewood grown later in the summer. Comparing the spacing between annual growth rings gives an indication of the growing conditions for different years. Wider-spaced growth rings indicate faster growth, but as already discussed in the introduction, can result in wood with lower density, strength, and stiffness.[7]

Figure 2.1: Cross-section of a tree illlustrating the annual rings and macrostructure of wood. (Photographer: Arnoldius)

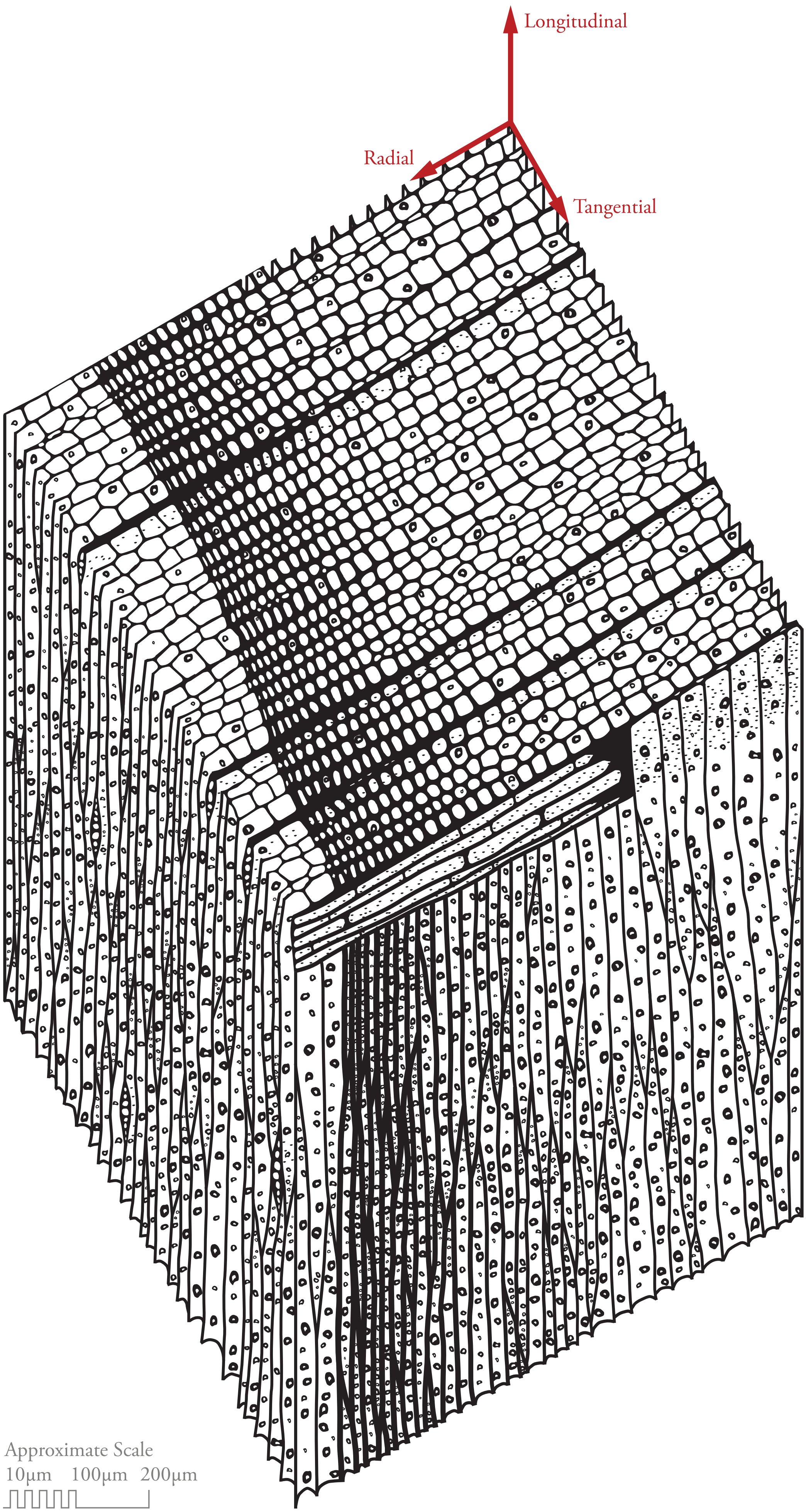
Moving slightly down to the scale of sawn and dimensioned wood products, structural engineers often describe wood as a non-uniform and orthotropic material, meaning that its mechanical properties vary with position, as well as direction of loading, respectively. For example, the compression strength of Norway spruce (*Picea abies*) along the direction of the longitudinal axis of the tree, or parallel-to-grain, is nearly ten times greater than that along the radial or tangential directions, perpendicular-to-grain.[8] Non-uniformities such as knots can also reduce the tension and bending capacities of wood considerably. The strength in bending for full-scale samples of UK Sitka spruce (with knots and curved grain) is approximately half that of small clear samples (with straight grain and no knots).[9] Furthermore, trees growing on sloping sites also produce 'compression wood' and 'tension wood' on the sides of the tree facing the downward and upward slopes, respectively. Large sections of compression and tension wood are generally avoided in structural applications due to their brittle nature and difficulty in cutting and shaping.[10] While most contemporary structural engineers consider all non-uniformities such as curved grain, knots, and bowed or curved sections of wood to be defects, traditional builders have often revered these characteristics for their unique qualities.[11] Little has changed in the material itself; only our perception of wood and its potential use changes with time.

Techniques for characterising the macroscale properties and behaviour of wood often involve standardised mechanical testing of small clear wood samples. As already noted, such tests may be unsuitable for assessing actual structural performance due to scale effects and non-uniformities, but nonetheless are useful for comparing the relative properties of different species, sites, and individual trees. For a given species, the variability in the mechanical properties of wood from different trees on the same site are usually more significant than the variability found between different sites.[12] Tests for bending strength and stiffness, compression and tensile strength, hardness, and other relevant parameters are outlined in BS 373:1957, which closely follows the wood research program conducted at the Building Research Establishment (BRE) in the UK.[13,14] An alternative standard in the United States is the American Standards and Testing Methods (ASTM) D 143 - 94, which in a similar manner as BS 373:1957 and the BRE work, appears to have been written based on the wood research at Yale University in the first half of the twentieth century.[15] In recent studies by Modén and Berglund[16], more advanced techniques based on digital-speckle photography have been used in tandem with these standardised tests to measure the whole surface strain of wood while undergoing mechanical testing. This somewhat advanced technique is especially useful in larger scale testing to assess the effects of knots and other complex mechanical effects at full-scale.[17] The standard BS EN 408:2010 gives guidance for conducting such full-scale tests. In the first instance, small clear specimens are a fairly inexpensive means for relative comparisons between species or processes, whereas full-scale structural tests can provide absolute results suitable for practical applications. Both of these testing strategies are employed and discussed in greater detail later on.

Microstructure

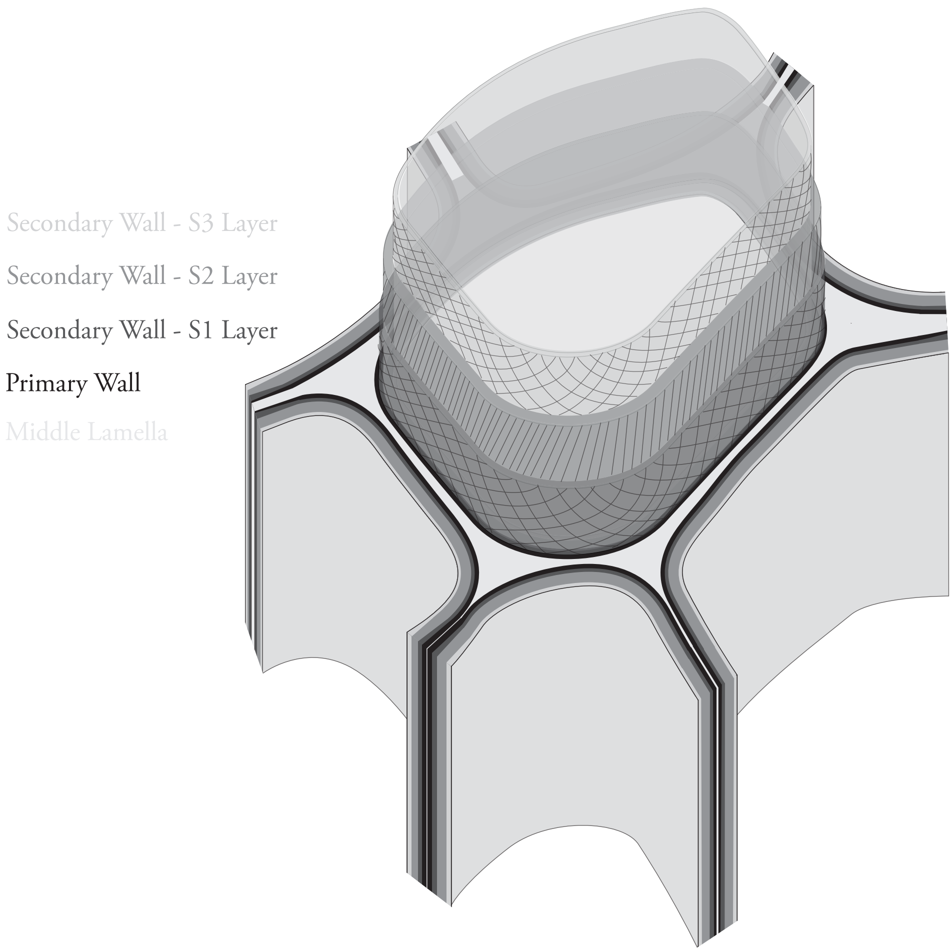
At the microscale, material scientists describe wood as a cellular material. Figure 2.2 shows a detailed axonometric illustration of the cellular microstructure of a softwood tree such as Sitka spruce, where most of the individual cells or tracheids can be seen in approximate alignment with the longitudinal direction along the tree. Some parenchyma cells or rays are also oriented outwards along the radial direction. In reality, the tracheids are actually aligned slightly off from the longitudinal axis, forming a spiralling pattern up the length tree; in extreme cases, the pattern is labelled accordingly as spiral grain.[18] The general cellular arrangement of wood, however, helps to explain orthotropic behaviour observed at the macroscale, as well as the tendency of wood to exhibit different crack propagation characteristics depending on the plane of propagation.[19] In the case of the tracheids, however, the darker coloured latewood cells also have thicker walls than the lighter coloured earlywood cells, and results in the latewood having an overall increased density at the macro level. Despite the difference in cell wall thickness, the density of the cell wall itself of both earlywood and latewood is approximately equal to 1500kg/m^3, and this value applies for all wood species.[20] By examining the mechanical data from a number of existing studies, Gibson and Ashby[21] have shown that the relative density of a given wood species, defined as the ratio of the cell wall density to the overall density of the wood, is the most influential parameter affecting the mechanical properties of small clear specimens. The macroscale mechanical properties of clear wood are not only strongly dependent upon the cellular arrangement of wood, but also the relative proportions of thicker latewood cells and thinner earlywood cells.

Figure 2.2 (opposite): Illustration of the cellular microstructure of softwood.


Longitudinal
Radial
Tangential
Approximate Scale
10µm 100µm 200µm

Models of the cell wall and its composition give a more detailed understanding of the microstructure of wood and its influence on macroscale properties. The cell wall can be thought of as a multi-layered structure composed of wire-like microfibrils. In the different cell wall layers shown in Figure 2.3, the microfibrils are helically wound in various patterns and orientations; in the primary wall, the microfibrils are randomly oriented, followed by three inner secondary layers with different microfibril orientations. In softwood species such as Sitka and Norway spruce, the thickest S2 secondary layer accounts for the majority of the cell wall and features microfibrils aligned 10-30° from the longitudinal axis.[22] The thinner S1 and S3 layers have microfibrils orientated 50-70° and 60-90° from the longitudinal,[23] although a more recent review contends that the S3 layer orientation remains unknown.[24] Regardless of the microfibril angle in the innermost and relatively thin S3 layer, the predominant alignment of the S2 microfibrils along the axis of the cell and tree further contributes to the orthotropic properties of wood at the macro level, especially MoE and stiffness. Therefore, changes in the S2 microfibril angle due to growing conditions can also affect the overall mechanical properties of wood.

Figure 2.3: Layered structure of the wood cell wall.



While the cellular structure of wood in a cross-sectional plane is closely related to wood's general orthotropic behaviour, examination of the microstructure along the length of the tree is needed to understand moisture effects. The moisture content of wood, defined as the mass ratio of water to dry wood, continually adjusts to seek an equilibrium state with surrounding conditions. For example, for a surrounding air temperature of 20°C with a relative humidity of 65%, the equilibrium moisture content of Norway and Sitka spruce is approximately 12%. Adjustments in moisture content towards equilibrium are made possible by two features of the microstructure: voids in the cellular microstructure, called lumen, allow nutrients and moisture to flow inside and along a single cell; and special apertures called pits allow nutrients and moisture to transfer to adjacent cells for further transport in subsequent cells.[25] As a typical tracheid or softwood cell is only roughly three or four millimetres long, pits are located along the length of a tracheid and especially towards each of its tapered ends, where the tracheids can overlap with neighbouring cells either above and below.[26] The pit apertures also feature a diaphragm-like membrane that restricts flow, although flow still remains possible. Without pits on the cell wall it would be difficult for softwood trees to grow and conduct nutrients and minerals, as well as to dry and remove excess moisture from wood after first being sawn. Although they are a relatively small feature, pits also play an important role in wood modification, and provide the basic gateways for wood to be impregnated with liquid-based treatments.

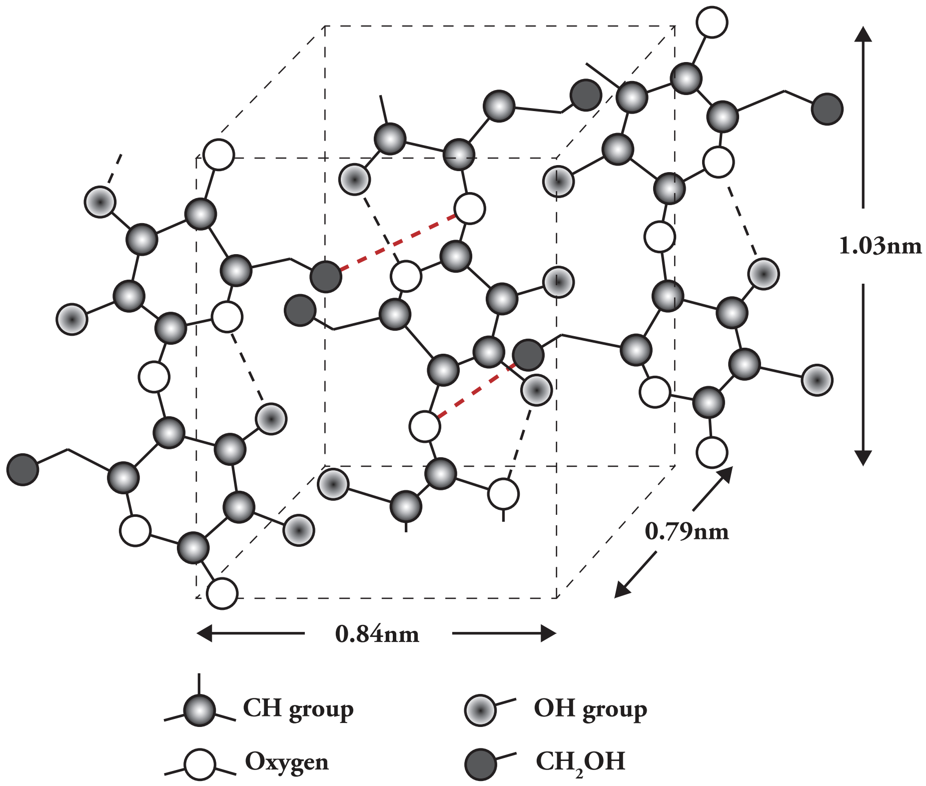
Changes in moisture content can sometimes lead to changes in the mechanical properties and dimensions of wood, and other times these properties remain independent of moisture conditions. This curious behaviour is explained by the difference between the moisture conducted through lumen and pits, usually referred to as free water, and the moisture held within the cell wall itself by intermolecular forces, referred to as bound water.[27] When an abundance of free water is present in wood, such as when a tree is first cut, the moisture content is said to be above the fibre saturation point, and the properties of wood remain constant with changes in moisture content. Note that for most species, the fibre saturation point corresponds to a moisture content percentage of about 30%.[28] In cases where no free water exists, the moisture content is said to be below the fibre saturation point and subsequent changes in moisture content affect the bound water in the cell wall, resulting in slight changes in the mechanical properties and dimensions of wood. This behaviour of the microstructure is important, as most temperature and humidity levels found in structural service conditions result in moisture content levels well below the fibre saturation point. Hence, wood can decrease and expand in dimension but also change mechanical properties while in service. The movement behaviour of wood should not be ignored, and can be seen either as an advantage or disadvantage of the material, depending on the designer and quality of workmanship of carpenters and builders involved.

Techniques for investigating the microstructure of wood include various forms of microscopy and spectroscopy, as well as chemical-based techniques for separating individual cells for subsequent examination of pits and other features. For example, research at the BRE has used a carefully prepared chemical solution to separate individual tracheids for studying their geometry and composition.[29] On the other hand, scanning-electron microscopy has been used extensively to observe changes in the microstructure of wood during loading and after failure.[30] X-ray diffraction has also been a useful technique for not only measuring the mean S2 microfibril angle in the cell wall, but also the distribution and variation in the S2 microfibril angle.[31] Due to the complexity of the microstructure of wood, a combination of measurement techniques is often employed. Studies based on the Silviscan measurement apparatus highlight the benefits of combining measurement methods.[32] Silviscan measurements typically involve automated x-ray measurements to determine localised wood density, microfibril angle, and tracheid dimensions such as length, diameter, and cell wall thickness simultaneously, providing a much more detailed understanding of the properties of wood.[33] Combining these measurement techniques has not only improved our understanding of the cells and their structure, but has also given a better understanding of how these properties interact and affect the macroscale properties.

Molecular Structure

At the molecular level, wood can be described as a composite material formed mainly of cellulose, hemicellulose, and lignin. In common softwood species, cellulose accounts for about 40% of the wood's chemical composition, while hemicellulose and lignin both account for roughly 25% each.[34] Minerals and nutrients used by the tree for growth, commonly described as extractives, are also found in wood and usually account for roughly 10% or less of the material's chemical makeup.[35] Different models have been proposed for the arrangement of the cellulose, hemicellulose, and lignin, but it is generally agreed that the cellulose forms hard chains of mainly crystalline regions, embedded within a hemicellulose and lignin matrix; this composite and layered arrangement makes up the longer strands of the microfibrils found in the cell wall layers.[36] However, cellulose, hemicellulose, and lignin can be further decomposed into their most basic molecular structures, where in the case of cellulose, the molecular chain-like structures of oxygen, hydrogen, and carbon, also feature the functional hydroxyl side groups (OH) among others shown in Figure 2.4. As the bonding between adjacent cellulose chains is dependent upon the nature of the functional groups, chemical modifications to the groups can be employed to affect overall changes in the cellulose and properties of wood. Inter-cellulose bonding is relatively simple compared to those between lignin, hemicellulose, and cellulose, but recognising this relationship between the smallest molecular scales of wood and its larger macroscale properties is the fundamental basis for chemical wood modification.

Figure 2.4: Proposal model for a unit cell at the moleular scale. (Adapted from Gibson and Dinwoodie)



Compared to homogenous materials, wood is a relatively difficult material to study at the molecular scale due to its composite nature and layered arrangement of cellulose, hemicellulose, and lignin. Employing dyes and other tracer chemicals can be useful techniques for understanding the porosity of wood and the chemical behaviour of the cell wall itself. On the other hand, Raman-based spectroscopy measures the vibrational energy associated with different molecules, and can provide detailed information on the chemical bonds and molecular structure of wood.[37] Near-infrared (NIR) spectroscopy is a technique that has only recently been applied to wood, and is based on correlating the changes of absorption spectra of near-infrared light reflected off of the surface of wood[38] or transmitted through it.[39] Wood scientists such as Kelley[40] and others[41] have developed this technique to better understand the chemical and mechanical properties of wood, and the interactions between the two. The NIR technique is also useful for measuring other properties such as density and moisture content non-destructively.[42,43] The multiple benefits and ability of the NIR technique to relate different scales of wood help provide a more holistic understanding of the material.

Summary

From the molecular to macroscale, the reoccurring physical characteristic of wood is its layered and fibrous nature: annual rings form layers of growth in the tree throughout its life, while the cellular microstructure of wood is formed of layered cell walls. The different layers of the cell wall itself are also formed from cellulose chains that are strong in tension, embedded in layers of relatively brittle hemicellulose and lignin. This concept of repeated layering at different scales in the structure of wood helps to explain its overall behaviour and orthotropic properties. This layering concept may also be useful for exploring the structural use of wood at taller and larger building scales usually associated with concrete and steel. However, in addition to the layered structure found at different scales, localised effects at different scales cannot be ignored: the functional groups and bonding behaviour of cellulose chains as well as macroscale features such as knots both play an important role in affecting the properties of wood. Well-established techniques for studying the structure and properties of wood exist at all scales and have been successfully exploited to great effect. Emerging techniques such as NIR that bridge and relate different scales and properties also offer future promise for improving our understanding, and how the different scales of wood can interact and influence overall properties and behaviour.

Embodied qualities of wood

Introduction

In addition to the physical structure of wood, the following two sections present a discussion on the embodied qualities and quantities of wood in relation to practical applications and design rationales for using wood in general. The term 'embodied qualities' is used here to intentionally contrast against the common but problematic concepts of embodied quantities, like embodied energy and carbon, and sequestered carbon, which will be addressed in due course. Before doing so, the present discussion aims to offer a sound reasoning for using wood in the built environment. A discussion of the embodied qualities of wood, including but not limited to its renewability, ease of use and working, health benefits, and cultural value, will show how the material's qualities can play an important role in design. Some of these qualitative aspects of wood have been considered by others alongside rationales based on embodied energy and carbon estimates, but they are usually done so in a marginalised way; in other cases these aspects of wood are overlooked completely due to their qualitative rather than technical nature. The character and qualities of wood are nonetheless a powerful reservoir from which designers can draw to produce compelling buildings and structures. A designer's interests and ambitions, however, must not ignore but extend beyond, the purely technical and aesthetic aspects of engineering and architecture. Such thinking is only possible if one considers the close relationship of structural design and architecture and their wider topographical, social, and cultural context. As an architectural and structural material, wood has much more to offer than the ability simply to sequester carbon while replacing concrete and steel.

Renewability

Compared to other modern structural materials, wood is the only renewable material that can be used by designers and builders today. Although engineered bamboo is also a material that is grown and managed, and hence, renewable, its certification and development for structural applications is still in progress worldwide.[44] Wood's renewability and the growth of trees, however, is one of the simplest reasons for its use in contemporary and future buildings. But renewability is also one of the strongest claims and reasons to build with wood. With continued forest management and growth, wood's undeniable character of being renewable and available for future generations of architects and engineers can serve as a readily understandable and tangible example of sustainability in the built environment. Unlike the processes for producing steel or cement, the growth of trees and use of wood for building is something that everyone can immediately grasp, but also an aspect of our built environment that enables a shared understanding. The grown nature of wood and its ongoing use establishes continuity in traditional, contemporary, and future buildings and construction.

The idea of wood as a renewable material does not necessarily need to be limited to the natural growth with trees, but may also apply to a building or structure if so designed. For example, a design could involve individual structural elements that could be easily replaced or serviced, one element at a time following scheduling or as the need arises. Such a design would ensure a robust structure that could be easily maintained. This type of serviceability touches on current issues of design for deconstruction, but with a focus on maintenance and continuity rather than end-of-life only. In any case, designing for renewability or serviceability, or deconstruction may be achieved in steel or precast concrete, but the cellular character of wood and its low density are particularly attractive material properties for such an approach.

Renewability as a design concept should also not be confused with flexibility or adaptability, where structures and spaces are typically designed with open plans to allow any possible future use. Some consider flexibility as a means to avoid early demolition and prolong building life, forgetting important lessons from architectural history and criticism. Modern architects in the early twentieth century considered new structural materials like steel and reinforced concrete to be flexible because they could support different spatial concepts in a design process. Flexible structures could be easily adjusted in the design process, offering different ways to support and engage with architectural and spatial intentions. Later on, however, notable architects in the 1970s reinterpreted flexibility into a notion of universal adaptability, with implicit claims of added value as 'flexible' buildings could be easily changed to support different uses in the future.[45] Colquhoun addressed and strongly criticised this latter concept of flexibility:

> This attitude assumes that architecture has no further task other than to perfect its own technology. It turns the problem of architecture as a representation of social values into a purely aesthetic one, since it assumes that the purpose of architecture is merely to accommodate any form of activity which may be required and has no positive attitude toward these activities.[46]

In contrast to later concepts of flexibility as adaptability, the case of renewability in structures requires a careful amount of planned redundancy in the original design and construction. Redundancy in this case should also not be seen in conflict with structural efficiency, as additional redundancy, beyond safety and load factors, is already required in general structural design to prevent disproportionate collapse. This existing design requirement of redundancy for safety and stability against disproportionate collapse can be further exploited to include provisions for renewability. After construction though, a particular design may only achieve renewability through its continued use and value by people and occupants, who ultimately determine the success or failure of the design.

Ease of reuse and working with wood

The design concept of renewability also raises important questions for the end life or perhaps second life of wood elements and structural components. Wood's ability to be reused even after a considerable amount of time is an important quality that should not be ignored. Current research on steel and aluminum also emphasises the importance of reuse, as optimising production and recycling processes have already reached a point of diminishing returns.[47] In the specific case of wood structures, however, the reusability of existing elements like beams and columns for subsequent structures requires careful consideration. Wood for reuse in structural applications may be actually limited due to creep and duration of load effects, or static fatigue. For example, after being in service and under loading for 50 years, the strength of wood reduces by approximately 40%.[48] Although current design codes account for this effect through safety factors (k_{mod} in the case of Eurocode 5), there are currently no provisions for duration of load effects of reused structural wood elements. Nonetheless, there may be more valuable uses of structural wood in non-structural applications such as furniture, flooring and cladding, or even lower value applications such as fencing or pulp. In the worst case, reused wood can be still put to good use as combustion fuel for heating and generating electricity.

Another important embodied quality of wood that can often be taken for granted is the material's ability to be easily worked. Primary processing like sawing, and secondary processing that transforms wood into value-added products are only possible due to the ease of working with the raw material in the first place. The reuse of wood elements is also strongly dependent on the material's ability to be reworked. The relatively low density of wood accommodates either minor or major working or processing to take place. The cellular nature of wood means that it can be easily separated, split, sawn, turned, shaved, or sanded, with either simple hand tools or powered and automated machines. Despite recent advancements in wood machinery, especially with computer-numerically-controlled (CNC) equipment, simple tools such axes and handsaws are still widely used today. Working with wood can and still does encompass both traditional and state-of-the-art techniques and tools. The ease of working with wood also means that it can be easily combined with other materials in composite structures. In any case, as a structural material on its own, or in a composite arrangement, or as a finishing material through reuse, wood can have either traditional or contemporary associations and value, or both simultaneously, depending on the design.

Availability

The important issue of the availability of wood from forests is a necessary condition for both our historical, current, and continued use of the material. In many parts of the world, the relatively widespread growth of trees meant that wood could be sourced and transported with relative ease and used for many purposes. Deforestation remains a critical issue that needs to be addressed in many parts of the world today, but managed forests in several countries and regions are also expanding significantly. The United Nations estimates that in about half of the world's countries, deforestation has either been halted or actually reversed.[49] The UK's expected increase in available Sitka spruce in the coming decades is an example of this trend of afforestation. Forest area throughout Europe, however, is also expanding and making additional low-grade wood available without diminishing future harvests. With world paper consumption also showing a gradual reducing trend, additional wood normally used for pulp and paper production might also be available for construction in the future. The general availability of wood and its expected increase in availability represent an important opportunity for designers and craftsmen in various fields, especially in architecture and engineering.

Aesthetics of wood and timber

A simple embodied quality of wood that is difficult to forget but also sometimes difficult to discuss is the material's look, feel, and overall aesthetic. This quality is frequently suggested when people vaguely describe wood as 'warm' or 'tactile'. The general aesthetic of wood may be difficult to express in an explicit way, but the overall positive perception and appreciation of wood by people is commonplace. The natural growth and unique features of wood, especially the complex grain and knot patterns that are produced after sawing, is one important factor that contributes to the appreciation of wood by designers and the general public alike. For example, countless non-wood products made from plastic or other synthetic materials are given similar textures and colouring to resemble wood[50], and architects and engineers often cite the aesthetic of wood as a simple design rationale. Furthermore, different sawing techniques and methods, such as quarter sawing and rift sawing, lead to different grain patterns in addition to improved movement characteristics with changing moisture content. In many applications such as in furniture and interior finishing, however, only the distinct quarter-sawn grain pattern of wood is sought after, without consideration of the reduced movement and minimal warping of quarter-sawn wood with changes in moisture. Researchers and designers interested in using and promoting wood in the built environment therefore should not trivialise or disregard the material's aesthetics. The visual and tactile qualities of wood can influence our perception of the material in a direct and physical way.

For almost all applications and uses, wood needs to be processed to some degree and then converted into elements for a future with a diverse range of uses. The mechanical properties of many wood products such as sawn timber, including strength and MoE, maximum dimensions, and local defects are still governed primarily by the natural growth of trees. In the case of building with standard sawn and dimensioned wood, the final products are sometimes referred to as lumber, timber or sawn timber, with the term 'wood' often used only to describe the material in its raw, unprocessed state. This distinction may be subtle, but is nonetheless misleading, as both the aesthetics, technical properties, and characteristics of many wood or timber products are far more dependent on the natural conditions of growth than any subsequent processing. For example, without the radial parenchyma cells or rays orientated in the radial direction of a tree, quarter-sawn wood would be much more visually similar to regular plane-sawn wood. In other cases, including but not limited to fibreboard and OSB, the natural aesthetic and appeal of wood can actually be lost and irrecoverable. These are examples of engineered wood products that are processed to such a degree that they no longer feature the natural characteristics and features of wood; the processes involved in their transformation from wood to a wood product determine their final properties, just like ordinary industrial materials. While heavily processed wood products in turn lose their natural aesthetic qualities and appeal, they are usually traded-off for improvements in dimensional stability and uniformity. The paper that these words are printed on provides an interesting example at hand of a heavily processed but still incredibly important timber or wood product. The paper's properties and aesthetics are determined predominately by its conversion from wood, rather than the growth of the wood itself. We do not even generally think of the paper's origins as wood. Both cases of maintaining the natural aesthetic and properties of wood, or heavily processing the material to achieve uniformity and homogeneity, can be put to good use in architecture and engineering, among other applications.

Health and psychological benefits

In recent years, psychologists are finding that the natural aesthetics of wood have surprising effects and health benefits in our built environment. A thorough review by Fraser[51] in 2011 of studies dating from 1995-2010 suggests numerous benefits of including wood in indoor environments: decreased pulse and stress levels, general promotion of well-being, and overall increase in positive feelings. These studies have been extended primarily from earlier research on the overall benefits of views of nature such as trees or even the presence of indoor plants. These earlier studies similarly showed how patient recovery time is shortened in rooms with views of natural settings, while rooms even with plants can help increase work performance and improve pain tolerance. In the later studies on the positive health effects of wood, Fraser acknowledges that this field is still in its infancy, but also that the statistically significant results from studies conducted so far should be taken up by wood companies for new types of marketing and promotional purposes. An important question that remains unclear from the work so far is whether these health benefits can be achieved simply with wood furniture, or if wood finishing, structures, and homogenous or unnatural wood products like OSB each have different effects. Future research for better understanding the positive health benefits of wood in indoor environments would therefore be of value and interest.

Wood as a cultural material

In addition to thinking of the availability, aesthetics, and usefulness of wood in both natural or unnatural states or products, the diversity of applications and reuses also make wood an important cultural material. The combination of being readily available, easily worked, and reusable has made wood an important material in very a wide range of activities, practices, and daily rituals in human history.[52] Figure 2.5 shows a small but diverse sample of wood artifacts, ranging from traditional toys and musical instruments to buildings and boats. This limited set of artefacts could easily be expanded upon, but for brevity and as illustrative examples, they show wood's diverse use in different human cultures and climates of the world. These examples also serve to show how the different characteristics, properties, and structure of different wood species can be related to design and use. The general softwood used in construction might be poorly suited for a musical instrument like a cello, while a birch bark canoe would be difficult if not impossible to construct with another tree species. The wide range of dates associated with this small group of artifacts also illustrates the traditional and contemporary interest in wood, and our sustained and ongoing engagement of working with the material.

Figure 2.5(a-d)
in descending order:
(a) Traditional whip and top,
(Source: National Museum of Wales)
(b) Cello,
(Photographer: Marco Tedaldi)
(c) Multihalle by Gion A. Caminada and Conzett, Bronzini, Gartmann AG, 2005, Vrin, Switzerland,
(Photographer: Michael Ramage)
(d) Birch bark canoe by Richard Kroeker and architecture students at the Technical University of Nova Scotia, Halifax, Canada.
(Photographer: Richard Kroeker)

Apart from human culture, activities, and uses, the artifacts in Figure 2.5 represent what Sennett describes as a tacit or intuitive knowledge of the material.[53] The idea of tacit knowledge is demonstrated in the difficulty of master craftsmen in sometimes explaining their techniques and skills to apprentices. Another example is the difficulty of learning a simple task like tying a knot or shoelaces following written instructions, compared to the ease of learning the task once being shown in person or following visual diagrams. Modern and scientific understanding of wood has expanded upon tacit and intuitive knowledge with a more explicit understanding of the material's properties and characteristics. At the same time as presenting such complexities and technical opportunities to scientists, designers, architects, and engineers, the material is still steeped with cultural value. In maintaining the natural characteristic of wood rather than heavily processing the material before use, wood can also maintain its general cultural value in addition to its aesthetic value. But as an architectural material, wood can also play an important mediating role between the built environment of an urban situation and the broader environment and landscape of the material's origins. Figure 2.6 illustrates one example of how architects and engineers have used wood as a material that lies at the intersection of tradition, contemporary technology, culture, and landscape. Although most designers tend to focus purely on the technical and aesthetic value and potential of wood, its underlying cultural value can contribute a powerful role in a completed project.

Summary

The embodied qualities of wood, like the material's natural growth and renewability, ease of working and ability to be reused, aesthetics and positive health benefits, and cultural value are important characteristics for designers and researchers to consider. Many of these qualities are straightforward and simple, but they should still not be discarded or forgotten about when thinking about designing and working with wood. The simplicity of the embodied qualities of wood is not a drawback, as the direct and physical qualities of wood are easy to understand and experience by everyone. The embodied qualities of wood can also play an important role in the design process, by extending the renewability of wood to develop a concept of renewable structures that are easily maintained and serviced. Focusing on the qualities of wood and what the material can offer a design situation is important, and can also be done alongside considering the qualities of other structural materials like steel and concrete. Each material has its own unique properties, different uses, and values and associations that are either suitable individually or more often, in combination with each other, for responding to a particular situation through design.

Embodied quantities of wood

Embodied energy, embodied carbon, and sequestered carbon

Many researchers, practitioners, and even policymakers are becoming increasingly focused on the embodied quantities of wood. This focus on embodied quantities mirrors an increasing awareness of climate change, the green-house gas emissions associated with building construction, and the need for ambitious targets to reduce overall emissions in the coming decades. No strict definitions have been established for widely used terms like embodied energy or embodied carbon, but as general concepts they refer to the abstract or symbolic embodiment of energy and carbon dioxide emissions associated with a material's production. Depending on the chosen scope and boundary of a study, embodied energy and carbon calculations can also include and account for disposal. In the case of an engineered wood product, its total embodied energy may be composed of, but not limited to, the energy associated with several activities: forestry processes like cutting and transporting trees to a sawmill, then primary processing like sawing and kiln drying; secondary processing such as lamination, followed by transportation to a building site, and finally removing and recycling or disposal at the end of the structure's service life. In a similar way, embodied carbon is used to describe the carbon dioxide emissions associated with the embodied energy. To further complicate the matter, in the specific case of

Figure 2.6 (opposite): Shore House by Mount Fuji Architects Studio, 2013, Kanagawa, Japan. (Photographer: Ken'ichi Suzuki) wood, some researchers also consider the actual (non-symbolic) absorbed or 'sequestered' carbon removed from the atmosphere during the growth of a tree. Note that often the embodied energy and embodied carbon of a material may be related, but in certain instances like the production of cement, there is an additional contribution to the material's embodied carbon. The chemical process of producing cement itself releases carbon dioxide that is accounted for in the embodied carbon of concrete.

These three terms, embodied energy, carbon, and sequestered carbon, are quickly becoming a common currency in structural design alongside more conventional material properties like strength, MoE, and density. In the case of architects, engineers, and policy makers who promote wood construction based on its environmental credentials as a response to climate change, wood's ability to sequester carbon is often held up as one of its characteristic benefits. In the following section, this prevalent claim is reviewed and forcefully questioned in light of various research studies that individually point to different limitations and problems of modelling and estimating embodied energy, carbon, and carbon sequestration. When these studies are considered together in a more broad and holistic view, one can see how such arguments for wood construction based on carbon sequestration and life-cycle analyses should not be prioritised in structural design.

Examples from research literature

A large body of research literature has already been stockpiled on the subject of embodied energy and carbon, and carbon sequestration in buildings. Moncaster[54] performed an extensive review in 2012, which in turn summarised reviews published from 2007–2010 by Sartori and Hestnes[55], Dixit *et al.*,[56] and Hernandez and Kenny.[57] One kind of study frequently seen in the literature uses embodied energy and carbon as metrics in comparing two or more ideal or representative building types or structural materials. In a relatively early pair of studies in 1994 and 1999 by Buchanan and Honey[58] and Buchanan and Levine,[59] respectively, the embodied energy and embodied carbon of wood was considered for typical building types such as houses, offices, industrial structures and multi-storey hostels. Using values of 600MJ/m^3 for the embodied energy in sawn wood and a range of 500-1000MJ/m^3 for the embodied energy in various engineered wood products in New Zealand, Buchanan and Levine's analysis considered the embodied energy and carbon of wood, steel, and concrete construction for the building types noted above.[60] Along with embodied carbon, they also accounted for the sequestered carbon in wood during growth with an estimated amount of 250kg of carbon per m^3 of wood.[61] Furthermore, the later study also considered the offset savings in embodied carbon or replacement value of using wood instead of concrete and steel, thereby further increasing carbon sequestration effects. Not surprisingly, they concluded that reductions in fossil fuel energy and carbon dioxide emissions could be achieved through increase wood use in New Zealand construction. Following a very basic extrapolation process, they also found a similar result for the global construction industry.

A more recent study in 2010 by Vukotic *et al.*[62] has commented on these specific studies and especially the sensitivity of their results to the assumed values for embodied energy and carbon of different materials. For example, in contrast to Buchanan and Levine's results, the embodied estimates of commercial UK buildings by Sansom and Pope[63] suggested that optimised steel construction leads to lower embodied energy and carbon than wood and concrete alternatives. Note that the British Constructional Steelwork Association also sponsored the study, although the authors did not acknowledge any conflict of interest. In contrast to Buchanan and Levine, Sansom and Pope chose not to account for the sequestered carbon in wood, and performed their estimates with the assumption that 80% of a timber structure would be directed to landfills instead of being recycled or reused at the end of a structure's life. In their discussion, Sansom and Pope admittedly recognise the difficulty in forecasting how materials might be reused or recycled in the future. Similarly, Vukotic *et al.* also discuss in greater length how the end-of-life stages of a building might influence embodied energy and carbon comparisons, as the purpose of their study was to identify the stages of a building's life where significant carbon reductions can be achieved.[64] While their focus on life-cycle stages is presented as an alternative to determining if a material is 'better' than another in embodied energy terms, they performed the same kind of familiar estimates and comparisons with alternative building designs in different structural materials. When concluding on the specific cases compared, a timber structure was 'the better option over the whole building life.' In their conclusion, Vukotic *et al.* further found that the most important stages for reducing carbon emissions is the initial material selection at the design stage, including sourcing local materials, and the reuse or recycling of materials at the end of a structure's service life.[65] If these conclusions seem self-evident, or perhaps even arbitrary in the case of Buchanan and Levine's and Sansom and Pope's studies, the value of such exercises should come into question. One might ask the simple question of what such analyses, fraught with complexity and being easily compromised by assumptions, uncertainties, and biases, are able to offer architects and engineers in the first place.

Examples from practice and practice-based research

In practice, architects and engineers interested in promoting large-scale wood construction are performing similar types of comparisons, but following a much more oversimplified method. In the case of the Murray Grove Stadthaus in London, Waugh Thistleton Architects used sequestered carbon estimates to support their CLT-based design for a nine-storey wood building.[66] Waugh argued that the wood structure for the project actually stores approximately 185,000kg of carbon, whereas a comparable reinforced concrete structure would emit approximately 125,000kg of carbon in the atmosphere in the form of carbon dioxide emissions.[67] In this comparison, the sequestered carbon in the wood was taken into account, along with the wood's embodied carbon due to transportation from aboard, but the embodied energy and carbon due to manufacturing and producing the wood for the structure was not considered. Engineers from Ramboll UK have also performed similar estimates to support a wood-based design for the Open Academy school in Norwich. They did consider embodied carbon and sequestered carbon together, citing embodied carbon values from the well-established Inventory of Carbon and Energy (ICE) database.[68] The database, however, does not account for sequestered carbon so this additional factor was included following the preference of the designers. On the one hand, references to such open databases and figures help reduce bias and increase transparency in embodied energy and carbon estimates. On the other hand, these two examples from practice illustrate how uncertainties can arise not only in the estimated values for embodied energy and carbon of materials themselves, but more critically, in the application of such values in actual comparisons and modelling efforts.

So far only the discrepancies and biases of a limited number of embodied energy and carbon estimates have been emphasised. One the most consistent and common elements seen in embodied estimates and studies for structures, however, is a very brief acknowledgement that embodied energy and carbon estimates are significantly lower than a building's operational energy and associated emissions. Operational energy is broadly defined in such cases as the energy needed to heat, cool, and operate a building during its service life. In many studies a further justification for embodied energy and carbon estimates is given by citing that the operational energies of buildings are gradually being minimised. Researchers and practitioners interested in embodied estimates almost seem compelled to claim that in the future, the embodied energy of structures will become more significant compared to operational energy. What such statements fail to recognise is that just like uncertainties in embodied energy estimates can arise due to future recycling and reuse, operational energy estimates can also be compromised with large uncertainties. Such statements therefore should be regarded as speculation, primarily based on a carbon dioxide emission-free supply of electricity and with little regard of how strongly the behaviour of building occupants can influence operational energy. The architect Peter Clegg has summarised this position in very blunt terms, with the acknowledgement that buildings by themselves do not create energy demands resulting in energy consumption, but the people inside them do.[69]

A 2008 study from Hacker *et al.*[70] of Arup Research + Development, the British Cement Association, and the Concrete Centre, illustrates in detail how speculations on future operational energy trends can undermine and influence embodied energy and carbon calculations. Their study begins by recognising how concrete construction has been set in a poor light in many embodied carbon studies and estimates. They attempt to redeem concrete construction by making a relatively simple argument: in the long-term future context of climate change and warmer UK summers, compared to wood-frame construction, concrete's thermal mass will actually deliver net savings in total energy and emissions by reducing overheating and operational energy requirements.[71] What is interesting about the study is that the typical semi-detached family house considered is not explicitly set in London, only generally in the south-east of England, while London weather data are used as the baseline for making future climate predictions. The authors do not acknowledge that heavyweight construction and adding thermal mass even in the central urban context of London today does not offer additional summertime cooling benefits due to the urban heat island effect.[72] Instead, the concept of thermal mass here is assumed to be universally applicable in low-energy building design, no matter what the context and climate. Invoking and manipulating a relationship between structural materials and operational energy can further allow energy estimates to be used as disguises for biases and unhelpful comparisons.

Policy-level example

In addition to research literature and practice, embodied energy and carbon-based arguments are also operating at the policy level by groups that support wood-based construction. Estimates from the Wood for Good UK-based group is one example of sequestered carbon estimates used as propaganda for wood construction and forestry. In response to the UK government's future industry investment of £1 billion in carbon capture development and construction, Wood for Good contended that this £1 billion could be used to plant approximately 1.1 billion trees, thereby sequestering 10 million tonnes of carbon each year, or approximately 37 million tonnes of carbon dioxide from the atmosphere.[73] Herein lies one of the main problems of embodied carbon and energy estimates; a figure like 37 million tonnes of carbon dioxide is misleading and difficult to interpret on its own. Furthermore, even if this estimate may be transparent and even reasonably accurate, those performing the estimate do not provide a more holistic view of the orders of magnitude involved. For example, the UK's current carbon dioxide emissions are in excess of 600 million tonnes annually, with a target to reduce annual emissions to approximately 160 million tonnes by 2050. So while an additional 1.1 billion trees might sequester carbon and lead to lower carbon dioxide emissions, the intended lowering effect can contribute to roughly only 8% of the UK's 2050 target. Work by Lauk *et al.*[74] has considered changes in global carbon stocks from 1900-2008, and similarly warns of the limitations of simply planting trees in an attempt to mitigate the complex social, economic, and political issue of climate change. While the Wood for Good group does cite other positive reasons for increasing forest cover and using more wood in the built environment, its promotions based on carbon sequestration show how these estimates can also be misleading. Instead of making a strong case for the use of wood in the built environment, these estimates undermine credibility and distract from other efforts for promoting wood construction and forestry.

Discussion on the question concerning sequestered carbon

The complexity, uncertainty, and relative order of magnitude of embodied energy and carbon estimates have been called into question thus far. A limited number of examples from research, practice, and a policy group have been used to show not only the shortcomings of these estimates, but also their pervasiveness. The general concept of sequestered carbon, however, still needs to be addressed more directly, as its inclusion or exclusion in estimates is often a contentious but decisive factor in determining results. From the perspective of an architect and structural engineer, the problem with considering sequestered carbon as a concept in the early stages of a design process is that it is fundamentally in conflict with designing efficient wood structures. While material and structural efficiency itself should not necessarily be the sole ambition of designers and especially engineers, sequestered carbon is an even more problematic concept in design. Any wood structure, no matter how poorly or well designed, can have some claim to sequestering carbon. Furthermore, pitting wood structures against alternatives in steel or concrete with embodied energy and carbon estimates and comparisons offers no consideration for innovative composite structures. Structural engineers and designers should therefore be suspicious of any design or policy-level promotion that heavily relies on sequestered carbon as its primary rationale or explanation. The work presented later on makes no reference to embodied energy or carbon, or sequestered carbon.

Several studies in the research literature also acknowledge the criticisms and problems of embodied energy and carbon estimates presented here by the author. Standards have been drafted in the recent years for reducing uncertainties and biases in embodied energy and carbon estimates.[75] They still include open provisions and different options for either including or excluding certain stages of material's life, such as transportation and disposal in embodied estimates. For those not operating in a design role but who are still interested in engaging in embodied energy and estimates, despite the criticisms presented so far, the issue of whether or not to consider sequestered carbon remains. Compared to embodied energy and carbon, one key advantage that sequestered carbon commands is that it is an actual material property. Sequestered carbon is not symbolically embodied, but is physically embodied, meaning that it is not plagued by the same uncertainties as embodied energy and carbon. At least not in the first instance, as one can actually pick up a piece of wood and immediately estimate its weight and sequestered carbon. The same cannot be said for the embodied energy of that same piece of wood, which would rely on many factors such as its transportation history and whether the wood was sawn and kiln dried by power generated from biomass, a coal-fired electricity plant, or a hydroelectric dam. Of the three concepts discussed, namely embodied energy, carbon, and sequestered carbon, sequestered carbon is the most straightforward and least fraught with uncertainty.

The concept of sequestered carbon should also not be limited to only wood structures. Concrete research is already starting to offer ways of sequestering carbon, albeit not as efficiently or effectively as in wood.[76] In the specific case of wood structures, however, sequestered carbon needs to be considered in light of the typical growing rotations of managed forests and the design-life of a building. The argument for including sequestered carbon usually follows that a wood structure is locking away carbon dioxide from the atmosphere through the managed growth of trees and their absorption of carbon. Arguments for including sequestered carbon with embodied carbon estimates therefore only become meaningful when a structure is designed to last, and actually lasts longer than the time it takes to grow trees. By designing structures with a shorter service life than the typical growing time of about 40 years for managed trees in the UK, the carbon already sequestered in a tree is only being moved from the forest to a building, at a faster rate than a replanted tree can sequester carbon from the atmosphere. While most UK buildings are designed with a service life of around 60 years, buildings can often be demolished prematurely. If timber buildings are demolished faster than the time for trees to regrow, this arrangement does not allow enough time for planted trees to mature and sequester the equivalent amount of carbon in the building. This additional contingency might be one final reminder about how the results of sequestered carbon estimates can be so easily skewed, misinterpreted, or misrepresented.

Summary

The carbon dioxide emissions related to structural materials have become a commonly discussed and cited metric in architecture and engineering. Researchers, practitioners, and policy groups are using the concepts of embodied energy, embodied carbon, and sequestered carbon for estimates and comparisons of the environmental credentials of different structural materials. The uncertainties, biases, and limitations of such comparisons in several examples have been reviewed and discussed, showing how estimates and modelling can be used as a rhetorical device to promote an agenda for a particular structural material. Instead of counting and trying to estimate the tonnes of carbon associated with one material and design or another, this review and discussion has aimed to discount such comparisons. Although the development of standard methods and guidelines may help reduce uncertainties and increase the transparency of results in the future, the order of magnitude of embodied energy and sequestered carbon in structural materials only accounts for a relatively small percentage of overall carbon dioxide emissions. Hence, their role in mitigating climate change is also significantly limited. Architects and engineers should therefore focus on designing structures and buildings that can deliver value to clients and occupants in the present and in future generations.

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Chapter 3 SMALL-SCALE TESTING OF IMPREGNATED AND STRESS-LAMINATED WOOD

Overview of approach, methods, and testing

The previous chapters have introduced and explained a number of relevant issues for the two main areas of the thesis, namely, wood modification by impregnation and stress lamination. So far the expected increase in low-grade UK wood and its incompatibility with current lamination standards and techniques have been discussed, along with developments in the design and construction of tall wood buildings. On the one hand, wood impregnation has the potential to increase the use of low-grade wood by enhancing the mechanical properties of the material. On the other hand, as an alternative form of lamination that is more compatible with low-grade wood than other standard techniques, stress lamination can also add value and broaden the scope of large-scale wood construction. This chapter and those that follow will build upon the previous discussions with original research and experimental work at different scales. With a clear need to improve the properties of low-grade wood and assess the performance of new stress-laminated elements, experimental testing is a straightforward and appropriate method that is used throughout the following work. Testing at different scales is required for determining the usefulness of new variations, techniques, and approaches in wood impregnation and stress lamination.

The introduction given in chapter one compared and emphasised the differences between wood modification and stress lamination. The former is marked by complexity, requiring advanced laboratory equipment and techniques. Furthermore, wood modification demands an acute understanding of not only physical and chemical structure of wood at various scales, as discussed in the previous background chapter, but also fluid-flow behaviour in the cellular structure of wood during the impregnation process. The approach taken here, however, is not to develop a completely new type or form of wood modification and impregnation, but only new techniques or variations on established research and knowledge. Although developing new wood impregnation techniques may be more challenging on a broader scientific front, new developments can be still compatible with existing and relatively simple wood treatment facilities and common industrial equipment for treating wood. Capital investment at the industrial or production scale can be minimised and focused on the synthesis of new monomer solutions that function within the existing framework of wood modification and treatment facilities. The key challenge in the present research therefore is to show the feasibility and effectiveness of impregnation solutions at smaller scales within the limitations of a typical chemical laboratory.

At an industrial scale, stress lamination would similarly demand minimal investment, requiring only very simple tools and construction expertise, thereby avoiding the expenses associated with large-scale presses, fixtures, and other industrial machinery. Stress lamination can still be a productive and especially flexible and cost-effective technique for both standard and custom large-scale building elements. The technique is particularly appealing for the UK context, where very little capital investment has been made in engineered wood technologies like glulam, LVL, or CLT. Stress lamination, however, also holds potential in other contexts with immediate access to low-grade wood but without accessibility to industry or more common engineered wood products. Despite relying on simple construction methods, a firm understanding of the microstructure and moisture-related movement of wood is still needed to control losses in prestress in stress lamination. The earlier introductory comparison between wood impregnation and stress lamination, with the former positioned as scientific and complex and the latter as crude and simple, can now be seen in a more subtle light. Both techniques contain aspects of simplicity and complexity in their development and later in possible larger-scale application. Development in either area, however, shares a common ground with an understanding of the physical structure of wood at different scales.

The experimental work that follows in both wood impregnation and stress lamination is similarly underscored by a unified approach and method through experimental testing. In the wood impregnation and stress lamination experiments that follow, initial tests are always carried out with small-scale models or specimens. Scale models are economical, easy to impregnate with standard glassware and chemistry equipment, quick to test, and also relatively easy to handle for exploring new structural forms through prestressing and stress lamination. In the case of wood impregnation, however, small-scale models or specimens of impregnated wood are inappropriate to gauge flow phenomena or depth of penetration issues. Small clear wood specimens and models are simply too small to represent larger-scale flow effects within the more complex and varied structure of full-scale sections, which include curved grain and imperfections such as knots. Similar scale effects are also present in small stress-laminated models, which also do not contain large imperfections and have higher material properties like bending strength and MoE when compared to full-scale structural sizes. Due to these known scale effects in wood, full-scale testing is needed to confirm research findings at smaller scales. Full-scale testing can also provide a valuable method for gaining first-hand experience and developing a physical or tacit understanding the behaviour of materials and structures. Based on full-scale test results, the structural performance of impregnated wood or new stress laminated elements can also be more reasonably compared with basic modelling, or estimates from current design codes for solid timber and other established lamination methods like glulam.

This combination of small-scale and full-scale testing methods is employed here as an intentional blending of the methods from architectural and engineering research. Alongside drawing, scale model making is synonymous with architecture and design, including in practice, research, and educational settings. Mechanical and structural testing is also commonplace within the history of engineering, and is also found throughout research, teaching, and practice, and especially in the historical development of new structural concepts and forms. Two well-known examples include the timber gridshell structure for the Multihalle in Mannheim by Frei Otto and Ted Happold in the mid 1970s, and the concrete bridges by Robert Maillart in the early twentieth century.[1,2] The two approaches from architecture and engineering taken together might be described as multidisciplinary, but they also offer a more important benefit; scale model making, mechanical testing and full-scale structural testing are natural complements to one another, much in the same way that architects and engineers in practice complement each other's skills and interests. Borrowing the methods of architects and engineers is suitable for research that is relevant to both fields.

Wood modification by impregnation

Wood modification by polymer impregnation has already been established as a way to adjust and tune different properties directly at the molecular scale in the cell wall. With these changes in properties and their potential benefits, the process of modifying wood by polymer impregnation for practical structural applications can be thought of overall as a material design problem. The impregnation process yields treated wood with customised or designed properties for enhanced use and new applications. There is a relatively large body of research literature related to the general process of wood impregnation with liquid monomers, followed by in-situ polymerisation by exposure to heat or radiation. The following section begins with a brief review of recent literature on wood-polymer impregnation, followed by mechanical test results of small clear samples of Norway and Sitka spruce. While the main research focus of this thesis is Sitka spruce, the two spruce species are notably similar and often described interchangeably as 'whitewood' in industry. In the early stages of research, while making contacts and arrangements with UK industry representatives for UK-grown Sitka spruce, C16-graded Norway spruce, imported from Sweden and sold by most UK timber suppliers, was used for preliminary testing. Furthermore, existing impregnation research in the literature includes work on Norway spruce rather than Sitka spruce, as the former species is commonly grown in many European countries, while the latter is the major softwood species grown in the UK. Following the results from small-scale mechanical testing, a discussion is given on the limitations of polymer-impregnation for full-scale applications, highlighting areas for further research efforts and development.

Recent literature on wood polymer impregnation

One of the most important issues in the early literature on impregnation and in practice is the interaction between polymer and wood, or often the lack of interaction. When referring to early studies on wood impregnation, Bodig and Jayne's text on the mechanics of wood-polymer composites emphasised that such interactions and the resultant material properties can be governed by a rule of mixtures.[3] In such cases, the increase in mechanical properties of a wood-polymer composite are linearly proportional to the amount of polymer impregnated into the wood, usually in the empty lumen of the wood rather than the cell wall itself. When wood-polymer composites follow the rule of mixtures, their resultant properties can also be predicted by simple calculations. More recently, however, Dinwoodie discussed some limited evidence for the possibility of going above and beyond the rule of mixtures, resulting from direct molecular interactions with polymers inside the cell wall.[4] Dinwoodie also reminds readers of the limitations of impregnation, and especially how the process increases the cost of wood by roughly three to four times.[5] It is perhaps interesting to consider how impregnation is framed only in terms of cost, rather than also adding value. This observation suggests that without establishing a strong market need and application, value-added processes like impregnating wood might be perpetually seen as cost-adding processes.

Since Dinwoodie's publication, Rowell,[6] Hill,[7] and Ansell[8] have performed more detailed reviews on wood modification and impregnation. Hill defines wood impregnation more strictly as processes that actually impregnate the cell wall, rather than those that only fill the voids and lumen of the wood.[9] Impregnation with resins, Furfuryl alcohol, monomer mixtures, silicon compounds, and polymers are discussed at length in these reviews, including different impregnation process parameters like vacuum and soaking times, and various temperatures for initiating reactions and polymerisation. For almost all previous investigations, the dimensional stability and anti-swelling efficiency (ASE) of impregnated wood samples are the important parameters of interest. Hill notes that several processes can further increase the decay resistance and MoE of wood.[10] Special attention is also given to how well processes can penetrate and lock impregnates into the depths of the cell wall, not only to create interactions and enhancements in material properties, but also to prevent any leaching out in service. Two key issues that prevent commercialisation, however, are scaling factors and the toxicity of ingredients in some impregnation processes and studies. For example, at the present time, the only commercially developed impregnation process uses Furfuryl alcohol, even though the resultant impregnated wood can gradually emit small levels of volatile organic compounds (VOCs) into the air, and especially in fires.[11] Although conclusions indicate that wood impregnated with Furfuryl alcohol does not pose a significant environmental risk in standard construction practice, materials that emit even small amounts of VOCs are not encouraged in stricter construction standards. Regardless of the benefits in material properties from a particular impregnation process, the potential for commercialisation is therefore severely limited by the use of toxic chemicals.

Preliminary testing in four-point bending

The most common monomer in previous impregnated wood studies is methyl methacrylate (MMA). MMA is commonly available and inexpensive, widely used in various plastic materials, and can also be combined with other monomers.[12] Accordingly, before trying any tests with new or custom-synthesised monomers, the first small clear wood samples tested for this thesis project were impregnated with MMA. The general steps of the impregnation and polymerisation processes are illustrated in Figure 3.1. Following the polymerisation process, wood samples measuring 7x7x70mm were conditioned for seven days at 24.0±0.5°C and 45±10% relative humidity to give an estimated equilibrium moisture content of 10%. Although conditioning at 20°C and 65% relative humidity, corresponding to a 12% equilibrium moisture content, is widely used in almost all wood standards, a conditioning chamber with this temperature and relative humidity was not available at the time of preliminary testing. Nonetheless, the difference in wood moisture content between 10% and 12% is estimated to increase MoE results by a relatively small amount of about 2-4%.[13]

Following conditioning, wood samples were tested in a basic four-point bending configuration. As UK-Sitka spruce is a stiffness-limited material, the MoE in bending was a parameter of interest in comparing samples of untreated and polymerised small clear wood. In contrast to non-destructive stiffness-based testing, destructive strength tests in bending and tension that induce failure are inappropriate for small clear wood samples due to scale effects with the wood's defects and grain.

Figure 3.1: Overview of process steps for wood impregnation and polymerisation. (Adapted from Janeček)

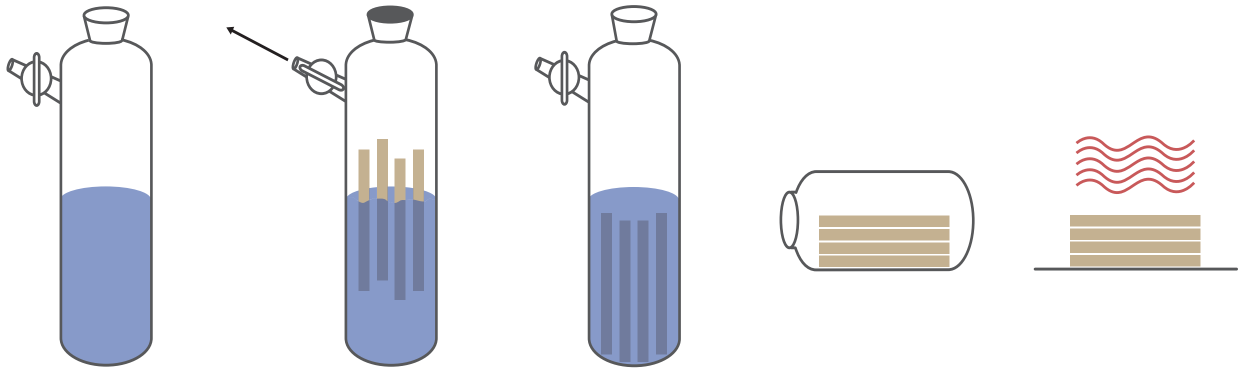


Figure 3.2 shows preliminary MoE results from the four-point bending tests, with untreated wood and wood impregnated with MMA under different vacuum and soaking conditions. These MoE results are plotted against the percent mass increase of the impregnated and polymerised wood. The percent mass increase is defined here as simply the percent difference between the mass of the final polymerised wood and the original untreated wood, taking the untreated wood mass as reference. Apart from the first treatment condition, labelled as R-A, which yielded a surprising loss in mass instead of an increase due to impregnation, there are several problems with these preliminary results that are immediately apparent, especially in the wide scatter of the data. Considering only the scatter in MoE along the vertical axis of the figure, even the natural, untreated wood samples alone lie in a range from roughly 4-16kN/mm^2. In terms of strength classes, this range covers C14 to C50. The standard deviation in the MoE of this control group is also roughly twice that of published results from Lavers and the BRE.[14] The small 7mm cross-section dimension of the wood samples is one cause for the wide range in the data. The 7mm width and height of the samples is on the same order of magnitude as the width

of the annual growth rings of the wood. Figure 3.3 shows the end grain of the 20 untreated control samples, where some samples had as many as four bands of high-density latewood, whereas others had as few as one and were predominately composed of low-density earlywood. The radial and tangential directions of each sample were also skewed with respect to the direction of loading, introducing further vertical scatter in the results due to the orthotropic nature of wood.

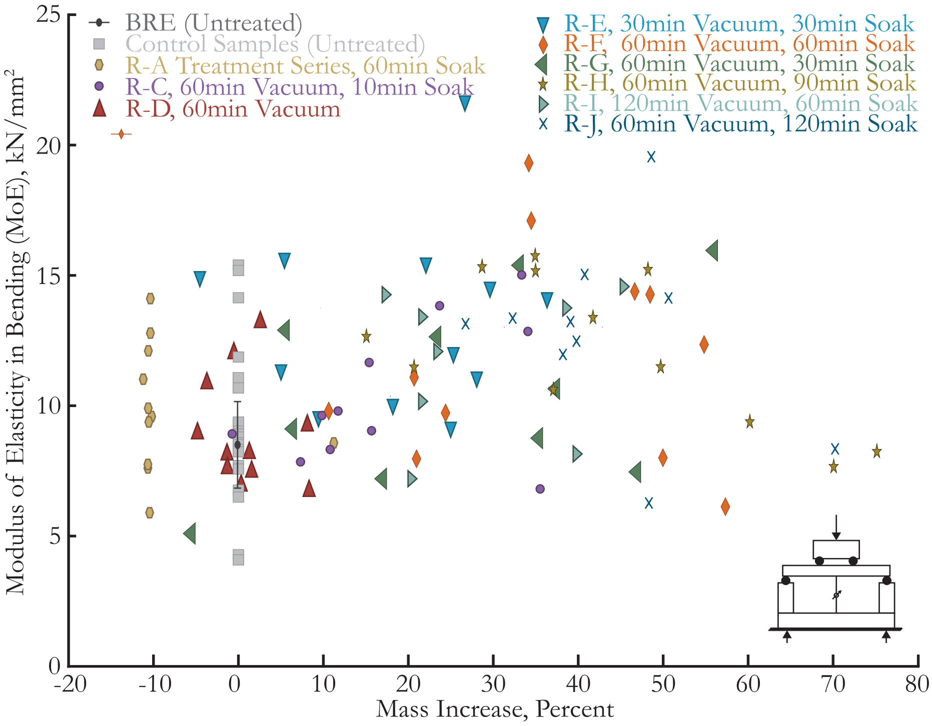


Figure 3.2: Preliminary results for MoE in bending for untreated wood and MMA-impregnated samples. (Mass data courtesy of Janeček)

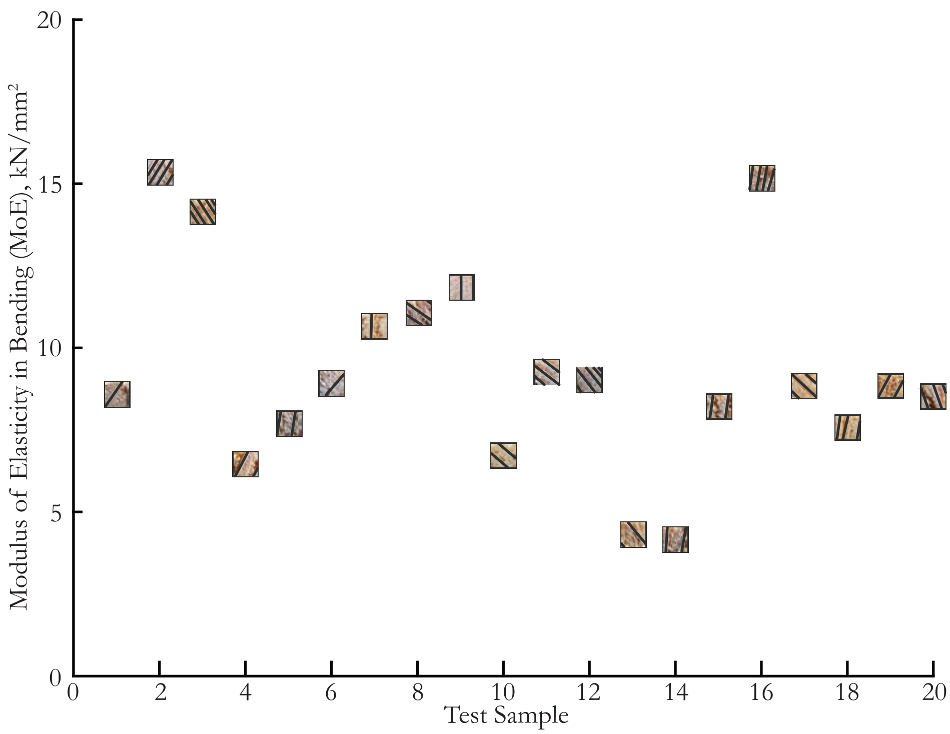
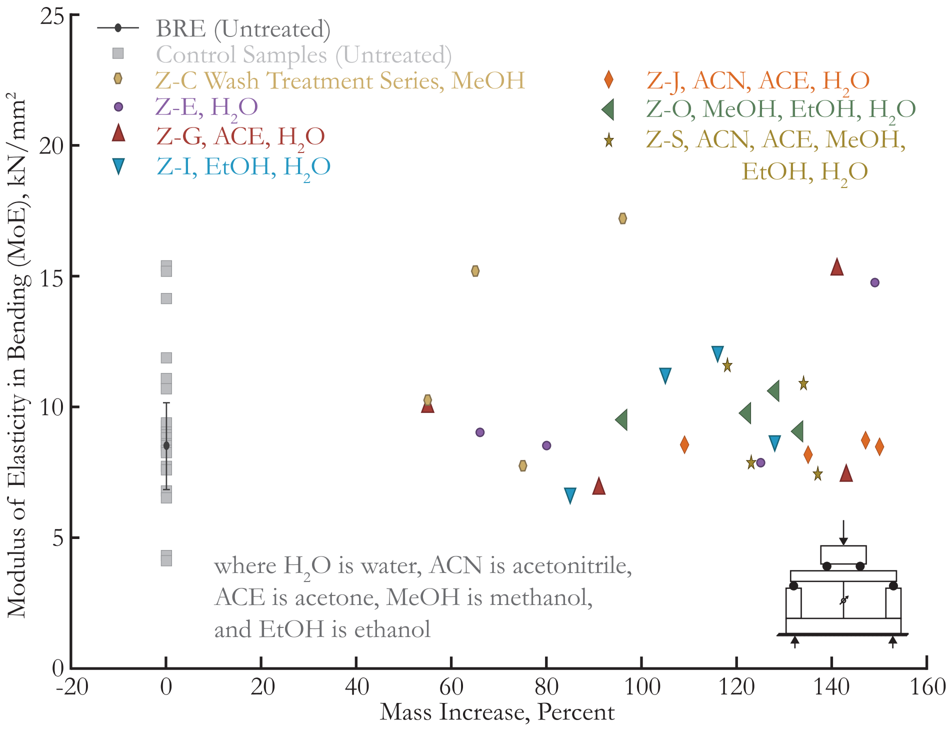


Figure 3.3: Preliminary results for MoE in bending for untreated wood showing end grain of samples.

Similarly wide scattering is also seen in the preliminary test data along the horizontal axis in Figure 3.2. For example, the treatment condition labelled R-F that is shown in orange, yielded some impregnated wood samples that increased in mass by as little as 20%, or as much or more than 50%. The small 7x7mm cross-section of the samples and the noticeably different proportions of early and latewood may be partly responsible for these variations in monomer uptake and final mass data. This scatter in the mass data also points to flow problems and raises serious questions regarding limitations in depth of penetration or the ability to uniformly and consistently impregnate even a small group of relatively small wood samples, let alone wood at full-scale. In an attempt to shed some light on these potential penentation and flow issues with getting the monomer in the wood, the author's chemistry collaborators performed additional impregnation trials with an initial washing procedure, with the results presented in Figure 3.4. Although only four samples of wood were tested for each washing treatment, the different conditions generally show that washing the wood and removing extractives before impregnation can result in higher mass increases after polymerisation. A possible explanation for this behaviour is that the minerals and nutrients making up the extractives hinder flow, especially around pits.[15] While the horizontal scatter of the data in Figure 3.4 is still significant, however, there are some patterns and trends suggested between some of the different washing processes. For example, the Z-C wash with methanol overall resulted in an significantly lower polymer mass increases than the Z-J and Z-S washes. The Z-S wash process that yielded the largest mean mass increase of over 130% also resulted in the MoE of the polymerised wood showing relatively lower scatter than other conditions. Once passing a 100% mass increase, there now is more polymer than wood in the sample, at least on a mass basis, and the polymer properties begin to become more dominant. In almost all cases in Figures 3.2 and 3.4, however, any potential gains in MoE from impregnating and polymerising the wood, either with or without washing, are difficult to identify. These preliminary results do not appear to follow a rule of mixtures and are inconclusive, raising further questions without providing many meaningful answers.

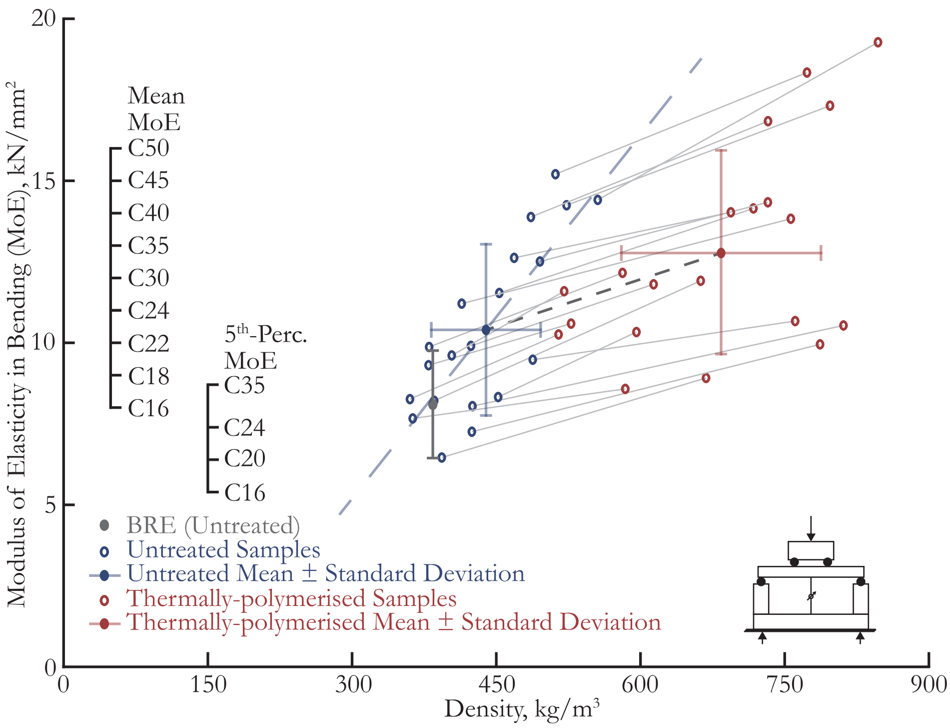
Figure 3.4: Preliminary results for MoE in bending for untreated wood and washed and impregnated samples. (Mass data courtesy of Walsh)



These preliminary 7x7x70mm wood samples were prepared according to specifications of the author's collaborators, with a resolve to maintain random sampling by cutting specimens randomly from larger pieces of dimensioned timber. Random sampling may be commonplace in laboratory work to avoid biases, but without understanding the growth of trees and the basic mechanics of wood, random sampling can also introduce random error and needless uncertainties. Furthermore, the small 7mm cross-section of the samples, along with their 70mm length giving a 1:10 height-to-length ratio, was chosen to comply with existing chemistry laboratory glassware for the vacuum-based impregnation process. The recommended minimum cross-section in standards for mechanical testing of small clear wood samples is 20x20mm, with a minimum length of 300mm for a height-to-length ratio of 15 to minimise shear effects. Testing standards also require that the loading force be directed along the tangential direction, so that the growth rings are aligned rather than skewed with the edges of the samples.[16] Despite these recommendations, larger samples measuring 20x20x300mm were not feasible for preliminary testing, not only due to the size of the glassware available for the impregnation process, but also the amount of monomer solution that needed to be prepared for impregnating 10-20 samples of this larger size. As a compromise for future testing, the author proposed a simple method of testing each sample in elastic bending before impregnation, as well as after the impregnation and polymerisation treatment. Each individual sample could be tracked during the impregnation and polymerisation processes, and then retested in the same orientation, thereby minimising variations due to grain angle and cross-section size.

The results from additional four-point bending tests conducted before and after the impregnation and polymerisation processes are shown in Figure 3.5. The impregnation process for these samples followed the R-J condition from Figure 3.2 in an attempt to maximise monomer uptake, without going past a kind of point of diminishing returns, as seen with other conditions in Figure 3.4 where mass increases could reach well over 100%. Lines are drawn between pairs of data points in Figure 3.5 to show how the MoE of each individual sample changed due to impregnation and polymerisation. These results are also plotted against density rather than percentage mass increase in an attempt to verify any possible relationships between MoE and density rather than MoE and mass increase. For instance, untreated MoE data in Figure 3.5 follow an approximate but linear trend that increases with density or mass, as expected. In contrast to comparing mean and standard deviation values for different groups of samples, the MoE values from before and after the treatment processes also provide a much clearer indication of how the impregnation and polymerisation is increasing the wood's MoE in bending. The slope of the lines that connect pairs of data for each individual sample before and after treatment is surprisingly similar. These results show that the MoE of each sample of wood has increased by just over 20%, or in terms of strength classes, from C22 to about C32 for mean values. The slope or rate of increasing MoE against density due to polymerisation, however, is much less than the slope of the linear regression line fitted to the untreated MoE data. The impregnation and polymerisation process may increase MoE, but does so at a much less efficient rate than in nature, at least on a density basis. Wood modification by impregnation therefore may be an effective strategy for improving the properties of low-grade wood, but not a very efficient one without more research. Finally, the wide scatter of the polymerised mass data also still suggests depth of penetration and consistency problems due to non-uniform impregnation phenomenon. Testing at larger scales is strongly recommended to better assess potential limitations in depth of penetration and the uniformity of impregnation.

Figure 3.5: Results from additional four-point bending tests conducted before and after treatment.

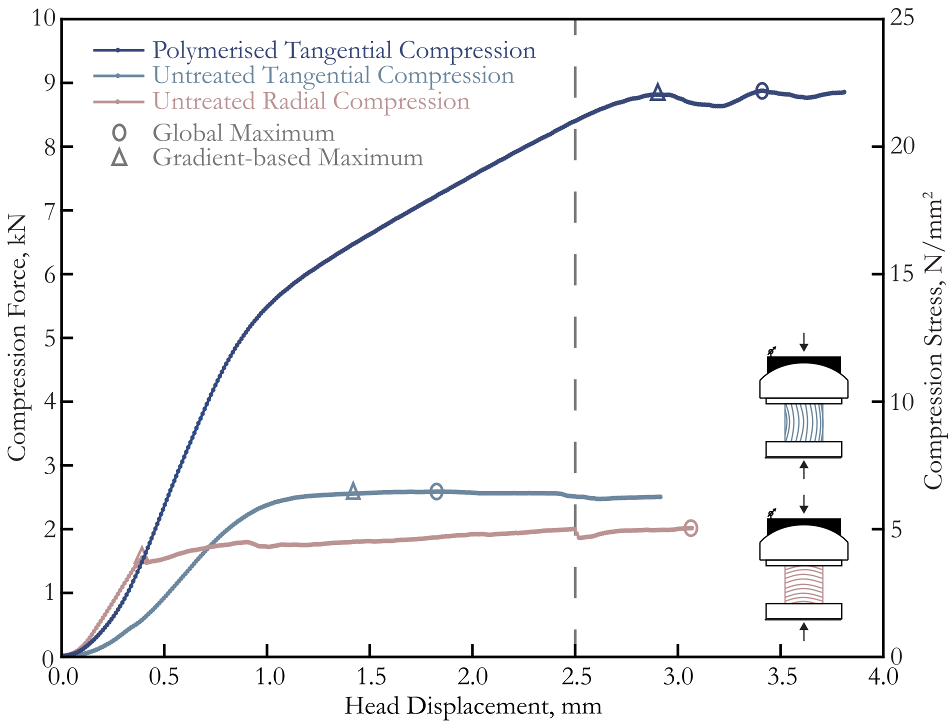


Compression strength testing

In an attempt to scale up the impregnation and polymerisation towards the recommended sample sizes in wood testing standards, 20x20x20mm samples of Norway spruce were treated for compression testing perpendicular-to-grain. Moving to a 20x20mm cross-section is an important intermediate step before scaling up to larger bending tests at 20x20x300mm, or even full-scale dimensions. Compression strength perpendicular-to-grain is also a relevant material property in many design situations. For example, in multi-storey platform construction, where solid floors of CLT, *Brettstapel*, or glulam are placed directly on top and underneath structural walls, the wood at the floor and wall joint is loaded in compression perpendicular-to-grain. Compression strength perpendicular-to-grain can also become a limiting material property for tall timber buildings.[17] The material property is therefore an interesting and relevant parameter to consider enhancing through impregnation and polymerisation.

Figure 3.6 shows the preliminary results of strength testing with two untreated wood samples and one impregnated and polymerised wood sample, all tested in compression perpendicular-to-grain. To quickly assess the potential of impregnation and polymerisation to improve compression strength perpendicular-to-grain, only three samples were tested here in the first instance following the same R-J impregnation conditions from Figure 3.2. The untreated samples, tested in the radial and tangential directions, show the typical plastic failure with a gradual crushing of the sample at a compression stress of about 5N/mm². Here, the compression strength in the tangential direction is slightly higher than that of the radial direction due to the reinforcement provided by the latewood. The initial polymerised sample, however, failed at a compression stress just over 20N/mm², or approximately five times that of the untreated sample tested in the tangential direction. This initial result may seem promising, but is actually only comparable to recent developments in timber engineering now found in practice; using self-tapping, axially-loaded screws in platform construction can also increase the perpendicular-to-grain compression strength of wood by similar margins of about four to five times.[18] For impregnation and polymerisation to become feasible for practical use, the improvements in material properties must go further and far beyond those in current practices[19] of reinforcing and locally strengthening wood with self-tapping, axially-loaded screws.

Figure 3.6: Preliminary test results for compression strength perpendicular-to-grain.



Subsequent compression strength testing in the perpendicular-to-grain directions was performed to confirm the previous result and provide more data. An inexpensive and small-scale conditioning chamber, converted from an old refrigerator, was also prepared at this time and used to properly condition the compression samples at 20.0±0.5°C and 65±5% relative humidity. These conditions give the wood samples the recommended equilibrium moisture content of approximately 12%, before both impregnation and testing. After the impregnation, polymerisation, and reconditioning processes, however, the treated samples stabilised to an equilibrium moisture content of only about 8-10%, as summarised in Table 3.1. The stable mass is defined here as the equilibrium mass of a wood sample that remains within a margin of ±0.01g over a 24 hour period. The lower equilibrium moisture content of the polymerised wood after reconditioning suggests that the impregnation and polymerisation processes are most likely enhancing the dimensional stability of the wood. This issue remains open for more detailed investigations on the moisture and movement characteristics of these polymerised samples.

Table 3.1: Stable conditioned mass of samples before treament, after polymerisation, and before testing.

Sample	Stable conditioned mass before treatment, (g)	Dry mass following polymerisation, (g)	Stable conditioned mass before testing, (g)	Estimated moisture content before testing, (%)
D1	-	-	3.54	12
D17	-	-	3.69	12
E20	-	-	3.60	12
F12	-	-	4.10	12
D3	-	-	3.69	12
E18	-	-	3.59	12
F10	-	-	4.04	12
F14	-	-	4.08	12
D2	3.68	7.37	7.67	9
E8	3.57	7.33	7.63	10
E21	3.61	7.33	7.63	9
F13	4.07	7.66	7.96	8
D18	3.58	7.16	7.46	10
E19	3.54	7.22	7.51	9
F11	4.08	7.56	7.88	9
F15	4.23	7.55	7.89	9

Figure 3.7 illustrates the results of subsequent perpendicular-to-grain compression strength testing, with samples modified according to the R-J impregnation conditions, but with an additional and preceding washing process following the Z-J condition from Figure 3.4. The washing process was introduced here to intentionally maximise the density and mass increase of the polymerised wood samples, with the hypothesis that the compression strength perpendicular-to-grain could be further increased. The results in Figure 3.7 confirm that washing the wood before the impregnation process is beneficial for further enhancing the perpendicular-to-grain compression strength of polymerised wood. The polymerised wood shows an increase in strength by a factor of roughly ten times compared to untreated or natural wood. The perpendicular-to-grain compression strength of many of the treated samples even surpassed that of common grades of concrete of about 30N/mm^2. These results are promising from a purely mechanical perspective, but additional work is needed to better understand the interactions between the polymer and wood cell wall and how the

washing, impregnation, and polymerisation processes can be further exploited. Advanced compression testing with an NIR-based technique for 50x50x50mm samples of UK-grown Sitka spruce was originally planned for this purpose. After a preliminary test, however, the NIR technique and testing was abandoned due to difficulties in obtaining reliable NIR transmission and reflection data.

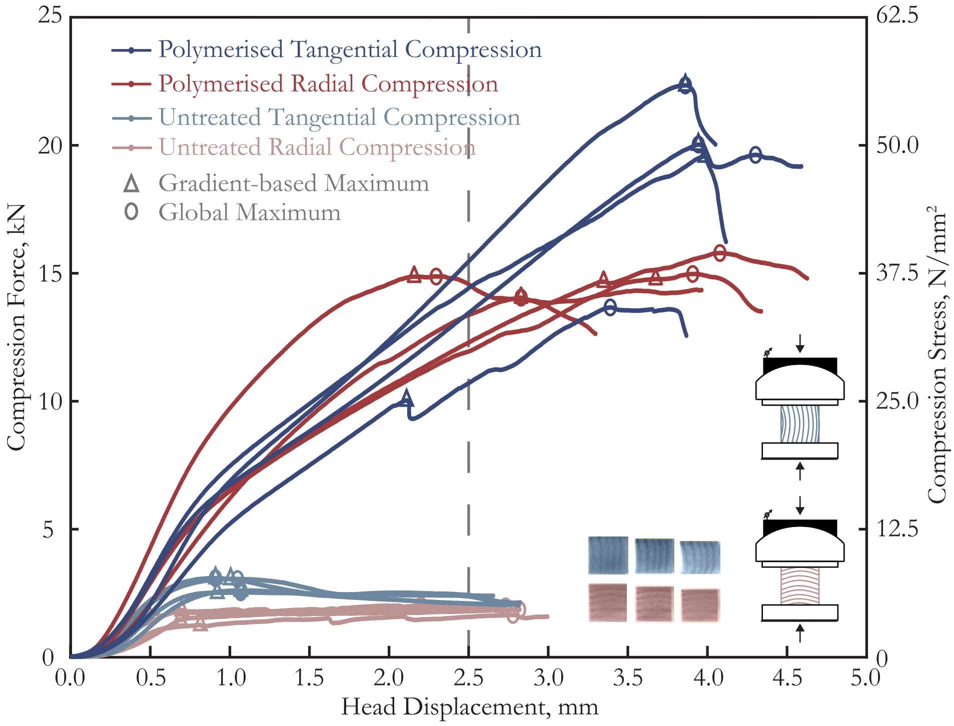


Figure 3.7: Test results for compression strength perpendicular-to-grain for untreated wood and MMA-impregnated samples.

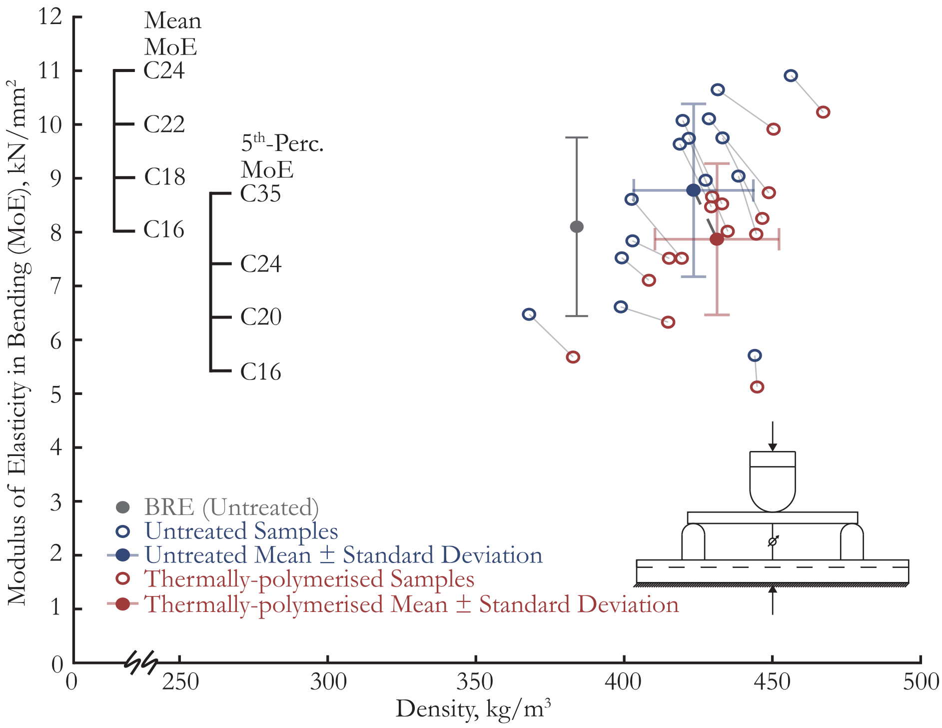
Scaling up and final testing in three-point bending

The results presented so far provide initial evidence for the ability of impregnation and polymerisation to modify and enhance the mechanical properties of low-grade wood. Trials are still needed with larger samples, and eventually, full-scale wood specimens. Full-scale trials are important for establishing confidence in the processing techniques for practical applications. They can also be used to study how the impregnated polymers might interact with glue-based lamination processes, or even large defects such as knots and cracks. For instance, the polymer may be able to reinforce the areas around defects, thereby increasing the strength of wood in bending. Unfortunately impregnating full-scale, dimensioned timber was deemed infeasible during the timeline of this thesis project. As a compromise, final testing with 15x15x225mm UK Sitka spruce samples were conducted after some older but also larger glassware was discovered in the chemistry laboratory. In parallel to this small discovery, impregnation and polymerisation trials for a separate fourth-year engineering undergraduate thesis also confirmed that Norway and Sitka spruce samples showed relatively similar increases in mass and shear strength following impregnation and polymerisation.[20] The polymerised wood showed similar increases in shear strength as the previous MoE results, with gains of about 20% in the shear strength of polymerised wood compared to untreated samples.[21]

While the previous 7x7x70mm samples were tested in four-point bending, these larger 15x15x225mm samples were tested in a new three-point test fixture that was developed for conducting tests according to BS 373:1957. These larger samples were also much more carefully prepared with the tangential and radial directions cut parallel with the samples' edges. The extra effort made in accurately preparing samples was done with the hope of achieving more consistent results with less scatter, thereby providing a clearer picture of the improvements from polymerisation. These larger samples were expected to yield more consistent results, but the same before-and-after testing routine was still followed as an extra precaution. Instead of impregnating the samples with common MMA monomers, a new custom-synthesised monomer mixture with ionic liquids was also tried in the impregnation process. The custom monomer mixture was designed with certain ionic liquids that are known to slightly dissolve cellulose,[22] thereby providing new opportunities for stronger molecular interactions between the wood's cell wall and polymers following the polymerisation process.[23]

Figure 3.8 shows the MoE results of the samples before and after the treatment processes, where the vertical standard deviation bars of the natural MoE data almost coincide with published results from 20x20x300mm tests.[24] A minor but distinct decrease, however, in the MoE of all of the samples following impregnation and polymerisation can be readily seen. These negative results suggest that the initial changes to the wood's cellulose may be working as intended, but the final impregnation and molecular interactions between the polymers and cell wall are most likely performing less than originally hoped, if at all. Theses results are still useful, however, and show that even a small step up in scale with more carefully prepared samples can greatly reduce scatter, leading to more accurate results. This improvement in accuracy may allow a subtler mapping of the resultant gains in mechanical properties associated with different monomer mixtures and impregnation conditions in the future.

Figure 3.8: Results from three-point bending tests conducted before and after treatment with a custom-synthesised monomer solution with ionic liquids.



Summary

Enhancing the properties of low-grade wood directly at the molecular scale through impregnation and polymerisation is a challenging task requiring concerted research efforts across different fields. While incremental increases in the MoE in bending of small clear wood samples can be achieved with low-grade wood, significantly more work is needed to establish a better case for polymerised wood in practice. Enhancements with gains in order of magnitude are needed, like those seen in preliminary compression strength testing. Reflecting back on this sequence of tests, a more systematic and perhaps more focused approach would have been beneficial. Publishing a cross-multidisciplinary literature review on the subject could help in the future with not only identifying potential challenges, but also providing a stronger quality of scholarship to future work. Chemistry issues can be examined with small-scale samples for efficiency and ease, with even smaller samples than those tested here. Once establishing a number of promising, custom-synthesised monomers and non-toxic mixtures, impregnation and flow issues can be considered incrementally by scaling up to larger sample sizes. In parallel to these chemistry and fluid areas, material science and structural engineering knowledge can be used to establish clearer guidance for relating how the small molecular scales can influence changes in macroscale properties and interactions with defects. Finally, when returning to Figure 1.6 and the primary processing of wood by sawing, peeling, or stranding, work so far has focused exclusively on impregnating only dimensioned and graded timber. Significantly smaller veneers and strands can also be considered, and may help bypass some challenging uniformity and flow problems at full scale. This work remains to be considered in the future by the larger research group.

Initial testing of stress-laminated models

Stress lamination is a tried and tested technique for the construction of the bridge decks, but is also useful for building construction with low-grade wood. One reason why stress lamination has not been fully developed already or extended beyond simple flat decks and arches is because of the earlier development and popularity of glulam in many countries. UK designers and structural engineers, understandably, have found it more convenient to specify higher-quality and imported glulam than to develop alternatives with low-grade wood grown in the UK. With the predicted increase in UK forestry, this convenience comes into question and opens up opportunities for new ideas and development in areas like stress lamination. Ongoing research based in Sweden continues to show how basic improvements are still possible in both conventional stress lamination construction for bridge decks and the modelling of their structural performance.[25,26] Flat decks could easily be introduced into building construction as floor plates for one-way spans. Just like dowelled *Brettstapel* floor panels, stress-laminated floor panels would be able to achieve greater span-to-depth ratios and structural efficiency than CLT in one-way spans. In addition to horizontal floor elements, stress lamination would also be beneficial for vertical elements like columns and shear walls. As a precursor to full-scale construction and testing, this section presents a study of scaled models and associated drawings of different structural building elements that can be achieved with stress lamination, including much more complex arrangements than flat decks.

Unrealised potential of stress lamination in buildings

Stress lamination in its most basic form works by threading timber planks or boards on edge, onto transverse steel prestressing rods or bars. Prestressing bars are usually 25-30mm in diameter, spaced at distances of 600-1000mm on-centre.[27] Once assembly is completed, with all the boards in place, the bars are tightened with post-tensioning jacks to provide friction forces between the individual boards in lieu of glue or adhesive. When a point load is applied on the surface of a stress-laminated bridge deck, such as a tire of a heavy truck, the load is shared and spread to adjacent boards through friction and shear forces. The compressive prestress also counteracts the lateral bending of the deck. Compared to conventional glulam with large-scale presses and fixtures, the manufacturing of full-scale stress-laminated elements can be accomplished with minimal energy input and simple tools. Furthermore, stress-lamination construction can be performed off- or on-site, depending on accessibility and scheduling. The stress-lamination technique, however, due its adjustable and serviceable nature, also lends itself very well to realising renewable structures in practice. If designed and detailed properly for fire resistance and accessibility, stress-laminated elements could be much more easily maintained, serviced, and replaced compared to glulam or other mechanical lamination techniques like nailing. Prestress losses still remain a critical issue in stress lamination, arising from creep and wood movement with the material decreasing dimensions due to losses in moisture content. As prestressing elements like steel bars are effectively stiffer than the wood, the bars loose their tensile forces as the wood reduces in dimension and ceases to resist through compression. Prestressing and properly accounting for prestress losses is therefore an important technical issue to keep in mind when designing and building stress-laminated elements.

One of the first steps in constructing stress-laminated elements at either model-scale or full-scale is drilling holes in the individual elements for the prestressing bars or rods. Instead of using the bearing action of conventional mechanical fasteners like dowels in drilled holes with tight-fitting clearances, stress-lamination construction guides recommend oversizing holes to allow easier assembly.[28] The oversized holes do not affect structural performance, as prestressing and friction forces can provide comparable shear resistance without any bearing forces or dowel-action. An initial estimate illustrated in Figure 3.9 shows how with a moderate prestressing of 0.5N/mm^2, friction forces can engage a large area of material and provide similar structural resistance compared to bearing resistance in the parallel-to-grain direction. This prestress level is also far below the typical compression strength perpendicular-to-grain of 5N/mm^2 for Sitka spruce seen in previous tests. Working well within the elastic region with prestressing provides ample opportunity for reuse.

In the case of a dowel connection loaded in the perpendicular-to-grain direction, however, the Hankinson effect applies and describes a significant decrease in strength when loading wood at an angle with respect to the grain. The bearing forces around a dowel therefore can cause splitting. As an alternative, friction forces can be more effective in such cases, but can also result in rolling shear. Rolling shear failure occurs with shear forces in the radial-tangential plane, causing cells and growth rings to shear and roll over each other. On the other hand, the Hankinson effect and splitting arise as tension forces are applied in the perpendicular-to-grain direction, where the wood has is lowest strength due to the cellular nature of the material. In the general cases of the normal loading parallel or perpendicular-to-grain, the shear area for friction forces is significantly larger than the localised area of wood around a dowel that becomes engaged in bearing. This principle of distributing forces over a larger area in shear rather than a localised one in bearing is the basic mechanism of both stress lamination and glulam. Split ring and dogtooth timber connectors also work in a similar way by transferring forces from a dowel to an enlarged bearing area through a connector. Stress lamination is based on simple principles but is also general and flexible enough to allow a wide variety of different configurations.

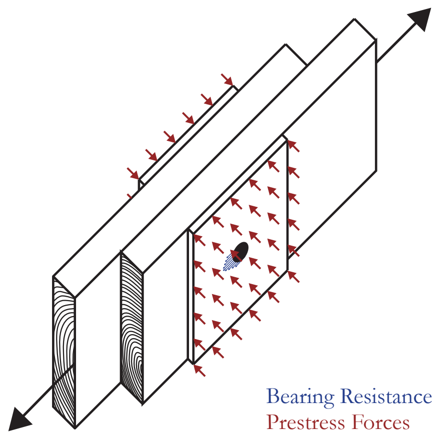


Figure 3.9: Illustration of the difference in area and material engagement between bearing resistance and prestressing against shear.

Flat panels, curves, fans, and twists with 1:10 models

Scaled structural models of wood are often limited due to scale effects associated with defects, but they can still play an important role for testing new design configurations and assemblies. The model study on stress lamination began with 1:10 models of a conventional flat panel or deck and a basic straight column. Threaded steel rods 2mm in diameter or 1.5mm diameter steel wire were used as scaled prestressing bars. Either small 12mm steel washers or acrylic pieces, laser-cut 10mm square, were used as scaled bearing plates at the edges of stress-laminated elements to prevent any localised crushing. In practice usually steel plates or steel channel sections are used at a panel's edges to distribute bar forces and prevent crushing. In the case of the models with steel wires, a custom-built jig for the scaled assembly and prestressing processes was also built to ensure the tension force in each wire was properly scaled, thereby achieving similar prestress levels in the model as in the full-scale case. Before fixing the copper wire ferule and completing the scaled prestressing process, the tension force in each steel wire was estimated using compression springs in the jig. Note that the compression springs in the jig were chosen with a relatively long length to minimise stiffness, thereby allowing deflection measurements to be made with a simple ruler. An improvement can be made with the use of shorter and stiffer springs for more accurate measurements with dial guages. Figures 3.10 to 3.13 show initial drawings, the jig for assembly and prestressing, and the completed 1:10 scale models of a straight floor panel or deck and a new kind of stress-laminated column. Compared to their state before prestressing, the completed models each felt like solid elements. The prestressing and friction forces from the lamination process appeared to work as expected, and the individual scaled boards could not be shifted or moved by hand relative to one another.

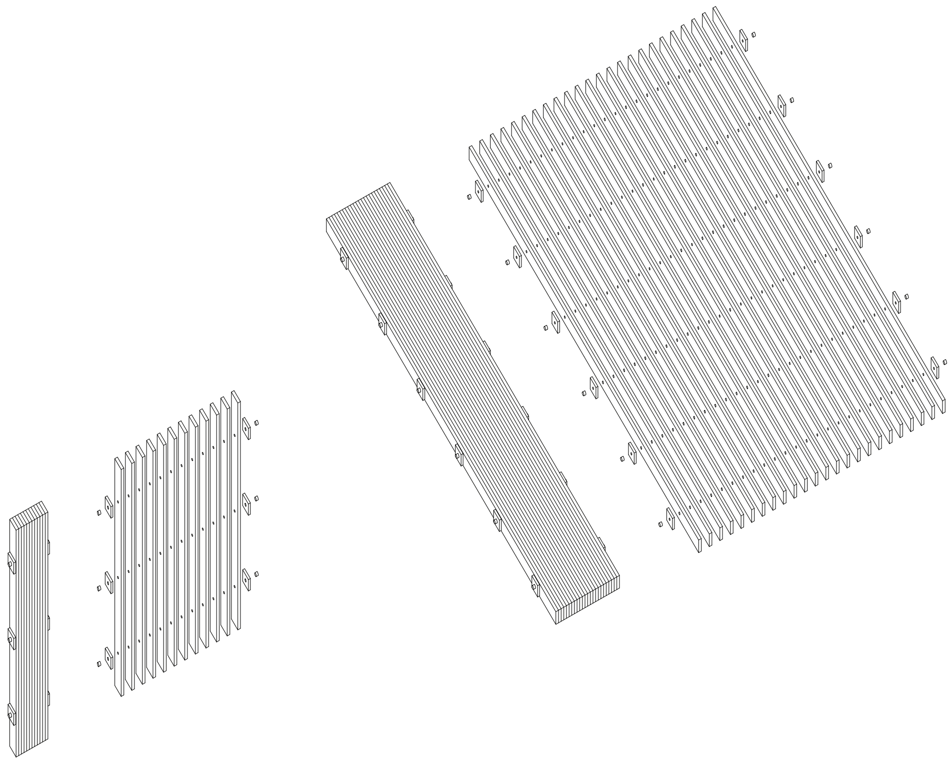


Figure 3.10: Drawings of a straight stress-laminated floor panel or deck and a new type of straight stress-laminated column.

Figure 3.11: Jig for prestressing and stress-laminating models at a scale of 1:10.

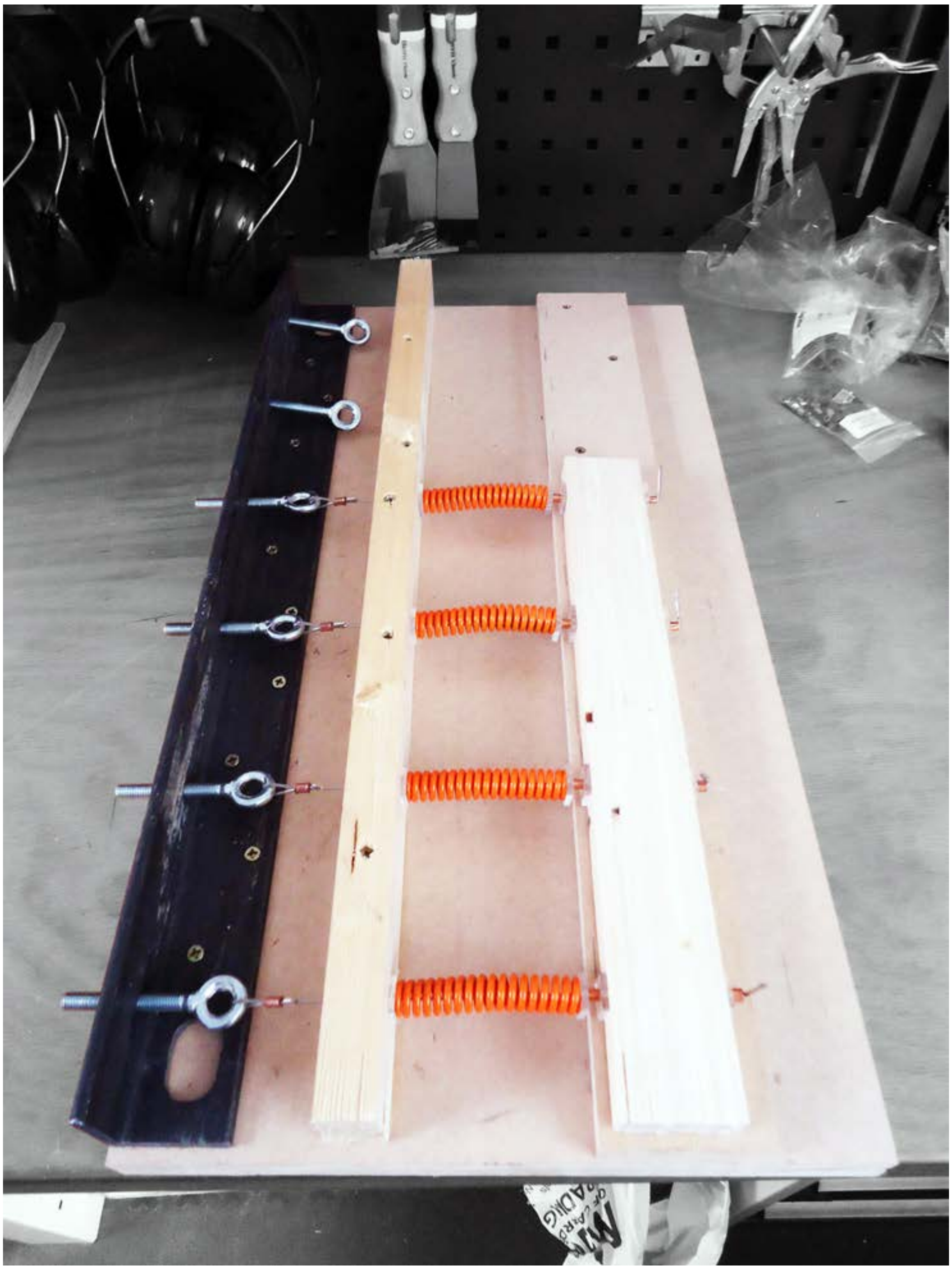


Figure 3.12: Model of a stress-laminated deck or floor panel at a scale of 1:10.

Figure 3.13: Model of a straight stress-laminated column at a scale of 1:10.

These basic straight elements in stress lamination can be extended to more complex three-dimensional forms with minor changes in the construction process, but resulting in potential gains in structural performance. Subtle offsets in the boards during assembly can increase a column's second moment of area and its resistance to bending and buckling. At the same time, the offset boards in a column-to-beam connection or wall-to-floor connection might also offer some bending moment capacity, thereby increasing frame-action through enhanced resistance to rotations. Figure 3.14 shows the drawings and a physical model of a curved stress-laminated column, where the holes in each individual board element are incrementally shifted to provide the overall curved profile. A simple template for offsetting the individual boards established the overall curve accurately during the assembly process. This curved variation highlights the simplicity of the stress lamination process but also its flexibility for creating new structural elements in wood.

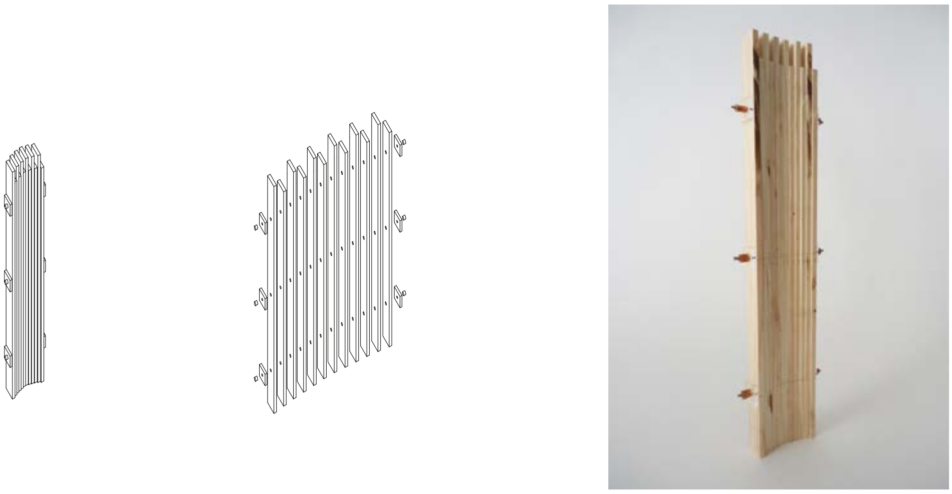


Figure 3.14: Drawings and 1:10 model of a curved stress-laminated column.

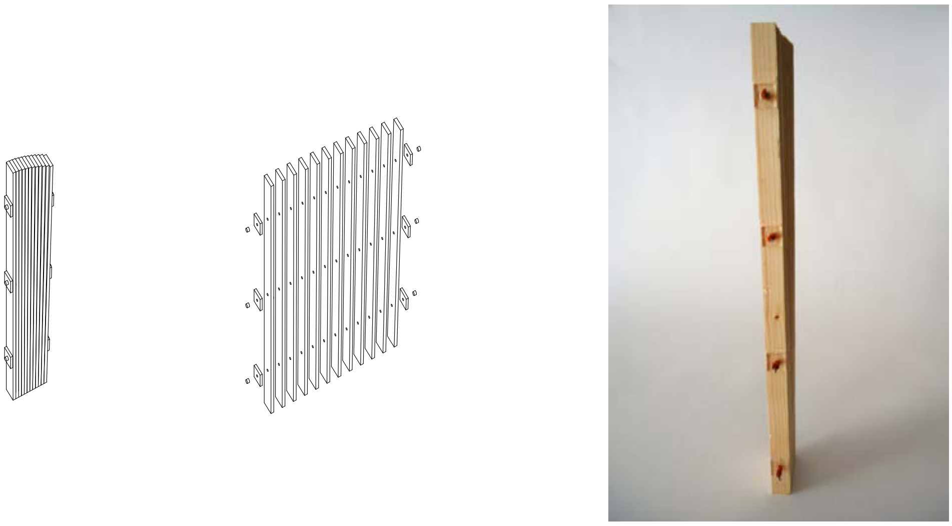


Figure 3.15: Drawings and 1:10 model of a fan-shaped stress-laminated column.

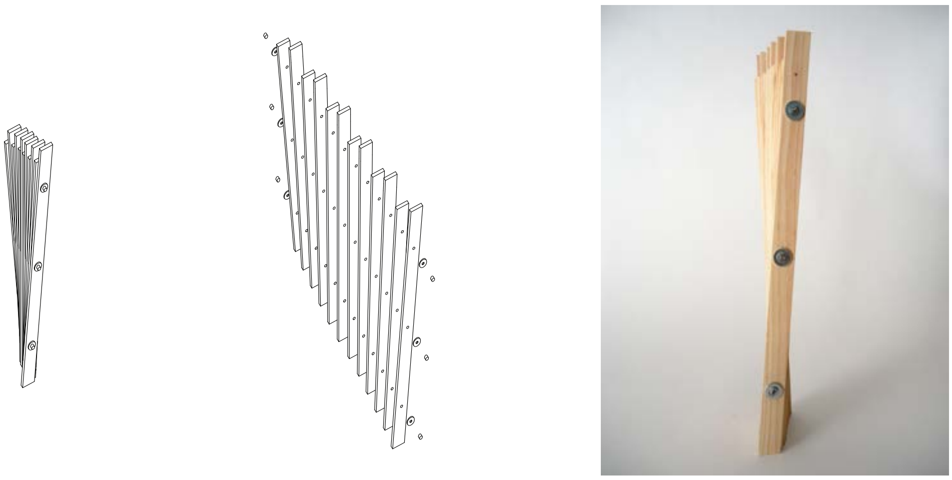


Figure 3.16: Drawings and a 1:10 model of a twisting stress-laminated column.

Even more complex fan-shape and twisting columns and shear walls can be achieved through simple changes in the stress lamination process. Instead of just offsetting the individual boards to form a curved column, adding subtle rotations to the individual boards about the bottom of the panel results in the non-prismatic, fan-shaped column shown in Figure 3.15. Changing the point of rotation to the centre of the column yields the complex twisting geometry seen in Figure 3.16, with a varying second moment of area along the column length for resisting buckling. These models show that offsetting and rotating boards by a small amount of 1-3mm in the models, or 10-30mm at full-scale, actually reduces the area for friction forces, making the prestressing forces essentially work harder to achieve the same level of shear resistance. Although the assembly and construction process of these relatively complex geometries is straightforward and simple with stress lamination, the final forms introduce noticeable challenges in other areas of construction, such as applying finishing and fireproofing materials. With more elaborate floor and wall details in basic platform construction, such as the example shown in Figure 3.17, the varying individual angles of the boards in a fan or twisting configurations are problematic at the larger construction scale. Combining the twisting concept with alternating straight boards might alleviate such construction concerns, as shown in Figure 3.18, while retaining the structural advantages associated with increased resistance to rotations in connections. Extending this new combination concept to multiple stories in Figure 3.19 shows how the upper section of the column's offset boards helps to support a beam or floor plate from underneath. The column above with its lower section of offset boards gives further rotational restraint to a connection. Testing and applying these lamination and construction concepts in an actual design process for a multi-storey timber building remains as future work and may yield further insights.

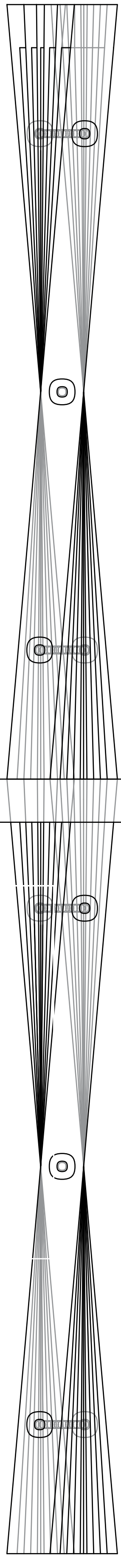


Figure 3.17: Drawing showing the varying angles present in a column-to-floor connection with twisting stress-laminated columns.

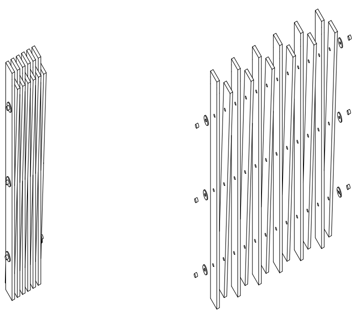


Figure 3.18: Drawings for a combination-type stress-laminated column with alternating straight and rotated layers.

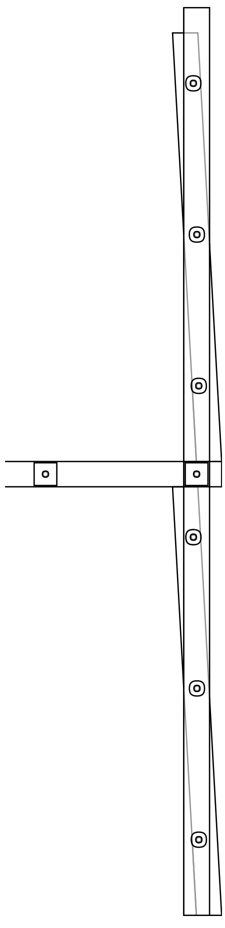


Figure 3.19: Drawing showing a potential column-to-floor connection with enhanced resistance to rotations from the combination-type stress-lamianted columns.

Testing of 1:10 and 1:5 model columns

These curved, fan-shape, and twisting models reveal the degree of complexity that can be achieved with stress lamination. In a critical light, however, these models also represent an overly elaborate structural formalism that could easily become too spatially dominant in a design. The gains in performance from simply offsetting or rotating layers can also be questioned. While structural gains can be made with more complex geometry, testing is first needed with simpler structures and forms that would be far more useful and commonly applied. Stress lamination has never been considered for columns or shear walls before, so backing away from these more complex configurations and returning to the simpler vertical straight models is merited for preliminary testing of scale models before full-scale trials. The first model tested was at the 1:10 scale, measuring 16x16x360mm and formed of four scaled Norway spruce boards 4x16x360mm in dimension. Threaded rods were used for the scaled prestressing bars, as they could be easily adjusted and readjusted in different preliminary testing trials. Figure 3.20 shows the model before and after a test trial. The results of testing the model until buckling at different prestress levels are shown in Figure 3.21, where the prestressing bars in each test was simply loose or 'finger' tight, 'medium' tightness, and 'fully' tight. This method and these labels are crude, as the tension force in the model's 2mm threaded rods was difficult to estimate, but the test results still provide important insights for relative comparisons before more detailed investigations. The test results show that large differences in prestress levels can strongly affect buckling performance. The final test results from fully-tightening the prestressing rods also compare surprisingly well with theoretical but conservative predictions of the Euler buckling load and the buckling load from unfactored or characteristic EC5 estimates. These estimates are conservative in several ways, even without factoring, as the mean MoE of 8kN/mm^2 was used instead of the characteristic or 5th-percentile MoE value of 5.4kN/mm^2. With the small size of the model, the mean MoE value was thought to better represent the material rather than the lower characteristic 5th-percentile value recommended for buckling calculations in the Eurocode for timber design.[29] The agreement between the test results and the predicted buckling loads implies that the stress lamination is performing similar to glue-based lamination, with full-composite action over the entire column section. This performance and the influence of spacing the prestressing elements can be further studied in more detail at a larger scale.

Figure 3.20: Before and after preliminary compression testing of a 1:10 model of a stress-laminated column.

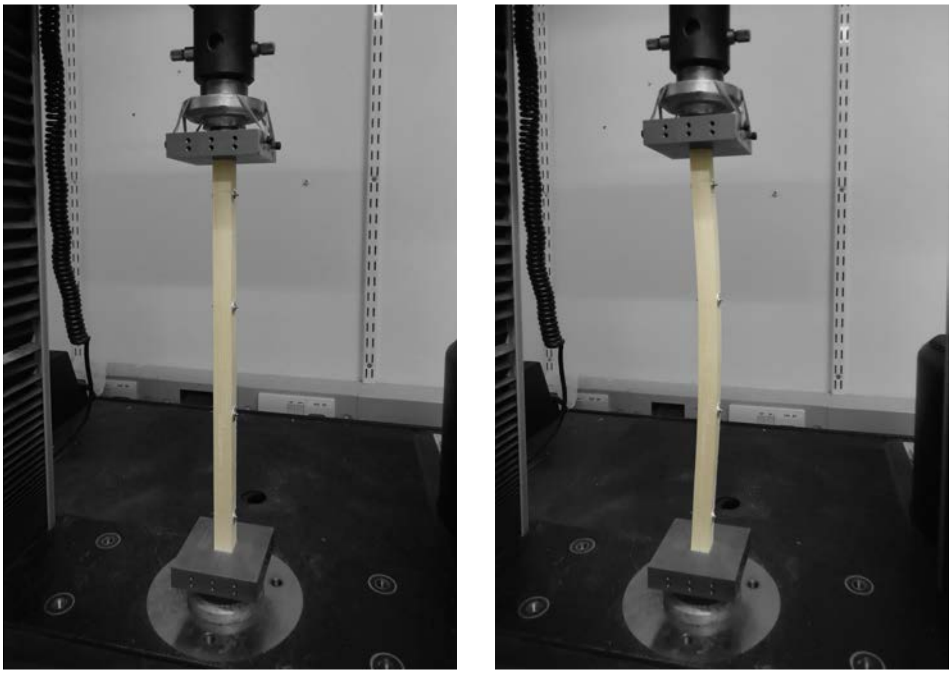
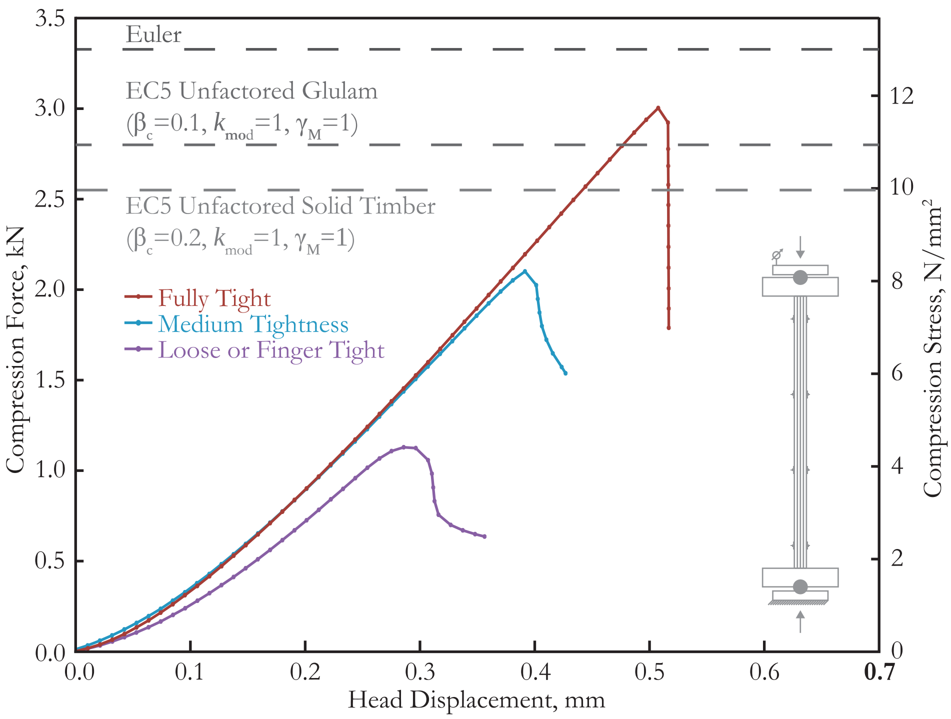


Figure 3.21: Compression test results for a 1:10 stress-laminated model, showing the influence of prestressing on performance.



Before moving up to working and testing at full-scale, an additional straight column model at the 1:5 scale was built and tested with UK-grown Sitka spruce, as shown in Figure 3.22. M6 (6mm diameter) stainless steel threaded rod was used for the scaled prestressing elements. Belleville or conical-disc spring washers were used on both sides of the prestressing rods as a way to try to estimate tension force, although the small deflections of the washers proved difficult to measure accurately. Note that Belleville washers are also one proposed way to control prestress losses in stress-laminated bridge decks at full-scale, which will be discussed in detail later on. The results of testing the 1:5 model, as shown in Figure 3.23, with loose or finger-tight prestressing elements follow a similar poor performance as in the 1:10 model testing. The individual boards in the section are not adequately laminated and cannot transfer shear forces effectively. Additional trials with fully tightened rods at different spacing of 100 or 200mm, corresponding to 500 or 1000mm at full-scale, also showed markedly different behaviour. Note that the trial with prestressed rods spaced at 100mm also included an extra prestressing rod at each end of the model, to provide extra shear resistance where shear forces are expected to be the greatest in magnitude. While the performance with the 200mm spacing was in good agreement with unfactored EC5 characteristic buckling estimate for glulam, the 100mm spacing exceeded the Euler buckling curve and the C16 characteristic or 5th-percentile compression strength parallel-to-grain of 17N/mm^2. The perpendicular-to-grain compression effect provided by the prestressing elements may also reinforce against compression failure parallel-to-grain, as the strength parallel-to-grain is also influenced mechanically by a buckling phenomenon at the scale of the cell wall.[30] This effect can be further studied with solid sections, but in the case of lamination, the friction forces provided by the prestressing is far more dominant and of more practical interest.

Figure 3.22: Before and after compression testing of a larger 1:5 model of a stress-laminated column.

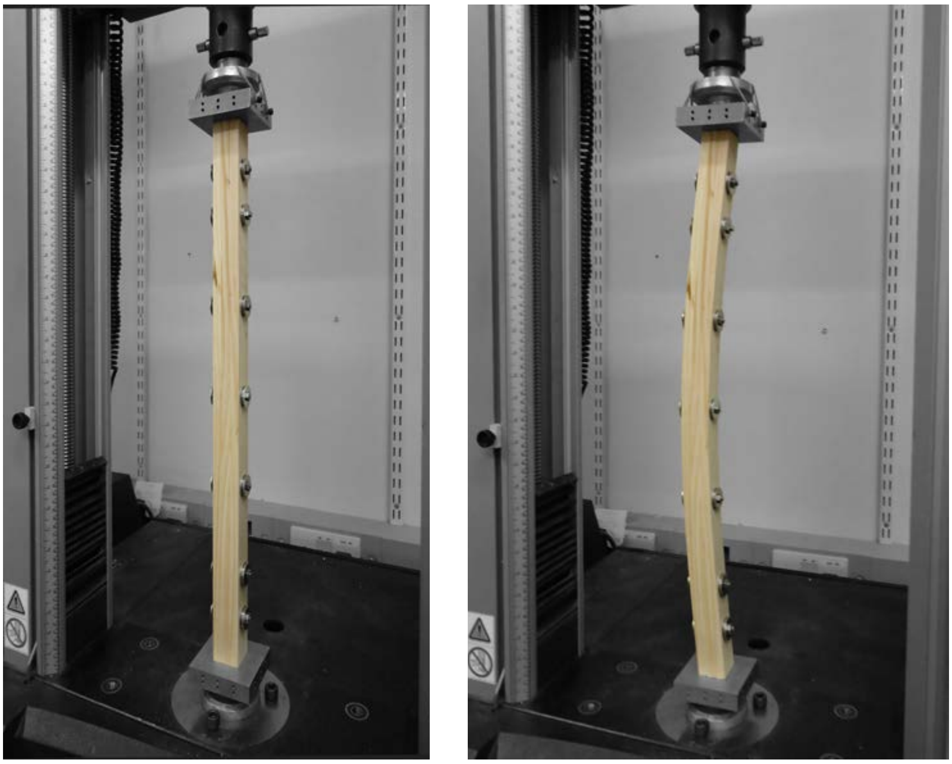


Figure 3.23: Compression force versus head displacement test results for 1:5 models of stress-laminated columns showing the effect of different spacings for prestressing elements.

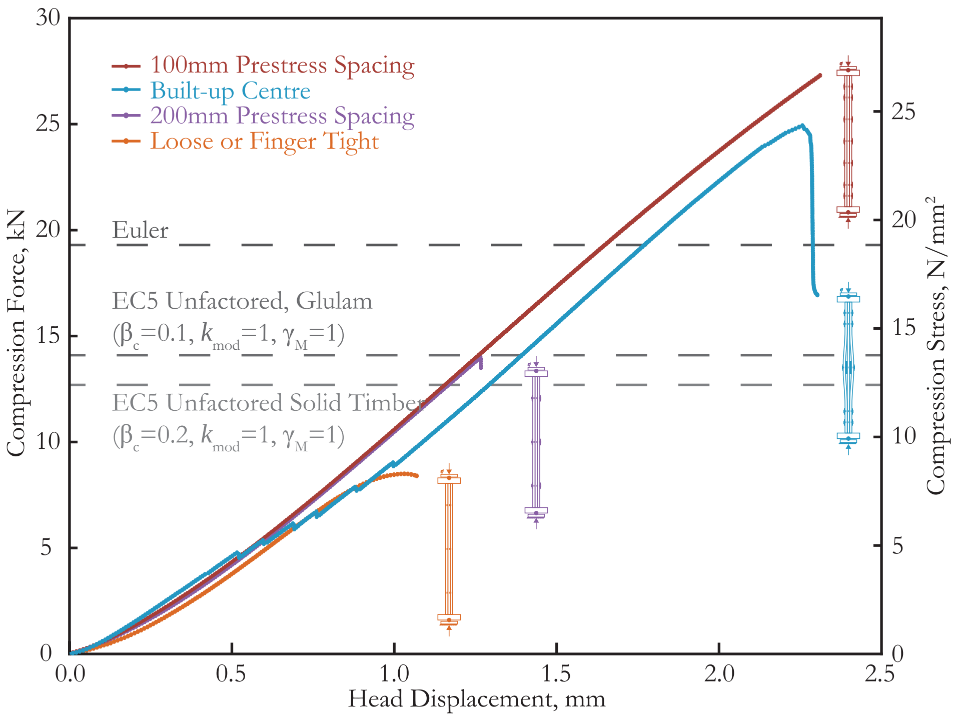


Figure 3.24 summarises the test results from both the 1:5 and previous 1:10 scale models. The data and Euler buckling curve and unfactored EC5 characteristic buckling curves are plotted in the conventional way with compression stress against slenderness ratio. The non-dimensional slenderness ratio, λ, is defined as the ratio of a column's height, *h*, to its radius of gyration, *r*, where the latter is the root of the second moment of area, *I*, over the column's cross-sectional area, *A*. Equation (3.1) defines the slenderness ratio in mathematical form:

$$\lambda = h / r = h / \sqrt{I / A}\,. \tag{3.1}$$

For the 1:10 model case, its slenderness ratio is approximately 78, and the 1:5 model has a slenderness ratio of about 65. In general, note how the Euler buckling curve does not take into account a material's finite compression strength, and that EC5 buckling curves vary and deviate away from the ideal Euler curve at lower slenderness ratios, in various degrees depending on the value of the straightness factor, β_c. Following Eurocode recommendations, for solid timber columns, the straightness factor is taken as 0.2, and for glulam, 0.1. The partial safety material factor, γ_M, is set to 1 in all cases, although in practice for full-scale cases, the EC5 design curves are noticeably lowered with γ_M equal to 1.3 for solid timber and 1.25 for glulam, as the latter reduces variability in material properties through the lamination process. The duration of load factor, k_{mod}, is similarly set to 1 to provide unfactored and conservative estimates for these initial tests.

The result of a trial test of a novel split and curved column design, shown in Figure 3.25 with solid ends and a built-up centre portion with packs, is also included in the previous results. The performance of this curved, built-up column design did not reach that of the solid column with 100mm spacing, but still exceeded the estimated Euler buckling load. Two factors or principles are influencing this result: firstly, the increased depth and second moment of area of the centre portion resists bending and shifts the buckling mode to a higher, second-order mode shape as seen in Figure 3.25; secondly, the spacing of the individual boards above and below the built-up centre packs does not allow friction forces, so all shear forces are transferred through the column ends and the centre packs. The increased resistance to bending of the centre portion provides support against buckling while the spaced configuration reduces the area for friction forces. Furthermore, as the column takes more compression load, the outer boards want to separate away from the centre packs, thereby inducing a tension force perpendicular-to-grain in the packs. Unlike gluing, the prestressing in compression with stress lamination directly acts against this tension force. Built-up columns, either straight or more novel designs like this curved example, are therefore another case where stress lamination is well suited and offers an effective alternative for laminating and connecting elements.

Figure 3.24: Buckling curves and summary of 1:10 and 1:5 model testing.

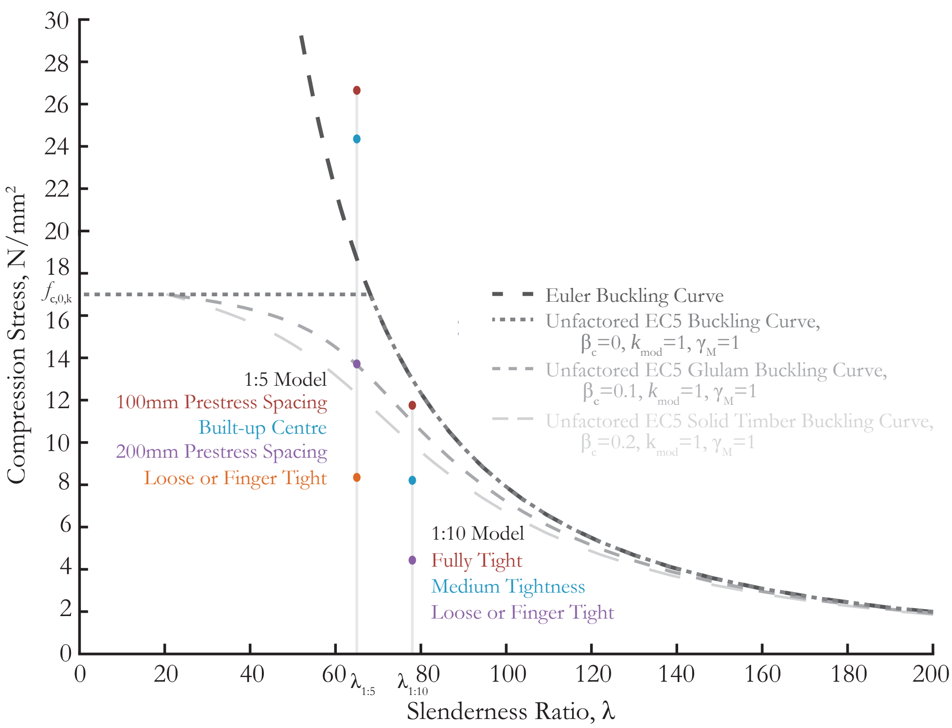


Figure 3.25: Before and after compression testing of a 1:5 model of a split stress-laminated column with a built-up centre.

Summary

Stress lamination has found little application in building design compared to its extensive use for bridge decks. The simplicity of the construction technique allows complex geometries and structural forms to be easily built with basic tools and resources. A model study was done to explore the formal potential of these complex alternatives, showing how curved, fan-shape, twisting, and alternating column and shear walls can be realised. Although these complex structural elements can be easily assembled and were derived with structural principles in mind, they are also expected to cause additional challenges in other areas of construction with finishing materials and fire protection. Preliminary testing of scaled models was therefore focused on more common and relevant straight columns, as even basic types of straight stress-laminated columns have never been proposed or examined. The results of 1:10 and 1:5 model testing highlight the importance of the prestress loss issue, as test trials with intentionally loose prestressed elements performed far below both theoretical Euler and Eurocode buckling estimates. Models that were fully prestressed, however, showed reasonable agreement with unfactored or characteristic Eurocode buckling estimates for glulam. For a larger 1:5 model with a configuration of closely spaced prestressing elements at 100mm on-centre, the buckling performance significantly exceeded the Euler estimate and surpassed the characteristic compression strength parallel-to-grain for low-grade C16 wood. A novel column configuration with solid ends and a curved, built-up centre portion with packs also showed similar performance beyond the Euler buckling estimate, suggesting a potential for stress lamination to be effective in both solid and built-up column design. Full-scale testing is needed now to realise and demonstrate this potential fully.

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Chapter 4 STRESS-LAMINATION DETAILING AND FULL-SCALE TESTING

Full-scale stress-lamination details

Connection detailing

Conventional stress-lamination details are formed of simple components: prestressing bars for applying force and bearing plates for distributing force to avoid local crushing failures. In scale models of stress-laminated elements, small diameter threaded rods and wire are relatively easy to work with and handle. At full-scale, prestressing bars with a diameter of 25mm or more are somewhat cumbersome, not only in the assembly process, but also in prestressing with hydraulic jacks. The ends of large diameter prestressing bars protruding at the edges of a bridge deck provide easy-access for maintenance, but their scale is poorly suited for interior design and detailing in buildings. In addition to detailing issues for finish materials, the ends of bulky prestressing bars are also problematic for fire protection and coverings. An alternative design detail needs to be developed for extending the stress-lamination technique to buildings. The author[1] presented such an alternative design at the World Conference of Timber Engineering in 2014. The following discussion expands on the previous work already presented with more detail on its development and use in full-scale testing.

Figure 4.1 shows a typical example from practice of the steel prestressing bars protruding from the edge of a stress-laminated bridge deck. Along with these steel bars, either steel bearing plates or channels, or steel and hardwood plates together are found on the sides of a stress-laminated bridge deck. A recent master's research project[2] in Sweden has shown that the configuration with steel and hardwood bearing plates can avoid crushing, but the bearing plates can also fail in bending if they are too thin or if the bars are over tightened. Elkhom proposed using self-tapping, axially-loaded screws underneath bearing plates to further reinforce the compression strength of the wood in the perpendicular-to-grain direction. This reinforcement works in exactly the same manner as in platform construction, and serves the same strengthening role.[3] Self-tapping screws help to distribute prestressing loads more effectively into the deck. This addition of self-tapping screws works well for bridge decks, and providing bearing plates are designed properly, the reinforcement from screws can allow prestressing bars to be significantly over tensioned. By further tensioning the bars, friction forces can be increased along with structural performance in bending. This updated detail is useful for stress-laminated elements in building construction, but still incorporates large diameter prestressing bars and bearing plates that require further consideration.

Figure 4.1: A stress-laminated bridge deck showing the bearing plates and ends of steel prestressing bars at the edge of the deck.
(Source: Anton Steurer, *Developments in Timber Engineering - The Swiss Contribution* (Basel: Birkhauser, 2006).)

An alternative prestressing configuration with several smaller diameter threaded rods is proposed in Figure 4.2. The six threaded rods in the detail are specified as M10, having a nominal 10mm diameter and shank area of about 78mm², giving a total area of roughly 468mm² for the group. Compared to this total area, a single prestressing bar specified as M24 with a nominal 24mm diameter has a similar shank area of about 450mm². This new configuration with several smaller diameter threaded rods offers many benefits compared to using a conventional single prestressing bar: one large diameter bar inherently concentrates prestressing forces, while several smaller threaded rods or bolts can achieve more even prestress distribution with thinner bearing plates; the two options are also comparable in cost, and using several prestressing threaded rods in a group also introduces additional safety and redundancy. This detail with multiple smaller prestressing threaded rods, however, requires slightly more labour and skill in construction. One large diameter hole is easier to repeatedly drill in boards, compared to many smaller diameter holes with tighter tolerances on their location relative to one another. Similarly in the assembly and prestressing processes, fewer prestressing elements is advantageous as the successive tensioning of bars affects others that have already been tensioned. Stress-laminated construction guides therefore recommend tensioning all the bars along the edge of the deck three times with a single hydraulic jack.[4] Despite these drawbacks, the decisive advantage of the alternative prestressing detail in Figure 4.2, with several smaller diameter threaded rods compared to one large bar, is that construction, prestressing, and maintenance shifts down in scale from the hydraulic jack to the human hand. M10 threaded rods or bolts can be easily tightened with a torque wrench, and they are much more compatible with different types of interior finishes and fire proofing than large diameter bars.

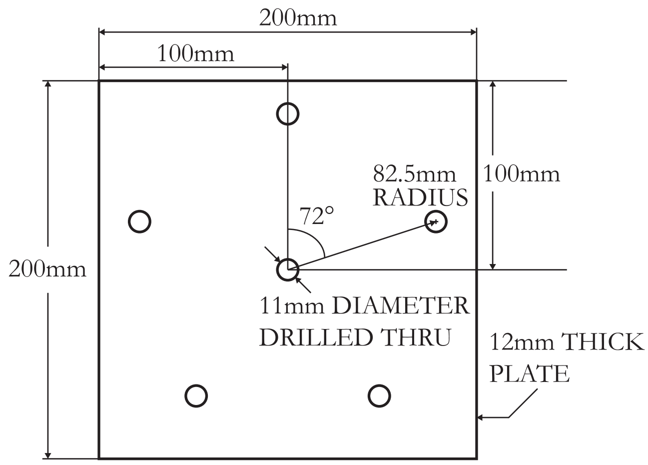


Figure 4.2: Drawing for an alternative prestressing configuration and bearing plate with six threaded rods.

Tension versus torque trials and estimates

Tensioning and prestressing bolted joints with a torque wrench is already done extensively in steel construction. Preloading or prestressing bolted connections is not commonly done in wood construction, however, due to fears about movement with changing moisture content and creep leading to prestress losses. The general correlation between bolt tension with torque is dependent on several factors: hardness and friction properties of the bolt head and contacting joint surfaces, thread conditions and lubrication, and operator accuracy.[5] Although a general correlation between bolt tension and torque can be strongly influenced by these various factors, a reasonable correlation can be established if these conditions are limited and accounted for appropriately. For example, in steel structures and construction, bolt and surface materials are usually limited to a few types for a given project or job site, and torque wrenches are calibrated daily to ensure the correlation between torque and bolt preload is not compromised by an inaccurate tool. Following this example, and to minimise inaccuracies in subsequent work, a calibration was performed of a manual torque wrench that was used later for prestressing threaded rods in stress-laminated timber elements. The wrench's calibration data are summarised in Figure 4.3, where the torque wrench was set to various torque levels in ft·lb, then converted to Nm for consistency with the other metric units used throughout testing. At each setting an actual applied load and torque on the wrench's handle was increased until the wrench disengaged. Figure 4.3 shows the expected linear behaviour of the torque wrench, although the wrench's torque setting is consistently 15% higher than the actual applied torque. A calibration was also performed at the zero torque setting of the wrench to determine the limit of its lower range. As Figure 4.3 shows, the wrench is unreliable at torque levels less than around 10Nm, as the zero setting itself produced an applied torque of approximately 8Nm. The data point corresponding to the 10ft·lb or 13Nm setting, however, performed well and followed the same linear correlation as in higher settings. This 10ft·lb or 13Nm torque setting, resulting in 12Nm of applied torque, therefore corresponds to the lowest allowable torque and prestressing condition in subsequent tests.

Figure 4.3: Calibration data for a manual torque wrench for prestressing threaded rods.

Figure 4.4 shows the results of basic trials to relate the tension in a threaded rod with the applied torque. In this case a pass-through type load cell was used around the rod and in between two pieces of wood and bearing plates to measure the tension in the threaded rod. Two trials are shown in Figure 4.4, where each shows a linear relationship between torque and tension, but at slightly different rates or slopes. Together these results show how the thread and friction effects introduce error and uncertainty, although they also confirm a rough but reasonable correlation between the tension

in the threaded rod and the applied torque in the first instance. Estimates of tension and prestressing forces will therefore be made from the applied torque in subsequent full-scale testing. For more accurate estimates of bolt tension, strain gauges mounted on threaded rods or external collars are needed and will be considered in detail later on.

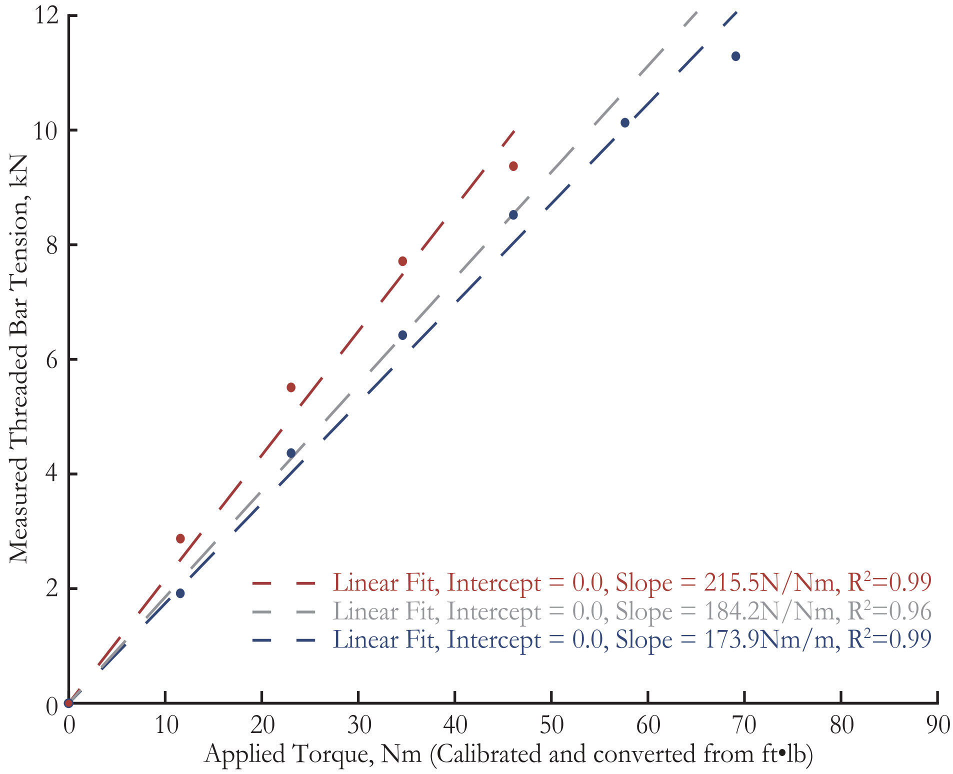
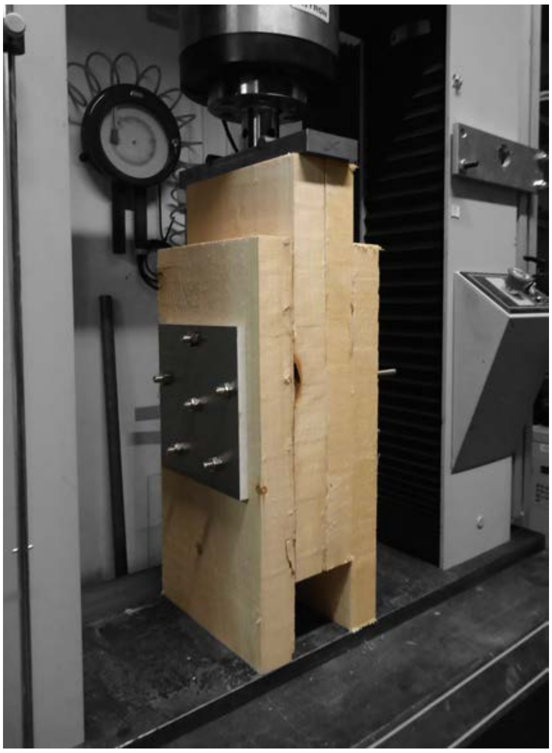


Figure 4.4: Tension versus applied torque for prestressing threaded rods.

Shear tests

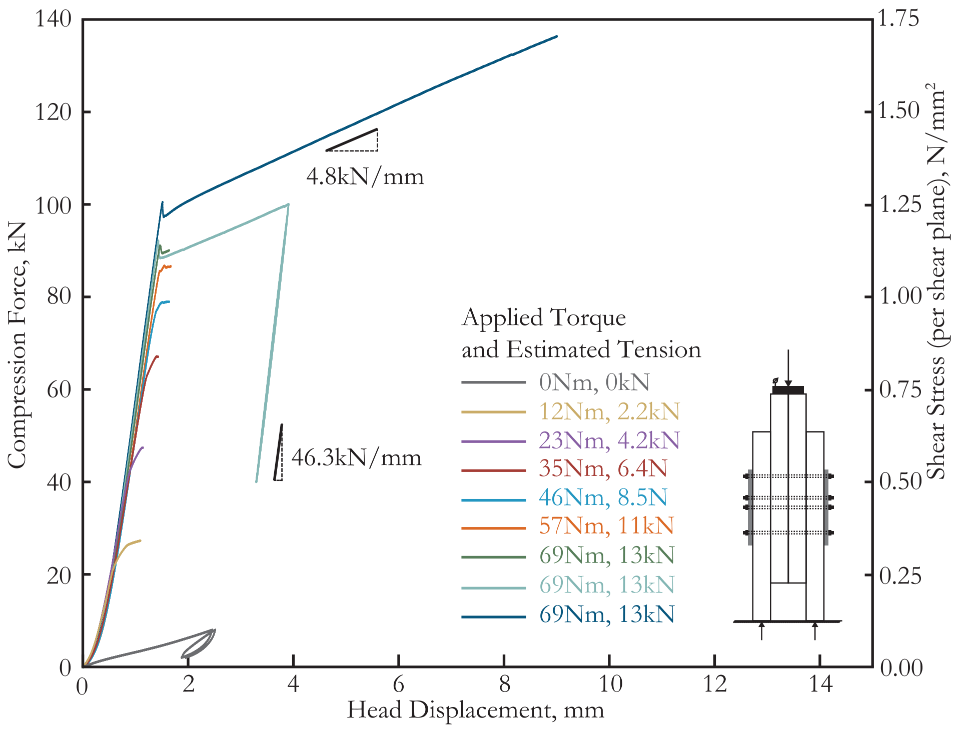
To examine the lamination performance and shear resistance of the proposed stress-laminated detail, two bearing plates were machined following the pattern shown in Figure 4.2. These plates were the same bearing plates used for the previous torque and tension trials. Each plate was machined to the final dimensions of 12x200x200mm in aluminium. The softness of aluminium compared to steel was desirable in case of small deviations in the drilling locations in the wood, as the plates would be less prone to damaging the threads of the steel threaded rods. While conventional stress lamination involves steel bars in plastic sleeves to prevent corrosion issues with the wood, M10 A2 stainless steel threaded rod was specified in the present case, simplifying the detail and avoiding the need for plastic sleeves. Using four rough sawn UK-grown Sitka spruce boards measuring 47x250x500mm, the bearing plates and threaded rod were used to build the full-scale stress-laminated prototype detail shown in Figure 4.5. Note that the holes drilled in each of the boards to allow the threaded rods to pass through were 11mm in diameter, for the nominally 10mm M10 threaded rods. Although stress lamination guides recommend oversizing holes to a greater extent, the 11mm diameter or 1mm of clearance is the upper limit recommended in Eurocode 5 for bolted connections.[6] The oversized diameter of the drilled holes still allows opportunity for static friction forces to engage before embedment occurs. Once static friction forces are overcome by applied actions, the arrangement can still act as a regular dowel connection with some initial slipping, followed by embedment and bearing resistance. To confirm this dual behaviour, the prototype was subsequently tested in shear at different levels of prestress in an Instron 5567 testing machine. The tension in the threaded rods during testing was estimated from the applied torque of the manual calibrated torque wrench.

Figure 4.5: Full-scale prototype of a stress-lamination detail in shear testing.



The results of shear testing the full-scale stress-laminated prototype detail with different levels of prestress are shown in Figure 4.6. With each different prestressing trial, the testing was stopped prematurely just as the static friction forces were overcome and the threaded rods started to bear and embed in the wood. Stopping each trial prematurely was done to minimise any embedding action of the rods, in turn giving the opportunity to test different prestress levels with only one prototype in the first instance. Comparing the different trials, the effect of prestressing on improving the stiffness or initial slip modulus in Figure 4.6 is noticeable. Note that the high stiffness or initial slip modulus seen in the prestressing trial is due to static friction forces, but after the static friction forces are overwhelmed, the prototype behaves as a normal mechanical connection. Although only the head displacement was measured rather than the actual relative displacement of the boards in the prototype, the improvement in stiffness between the first trial with no prestressing and all subsequent trials with prestressing is nearly tenfold; the effect of prestressing increases the stiffness from about 4.8kN/mm to 46.3kN/mm. Improvements in strength can also be seen due to prestressing, with the most optimum prestress level coming from the 35-46Nm of applied torque. At higher prestress levels, gains in strength can be seen but are minimal as the rods and threads are being overstressed. For lamination and especially columns and their stability and buckling behaviour, the gains in stiffness from prestressing are far more relevant than strength issues. In general, glue-based lamination has a far higher stiffness than mechanical-based lamination with nails or screws. In turn, the former can achieve far higher composite action and more efficient shear transfer in laminated elements. These results suggest that stress lamination can also outperform conventional mechanical connections, and may provide comparable performance to gluing.

Figure 4.6: Shear test results for the full-scale prototype at different prestress levels.



In Figure 4.6, the penultimate trial conducted with an estimated 13kN of prestress also illustrates how the stress-laminated detail reacts to cyclical loading. As the loading overcomes the friction forces at roughly 90kN, the rods start to embed and bear in the wood and the stiffness reduces. After reaching 100kN, the loading begins to cycle down to 40kN, and as the load is reduced the static friction forces also immediately re-engage, with the stiffness returning to its initial higher level of about 46kN/mm. Due to the embedding action, the inner surface of the drilled holes has also been shaped to closely match the threads of the rods, resulting in higher stiffness. The re-engagement of the static friction forces, however, means that if a variable short-term action or load was applied and exceeded the static friction forces from prestressing, the effects of prestressing would not be lost once the action was removed or stopped. Note that this somewhat complex structural behavior arises from the simple act of prestressing and tightening a timber connection with threaded rods or bolts. While these results are a promising start for demonstrating the structural performance of stress lamination at full-scale, more than one model or prototype is needed to confirm these trends in further testing. Future trials with an updated test method in line with standards for testing timber connections, such as BS EN 26891:1991,[7] would also be beneficial for more straightforward comparisons with the performance of standard mechanical connections and estimates.

The author received valuable feedback when discussing the results in Figure 4.6 with various colleagues and structural engineers familiar with timber connections and design. In particular, Mr. Simon Smith and Mr. Tristan Wallwork offered some constructive criticism and feedback that is worth discussing here, as two specific issues arose that required more detailed clarification. The first issue regarded the use of one prototype with multiple trials at different prestress levels. Even though each trial was stopped before significant bearing or embedding of the threaded rods could occur, the first trial was performed without any prestress. Could the stiffness in subsequent trials with prestressing be influenced by the preceding test or tests? This is a relevant concern, but one that can be addressed by considering how the static friction forces can first act independently of bearing resistance and the potential embedding and bending action of the threaded rods. As the shear resistance associated with static friction exhibits a higher stiffness than the resistance associated with bearing or embedding, the forces are attracted to and transferred between the boards through the path with higher stiffness, that is, through friction. The second question regarded the different prestress levels yielding the same stiffness value or slip modulus. Surely the slip modulus should increase with increasing prestress? The behaviour implied in this question seems intuitive and logical, but the question forgets that the friction forces in question are static friction forces rather than kinetic. The distinction between these two is subtle but significant, as static friction forces and static coefficients of friction are generally greater in magnitude than their kinetic counterparts. Static friction occurs before any slipping initiates, and once overcome, gives way to kinetic friction that is lower in magnitude compared to the preceding static friction. The static friction force, however, can also vary in magnitude, and within its capacity, will resist and match applied loads to maintain equilibrium. The similar stiffness values or slip moduli associated with different levels of prestress in the various trials therefore corresponds to the time before slipping, and before kinetic friction has been initiated.

Figure 4.7 displays the results of simplified testing with two smaller models having only one M10 threaded rod each. The two models are nearly identical except for one model is prestressed, while the other is only finger-tight. Here the influence of prestressing becomes much more explicit. The higher stiffness and initial slip modulus associated with prestressing and static friction forces can be clearly seen. The prestressing shifts the typical embedment curve upwards. As most timber connections and structures are heavily factored, with factors applied to both material properties and loads or actions, this upward shift and gain in strength is less significant compared to the increased initial slip modulus and stiffness. In practice, timber connections will function in this initial region, instead of being pushed toward the ULS and approaching failure. Awaludin *et al.*[8] made a similar argument when they examined the benefits of prestressing timber connections for resisting rotations and moments. They also concluded that prestressing is beneficial due to the addition of static friction forces for increasing stiffness. Prestressing may be advantageous for other types of timber connections, but the present work will remain focused on its use as a simple but effective alternative to gluing and other types of mechanical-based lamination techniques.

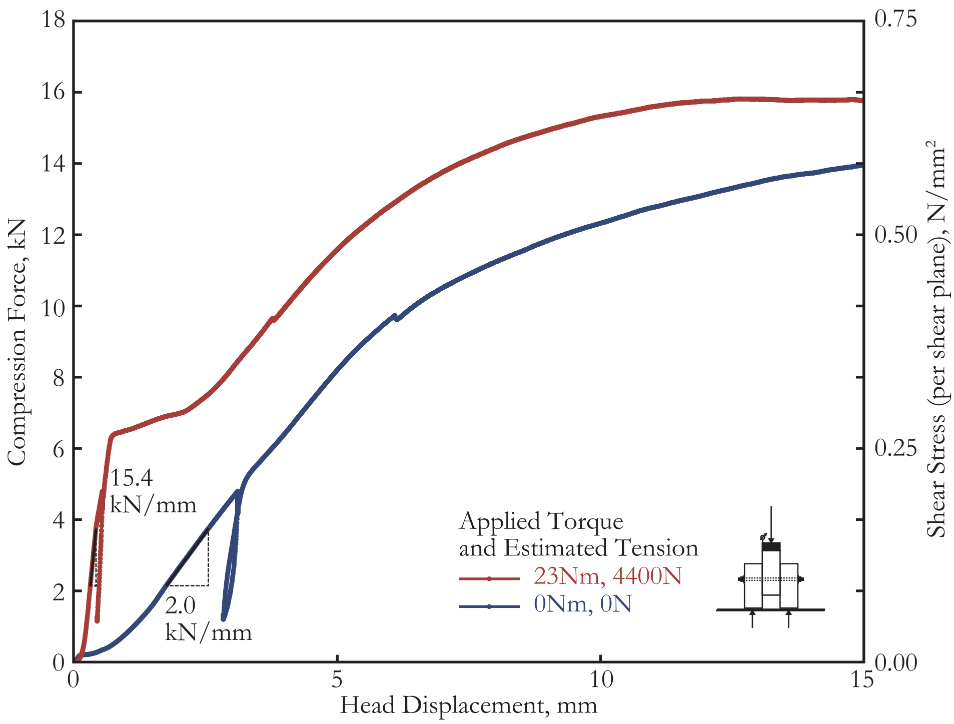


Figure 4.7: Shear test results for models with a single threaded rod, with and without prestressing.

Comparison to dowels, nails, screws, and tooth-plate connectors

The test results presented in Figures 4.6 and 4.7 show how prestressing can improve the structural performance of a mechanical connection. Comparison is also needed with other types of mechanical fasteners in timber, including those with dowels, nails, screws, and special toothed-plate connectors. Following EC5 estimates, the slip modulus of connections with regular screws and nails is similar to that of dowels, and they are all calculated with the equations provided in Table 7.1 of BS EN 1995-1-1:2014. The stiffness or slip modulus per shear plane per fastener for a connection with dowels, screws, and nails with predrilling, K_{ser}, is defined as follows:

$$K_{ser} = \frac{\rho_m^{1.5} d}{23}; \tag{4.1}$$

and for nails without predrilling,

$$K_{ser} = \frac{\rho_m^{1.5} d^{0.8}}{30}; \tag{4.2}$$

and for a basic Type C1 tooth-plate connector,

$$K_{ser} = \frac{1.5 \rho_m d_c}{4}, \tag{4.3}$$

where ρ_m is the mean density of the wood in kg/m^3, d is the diameter of the dowel, regular screw, or nail in mm, and d_c is the connector diameter in mm. K_{ser} has units of N/mm. Taking the mean density of UK-grown Sitka spruce as 390kg/m^3,[9] with a smooth 10mm dowel, the slip modulus is calculated at 3350N/mm per fastener per shear plane. Based on slip moduli estimates, Table 4.1 summarises the total stiffness in kN/mm if the models in Figure 4.7 were made with a smooth dowel, nails, screws, or connectors. The experimental stiffness results from Figure 4.7 are also included for reference. Note that the model tested from Figure 4.7 featured two shear planes, so the estimated slip moduli of 3350N/mm for a dowel is doubled accordingly to 6.7kN/mm.

Table 4.1: Stiffness comparisons between prestressing and estimates for other types of mechanical fasteners for timber.

Type	Number of Fasteners	Total Stiffness, kN/mm
10mm Threaded Rod	1	2.0
10mm Prestressed Threaded Rod	1	15.4
10mm Dowel	1	6.7
4mm Regular Timber Screws	8, (4 fasteners per shear plane)	5.4
4mm Nails (with predrilling)	8, (4 fasteners per shear plane)	5.4
4mm Nails (without predrilling)	8, (4 fasteners per shear plane)	3.1
Toothed-plate Connector (C1 type, 50mm diameter)	2, (1 connector per shear plane)	14.6

The experimental results and measured stiffness for the regular and prestressed 10mm threaded rod in Table 4.1 have the lowest and highest values, respectively. The connection stiffness of a 10mm dowel is just over three times that of the threaded rod, highlighting the strong influence of the smooth and rough surface conditions the two different fastener types. While the inner wood surface of the drilled hole can also strongly influence stiffness and load results, up to a factor of three times,[10] high-quality Famag lip and spur wood drill bits were used throughout to minimise this effect. Moreover, the inherent compliance and stiffness of the Instron testing machine that applied the load in the experiments also may be influencing the test results. Attaching displacement transducers directly on the models is needed to eliminate this possible influence. Of all of the mechanical fasteners in Table 4.1, only the estimated stiffness of the toothed-plate connectors can come close to matching the prestressed threaded rod test results. This comparison will be examined further, along with other more advanced types of fasteners, in future shear testing conducted after full-scale construction and buckling experiments.

Summary

Conventional stress lamination detailing with heavy bars on the order of 25mm or more is suitable for bridge decks but not for buildings. An alternative proposal with thinner bearing plates and multiple smaller diameter threaded rods for prestressing was proposed and tested at full-scale. Using a calibrated torque wrench to prestress the threaded rods, a full-scale prototype connection made with UK-grown Sitka spruce was constructed and tested in shear at various prestress levels. The tension of each threaded rod was roughly estimated from the torque wrench setting. Experimental results show that prestressing increases the strength capacity and especially the stiffness of the connection or laminated detail. The structural performance and stiffness due to prestressing is only slightly above that of estimates for tooth-plate connectors. Theoretical estimates for other types of mechanical fasteners offer noticeably lower stiffness values and lamination potential than prestressing. Further testing is needed with displacement transducers to better quantify how prestressing enhances stiffness. These tests can be conducted either in parallel or following full-scale construction efforts and structural testing of stress-laminated columns.

Full-scale construction and testing

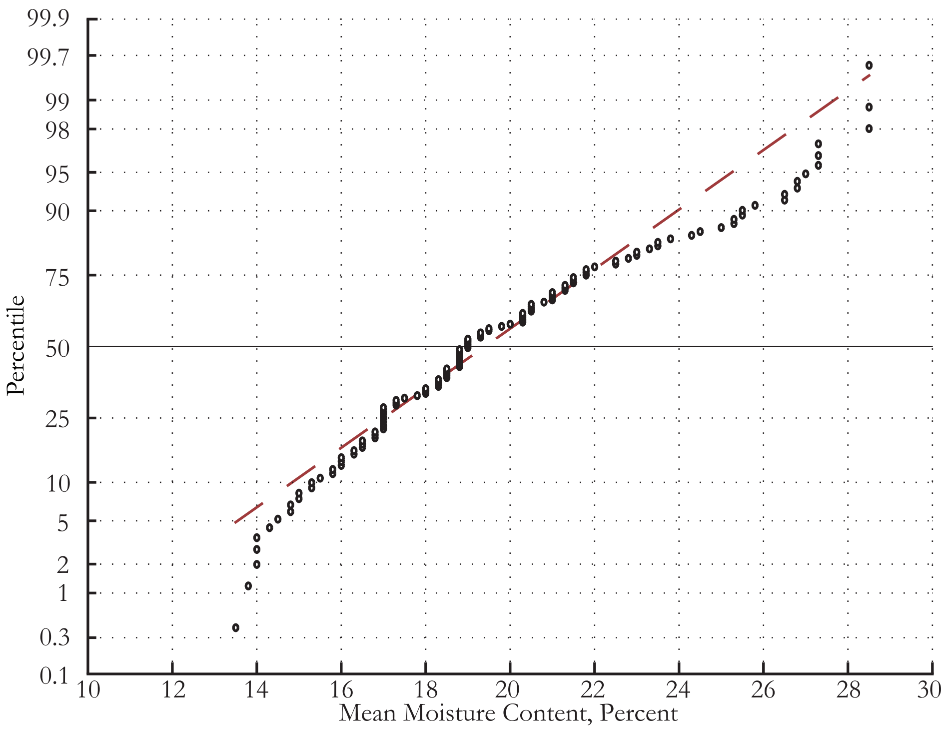
To verify the results from scale model studies and the previous detailing at full-scale, a large amount of UK-grown Sitka spruce was arranged and generously donated by BSW Timber Limited. With the support of Mr. Alex Brownlie and Mr. John Smillie, the author collected and transported 128 boards of UK-grown Sitka spruce, each measuring 47x250x3600mm, from the BSW sawmill at Newbridge on Wye in Wales. The timber was specified and provided as rough sawn, without being planed or graded, and was kiln dried to 20% moisture content following the sawmill's standard practice.[11] Note that the timber was intentionally specified as rough sawn, as the rougher surfaces were expected to increase friction forces with the stress lamination process while adding additional value by saving the cost of planing. The timber was collected over the course of two days, with two return trips made between Cambridge and Wales with the timber in a large enclosed van, keeping the material dry throughout. While storage and workshop facilities are limited at the Cambridge University Architecture Department, a well-sheltered place was arranged outside of the Department's main wood workshop, shown in Figure 4.8. During subsequent construction and testing, the timber was therefore exposed to outdoor conditions but kept safe from any rain.

Figure 4.8: Rough sawn UK-grown Sitka spruce supplied by the BSW Timber Limited sawmill at Newbridge on Wye in Wales.

UK Sitka spruce moisture content and MoE characterisation

Before embarking on a program of full-scale construction and structural testing, each board in Figure 4.8 was characterised in terms of its moisture content and estimated MoE in bending. Four moisture content measurements were taken along the length of each board with an Extech Model MO220 capacitance-based moisture meter. The results of these moisture content measurements are shown in Figure 4.9, where the mean of the four measurements for each board is shown in a normal probability plot. The overall mean moisture content of all of the timber is 19.7%, closely matching the Newbridge sawmill's kiln drying target of 20% moisture content. The lowest moisture content is around 13.5%, while the highest is around 28.5%, with the latter allowing fungal growth if sustained over time. Note that about 12 boards or 10% of the total already meet the requirements for glulam, with moisture content levels between 8 and 15%. Around 20 boards also meet the recommended moisture content of 16% or less for stress lamination and prestressing, although no strict requirements are specified in BS EN 1995:2:2004 for bridges.[12] The boards' mean moisture content distribution also roughly follows a linear behaviour on the normal probability plot, meaning that the measured moisture content data also approximately follows a normal distribution.

Figure 4.9: Normal probability plot of mean moisture content of UK-grown Sitka spruce boards.



An approximately normal distribution can also be seen in Figure 4.10 for the MoE estimates of the boards. The mean MoE of 7.12kN/mm^2 is somewhat lower than expected, with the maximum and minimum values of the MoE data around 4.9 and 10.3kN/mm^2, respectively. All of these MoE data are derived from non-destructive dynamic measurements with a handheld Brookhuis Timber Grader MTG device, kindly provided by Mr. Paul Charnaud of Scott+Sargent Woodworking Machinery Limited. The device has already been benchmarked against MoE data from static bending tests, and is also approved for strength grading purposes in EN 14081-4:2009.[13,14] To estimate MoE, the device requires either the mass of the board, or input parameters including the dimensions of the board, a reference density at 12% moisture content, and the board's actual moisture content. Rather than weighing each individual board, the reference density was set to 390kg/m^3 for UK-grown Sitka spruce following published data from the UK Forestry Commission.[15] These MoE measurements were conducted in parallel with the previous moisture content data, so based on the measured moisture content of the first few boards, an assumed 18% moisture content was programmed into the MTG device for all measurements. Instead of stopping and reprogramming the device for each board, the error introduced from the difference in the actual moisture content of each board and the assumed 18% value was eliminated in post-processing. After completing measurements, the MTG device's performance with respect to different moisture content settings was characterised with two reference boards. The device's behaviour was linear, as expected, with the MoE estimate increasing by 0.65% for every 1% difference in the actual and assumed or programmed moisture content. Following this correlation, Figure 4.10 shows the MoE estimates corrected for moisture content.

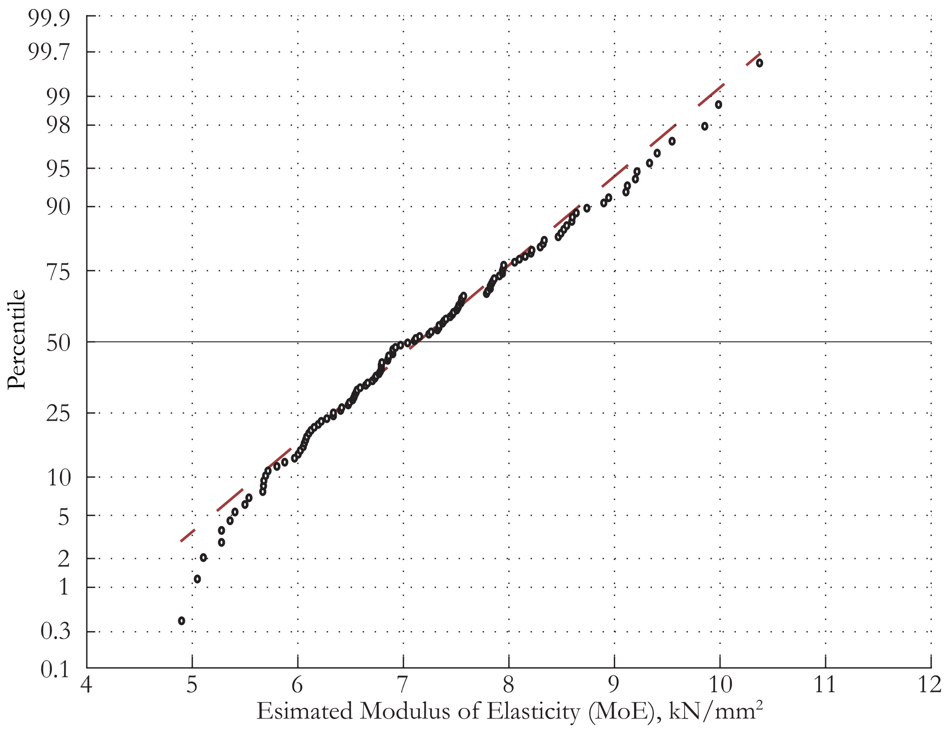


Figure 4.10: Normal probability plot of MoE estimates from dynamic measurements of UK-grown Sitka spruce boards.

Construction and structural testing methods

Having successfully characterised enough low-grade timber for a significant series of full-scale structural tests, construction began in earnest. The author machined an additional 40 aluminum bearing plates following the dimensions in Figure 4.2. An additional 120 M10 A2 stainless steel threaded rods were also ordered at a length of 300mm. Altogether this hardware could allow a maximum set of four stress-lamianted columns to be constructed and tested at a time; care was needed to avoid damaging the plates and especially the threaded rods so they could be reused in additional column sets. With the threaded rods loaded elastically, the only potential problem that could arise preventing their reuse would be physical damage to their threads. The author began by constructing four stress-laminated columns, each with five layers or lamellas of 47mm thick Sitka spruce boards. The full 3600mm board length was used in the column, thereby giving a height of 3600mm, while the columns' cross-sectional area was 235x250mm^2. Assuming an ideal lamination and connection efficiency of 100%, as is commonly associated with gluing, these dimensions yield a column slenderness ratio, λ, of 53. This slenderness ratio falls in the intermediate range of the EC5 buckling curve, and was chosen intentionally as the slope of the buckling curve is steepest in this range. The buckling curve becomes relatively flat at both low and high slenderness ratios, meaning that stocky and long slender columns are less sensitive to changes in structural efficiency and performance. With stocky columns having low slenderness ratios, the compression strength of the material dominates. At high slenderness ratios, the geometry and bending stiffness of the column governs with little influence from compression strength. By constructing columns with intermediate slenderness, both factors are influential, so any performance inefficiency with stress lamination would therefore be easiest to detect and measure.

A series of photographs taken during the construction, testing, and disassembly processes is presented in Figure 4.11. For construction and assembly, five boards making up a column were first clamped together. A Famag drill guide and an 11mm diameter auger drill bit was then used to accurately drill the holes for the threaded rods for prestressing. Even with the drill guide, however, each prestressing detail or group of six holes would require one or two of the holes to be slightly enlarged with additional drilling to properly fit the pattern of the bearing plates. The top of the holes where the drilling process began would always match the bearing plate closely, but at the bottom surface where drilling finished, one or two of the holes would be misaligned with the bearing plate pattern and would require redrilling. Note that some boards appeared to have a slight twisting or cupping, and although they were within the limits of standards for sawn timber,[16] even a slight twisting or cupping could cause accuracy problems with drilling. Clamping the boards helped to minimise these errors, but even minute inaccuracies or misalignments when starting the drilling process are exaggerated when drilling down with an auger bit 320mm in length. Drilling therefore was by far the most time consuming stage of construction, and could most likely be optimised in the future.

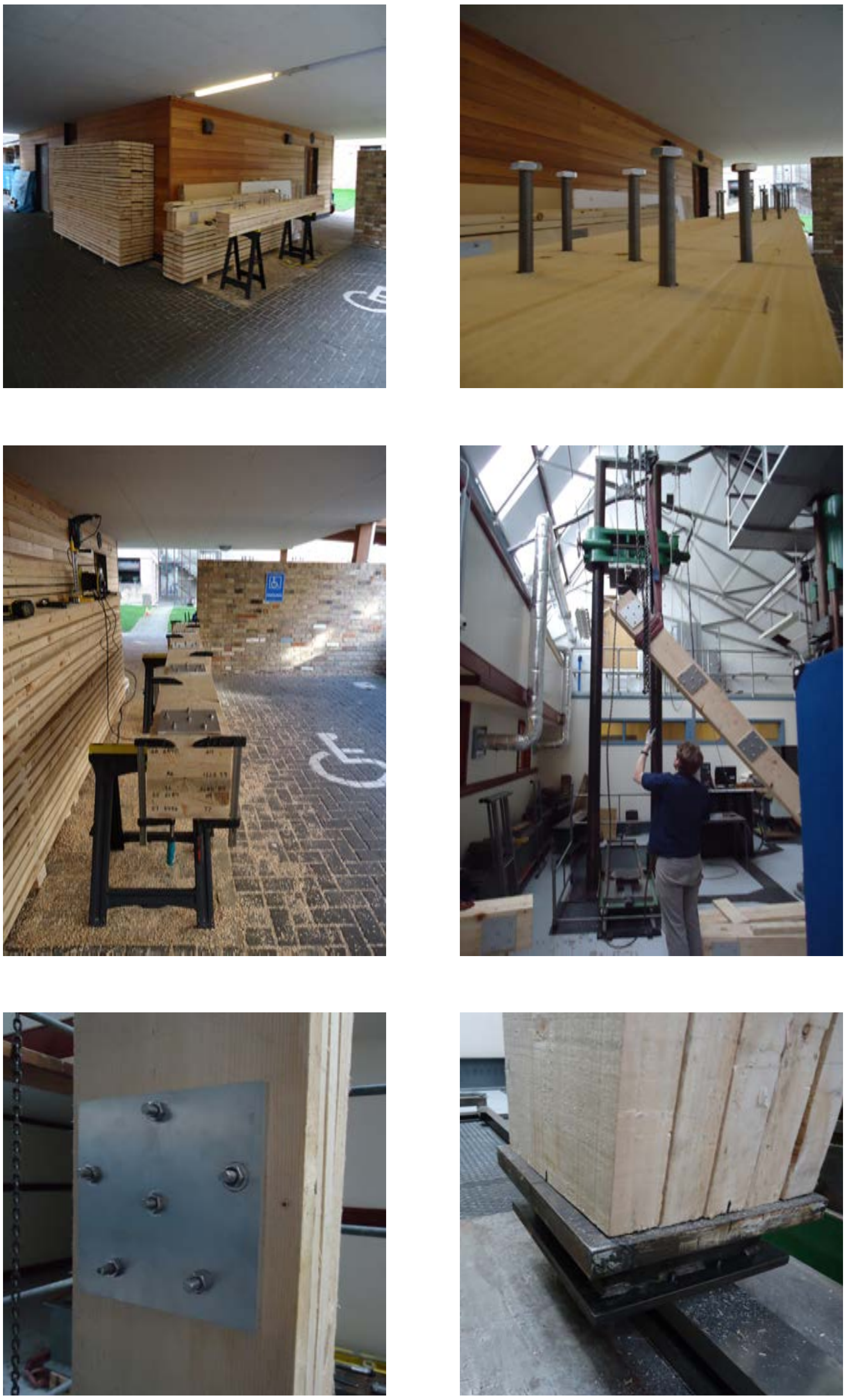


Figure 4.11 (right and opposite): **(a)** Drilling and assembly of stress-laminated columns and **(b)** the threaded rod arrangement before installing the bearing plates.

(c) Clamps are used to hold the individual column layers together while drilling and installing the bearing plates. **(d)** After prestressing and recording moisture content values, a column is hoisted into the Amsler press.

(e) Typical bearing plate and threaded rod detail, and **(f)** an initial alignment of a column in the Amsler press.


(g) The base and **(h)** top of a column are adjusted and recheked for alignment and level prior to testing.


(i) A stress-laminated column before and **(j)** after testing.


(k) Shear deformation of a column after buckling, where **(l)** deformation is still held by friction forces prior to disassembly.


(m) Deformation is significantly relieved after loosening the threaded rods during disassembly. With the disassembly process completed **(n)** and the threaded rods removed, the individual layers of a column remain suitable for reuse.

The leftside of Figure 4.12 shows a scaled elevation drawing of the first set of four columns constructed, with a spacing of 700mm on-centre for the prestressing bearing plates. Note that the distance between the bearing plate edges is 500mm. Upon completing the first column of the second set, or the fifth column in total, the author noticed that the column ends were not pressed together as tightly as expected. Shear stresses and deformation due to bending are also expected to be at their maximum at the ends, as shown in Figure 4.11(k), where friction forces need to work the hardest. All subsequent columns were constructed with an adjusted spacing of 800mm on-centre for the bearing plates, or a distance of 600mm between adjacent plate edges. This second configuration, shown in Figure 4.12 on the right, appeared to press and clamp the ends of the columns together much more effectively than with the initial configuration. For both spacing configurations, after drilling and assembling the boards with bearing plates, threaded rods, and nuts, a column was lightly tightened for subsequent transportation to the nearby Cambridge University Structures Laboratory for testing. The estimated mass of a column was about 80kg, so a single person working alone could lift one end at a time to pivot and move a column onto a small cart with relative ease. Following one to two days in the laboratory, the moisture content of each column was measured and recorded again, before prestressing the column with a calibrated manual torque wrench. An applied torque of 35Nm was used, achieving an estimated tension of 6.4kN in each threaded rod. Following prestressing, each column was then tested in compression until buckling. Figure 4.13 illustrates the ends and annual ring orientations of the boards in all the tested columns. Summaries of the mean moisture content values of each board measured at the time of testing are included, with updated moisture-content corrected MoE values based on the previous dynamic measurements. Photographs from each test case along with individual test data are included and summarised in the Appendix for completeness. Note that the MoE values are also adjusted according to the timber testing standard BS EN 384:2010.[17] No specific moisture content corrections are available in the Eurocode 5 design standard, apart from the more general k_{mod} factor for different service classes and duration of loads.

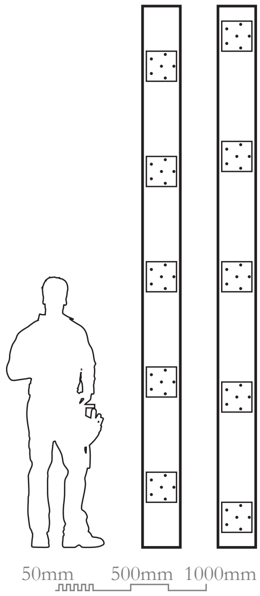


Figure 4.12: Different bearing plate spacing configurations for the full-scale stress-laminated columns.

A 5000kN Amsler compression press was used to test each column at full-scale until buckling occured. Although the press is powered by a hydraulic system, it can be operated in a manual control mode to effectively offer displacement-controlled testing. Figure 4.14 illustrates the Amsler press arranged with simple pinned-pinned end conditions. These end supports were made up of existing parts in the Cambridge University Structures Laboratory. Previous timber column studies by Steiger[18,19] have used far more sophisticated testing apparatus and end supports, with large dowels set in each end of a solid wood column with smooth bearings and hydraulic clamps. The sophisticated end supports by Steiger are advantageous for more advanced studies with combined compression and bending, and they do not extend the buckling length outside of the column. However, the large dowel and clamps of this arrangement reinforces a column in shear. Buchanan's doctoral research on combined buckling and bending interaction of timber columns also used a similar type of clamped end support.[20] Furthermore, the same artificial shear reinforcement is present in a recent timber buckling study for an undergraduate thesis project in the Cambridge University Structures Laboratory, where steel end plates were attached directly to CLT elements with self-tapping screws.[21] While shear effects are not influential for buckling studies with glulam or solid timber,[22] they are potentially critical for stress-laminated elements. If shear forces, associated with bending due to compressive loads and eccentricity, are able to overcome a column's prestress and static friction forces, shear effects become significant. Conventional simply-supported end conditions outside of the column are therefore a more conservative approach for the present testing program.
To avoid exaggerating test results and buckling performance, however, the buckling length in testing is taken as the shorter length of the timber column, rather than the longer distance between the two end pins or rollers.

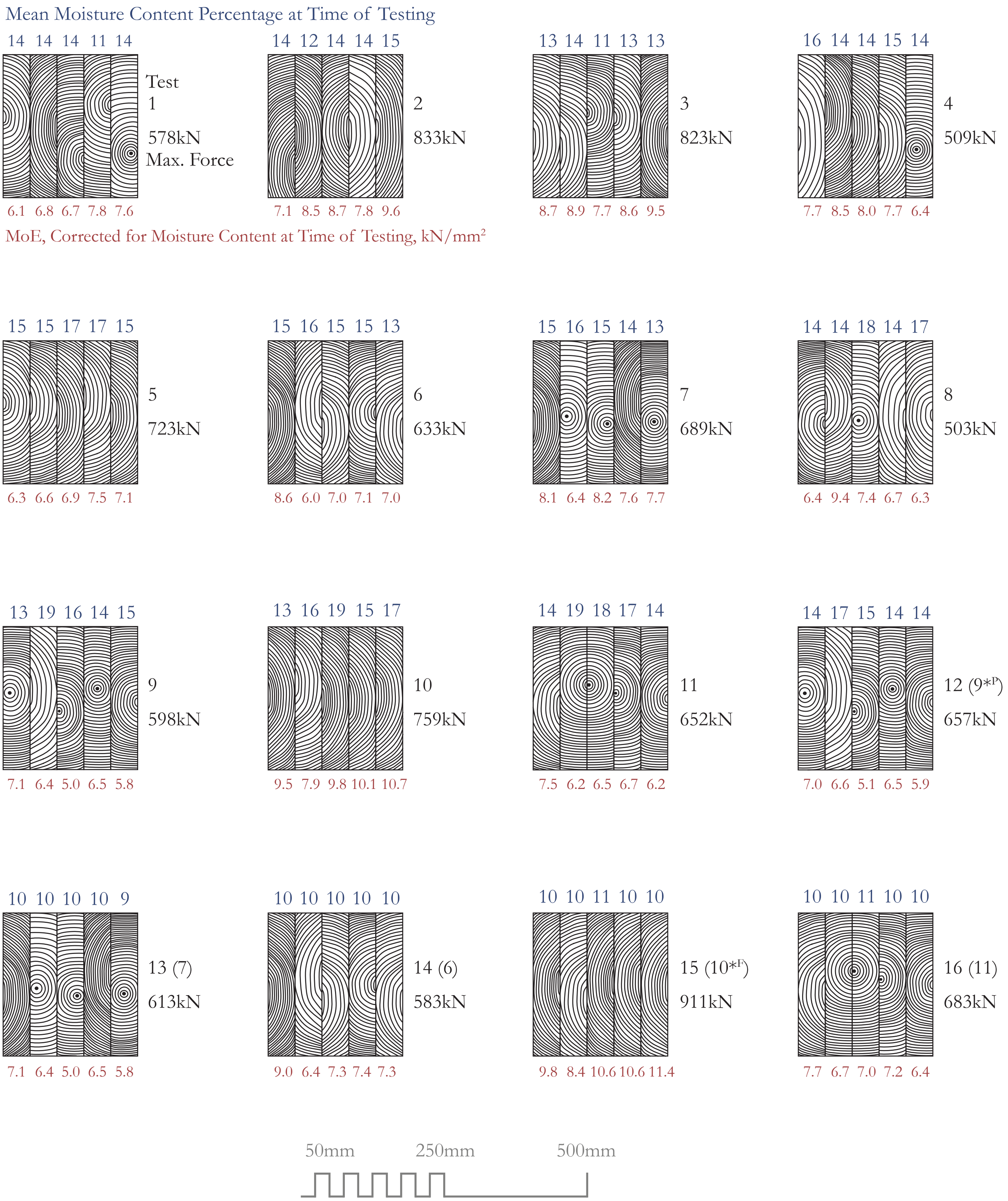

Mean Moisture Content Percentage at Time of Testing
14 14 14 11 14
Test
1
578kN
Max. Force
6.1 6.8 6.7 7.8 7.6
14 12 14 14 15
2
833kN
7.1 8.5 8.7 7.8 9.6
13 14 11 13 13
3
823kN
8.7 8.9 7.7 8.6 9.5
16 14 14 15 14
4
509kN
7.7 8.5 8.0 7.7 6.4
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²
15 15 17 17 15
5
723kN
6.3 6.6 6.9 7.5 7.1
15 16 15 15 13
6
633kN
8.6 6.0 7.0 7.1 7.0
15 16 15 14 13
7
689kN
8.1 6.4 8.2 7.6 7.7
14 14 18 14 17
8
503kN
6.4 9.4 7.4 6.7 6.3
13 19 16 14 15
9
598kN
7.1 6.4 5.0 6.5 5.8
13 16 19 15 17
10
759kN
9.5 7.9 9.8 10.1 10.7
14 19 18 17 14
11
652kN
7.5 6.2 6.5 6.7 6.2
14 17 15 14 14
12 (9*P)
657kN
7.0 6.6 5.1 6.5 5.9
10 10 10 10 9
13 (7)
613kN
7.1 6.4 5.0 6.5 5.8
10 10 10 10 10
14 (6)
583kN
9.0 6.4 7.3 7.4 7.3
10 10 11 10 10
15 (10*F)
911kN
9.8 8.4 10.6 10.6 11.4
10 10 11 10 10
16 (11)
683kN
7.7 6.7 7.0 7.2 6.4
50mm
250mm
500mm

Figure 4.13 (opposite): End grain illustrations, mean moisture content, and estimated MoE of the individual layers in the full-scale stress-laminated columns in structural testing.

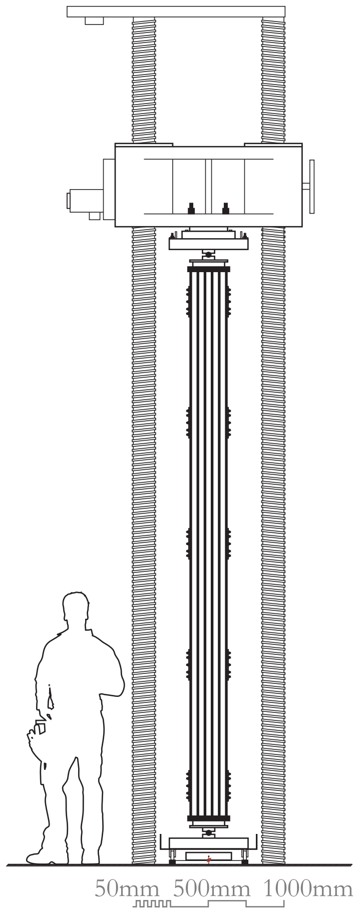


Figure 4.14: Drawing of the 5000kN Amsler press used for full-scale structural testing.

During the general testing process, some minor twisting of the columns was observed while hoisting and aligning them into the Amsler press. While aligning a column, any eccentricities or errors in alignment due to twisting were balanced evenly between each side of the column, and also between the top and bottom plates. Note that standards for structural timber graded above C18 have a limitation for twisting of 1mm per 25mm of width over a length of 2m.[23] This limit corresponds to a maximum allowable 18mm of twist over the 250mm width and 3600mm height of the columns. The maximum twisting limit for C18 and lower graded material is also double that of the higher grades. The twisting in the columns while aligning them in the Amsler press was approximately ±5mm, well below the standard limits for both C16 and higher grades. Despite a slight twist in some columns, each was also aligned in the Amsler press with its centroid located as close as possible to the centre marks on the end support plates, as shown earlier in Figure 4.11. Any initial misalignment or eccentricity in turn lowers a column's buckling resistance, as the column experiences a greater degree of bending due to eccentric loading. In practice, however, timber columns with perfect alignment and straightness cannot be achieved. EC5 suggests that solid timber columns should have a maximum initial eccentricity of 1/300 of their effective length or height.[24] For a 3600mm column, this EC5 estimate gives an initial eccentricity of 12mm. Compared to in-situ measurements of columns by Elhbeck and Blass,[25] this EC5 design guidance is somewhat overcautious. Nonetheless, to minimise errors in test results, attempts were made during column construction and prestressing to also minimise any initial eccentricities, deviations in straightness, and column curvature. All columns were checked for straightness by simple visual inspection, and the worst four of the sixteen stress-laminated columns constructed for the study were also carefully measured before and after testing. The columns had a measured initial eccentricity of 4mm or less over their length of 3600mm. These measurements indicate that stress-laminated column construction can achieve initial eccentricities well within the 12mm or 1/300 limit suggested in EC5 for solid timber.

Structural test results and discussion

The force versus head displacement curves from testing full-scale, stress-laminated columns are shown in Figure 4.15. In all cases the Amsler press was operated in a manual control mode, with the head displacement increasing at a constant rate of 2.4mm/min. Five of the sixteen tests in Figure 4.15 were performed as repeated test cases. Two of five repeated tests were also made with special conditions that are described in detail later on. In these five repeated cases, however, columns that were previously tested and disassembled were then reassembled and prestressed. They were tested again to confirm reusability of the wood and repeatability of the initial results. Note that the original and repeated columns are also numbered in Figure 4.13, with the original column number in parentheses. The maximum force measured from each test is further summarised in Figure 4.16 on a normal probability plot, with solid data points representing the repeated test cases. Figure 4.17 also shows the linear elastic slope plotted against the maximum compression force from each test, illustrating a general correlation between the two parameters and the expected influence of stiffness on buckling performance. This relationship and more general stiffness effects will be studied and discussed in more detail in subsequent testing.

Figure 4.15: Force versus head displacement curves from full-scale testing with stress-laminated columns.

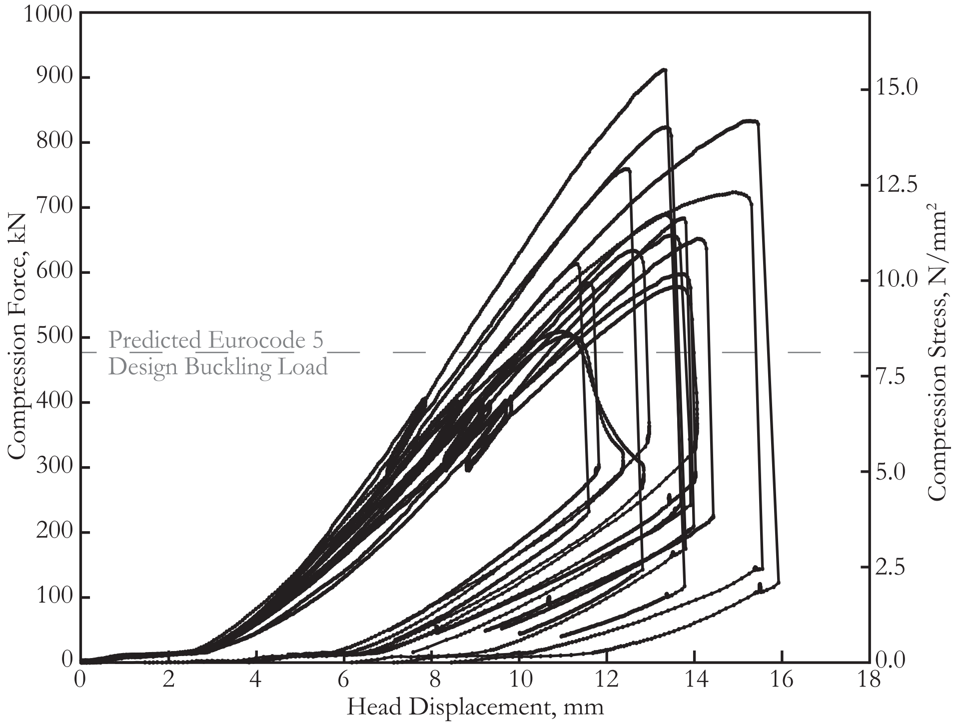


Figure 4.16: Normal probability plot of the maximum force data from full-scale testing.

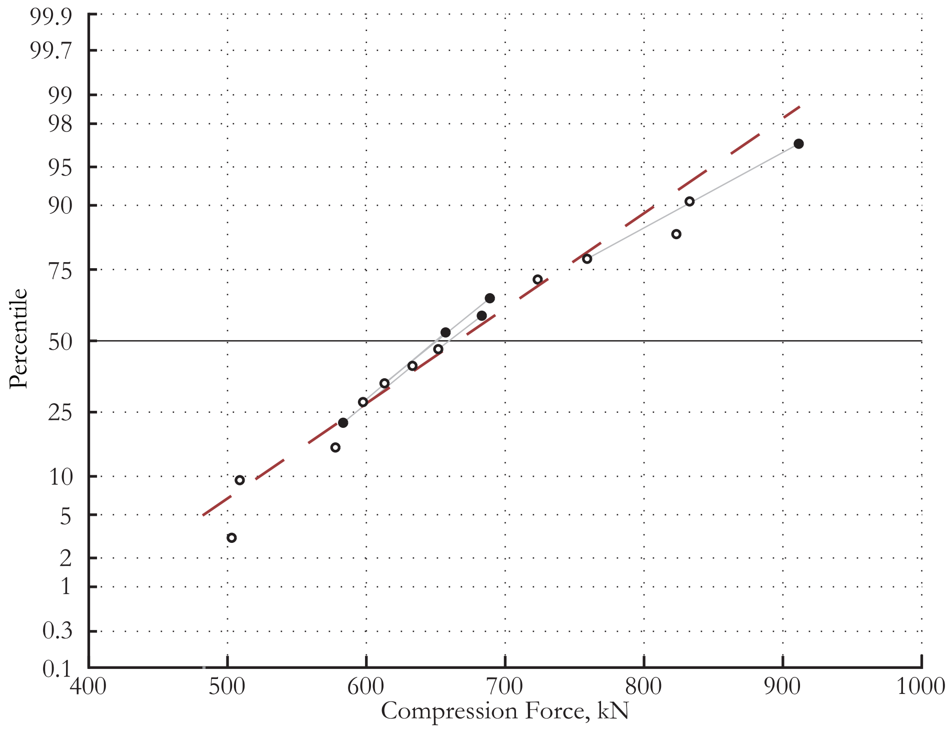
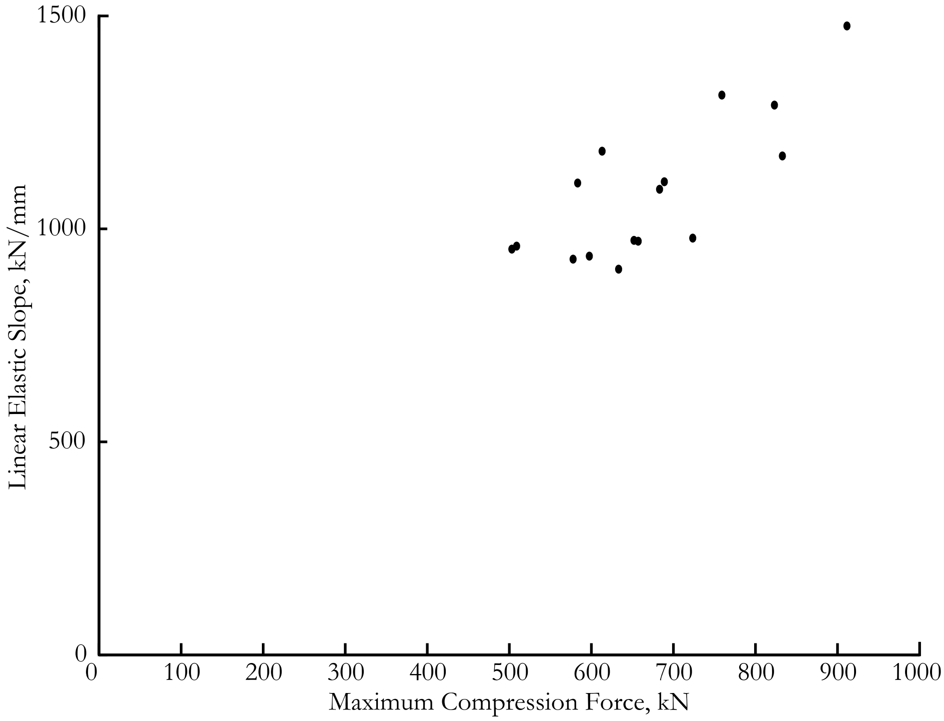


Figure 4.17: Linear elastic slope plotted against maximum compression force for each full-scale column test.



Examining Figure 4.15, in all cases with either initial or repeated testing, the maximum force data are above the predicted EC5 design estimate of 477kN for C16 solid timber columns measuring 235x250x3600mm. This estimate was made following EC5 recommendations for solid timber, with a straightness factor, β_c, set to 0.2, a partial safety material factor, γ_M, of 1.3, and a short-term duration of load factor, k_{mod}, equal to 0.9. While most studies performed in controlled laboratory conditions compare experimental results to estimates using a k_{mod} value set to 1, the use of the short-term value of 0.9 is justified here on multiple counts. Firstly, the k_{mod} factor is also used in EC5 to account for moisture content effects in addition to duration of load effects. Although the construction process was done in sheltered conditions, the columns were still constructed outdoors and then tested with varying moisture content levels above 12%. In addition, the main focus of the Cambridge University Structures Laboratory is steel and concrete structures, and hence the laboratory lacks temperature and humidity control, further justifying the use of the k_{mod} value of 0.9. Finally, in the UK National Annex (NA) to Eurocode standards, short-term loads are defined as those lasting less than 1 week, such as with a maintenance person on a roof, but longer than instantaneous loads such as explosions or impacts.[26] Each structural test conducted here lasted between five to ten minutes. A short-term duration of load factor is therefore the most suitable for comparing the structural performance of stress-laminated columns to design code estimates.

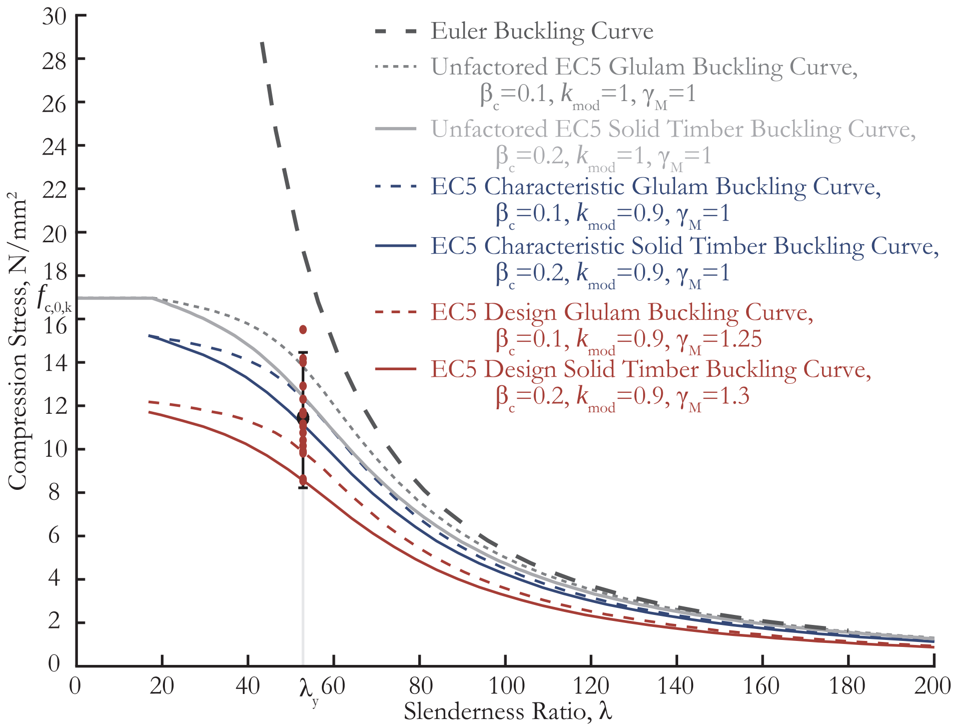


Figure 4.18: Summary of experimental test results with EC5 buckling curves.

The experimental data in Figure 4.16 approximately follow a straight line on the normal probability plot, meaning they are also roughly distributed normally. This result is important given the relatively limited number of tests and data points, especially for determining the characteristic or 5th-percentile buckling load value of the tested stress-laminated columns. From the linear interpolation of the data, the mean or 50th-percentile corresponds to a compression force of 665kN, but the line on the normal probability is also extrapolated down to the 5th-percentile level, corresponding to a buckling load of 483kN. This value falls below the EC5 solid timber estimate of 654kN for the characteristic buckling load. This EC5 estimate is again calculated with the straightness factor, β_c, set to 0.2, a short-term duration of load factor, k_{mod}, equal to 0.9, but with a partial safety material factor, γ_M, of just 1. Note that the general difference between design and characteristic values in the Eurocodes simply arises from the k_{mod} and γ_M factors. This comparison and shortfall confirms that with stress-lamination there is a performance penalty or loss of approximately 26% due to finite connection stiffness and inefficiency. The 5th-percentile value from experiments, however, agrees relatively well with the estimated EC5 design value of 503kN, with the former being only 4% lower than the latter.

Compared to EC5 glulam estimates, the 5th-percentile buckling load of 483kN from testing stress-laminated columns is lower. For a hypothetical C16 glulam column with the same dimensions as those tested, the EC5 design and characteristic estimates are 654 and 725kN, respectively. The characteristic value estimated from the experimental data is approximately 17 and 33% lower than these equivalent glulam estimates. All of these comparisons are illustrated for clarity in Figure 4.18, where error bars are used to mark the estimated 5^{th}- and 95^{th}-percentile levels from testing. While the stress-lamination technique produces results that closely match the EC5 design estimates for solid timber, when compared to unfactored or characteristic estimates there are losses in structural performance, especially when compared to glulam. The finite connection or limited lamination stiffness provided by friction forces is most likely responsible for this performance deficiency. This performance loss, however, needs to be balanced against the adjustable nature and wider compatibility of stress-lamination with low-grade wood, providing prestress losses can be properly controlled.

Disassembly, retesting, and buckling mode shapes

Full-scale structural testing is useful for assessing performance and making comparisons, but the construction and disassembly processes also give valuable insights. The five repeated test cases showed that hardware and boards could be easily disassembled and reused to achieve similar structural performance. Furthermore, of the five repeated test cases, two cases were also performed with special conditions to assess the influence of prestress levels and friction effects: one retesting condition involved higher prestressing, conducted with column 12 in Figure 4.13, while the other involved flipping or reversing each board in the reassembly process of column 15. In the first special case of column 12, the column was retested with a higher applied torque of 58Nm, giving an estimated threaded rod tension of 10.6kN. Note that the original test was conducted with 35Nm of applied torque or 6.4kN of estimated tension. The higher prestressing had little effect on the test results, though, and the column buckled under an applied load of 657kN, only 10% higher than its original buckling load of 598kN. This difference is similar to the other reassembled columns that were retested with no changes in prestress. They also showed repeatable results within about a 10% margin. Note that most retesting cases were performed with slight increases in MoE due to lower moisture content, but this slight increase in stiffness also had minimal effect on structural performance. Although only one column was retested with higher prestressing, the limited change in structural performance suggests a similar finding from the previous detailing tests in shear; provided that prestress levels are controlled and remain above a basic minimum, differences in prestress levels have little influence on structural performance related to stiffness.

In contrast to all the retesting cases, column 15, which had each of its boards intentionally flipped or reversed in the reassembly process, showed an improvement of about 20%. This reversal was performed for a preliminary assessment of how friction effects might affect overall structural performance. Note that simply flipping each board does not alter the structure's effective bending stiffness. Every board retains is original location in the overall column assembly, with each at the same distance from the column's centroid as in original testing. Flipping each board in the reassembly process therefore has no affect on the column's second moment of area. The resultant increase in structural performance of 20%, however, indicates that the surface conditions of the boards and their localised coefficients of friction can noticeably affect the performance of stress-laminated elements. Still, only two limited retesting cases have been altered here to assess the relative influence of prestressing and friction effects. Friction forces are a product of both prestressing forces and the coefficient of friction. The limited results discussed here point to the latter being most likely the more uncertain of the two and capable of affecting performance more profoundly, but more experimentation is needed.

Localised friction effects might also play a role in affecting the buckling mode shape of stress-laminated columns. During testing, some of the columns' buckling mode shapes appeared to slightly deviate from the expected curved or sinusoidal shape. Figure 4.19 illustrates this observation and shows photographs of column 12 before and after testing. The variable shear resistance, due to the localised friction coefficients along the length of the column, may be the cause of this behaviour immediately following the onset of buckling and unrestrained lateral deformation. Theiler's theoretical simulations showed that shear effects have little influence on buckling performance in solid timber and glulam,[27] but after observing this behaviour, the resultant buckling mode shapes were still measured for the last set of four retested columns, as shown in Figure 4.20. The shapes appear to follow the expected sinusoidal shape or simple curvature, but minor deviations away from symmetry are present. Future research and testing can be done to study post-buckling behaviour in more detail.

Figure 4.19: Column 12 before and after structural testing.

Figure 4.20: Final buckling mode shapes in retesting trials with columns 13-16.

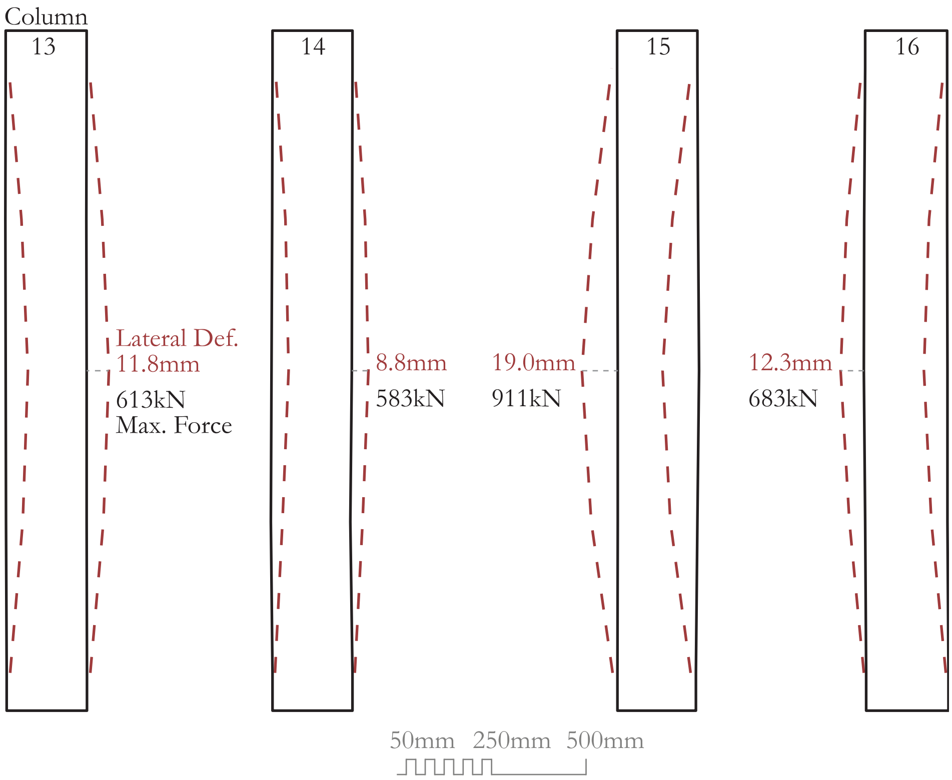


Figure 4.11(k) is also reproduced in a larger format as Figure 4.21. This larger figure shows how the overall buckling shape and lateral deformation of a stress-laminated column includes noticeable shear deformation between adjacent layers. Shear effects playing a visible role in the final lateral deformation of the columns is consistent with the previous observations of deviations away from symmetry due to localized friction effects that act in shear. Although the full-scale columns tested all showed different amounts of shear deformation, this shearing was generally observed to occur after the onset of buckling, as lateral deformations increased rapidly. The shear forces associated with bending from the eccentricity and compression forces overcome static friction forces, yielding lower lamination stiffness and resulting in shear deformation. The relatively little amount of rotation seen in the pinned end support base in Figure 4.21 also emphasises the importance of shear deformations in stress-laminated columns after buckling occurs. More complex end supports that allow for local shear deformation between layers in addition to overall base rotation, like those used by Jumaat[28] for model testing or Alvim and Almeida[29] for full-scale spaced timber column testing, would have been an improvement to the current basic arrangement. Despite the use of more common and simple end supports here, they do not result in unforeseen rotation resistance or fixed end conditions that can significantly affect the buckling length. The overall rotation of the columns at their ends shows that they were not fixed and allowed to rotate. As shear effects can play a subtle but important role

in the buckling of stress-laminated columns, their influence on structural performance is discussed from a theoretical perspective in the following section.

Figure 4.21: Shear deformation between adjacent layers after the onset of buckling.

Summary

Full-scale construction of stress-laminated columns in preparation for structural testing demonstrates the relative ease and simplicity of the lamination technique. Accurate and straight columns within the EC5 initial eccentricity guidelines can also be achieved provided care is taken during the assembly and prestressing stages. After characterising the moisture content and MoE of 128 UK-grown Sitka spruce boards, eleven stress-laminated columns measuring 235x250x3600mm were constructed and tested. Test results confirm that stress-lamination performs in line with EC5 design estimates for solid timber, and is only marginally lower than design estimates for glulam. Compared to EC5 characteristic estimates, stress lamination yielded some losses, displaying lower performance than solid timber and glulam by approximately 26 and 33%, respectively. These performance losses are the prices paid for the increased compatibility but finite stiffness of stress lamination with low-grade wood. Disassembly of the columns also confirmed that the material could be easily reused for structural purposes. Retesting a limited number of reassembled columns showed the repeatability of initial test results. Compared to surface related friction effects, increasing prestress levels appears to offer little advantage for bolstering structural performance, as demonstrated in one of the retested columns. To ensure the rigour of these full-scale results and especially their interpretation, further experimentation but also modelling and comparisons to theoretical predictions can be performed. Additional trials with columns at different slenderness ratios are recommended for future testing.

Theory and modelling related to laminated columns

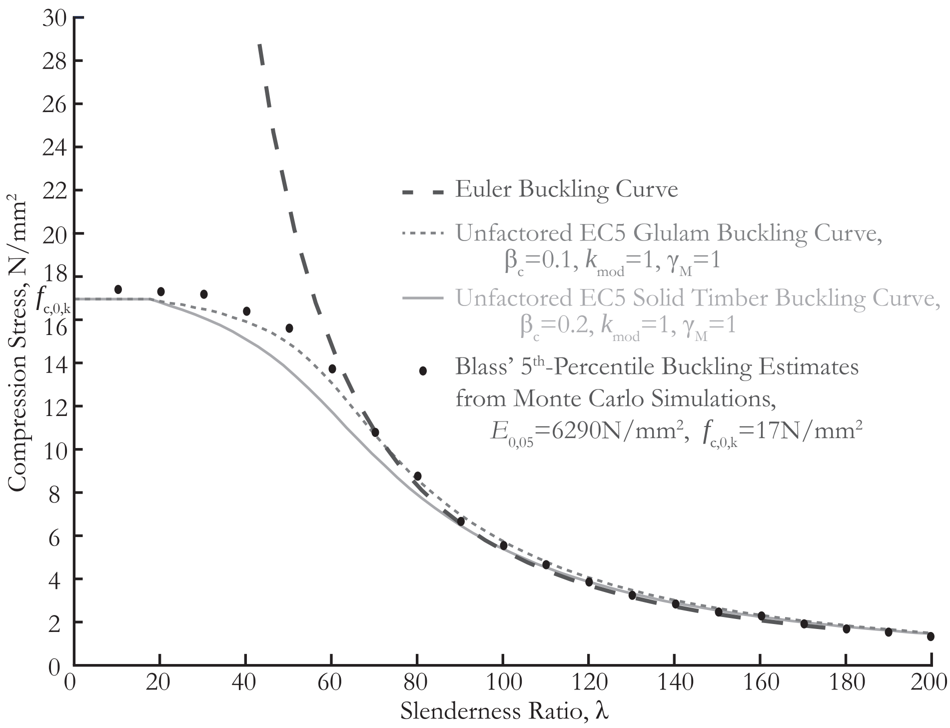
Theory and modelling provide a useful complement to scale models, construction, and full-scale testing. The latter are invaluable for gaining first-hand experience of structural behaviour and for design purposes, but the former is the basis for making predictions and comparisons with existing codes and estimates. The following section therefore reviews and discusses theoretical aspects of solid timber, glulam, and mechanically-laminated columns. There is a rich body of research literature already established on the subject of timber column buckling, following some similarities but also several key differences from literature and analytical techniques for steel. These differences arise due to the unique mechanical properties of wood and their influence on buckling behaviour. Several well-known timber engineering researchers including Blass,[30] Buchanan,[31] and Steiger,[32] all originally performed their doctoral work on timber column buckling. In the case of Blass and Steiger, their efforts were also expanded and updated in 2014 by Theiler,[33] who focused mainly on modelling second-order plastic effects in timber column buckling. By discussing the theory behind timber column buckling and the development of the EC5 buckling curves, more rigour can be established in the previous comparisons with experimental results and their interpretation. Although Eurocodes in general are widely accepted and applied, it is important to understand what the EC5 equations actually represent, the assumptions made in their derivation, and what one is actually comparing experimental data to, thereby ensuring comparisons are unbiased and justified.

Theory and establishment of the EC5 buckling curves

The overall forms of the EC5 buckling curves generally resemble those for steel, but their origins are traced back to the work of Larsen[34] in the 1970s and Blass[35] in 1980s. In 2011, Larsen also wrote a useful summary of the original Eurocode 5 development including a valuable bibliography.[36] As the original EC5 lead author and as part of the CIB-W18 working group, he oversaw the early drafts of EC5 to harmonise the differences in national codes throughout Europe.[37] In the case of timber column buckling, existing codes preceding EC5 were based on linear elastic theory, with varying modifications to take into account initial eccentricities for applied loads or deviations in straightness. For example, in the UK, the Perry-Robertson formula from 1925 preceded the Eurocode buckling curves, and included estimates for the eccentricity of applied load, but not for geometrical imperfections or deviations in straightness.[38] The opposite was true of buckling curves in North American design codes, which failed to consider the eccentricity of loads, while including estimates for the eccentricity due to initial deformations. Larsen's buckling curve proposed in early drafts of EC5 was still based on linear theory, but addressed discrepancies between existing codes by accounting for both types of eccentricities. Estimates for total eccentricity were based on the requirements of sawn timber. The buckling curves were validated with experimental data, but deficiencies in material properties and more subtle influences still could not be adequately modelled without extensive testing.

In cases of combined compression and bending, Buchanan[39] noted that second-order plastic effects in the compression zone of a section could have a significant influence in column buckling. Steiger[40] also made similar observations when studying the interaction of applied moments and compressive forces in columns. These effects were not adequately represented by linear elastic theory in Larsen's earlier EC5 drafts. Relying on the extensive testing and material models of Glos, Blass performed Monte Carlo simulations to recalibrate and update the EC5 buckling estimates.[41] The simulations accounted for variations of several material properties and influences: density, knot area ratio, moisture content, portion of compression wood, and the presence of finger joints in a column. Blass performed hundreds of simulations to determine the characteristic or 5th-percentile buckling load at a given slenderness ratio.[42] The simulation process was then repeated at different slenderness ratios, leading to data points that form a semi-empirical characteristic buckling curve, as illustrated in Figure 4.22. The result of curve-fitting these points with a non-linear function, defined as the buckling instability factor, $k_{c,y}$, is the basis of the present-day Eurocode 5 approach to column buckling. Blass' approach is efficient and thorough, requiring designers to supply only simple parameters including a column's equivalent length or height, basic geometry, and material properties. The non-linear function described by the buckling instability factor accounts for more complex influences and second-order effects, especially in the case of combined compression and bending. There is some conservative aspect to this approach, however, by already accounting for moisture content effects in the characteristic values. Such accounting typically should be done at the stage of determining design values with k_{mod}. When discussing this issue in person with Steiger, the author was reminded that designers following EC5 have no explicit way of accounting for moisture content effects on buckling.[43] The Swiss SIA 265:2014 design code addresses this issue, and includes two separate factors: one for duration of load, and another for moisture content effects, instead of one general k_{mod} factor.

Figure 4.22: Blass' 5th-precentile buckling estimates from Monte Carlo simulations and unfactored EC5 characteristic buckling curves.



Influence of compression strength parallel-to-grain and creep

Around the same time as Blass was formulating an updated EC5 buckling curve, Leicester[44] proposed a simpler approach for the torsional buckling of beams. Leicester noted that the interaction of various parameters and properties influencing buckling in timber are difficult to account for, except at the extremes of the buckling curve: compression strength parallel-to-grain governs at very low theoretical slenderness ratios, while at high slenderness ratios, various effects are all captured by the Euler buckling curve. Leicester's argument is relevant here because the assumed value of the characteristic compression strength parallel-to-grain can affect a major portion of the buckling curve used in practice. In the previous testing and comparisons, the characteristic compression strength of 17N/mm² was taken from the C16 strength class and used throughout.[45] This standard value is also what designers have available to them. Recall, however, that UK Sitka spruce is a stiffness-limited material, with a characteristic bending strength in excess of that specified in the C16 strength class. The UK-grown softwood may also have excess compression strength parallel-to-grain that can bias comparisons.

No full-scale compression strength tests parallel-to-grain have been performed or published for UK-grown Sitka spruce. The BS EN 338:2009 strength class standard offers a means to verify the material's actual characteristic compression strength. Density and MoE are properties determined in grading while all other secondary properties, including compression strength parallel-to-grain, are determined analytically.[46] Steiger and Arnold[47] have also recently rechecked and confirmed the EN 338 formulas with the testing of Norway spruce samples. In the case of UK-grown Sitka spruce with a mean density[48] of 390kg/m^3, and a characteristic bending strength[49] of 18.5N/mm^2 determined from full-scale testing, the BS EN 338:2009 relations yield an estimated characteristic compression strength parallel-to-grain of 18.6N/mm^2. This estimate is about 10% higher than the 17N/mm^2 value used for the previous buckling curves and estimates, suggesting that the previous comparisons are slightly exaggerating the performance of stress-laminated columns. This comparison, however, is only applicable for wood at the reference moisture content of 12%, and moisture affects compression strength far more than the MoE.[50] For softwood with a compression strength parallel-to-grain of 18.6N/mm^2 at a reference moisture content of 12%, its compression strength at 20% moisture content is roughly 24% lower, or about 14.1N/mm^2.[51] Considering the affects of moisture content on compression strength together confirms that the previous comparisons are not exaggerated, but actually somewhat conservative and safe.

Like compression strength and moisture content effects, creep is another factor that can influence buckling behaviour and estimates. While considering the general format of design codes and formulas for buckling, Leicester[52] also studied the effects of creep on timber column buckling. His estimates show that creep effects can lower buckling loads of centrally-loaded columns by roughly 8-10%, depending on the model assumed for the interaction between compression and bending effects. Creep has not been examined or discussed in detail here due to the relatively short duration of tests, but it is important to consider how creep has a lowering effect on buckling loads in testing. For example, the previous EC5 estimates have not included creep effects, thereby introducing another conservative factor in performance comparisons. The uncertainties introduced from compression strength, moisture content, creep, and their mutual interaction, also highlight the complexity involved in column buckling with a natural material like wood. The benefits of reducing uncertainties through controlled temperature and humidity laboratory conditions cannot be understated. The Cambridge University Structures Laboratory would benefit from the addition of such climate controls, even in a small area for full-scale testing of natural materials. Furthermore, designers may not be aware of the complexity embedded and already accounted for in relatively simple EC5 buckling formulas. The task of code writers charged with providing safe but also efficient frameworks and guidance for designers should not be underestimated.

Theory for mechanically-laminated and built-up columns

Compared to solid timber and glulam columns, built-up and mechanically-laminated columns are significantly more complicated to design. While Larsen performed experiments and proposed an initial theory of timber column buckling in early drafts of EC5, he also studied the more complex cases of mechanically-laminated and built-up columns.[53] In the case of solid columns without spacers or packs, with each layer laminated with mechanical fasteners, an efficiency factor, γ, is usually used to account for the finite stiffness and lamination efficiency between adjacent layers. This efficiency factor approach is presented in Annex B of EC5, although not in detail.[54] Larsen's original paper is also not readily available, and the author was only able to receive a copy after directly contacting Dr. Jørgen Munch-Andersen, a senior adviser at Danish Timber Information.[55] The original derivation is somewhat difficult to follow, however, due to the notation and coordinate system chosen. A clearer summary can be found also in Larsen's more recent essays of the development of EC5.[56] The main purpose of the efficiency factor is to calculate an effective bending stiffness of a column, reducing the terms associated with the parallel-axis theorem when calculating the second moment of area. To illustrate, the effective bending stiffness, $(EI)_{\mathrm{ef}}$ is defined as

$$\left(EI\right)_{\mathrm{ef}} = \sum_{\mathrm{n}=1}^{\mathrm{N}} (E_{\mathrm{m,n}} I_{\mathrm{n}} + \gamma_{\mathrm{n}} E_{\mathrm{m,n}} A_{\mathrm{n}} r_{\mathrm{n}}^{2}), \qquad (4.4)$$

where the subscript n denotes a property of the n^{th} layer or lamella: $E_{\mathrm{m,n}}$ is the mean MoE, A_{n} is the cross-section area, γ_{n} is the efficiency factor, and r_{n} is the projected distance from the lamella's centroid and the overall centroid. Figure 4.23 illustrates general strain distributions for γ_{n} values representing an ideal or perfect connection between layers ($\gamma_{\mathrm{n}}=1$), a partial connection ($0<\gamma_{\mathrm{n}}<1$), and no connection ($\gamma_{\mathrm{n}}=0$). Instead of estimating a normal slenderness ratio for a column, an effective second moment of area, I_{ef}, can also be calculated by removing the E terms in equation (4.4). The I_{ef} value can then be used to determine an effective column slenderness ratio, λ_{ef}. While a detailed derivation of this approach is not reproduced here, Larsen did show its validity for centrally-loaded columns, providing certain conditions are met: the bending moment experienced along the column follows a sinusoidal distribution, which is reasonable if the initial eccentricity along the column is also sinusoidal; and the spacing between fasteners, a_{n}, is sufficiently small so that the columns acts as though the lamination is continuous.[57] As the connection details of stress-laminated columns are widely spaced at about 500mm, however, the approach is not applicable. Nonetheless, this efficiency factor approach can still be useful for estimating the performance of alternative forms of mechanical lamination with nails or regular screws.

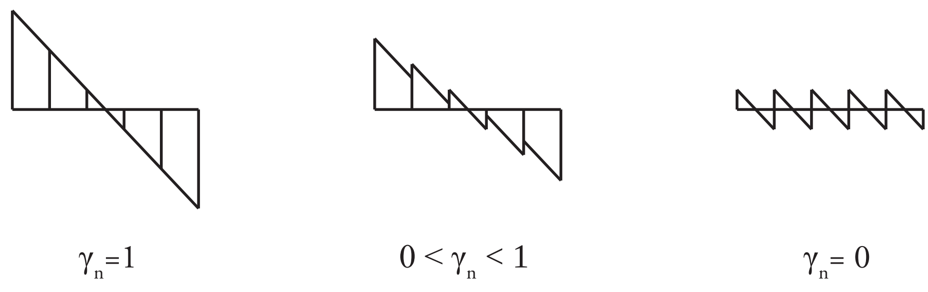


Figure 4.23: Strain distributions of a five-layer section subject to pure bending, with different connection efficiency factors.

The efficiency factor for a layer in a mechanically-laminated column can be calculated considering the slip modulus of a fastener at the ULS, $K_{u,n}$, the fastener spacing, a_n, and effective height of the column, h:

$$\gamma_n = \left(1 + \frac{\pi^2 E_{m,n} \; A_n a_n}{K_{u,n} h^2}\right)^{-1}. \tag{4.5}$$

For nail-laminated and regular screw-laminated columns, their slip moduli can be estimated from equations (4.1) and (4.2), and then used to provide simple performance estimates for comparison purposes. Table 4.2 summarises γ_n values for nails and screws at different spacing. Note that the slip modulus at the ultimate limit state is simply two-thirds of the initial slip modulus, $K_{ser,n}$. The values summarised in Table 4.2 are also calculated using the same dimensions of the columns already tested. $E_{m,n}$ is taken as 8kN/mm^2 following the standard mean value for the C16 strength class representative of UK-grown Sitka spruce. The results in Table 4.2 show that even with a close fastener-spacing of 20mm along the entire length of the column, the efficiency factor and effective second moment of area are significantly lower than those associated with both non-laminated and glued structures, where γ_n=1. Figure 4.24 illustrates the 20mm spacing and high density of fasteners needed. Even in the best case of using regular screws at a spacing of 20mm, the slenderness ratio jumps from a value of 53 in the ideal case of γ_n=1, to an effective slenderness ratio of 86. Shifting along the characteristic EC5 solid timber buckling curve from a slenderness ratio of 53 to 86 in Figure 4.18 results in an estimated 50% loss in structural performance. The relatively low slip moduli of nails and regular screws influence lamination performance considerably. In light of these estimates, experimental test results demonstrate that stress lamination can provide better structural performance compared to conventional nail- and regular screw-laminated columns.

Table 4.2: Lamination efficiency factor estimates for nails and regular timber screws at different spacings.

Type	ULS Slip Modulus, $K_{u,n}$, N/mm	Spacing, a_n, mm	Efficiency Factor, γ_n	Effective Second Moment of Area, I_{ef}, mm^4	Effective Slenderness, Ratio, λ_{ef}
4mm Nails (without predrilling)	518	20	0.27	79.9×10^6	98
		40	0.15	50.6×10^6	123
		80	0.08	32.4×10^6	153
4mm Regular Timber Screws	890	20	0.38	110.5×10^6	83
		40	0.20	72.5×10^6	102
		80	0.11	45.8×10^6	129

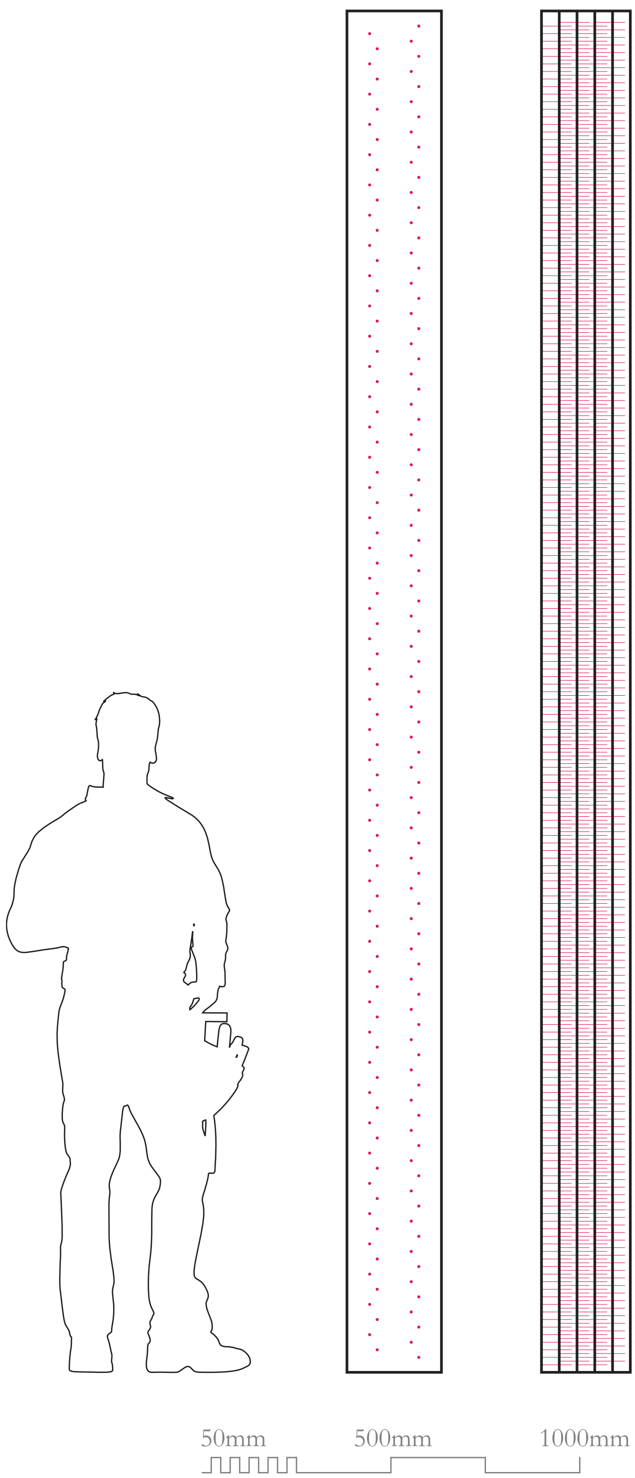


Figure 4.24: Drawing of a nail-laminated column with an effective fastener spacing of 20mm.

An alternative theoretical approach also worth considering is that for built-up columns, spaced and joined with short blocks or packs and mechanical fasteners. Compared to solid mechanically-laminated columns with nails or screws, built-up columns have more in common with the stress-laminated varieties already built and tested. In both built-up and stress-laminated columns, shear resistance is concentrated at a few widely spaced locations along the column, rather than being evenly spread with closely spaced and small fasteners like nails or regular screws. EC5 estimates for built-up columns, however, do not offer significant details on stiffness and lamination effects. Ceccotti[58] has also discussed the mechanics of built-up timber structures in somewhat greater detail, although the study is mainly a reproduction of Larsen's earlier work. Furthermore, Ceccotti and Larsen appear to follow the approach taken in the original German standard DIN 1052, which was most likely derived from Möhler's earlier experiments and modelling.[59] Rather than returning to first principles, the approach in EC5 for built-up columns appears to be adapted from the existing theory for mechanically-laminated columns, ignoring the small fastener-spacing assumption. To quote Ceccotti,[60] for estimating an effective slenderness ratio for built-up columns, the 'resulting formula is a sort of average square root of the slenderness of the entire column and the slenderness of single shafts' or layers:

$$\lambda_{ef} = \sqrt{\lambda^2 + \mu_{pb}\frac{N}{2}\lambda_n^2}\ , \tag{4.6}$$

where λ is the slenderness ratio associated with an ideal lamination efficiency, the number of layers in the built-up section is N, while λ_n is the slenderness ratio of an individual layer, and μ_{pb} is an alternative lamination efficiency factor. This last μ_{pb} factor is set to 1 for packs connected with glue, 2.5 for bolts with timber connectors, and 3 for nails in the case of short-term loading.[61] This approach is problematic as the spacing between packs is only indirectly accounted for through the λ_n value. Furthermore, neither Ceccotti nor Larsen give details on how this expression was derived. Additionally, any non-zero value of λ_n results in an increase in λ_{ef}, even if $\lambda_n < \lambda$. This behaviour suggests that even built-up columns with closely spaced and glued packs will display performance losses, which is rather dubious. For the stress-laminated columns tested, although the layers are actually in contact rather than spaced with packs, equation (4.6) yields an estimated effective slenderness ratio of 97, assuming an ideal lamination efficiency with gluing. This high estimate for the effective slenderness ratio suggests structural performance would be significantly reduced, and does not agree well with experimental evidence. For updating design codes to include provisions for stress-laminated columns, new theoretical derivations are therefore recommended for future studies.

Summary

The theory and modelling for timber column buckling is complex, primarily due to material and geometrical influences, even at the code level in EC5. Much modelling work has already been done for the basic types of solid timber and glulam columns. Uncertainties and the complex interaction of influences are greatest in the intermediate slenderness range, and the previous comparisons with EC5 buckling curves can also be influenced by the assumed value for the compression strength parallel-to-grain. Rather than relying on the given C16 strength class value, the compression strength of UK-grown Sitka spruce was estimated based on density and bending strength and also considering moisture content effects. The estimate for UK-grown Sitka spruce is about 10% higher than the standard C16 value for the compression strength parallel-to-grain, but moisture content and creep effects are also expected to lower standard estimated buckling loads by roughly 30%. The previous performance comparisons of stress-laminated columns are therefore somewhat cautious and conservative. EC5 estimates for mechanically-laminated columns with nails and regular timber screws further show that these types of conventional fasteners offer lower performance than the full-scale stress-laminated columns built and tested. Theoretical work and modelling for spaced, built-up columns has been improvised based on mechanically-laminated columns, and leads to suspicious results that do not agree with experimental data. The theory and design codes for built-up timber columns should therefore be revised with considerations from first principles. The development of new theoretical models for built-up columns could also be done in such a way to include provisions for stress-laminated columns.

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Chapter 5 FURTHER TESTING AND ACCOUNTING FOR PRESTRESS LOSSES

Improved connection detail with hardwood

Small changes and further development can lead to improvements in the construction and structural performance of stress-laminated columns and building elements. Drilling was the critical stage identified during full-scale construction, so updating the threaded rod and bearing plate detail was an important area for improvements. By simply reducing the number of threaded rods and holes in the bearing plates, the lengthy and sometimes tedious drilling process can be shortened. While reducing the number of threaded rods also lowers the maximum shear strength provided by the connection or detail, stiffness rather than strength has been identified as an important parameter in previous testing and modelling. Furthermore, steel bearing plates are commonly used in bridge construction, primarily for their strength and durability for withstanding weather conditions outdoors. The strength of steel or aluminum bearing plates is not needed when prestress forces are effectively distributed over multiple threaded rods. With indoor rather than outdoor conditions, durability requirements for the plates are also eased, providing an opportunity for alternatives to steel and aluminum. Reducing the overall amount of metal in stress-laminated detailing for buildings would also be beneficial from cost and fire safety perspectives, as metal connections conduct heat into the wood more effectively.

Figure 5.1 shows a drawing of a new bearing plate for stress lamination that was made from hardwood grown and readily available in the UK. Compression strength perpendicular-to-grain was the first consideration for selecting a suitable hardwood species for the plates. Even the lowest grade of structural hardwood in the D18 strength class listed in BS EN 338:2009 has a characteristic compression strength perpendicular-to-grain of 7.2N/mm^2, over three times that of C16 graded softwood.[1] The estimated threaded bar force of 6.4kN in the previous testing was concentrated over a 40mm diameter washer, or an area of about 1180mm^2 to give an average compression stress of roughly 5.4N/mm^2. Common UK-grown hardwood species that are worth considering for bearing plates include oak (*Quercus rubra*), ash (*Fraxinus excelsior*), beech (*Fagus sylvatica*), and birch (*Betula spp.*). Table 5.1 summarises the mean density values from measurements by Lavers[2] of these UK hardwood species of interest. The characteristic density and compression strength perpendicular-to-grain of each species are estimated from the mean density values following the simple linear equations in BS EN 338:2009.[3] All of the species surpass the D40 strength class for density and compression strength perpendicular-to-grain, and would be suitable for the new bearing plates.

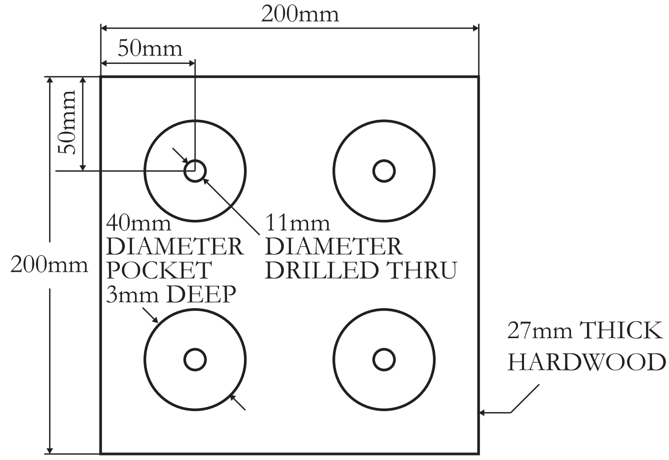


Figure 5.1: Drawing for new hardwood bearing plate.

Table 5.1: Density and perpendicular-to-grain compression strength of common UK-grown hardwood species.

Species	Mean density, ρ_m, kg/m^3	Characteristic density, ρ_k, kg/m^3	Characteristic compression strength perp.-to-grain, $f_{c,90,k}$, N/mm^2
Oak	705	588	8.8
Ash	689	574	8.6
Beech	689	574	8.6
Birch	674	562	8.4

Overdrying plates for controlling prestress losses

A second key advantage of switching from metal to hardwood for the bearing plates is that the new plates could also be overdried before prestressing. Overdrying hardwood plates would allow them to gradually expand while reaching an equilibrium moisture content in-service after prestressing. Expansion can lessen or potentially mitigate prestress losses due to softwood movement and creep. This idea of overdrying hardwood elements for prestressing benefits is unique, going hand-in-hand with the concept of using stress lamination for buildings. Analogies can be drawn to *Brettstapel*, however, which uses overdried hardwood dowels that subsequently expand and lock the panel boards together without glue. An interesting study by Anshari *et al.*[4] also inserted small densified pieces of compressed and overdried Japanese cedar (*Cryptomeria japonica*) into pockets cut in glulam beams to prestress them, thereby increasing their stiffness and load carrying capacity. In most structural applications, however, species with high rates of movement with changes in moisture content are undesirable. In many cases they are strictly avoided, especially in outdoor applications. The hardwood species with the highest rate of movement, however, is the most desirable in the present application, providing other considerations like strength and availability can be met.

In principle, the softwood in stress-laminated columns could also be overdried, but such action would be costly given the volume of wood involved. Low-grade wood like UK Sitka spruce would also tend to exhibit significant twisting after being overdried to moisture content levels far below those found in service conditions. Hardwood plates are much smaller in size and are therefore easier to overdry. In addition to solid columns, the idea of overdrying hardwood plates would also be very useful for blocks or packs in spaced or built-up stress-laminated columns. The expanding hardwood used for packs or spacers could counteract prestress losses in the case of stress lamination, but also lessen the potential for tension forces perpendicular-to-grain in the packs. In solid stress-laminated columns, the overdried plates might also be useful as simple shear keys, but this more complex configuration would require additional construction time and effort, especially with routing keyways in the softwood boards. In all these situations, the natural characteristics and properties of wood are put to good use for structural benefit instead of avoiding or suppressing these properties through processing.

Hawker and Turnbull[5] of Edinburgh Napier University have already conducted a preliminary study of the moisture-movement characteristics of common UK hardwoods. Their work was part of a pilot study on UK-based *Brettstapel* production. The results of their testing show that beech and birch are the most desirable species from those listed in Table 5.1. A way to further enhance the high movement rates of these species is by making the plates from quarter-sawn wood. Figure 5.2 shows how the tangential direction of the close-grained, quarter-sawn hardwood plates is aligned with the axial direction of the threaded rods. This small detail is meaningful as the movement rates due to changes in moisture content in the tangential direction of wood are typically double that of the radial direction.[6] Using quarter-sawn wood for the plates would therefore maximise the high-movement characteristic of beech or birch. Although quarter-sawn material is usually associated with furniture or cabinets and is more costly than regular plane-sawn wood, the small volume of the bearing plates would not lead to significant cost increases overall. The challenge is finding a sawmill or supplier with suitable quantities of quarter-sawn beech or birch already on hand for full-scale testing. After contacting several mills in northern England and Scotland, the author found a small sawmill, White Wood Management, near Devon that could offer both species in quarter-sawn patterns. The mill's operator, Mr. Jim White, was also very much interested and supportive of the idea of using UK-grown hardwoods in new ways for structural purposes. He noted during correspondence that current standards for visual grading are severely limited in terms of their allowable species.[7] The mill already had sufficient quantities of air-dried and quarter-sawn beech on hand, but was also willing to quarter-saw some forthcoming birch logs. Quarter-sawn beech was therefore chosen for the plate material due to its immediate availability. Using beech for overdried bearing plates in stress lamination also contrasts and shows an alternative use of the species next to the growing interest in beech glulam and LVL in Switzerland and Germany.[8]

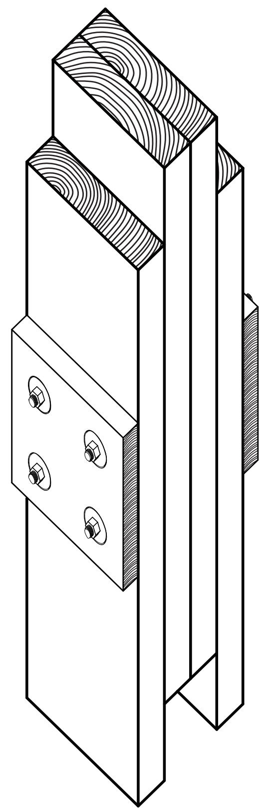


Figure 5.2: Detail of the new hardwood bearing plates with their tangential grain direction aligned with the threaded rods.

Updated shear testing and results

Five pairs of bearing plates were made according to Figures 5.1 and 5.2 and used for subsequent shear testing. As an improvement of the previous full-scale shear tests, five identical prototype details were made at full-scale with UK-grown Sitka spruce. They were assembled with the new bearing plates for further refined testing. Figure 5.3 shows one of the five prototypes. Before shear testing, each prototype was prestressed to a different level with a calibrated manual torque wrench. Strain gauges were also bonded to special 25mm long M10 stainless steel collars, as illustrated in Figure 5.4, thereby providing simple transducers for fitting around each threaded rod and measuring its tension. Finally, two linear variable differential transformers (LVDTs) were mounted on each prototype before testing to provide an accurate measurement of each detail's slip and stiffness. These additional transducers, and the use of different prototypes for investigating the performance of different prestress levels, address the limitations of the previous shear testing. Finally, testing was also performed according to BS EN 26891:1991, with each test involving cycling between the 10 and 40% levels of the estimated EC5 ultimate load.[9]

Figure 5.3: Updated shear testing with one of five new prestressed connection details with plates made from UK-grown beech.

Figure 5.4 shows the results of the shear testing, with the applied compression force plotted against the actual displacement or slip of the prototypes. In contrast to the previous relative comparisons and repeated testing of one prototype detail, using the displacement transducers gives a much clearer and absolute picture of the enhanced stiffness performance from prestressing. The initial slip moduli or stiffness of all the prestressed and stress-laminated prototypes is approximately 1500kN/mm, where the benchmark prototype detail with no prestressing showed an initial slip modulus or stiffness of about 7kN/mm. On the other hand, the earlier estimates for the stiffness of various mechanical connections in Table 4.1 was about 3-14kN/mm, depending on the number of shear planes and

fasteners used. These new test results confirm how the initial stiffness from prestressing and stress lamination can compete with that of gluing and outperform mechanical connections with nails, regular screws, and typical tooth-plate timber connectors.

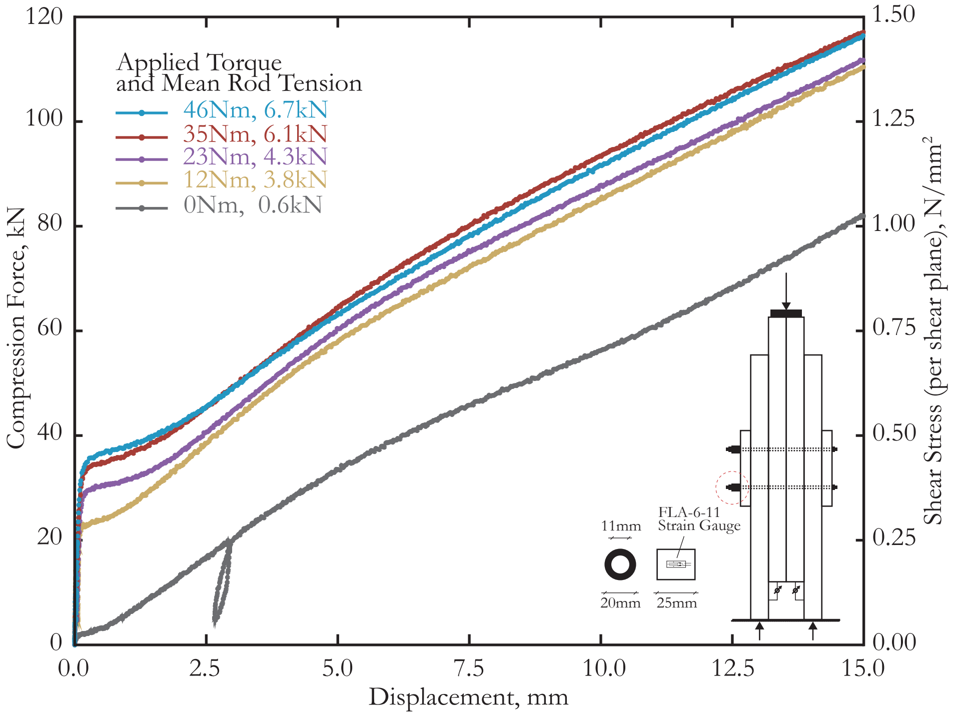


Figure 5.4: Results from updated shear tests with different prototypes having different prestress levels.

The point where prestressing and friction forces are overcome in each test can also be seen in Figure 5.4. For comparison purposes, Figure 5.5 presents the sum or total of the measured initial tension in the threaded rods of each test. The total tension is defined here as the sum of the tension in each of the four individual threaded rods in each detail. Considering the total tension and the applied compression force when friction forces were overcome in each test, the estimated mean static coefficient of friction for the wood is about 0.37. This value lies firmly between Taylor's et al.[10] published friction factors of 0.31 and 0.48 for planed and rough-sawn timber, respectively. Figure 5.6, however, also shows the individual and mean threaded rod tension measured in all tests. Considerable differences in the tension of different threaded rods prestressed at the same torque level can be seen. The total and mean tension for the lowest 12Nm and highest 46Nm prototypes are about 70% higher and 26% lower, respectively, than the estimated levels from the applied torque. The torque-tension estimates for the intermediate applied torques of 23 and 35Nm agree well with the estimated mean and total tension levels, as shown in Figure 5.5. Note that all the previous constructed columns were prestressed in this intermediate range, with an applied torque of 35Nm. The overestimation of the tension in the highest torque case sheds further light on why a higher prestress level in the previous repeated testing of column 12 showed little difference. With an applied torque of 46Nm and higher, the threaded rods are being over-tightened with only marginal gains in tension. The applied torque of 35Nm used in previous full-scale column testing is therefore recommended for future testing and construction.

Figure 5.5: Comparison between the estimated total tension based on applied torque and the measured total tension in shear tests.

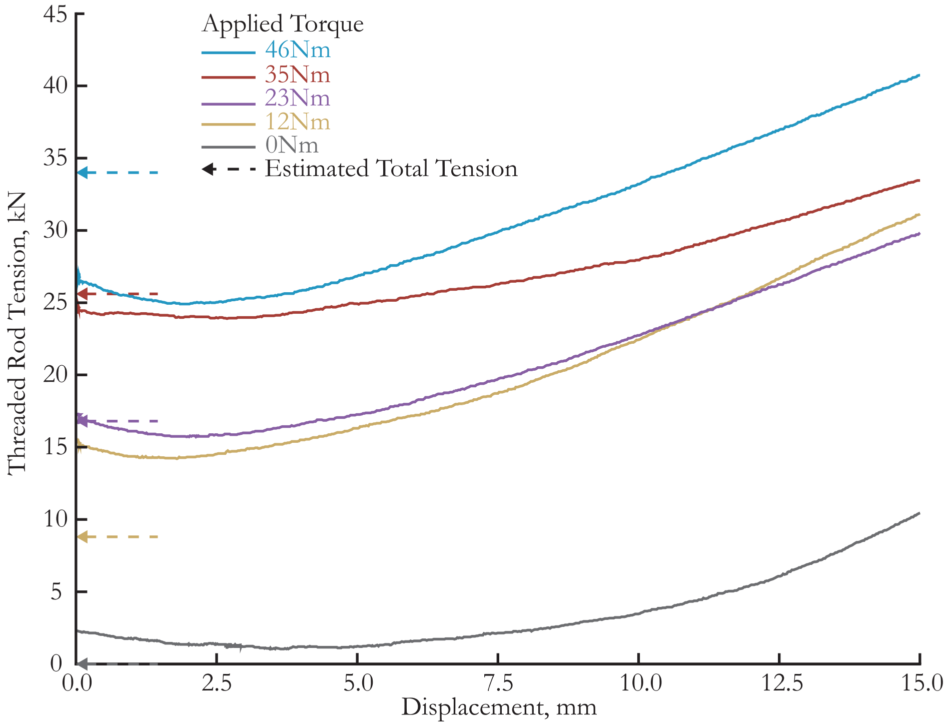
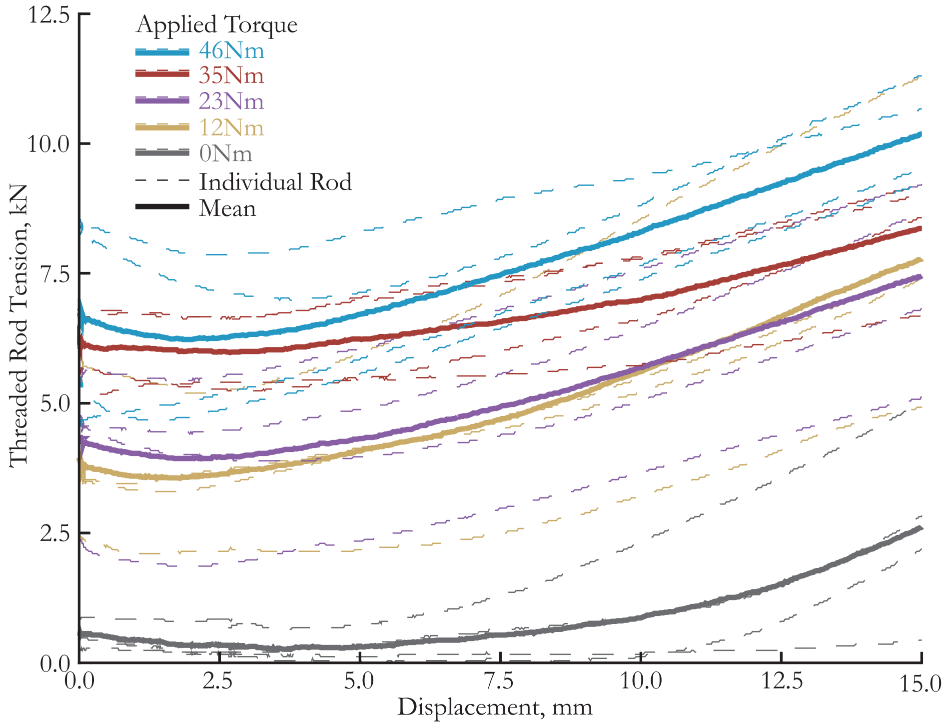


Figure 5.6: Individual and mean threaded rod tension results from updated shear tests.



Alternatives to stress lamination with self-tapping, axially-loaded screws

Table 4.1 summarised the estimated stiffness performance for conventional mechanical connection types covered by EC5. While performing the updated shear testing, the author realised that two special types of connections are not adequately covered by these estimates. Based on discussions with Dr. Jørgen Munch-Andersen, who is currently overseeing the updated EC5 section on connections, the author learned that EC5 is in the process of a major update.[11] The current EC5 stiffness or slip moduli equations and estimates have not yet been updated for self-tapping, axially-loaded screws when installed at an incline. In contrast to regular timber screws, self-tapping screws feature threads that are rolled rather than cut, with the threads extending further out from their shank than in conventional lag bolts or regular timber screws.[12] A hardening process further enhances their strength in bending, tension, and torsion after rolling the threads. The screws' thread dimensions increase their axial withdrawal capacity to about 1-10kN or more depending primarily on the screw length, l, angle with respect to the grain, α, and mean wood density, ρ_m, among other secondary factors.[13] Angling these axially-loaded screws in a connection therefore engages this high withdrawal capacity along the length of the screw.[14] Self-tapping, axially-loaded timber screws can also come in long lengths, up to several meters, and when specified as fully-threaded, they can be used in connections or reinforcing roles in either tension or compression perpendicular-to-grain. The screws have already been discussed in their ability to reinforce against crushing, but they are also useful for preventing splitting and for providing local reinforcement around dowels.[15]

Timber rivets are another type of specialised fastener that are similar to nails and used extensively in North America for connections, especially in trusses. Like self-tapping screws, they are also hardened and useful for connections requiring very high load capacity and stiffness.[16] Rivets are used in combination with predrilled steel plates, and in recent years they have found new application in New Zealand for connections between dampers or anti-seismic devices and timber structures.[17] When used in groups of tens or hundreds, however, they can provide significant stiffness in a connection. Their ease of installation and small size are relatively cost-efficient, but group-effects and brittle failures can be also critical.[18] Like self-tapping screws, timber rivets are not properly covered by the current EC5 equations estimates, as they are seldom used in Europe. The maximum stiffness of a timber rivet connection in LVL, tested by Zarnani and Quenneville with groups of 64 timber rivets per shear plane, was about 380kN/mm, corresponding to 3kN/mm per fastener per shear plane. Ignoring the effects of the increased density of the LVL, this rivet slip modulus is just over four times that of a 2mm nail following EC5 estimates with solid timber. The estimated stiffness of a timber rivet, though, is still significantly lower than the initial slip modulus measured in the prestressed prototype testing.

Compared to rivets and other types of mechanical fasteners and connectors, estimating the slip modulus of self-tapping screws is far more complex. Instead of following the current slip modulus equation for screws in EC5, timber engineers in practice[19] use the equations and estimates found in European technical approvals (ETAs) for the particular type of self-tapping screw they are specifying for a project. These estimates are still somewhat simplified, and Tomasi *et al.*[20] and Jockwer *et al.*[21] have performed more detailed studies on the strength and stiffness of simple self-tapping screw connections in shear to assess the effects of a screw's angle with respect to the grain. Table 5.2 summarises the slip moduli on a per fastener per shear plane basis, observed in their studies for various angles and configurations: shear with the screws in compression (S-C), shear with the screws in tension (S-T), and an 'X' configuration (S-X) with pairs of screws in shear, one in tension and the other in compression. An ETA estimate[22] for a self-tapping screw's axial withdrawal slip modulus, along with the EC5 estimate for regular timber screws are both included in Table 5.2 for reference. When used properly in tension at an ideal 45° angle to the grain, self-tapping screws significantly outperform older types of mechanical fasteners like screws, nails, and bolts. Like timber rivets, though, they offer far less initial stiffness and lamination efficiency than that measured in the stress-laminated prototypes.

Table 5.2: Slip moduli for a prestressed threaded rod, regular timber screws, and self-tapping, axially-loaded timber screws.

<table>
<tr><th>Type</th><th colspan="12">Slip Modulus, K_{ser}, kN/mm</th><th>Notes</th></tr>
<tr><td>Prestressed 10mm threaded rod</td><td colspan="12">187.5</td><td>Measured from Figure 5.4</td></tr>
<tr><td>4mm regular timber screw</td><td colspan="12">2.0</td><td>EC5, Table 7.1, ρ_m=390kg/m^3</td></tr>
<tr><td>8mm self-tapping screw, l = 100mm</td><td colspan="12">7.5</td><td>ETA-11/0190, ρ_m=390kg/m^3, Axial withdrawal stiffness, independent of α</td></tr>
<tr><td rowspan="2">8.5mm self-tapping screws,
l = 155, 127, 110mm</td><td colspan="4">α=45°</td><td colspan="4">60°</td><td colspan="4">90°</td><td rowspan="2">From Jockwer et al., ρ_m=420kg/m^3</td></tr>
<tr><td colspan="4">13.6</td><td colspan="4">5.9</td><td colspan="4">1.9</td></tr>
<tr><td rowspan="4">8.2mm self-tapping screws,
l = 190, 220mm</td><td colspan="3">α=45°</td><td colspan="3">60°</td><td colspan="3">75°</td><td colspan="3">90°</td><td>From Tomasi et al., ρ_m=426kg/m^3</td></tr>
<tr><td colspan="3">1.1</td><td colspan="3">1.4</td><td colspan="3">1.6</td><td colspan="3">1.1</td><td>(S-C)</td></tr>
<tr><td colspan="3">8.2</td><td colspan="3">4.9</td><td colspan="3">3.1</td><td colspan="3">1.1</td><td>(S-T)</td></tr>
<tr><td colspan="3">14.4</td><td colspan="3">10.5</td><td colspan="3">7.0</td><td colspan="3">2.3</td><td>(S-X), where K_{ser} values are for an 'X' pair of screws working together across 1 shear plane</td></tr>
</table>

Alternative forms of stress lamination with self-tapping screws still might be useful, albeit with somewhat lower performance than prestressing with threaded rods. A study by Steilner[23] has reported on the use of self-tapping screws with varying thread pitch that can prestress two pieces of wood together. The study, however, only considered wood already kiln dried and conditioned to 12% moisture content. If used with low-grade wood at higher moisture content, a screw's prestressing effect would still be subject to the same prestress losses from movement and creep as in stress lamination. Like nail-lamination, there would be no way to adjust or retighten inner layers prestressed with self-tapping screws. Stress lamination was originally developed as a repair technique for this specific problem in older nail-laminated bridge decks. A better way to use self-tapping screws for prestressing and stress lamination might be to first start with relatively short, partially threaded and large head-diameter screws to temporarily hold adjacent layers together. Larger diameter and longer fully-threaded screws can then be applied at a 45° angle to the grain for stiffening and strengthening the lamination in shear. The performance of such a configuration is examined in greater detail later on. Self-tapping screws would avoid the need for any drilling, but unlike threaded rods, they would not be able to retighten layers back together after movement from changes in moisture or shrinkage. A hybrid solution involving both threaded rods and self-tapping screws may prove more feasible than stress lamination with only one type of fastener. Hybrid configurations are recommended for future research.

Summary

An updated but simplified bearing plate detail made from quarter-sawn, UK-grown beech reduces the drilling and threaded rod requirements in stress-laminated columns. Specifying the plates as quarter-sawn and overdrying them prior to prestressing is expected to minimise prestress losses. Several full-scale prototype details tested in shear illustrate the benefits in stiffness from prestressing and stress lamination more clearly. The prestressed prototypes all outperformed the benchmark test with no prestressing, along with the estimated slip moduli of all other types of mechanical fasteners. Self-tapping, axially-loaded screws are the highest performing mechanical fasteners available, if used properly and installed at an angle with respect to the grain. They require no drilling compared to threaded rods, but will most likely result in lower stiffness and lamination efficiency. Instead of following their dominant use in connections or in reinforcing roles perpendicular-to-grain, they may be also useful in alternative forms of stress lamination, or in hybrid configurations with threaded rods and self-tapping screws. Before recommending such new applications, further full-scale testing is needed with the new hardwood bearing plates, along with testing the idea of overdrying the plates to reduce prestress losses.

Additional full-scale tests

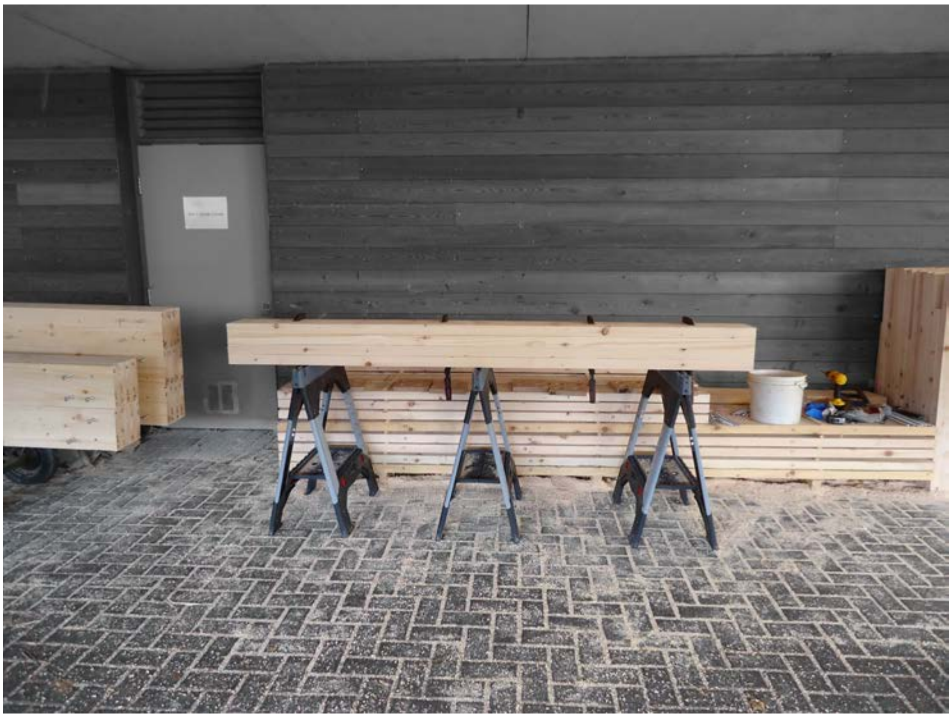
The new detail with quarter-sawn beech bearing plates can be put to further use for additional full-scale construction of columns, in particular those with a different slenderness ratio than those already tested. The previous columns measuring 235x250x3600mm had an ideal slenderness ratio of 53 in the intermediate range. A smaller slenderness ratio of 44 was chosen for further testing rather than building and testing very slender columns that are rarely seen in practice. A slenderness ratio of 44 corresponds to columns measuring 188x200x2400mm, made of four layers each measuring 47x200x2400mm. A lower slenderness ratio means that the columns buckle at a higher compression stress, closer to the wood's characteristic compression strength parallel-to-grain. For the new column dimensions, the majority of the remaining untested UK-grown Sitka spruce was accurately ripped down to a width of 200mm and cut to a length of 2400mm, as shown in Figure 5.7(a). Figure 5.8 illustrates the bearing plate spacing of 700mm on-centre for all the columns built and tested, with 500mm between the edges of adjacent plates. As all of the boards used in the previous columns were not cut to width, and being rough sawn, their width varied by a few millimetres along their length. After ripping the remaining boards, having consistent and accurate parallel edges for measuring and aligning the bearing plates before drilling was helpful. Figures 5.7(b) and (c) show the parallel edges during assembly. Furthermore, having the width of the bearing plates match the full width of the columns also reduced drilling inaccuracies and greatly improved construction time and the overall quality of workmanship. With the plate width matching the column width, each plate needed to be only accurately positioned along the length of the column. The previous plates in chapter four had to be accurately positioned along the length and width of a column, and also aligned without rotation.

Figure 5.7 (below and opposite): Construction sequence of additional stress-laminated columns.

(a) Ripping and cutting individual boards to a consistent width of 200mm and a length of 2400mm, respectively.

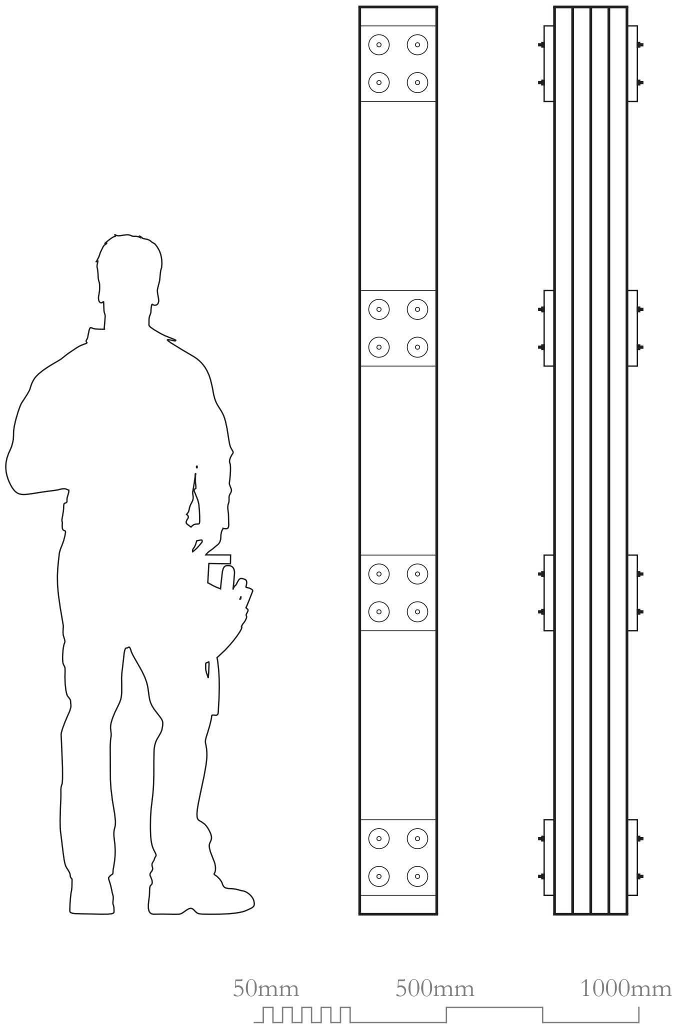
(b) Assembly of inidividual boards with accurate alignment of the boards' parallel edges.

(c) Clamping of the assembled column prior to marking the bearing plate positions.



(d) The bearing plates are finally positioned and clamped in the assembly for drilling.

Figure 5.8: Updated column spacing and dimensions with new hardwood bearing plates.



Test methods and updated equipment

After making an additional seven pairs of quarter-sawn beech bearing plates, a total of twelve pairs of bearing plates could allow a set of three complete columns to be built and tested at one time. Note that the bearing plates were made in pairs and drilled simultaneously to ensure precision and accuracy. Figure 5.9 illustrates the annual rings at the bottom of the nine columns built and tested. Figure 5.9 also summarises the measured mean moisture content of each board at the time of testing, and the estimated MoE values corrected for moisture content. The same 5000kN Amsler press was used again for the full-scale structural tests, although with significant additions in testing instrumentation. The updated testing equipment is shown and illustrated in Figures 5.10 and 5.11, respectively: five wire gauges for measuring a column's buckling mode shape and lateral deformations or eccentricity during testing, and LVDTs on the top and bottom plate for examining overall stiffness effects. One of the special advantages of glulam is the ability to spatially redistribute defects such as knots in a column or beam. As a result of this random redistribution through ideal glue-based lamination, improvements in overall strength and stiffness are achieved compared to the constituent timber of a beam or column. Examining stiffness effects in column testing is useful for assessing if stress lamination offers similar improvements in stiffness.

Figure 5.9: End grain illustrations, mean moisture content, and estimated MoE of the individual layers in the new stress-laminated columns in structural testing.

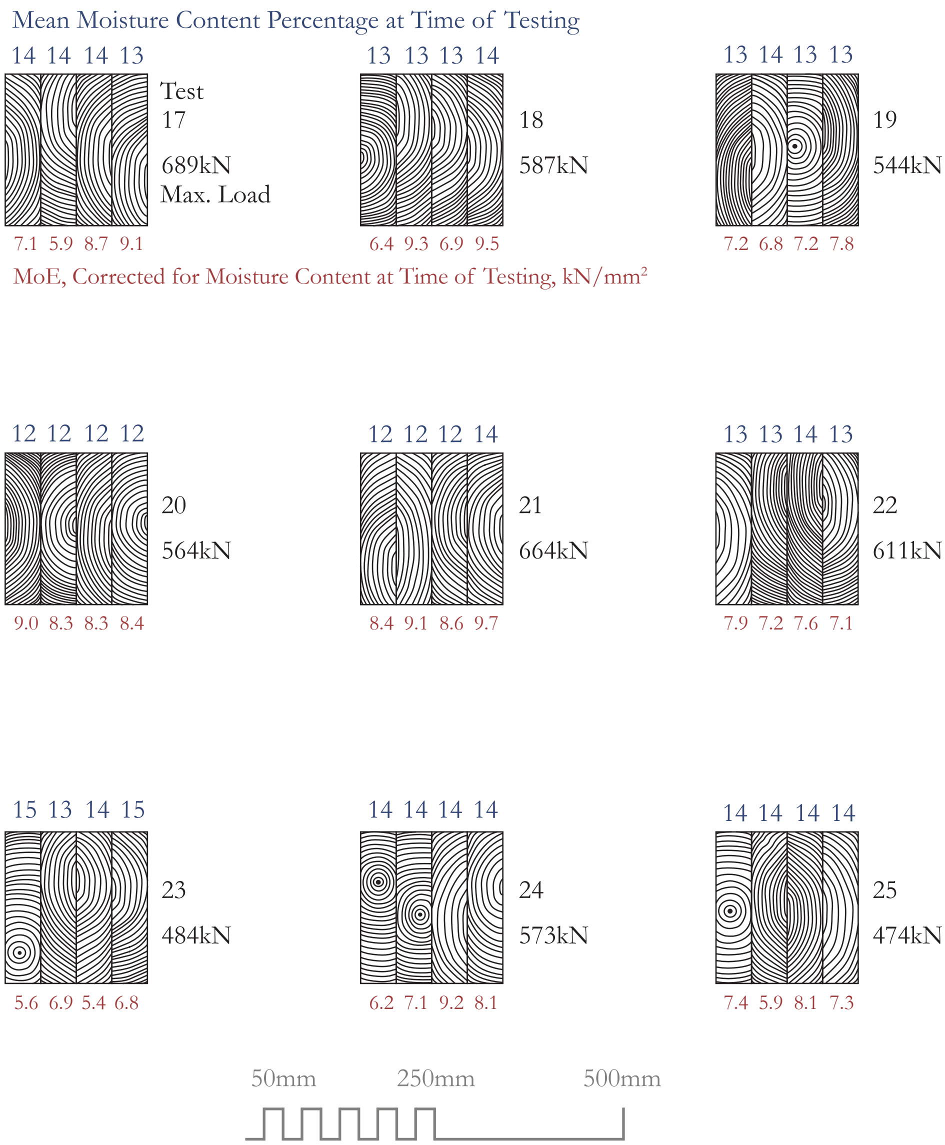


Figure 5.10(a) above left, (b) above right, and (c) right: positioning of the LVDTs for measuring the displacement of the **(a)** top and **(b)** bottom plates. **(c)** Arrangement of wire guages along the column for measuring lateral deformation and buckling mode shapes.

Figure 5.10(d) opposite above, (e) opposite below right, and (f) opposite below left: (d) alignment of a column prior to testing, and **(e)** buckling shape and **(f)** shear deformation after testing.

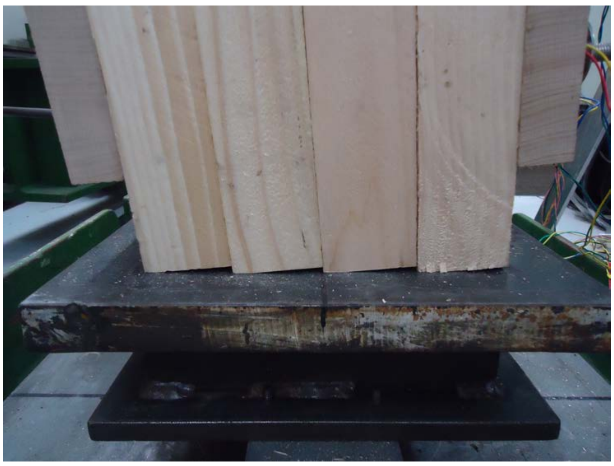
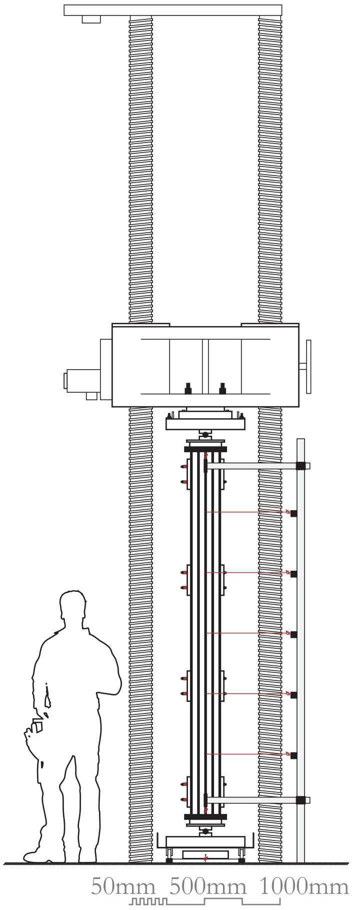


Figure 5.11: Amsler press with additional instrumentation, including LVDTs on the top and bottom plate, wire gauges, and tension transducers for each threaded bar in a column.



As most of the wire gauge and LVDT transducers were either uncalibrated or missing calibration factors, they were all calibrated and checked before testing. At this time the author also checked the Amsler head displacement displayed and recorded by control software, finding that the displayed head displacement is approximately 10% low compared to actual dial-gauge measurements. Since only the maximum load and strength effects were the primary factors of interest in previous testing, this error has no effect on the discussion and conclusions in the previous chapter. For examining stiffness effects in more detail in the previous testing, the head displacement data would need to be corrected with a suitable calibration factor. For proceeding with new testing, sixteen additional M10 stainless steel collars were fitted with strain gauges for tracking threaded bar tension throughout compression testing like in the previous shear tests. Each column was therefore prestressed in-situ in the Amsler press with the simple collar transducers measuring the tension in each threaded bar. With each column weighing an estimated 35kg, they were much easier to handle, position, and prestress in the Amsler press as compared to the previous larger 80kg columns. The first set of new columns tested were also checked for straightness, with the deviations in their initial eccentricity or straightness measured to be within one millimetre. This improvement in straightness is most likely due the columns' smaller dimensions, especially their shorter length, but also the improvements made in the construction process. Furthermore, no noticeable twisting was observed in these smaller columns. After testing nine columns having a lower slenderness ratio of 44, reassembly and repeated testing were not performed due to scheduling limitations with the Amsler press. At this stage, approximately 107 of the 128 UK-grown Sitka spruce boards had also been used up in the testing program, so the amount of characterised material was also nearly exhausted. The remaining boards were mainly those that showed noticeable defects and twisting.

The increasing complexity of testing equipment also comes with more potential for errors or misconfigurations, at least initially. The first two structural tests were marred by two minor setbacks that needed to be addressed in post-processing. The first involved a poorly positioned LVDT on the top plate, causing the displacement transducer to slip off the top plate part way through the first test. Rather than stopping the test and unloading the column, the LVDT was repositioning properly for subsequent testing. Compared to the more complex lower section of the Amsler press, where the load is applied from its hydraulic cylinder located below, the LVDT on the top plate is only needed to characterise the upper part of the press' compliance or stiffness. The eight remaining tests showed that the upper section of the press behaves in a repeatable manner, with a consistent initial load versus displacement behaviour and a constant stiffness of 1080±140kN/mm for the complete range over approximately 20kN. Following this characterisation of the upper section of the press, the missing data from the top LVDT that slipped in the first test was reasonably estimated from the subsequent test data.

The second error that hampered initial testing involved the data acquisition software and prematurely clipping the load signal in two of the first three trials. The author incorrectly entered the upper voltage limit for the input channel for load, with the error going unnoticed until after completing two tests. The load signals were clipped above 265kN, although the initial loading and cycling of the columns and their post-buckling and unloading behaviour were correctly acquired and recorded. The latter is important, as the head displacement for applying load moved at a constant rate with respect to time, with the load curve plotted against time being nearly linear leading up to buckling. The time at which the columns buckled was also clearly captured as the load immediately dropped below the clipping level of 265kN. Rather than omitting the clipped portion of the two tests, the short clipped segments were estimated with a simple linear interpolation in post-processing. This interpolation is shown in red for clarity in the subsequent load-displacement curves. The corresponding maximum loads from this interpolation process also matched those recorded in notes and written observations taken at the time of testing.

Buckling load results and strength comparisons

Figure 5.12 presents the force-displacement data for the nine columns tested in the Amsler press. Test data from each individual trial is also presented in the Appendix. Compared the EC5 design buckling loads of 368kN and 415kN for solid timber and glulam, respectively, all of the columns performed well and surpassed these estimates. The majority of the columns also surpassed the unfactored characteristic EC5 buckling estimates for solid timber and glulam columns of 532 and 576kN, respectively. For comparisons to the estimated characteristic or 5th-percentile buckling load from testing, Figure 5.13 further shows the maximum force from each test on a normal probability plot. The number of columns and corresponding data points is more limited here than in previous testing, but the data still follow a normal distribution reasonably well. The estimated 5th-percentile value from the test data corresponds to a compression load of 462kN, which is approximately 13 and 20% lower than characteristic EC5 estimates for solid timber and glulam, respectively. Figure 5.14 illustrates these comparisons with the maximum force test data plotted with the EC5 design and characteristic buckling curves. Again, error bars are used to illustrate the 5th- and 95th- percentile estimates from testing.

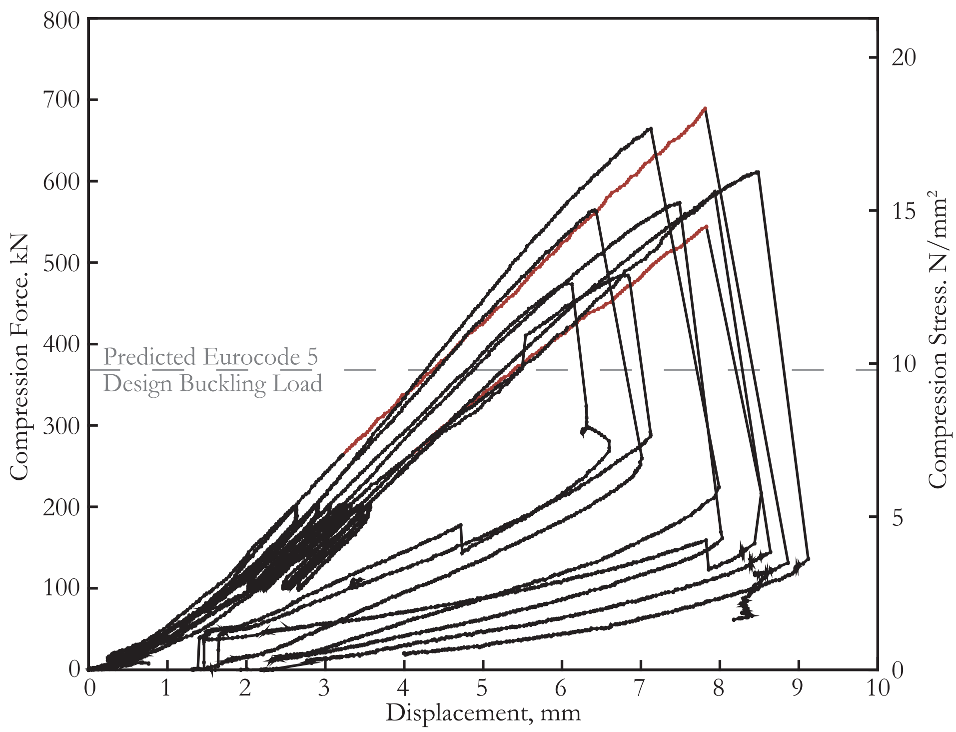


Figure 5.12: Force versus head displacement curves from additional full-scale testing with stress-laminated columns.

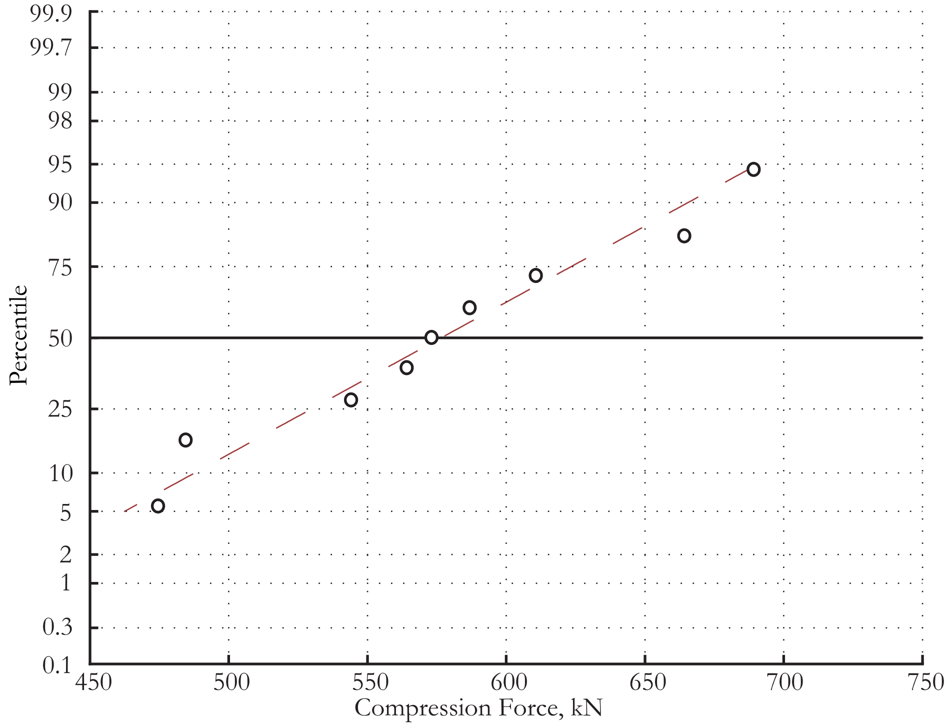


Figure 5.13: Normal probability plot of the maximum force data from updated full-scale testing.

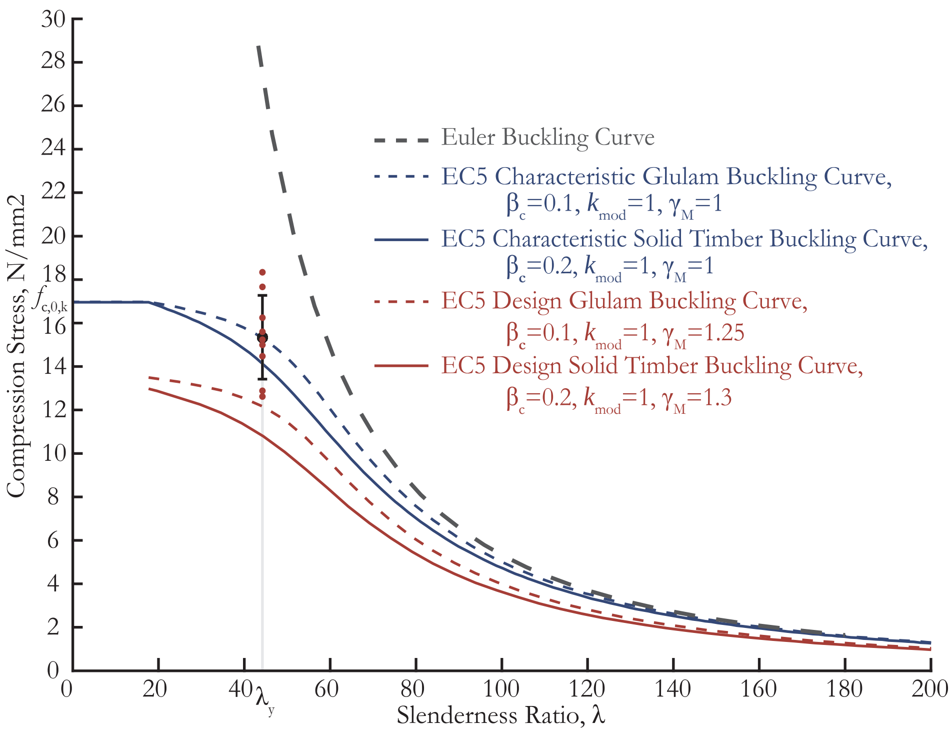


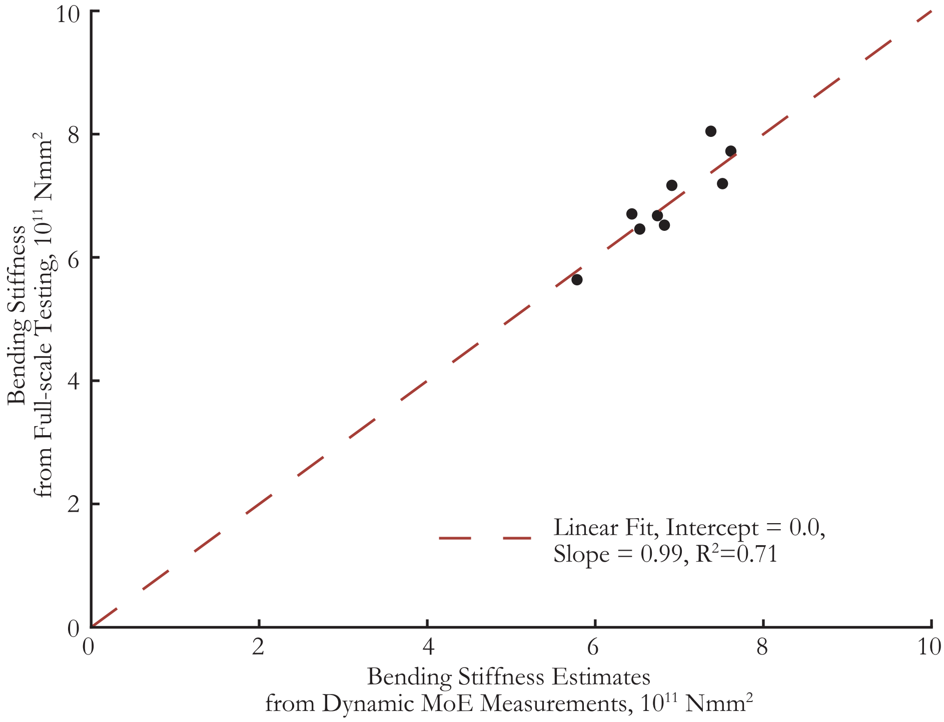
Figure 5.14: Summary of updated experimental test results with EC5 buckling curves.

The previous full-scale column tests showed about 26 and 33% lower performance when compared to characteristic EC5 estimates for solid timber and glulam. The improvement in the structural performance summarised in Figure 5.14 is due partly to higher construction quality, but also likely from the lower slenderness ratio. By moving from a slenderness ratio of 53 to the slightly lower value of 44, the columns are approaching the short or stocky region of the buckling curve; here the characteristic strength of the columns' timber becomes more influential next to stiffness and geometry effects. As hypothesised before full-scale testing began, this short column region is also relatively insensitive to inefficiencies in lamination, with the buckling curve eventually flattening as the slenderness ratio lowers below about 20. The previous full-scale test results from Chapter four, in the intermediate range of the buckling curve where the slope is the greatest, therefore represent a worst-case scenario with a 26% performance loss or penalty due to stress lamination when compared to solid timber. This performance loss in the intermediate region has been halved in the present tests by moving slightly towards the short region. Designers interested in specifying stress-laminated columns would therefore benefit by adjusting the dimensions of their columns to yield slightly lower slenderness ratios, if feasible to do so considering other architectural factors. At lower slender ratios, overall structural performance increases as the buckling curve rises and reaches higher values. With lower slenderness ratios, the buckling curve also gradually flattens and as the test results so far illustrate, the penalties from lamination inefficiency also noticeably decrease.

Stiffness comparisons with MoE estimates from dynamic measurements

Stiffness assessments and comparisons can also be made with the measured data from testing. Based on the LVDT displacement measurement data, the overall MoE of a column can be calculated and compared to the MoE estimates from dynamic measurements for the boards making up the column. Rather than comparing the measured overall MoE with mean MoE values of the individual boards, a better approach is to compare the measured and estimated bending stiffnesses. The bending stiffness of the tested columns is simply the product of the overall MoE acquired from measurement data and the column's second moment of area. By comparing bending stiffness, the MoE of each individual board and the effect of the board's position in the column are both taken into account. Figure 5.15 shows the actual bending stiffness values derived from test data, plotted against the estimated bending stiffness based on the layers in the columns. The latter estimates are calculated using equation (4.4), with the MoE values from dynamic measurements substituted for E_n, and assuming an equivalent efficiency factor, γ_n, of 0.8. By considering the effects of lamination efficiency through an equivalent efficiency factor, the estimated values for the bending stiffness show reasonable agreement with those derived from test measurements. Although the range of values in the comparison is quite limited, the agreement and the corresponding efficiency factor of 0.8 gives further indication of the superior performance of stress lamination compared to conventional fasteners and other mechanical-lamination techniques.

Figure 5.15: Bending stiffness comparison between experimental data and individual MoE estimates from dynamic measurements.



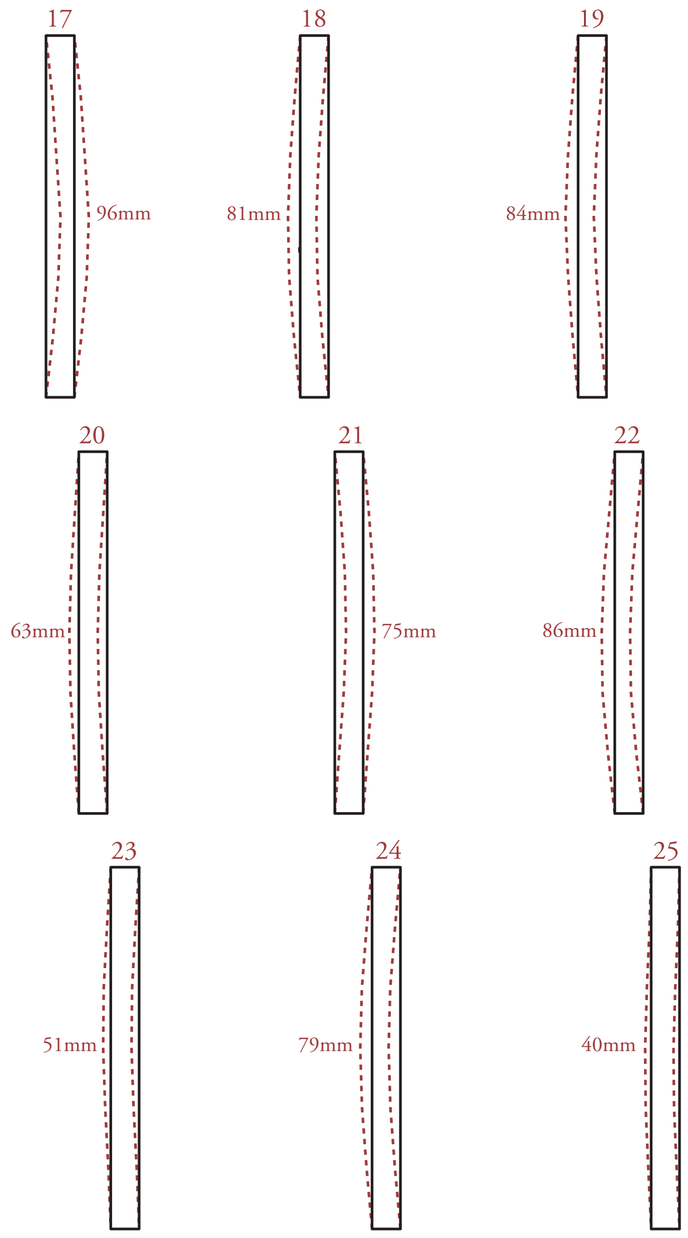
In EC5, an ideal efficiency factor of unity is usually associated with glue-based lamination. While an efficiency factor of 0.8 for stress lamination highlights an improvement over other types of conventional mechanical lamination, its value less than unity also emphasises the enhanced performance of glue over any mechanical alternatives. Gluing may offer brute lamination performance, but this benefit is not its true decisive advantage. The improvements in overall strength and stiffness of glulam due to spatial averaging and decreased variation far outweigh the benefits of efficient lamination on its own. For example, the characteristic MoE of softwood graded at the C24 strength class is 7.kN/mm^2, but when C24 wood is used to produce GL24C and GL24H grade glulam, the glulam's overall characteristic MoE rises to 9.1 and 9.6kN/mm^2, respectively.[24] Taking the overall column dimensions of 188x200x2400mm from the present testing as a reference, this increase in the characteristic MoE and overall bending stiffness from gluing corresponds to an efficiency factor greater than unity, equal to almost 1.3. This estimate illustrates the benefits of glulam in a new light, but also how glulam is represented like a new wood species or grade with enhanced properties in the Eurocodes, as opposed to a composite structure formed of individual layers. There may be similar advantages with spatial averaging and decreased variation found with mechanical-based lamination, but there is currently no way to account for such benefits in the present EC5 framework for built-up structures based on simple efficiency factors.

Despite displaying lower lamination performance compared to glue, there are still other secondary benefits of mechanical lamination worth emphasising. For example, the ductility of mechanical lamination is often overlooked compared to the brittle behaviour of glue, while glue-based lamination offers little resistance to twisting or tension perpendicular-to-grain. The latter is especially relevant for use with low-grade wood like UK-grown Sitka spruce. For example, if two adjacent layers in a glulam column or beam exhibit a high degree of twisting in the opposite direction from one another, they would create tension forces in the perpendicular-to-grain direction. Even with strong and reliable glue, the two twisting layers of glued wood would still have a tendency to split apart. Like in the case of built-up columns with spaced packs, the packs can be glued but still need to resist tension forces perpendicular-to-grain. In many cases, however, gluing is a highly effective technique for normal and high-grade wood, allowing forces to efficiently flow and transfer through different lamination layers, finding the paths of greatest stiffness through a beam or column. Stress lamination and other types of mechanical lamination with fasteners are still far more useful and compatible with low-grade wood due to their resilience, especially against twisting. Recent research efforts for increasing the compatibility of gluing with green wood may offer improvements with regards to moisture content and kiln drying requirements.[25] Fundamentally, glues still cannot address potential problems with twisting and other defects because of the orthotropic nature of wood and its weak tensile strength in the perpendicular-to-grain direction. Glue-based lamination works in series between different layers of wood, while mechanical fasteners and stress lamination can work in parallel across all the layers.

Buckling mode shapes and shear effects

The columns' final mode shapes after buckling and eccentricity or lateral deflection measurements during testing are presented in Figures 5.16 and 5.17, respectively. In general, before buckling occurs, the columns showed little lateral deformation, up to roughly 2-4mm. At the onset of buckling, however, the columns' lateral deflection increased by an order of magnitude. Some asymmetrical behaviour was also present both before and after buckling, as this behaviour was most likely due to varying stiffness and friction effects along the length of the columns. Apart from these general observations, three test cases showed more particular behaviour. Before buckling occurred, measurements from column 22 displayed an apparent higher-order mode shape in bending, although the differences in lateral deformations along the column were very minute, on the order of 1mm or less. Following buckling, column 22 along with all other tests displayed the typical first-order mode shape. In addition, column 17 and 22 also exhibited tension failures on the outer surface of their outer layer, somewhat distorting their final buckling mode shapes with some localised delamination. Figure 5.18 shows details of the tension failures in both cases, and the influence of knots in the failure. These tension failures arose from the significant bending after the onset of buckling, and did not likely play a role in the columns' instability. For example, column 21 withstood similar amounts of compression force before buckling and did not feature any tension failure after buckling. All three of these high-performing columns did buckle near or above the wood's characteristic compression strength parallel-to-grain, so minor plastic effects in the compression zone of the column may have been present. Such plastic effects could encourage larger bending deflections upon buckling. Advanced modelling with second-order plastic theory can be performed in the future to better understand this structural behaviour and these more complex failures.

Figure 5.16: Final buckling mode shapes from updated testing.



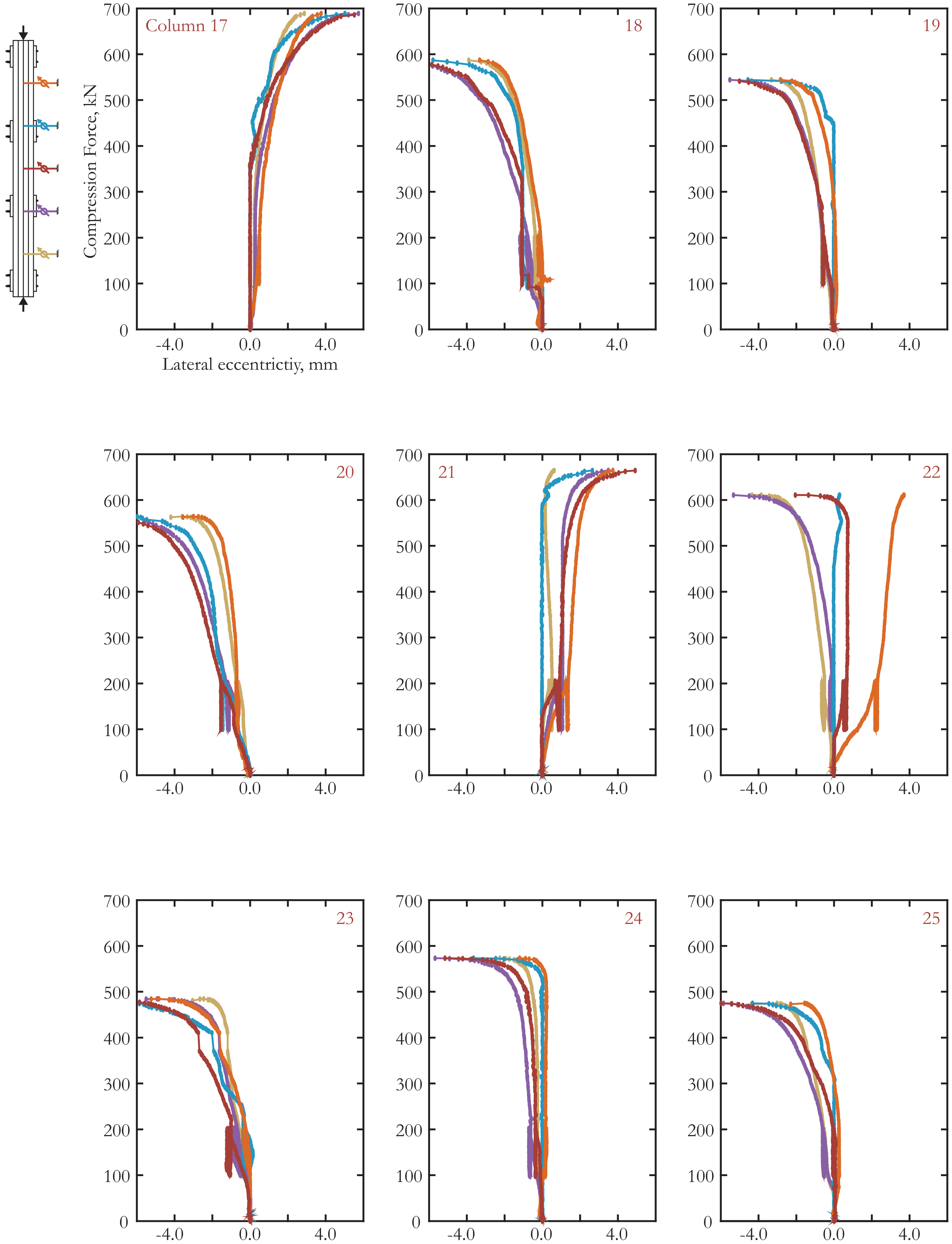

Column 17
18
19
20
21
22
23
24
25
Compression Force, kN
Lateral eccentrictiy, mm
0
100
200
300
400
500
600
700
-4.0
0.0
4.0

Figure 5.17 (opposite): Lateral deformations and column eccentricity measurements during full-scale column testing.

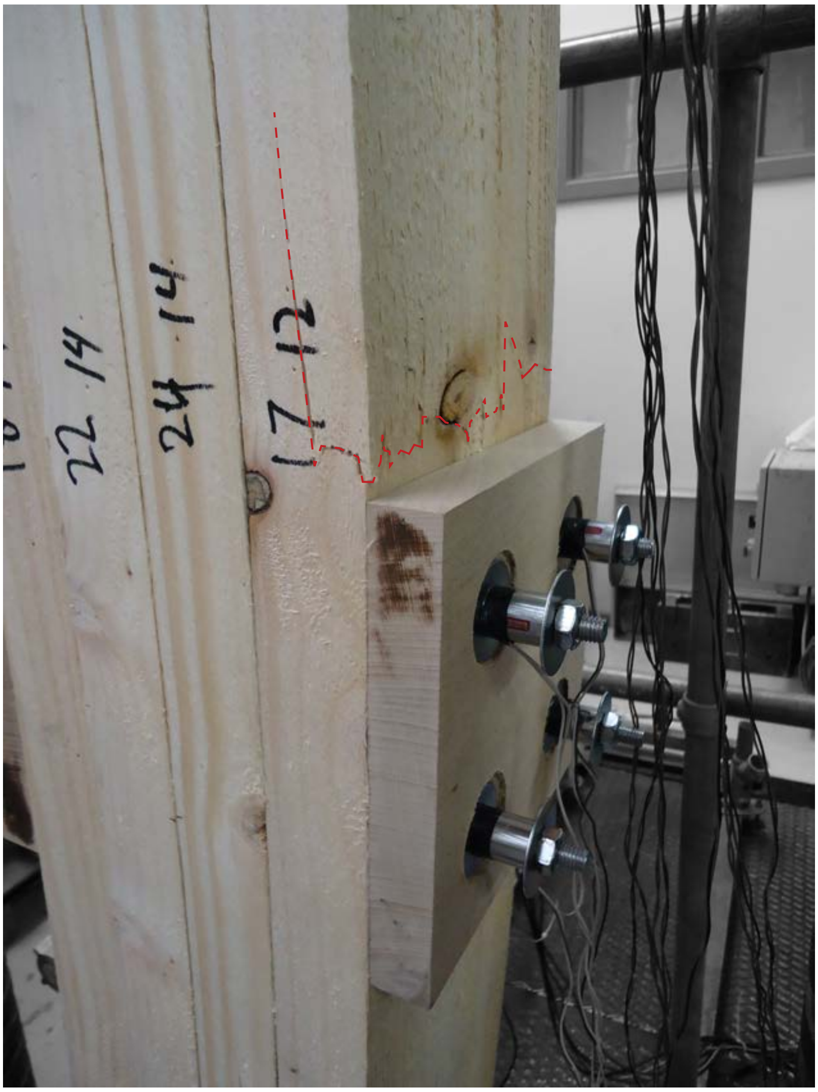
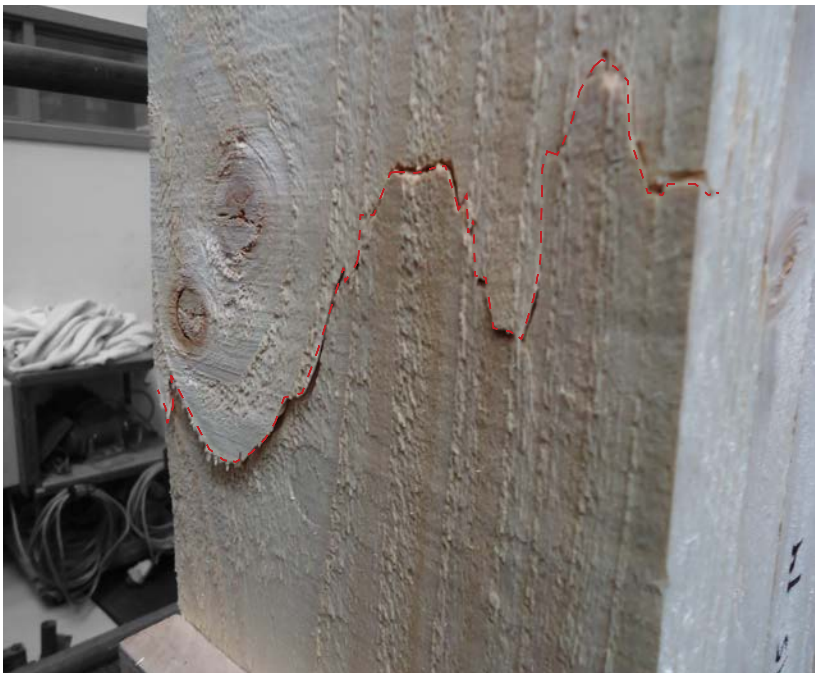


Figure 5.18: Tension failures in column 17 (top) and column 22 (bottom) following buckling.



In addition to the columns' buckling mode shapes, the lateral deformation measurements from testing can help to provide estimates of the shear between adjacent layers. EC5 estimates[26] for shear, V_d, in mechanically-laminated and built-up columns are given by a simple piece-wise function:

$$V_{\mathrm{d}} = \begin{cases} \dfrac{F_{\mathrm{c,d}}}{120\ k_{\mathrm{c,y}}} & \text{for } \lambda_{\mathrm{ef}} < 30 \\ \dfrac{F_{\mathrm{c,d}}\lambda_{\mathrm{ef}}}{3600\ k_{\mathrm{c,y}}} & \text{for } 30 \le \lambda_{\mathrm{ef}} < 60 \\ \dfrac{F_{\mathrm{c,d}}}{60\ k_{\mathrm{c,y}}} & \text{for } 60 \le \lambda_{\mathrm{ef}} \end{cases} \tag{5.1}$$

where $F_{c,d}$ is the design compression force. These EC5 shear estimates are based on Larsen's original 1974 study,[27] which used elastic theory throughout. Unlike the EC5 sections on solid and glulam columns that were corrected with second-order plastic theory through Monte Carlo simulations, the EC5 theory and design formulas for mechanically-laminated and built-up columns have never been updated. Larsen's complete derivation is omitted here, but the original shear function can be derived from the starting point that shear in beams and columns can be related to the bending moment, M_y, through differentiation in the x-direction along the length of the beam or height of the column, h:

$$V_{\mathrm{d}} = \frac{dM_{\mathrm{y}}}{dx}. \tag{5.2}$$

The bending moment increases as the product of the eccentricity of the column, e_y, and the applied force, F_c, so equation (5.2) becomes

$$V_{\mathrm{d}} = \frac{d\left[\mathrm{F}_c e_{\mathrm{y}}\right]}{dx}. \tag{5.3}$$

If the eccentricity of the column, e_y, is sinusoidal, with a final eccentricity or lateral deformation at mid-height of e_2 at the time of buckling, and the axial force does not vary along the height of the column, the shear expression can be simplified as

$$V_{\mathrm{d}} = \frac{d\left[\mathrm{F}_c e_2 \sin\left(\dfrac{\pi x}{h}\right)\right]}{dx}, \tag{5.4}$$

$$V_{\mathrm{d}} = \mathrm{F}_c e_2 \frac{\pi}{h} \cos\left(\frac{\pi x}{h}\right). \tag{5.5}$$

The maximum shear occurs at the ends of the columns, where $x=0$ or $x=h$, so the shear estimate can be again further simplified:

$$V_{\mathrm{d}} = \mathrm{F}_c e_2 \frac{\pi}{h}\ . \tag{5.6}$$

With the initial eccentricity measurements of the tested columns measured at 1mm or less, adding the typical eccentricity of about 3mm observed during testing gives a total eccentricity, e_2, of approximately 4mm at the onset of buckling. If the compression force is taken at the estimated characteristic value of 462kN from testing, with the column height of 2400mm, the shear is calculated at nearly 2430N. After taking into account the spacing of the prestressing elements and the thickness of the individual layers, estimates for the shear force on the end connections becomes 18kN. This shear force value is comparable to the shear force per shear plane needed to overcome prestressing, as observed in earlier shear tests in Figure 5.4. A photograph of the final shear deformation between layers from each test, like that shown in Figure 5.10(f), are also summarised in the Appendix, where the maximum shear deformation roughly varies from 0-3mm. These rough estimates from photographs are also comparable with a shear deformation of about 2mm seen earlier in Figure 5.4 for a load of 18kN per shear plane. Therefore, shear estimates from measured data, photographs, and Larsen's derivation suggests that shear effects can play a role in stress-laminated column buckling after and also near the onset of buckling, when eccentricity or lateral deformations begin to increase more rapidly.

The EC5 estimate for shear given by equation (5.1) yields a far larger value than that shown above based on Larsen's derived formula. If the effective slenderness ratio is 44 as in the case of a solid column, with the design compression force at 368kN and an instability factor, $k_{c,y}$, of 0.9, the EC5 shear estimate is about 5000N. Again, considering the spacing between prestressing bearing plates and the layer thickness, the EC5 estimated shear force on the end connections reaches around 32.2kN. Compared to the previous shear force of 18kN, this much higher EC5 estimate of 32.2kN is unexpected; Larsen's formula is supposedly the basis for the EC5 piecewise function in question. The significant discrepancy between the EC5 estimate and that from Larsen's formula is not immediately evident, but can be explained by considering the assumptions implied in the EC5 formula. In the previous calculation with Larsen's formula, the measured eccentricity before and during testing was used. Designers, however, do not have such test data available in most cases, so Larsen considered and approximated the so-called 'P-delta' effect with a moment magnification factor in lieu of actual measurement data. Like in steel column design, Larsen defined an initial eccentricity, e_1, and a second-order factor that exaggerates this eccentricity and the column's bending moment with increasing load. The product of e_1 and the magnification factor is used to estimate e_2:

$$V_{\mathrm{d}} = \mathrm{F}_c e_1 \left[\frac{\pi^2 E_{\mathrm{m}} / \lambda_{\mathrm{ef}}^2 f_{\mathrm{c,0,k}}}{\pi^2 E_{\mathrm{m}} / \lambda_{\mathrm{ef}}^2 f_{\mathrm{c,0,k}} - \sigma_{\mathrm{c}} / f_{\mathrm{c,0,k}}} \right] \frac{\pi}{h} \cos\left(\frac{\pi x}{h} \right), \qquad (5.7)$$

where the magnification factor is shown inside the square brackets, with E_{m} being the column's mean MoE, $f_{\mathrm{c,0,k}}$ is the 17N/mm^2 characteristic compression strength parallel-to-grain of the C16 strength class, and σ_{c} is the column's axial compression stress caused by the applied load at buckling. This magnification factor is a simplification attempting to capture more detail of the basic buckling mechanism: increasing axial force in turn increases the column's initial eccentricity; the increase in eccentricity leads to greater bending, which results in further deflection. Eventually the column reaches a point of eccentricity, e_2, where a stiffness-based instability is reached and buckling occurs.

The magnification factor can be quickly calculated for the case of the tested columns. Taking the column compression stress, σ_{c}, as the estimated characteristic force value of 462kN over the column's typical cross-sectional area of 188x200mm^2, the factor simplifies to

$$\left[\frac{\pi^2 E_{\mathrm{m}} / \lambda_{\mathrm{ef}}^2 f_{\mathrm{c,0,k}}}{\pi^2 E_{\mathrm{m}} / \lambda_{\mathrm{ef}}^2 f_{\mathrm{c,0,k}} - \sigma_{\mathrm{c}} / f_{\mathrm{c,0,k}}} \right] = \left[\frac{2.40}{2.40 - 0.72} \right] = 1.43. \qquad (5.8)$$

With an initial eccentricity of approximately 1mm measured from the first set of built columns, the final eccentricity at the onset of buckling is estimated to be also 1.43mm after applying the magnification factor. This estimate is conservative compared to measured data, where the columns approximately deflected an additional 3mm from their initial 1mm eccentricity, giving a final eccentricity of about 4mm before buckling. A misalignment of 1-2mm in the Amsler press could account for this difference. Alternatively, in the characteristic case, some part of the discrepancy between the EC5 shear estimate and that from the test data with Larsen's formula may arise from the magnification factor or the use of elastic theory in general. The discrepancy between the two estimates, however, most likely comes from the difference between the measured and assumed value of a column's initial eccentricity. If the conservative guidelines from EC5 for an initial eccentricity 1/300 times the height of the column are followed, e_1 becomes 8mm. With this large initial eccentricity of 8mm, Larsen's shear equation nearly converges with the piecewise EC5 function. These results are significant, showing that if stress-laminated columns are poorly constructed with relatively large initial curvature and eccentricity, shear effects can become influential and affect structural performance. As the full-scale columns constructed and tested were far straighter than EC5 guidelines, similar to most columns in practice,[28] performance remains comparable to solid timber columns with only marginal losses. Twisting over longer periods of time under service conditions still needs to be carefully considered, and could lead to slight increases in column eccentricity. This discussion also emphasises how buckling and overall structural performance can be influenced by even small differences in initial eccentricity, just like moisture content, compression strength, and creep effects. The adjustable nature of stress lamination provides leeway for carpenters to ensure that a column's initial eccentricity can be minimised during column assembly, or even corrected later, either off- or on-site during construction. Designers interested in specifying stress-laminated columns are recommended to design with and specify reasonable values for their columns' initial eccentricity. Designers might also encourage ample quality control checks following assembly and during construction to ensure that their specifications are not exceeded.

Performance comparison with self-tapping, axially-loaded screws

With the estimated slip moduli in Table 5.2, equations (4.4) and (4.5) can provide final performance estimates for mechanically-laminated columns with self-tapping, axially-loaded screws. Table 5.3 summarises spacing and efficiency factors for different configurations with self-tapping screws installed at a 45° angle with respect to the grain. Compared to regular timber screws, self-tapping screws are a marked improvement with significantly higher axial stiffness and overall lamination efficiency. Their disadvantage, however, is that they only exhibit high stiffness in one direction. The stiffness results of Tomasi *et al.*[29] captured this one-way behaviour clearly, with tests in shear with screws loaded in compression (S-C) performing similar to regular timber screws. Self-tapping screws display high-stiffness only when engaged with a tension component acting along their axis. Efficient lamination performance limited to one direction is not problematic with the common use of self-tapping screws in timber-concrete composite floors, as shear occurs predominatly in only one direction, associated with downward bending. Columns are slightly more complex and for a given primary buckling axis, can bend in two directions, either left or right. Self-tapping screws in stress-laminated columns therefore need to be installed in 'X' pairs or configurations, as shown in Figure 5.19, so they can provide stiffness in both shear directions for a given axis. For self-tapping screws, EC5 recommends a minimum inline spacing of about 10*d* or 80mm, yielding improved lamination performance compared to regular timber screws spaced at 20mm. A closer spacing of 40mm with the former gives an efficiency factor of 0.65, while the latter's efficiency factor for lamination is about 0.38. The efficiency factors of both of these options, however, are still lower than the estimated 0.8 efficiency factor achieved with stress lamination. Furthermore, the higher performance of self-tapping screws over regular timber screws also comes at a higher price. The cost of hardware used for laminating a column with self-tapping screws is about four to five times that of stress lamination. Compared to regular timber screws or the basic hardware needed for stress lamination, self-tapping screws can therefore be relatively expensive for use in mechanical-based lamination with low-grade wood.

Table 5.3: Lamination efficiency factor estimates for self-tapping, axially-loaded timber screws at various spacings.

Type	ULS Slip Modulus, $K_{u,n}$, N/mm	Spacing, a_n, mm	Efficiency Factor, γ_n	Effective Second Moment of Area, I_{ef}, mm^4	Effective Slenderness, Ratio, λ_{ef}
8mm self-tapping screws, *l*=240mm	7100	40	0.58	66.9×10^6	57
		80	0.41	49.1×10^6	66
		160	0.26	33.4×10^6	81
8mm self-tapping screws, *l*=240mm	9600 (for 'X' pair)	40	0.65	74.5×10^6	54
		80	0.48	57.0×10^6	62
		160	0.19	39.9×10^6	74

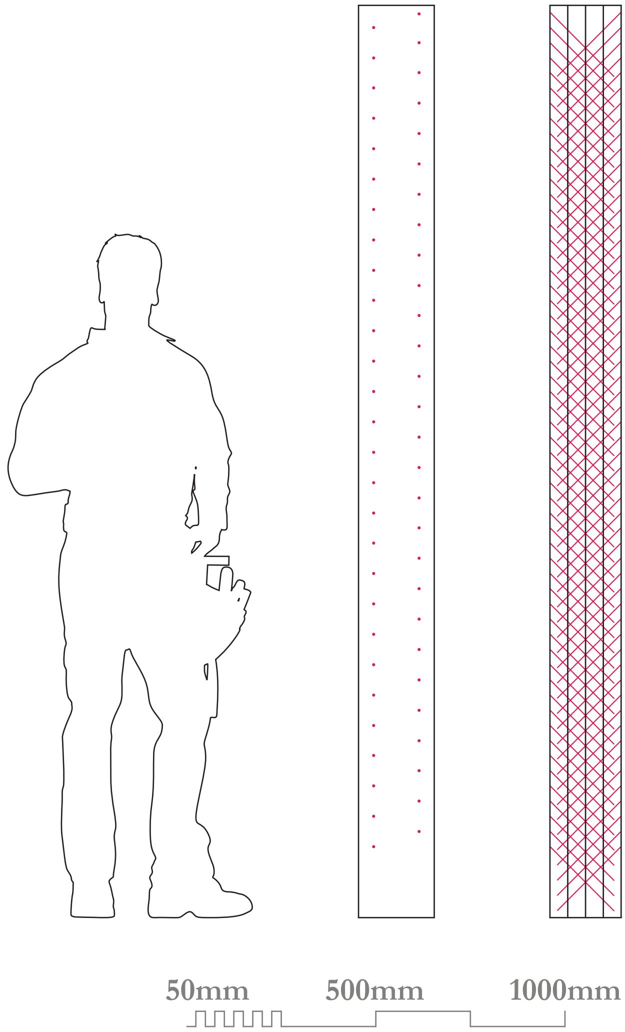


Figure 5.19: Mechanically-laminated column with angled 'X' pair arrangements of self-tapping screws at a spacing of 40mm.

Summary

Full-scale testing with updated instrumentation and improved detailing and construction methods show enhanced structural performance for stress lamination. Connection details tested in shear and full-scale columns in compression establish more confidence in the performance and application of stress lamination for building elements. Results from additional testing of stress-laminated columns at a slenderness ratio of 44 show that the characteristic buckling load is only 13% lower than estimates for solid timber. Stiffness measurements further show that stress lamination performs at an equivalent lamination efficiency factor of 0.8, outperforming other types of mechanical fasteners. Shear effects near or after the onset of buckling can also become significant compared to the shear strength from friction and prestressing. EC5 estimates suggest that shear effects become much more critical with poor construction that produces columns with large initial eccentricities. Self-tapping screws show noticeable improvements in lamination performance compared to regular timber screws and nails, but they cannot match the performance of stress lamination and are expected to be more costly. For completing a sound argument for the use of stress lamination in building elements with low-grade wood, the important issue of prestress losses and their effects on structural performance can now be addressed in detail.

Threaded rod tension and prestress losses

Throughout full-scale connection and column testing, the benefits and performance of stress lamination have been emphasised by comparisons to alternatives such as glulam, solid timber, and other types of mechanical fasteners. Prestress losses have also been acknowledged to play a crucial role in the performance of stress lamination. Up to this point, however, the issue of prestress losses has not been addressed directly. Prestress losses are certainly important, but timber engineers in general have tended to criticise and avoid stress lamination in buildings, citing its potential susceptibility to loss in prestress from creep and movement.[30] For example, even though stress lamination has been accepted by bridge engineers for timber bridge decks globally, the technique is completely omitted from the well-known European *Timber Construction Manual* by Herzog *et al.*[31] Furthermore, no allowance for friction effects is made in Part 1 of EC5 for buildings, except for the secondary 'rope' effect of bolted connections after yielding.[32] In contrast, friction effects and prestressing represent a fundamental structural principle in Part 2 of EC5 for the design of bridges.[33] Engineers working on buildings have developed a fundamentally different perception of the issue of prestress losses when compared to bridge engineers, hence the major discrepancy in the two parts of EC5. Under Johansen's influence, timber building engineers see prestressed connections as offering benefits, but any gains are also laced with uncertainty through potential prestress losses due to the moisture-dependent movement and creep behaviour of wood.[34] On the other hand, the necessity to use stress lamination for timber bridge decks has forced bridge engineers to actively engage with the challenge of controlling prestress losses rather than avoiding the issue entirely.

During the course of the past four decades, bridge engineers have developed a number of different ways of dealing with prestress losses. These methods are worth considering for use in stress-laminated timber elements with low-grade wood for buildings, alongside the author's own proposal of using overdried beech bearing plates for limiting or predicting prestress losses. Two basic examples have already been discussed in earlier chapters: overtightening prestressing bars, as is recommended in Part 2 of EC5; or using self-tapping screws for local reinforcement against compression failure in the perpendicular-to-grain direction, thereby allowing even greater overtightening in prestressing bars. The most straightforward way to avoid prestress losses and the resulting decline in structural performance, however, is to simply periodically retighten prestressing bars or elements. This manual approach may seem obvious, but also may not be feasible in buildings due to access and fire protection systems. To minimise retightening and to reduce maintenance efforts, timber bridge engineers have also proposed using Belleville washers or conical-disc springs, or leaf springs at the ends of prestressing bars.[35] Belleville washers are shaped like a shallow cone, acting as compact compression springs that can be easily stacked in parallel or series configurations. Belleville washers can reduce the effective stiffness of prestressing bars, increasing compliance and allowing some movement to occur in the wood without significant prestress losses. Some report, however, that when subject to outdoor service conditions, Belleville washers in the field display heavy corrosion and have ceased to function effectively.[36] Corrosion issues should not be problematic indoors, but the stacks of Belleville washers needed at the end of each threaded rod in a column, floor plate, or wall panel, would not be well suited to interior detailing and fire protection. The large number of Belleville washers required can also be costly. In response to the corrosion issue with Belleville washers on outdoor bridge decks, timber bridge engineers continued to develop more reliable means for controlling prestress losses.

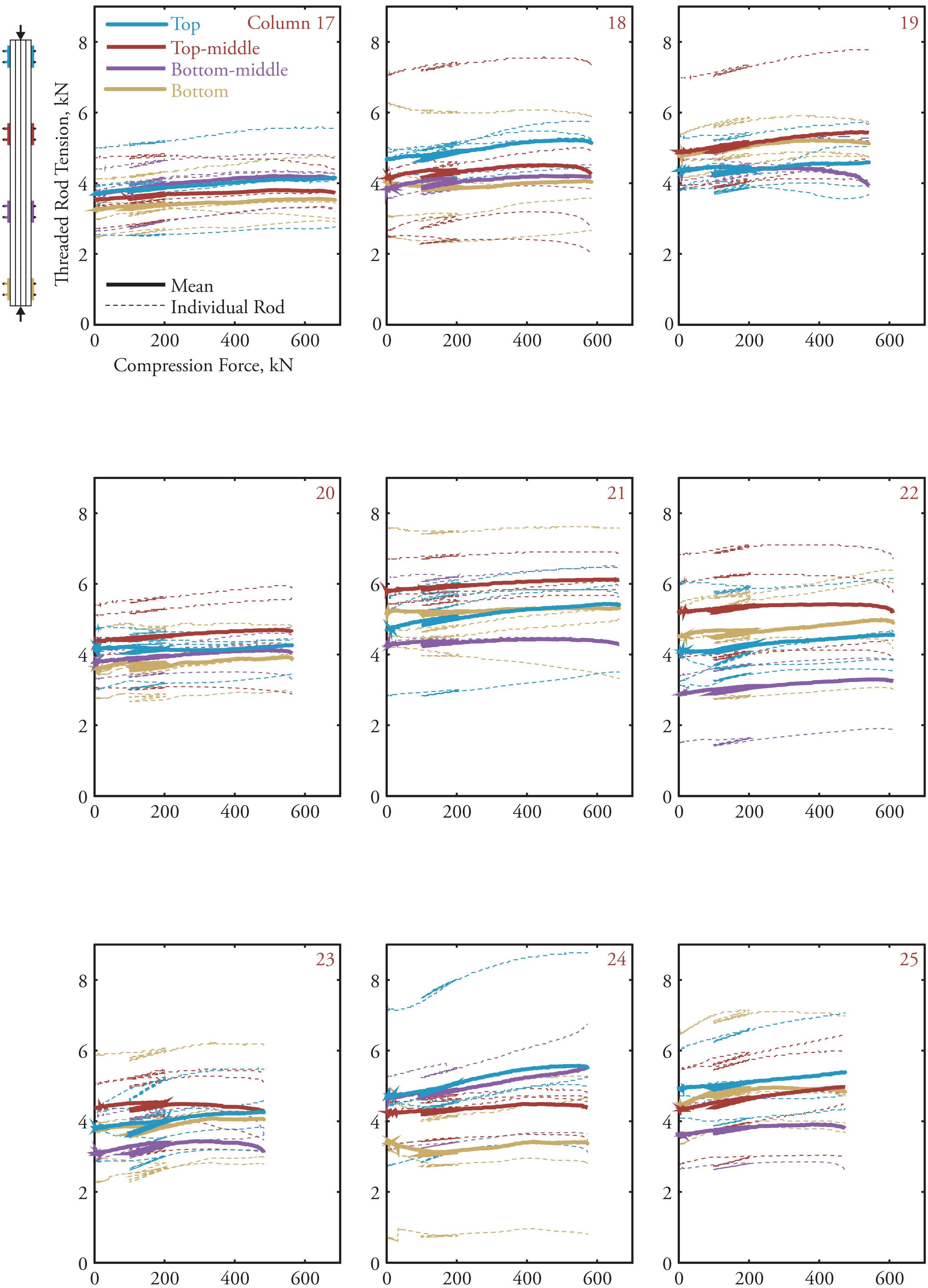
A more promising way of controlling prestress losses involves substituting more flexible materials for the standard components in stress lamination, instead of adding further complexity with Belleville washers or leaf springs. The author's proposed beech plates follows the former strategy, along with work by Dagher *et al.*,[37] who substituted common glass-reinforced plastic (GRP) tendons for steel prestressing bars. GRP is well suited for stress lamination because its MoE is roughly one-quarter of that of steel, while the strength of GRP is nearly double. GRP tendons therefore have excellent compliance and flexibility to compensate for any movement or dimensional changes in stress-laminated timber elements, especially those with low-grade wood with relatively elevated moisture content. Dagher *et al.*[38] found that long-term prestress levels could be stabilised at 80% of their initial levels, compared to steel configurations that tend to stabilise at about 30% of their initial levels. Their results show that using GRP is an effective method for controlling prestress losses in the long term. The effectiveness of GRP in stress lamination is encouraging, but the use of overdried beech plates may be able to mitigate prestress losses completely, leading to net increases in prestress over time. Long-term experiments will be performed to test overdried beech plates and their effects on prestressing.

Figure 5.20 (opposite): Individual and mean threaded rod tension measurements recorded throughout full-scale column testing.

Before conducting any preliminary or long-term experiments for examining prestress losses, it is worth noting the possibility of combining various techniques for controlling prestress losses together. Previous studies have usually only considered one method or strategy for controlling prestress losses, but they are all fully compatible with one another and can be used in tandem. For example, overdried beech plates can be used with Belleville washers and GRP bars if so desired. The bars can also be overtightened, retightened, or even used with self-tapping screws under the plates for maximising initial prestress levels. Considering different combinations of methods for controlling prestress losses is an interesting area recommended for future research. Such combinations might yield optimised, cost-effective, and innovative approaches for controlling prestress losses with stress lamination in building applications.

Threaded rod tension in full-scale column testing

Before discussing testing for prestress losses, it is also important to consider the threaded rod tension and its effects in full-scale column testing, as presented in Figure 5.20. Even though the threaded rods were tightened with the same calibrated manual torque wrench, considerable differences exist in the resultant tension of each rod. The mean tension of the group of rods in each bearing plate also varies between roughly 3 and 6kN, which is in many cases noticeably lower than the intended target of 6.1kN based on torque-tension estimates. Some minor prestress losses due to creep are expected after tightening the threaded rods in a column, but such short-term creep effects cannot account for the lower tension levels. The torque-tension relationship may be more sensitive to the surface conditions of threads, nuts, and contacting washers than initially estimated. Furthermore, while tensioning and tightening the rods, out of the sixteen transducers each attached to a threaded rod in a full-scale column, two transducers were initially damaged while tightening the threaded rods in-situ in the Amsler press. These damaged transducers were repaired and replaced for the subsequent testing, with more care taken during the tightening process for subsequent tests. After repairing these transducers, one other transducer also repeatedly experienced a short-circuit from its lead wires making contact with the adjacent washer surface during tightening. Better transducer design could prevent this problem from occurring in the future. The data from this problematic transducer and the first two damaged cases from initial testing are therefore omitted from mean calculations and Figure 5.20 for clarity.


Top
Top-middle
Bottom-middle
Bottom
Column 17
Mean
Individual Rod
Threaded Rod Tension, kN
Compression Force, kN
18
19
20
21
22
23
24
25
0
2
4
6
8
0
200
400
600

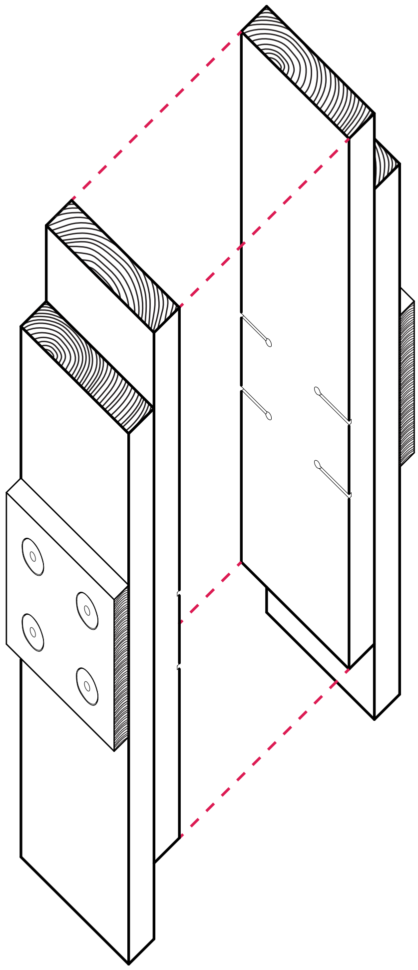
As can be seen in Figure 5.20, some threaded rods displayed a minor correlation between their tension and the applied compression force load from the Amsler press. Although the drilled holes for threaded rods were intentionally oversized by 10% or 1mm in diameter, some incidental bearing or dowel-action was most likely present during testing. In the most extreme case of a threaded rod in the top bearing plate detail in column 24, the change in tension is approximately 1.5kN. Considering the load-tension plots from previous shear tests in Figures 5.5 and 5.6, this magnitude of change in tension can easily occur before any embedding or major bearing occurs. The majority of the threaded rods showed relatively little change in tension throughout compression testing. Further, no significant changes in tension were observed between the threaded rods in the plates located at the ends of columns and those towards the middle. As shear forces vary between the two locations, these observations are consistent with the previous shear estimates; shear from bending due to the applied compression force and eccentricity do not overcome the prestressing forces until just near or after buckling occurs. Different bearing plate configurations with variable spacing along a column therefore might offer some flexibility to designers considering access and interior detailing issues.

Logging prestress losses

In parallel with shear and compression testing, longer-term tests for assessing prestress losses at full-scale were also prepared and executed. The longer-term tests were conducted in the constant-temperature and humidity room in the Cambridge University Structures Laboratory. Different prototypes or full-scale connection details with the same dimensions as those used in shear testing were tested for losses in prestress over a six-month period. Five models were prestressed under different conditions: either with overdried or regular beech plates, with their threaded rods tightened at either 12 or 35Nm torque levels with a manual torque wrench. The starting moisture content of the Sitka spruce in all models was 14-15% at the time of initial prestressing and logging. The softwood was expected to reach approximately 11 or 12% moisture content after stabilising with the surrounding air temperature of 20±0.5°C with 60±10% relative humidity. Before prestressing, three pairs of beech plates were also thoroughly overdried for several days at 100°C, and were expected to restabilise at 9-10% moisture content during testing. For logging the changes in tension, strain gauges were mounted at the mid-point along the length of each threaded rod, with a small length of threads turned down to create a smooth surface for the gauges. To bring the gauge's lead wires outside of the model, small channels were also routed in the middle softwood layers, as illustrated in Figure

5.21. The rods and channels were carefully prepared, but despite exercising caution when tightening the threaded rods and prestressing the models, five of the twenty gauges still broke away from their lead wires. Even a small imperfection in the threads of a rod would cause the rod to spin, rather than the nut spinning around the stationary rod. Just a small amount of rotation in a threaded rod during tightening would then wind the gauges' lead wires and break them. Learning from these observations and to improve this overall configuration, external transducers were made up accordingly and used much more effectively throughout the shear and compression testing already discussed.

Figure 5.21: Exploded view of the full-scale connection detail before assembly for prestress testing, showing internal channels routed in the middle layers for strain gauge lead wires.



Results from six months of logging the threaded rod tension in five separate prestressed connection details are presented in Figure 5.22. Note that the change in tension of the individual threaded rods is presented instead of the tension readings in absolute terms, as capturing the initial tension levels while prestressing proved difficult. The mean tension data for each connection detail is similarly presented in Figure 5.23. Differences between individual threaded rods in the same connection are seen in Figure 5.22, but the mean values plotted in Figure 5.23 give an interesting comparison between the overall trends exhibited from the different prestressing conditions. For example, the prestress losses in models without overdried beech plates can be clearly seen, especially in the higher case of torqueing the threaded rods to 35Nm rather than 12Nm. At the end of the six-month logging period, major prestress losses of approximately 1.5kN and 3.5kN are seen for these cases of 12 and 35Nm torque levels without overdried beech plates. If these threaded rods were initially tensioned to a range of roughly 4-6kN, as demonstrated in the previous column testing, the magnitude of these losses is significant and could lead to major reductions in structural performance through shear strength effects. With the majority of these losses occurring in the first two months following initial prestressing, construction guides for stress lamination recommend retensioning prestressed elements after one week and then again after six to eight weeks to alleviate these losses.[39] Existing studies by Taylor[40] also show that subsequent prestress losses following retensioning are also significantly diminished with much lower rates of exponential decay. Final retightening or retensioning after roughly eight weeks, followed by periodic checking with scheduled maintenance activities, can ensure structural reliability. An engineering-based solution to reduce checking and maintenance requirements is still more desirable.

Figure 5.22: Change in individual threaded rod tension and prestressing over a six-month test period.

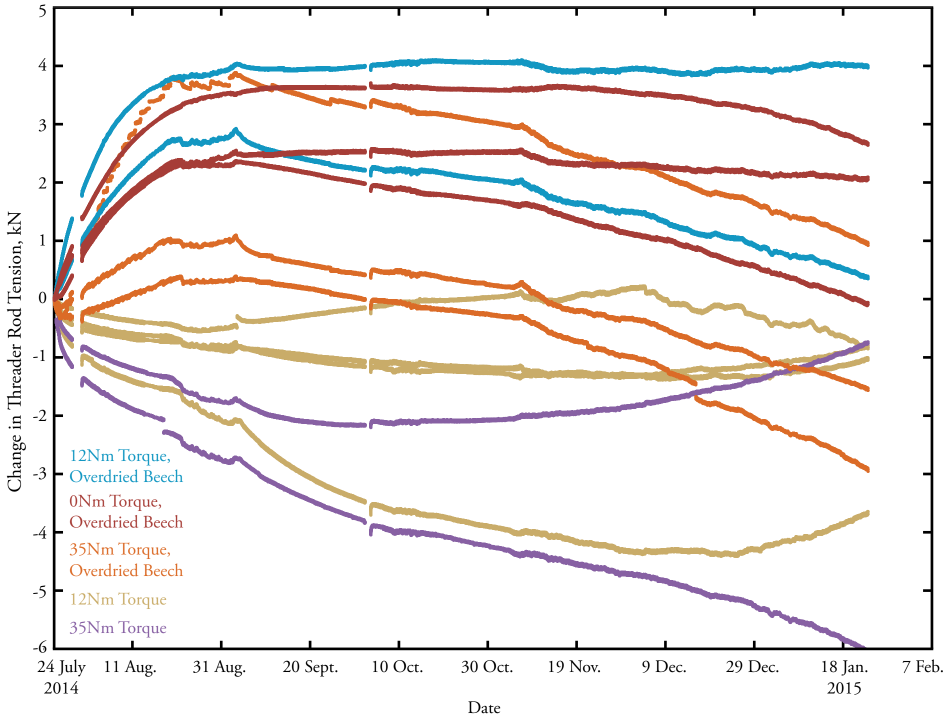
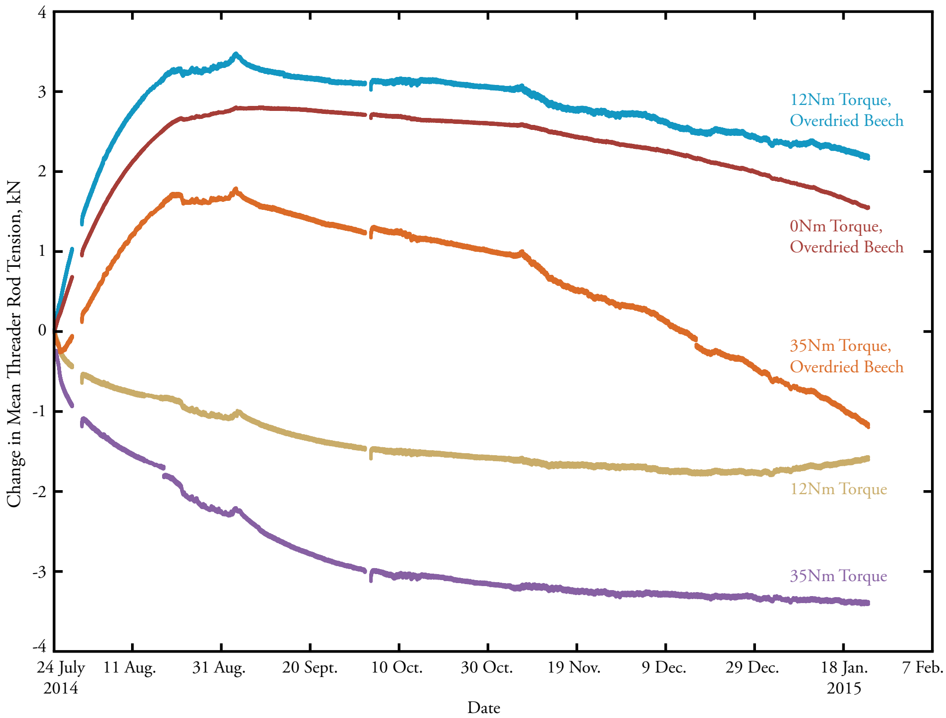


Figure 5.23: Change in mean threaded rod tension and prestressing over a six-month test period.



In contrast to these two basic stress-laminated connections with no overdrying, the details with overdried beech plates show a definitive tensioning effect. With little or only moderate amounts of prestressing of 12Nm or less, the overdried beech plates are the most effective and show immediate gains in tension. In the case of using overdried beech plates with a higher torque of 35Nm on the threaded rods, however, creep and movement of the spruce are immediately evident. Compared to creep effects, the beech plates are slower to gain moisture and to begin expanding, so the tension initially decreases in the first week following initial prestressing. Once expanding, however, the beech plates are able compensate for this initial loss and exceed the original threaded rod tension with a maximum mean gain of approximately 1.5kN. As the beech plates are thinner and smaller in size compared to the spruce, they also stabilise with the surrounding air and humidity much faster. After approximately one month, with the beech reaching equilibrium with the surrounding air and humidity, creep effects once again contribute to longer-term prestress losses. Despite these losses, in the cases with overdried beech plates and threaded rods subject to only moderate amounts or prestressing and initial torque, the overall resultant mean gain in tension after six months is about 1-3kN. This gain is similar in magnitude but opposite in sign to the net loss in mean tension of about 1.5-3.5kN seen in the models without overdried beech plates. The overdried beech plates are therefore an effective means to mitigate initial prestress losses.

In light of these promising preliminary results, further testing can be performed, especially with UK-grown Sitka spruce starting at 20% moisture content. During the course of testing, external collars fitted with strain gauges have been shown to be far more reliable than attaching the gauges directly on the threaded rods inside the detail. As a noticeable gradient is still present in the lowest-performing model with overdried beech plates and a torque of 35Nm, future testing over a longer period is also recommended. Finally, GRP rods in combination with the overdried plates would be of interest for future tests, along with the effects of retightening after an intermediate time of about eight weeks. Following the work of Awaludin *et al.*,[41] numerical modelling of the present results with straightforward spring and damper systems would also be of great benefit, allowing future predictions to be made for different configurations without extensive long-term tests.

Summary

Prestress loss is an important issue in stress lamination, but one that is often misperceived by building engineers. Compared to the technique's wide acceptance by bridge engineers, especially with low-grade wood, stress lamination has found little to no application in buildings. Several methods like overtightening and retightening prestressing elements, and the use of Belleville washers and GRP bars have been developed by bridge engineers, with varying degrees of effectiveness. While the latter have shown promise in limiting prestress losses, all of these methods can work together and be applied in parallel. Such combinations remain to be studied. Data from a six-month test period show that by only overtightening steel elements, prestress losses can be significant, especially with reference to the threaded rod tension measured in column testing. Tests with overdried beech plates, however, confirm that prestress losses can be counteracted and lead to increases in prestress, even when softwood is subject to creep and movement due to decreases from around 14 to 12% moisture content. Further modelling efforts and testing would be helpful for assessing alternative test conditions and configurations, especially with the more robust external transducers already developed for shear and compression testing.

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Chapter 6 CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Motivation and context

To conclude the thesis, reflection is merited on the accomplishments that have been made, starting with the original context and problems that motivated the work and to which the work responds. The expected increase in available softwood from UK forests in the upcoming decades is an important starting point worth re-emphasising. The majority of this expected increase is made up of relatively low-grade Sitka spruce, which when processed is incompatible with current gluing standards. Companies in the UK wood industry have also made little to no investment in the production of value-added massive engineered wood products like glulam, CLT, or LVL. Instead, these higher-value and higher-cost wood products are imported for use in large-scale construction, while UK-grown wood is processed for lower-value applications like fencing, pallets, and small-scale construction. New techniques and ways of working and building with low-grade wood should be developed. Current research has been focusing on *Brettstapel* and CLT as ways to add value to UK-grown wood for use in large-scale construction. The present thesis considered new types and variations of wood modification and stress lamination. These techniques have potential for adding value to low-grade wood and can find applications both in and outside of the UK in the future.

The emerging trend of building bigger, faster, and taller with massive engineered wood products is another important driver for developing alternative value-added processes for low-grade wood. While depending on imports of massive engineered wood products, the UK has already taken an early leading role in this new area of construction. Pioneering projects like the nine-storey Murray Grove Stadthaus residential building in London have inspired tall wood construction in many other countries. Although the Stadthaus and other first generation tall-wood buildings might lack compelling design, they are playing an important role in demonstrating the basic feasibility of building with wood at larger scales, especially with regard to fire, acoustics, construction, and cost. Massive engineered wood products can play a more powerful role in a design process and completed project, acting as more than replacements for steel and concrete precedents. Expanding the scope of large-scale construction with stress lamination while moving beyond basic design concepts and the short-sighted quest for height is now required; engaging with broader architectural issues will become more feasible and commonplace in the future of this field. The UK's rich tradition of working with wood may sometimes be overlooked next to accomplishments in masonry, steel, and concrete construction, but architects and engineers can engage with tradition, alternative ways of working with wood, and other broader concepts to produce more thoughtful and compelling designs.

Characteristics of wood

Understanding the characteristics of wood at different scales has provided a solid foundation for developing and investigating new ways of modifying and stress-laminating low-grade wood. Fundamental research is still necessary on the complex and layered structure of wood as a natural material, including the interaction of properties at different scales. Nonetheless, changes through processing and lamination can enhance mechanical properties and structural performance. In the specific case of wood modification and impregnation, improvements can also come about from working at the smallest scales to achieve interactions between impregnated polymers and the natural polymers in the wood's cell wall. In addition to the ability to modify the properties of wood, the ease of working the material by hand, its renewable nature, positive health benefits, and aesthetic and cultural value taken together provide a strong, general argument for its continued use in buildings. Some proponents tend to emphasise wood's ability to sequester carbon and mitigate climate change through the growth of trees, but equivalent embodied carbon dioxide emissions associated with structural materials in construction account for a small percentage of atmospheric emissions carbon-based estimates. Estimates for embodied energy and carbon of structural materials are also undermined by significant uncertainties. These uncertainties are commonly exploited in research, practice, and policy-level debates, and can be used to justify *a priori* biases for steel, concrete, or wood construction. Design rationales for the use of wood based on carbon sequestration and emissions are therefore highly questionable. These arguments have little to offer in addressing any doubts or public misperceptions about new types of large-scale wood construction.

Wood modification by impregnation

Wood modification by impregnation is an established research field for enhancing the properties of wood such as durability and stability. The treatment techniques, however, also offer potential to improve the mechanical properties of wood through interactions with the cell wall.

Value-added processes for impregnating wood with mixtures of monomers, followed by polymerisation through the application of heat or radiation have found only limited commercial development. Limitations to commercialisation are centred on general cost and potential toxicity issues. Methyl methacrylate has been the most popular monomer used in previous studies on impregnating wood, and was chosen for initial testing of small-scale samples of Norway and Sitka spruce in bending and compression. Compared to untreated or natural wood, initial results from testing polymer-impregnated small clear wood samples showed a large degree of scatter in the data for both mass and MoE. The effects and initial results from different impregnation conditions were therefore difficult to assess, including important aspects such as depth penetration and flow uniformity issues. Testing samples non-destructively before and after treatment provided a better account of improvements, with increases in MoE of about 20% compared to natural wood. Additional compression testing in the perpendicular-to-grain direction yielded more promising results, with gains in strength of roughly tenfold. The enhanced compression strength of the impregnated and polymerised wood samples could therefore match or surpass that of common mixtures of concrete. On the other hand, final testing of a new custom-synthesised monomer mixture with ionic liquids gave negative results, with consistent decreases in MoE. The overall improvement in test methods used to determine this outcome was still beneficial, and can be useful for accurately characterising wood in future research with new monomer mixtures and treatment solutions.

Full-scale stress-lamination construction and performance

Compared to wood modification and polymer impregnation, stress lamination is a relatively simple technique, requiring only basic hand and power tools. Stress lamination has been widely used in timber bridges for several decades, including construction with low-grade wood. The technique has found little to no application in buildings thus far, primarily due to concerns about losses in prestress forces over time from wood movement, arising from creep effects and changes in moisture content in service. Prestressing assemblies of wood on edge and using friction forces instead of glue for lamination is still open to further development. As a general concept, stress lamination can be applied to a wide variety of structural elements for both bridges and buildings. Scale models at 1:10 and 1:5 demonstrated the possibilities of making twisting, wave, split, and fan-like structural forms, alongside straight stress-laminated columns and panels. Instead of immediately scaling up these more complex structural forms for testing at full-scale, construction and testing focused on evaluating the structural performance of simpler straight stress-laminated columns.

Successfully adapting stress lamination for buildings requires designing new details, as stress-laminated bridge decks use bulky large-diameter steel bars as prestressing elements. A new prestressing detail was developed by distributing prestressing forces with several smaller bars or threaded rods instead of a single large bar. A prototype of a full-scale detail tested in shear showed promising results, with noticeable gains in stiffness and strength compared to a benchmark trial with no prestressing or stress lamination present. After characterising the moisture content and MoE of 128 boards of UK-grown Sitka spruce, eleven columns with an overall slenderness ratio of 53, measuring 235x250x3600mm, were constructed at full-scale and tested until buckling occurred. The performance of stress-laminated columns was comparable to EC5 design estimates, but approximately 26 and 33% lower than estimated characteristic values for solid timber and glulam, respectively. This performance still exceeded estimates for lamination with conventional mechanical fasteners like dowels, regular timber screws, and nails. After buckling, every column could also be easily disassembled for efficient reuse of the material. Reassembling five columns for retesting further demonstrated the repeatability of results within a 10% margin. Two special cases in the reassembled columns suggested that friction effects could be more influential in structural performance than the initial level of prestress forces. Further modelling work can be done to update design codes for full acceptance of stress-laminated columns, along with improving the design equations for mechanically-laminated and built-up columns.

Additional improvements in detailing at full-scale were made for optimising and demonstrating the full potential of low-grade wood with stress lamination in buildings. Bearing plates made from UK-grown hardwood with fewer threaded rods improved accuracy and reduced overall construction time. Using the plates in full-scale testing with enhanced instrumentation yielded improved results. Nine columns measuring 188x200x2400mm showed improvements in buckling and structural performance. These more stocky stress-laminated columns, with a slenderness ratio of 44, exceeded EC5 design estimates and were only 13 and 20% lower than unfactored characteristic estimates for solid timber and glulam, respectively. Stiffness results further revealed that these columns behaved with a relatively high equivalent lamination efficiency factor of 0.8. Although stress-lamination cannot achieve an ideal factor of 1 associated with gluing, the technique is far more compatible for use with low-grade wood due to its adjustable and mechanical nature. Stress lamination also delivered higher performance than estimates for other mechanical-based lamination techniques, including relatively new kinds of self-tapping, axially-loaded screws. Compared to relatively short-term testing of stress-laminated columns, structural integrity and performance in the long-term are ensured by properly accounting for and controlling losses in prestress forces. A review of literature for controlling prestress losses in stress-laminated timber bridge decks offers several promising methods. Test results from a period of six-months also showed that overdrying hardwood bearing plates can minimise and in some cases even mitigate prestress losses due to moisture-related movement and creep. All together the suite of full-scale tests for shear, compression, and prestress forces established that it is possible to meet the challenges of building with low-grade UK-grown wood using a simple technique like stress lamination.

Recommendations

During the course of this thesis, several areas were identified for future work that can build upon the present achievements. Recommendations for future research and actions are summarised here.

UK wood industry

The lack of diversity and investment in secondary processing for massive engineered wood products in the UK wood industry has been identified as a major area requiring further attention. The growing demands for massive engineered wood products in large-scale and multi-storey wood construction and the expected increase in availability of UK-grown wood provide ample opportunity to encourage investment and further research activities. Current interest in establishing CLT and *Brettstapel* production with UK-grown softwood is a positive step forward, but much more work is needed on this front. Further work with stress-laminated elements, including floor plates and wall panels, can highlight how low-grade UK wood can meet the demands of multi-storey timber construction, but without large capital investments or industrial-scale production. Additional construction and testing of different types of stress-laminated building elements in the context of a built project would further emphasise the strength of the technique for different structural elements. Hybrid configurations with self-tapping screws may prove to be a useful stepping-stone for building engineers who are unfamiliar with stress lamination but interested in its potential benefits.

For establishing more diversity in the UK timber industry in the longer term, existing small-scale UK glulam producers working exclusively with imported wood can play an important role. Such producers have already demonstrated that small-scale glulam production with imported softwood is a successful and profitable business model in the UK. Working with more locally-grown material, including hardwoods, alongside imported softwood could also find similar success, with potential for scaling up production while leading to additional economic and social benefits in the UK. Involving practitioners and research in industrial pilot studies for glulam production with UK-grown wood is therefore recommended. In reference to the successful development of CLT in Europe, combining interested parties from the three areas of practice, research, and industry is also strongly encouraged. Together these three interests can maximise impact and increase the likelihood of future investment and production. For example, in the case LVL production, a study performed by Timber Research and Development Association (TRADA) on behalf of the UK Forestry Commission has already demonstrated the feasibility of producing LVL from UK-grown Sitka spruce. Without input from practice to demonstrate successful application or encourage demand in the market, however, feasibility studies remain obscure and less likely to initiate investment and positive outcomes.

Wood modification by impregnation

With this thesis work on wood modification and impregnation, limitations in laboratory equipment and apparatus prohibited some testing at full-scale structural dimensions. For taking the work forward, toxic monomers and treatment solutions should be strictly avoided for potential use in buildings. Before moving up in scale, further experimental work with additional input from plant and material scientists is needed to bridge the gap between the molecular and structural scales. Based on the author's experience, it is important to first address the issues of characterising interactions at the cell wall, then to address scale and flow issues later. For example, once a new monomer mixture is thoroughly tested and proven successful with small samples, the challenges of scaling up successful treatment mixtures can be tackled with help from researchers specialising in fluid mechanics. From a mechanical and structural engineering perspective, trying to resolve the influence of both cell wall interactions and inconsistent impregnation proved difficult, especially without the support of plant and material scientists and a fluid mechanics researcher. With the recent development of the Centre for Natural Material Innovation at the University of Cambridge bringing these individuals together, future work should be able to significantly expand upon the preliminary wood modification results presented in the thesis. Rather than strictly focusing on sawn timber, investigating alternative treatment methods with small strands and veneers may also may prove fruitful.

When this thesis project first began, the author and other collaborators struggled with the tension of improving performance by either adjusting mechanical properties, or by designing more efficient structures with different cross-sections or details. Finding a design situation where only one solution exists is unlikely. For example, limitations in perpendicular-to-grain compression strength in platform construction can be addressed by several approaches, with some more difficult to implement than others: strength can be directly increased with impregnated timber, self-tapping screws can be installed as reinforcements, joints can be staggered in the panels, or pockets of dimensionally-stable grout can be used to transfer vertical forces around the ends of the floor plates. Rather than envisioning impregnated wood as a stand-alone design solution in itself, more work can be done on how to better integrate the modified wood with other design options. For example, due to densification effects, potential improvements from simple withdrawal-strength tests with modified wood and self-tapping screws might far outweigh any direct gains in material properties. Providing new wood impregnation processes can be scaled up to structural dimensions, a broad set of quick, initial tests focused on design issues and detailing is recommended in addition to focusing on relevant material properties like MoE.

Stress lamination

In the future, general structural testing with different types of stress-laminated building elements can be performed in the context of a design project, and more specific issues can also be considered with future research. The present research focused on buckling and the performance of stress-laminated columns, offering a platform for investigating several different avenues in the future. General connections and detailing between stress-laminated columns and other structural elements like beams and floor plates are important. Future research efforts can therefore extend the current work by examining general connection issues between different stress-laminated elements, alongside connections with other types of massive engineered wood products like glulam, LVL, and CLT. For example, in the case of joining columns in multi-storey construction, staggering the layers in stress-laminated columns might be beneficial for construction. Furthermore, standard connections with either dowels or self-tapping, axially-loaded screws are feasible with stress-laminated elements, but the bearing plates needed for stress lamination might also serve an additional bearing role for secondary beams. Testing connection details developed with structural engineers from practice is recommended for giving a collaborative element and direction to future research.

Through a detailed discussion of test results, the theory and modelling behind design equations in EC5 for solid timber and glulam columns have incorporated second-order analysis and plastic theory. On the other hand, the EC5 design equations and theory for mechanically-laminated and built-up columns have not received the same level of attention as solid columns. Accordingly, the former have not been updated since the original EC5 draft. Further theory and modelling work is recommended for these more complex column configurations. Making the design procedure and equations simple to use and apply should also be emphasised. The simplified approach taken for solid columns with a non-linear instability factor representing second-order and plastic effects can serve as an important starting point. In the cases of more complex load combinations with applied axial forces and bending moments, full-scale testing would most likely be necessary to validate proposals for new models. Additional full-scale testing with more complex load cases is therefore recommended alongside more advanced modelling efforts, as gains in strength from plastic effects unaccounted for in linear theory are expected with these more complex load cases. While buckling can be influenced by compression strength, moisture and creep effects, and initial eccentricity, more care in future full-scale testing on characterising the initial eccentricity of columns and applied loads is recommended.

Modelling and additional testing for controlling prestress loss with external transducers is also recommended for future investigations. The external transducers developed for shear and compression testing avoided the major problem of breaking internal strain gauge lead wires during prestressing. Starting with UK-grown Sitka spruce at about 20% moisture content, combinations of GRP rods, overdried beech bearing plates, and retightening actions could be examined over a longer test period of one year. Plans for such tests are already underway. After completing future prestress loss testing, the full-scale connection models could also be used in subsequent shear testing.

Full-scale compression strength testing parallel-to-grain with UK-grown Sitka spruce according to BS EN 408:2010 is also recommended for removing uncertainty from existing results. Such testing is commonly done but has not yet been performed for UK-grown Sitka spruce. Full-scale compression testing parallel-to-grain would be useful for assessing the accuracy of the standard C16 characteristic compression strength parallel-to-grain of 17N/mm^2. On the other hand, the BS EN 308:2009 strength class standard gave an estimated value of 18.6N/mm^2 for the characteristic compression strength parallel-to-grain of UK-grown Sitka spruce. The latter estimate is based on the bending strength and density for UK-grown Sitka spruce, and therefore may be more accurate. Full-scale testing can resolve this discrepancy and would be fairly straightforward to perform. The compression test results would most likely be of general interest to UK researchers in several different fields.

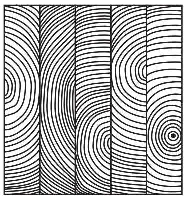
Finally, after gaining several years of experience in timber engineering through the course of this doctoral thesis, significant potential exists within this broadening field, either through stress lamination or other techniques and products like glulam, LVL, and CLT. The present work has focused on the UK context to demonstrate how low-grade wood can broaden and also be useful for large-scale construction. The thesis is also generally applicable in many other contexts where low-grade wood is available and undervalued. Practitioners working in such contexts further afield are recommended and encouraged to consider stress lamination alongside other feasible techniques for working with low-grade wood.

Appendix
FULL-SCALE TEST PHOTOGRAPHS AND DATA

Test 1:

Mean Moisture Content Percentage at Time of Testing

14 14 14 11 14

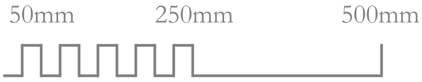


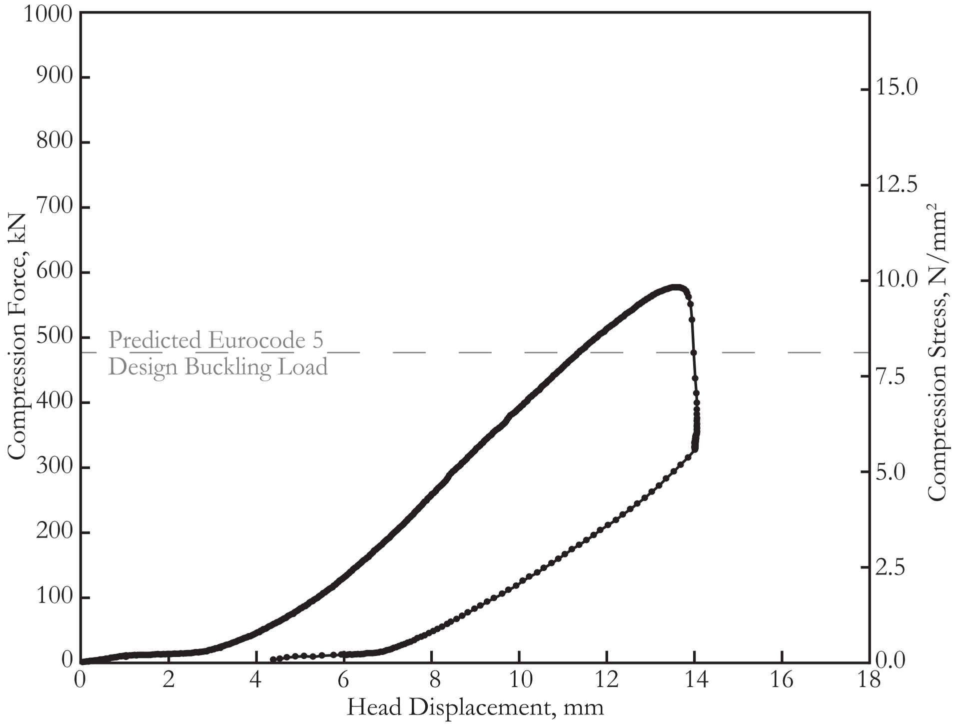
Test 1

578kN Max. Force

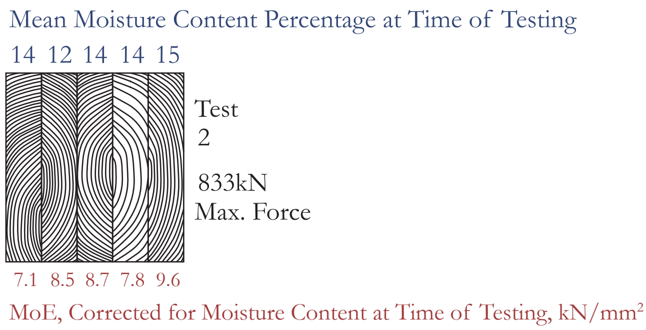
6.1 6.8 6.7 7.8 7.6

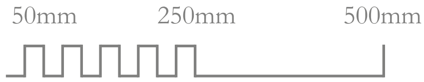
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

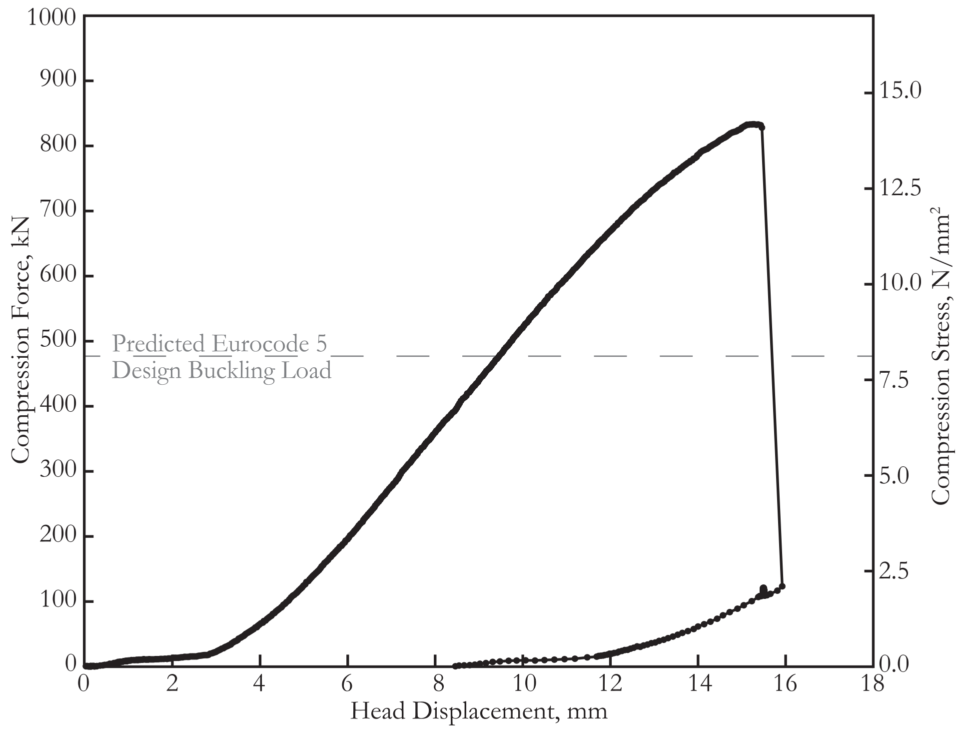


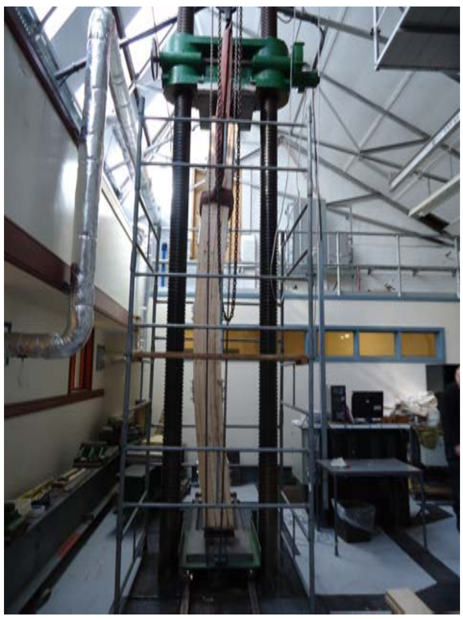


Test 2:


Mean Moisture Content Percentage at Time of Testing
14 12 14 14 15
Test
2
833kN
Max. Force
7.1 8.5 8.7 7.8 9.6
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²


50mm
250mm
500mm

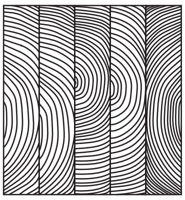

1000
900
800
700
600
500
400
300
200
100
0
Compression Force, kN
15.0
12.5
10.0
7.5
5.0
2.5
0.0
Compression Stress, N/mm²
0
2
4
6
8
10
12
14
16
18
Head Displacement, mm
Predicted Eurocode 5
Design Buckling Load



Test 3:

Mean Moisture Content Percentage at Time of Testing

13 14 11 13 13



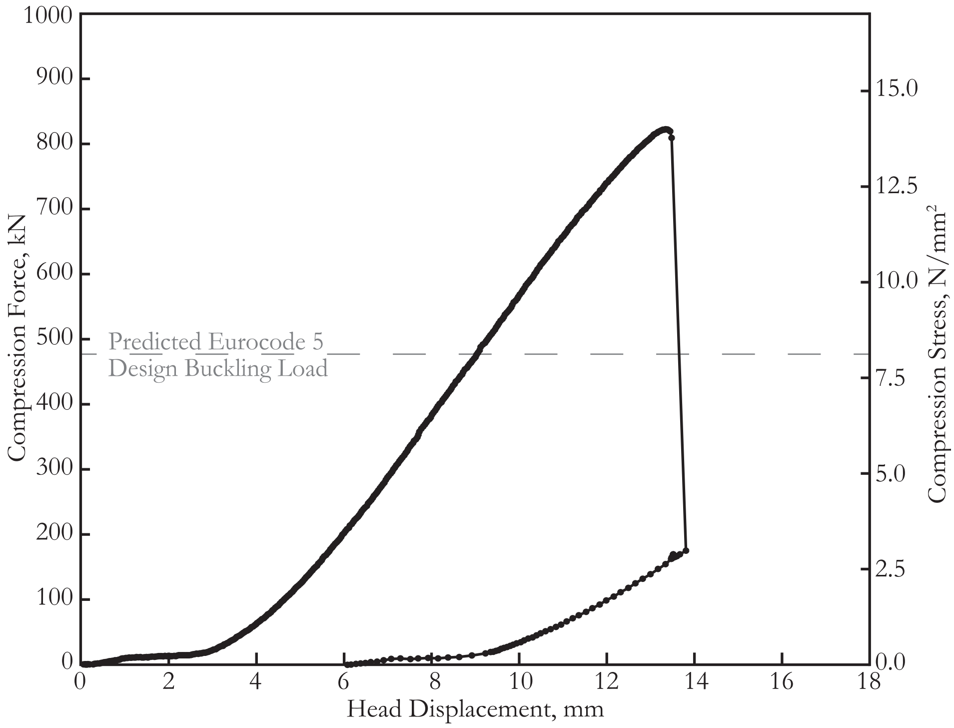
Test
3

823kN
Max. Force

8.7 8.9 7.7 8.6 9.5

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

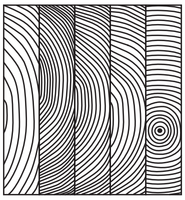
50mm 250mm 500mm



Test 4:

Mean Moisture Content Percentage at Time of Testing

16 14 14 15 14



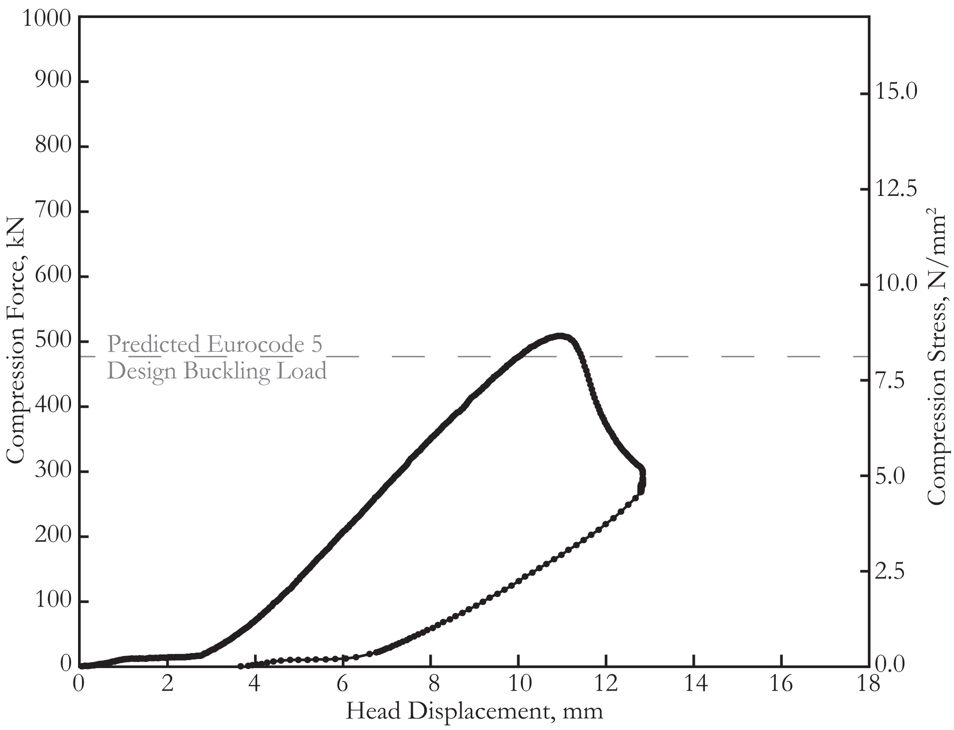
Test
4

509kN
Max. Force

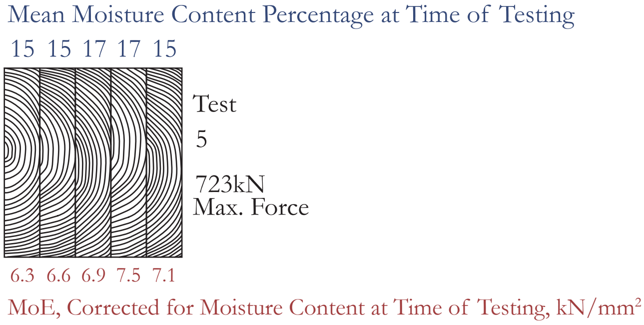
7.7 8.5 8.0 7.7 6.4

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

50mm 250mm 500mm

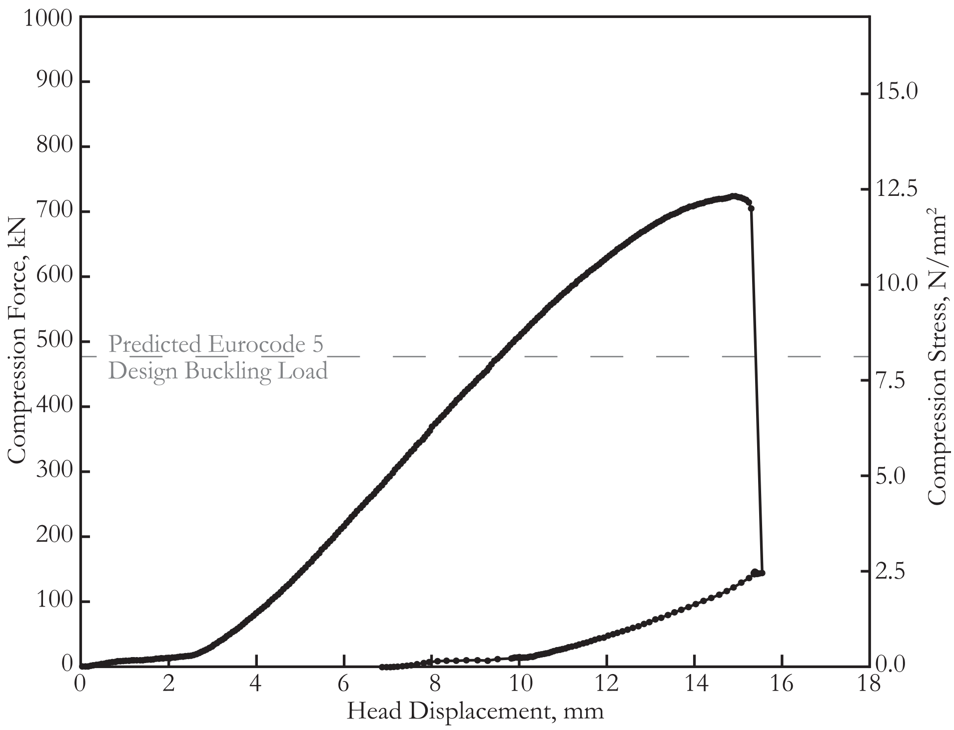


Test 5:



Mean Moisture Content Percentage at Time of Testing
15 15 17 17 15
Test
5
723kN
Max. Force
6.3 6.6 6.9 7.5 7.1
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

50mm
250mm
500mm

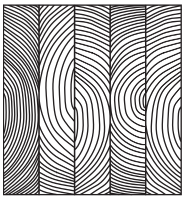


Compression Force, kN
1000
900
800
700
600
500
400
300
200
100
0
Predicted Eurocode 5
Design Buckling Load
0
2
4
6
8
10
12
14
16
18
Head Displacement, mm
15.0
12.5
10.0
7.5
5.0
2.5
0.0
Compression Stress, N/mm²

Test 6:

Mean Moisture Content Percentage at Time of Testing

15 16 15 15 13

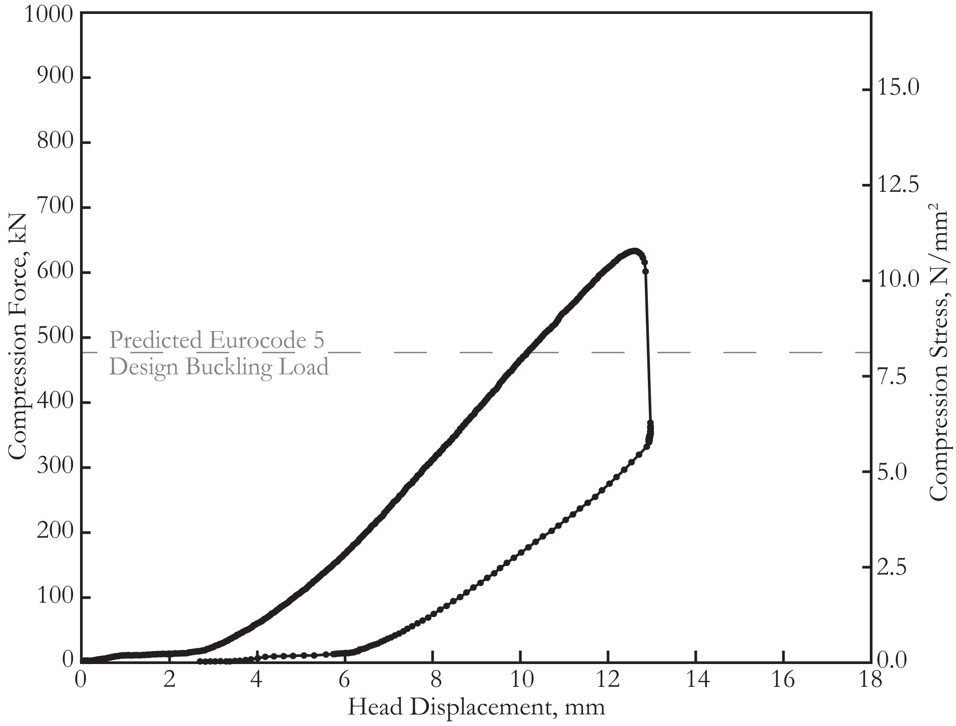


Test
6
633kN
Max. Force

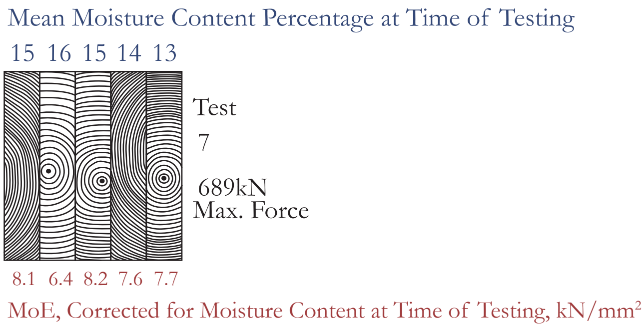
8.6 6.0 7.0 7.1 7.0

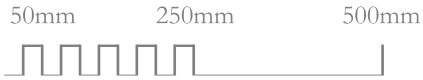
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

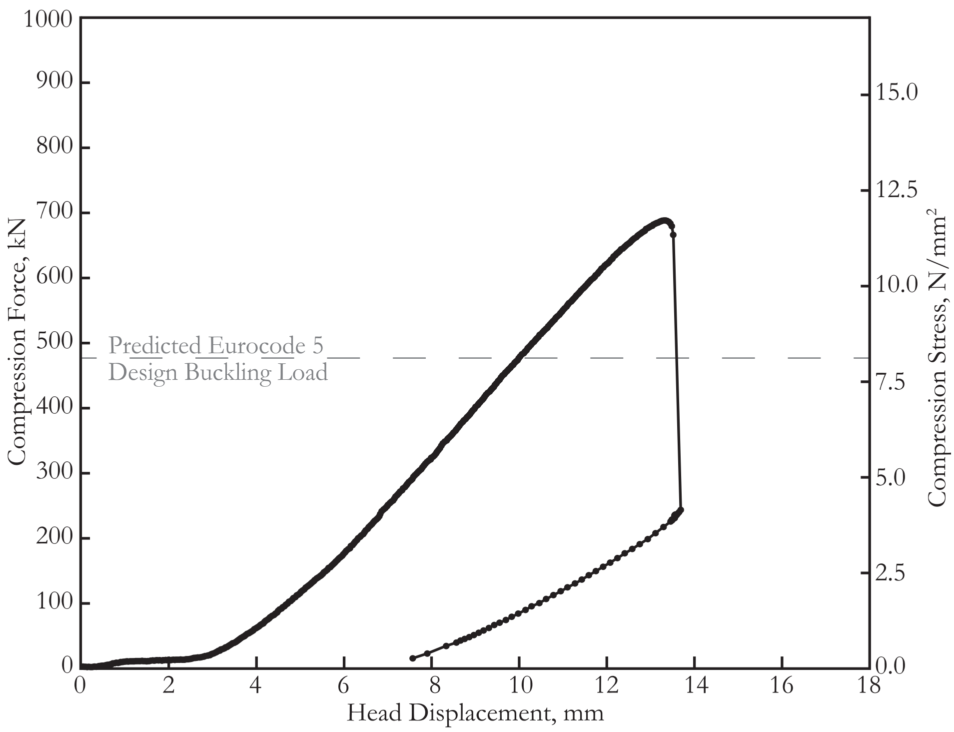
50mm 250mm 500mm



Test 7:


Mean Moisture Content Percentage at Time of Testing
15 16 15 14 13
Test
7
689kN
Max. Force
8.1 6.4 8.2 7.6 7.7
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

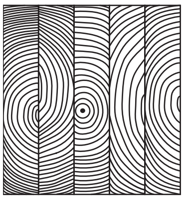

50mm
250mm
500mm


Compression Force, kN
1000
900
800
700
600
500
400
300
200
100
0
Predicted Eurocode 5
Design Buckling Load
Compression Stress, N/mm²
15.0
12.5
10.0
7.5
5.0
2.5
0.0
0 2 4 6 8 10 12 14 16 18
Head Displacement, mm

Test 8:

Mean Moisture Content Percentage at Time of Testing

14 14 18 14 17

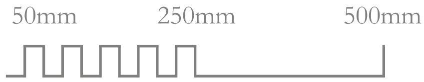


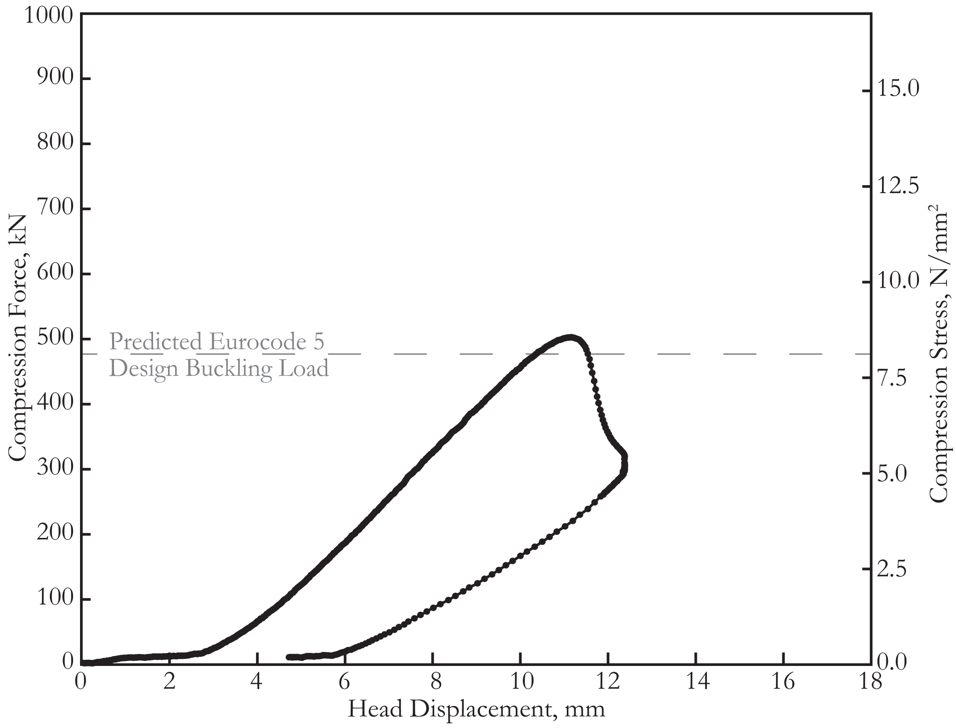
Test
8

503kN
Max. Force

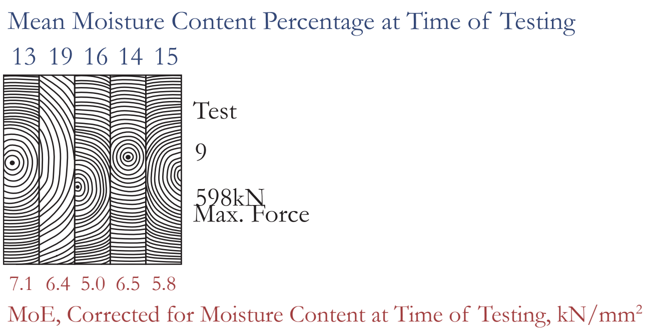
6.4 9.4 7.4 6.7 6.3

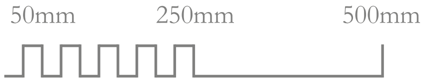
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

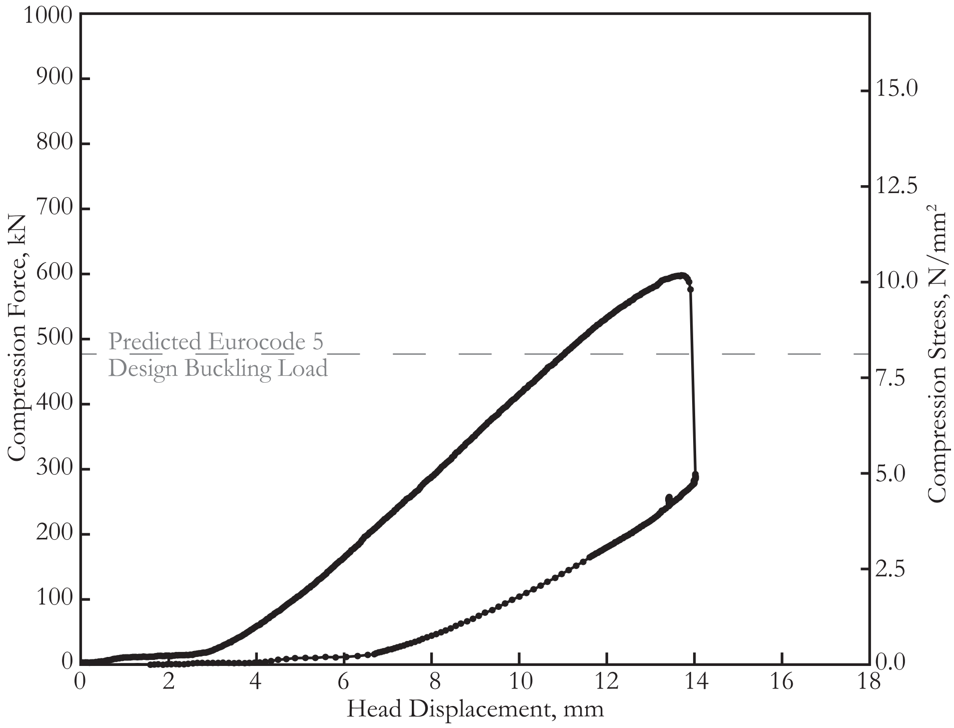




Test 9:


Mean Moisture Content Percentage at Time of Testing
13 19 16 14 15
Test
9
598kN
Max. Force
7.1 6.4 5.0 6.5 5.8
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

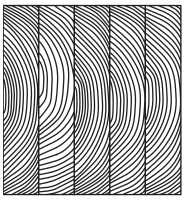

50mm
250mm
500mm


1000
900
800
700
600
500
400
300
200
100
0
Compression Force, kN
Predicted Eurocode 5
Design Buckling Load
0
2
4
6
8
10
12
14
16
18
Head Displacement, mm
15.0
12.5
10.0
7.5
5.0
2.5
0.0
Compression Stress, N/mm²

Test 10:

Mean Moisture Content Percentage at Time of Testing

13 16 19 15 17



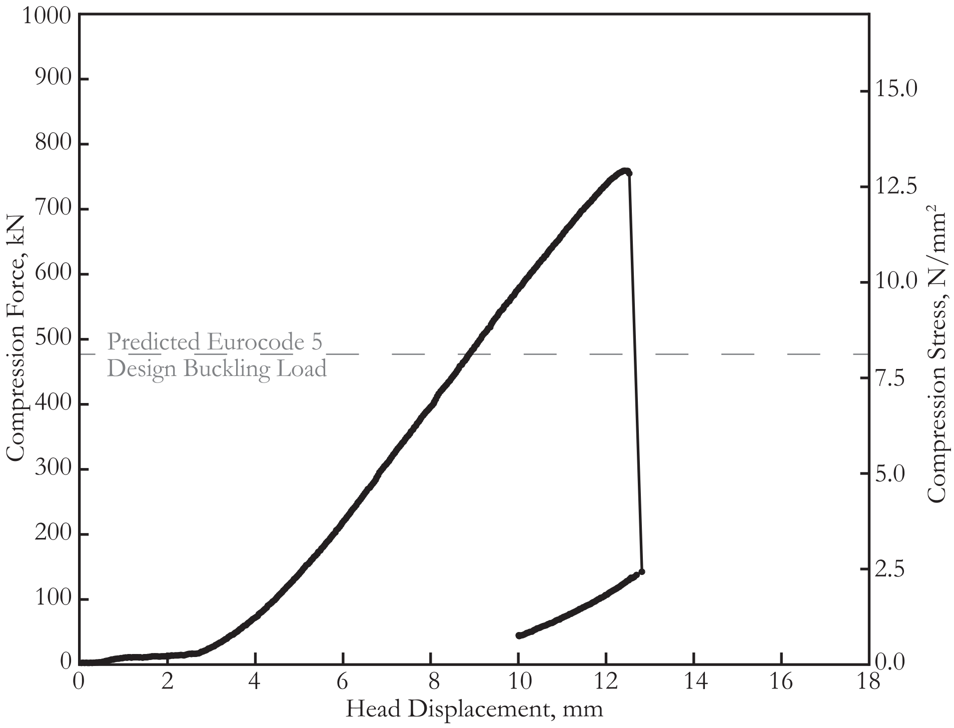
Test
10

759kN
Max. Force

9.5 7.9 9.8 10.1 10.7

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

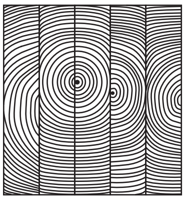
50mm 250mm 500mm



Test 11:

Mean Moisture Content Percentage at Time of Testing

14 19 18 17 14



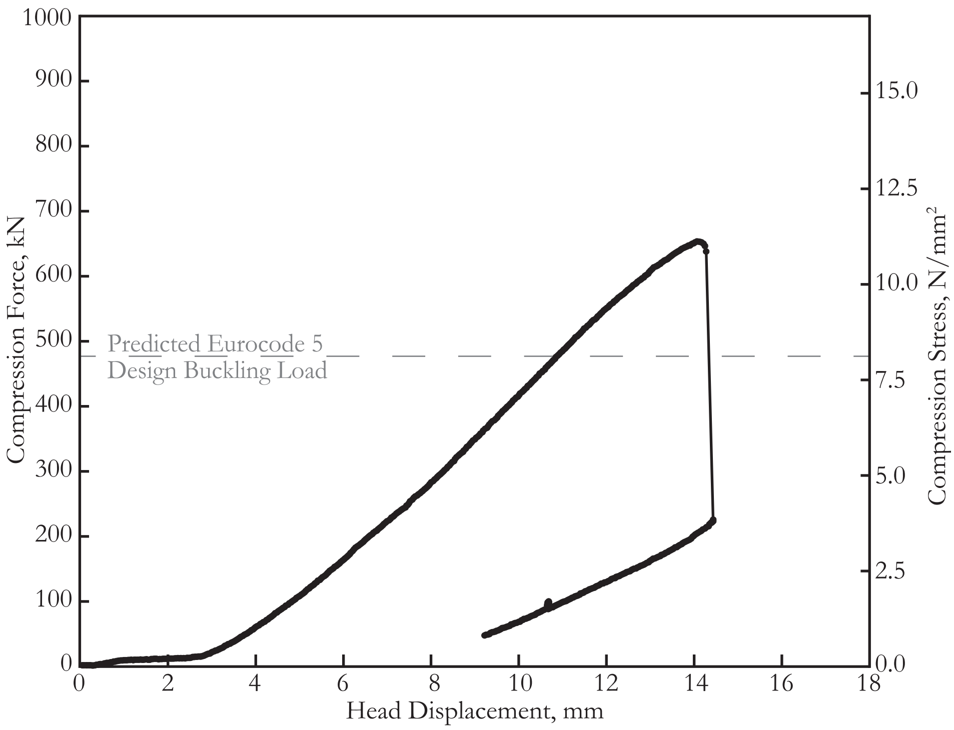
Test
11

652kN
Max. Force

7.5 6.2 6.5 6.7 6.2

MoE, Corrected for Moisture Content at Time of Testing, kN/mm^2

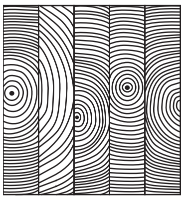
50mm 250mm 500mm



Test 12:

Mean Moisture Content Percentage at Time of Testing

14 17 15 14 14



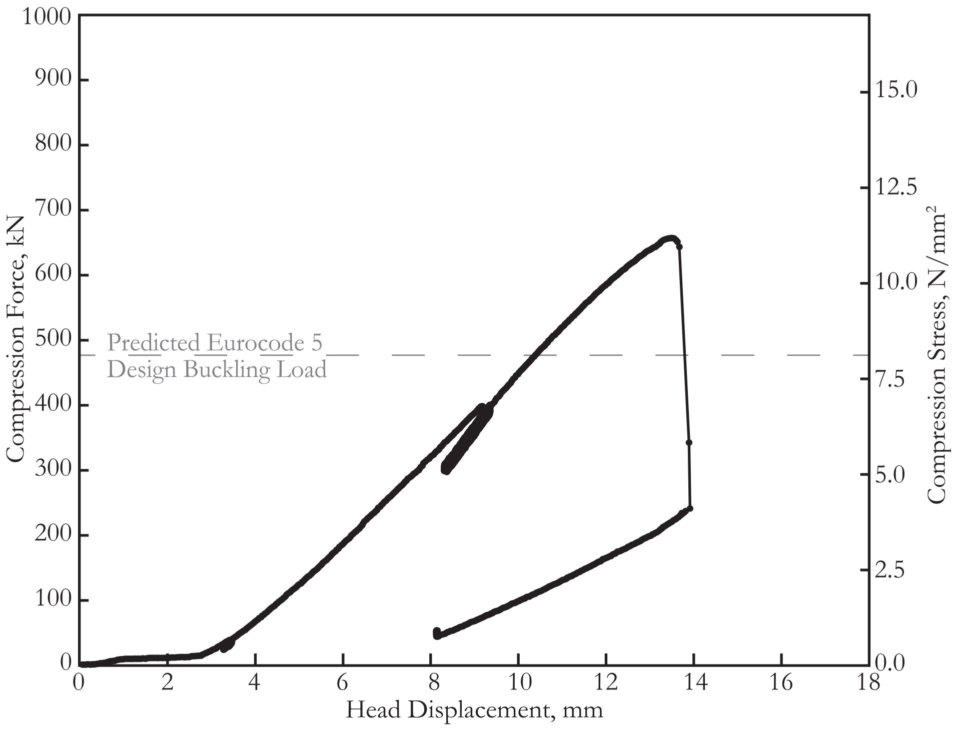
Test
12 (9*^{P})

657kN
Max. Force

7.0 6.6 5.1 6.5 5.9

MoE, Corrected for Moisture Content at Time of Testing, kN/mm^2

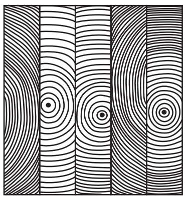
50mm 250mm 500mm



Test 13:

Mean Moisture Content Percentage at Time of Testing

10 10 10 10 9



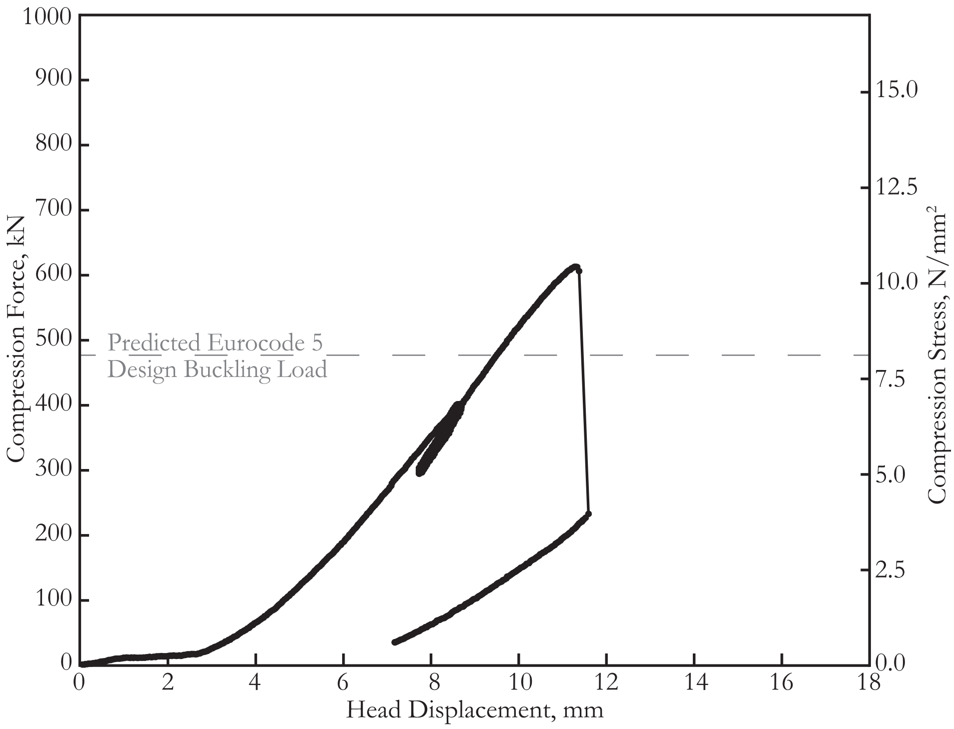
Test
13 (7)

613kN
Max. Force

7.1 6.4 5.0 6.5 5.8

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

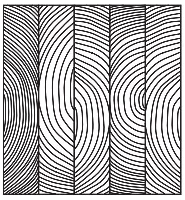
50mm 250mm 500mm



Test 14:

Mean Moisture Content Percentage at Time of Testing

10 10 10 10 10



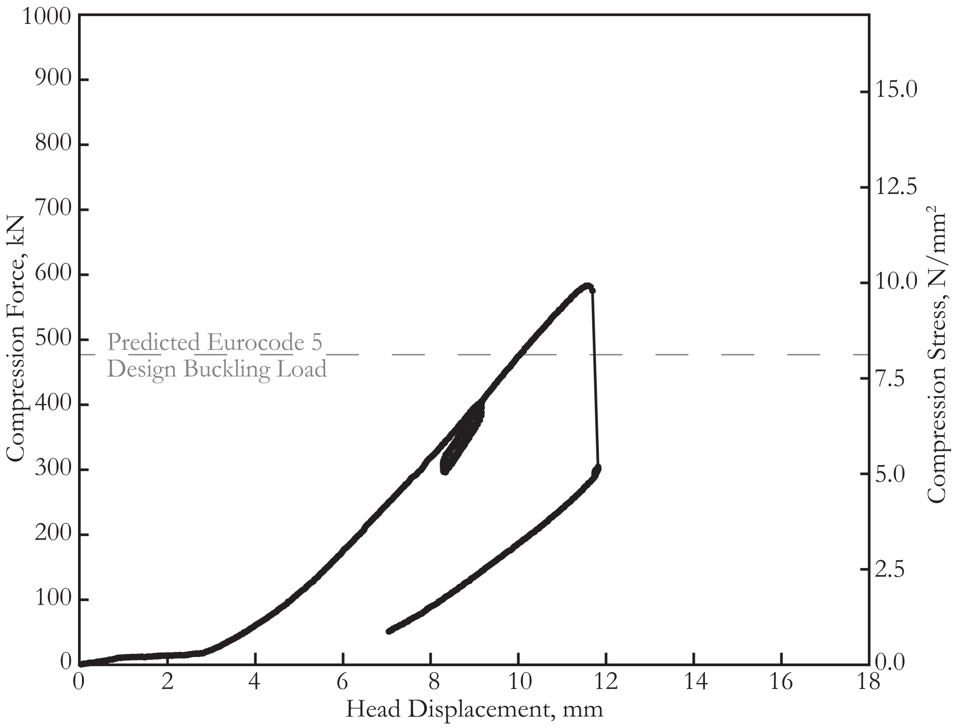
Test
14 (6)

583kN
Max. Force

9.0 6.4 7.3 7.4 7.3

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

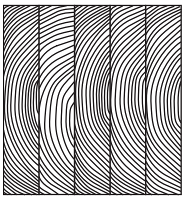
50mm 250mm 500mm



Test 15:

Mean Moisture Content Percentage at Time of Testing

10 10 11 10 10



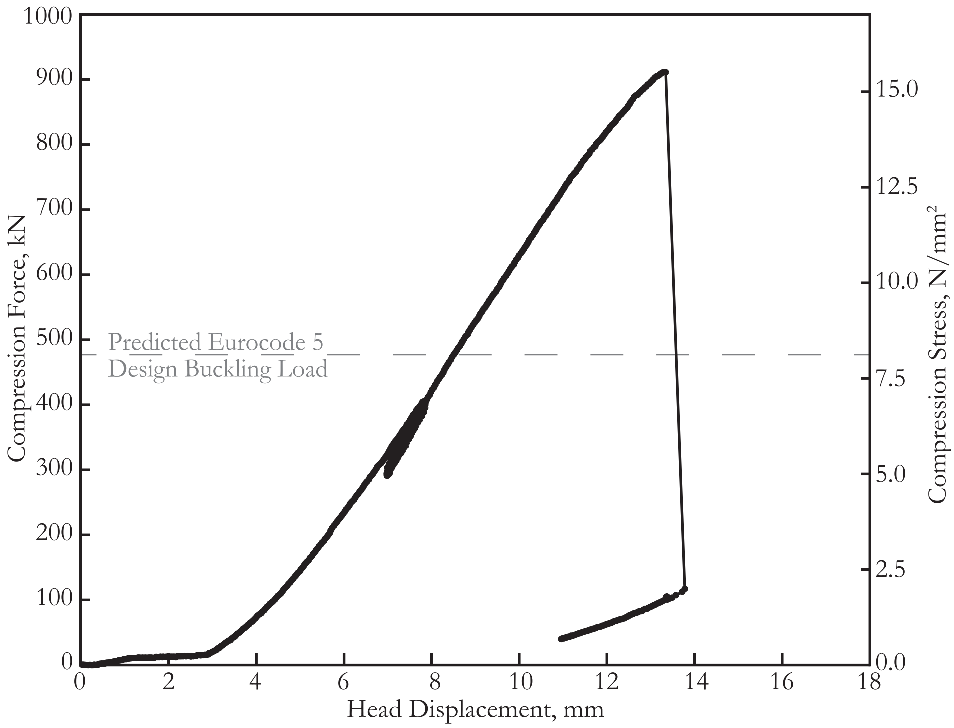
Test
15 (10*F)

911kN
Max. Force

9.8 8.4 10.6 10.6 11.4

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

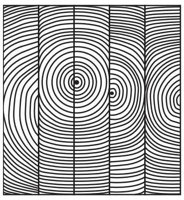
50mm 250mm 500mm



Test 16:

Mean Moisture Content Percentage at Time of Testing

10 10 11 10 10



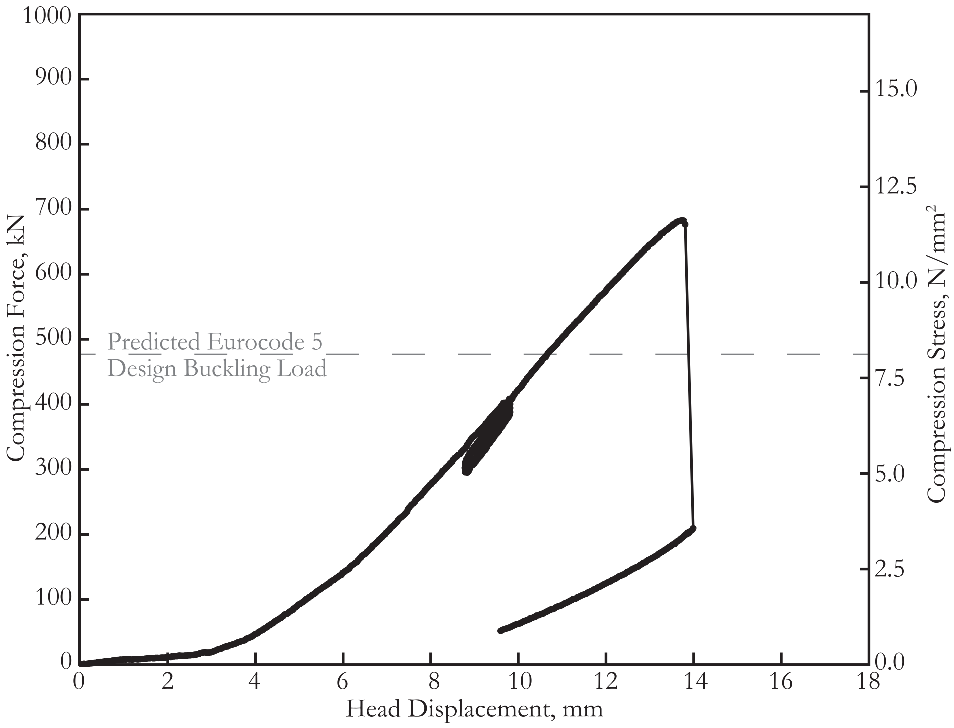
Test
16 (11)

683kN
Max. Force

7.7 6.7 7.0 7.2 6.4

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

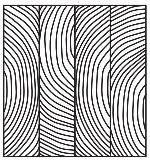
50mm 250mm 500mm



Test 17:

Mean Moisture Content Percentage at Time of Testing

14 14 14 13



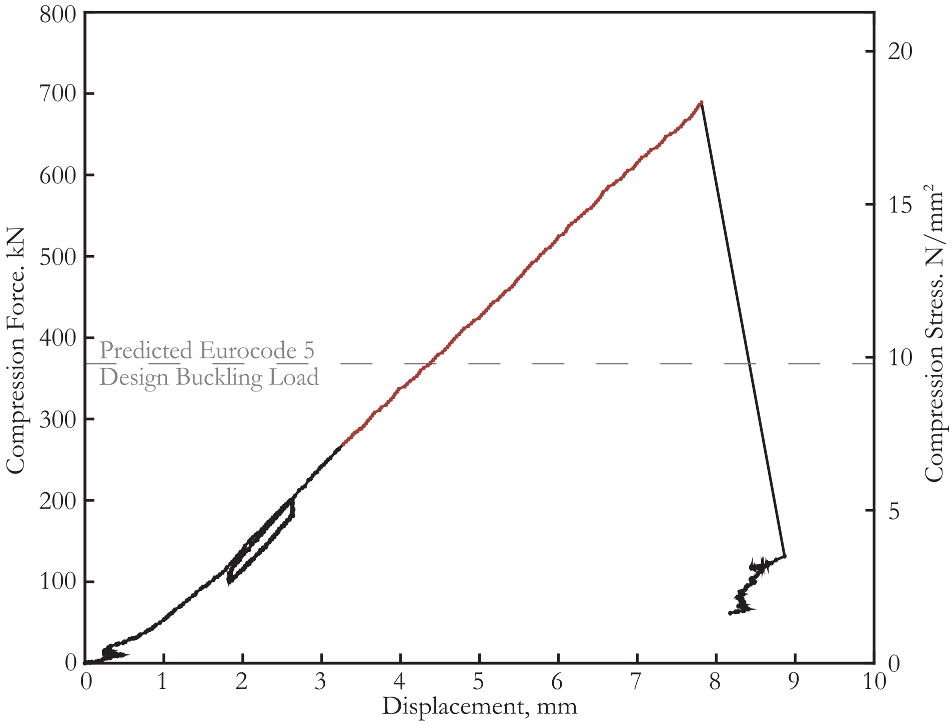
Test
17

689kN
Max. Load

7.1 5.9 8.7 9.1

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

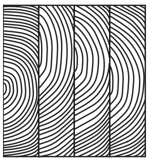
50mm 250mm 500mm



Test 18:

Mean Moisture Content Percentage at Time of Testing

13 13 13 14



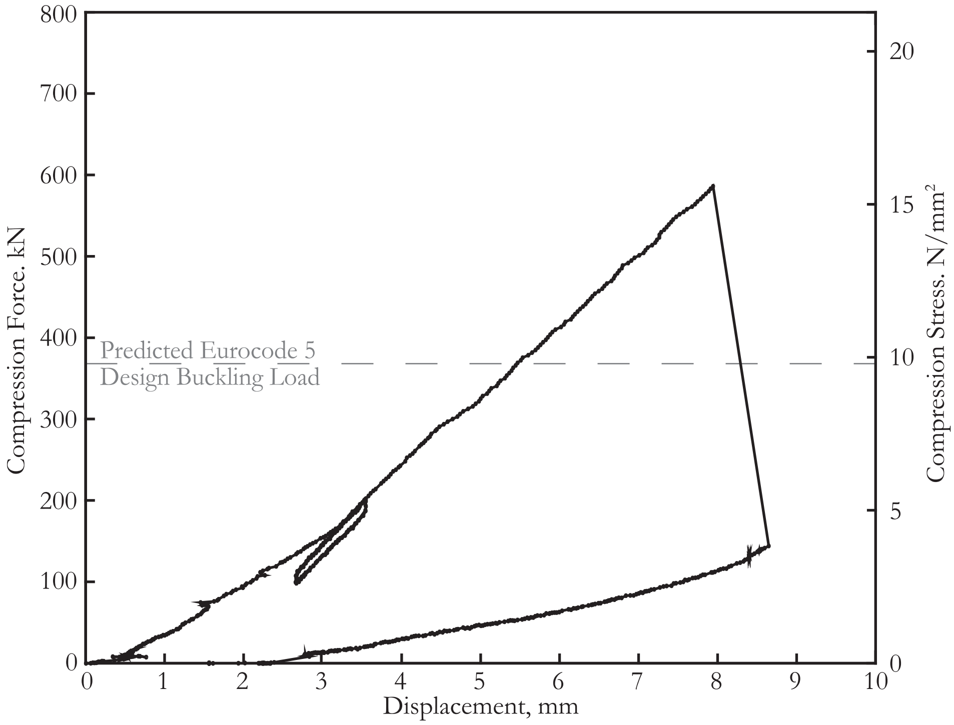
Test
18

587kN
Max. Load

6.4 9.3 6.9 9.5

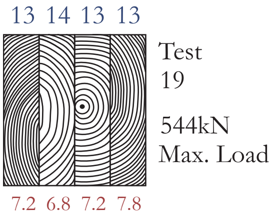
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

50mm 250mm 500mm

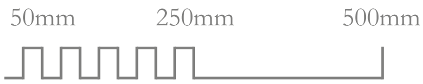


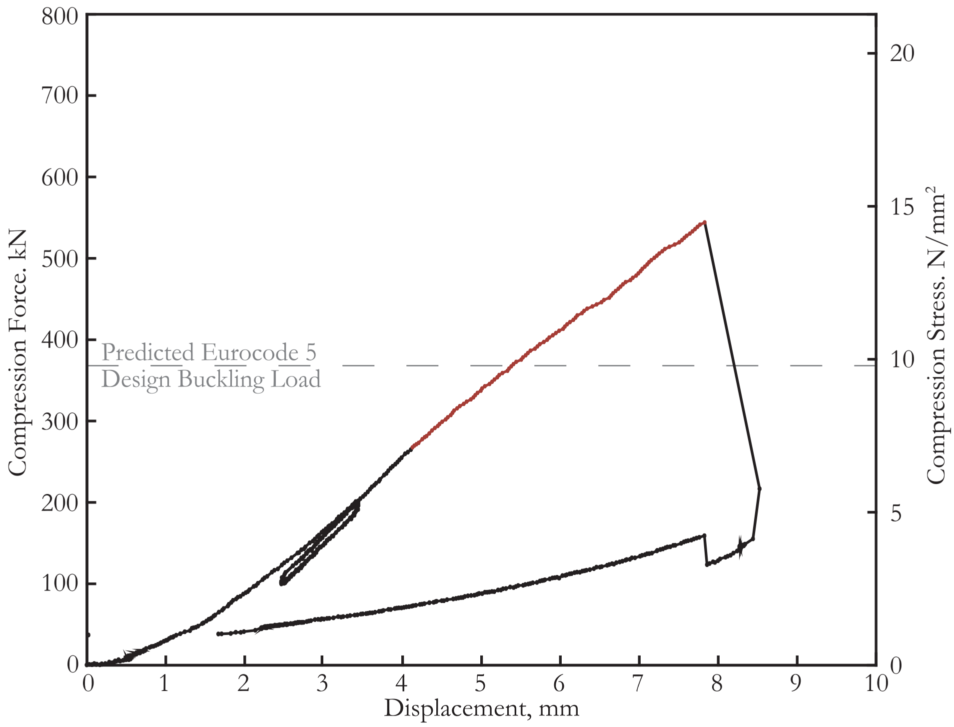
Test 19:

Mean Moisture Content Percentage at Time of Testing



MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

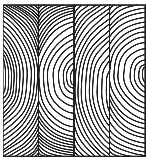




Test 20:

Mean Moisture Content Percentage at Time of Testing

12 12 12 12

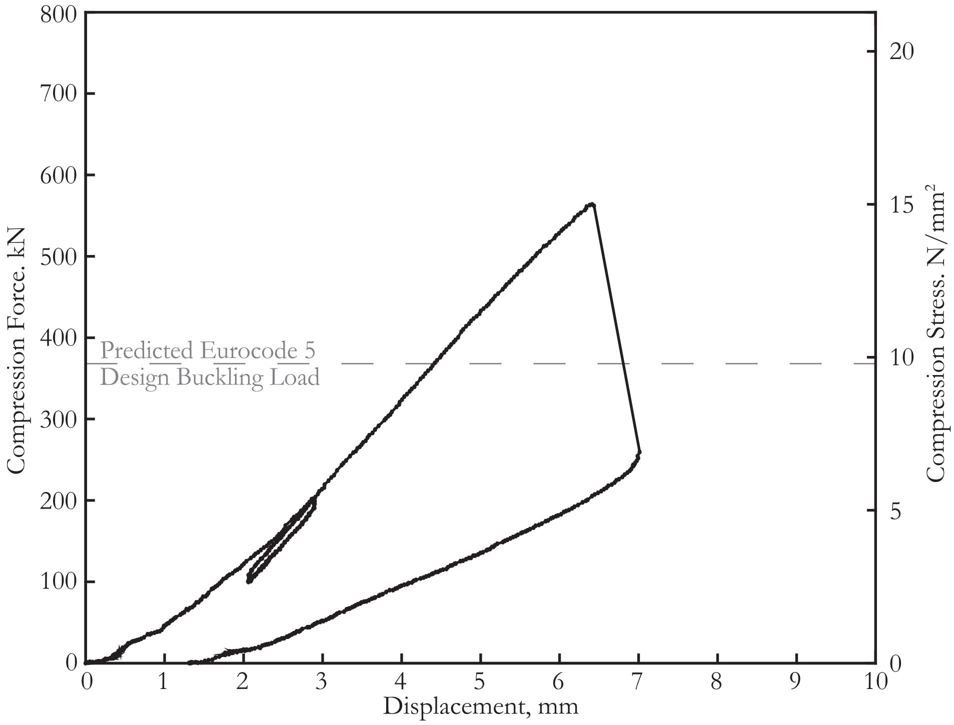


Test
20

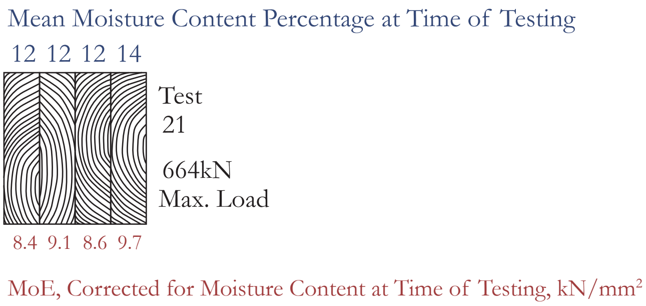
564kN
Max. Load

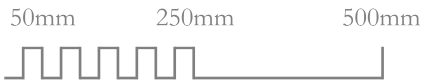
9.0 8.3 8.3 8.4

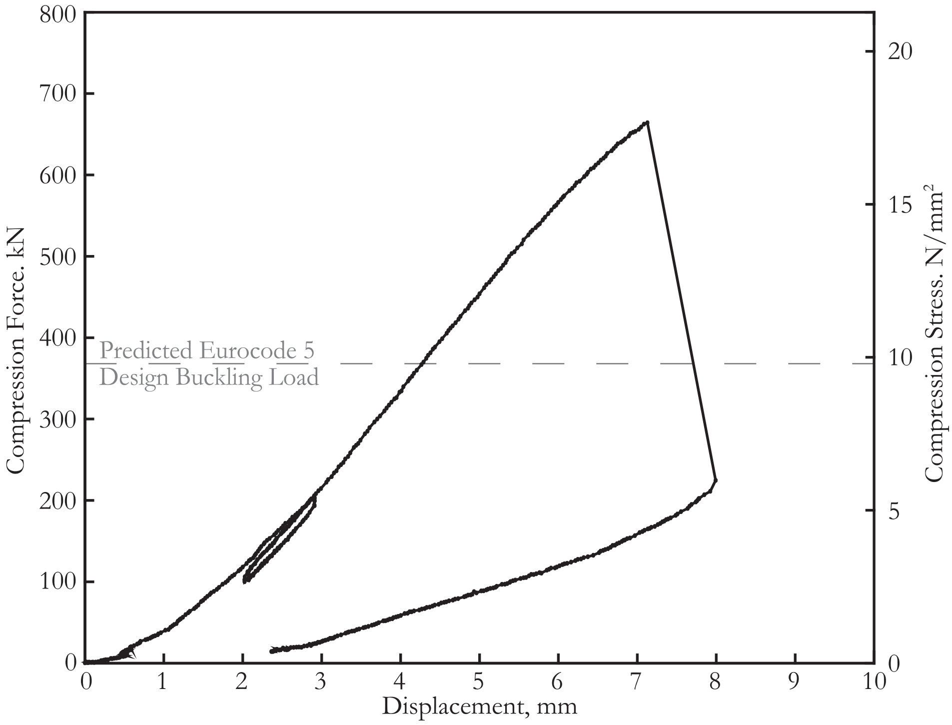
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²



Test 21:


Mean Moisture Content Percentage at Time of Testing
12 12 12 14
Test
21
664kN
Max. Load
8.4 9.1 8.6 9.7
MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

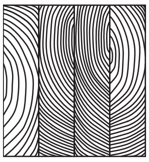

50mm
250mm
500mm


Compression Force. kN
0
100
200
300
400
500
600
700
800
Compression Stress. N/mm²
0
5
10
15
20
Displacement, mm
0
1
2
3
4
5
6
7
8
9
10
Predicted Eurocode 5
Design Buckling Load

Test 22:

Mean Moisture Content Percentage at Time of Testing

13 13 14 13



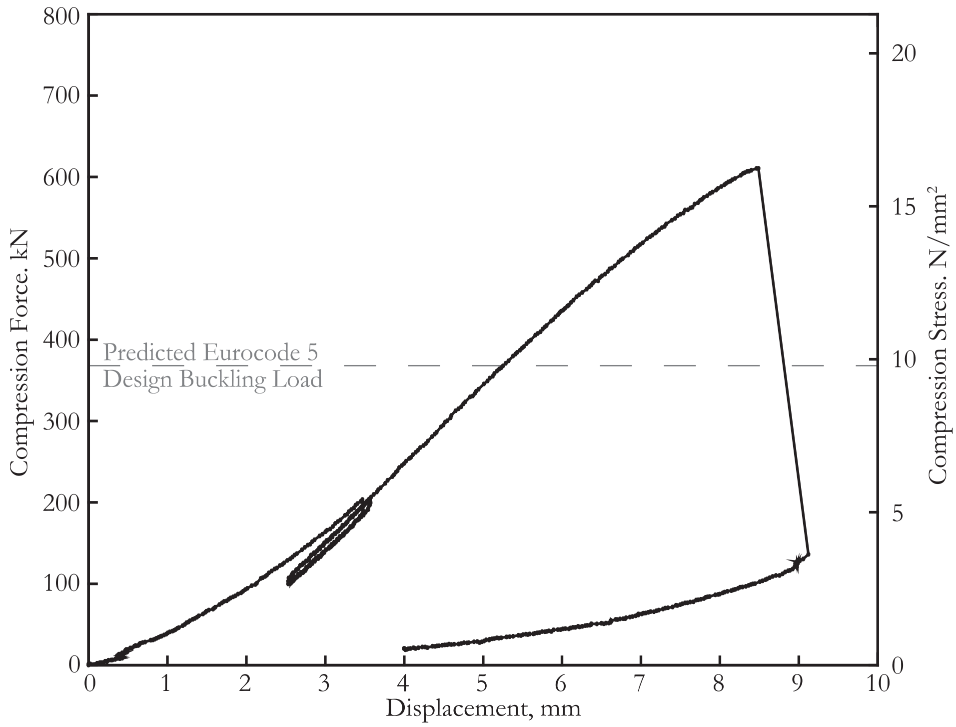
Test
22

611kN
Max. Load

7.9 7.2 7.6 7.1

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

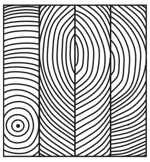
50mm 250mm 500mm



Test 23:

Mean Moisture Content Percentage at Time of Testing

15 13 14 15



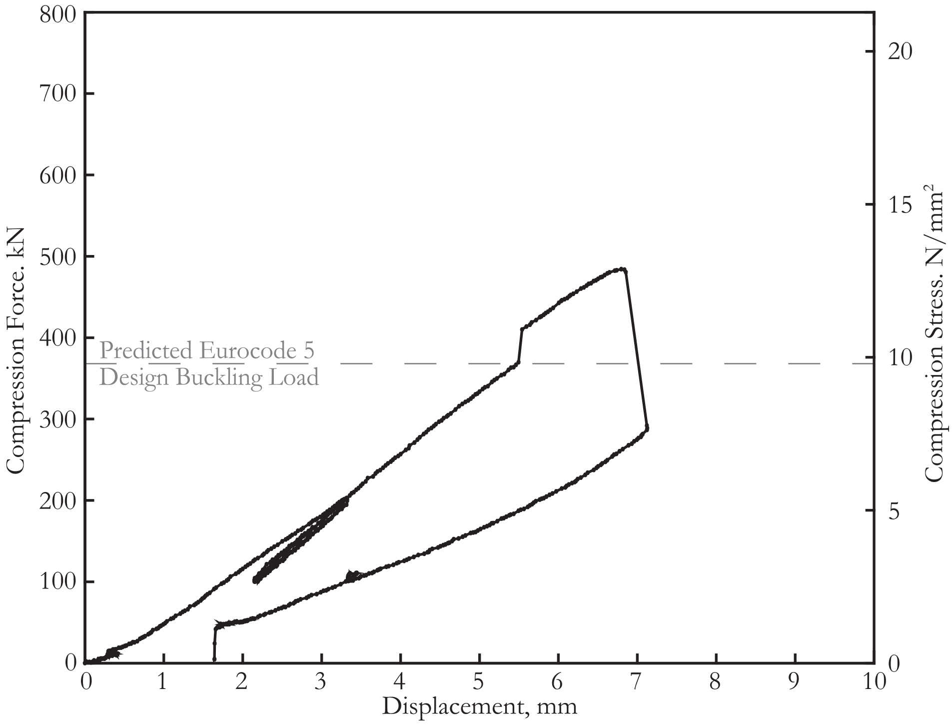
Test
23

484kN
Max. Load

5.6 6.9 5.4 6.8

MoE, Corrected for Moisture Content at Time of Testing, kN/mm^2

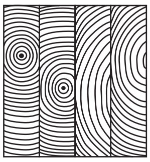
50mm 250mm 500mm



Test 24:

Mean Moisture Content Percentage at Time of Testing

14 14 14 14



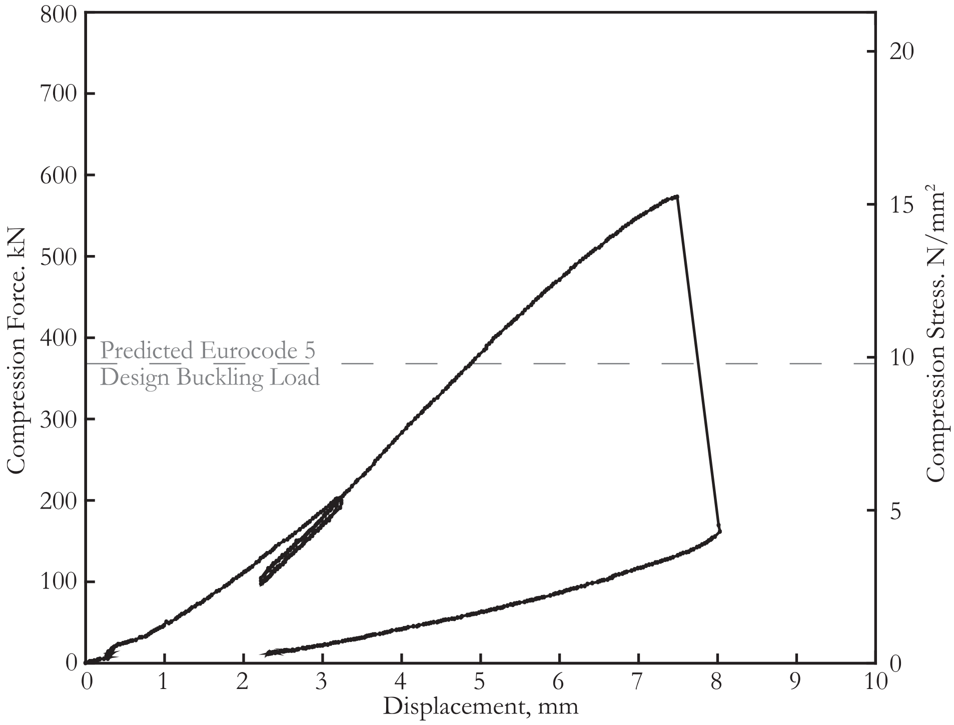
Test
24

573kN
Max. Load

6.2 7.1 9.2 8.1

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

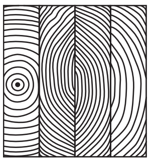
50mm 250mm 500mm



Test 25:

Mean Moisture Content Percentage at Time of Testing

14 14 14 14



Test
25

474kN
Max. Load

7.4 5.9 8.1 7.3

MoE, Corrected for Moisture Content at Time of Testing, kN/mm²

50mm 250mm 500mm