A state-of-the-art review of carpentry connections: From traditional designs to emerging trends in wood-wood structural joints

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Highlights

- Overview of history and context of steel-free, adhesive-free, wood-wood connections
- The advantages and disadvantages of timber carpentry connections are explored
- A literature review of research related to traditional connections is presented
- Experimental, analytical and numerical approaches to wood-wood joints are discussed
- Contemporary connections and emerging fabrication technology are presented

Abstract: Interlocking timber connections that are steel-free and adhesive-free are uncommon in modern construction, but the necessity to maintain cultural heritage alongside the rise in demand for sustainable buildings has revived interest in traditional carpentry techniques, as well as contemporary wood-wood and all-wood connections. This review aims to draw together the past, present, and future applications of structural timber joints from a holistic perspective. In response to inadequate design guidelines, research on the mechanics of timber joints that incorporate traditional techniques has increased globally. Furthermore, sustainability objectives are prompting the modernization of connections, particularly in digitization and fabrication. The drivers of the rises and falls in the popularity of timber carpentry are explained and give meaning to the ongoing increase in interest in timber connections. Perceived advantages to timber joints, as well as the limitations, are explored herein. This work includes a comprehensive literature review of emerging research and highlights connection evaluation methods, including experimental, analytical, and numerical modeling approaches. A discussion of modern trends in technological advancements, particularly those utilizing digitization and robotic milling, foreshadows the future of carpentry in modern construction.

Keywords: timber joints, timber connections, carpentry, wood-wood, traditional, cultural heritage, contemporary, evaluation methods, digitization.

1 Introduction

Carpentry is the craft of joining wood, and in the context of structural connections, this term “traditionally” refers to wood-wood connections that are linked together by geometries that can provide the interlocking and friction necessary to transfer loads between elements while maintaining rigidity. Heritage structures with wood-wood connections can be found worldwide and date back hundreds or thousands of years depending on the locality. It is only in relatively recent history that steel fasteners and glued joints have supplanted wood-wood connections in popularity. Today, metal fasteners and glues are commonly used due to convenience and cost efficiency, but with changes in technology and demands for sustainability, the idea of reintroducing wooden joints into modern construction is increasingly appealing.

Wood has been a key material throughout history and across cultures. With the emergence of civilization came the cutting and shaping of wood, from chopping down trees to building structures [1]. For centuries carpenters across developing civilizations refined the practice of shaping wood into timber structural elements with interconnecting parts to build countless structures. Many traditional techniques are millennia old, like Tibetan building techniques that date back 4500 years, and centuries-old timber structures like the Potala Palace and Jokhang Temple still stand today [2]. Structures with Dou-gong brackets, a common joint dating back as far as 770 BC, can still be found throughout heritage timber buildings in China, Korea, and Japan [3]. Egyptian archaeological sites have been known to exhume millennia old woodworking tools like hand saws, chisels, and mallets alongside artworks depicting carpenters using the same tools [4]. Likewise, wood may have been the most important material of the Roman Empire as timber carpentry was used in temporary construction structures, buildings, ships, wagons, infrastructure, and household items [5]. Archaeological excavations have even shown that some carpentry techniques used in Ancient Rome can still be found today in heritage buildings [5]. Carpentry has been a fundamental contributor to the rise of civilization and is a part of the heritage of many cultures. As such, it is cultural heritage that has prompted researchers to seek knowledge related to carpentry as a trade and a science.

Many building practices seen today were also in use by ancient carpenters, like the regional standardization. Historical standardization of practices was often presented as localized trade customs, a code of direction, or design manuals featuring logistical recommendations. From this natural evolution of standardization, unique regional carpentry styles were adopted from trial and error, which brought tradespeople to design better structures. Local tradespeople possessed skills and knowledge that were refined and passed down over generations. The knowledge of materials, tools, and techniques often resulted in buildings that lasted centuries, though much of the technical knowledge is now lost as it has disappeared with the craftspeople who possessed it. Few carpenters today work with wood-wood connections.

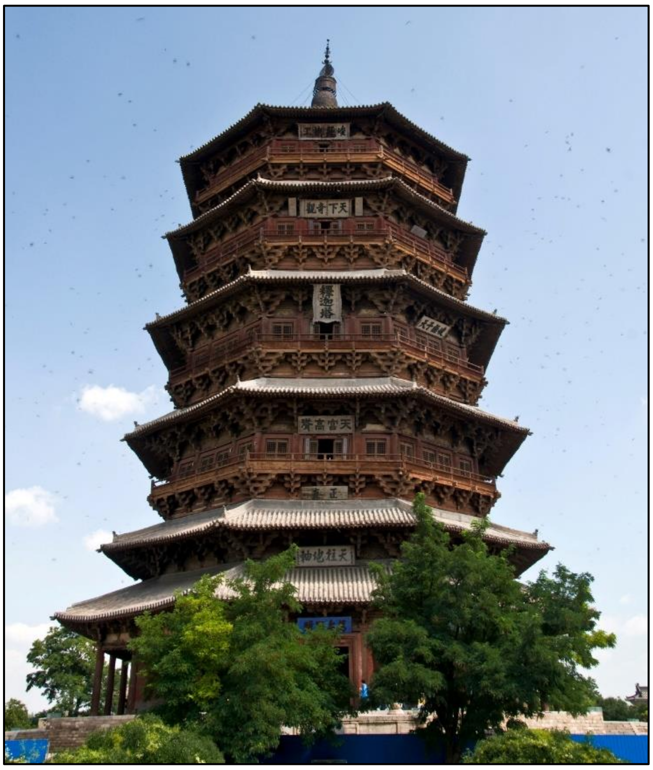


Figure 1: *Yingxian* wooden pagoda of *Fogong* temple, first built in 1056AD in China (Stock image [6]).

Modern masters and apprentices of traditional carpentry are extremely rare. The Japanese are often considered the best example of preservation of carpentry culture as the continued use of carpentry has shaped exemplary craftspeople. Traditional Japanese timber structures date back as far as the 3rd century, and temples dating back to the 9th century still partially survive today [3]. Traditional Japanese timber structures have also shown to have high resilience as they resist forces like earthquakes and typhoons by using imbedded timber connections [7]. One comprehensive overview of this preserved skill and tradition is given by Brown (2013), who documented the history, techniques, and craftspeople involved in Japanese carpentry and recounted his experience shadowing Tsunekazu Nishioka, one of the last remaining Japanese master carpenters, as he rebuilt a traditional temple in the 1980s [8]. Elsewhere, masters in wood-wood carpentry are extremely uncommon.

Perhaps one of the most significant consequences of localized recognition of best practice is the development of standardized design guidelines, of which the best early example might be the ancient Chinese building codes, which nominated techniques for timber buildings and featured carpentry connections. Most ancient Chinese architecture might more accurately be described as construction since the ancient methods used over centuries is a set of regulations called the *Yingzao Fashi*, or simply "Building Standards", of which the document was rediscovered during the time of the restoration of the Forbidden City in 1919 [9]. These standards in updated revisions existed from the 9th to the 14th centuries, which prescribed standard geometries and relative ratios of timber elements and connections to aid craftspeople in designing and cutting unique elements by hand [9]. Heritage structures built to these codes, like the *Yingxian* pagoda (Figure 1), can still be seen today. Alongside the building standard, master carpenters used carpenter's manuals passed down in each carpentry family with secret records of family techniques, one well-known example of which is the *Yingzao Fayuan* (Basic Rules for Building) [10].

Tradition and standardization were also seen throughout the development of Western civilizations. Early Western carpenters learned trades from master carpenters, and since construction books did not exist, educational deficiencies prompted the existence of manuals to address the lack of knowledge on matters like measurement, pricing, geometry, and surveying [11]. The manuals inadvertently recorded the progression and evolution of local connection typologies and those in research and industry often refer to designs prescribed in historical manuals, many of which can be digitally accessed as public domain publications. Manuals were trusted to nominate designs until the beginning of the 19th century when the engineering profession emerged separately from architecture to specialize in recommendations dictated by scientific design [11]. Though today nearly everywhere has building codes based on scientific methods, not all building codes provide scientifically derived design guidelines for traditional or modern wood-wood carpentry connections, and those that do are often regarded as insufficient. Design guidance of wood-wood connections is a contentious topic throughout the literature, and improvements are a ubiquitous aim across research.

Historical connections can still be seen in traditional buildings throughout the world. Existing structures with historical carpentry connections are most notably seen in North America, Europe, China, and Japan [12]. Following centuries of carpentry in common use, few new buildings with traditional connections have been erected since a global shift away from wood-wood connections during the 20th century. Some reasons for this include supply chain insufficiencies, advances in competing materials technology, and economic drivers. First, a decline in suitable material (tree) availability meant limited supply [13]. Historically, structures could be built from trees with large diameters while today the global supply is widely consistent of logs of poorer quality and smaller diameters [14]. For example, in North America old growth forests had historically provided trees that grew large enough to make incredibly tall structures, known as “heavy timber” structures, but as alternative materials became more economical alongside material depletion, this building style disappeared throughout the second half of the 20th century [15]. A further deterrent was the high costs of skilled labor from craftspeople capable of shaping connections due to the time required [13]. Consequences of this cost in the market drove demands for more economically competitive technology, which was achieved by metal fasteners that eventually depleted the demand of skilled tradespeople nearly to the point of nonexistence. Similarly, timber was once a very common building material in China, but resource depletion reduced usage and therefore resulted in the loss of demand for knowledge of traditional timber construction and timber engineering [16]. The trend of material depletion and loss of traditional knowledge and craftspeople is common across global construction trends with a few notable exceptions in the far east where carpentry culture, adequate materials management and world class craftspeople still persist [3]. Elsewhere the impact of supply changes, economic interests, skilled labor deficiencies, and new technology made the abandonment of traditional carpentry inevitable.

The returning interest in traditional carpentry has two main drivers: cultural heritage preservation and sustainability. While interest in cultural heritage has been fueling motivation in research related to maintenance, preservation and restoration of historical connections for several decades, the emergence of interest in new buildings that incorporate aspects of traditional structural timber carpentry connections is still in its infancy. Building codes generally have limited design guidance for carpentry connections, therefore they are rarely used. To bypass roadblocks from lack of design guidance, researchers are more thoroughly exploring the empirical principles incorporated in historical designs. By better quantifying material behaviors of various carpentry connections, eventually the disparity between existing connection details and complimentary design guidance might be rectified. To better understand the globally ubiquitous considerations both propelling interest in carpentry and holding back meaningful integration into modern building practices, this review includes (1) an introduction of the historical context of carpentry, (2) a discussion of the perceived advantages of carpentry, (3) drawbacks and limitations of wood-wood connections, (4) a literature review of traditional carpentry connections, (5) an overview of experimental, analytical and numerical evaluation methods, and (6) an investigation into contemporary carpentry connections.

2 The modern revival of carpentry and the perceived advantages

2.1 Preservation of heritage structures

Following the decreased popularity of carpentry during the early 20th century, interest in carpentry among researchers was limited for many decades. Consequentially, many modern building codes still have roots from design manuals dating back to the 19th century or earlier, as this is the era when carpentry was abundant [17]. After decades of disinterest, research of wood-wood carpentry connections is currently seeing a global modern revival. One reason for this is the desire to conserve heritage building stock [12,18]. For example, in China heritage structures have been studied as early as the 19th century, but modern research is reviving the topic and approaches it with the aid of modern methods [18]. Likewise, to accommodate the demand for restoration, the Chinese have been seeking traditional carpentry apprentices since 2009 to keep up with demands [19]. Since the end of the 20th century, a great deal of global interest in experimental research regarding historical carpentry has emerged [20]. This is a widespread trend.

The deterioration of heritage structures has prompted both general studies of connections and technology to strengthen aged connections. Mostly for restoration purposes, investigations have aimed to better understand ancient techniques, including the various tools and systems involved in woodworking [1]. Heritage preservation is of great interest to researchers as aging building stocks worldwide constantly require repairs and enhancements to resolve structural problems in culturally valuable heritage structures [21]. Since replacing whole elements is complicated, historical joints might otherwise be retrofit, for example, by employing the use of either metal connectors or epoxy resin [22]. Retrofitting is often more economical to repair damaged joints or increase performance, but sometimes structural elements and joints are replaced entirely [21]. For instance, despite limited understanding of historical carpentry joints, traditional connections are still often used in the reconstruction of historical trusses in need of replacement parts [23]. Others have offered guidance on enhancing connection performance, as seen in a broad overview of common historical connections and retrofitting strategies where Branco and Descamps (2015) offer strengthening techniques and evaluation methods of joints as semi-rigid springs by evaluating stiffness and strength to better understand global behavior and determine optimal retrofitting locations to avoid overdesign [24]. It should be noted that despite the complications of aging heritage buildings, most demolitions of ancient timber buildings are carried out for nonstructural reasons and not due to degradation [25]. Despite widespread concerns of using wood-wood connections, timber elements generally perform well over centuries, though further evidence could reassure the public of performance reliability [1]. Therefore, researchers often focus on the robustness of aged joints, new joints, and retrofitting.

2.2 Sustainability

While the construction industry grapples with increased sustainability expectations, the urgency to use sustainable materials to counter climate change has become the most recent cause of the revived interest in wood-wood connections. At the turn of the 21st century, the desire to design both sustainable and aesthetic buildings have driven an increase in timber structures with a renewed focus on new approaches to traditional methods [3]. Building construction is estimated to account for 36% of global energy consumption and releases 39% of CO_2 from energy use [26]. Consequentially, there is growing interest to instate sustainable construction methods and use materials with low embodied energy [12]. As buildings are one of the greatest contributors to CO_2 emissions, building material choices are being driven toward timber as it is a renewable material with carbon storing capabilities and low embodied carbon from production [27]. Engineered timber (ET) products like plywood, cross-laminated timber (CLT), laminated veneer lumber (LVL), and glue laminate (glulam) are often the materials selected for modern tall buildings with sustainability objectives since timber can store carbon and engineered timber can have increased strength or geometries compared to sawn timber. Though the Mass Timber (MT) building movement involving large engineered timber structures began in the 1990s, the late 2010s have seen a global increase [28]. A study on the potential impact of replacing concrete with engineered timber suggested a decrease of between 34-84% in greenhouse gas emissions [29]. A lifecycle analysis of high-rise mass timber buildings found that a switch from concrete to engineered timber would mean a significant decrease in greenhouse gas emissions from buildings in addition to stored carbon, an overall decrease of about 69.5% [30]. However, one disadvantage is the increased embodied energy and CO_2 from extensive transformation of natural timber into engineered timber [31], and a product's stored carbon versus embodied carbon is often unapparent.

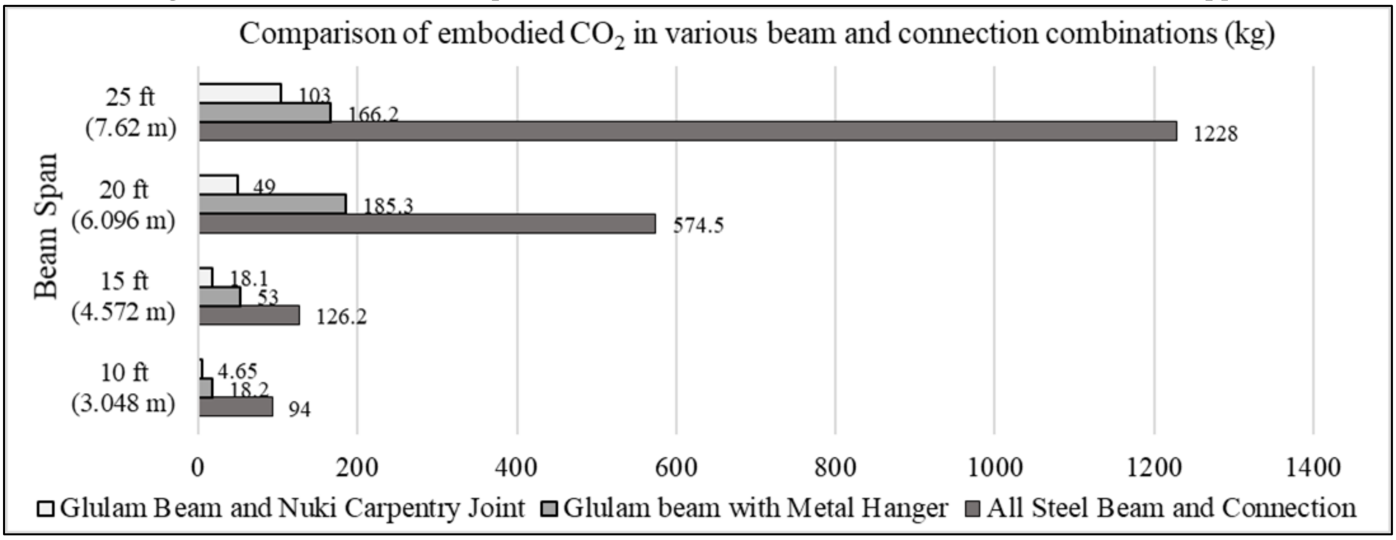


Figure 2: Comparison of embodied CO2 emissions data in glulam beam with *Nuki* carpentry joint, Glulam beam and metal joist hanger, and steel beam and steel connection [9].

Replacing the use of metal fasteners with interlocking wood-wood connections is one further sustainability approach. Steel must be mined and fabricated, both of which incur high energy use. Embodied energy from primary production of cut Sitka spruce is approximately 10.5–11.6 MJ/kg, whereas structural grade steel uses more than twice that [25]. As seen in Figure 2, Feng (2020) proposed incorporating wood *Nuki* beams with *Nuki* joints as an alternative to timber beams with metal joist hangers and steel beams with steel connectors in modern construction to drastically reduce emission from embodied CO_2 [10]. Structure lifespan may also be increased by avoiding steel components. Steel and wood suffer thermal incompatibility in (i) expansion and contraction where changes in temperature and humidity cause differences in local stiffness which leads to failure parallel to the grain at embedment locations and (ii) geometric moisture variations that cause bolts to become loose when timber shrinks [32]. A further concern of timber elements with steel connectors is the increased moisture retention of steel connectors compared to wood-wood connections due to steel plates trapping moisture and hindering the drying process therefore prolonging the effects of moisture exposure [33]. Wood-wood connectors are the next logical step in sustainable design, but limitations are underexplored.

It should be highlighted that the topic of increased timber usage also prompts sustainability concerns, particularly regarding deforestation and material lifecycle management. First, many concerns result from the widespread deforestation in the early 20th century. To address climate change with urgency while maintaining ecosystem quality, adequate controls are necessary to balance the many sensitive factors impacted by a widespread material shift [34]. Material supplies must be planned far in advance to avoid deforestation since trees take decades to reach a size suitable for building. Various fast-growing species of structural timber have been nominated in respective regions as strong candidates for mass production though this is an ongoing area of research [14]. However, effective forest management could address both deforestation and materials supply. Ramage *et al.* (2017) found that even a small increase of total forest area intended for timber construction would make a meaningful difference in carbon reduction [25]. Material lifecycle issues present further concerns, particularly when considering supply and demand implications, allocation of land for forestation, and protection of existing ecosystems with increased tree farming [27]. Additionally, end of lifecycle logistics of recovered timber would ideally be arranged for wood to be (1) re-purposed into a secondary life flooring, (2) reused as a new material like paper, or (3) converted into energy [25,27]. Unfortunately, advanced lifecycle planning that includes tree growth, ecosystem management, harvesting, building management, and reuse and recycle of materials is complex and requires cooperation across several industries. Furthermore, there is a disconnect between material suppliers and building designers. Future designers must have both the foresight to nominate materials that are in good supply and the knowledge necessary to adapt designs throughout changing material availability [35]. Overall, to effectively use timber in a meaningful way, more intimate lifecycle systems between key industry stakeholders are required.

A further difficulty in using sustainable timber products is the unaffordability of materials. New large timber buildings are generally engineered timber with special connection detailing, both of which draw presumptions of high costs. One economic disadvantage of selecting engineered timber products is that the processing required to transform wood incurs costs that make the most popular engineered timber materials expensive compared to alternative materials [31]. It is not uncommon for the price of timber to increase significantly from sawing, grading, planing, gluing, and transformation into engineered timber products, which can make timber less economically competitive than concrete. In a regional analysis of the economic impact of high-rise mass timber buildings with engineered timber in Oregon, United States, engineered timber was found to drastically increase the materials cost when compared to a concrete equivalent while other costs remained comparable, though ultimately a timber building was found to cost nearly double that of an equivalent concrete building [36]. In addition, government crackdowns on illegal logging, timber trafficking, and deforestation has resulted in various requirements to verify timber legality though identification of country of origin, genus and species, though regulation and identification methods drive up the initial cost of raw materials in many countries [37]. Consequentially, compounding costs often deter investors from building with timber. Furthermore, contractors might also increase cost estimates to account for risk of potential expenses incurred due to unforeseen problems like inaccurate pricing of unfamiliar materials, delays from unfamiliar installation practices, unfamiliar detailing resulting in incorrect installment, or other unexpected expenses. Kremer *et al.* (2019) conducted a series of interviews regarding modern mass timber construction across industry professionals and reoccurring responses indicated that a key concern in incorporating more mass timber in Australia is related to financial risk [38]. As carpentry connections in large projects are uncommon and unfamiliar, additional costs may exist from perceived risks of stakeholders. While hand carpentry in modern building is extremely uncommon and very expensive, one common observation in modern research is the potential of computerized milling (See section 6.1 Digitizing carpentry) to reduce the cost of shaping connections thus making sustainable connections more affordable, though examples of carpentry connections in large timber structures have been limited throughout industry (See section 6.3 Commercial construction case studies).

3. General limitations of timber joints

Unlike other building materials, timber is a biological material. As such, trees are prone to significant defects and physical variations, as well as disease, infestation, and decay. This reality presents problems when idealizing a structure because each structural member has unknown mechanical properties present at defects, as well as vulnerability to degradation and unique susceptibilities. Key shortcomings of timber include moisture sensitivity (geometric and biological), increased sensitivity to critical lateral loads with increased building height due to light-weight materials, orthotropic behavior (See section 5 Experimental, analytical, and numerical methods in carpentry connection evaluation), relatively lower connection stiffness, brittle failure modes, and limited knowledge of load distribution systems, impacts of structural element size, and expectations of material creep behavior over time [15], though additional shortcomings exist. It is understandable why industry preferences favor materials with reduced complexity and greater predictability. Additionally, these unknowns often prompt justifications for lack of design guidelines and building codes addressing wood-wood joints. Designing connections without code specifications is an area where few engineers specialize and such connections create risks that stakeholders are often reluctant to accept. This section explores how connections are impacted by the general shortcomings of timber, including (i) natural defects, (ii) moisture sensitivity, (iii) connection degradation, (iv) fire vulnerability, and (v) design guidance.

(A) (B) (C) (D)

Figure 3: Examples of (A) variant grain direction around a knot in tree trunk, (B) log cracking radially and longitudinally, (C) dried trunks with longitudinal cuts for crack prevention, and (D) various fungal growths at cut end of logs (photos by authors).

3.1 Natural defects

There is a range of natural defects to consider that might impact the mechanical behavior of timber connections. Material defects might include knots, grain direction, density variation, growth stresses, or cracking, and may hinder adequate load transfer or cause connection failures. Even in engineered timber products with improved material properties, weaknesses where failures occur are often associated with material characteristics like knots, cracks, and gaps [13]. Knots occur in sawn timber where branches were cut from a tree (Figure 3A). Before branches are removed, the tree had previously naturally strengthened the area to withstand the weight of the branch, but the discontinuous fibers made by the cut are now weaknesses. Furthermore, change in grain direction is caused by the occurrence of knots. Grain angle is a key indicator of material strength [39], therefore variations in angle are reflected in variations of mechanical properties and the intended force distribution of a design may not behave predictably. A further concern is the development of cracks. A tree that has just been harvested may not have any cracks, but as a tree loses moisture it shrinks, causing tension cracks between fibers. Timber may crack radially or longitudinally (Figure 3B) due to low tensile strength between fibers, though this can be mitigated by managing how the timber is dried or by incorporating cuts to relieve tension and guide cracking (Figure 3C). Cracking in joints can also change how forces are distributed, making the mechanical behavior unpredictable. This being the case, decreased defects means increased reliability of predictions of joint behavior.

3.2 Moisture sensitivity

Timber is sensitive to moisture and may respond undesirably to both drying and exposure to moisture. A joint is perhaps the most vulnerable part of a timber structure with respect to moisture as the exposed fiber ends and enclosed connections increase probability of shrinking, swelling, biological attack and degradation at a location requiring high mechanical performance.

One risk at a connection interface is that trapped moisture can foster an agreeable environment for bacterial and fungal growth, like that seen in Figure 3D. While timber buildings with adequate maintenance are expected to live past serviceability life, new buildings with inadequate waterproofing have resulted in demolition [35]. The threat of moisture is often understated as many case studies of modern timber buildings are located in northern Europe where the construction practices are thorough, biological attack is geographically less likely, and therefore durability observations may be unintentionally exaggerated [28]. Consequentially, risks of moisture may be downplayed. High risk of biological attack results from high vulnerability of wood and high potential for exposure in the absence of proper management. As a biological material, wood is susceptible to biological attacks including fungi, bacteria, and insects when moisture content exceeds 20%, though many treatments exist to combat degradation and several have additional benefits like fire resistance [25]. Increased rainy periods due to climate change also increase the risk of moisture contact and accelerated deterioration from moisture related threats like biological attacks and decay [33]. In an evaluation of degradation of timber heritage buildings in China, rainwater leakage and biological attack were found to be most critical among connections, particularly in the tenons of beams [40]. Therefore, moisture protection in turn prevents biological attack.

Furthermore, with changes in moisture content a timber connection will also change geometric dimensions. Fibers swell as they absorb moisture and shrink as they dry, which causes dimensional changes that can impact stiffness and transfer of forces at joints. This is because timber is hygroscopic, meaning that the moisture content of the material changes with respect to the moisture in the surrounding environment, therefore timber is often dried to the geometrical size it is expected to have within the moisture content anticipated within the built environment [25]. Despite drying a timber element, there is a risk that construction sites may provide opportunities for exposure during assembly, resulting in geometric variations in swelling and shrinkage which could cause defects that reduce the mechanical properties of the material [25]. Intensification of climate change also implies prolonged rainy periods under which increased moisture exposure can exacerbate cracking or splitting risks resulting from geometric fluctuation [33]. Three typical wood treatments against water absorption include impregnating part of the fibers with a material to block moisture, coating the exposed surfaces, or modifying the cell wall, which can be done either thermally or chemically [25]. More resilient timber might be both seasoned (dried, particularly in a kiln) and treated, which might increase both the cost of the product and the embodied CO_2, where a non-treated timber will be at higher risk of geometric fluctuation and warping.

A further risk of shrinkage is the potential for gaps to form between interconnected members. Tolerances must be made in manufacturing to fit members together with ease, but this may create problems if large allowances result in unwanted gaps. Upon inspecting almost any historical carpentry connection, shrinkage gaps can be found [20]. Key findings of a study on dovetail tenons showed that irregular manufacturing did not greatly affect capacities, though the combination of increased gaps and inaccurately cut interfaces reduced the effectiveness significantly [41]. Contrarily, results of the same study saw that specimens with no gap also exhibited lower capacities [41]. Similarly, in an experimental campaign involving beam to column dovetail mortise tenon connections, gaps were identified as a significant influencer on flexural behavior of the joint, particularly on the sides but less so on the top [42]. As this is such a common problem, Geno *et al.* (2022) proposed an end-notched connection design of reciprocal structures with angles cut to accommodate geometric fluctuation [31]. Anticipation of gaps can therefore reduce performance loss.

In addition to these drawbacks, moisture can also encourage degradation. In evaluations of Chinese mortise and tenon beam connections, specimens with intentional moisture degradation were experimentally evaluated, and while the connections still performed semi rigidly, degradation of the male part of the connection resulted in a clear decrease in rigidity [39]. In a study to evaluate the effects of weathering, wetting and drying cycles designed to simulate intensified rain and flooding climate events were performed on wood-wood half-lap joints and steel reinforced joints to evaluate pull out capacity and found that the climate-damaged specimens performed 30% poorer in capacity due to the reduction in strength, stiffness, and energy dissipation [33]. Furthermore, in an experiment involving retrofitting step joints with moisture damage, moisture content was not a relevant factor in capacity, however, slippage occurred in some retrofitting mechanisms [43]. Moisture exposure should therefore be avoided and adequate maintenance and monitoring are needed to retain mechanical attributes of joints.

3.3 Connection degradation and assessment

Since natural degradation is unavoidable, evaluation of connections as they age is of the utmost importance. In historical timber structures, failures usually occur in the joints [20]. Furthermore, aging joints experience increased unpredictability of connection behavior from defects [17]. If unmanaged, a connection might degrade such that it cannot transfer the building loads. Unfortunately, there are difficulties in evaluating timber joint degradation since the interlocking of elements makes some parts of the connection inaccessible. This has prompted increasing interest in non-invasive techniques to evaluate connections. To address the difficulties in evaluating structural integrity of aged joints, an experimental campaign to validate material testing methods and propose anisotropic failure criterion representative of tenoned step joint behavior was conducted by Feio *et al.* (2014), which employed semi and non-destructive methods of materials testing including ultrasonic testing, micro-drilling and surface penetration for evaluations of the strength properties using both new and historical timber [44]. Though the method remains unrefined, non-destructive connection evaluation methods are necessary to asses aging connections.

3.4 Fire vulnerability

Fire strategies in timber buildings are commonly viewed as inadequate, particularly with timber connections. There is inconsistent advice regarding exposed timber sections, charring areas, wood glue fire resistance, connection exposure, and penetration vulnerability, making it difficult for suppliers to make products that accommodate various building codes [35]. Interestingly, timber elements with steel connectors may perform poorly in fire as the steel heats the timber internally and degrades the mechanical properties [25]. Brandon *et al.* (2015) point out that in heavy timber frames with metal fasteners the fire performance might be greatly reduced as the metal conducts heat into a connection, while heavy timber elements are otherwise considered to perform well in fire [45]. Likewise, an all-wood doweled connection used in the Heartwood Building, the first type IV-C building to be fire rated for 2 hours in North America, is now being considered for various projects due to the fire rating [46]. However, a common solution to enhance fire performance is encapsulation, though recent research shows that encapsulation is expensive, ineffective, and aesthetically unappealing [45]. Char layer prediction in wood sections presents further shortcomings in capabilities with limited data. Špilák *et al.* (2022) conducted experiments to observe the char layering of spruce square and round sections to compare with existing models and a proposed finite element analysis (FEA) [47]. While FEA shows potential in replacing experimental evaluations, there is limited availability of data and models cannot be confidently validated [47]. There is speculation that heavy timber carpentry might perform well as they are interlocked without protrusions to ignite easily, but further research is needed to provide confident advice regarding wood-wood connection resistance.

3.5 Lack of design guidance

One globally reoccurring theme is the lack of design recommendations for detailing in building codes and standards that have hindered the use of carpentry techniques in modern building [11,16,47–50]**.** Even in Europe where many heritage buildings exist and the building code is quite advanced, design guidelines for wood-wood carpentry connections are limited. With the exception of a select few historical joints, many carpentry techniques are unaddressed in Eurocode 5 though national annexes of some countries individually address guidelines for joints common to local typologies despite that European carpentry joints are often similar between countries [48]. Therefore, widespread use of wood-wood carpentry is mostly limited to smaller buildings, though low-rise timber framing with metal fasteners is common and mass timber buildings with engineered timber and metal connections are gaining popularity.

This inability to refine design guidelines for different connections is widely due to the uncertainty in connection behavior caused by both material orthotropy and the unique mechanical material properties of each structural member. As previously stated, uncertainty of material attributes such as (i) variable strength, (ii) inconsistent density, (iii) excessive defects like knots and cross grain, (iv) high content of juvenile wood (which lowers stability and increases risk of drying defects), and (v) small log diameter, cause variation in material performance [14]. Historically, these variations would have been considered by master carpenters, but mass-produced timber does not account for this level of design and evaluation skills at this level are rare. Further to this, a complimentary effect of a concrete-driven industry is that timber engineers are uncommon and opportunities to learn this skill are limited across both universities and industry projects, regardless of increasing demand [35]. This means that few people are qualified to design with timber outside of building code guidelines, thus most new timber buildings use engineered timber and steel fasteners since building codes are more comprehensive for engineered timber and steel, as opposed to sawn or whole timber and carpentry connections. Overall, the topic of structures with wood-wood connections draws more questions than answers, despite the survival of ancient multistory structures over centuries being a testament to the potential longevity of timber.

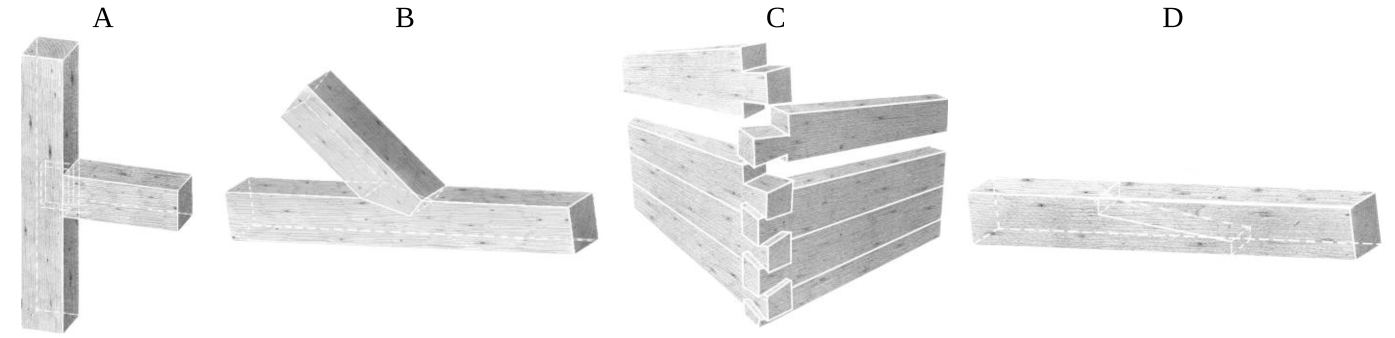


Figure 4: Examples of four joints for each common joint type including (A) Mortise tenon joint (Classic western timber framing mortise tenon), (B) Notched joint (Western roof truss single step joint), (C) Lap joint (Dovetailed corner joint), and (D) Scarf Joint (Nibbed scarf joint of a purlin). Drawings by Eva Kévers-Leclercq.

4 Literature review of wood-wood carpentry connections in heritage structures

There are countless ways to connect structural elements in buildings, bridges, structures, furniture and infrastructure using wood that has been shaped to interlock with adjacent elements. Though wood-wood connections often have similarities, several hundreds of carpentry joint types can be seen in historical structures worldwide [49]. This overview focuses on the connections seen in academic literature, though many connections exist outside of the spotlight of ongoing research. There are four main categories of common historic carpentry joints (see Figure 4) [22,24]. Studies on respective joint types are discussed herein with respect to the purpose and structural elements it joins. While these are general categories, studies involving carpentry frames and bracket systems (Figure 8) consisting of many joints have also been included in this study. This section explores the state of research in historical carpentry connections, which is most abundant in North American, European, and Asian carpentry. Following this literature review in section 4, methods employed during the research are discussed in section 5 Experimental, analytical, and numerical methods in carpentry connection evaluation, as summarized in Table 1 of Section 5 where the table key can be found in Table 2.

4.1 An overview of common carpentry joints

Mortise and tenon: A mortise and tenon is a very basic carpentry joint that is seen around the world and throughout time. Though reviews across Europe yield archaeological findings of timber frame constructions dating back to the Roman Empire, it was not until the 15th and 16th centuries that mortise and tenon style construction saw a turning point in technology [3]. Mortise and tenon connections are common in post to beam and roof structure connections, among others. In fact, mortise and tenon joints are one of the most common connections in historic carpentry, and are characterized by interlocking joint that connects two or more elements in a “L” or “T” shape configuration [44]. This joint acts like a key in and hole where the key piece (tenon) fits into the hole (mortise). A mortise and tenon might be a square, rectangular or round in shape, though it might also be tapered. For example, a goose neck, dovetail or partial dovetail shape increases interlocking since a simple mortise and tenon might be structurally inadequate to transfer tensile forces or resist rotation. Due to the widespread use of a mortise and tenon in ancient connection design, a significant amount of current research focuses on evaluating variations of this joint.

Notched joints: Notched joints are connections where members are joined such that a bottom member is fitted with a seat for an upper member to rest. These connections are particularly common in Western-style roof structures, and often involve a compressive member with a tenon, potentially at an angle, inserted into a tensile member with a notch within the foot truss system, commonly a kingpost system [24]. The most commonly used notch joint is a single step joint [48]. This joint might be seen where an angled truss member in compression meets a tie beam in tension or where a roof element connects to the envelope of a building. End-notched beams can also be seen where the notches situate a simply-supported or double-cantilevered beam. Dimensions used in historical notch design are often governed by rules of ratios from manuals written in the 19th century or earlier which explain the relationship between notch depth and contact surface area where increased notch depth increases embedment and friction area but weakens the section, and a shallow notch has decreased embedment and friction but increased member thickness and strength [17]. Similar concepts are still considered today, as found in Jockwer, Steiger, and Frangi’s (2014) state-of-the-art review of design of notched timber beams [50]. This relationship is still of great importance in domestic structures incorporating traditional designs.

Lap joints: Lap Joints are comprised of a top member and a bottom member where the two members are either connected by a pin, or part of the material of one member or both is removed to form complimentary male and female notches to connect the two parts. These members are often fit perpendicularly such that once assembled, the two parts together are the same depth as the deepest member. Notch patterns are commonly a square, a square with a step, or a triangle referred to as a 'dovetail' [24]. Increasing the length of a member can also be accomplished with a half-lap joint, which can be used to join two members in a right angle at the ends of a horizontal and vertical member, but it can also join the ends of two members that are meeting horizontally at a right-angled corner, called a corner halving joint [51]. For example, this might be seen in log buildings, or retaining wall structures. Most research from the end of the 20th century has focused on ultimate loads and lap joints [20].

Scarf and splice joints: Scarf and splice joints, which can have similarities with lap joints, are made to join two members at the ends, therefore making a longer beam, purlin, or sometimes column type elements. A scarf (or splice) joint joins two horizontal members at an interface at the ends by cutting the ends to create an area of maximized contact surface to increase length, particularly in roof members like purlins or roof members in tension [51]. These members often consist of a sloping interface and might include either pins or keys to increase friction between the members in tension, but the interface might also be a simple step with pins or a dovetail [24]. Early scarf joints were also used in industrial structures. Excavations have found that nailed scarf joints were used in ships during the 1st century in the Roman Empire, and the keel of cargo ships recovered from the 3rd century showed complex scarf joints without metal [51]. More information about scarf and splice joints can be found in a recent state of the art review [49]. Today scarf and splice joints are rarely used to join element ends due to low capacity compared to glued sections.

4.2 Literature review of carpentry connections

This section reviews existing literature and is broken up into four sections to reflect the natural categories of the literature: 4.2.1 Beam to column connections, 4.2.2 Roof Systems, 4.2.3 Elements lengthened with scarf and splice joints and 4.2.4 Lap joints. A detailed summary of the literature can be seen in the Section 5 where Table 2 can be used as a key to Table 1, which lists and classifies all literature used herein, including literature summaries of studies and information assessed in section 5 Experimental, analytical, and numerical methods in carpentry connection evaluation.

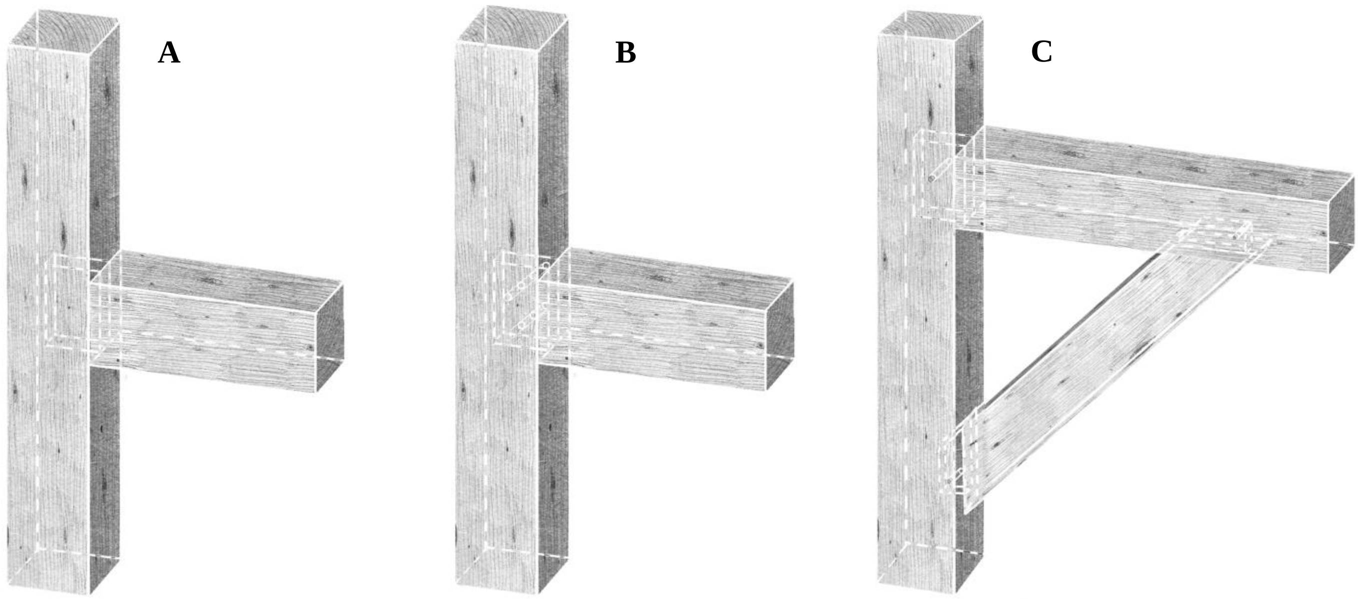


Figure 5: Examples of mortise tenon connections seen in timber framing within Europe, UK, and North America, including (A) Mortise-tenon beam to column connection, (B) Mortise-tenon with pegs though the post and tenon, and (C) Mortise-tenon with shoulder and pegs. Drawings by Eva Kévers-Leclercq.

4.2.1 Beam to column connections

4.2.1.1 Western mortise and tenon connections

Mortise and tenon connections are commonly seen around the world, and particularly in post to beam connections in timber framing. A mortise tenon works well in compression and can work in rotation, but it has negligible tensile capacity. To hold a joint in place, increase stiffness, or increase connection capacity, the intersection of a mortise and tenon might be penetrated with a small hole (or holes) to hold circular or square shaped elements such as keys, pins, pegs or dowels to increase performance. This is particularly common in North America, the UK, and Europe where timber framing is common (Figure 5). With the use of dowel-type elements, a connection is less likely to pull-out and might instead fail from bending or crushing of the dowels, or by crushing or tear-out near holes, or splitting where holes are close to an edge. In an exhaustive evaluation from Bulleit *et al.* (1999) on variations of pegged post and beam joints, 60 specimens across 4 historical joint types, including "mortise tenon, mortise and tenon with shoulder, mortise and tenon with shoulder and knee, and fork and tongue", were tested to evaluate discrepancies between new and salvaged timber connections with the output of a framework of evaluating frame structures providing model recommendations concluding that models must account for shear effects, moment-free joints, eccentricity and contact surfaces evaluation [18]. An experimental campaign from Burnett *et al.* (2003) focusing on the end distance of dowels showed that distance from the end did not impact stiffness, however, specimens with pegs closer to the end of the beam exhibited brittle and catastrophic failures [52]. Eckelman and Haviarova (2008) compared the rigidity of mortise and tenon joints with and without oak cross pins, though among eleven cases there was very little difference in rigidity of pinned and unpinned specimens [53]. Shanks and Walker (2009) showed through an experimental campaign alongside analytical evaluation of green and seasoned oak pegs in traditional UK mortise tenon connections that incorporation of high-quality, non-tapered, well-fit pegs with a reduced shear span increases connection strength and stiffness and induce elastoplastic yielding [12]. Shields and Hindman (2015) experimentally evaluated 40 keyed through tenon joint specimens in tension across 5 configurations with consideration for two wood species, two tenon lengths, and either 1 or 2 keys with outcomes found to be dependent on the tenon length, number of keys, and whether the keys were hard wood or soft wood to determine brittleness, capacity, and the presence of one of four failure modes of the connection [54]. Furthermore, in a study involving tropical wood, Amaruddin *et al.* (2013) compared past experimental results and finite element modelling of dowelled mortise and tenon connections to identify the European Yield Model failure mode [55].

4.2.1.2 Seismic performance of connections

The evaluation of traditional frames under dynamic loading is of great interest to heritage preservation in seismic regions. This is a key driver of evaluations of wood-wood connections and timber frames in Asia. Case studies of post-disaster heritage timber structures have shown that traditional timber structures can be resilient in seismicity and connections are a key factor in robustness and energy dissipation [56]. It is due to the semi-rigid and ductile behavior of the joints in addition to this high robustness that many historical structures have exhibited high performance during earthquakes and high wind loads, even as buildings age [24]. A structural case study of the Yingxian Pagoda, the oldest surviving traditional Chinese pagoda, is nearly 1000 years old and 67.31 m high with nine levels, and has survived 40 seismic events, including two magnitude 7 and six magnitude 6-6.9 (Richter scale) events [16]. Xue and Xu (2018) point out that many timber heritage structures survive through intense seismicity due to flexible frames that absorb forces [57]. However, as the weakest part of a structure, timber joints are critical and tend to suffer brittle failures while still mechanically functioning within the elastic region [56]. Šobra and Avez *et al.* (2016) identified the main causes of poor seismic performance in historical timber buildings as (1) reduced mechanical properties due to age, degradation, or biological attack, (2) insufficient maintenance, (3) structural alterations from building function changes, (4) cumulative damage from successive seismic events, (5) excessively heavy roofing, and (6) masonry failures in buildings with mixed material typology [56]. This means that to induce the best performance possible from a wood-wood connection there are many considerations, and ultimately the behavior is difficult to predict within a material already known for mechanical inconsistencies and defects. Due to the high performance of timber frames with risk of failures in joints, experiments in the literature reviewed often used cyclic or quasi-cyclic loading to anticipate the seismic behavior.

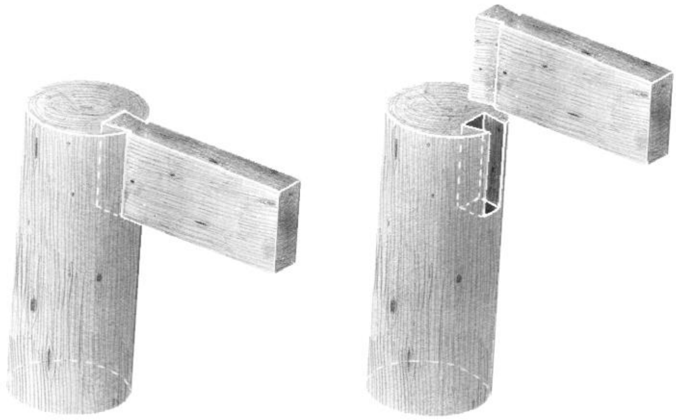


Figure 6: Round column and rectangular beam with top-column dovetail mortise and tenon connection, assembled (left) and disassembled (right). Drawing by Eva Kévers-Leclercq.

Mortise and tenon

In a study of aging traditional Chinese beam-to-column connections with column mortise and beam tenon, King *et al.* (1996) tested connections subjected to artificial mechanical degradation, which was found to be critical to connections during experimentation to determine the semirigid behavior, and which was used to validate analytical methods [40]. Dovetail mortise and tenon post and beam connections, like those seen in Figure 6, seen as early as the Ming dynasty (1368 – 1644) in China, are often considered critical as the joint rotational stiffness and strength dictates the stability of the structure [42]. Chen, Qiu and Lu (2016) combined experimental, theoretical, and FEM methods to evaluate two typical Qing Dynasty (1644 – 1912) joints with respect to dovetail geometries and flexural relationships with (i) moment to rotation and (ii) critical pull-out to rotation, and upon observations of crushing failures but increased capacity of dovetails versus straight tenons, an analytical method was proposed for practical use [42]. A parametric evaluation of Korean dovetail connections by Jeong *et al. (*2013a) considered different angles and dimensions using stochastic finite element methods (SFEM) which were validated through an experimental campaign to propose a numerical method of predicting connection capacity based on varying geometric design parameters [58]. Xie *et al.* (2021) investigated contact characteristics of a mortise and tenon using analytical methods and finite element modelling considering elastoplastic contact under reverse cyclic loading where contact simulations were successful but may have limitations in applications to aged wood [59]. Mortise tenon joints are very common throughout timber framing in China, Korea, Taiwan, and Japan.

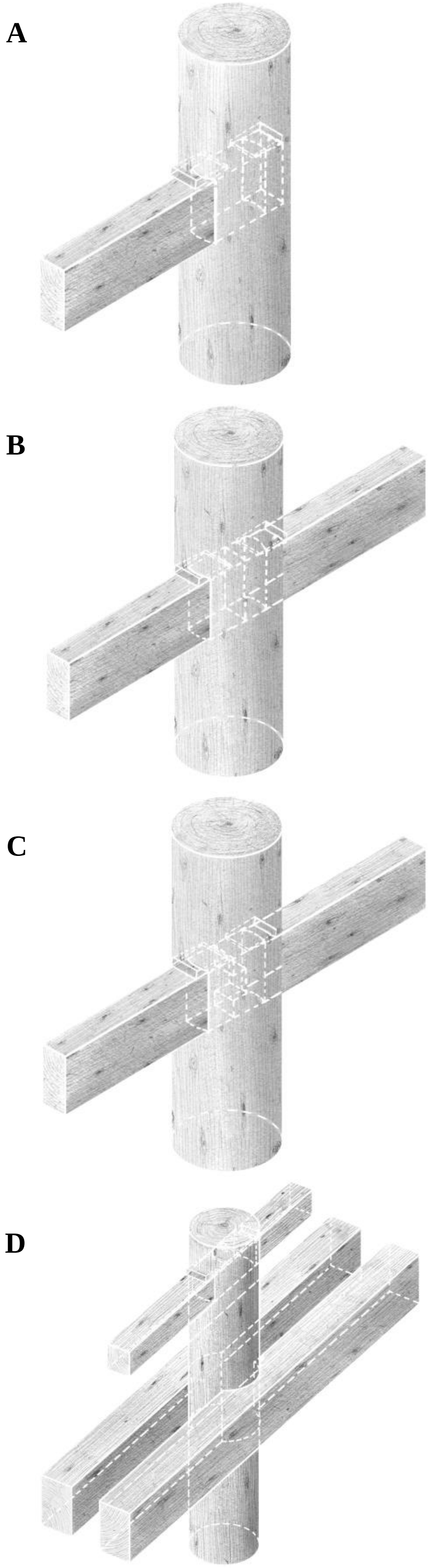


Figure 7: Examples of (A) Continuous *Nuki* joint (end beam shown here), (B) Simple butted *Nuki* joint, (C) Dovetail butted *Nuki* joint, (D) *Nageshi* beams (bottom beams) with continuous *Nuki* joint (top beam). Drawings by Eva Kévers-Leclercq.

Through and side tenon connections

Though timber framing often uses mortise and tenon style connections, some connections might instead have alternative configurations. For example, Japanese and Korean frames often utilize *Nuki* joints (Figure 7A, B, C), where *Nuki* beams penetrate columns, or *Nageshi* beams, which sit outside the columns (see Figure 7D). Modern research on Japanese heritage carpentry often focuses on *Nuki* joints. Though these joints are Japanese in origin, *Nuki* joints can be found in Japan, Taiwan, and China and perform well in moment resistance and ductility due to the beam embedment and friction [60]. These connections, often seen in shrines, temples, and houses, are characterized by a *Nuki* beam member that penetrates a column and is often locked in place with a tightly fit wedge called a called *Kusabi wedge* [60,61]. A *Nuki* joint beam can penetrate a post with either a continuous member, or two members butted together within the connection, called a butted *Nuki* joint. Kato and Komatsu (2000) observed the role of wedges in Nuki joints as well as embedment in experimental research that focuses on the moment rotation relationship of a beam to column joint that considers the use of *Keyaki*, *Moabi* and *Hinoki* wedges, which found the most favorable results relating to low wedge angle, deep embedment of wedge, and the use of hardwood materials [62]. In a two-part study of *Nuki* joints, initial condition stresses and stresses induced by test conditions on the beam and column of a *Nuki* joint with wedges of different sizes were observed. Guan, Kitamori and Komatsu (2008a) focus on initial conditions of the distribution of surface stress in critical areas of experimental elements inside joints when oversized wedges are introduced into the joint assembly to create improved stiffness and load carrying conditions, as well as model accuracy, as it was found that interlocking increased with wedge size [61]. It should be noted that internal surfaces of joints are of great interest to researchers, however, limitations in analytical and experimental methods often prevent researchers from gaining meaningful knowledge about these areas, which makes this study unique in its collection of data and replication in 3D finite element modelling. In a complimentary study, Guan, Kitamori and Komatsu (2008b) evaluate the stress states of the same *Nuki* joints with increasing wedge sizes of hard and soft wood in raking with the outcome of a reasonable parametrized numerical model to replicate a joint under raking in FE models while the authors note that increased wedge size increases stiffness and initial stress state does not result in early failure, but oversizing a wedge even 2-3 mm decreases connection capacity [63]. Chang and Hsu (2007) tested 21 historical Japanese butted *Nuki* joints for comparison with previously obtained results from continuous *Nuki* joint testing to show through experimental and analytical methods the typical failure modes in relation to friction, embedment, and compression of butted joints, which have drastically reduced stiffness and moment resistance in comparison with continuous joints [60].

Some further studies exist on the topic of Asian post and beam connections. To better understand the Taiwanese building stock, 87 historic buildings were surveyed by Chang, Hsu and Komatsu (2006) to identify the three main types of Taiwanese Nuki joints, which were experimentally evaluated alongside Japanese Nuki joints then modelled to predict decreased rotational performance of Taiwanese Nuki joints with performance reducing gaps due to the absence of wedges, resulting in a practical theoretical model for rotation stiffness [64]. Experimental research by Shiratori, Komatsu and Leijten (2008) focused on retrofitting tightening bolts into Japanese Nageshi beams to potentially reduce gaps present in joints and therefore increase stiffness and seismic performance [14]. A *Nageshi* beam is like a *Nuki* beam, however, this beam partially penetrates the side of the post and acts as structural bracing that is also held in place by a *Kusabi* wedge. Jeong *et al.* (2013b) experimentally evaluated the potential geometrical configurations of *Sagae* joints, a Korean connection where a square column with 4 square tenons seats two beams, and the results were used to validate FE models of moment carrying capacity to illustrate the impact of various geometries among which an optimal geometry was identified and a numerical model was proposed to predict failure criteria [65]. To evaluate the relationship between geometric variation and moment carrying capacity, Pang *et al. (2011)* undertook an experiment incorporating geometrical variations of a traditional Korean post and beam joint characterized by beams over columns where beams were either dovetail mortise tenon joints or a H shape that is seated into a column shoulder that often involves several members intersecting at a joint in three directions [66]. It was found that the shoulder can reinforce the mortise and tenon, and is therefore an important factor in moment carrying capacity and failure mode [66].

Frames

Seo *et al.* (1999) applied static lateral and shaker table cyclic loading to full-scale Korean heritage home frames with typical dovetail mortise tenon connections, where it was found that ultimate load capacity was dependent on various joint attributes, secant stiffness reduced with increased displacement, and load direction greatly impacted the viscous damping ratio [67]. To better understand the seismic response of traditional structures in southern China, Chun, Yue, and Pan (2011) evaluated four typical mortise tenon joint types including *Mantou*, *Ban*, *Shizihutou*, and *Yanwei* in half-scale frames tested in in-plane cyclic loading, where failures largely occurred by tenon breakage in 3 cases and pull out in the remaining case [68]. To compare seismic performance of carpentry connected frame with various panel infill scenarios, Xue and Xu (2018) evaluated scaled Chinese traditional buildings with mortise tenon connections and with and without frame panels in shaker table tests [57]. Similarly, Yang *et al.* (2021) tested traditional Chinese frames with mortise tenon joints with and without infill under full-scale, in-plane, quasi-static loading to evaluate stiffness, strength, energy dissipation and failure modes [69]. Likewise, Huang *et al.* (2018) observed four full-scale tests of traditional Chinese *Chuan-Dou* system structures with mortise tenon (dovetail and through) beam to post connections in low-cycle reversed cyclic loading where panels were considered in some cases [70]. Finally, Crayssac *et al.* (2018) built three traditional Chinese beam-column frames with mortise tenon connections and three different panel infill cases for evaluation under cyclic loading and found that mortise tenon frames with panels exhibited non-linear load displacement, showed good ductility and endured damage well, especially among frames with panel infill [71]. All experimental campaigns named involve frames made only with carpentry connections.

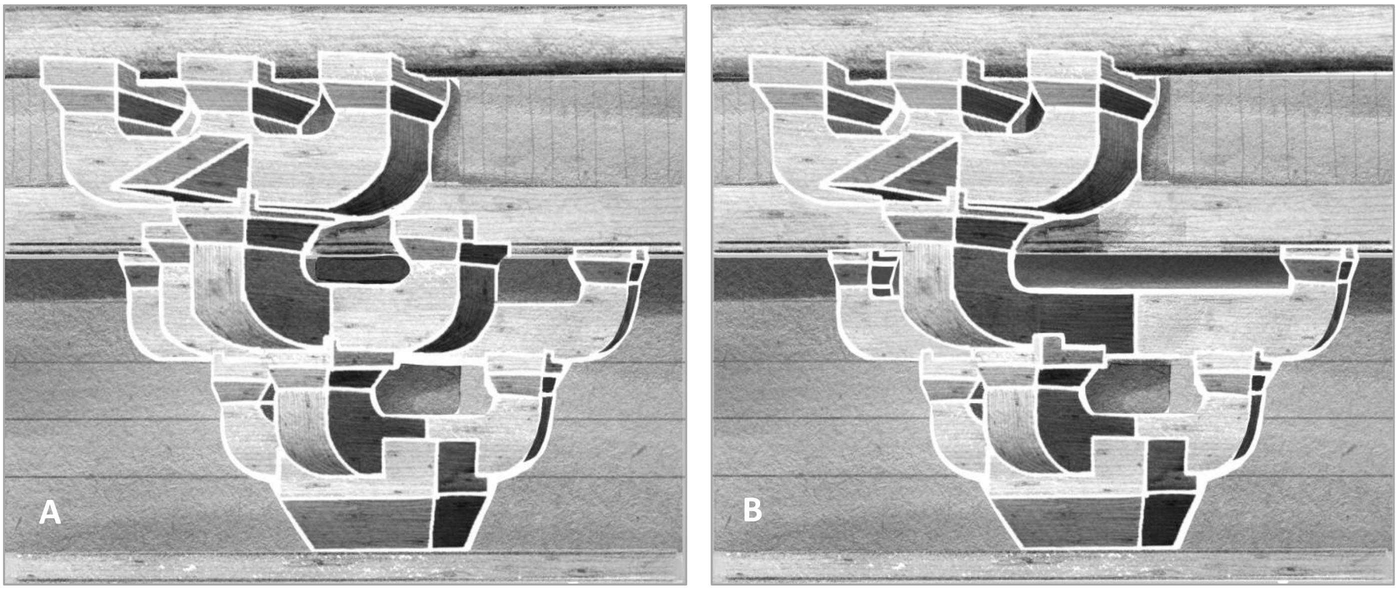


Figure 8: Example of Song Dynasty (A) Jixin and (B) Touxin bracket details from the ancient Chinese building code, Yingzao Fashi. Details adapted from [9] and drawn by Eva Kévers-Leclercq.

Bracket systems

In the majority of Chinese timber heritage buildings, the connection system used is as prescribed in the ancient building standards, the *Yingzao Fashi*. The brackets that made up the connection system that works as a joint is known as a *Puzou* (Figure 8), and is made up of several members but the two main parts are the *Dou*, a block, and the *gong*, the arm, and the system is called a *Dou-gong* joint or bracket [9]. These traditional bracket systems date back to 770 BC [3]. *Dou-gong* joints transfer vertical gravity loads and work together in to resist horizontal dynamic loads [72]. Specifically, the *Dou-gong* system is intended to convert bending forces from beam elements into vertical forces into columns, where the *Dou* is a wooden block that supports the bracket and the *gong* is a short cross-arm that sits with an *ang* as the lever arm in which both arms reduce the beam element span [16]. These elements of traditional Chinese buildings may contribute to good seismic response of heritage buildings like pagodas by providing structural flexibility when natural period frequencies occur, exhibiting high damping capability from joint friction under movement, and by utilizing relatively short columns and lateral compression elements [16]. Various formations of *dou-gong* joints exist to meet different structural needs where beams and columns connect. There are four *Dou* types and five *gong* types, and therefore many combinations of *Dou-gong* brackets systems exist [16]. A humble building might only have four types with few arms, however, some structures are very complex, like the Timber Pagoda of the Liao period, which has 56 different *Dou-gong* bracket formations, each with several arms per bracket [9].

Testing *Dou-gong* systems remains a popular topic due to the complexity of the connection. D'Ayala and Tsai (2008) tested "*Dou-gon*" (*Dou-gong*) brackets for seismic response of global models of Taiwanese *Deih-Dou (Dei-Dou)* timber structures, of which these brackets are the main connections, by applying case study findings after a large seismic event to model member stiffness, ultimate capacity, and seismic response of historic buildings [73]. Wu, Song and Li (2018) experimentally evaluated single and double configurations of *Dou-gong* connections with offset column load transfers like those seen in traditional pagodas to assess nonuniform compressive loading and dynamic response with eccentricity and found that the eccentricity decreased performance and exhibited unusual failures like overturning, but double *Dou-gong* connections and increased vertical load increased performance [72]. Zhang *et al.* (2020) modelled a frame that considered column sliding, mortise tenon lintels and energy-isolated *Dou-gong* brackets which was validated through observations about seismic behavior of individual elements working in the frame system [74] which compared results with 4-pillar frame Dou-gong shake table test results from Zhang *et al.* (2011) [75]. Xie *et al.* (2020) evaluated five forms of *Dou-gong* brackets in experimental reverse cyclic loading to compare seismic response of brackets with varying features [76].

As the Buddhist building style spread throughout Asia, similar connections consequentially developed in Japan and Korea [72]. Suzuki *et al.* (2001) conducted shake table tests on a full-scale frame of a traditional Japanese structure with Japanese *To-Kyou* brackets and *Nuki* beams and found that frames exhibited high damping and energy dissipation, bracket rocking was sufficient to restore structure equilibrium, and tie beams connections were found to be critical [77]. Katagihara (2001) analytically evaluated the shake table test data carried out by Suzuki *et al.* (2001) with particular interest in evaluation of varying structure types with *To-Kyou* brackets and *Nuki* beams [78].

Though less elaborate than far eastern brackets, traditional Tibetan beam and column systems are made of beam system secured with dowels over a block above each column. An experimental campaign by Guo *et al.* (2021) was prompted by excessive seismicity and tourism in heritage buildings, and moment rotation of six specimen configurations were evaluated [2]. All specimens failed in compression perpendicular to the grain within the Ludou, however, greater length ratio of Gongmu to beams resulted in greater moment capacity and dowels spaced further from columns resulted in increased stiffness, therefore two moment rotation types were identified and simplified trilinear models were presented [2].

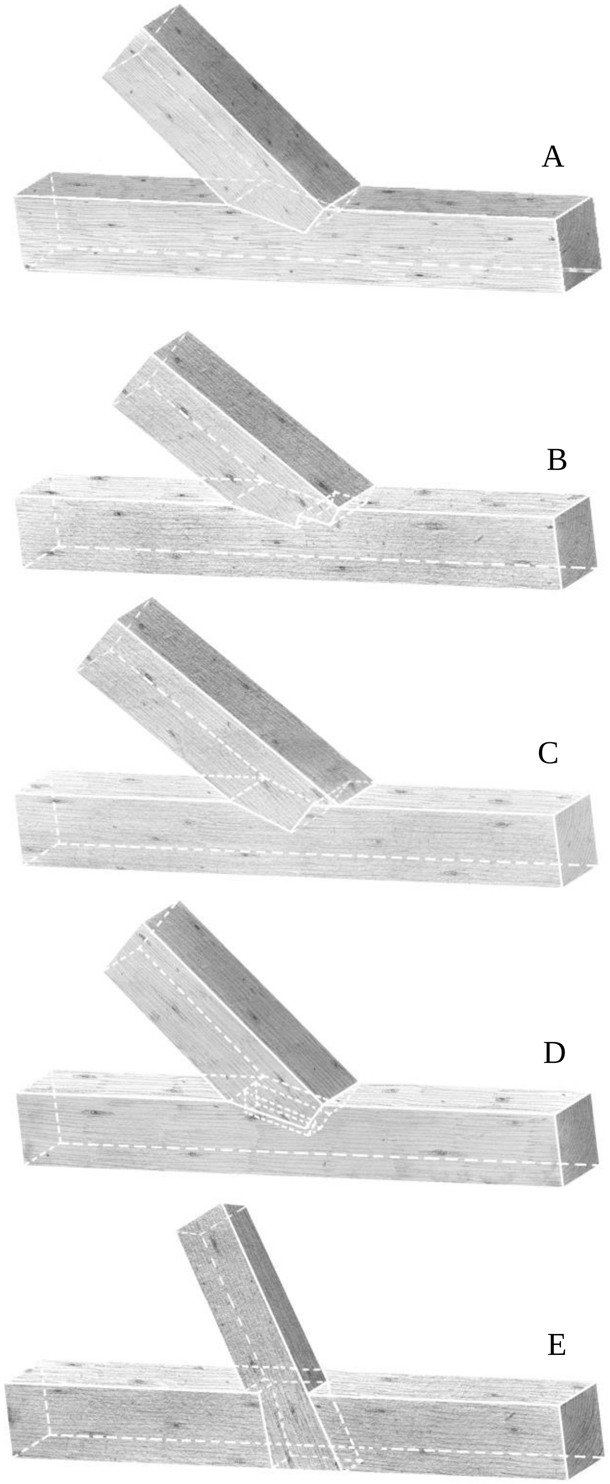


Figure 9: Examples of common roof truss connections, including (A) Single step joint, (B) Double step joint, (C) Reverse step joint, (D) Tapered-tenon step joint, and (E) Half dovetail mortise-tenon joint. Drawing by Eva Kévers-Leclercq.

4.2.2 Roof Systems

4.2.2.1 Western truss roof systems

Particularly throughout Europe, many existing low-rise buildings are heritage structures with carpentry connections, and therefore many studies focus on the various connections that might exist in a traditional roof structure. Heritage conservation and retrofitting have resulted in a wide range of research related to European carpentry, much of which involves traditional roofs. In an approach to global structural analysis, Holzer and Kock (2009) conducted a field study and finite element structural evaluation of 3 case studies of German church roof systems based on structural drawings from the 16th, 17th, and 18th centuries, of which the author evaluates systems in static and dynamic (wind) loading and explains the structural evolutions and systems in detail [20]. Though some studies include truss analysis, most literature focuses on experimentation, particularly of step joints. Step joints are a common connection found at the end roof rafter and into the middle of a roof beam, tie beam, or post in a truss system. Many variations of step joints exist, for example, the joint may have steps in different locations, more than one step, or a tenon. The most common notched joints evaluated in European roof structures are step joints (Figure 9), including single (A), double (B), reverse (C), and tapered tenon (D) step joints. A step joint relies on friction and is meant to resist compression in the rafter and shearing through the beam, but step joints are not designed to resist tension. Step joints are shown to have geometrical rules of design to govern failures modes, which are of great interest since when a crushing failure occurs it is not a critical failure, unlike when shear failures occur [17,79]. Many studies focus on this point of interest.

Several studies focus on single step joints. To fill a reoccurring knowledge gap in European building codes, Verbist *et al.* (2017) undertook an experimental campaign that focused on brittle shear crack failures in horizontal members and compressive crushing with plastic behavior at the contact interface of single step joints to evaluate design equations [79]. With a further focus on heritage restoration, an analytical campaign from Branco, Verbist and Descamps (2018) assessed crushing, shear cracking, tensile cracking, and rolling shear failures when loading a diagonal element of less than 50 degrees from the horizontal element of single step joints, double step joints, and step joints with mortise tenon to anticipate failure modes and capacities, as this is not described in EU building codes [80]. Further analytical evaluations by Verbist, Branco and Descamps (2020) include finite element models and cohesive zone modelling with consideration for hammock shape shear stress distribution (HSSSD) with the proposition of a reducer coefficient $k_{v,red}$ based on relevant failure parameters, like joint geometry and angle, to better understand the non-linear stress distribution and ultimate shear cracking failure parallel to the grain of the tie beam in a single step joint to better prevent brittle failures [81]. Retrofitting of single step joints is also common. Šobra, de Rijk, *et al.*, (2016) evaluated glue laminate single notch step joints under compression in a series of experiments then retested the specimens after being retrofit with self-tapping screws [21]. Parisi *et al.* (2008) compared single step joints rotational stiffness to seismic retrofitting of a bolted connection and a binding strip retrofit connection [82]. In a variation of detailing, Parisi and Cordié (2010) experimentally and numerically evaluated double-step joints with typical traditional detailing in compression and cyclically in rotation to better understand limitations in building codes and complex material modelling [17]. For the purpose of validating finite element models, Villar-García *et al.* (2018) experimentally evaluated two glue laminate step joints types, front notch joints and reverse step joints, but found that in all tests plastic deformation from crushing never occurred and all specimens failed with shear cracks on the tie end at ring interfaces and parallel to the grain [83].

It is also common to see step joints with a mortise in the rafter tie beam and tenon in the foot of the diagonal brace. Experimental evaluations of tapered tenon step joints from Koch and Seim (2012) display three failure modes and causational variables, which were evaluated individually with reasonable agreement to the German building code, though concerns remained with respect to accounting for friction [84]. The same 3 failure modes were observed again by Koch, Eisenhut and Seim (2013) where tapered tenon joints were then evaluated with analytical and numerical methods providing analytical validation, though this study focused on separation of lamella at shearing areas [12]. During validation of analytical and finite element models of step joints with tenons, Kekeliak, Gocál and Vičan (2015) considered experimental results of contact interfaces from the thesis of Feio (2006) [85], including sensitivity of joint stiffness on contact interfaces, load to grain angle, and hardness modulus [86]. Similarly, some step joints with and without mortise tenon have been compared. Palma *et al*. (2012) undertook an experimental campaign applying compression and rotation to damaged and undamaged, front-notched step joints both with and without mortise and tenon to evaluate potential repair techniques of historical structure with the aim of producing design guidelines related to rotational load carrying capacity, stiffness, static ductility, and failure mode [43].

Some roof connections commonly employ a mortise and tenon with full or half dovetail to improve interlocking with tapering (See Figure 9E). Unlike step joints, these connections can resist some tensile loading. Dovetail joints are known for their distinctive fan shape that is used at the end of timber members to tie them into an adjacent member, such as in a floor or ceiling joists, though half dovetails are commonly seen as well as they are faster and easier to make [51]. One seismically motivated experimental evaluation by Šobra, Avez, *et al.* (2016) involved dovetail rafter connections both with and without dowels, which when tested under tension and compression found that analytical models accurately reflected connection capacities of joints without dowels but drastically underestimated capacities of joints with dowels [56]. In a similar experimental campaign involving quasi-static pull-out tests, Kunecký *et al.* (2016) focused on verifying specific parameters of historic dovetail joints with and without dowels for the purpose of validating analytical and numerical models for connections in tension and compression, which identified failure modes of the connection and found that there are limitations to Eurocode 5 when estimating capacities of connections with dowels [41]. Furthermore, in transferring the findings of behaviors of pegged dovetail joints in roof truss elements to a global model, it was concluded that precise rotational stiffness of dovetail joints does not need to be considered since the moment transfer is negligible and joint stiffness of the model is sufficient [41].

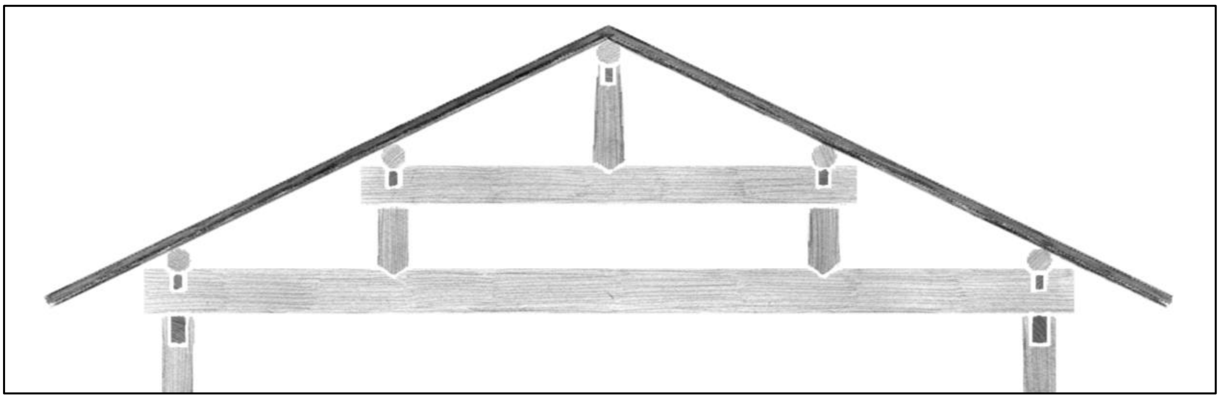
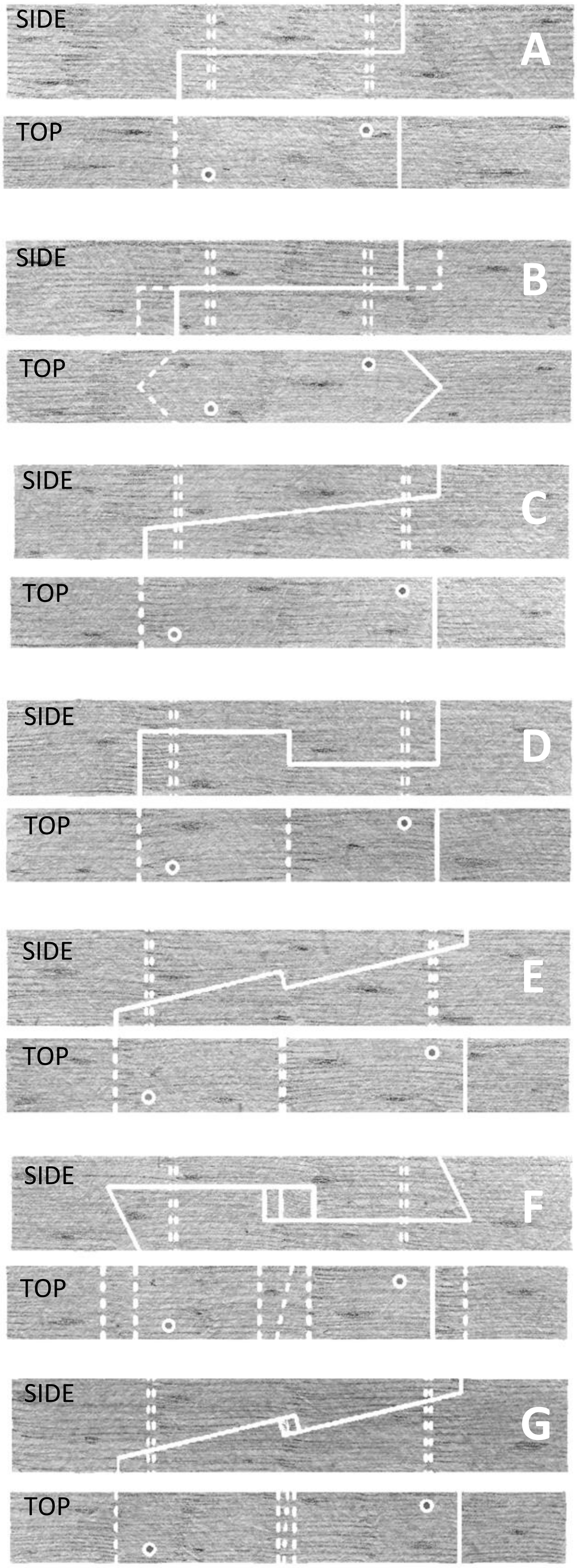


Figure 10: Classic Chinese post and beam roof structure with *Zhi* and *Mantou* joints. Similar roof designs can be found in dwellings and bridges alike. Drawing adapted by Eva Kévers-Leclercq based on [87].

4.2.2.2 Eastern post and beam roof systems

Simple Chinese heritage buildings also often feature roof details found in the ancient Chinese building codes (Figure 10), including a *Mantou* joint which connects columns to the beam at the base of the roof and *Zhi* joints between the bottom beam and a second beam above it. In an experiment with two main historical Chinese post-and-lintel style roof structure connections including *Mantou* mortise-tenon joints and *Zhi* mortise-tenon joints (short column), Chun *et al.* (2017) compared experimental results with FE models to evaluate failure modes, seismic response, and connection rigidity [87]. Often in these roof systems, two beams are seen stacked vertically to create a stitching beam with increased depth to improve beam performance. Stitching small top and large bottom or large top and small bottom beams increase bending capacity and are common in traditional Chinese buildings, particularly heritage buildings from the *Tang* Dynasty and *Qing* Dynasty (618 AD – 1912 AD) [88]. Chun, Balen and Pan (2015) experimentally and analytically evaluated traditional Chinese stitching beams according to ancient *Yingzao Fashi* (Building Standards) and the later *Yingzao Fayuan* (Basic Rules for Building) to better understand failure modes, bending capacity, mid-section strain distribution, and deflection of beams with and without knots on the tensile sides of beams [88]. Roof systems that are characterized with posts and beams make them noticeably distinct from Western-style roofs, which employ truss systems. One similar system recently gaining interest is historic *langqiao* (corridor bridges) in China, which employ the same craftmanship techniques as in historic buildings like houses and temples, like *Chuan-dou* frames, within the enclosed structures above the bridges, of which many case studies have stood throughout centuries in the Chinese provinces and are now making a comeback [19]. In a global case study review of Chinese corridor bridges, Knapp, Miller and Liu (2020) explore remains of rural bridges and the recovery and restorations efforts for cultural heritage purposes [19]. Among low-rise buildings and corridor bridges alike, post and beam style roof systems (eg: *Chuan-dou* or *Chan-dou*) are common.



684 Figure 11: Common scarf and splice joints where side view (top) and top view (bottom) details are seen, including (A) Simple splice joint, (B) Nibbed splice joint, (C) Nibbed scarf joint, (D)Tabled splice joint, (E) Stop-splayed scarf joint, (F) Keyed-hooked joint, (G) Stop-splayed and tabled scarf joint [49]. Drawings adapted by Eva Kévers-Leclercq based on geometric diagrams by Karolak et al. [49].

4.2.3 Elements lengthened with scarf and splice joints

The most common connections chosen to increase the dimensions of a structural element are scarf and splice joints (Figure 11). In many cases, scarf and splice joints are used at the ends of flexural and tension members to increase length, though they might also be used to increase other dimensions of a member [49]. A recent state-of-the-art review by Karolak *et al.* (2020) discusses various types of scarf and splice joints (Figure 11) and the overview includes past study details on experimental and analytical methods, and key findings or capacities [49]. To avoid redundancy, the studies contained in this state-of-the-art will not be covered in length in this literature review, however, some studies listed in Karolak *et al.* (2020) that contain analytical evaluations are included in Table 1. Following this state of the art, the authors then carried out an experimental and finite element numerical investigation on connections considering steel bolts for replacement joints used in repair or retrofitting [89]. After the completion of the Karolak *et al (2020)* review, Ceraldi *et al.* (2021) explained the relevance of the use of pegs in stop-splayed scarf joints and material choice of pegs in an experimental campaign that showed increased rigidity and increased yield and post-elastic behavior in joints with timber pegs, and significantly increased strength with steel pegs at the cost of brittle failure as the steel did not yield [22]. Further to this, it is important to note that splice and scarf joints are also common in Japanese carpentry. Uyko *et al.* (2008) undertook a study of strain distribution in *Kama Tsugi* (a half-lapped gooseneck joint) and *Okkake Daisen Tsugi* (a doweled wedged tabled joint) under tensile loading using Digital Image Correlation (DIC) and compared results to finite element method calculations [90]. Finally, one study included the rare topic of connections that lengthen columns. Among the techniques employed in Korean carpentry, the Utgulisanji connection is a complex keyed scarf joint claimed to have been found as the strongest wood-to-wood connection [91]. Despite the existence of building code guidelines, the complex geometry creates consequentially complex failure calculations, therefore an experimental campaign by Jeong (2020) focused on tensile capacities compared against three numerical evaluation methods [91].

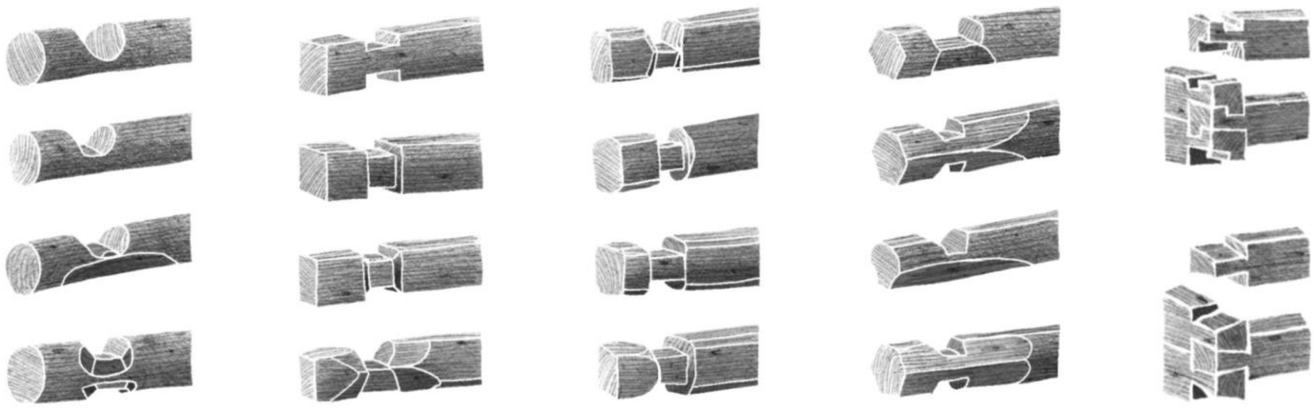


Figure 12: Example of various lap joints found in heritage log structures [94]. Reinterpreted drawing by Eva Kévers-Leclercq based on [94].

4.2.4 Lap joints

Lap joints, which might also be called saddle joints, are very simple in concept and fabrication, and therefore they can be found in very old structures. For example, retaining walls with saddled joints at the corners were used during the Bronze Age since the strength of a wall was required at the wall center and not the intersecting corner [51]. Today, a lap joint might be most recognizable in corner connections of log buildings like those found in Scandinavia and Central Europe, similar to those seen in Figure 12. Though this typology has European origins, it is also common in the Americas. In a case study review of historic log buildings in the United States, Murphy (2004) explored the building stock of log buildings built with traditional Czech carpentry techniques brought by European immigrants [92]. In a comparison between corner joints with saddle notches and dovetails, Kłosowski *et al.* (2018) used a computationally expensive stochastic probabilistic approach to numerical modelling to make assumptions that predict the likelihood of flaws in timber which concluded that saddle joints can endure lower stresses but are more resilient to moisture, and dovetails are more resistant to loading but more susceptible to moisture damage [93]. Lap joints can also be found in Asia. One vastly different study focuses on the *Watari-ago* joint, a ductile horizontal joint common to roof and floor systems which are made up of two beams with notches and are interlocked with friction [7]. Ogawa, Sasaki and Yamasaki (2015) undertook an experimental campaign under cyclic loading which produced key findings of joint resistance existing past joint yielding and a relationship between mechanical behavior and notch geometry, while theoretical methods validated the geometry required to achieve optimal mechanical performance [7].

5 Experimental, analytical, and numerical methods in carpentry connection evaluation

There are numerous methods to evaluate carpentry connections. In hand with experimental evaluations, various analytical and numerical methods exist. The reason for this is that the orthotropic and anisotropic nature of timber paired with individual tree variability like age, density, knots, cracking, or moisture related deformations or defects cause difficulty in creating general models that are suitable for a large range of specimens. The commonality across work in timber carpentry connections is the aim to use experimental data to best validate methods to approximate behavior of a nominated connection given that there is no significant material variability between specimens. These methods often include comparisons between existing theoretical equations, newly proposed theories or equations, new versions of models proposed from existing models, or finite element modelling using software such as Abaqus or Ansys.

Unfortunately, all models have various limitations. Creating a highly accurate model is extremely important for structural connections since timber, in critical circumstances, is an elastic-brittle material and the risk of catastrophic (irreparable) failure is high. While a model might very closely predict the average behavior of a connection with a given species and geometry of joint, specimens with severe defects can fail outside of the deviation accounted for in a factor of safety. Therefore, it must be assumed that industry grading is sufficient to provide quality of timber with adequate material consistency while accepting that most connections are overdesigned to account for unknowns. Consequentially, modelling must make allowances for material unknowns, like (i) strength of a material is often assumed from the species and individual tree mechanical properties vary, (ii) all wood has randomly distributed defects that cannot be accounted for in material models except probabilistically, and (iii) material mechanical properties degrade at varying rates over the lifecycle of the material depending on maintenance and exposure. In most cases, considering these variables is beyond the scope of the model. As a result, simplified models are most common.

In this section evaluation methods occurring in literature are compiled. First, the evolution of various failure criteria is explored and discussed with respect to model applicability. Second, research from the literature is classified by connection type and evaluation methods, with attention to experiment details and whether analytical or numerical methods are applied. Trends in research are highlighted, including topics of interest by region and modern evaluation approaches like finite element modelling. A summary of evaluation methods from the research found in the literature can be found in Table 1 of Section 5.

5.1 A short history of the analytical evaluation methods applicable to timber joints

It can be intimidating to select how to apply failure theories when evaluating wood. The expected failure in a given plane relative to the grain might display brittle fracture or plastic crushing. This is why different failure criterions are appropriate for different connections with different loading. Compared with the ease in which most materials can be evaluated, timber evaluation is a tedious task that is constantly being revised.

Very few material models have been made specifically for timber, and often as new anisotropic and orthotropic material models are developed they are consequentially applied to wood. Some material models are considered to work better for different applications since timber behaves differently in the radial, tangential and longitudinal directions and discontinuous fibers, like those at a joint, behave differently than continuous fibers. Furthermore, there are additional considerations required for forces applied to different directions relative to the grain. Some very old classical models are still used in various applications, though the second half of the 20th century saw developments in orthotropic and anisotropic models applicable to timber.

Wood is often modelled elastically, though it mostly displays brittle failures. As an elastic-brittle material, sudden brittle failures are likely. Rankine and Coulomb-Mohr are two well-known classical models for brittles failures. The Rankine (1857) yield criterion was originally called maximum-normal stress theory and was derived for applications in soil mechanics [94]. This criterion is often used in concrete or masonry but has also been applied to timber for tension [95]. The theory is suitable for brittle materials; however, this is not a common or effective practice as the theory is very simplified and only considers tensile stress as an indicator for failure while timber is extremely complex and has different yield strengths in different directions. A predating theory is the Coulomb criterion (1776) which was first presented 1773 in the "Essay on maximums and minimums of rules to some static problems relating to architecture" [96]. Otto Mohr's (1871) proposed theory, "Contribution to the theory of earth pressure", built on the Coulomb criterion as a simple representation of 3-dimensional material stresses [97]. This combined representation is known as the Mohr-Column theory, and it considers compressive and tensile principle stresses and maximum shear stress and is widely used. Coulomb-Mohr incorporates the commonality that brittle materials often have lower tensile than compressive properties, and therefore the two Mohr circles represent failure boundaries of compressive and tensile failure values that when considered together express a failure envelope. Mohr's condition was a variation of Tresca's yield condition in which maximum shear stress results as a dependency on the mean stress for present failure limits [98]. As the plane stress yield surface did not produce results consistent with materials testing, Modified Mohr has since been created to increase consistency. However, it was Henri Tresca's yield condition (1864), or "maximum shear stress theory", [99] that presented use of the difference of principle stresses to express maximum shear stress before a material experiences plasticity. Tresca implies that hydrostatic stresses do not impact yield as Tresca concludes that when maximum shear stress is equal to the yield value of shear stress in uniaxial testing, this is the yielding point. Von Mises' theory considers deviatoric stresses and ignores hydrostatic stresses, determining that yielding has occurred when maximum distortion energy and yielding are equal [100]. This idea has been refined repeatedly. Though these classical models are still commonly used to model material in timber connections, they have widely fallen out of popularity in favor of more comprehensive orthotropic and anisotropic models.

One of the most widely used failure criterions based on timber material testing is Hankinson's. Hankinson's criterion (1921) has been widely adopted as the classical model for compression and elastic limit at varying respective grain angles. This model also branches from Von Mises' criterion which is extended to consider orthotropic nature of wood by extending to derive an equation for predicting strength at angles respective to the grain [101]. Through an experimental campaign, Hankinson derived a formula for crushing strength of timber at various angles (0°, 3°, 6°, 10°, 15°, 20°, 30°, 45°, 60°, 75°, 90°) with respect to the grain using kiln-dried timber of high quality and experimental results were plotted to a curve of best fit to approximate the average compressive material behavior [39]. Hankinson's is cited as having general accuracy, but it considers only compression and neglects shear strength because it assumes the relationship between two compressive strengths to resolve the shear strength [102]. Despite this, Hankinson's criterion is widely used in research and industry, for example, Eurocode 5 considers Hankinson's criterion for some carpentry connections.

Hencky-Von Mises' criterion (1924), also called the 'maximum distortion strain energy criterion', is a refinement of the original theory. Hencky-Von Mises' considers Hooke's law for elastic deformation to replace the energy deformation of an isotropic cube to find total energy deformation and consequential change of volume and shape with the assumption that a material behaves elastically in a linear proportion until failure [102]. Norris criterion (1962) was later developed to evaluate the strength of orthotropic materials, including samples of Douglas fir, in combined loading based on Hencky-Von Mises' theory, which is often used for orthotropic ductile materials [102]. Today, some German design recommendations (DIN) are based on the Norris criterion which considers shear strength where resistance for combined shear and compression perpendicular to the grain are doubled [48].

To address the shortcomings of common methods of materials evaluations like Hooke's theory of elasticity and Coulomb or Tresca's methods of strain evaluation, Huber (1904) proposed an equation of material evaluation called "Specific Work of Strain as a Measure of Material Effort" with the recognition that further validation was necessary [103,104]. Consequentially, Huber-Mises criterion has been validated extensively for determining the elastic limit of metals under combined stresses with the exception of annealed mild steel, which is more accurately predicted by Coulomb or Tresca as this considers that the crystal planes that form in low ductility steel have a different structure in each plane and therefore are anisotropic as the different directional configurations results in different directional material properties [105]. Many adaptations to Huber exist, but for applications in wood, Koczan and Kozakiewicz looked a generalized Huber criterion for anisotropic, fibrous materials, which is similar to that of Norris as the criterion obtained incorporates special case conditions of the Tsai-Wu criterion for composite materials [106][106]. These adaptations to Huber criterion are examples of criterion used in anisotropic or orthotropic materials, or composite materials where the criterion can also be applied to wood.

For applications in timber, the Hill yield criterion might be one of the most common failure criteria for anisotropic material, and applications in wood-wood joints are no exception. Hill yield criterion (1948) was derived as a theory related to yielding and plastic flow when considering anisotropic materials, specifically cold rolled steel [105]. Variations of Hill Yield criterion are often presented in research, for example, the 'quadratic Hill yield criterion' and the 'generalized Hill yield criterion for anisotropic material'. Hill (1993) later presented a simplified theory of orthotropic plastic response and yield of metallic polycrystalline rolled sheets [107]. Across all literature encountered in the research presented herein, Hill was the most cited theory used among researchers.

To better understand composite materials, Hoffman's criterion uses elements of Mises-Schleicher's isotropic and Hill's orthotropic yield conditions to propose a fracture condition for brittle orthotropic materials for compressive and tensile strength in various directions, though at that time it was not verified [98]. Hoffman (1967) found that a 'fracture condition' best describes the physical phenomena experienced by brittle materials as they undergo an elastic phase followed by catastrophic failure, where a ductile material experiences a gradual failure in plastic deformation after the elastic phase, explained most accurately as a 'yield condition' [98]. For anisotropic materials it is important to define the principal stress axes orientation with respect to the strength planes which must be known such that geometric and stress parameters are considered jointly [98]. On the back of this concept, a report by Tsai and Wu proposing a theory of strength of anisotropic materials was driven by the need to evaluate composite materials [108]. Today, Tsai-Wu failure criterion is most widely used for elastic-brittle material though it is often found to be conservative in prediction of failures [83]. Following Tsai-Wu strength theory, further criterions have been proposed. For example, Cowin (1978) built on Tsai-Wu to propose equations for tensile, compressive and shear strength for orthotropic materials with respect to off-grain direction for materials such as bone and wood [109].

Like Hankinson's, some criterions have been proposed specifically for wood and are not models for arbitrary elastic-brittle, orthotropic, or anisotropic materials. Johansen's theory of timber connections (1949) provides design recommendations for timber connections secured into place using dowels, friction, and "toothed dogs", which are similar to nail plates [110]. While it also considers metal, this theory has served as a guideline for decades in timber connections with wooden dowels, is regularly referenced in research, and is commonly incorporated into building codes. 'A general failure criterion for wood' proposed by van der Put (1982) considers tangential, radial, and longitudinal stresses and follows analytical approaches using sample testing characterized by material yielding and fracture with respect to grain direction [111]. Serrano (2020) responded to the drawbacks of complex failure criteria by developing a simplified plasticity model for wood in compression [112].

As timber behavior is circumstantial, no perfect evaluation method for timber exists. Often research focuses on comparisons between analytical approaches regarding different materials and connection types to validate which methods are most circumstantially appropriate, particularly among building codes. For instance, Šobra, de Rijk, *et al.* (2016) used both Norris' and Hankinson's theories to obtain maximum allowable stress at a given an angle to the grain in a single step joint [21]. Since Eurocode 5 does not explicitly state standards for calculating capacities of timber joints, the Dutch (based on decomposing forces into components), German (based on Norris's theory), and Swiss (based on Hankinson's theory) national annexes were used and compared to experimental testing of single notch step joints in compression, all of which resulted in significant underestimations by building codes [21]. Likewise, some studies consider different theories in complimentary evaluations. Akter *et al.* (2021) approached the modelling of combined loading and compared results with Hill's orthotropic yield criterion, Hoffman's fracture condition and quadratic multi-surface (QMS) failure criterion which is validated with experimental data and finite element modelling [113]. Mascia, Nicolas and Todeschini (2011) compared Tsai-Wu and Hankinson's failure criterions for tension [114]. Mascia and Simoni (2013) carried out comparisons between Tsai-Wu, Tsai-Hill, Hoffman, Norris and Rankine failure envelopes to experimental data [115]. Instead of fitting the phenomenological failure criterions of composite materials to wood, Cabrero *et al.* (2012) used a number of criterion predictions to evaluate the suitability of use for wood, including the Linear Criterion, Quadratic Criterion, Von Mises' classical composite criterion, Tsai-Hill criterion, Norris Criterion, Tensor derived criteria, and the Cowin, Hoffman, and van der Put criterions [116].

One considerable risk of wood-wood connections is the combined discontinuity in the timber fibers and transfer of forces occurring at the same place where small cracks could propagate into fractural failures parallel to the grain. These fractures are consistent with one of three fractural modes seen in fractural mechanics. A fractural mode refers to tendency of a crack tip stress to decompose into a fracture from one of three corresponding critical stresses and consequential fracture type, commonly known as mode I for stresses orthogonal to the local crack plane surface, mode II for stresses parallel to the local crack plane surface but orthogonal to the crack tip surface, and mode III for stresses parallel to the local crack plane surface and crack tip surface [117]. The first case is when a fracture occurs as an opening perpendicular to the normal force is called mode I, and in timber this is exhibited by splitting down an element through the grain of the timber when vertical forces are applied and the tension between the grains cannot hold the material together. The second case of fracture, mode II, is seen as cracking down the center of an element as a result of shear forces and fracture is in plane with the shear forces, which in timber is seen as sliding while fibers separate in shear. The final fracture case, mode III, occurs from out of plane shear where a tearing fracture is perpendicular to the shear force direction, which might occur from torsional rotation but is significantly less common compared to mode I and II in timber structural elements.

The evaluation of each fracture mode can be achieved through experimental and analytical methods. Analytical methods are often dependent on a Stress Intensity Factor which is expressed as the fracture toughness (K, highest stress intensity that a material can withstand) to predict material resistance to crack propagation. Experimental testing can be used to extract the relationship (R-curve) between the crack growth resistance in terms of fracture toughness (K) and change in crack length of a specimen of a specific geometry and species. For in-depth detail on this topic, see Jockwer, Steiger, and Frangi's (2014) state-of-the-art review of design approaches to notched timber beams without reinforcement, which describes the context of notched timber, considers several fracture models, describes design applications, and comments on experimental validation methods from literature [50]. Dewey *et al.* (2019) evaluated different design recommendations of end notched girders and associated fractures modes and found that building codes were inadequate in reflecting real notch behavior, though notably the experimental campaign also concluded that tapering of end notches can increase capacities [118]. Contrarily, many evaluations of joints with notches now favor Cohesive Zone Models (CZM) made with the assistance of finite element modelling (FEM).

5.2 Evaluation of connections in literature

The evaluation of wood-wood carpentry joints can be complex, particularly considering the indeterminate interfaces of multiple pieces where forces transfer at a location where grain directions of contact surfaces differ. Elastic models are common though failures are often brittle but may have a plastic phase characterized by fibers crushing. Identifying likely failure modes is paramount since a brittle failure may be catastrophic, meaning that the failure is irreparable and there is a risk to safety and structural integrity. Such failures can occur in typical loading during serviceability when a material ages, weakens or is defective, or during dynamic loading like wind or seismicity which might cause member pull-out, crushing, or form fractures due to moment rotation. These actions affect load transfer, which might act on surfaces with different grain directions and may cause joints to perform differently. Consequentially, experimental testing for validation is extremely common among timber carpentry research, and is frequently paired with analytical or numerical evaluations.

5.2.1 Classifying connection types and evaluation methods found in literature

To better understand where areas of interest and neglect exist in timber joint evaluation, the following is a summary of the various evaluations of individual joints, brackets, or frames. This summary reflects a large part of academic literature, though the authors acknowledge a blind spot where material is unavailable in the English language. As seen in Table 1, literature is categorized by (i) connection type, (ii) locality of that connection, (iii) whether an experimental, analytical, or numerical (E/A/N classification, respectively) component was included in the study, (iv) applied load, (v) load protocol, (vi) material description, and (vii) a brief description. Note that in cases where the literature did not describe an experimental evaluation, all studies state therein where the data was sourced for application of analytical methods. See Table 2 for abbreviation information.

Observations of Table 1 show many trends among research. Beam to column connection research is the most popular topic with the USA, UK, Japan, Taiwan, Malaysia, and China having conducted research mixed between mortise tenon and through tenon connections, where pegs (dowels) are most commonly the focus in the USA and UK. Among Asian studies of beam to column connections, load action and protocol cater to seismic evaluation with an interest in seismic evaluations of frames in China. Timber frame connections and column to column connections in Asia are rarely seen, though bracket performance in seismic actions is a popular topic for experimental research in Tibet, Taiwan, China, and Japan, though analytical and numerical models are not common due to the complexity of the brackets. Beam to Beam splice and scarf joint research remains popular, particularly across Europe (See state-of-the-art review [49]), in monotonic flexural and tensile actions which are often paired with finite element modelling. In Europe there is a great interest in roof truss elements under monotonic compression, and experimental, analytical and numerical models are mixed in use.

Table 1: Summary of evaluation methods implemented in the literature

Reference	**Connection Type**	**Locality**	**Evaluation Type (E, A, N)**	**Load Applied**	**Load Protocol**	**Joint Material**	**Description of Research**
Beam to Colum Connections							
[18] 1999	MT WD WoD	USA	E, A, -	4PT	MN	Eastern White Pine, Northern Red Oak, Douglas Fir	Comparison of, (i) mortise tenon, (ii) mortise tenon (with shoulder), (iii) mortise tenon (shoulder and knee brace), (iv) fork and tongue.
[52] 2003	MT WD	USA	E, - , -	T	MN	White Oak, Red oak, Douglas Fir	Experimental evaluation of peg end distance impact on connection performance.
[53] 2008	MT WD	USA	E, A, -	MR	MN	Yellow Poplar	Experimental evaluation of joint rigidity and frame deflection.
[54] 2015	TT KY	USA	E, - , -	T	MN	White Oak,Red oak, Douglas Fir	Compares keyed-through tenon and pegged mortise tenon.
[12] 2009	MT WD	UK	E, A, -	PO[1] 3PT[2] 4PT[3]	MN[1] MN[2] MN[3]	Green Oak	Pull out on green oak beam with bending test seasoned oak pegs.
[62] 2000	TT NC	Japan	E, - , -	MR	QS	Sugi with Hinoki, Keyaki, and Moabi wedges	Evaluation of relationship between wedge embedment and moment rotation in *Nuki* joints.

[61] 2008	TT NC	Japan	E, - , N	O	O	Japanese Cedar, Moabi wedges	Analysis of initial stress state of *Nuki* joint with various wedge sizes through experimentation and (ABAQUS) FEM.
[63] 2008	TT NC	Japan	E, - , N	MR	MN	Japanese Cedar, Moabi wedges	Analysis of *Nuki* joint racking behavior with various wedge sizes through experimentation and (ABAQUS) FEM.
[14] 2008	TT NC NG	Japan	E, - , -	MR	CY	Beech wedges and beams. Poplar, Sugi, Falcata (posts).	Retrofitting proposed to increase stiffness and seismic performance. *Nuki* beam and *Kusabi-Nageshi* compared.
[64] 2006	TT NC NB	Taiwan	E, A, -	MR	MN	Chinese Fir	Taiwanese *Nuki* in rotation with various gap sizes
[60] 2007	TT NC NB	Taiwan	-, A, -	MR	MN	Chinese Cedar	Considers gaps and moment rotation in butted versus continuous Taiwanese Nuki joints.
[55] 2013	MT	Malaysia	-, A, N	PO	MN	Kempas	(LUSAS) FEM evaluates connection using tropical species of timber.
[66] 2011	DMT	Korea	E, - , -	MR	MN	GluLam (Japan larch)	Beam over column connection with H shape seat and DMT. Comparison between connection with and without beam shoulder.
[58] 2013	DMT	Korea	-, -, N	T	MN	GluLam (Larch)	Parametric considerations for capacity of DMT with H shaped column seat. Evaluation includes stochastic finite element method (SFEM) and (ANSYS) FEM.
[65] 2013	MT SN	Korea	E, - , N	MR	MN	GluLam (Red pine)	Geometric parametrization for increased moment carrying capacity of *Sagae* joint using (ANSYS) FEM.
[67] 1999	FS DMT TT	Korea	E, A, -	SR	CY	Pine (unspecified)	Seismic study of traditional Korean house featuring two full-scale tests connections of DMT and TT MT seated on column.
[40] 1996	MT	China	E, A, -	MR	MN	Shorea	Evaluation of artificially degraded MT joint in beam to round wood column connections.
[42] 2016	DMT	China	E, A, N	MR[1] PO[2]	MN[1] MN[2]	Chinese Fir	Compares two DMT connection types in experiments and (ABAQUS) FEM, proposed analytical models.
[71] 2018	FP DMT	China	E, - , -	SR[1] SR[2]	ST[1] CY[2]	African Rosewood	Scaled full pagoda evaluated and sections of beam, post, and panels evaluated (without fill, partial fill, filled)
[69] 2021	FP MT TT	China	E, - , -	SR	QS	Undisclosed	Seismic evaluation of timber framed shear walls with MT TT connections.
[68] 2011	FS MT	China	E, - , -	SR	CY	Chinese Fir	Characterization of seismic response of various mortise tenon connections in southern Chinese traditional timber frames.
[57] 2018	FP MT	China	E, - , -	SR	ST	Undisclosed	Seismic response of scaled whole traditional column and tie structures including seven varieties of mortise tenon connections.
[70] 2018	FS TT DMT	China	E, - , -	SR	LC, LRC	Chinese Fir	Seismic evaluation of four full-scale *Chuan-dou* timber frame types with DMT and TT connections.
[59] 2021	MT TT	China	-, A, N	RC	CY	Northeast Chinese Dahurian Larch	Evaluates a typical historic mortise tenon (*Chuan-dou* frame) considering friction in theoretical analysis and FEM (ABAQUS)
Column to Column Connections (End to End)							
[91] 2020	SJ KY	Korea	E, - , -	T	MN	GluLam (Larch and red pine)	Tensile properties evaluated in a traditional column-column connection of relative complexity.
Beam to Beam Connections (End to End)							
[23] 2013	SC	USA	E, - , N	T	MN	Hemlock	Experimental campaign of two joints compared to (ABAQUS) FEM to investigate defects and geometry.
[119] 2016	SP	Czech Republic	E, - , N	4PT[1] 3PT[2]	MN[1] MN[2]	Undisclosed	Compares experimental data with (ATENA 3D) FEM for connections with skewed angles of 45° and 63.3°.
[32] 2018	SC	Spain / Europe	- , - , N	T	MN	Scotch Pine	Experiment failure modes are recreated in (ANSYS) FEM. Failure mode verification equations are proposed.
[49] 2020	SC	Central Europe	-, A , N	3PB	MN	Norway Spruce, oak pegs	Numerical analysis of spliced rafters in (ANSYS) FEM, and various trusses in (SCIA Engineer) FEM.
[89] 2021	SC SP	State-of-the-art	For more information on the above beam – to – beam connections, see this paper.				
[22] 2021	SP	Poland	E, - , N	4PB	MN	Scotch Pine	Three connection types are experimentally evaluated in bending and compared to (ANSYS) FEM.
[23] 2013	SC WD	Italy / Europe	E, - , -	T	MN	European silver fir	Timber pegs and steel pins connections capacities compared to unreinforced connections capacity.
Log walls with corner lap joints							
[93] 2018	LJ, DMT, SN	Poland / Ukraine	E, - , N	O	MN	Pinewood	Experimental evaluation of log walls under axial and lateral loading with probabilistic (software undisclosed) FEM.
Floor Joist Connection							
[7] 2015	LJ	Japan	E, A, -	MR	CY, RC	Japanese Cedar	*Watari-ago* floor joint moment-deformation angle relationship considering 6 cases of geometric parameters.
Truss Roof Systems							
[82] 2008	BSJ, DSJ	Italy / Europe	E, - , N	C MR	MN CY	Unspecified	Experiment compares various seismic retrofitting techniques with FEM (limited model details).
[17] 2010	DSJ, BSJ	Italy / Europe	- , - , N	C MR	MN MN	Norway Spruce	Seismically motivated numerical study of members under increased axial loading and rotation in (ANSYS) FEM.

[43] 2012	SSJ, SSJ MT	Portugal / Europe	E, A, -	C MR	MN	Maritime Pine	Evaluation of joint mechanics to failure then retrofitting joints and re-testing to evaluate retrofit performance.
[44] 2014	SC	Central Europe	- , - , N	3PB	MN	Norway Spruce, oak pegs	Numerical analysis of spliced rafters in (ANSYS) FEM, and various trusses in (SCIA Engineer) FEM.
[86] 2015	SSJ MT	Portugal / Europe	E, A, -	C	MN	Portuguese Chestnut	Study on aged wood in connections, including comparisons between new and old timber.
[83] 2018	SSJ MT	Slovak Republic / Europe	- , - , N	C	MN	Chestnut, Spruce, Redwood, Douglas Fir, Red Oak	Numerical evaluation of connection stiffness focusing on contact elements in (ANSYS) FEM and parametric study.
[79] 2017	SSJ, DSJ, RSJ	Spain / Europe	E, - , N	C	MN	GluLam (Norway Spruce)	Experiments and (ANSYS) FEMs of stepped joints at 30° where gaps between elements are considered.
[80] 2018	SSJ	Europe	E, A, -	C	MN	Scotch Pine	Overview of geometric parameters, common failure modes, and corresponding evaluation methods discussed.
[81] 2020	SSJ, DSJ, MT	Europe	-, A, -	C	MN	Scotch Pine	Authors propose an analytical evaluation method following evaluations of existing methods across three step joint types.
[13] 2013	SSJ	Europe	- , - , N	C	MN	Scotch Pine	Cohesive zone model in (ABAQUS) FEM considers geometrical parameters influencing shear cracking.
[41] 2016	SSJ TP	Europe	E, A, N	C	MN	Spruce	Experimental campaign, analytical model and (ABAQUS) FEM compared for 3 failure modes. In German Language.
[56] 2016	SSJ TP	Europe	E, A, N	C	MN	Spruce	Experiment, analytical models, and (ABAQUS) FEMs to evaluate relationship between failure mode and joint angle.
[20] 2009	DMT, TP, WD, WoD	Czech Republic	E, A, N	T MR	MN CY	Norway Spruce	Impact of defects and gaps are analysed from experimental data. Lap joints explored as retrofitting option for decaying timber. Roof truss' (SCIA Engineer) FEM presented.
[82] 2008	DMT WD WoD	Europe	E, A, -	T[1] C[2]	MN[1] MN[2]	Scotch and Maritime pine, massaranduba dowels, Lodgepole pine numerical values.	Eurocode 5 evaluated for adequacy in predicting dovetail rafter capacity with and without dowel.
[17] 2010	Case studies of German Church roof systems.						
Post and Beam Roof Connections							
[88] 2015	WD	China	E, A, -	4PT	MN	Chinese white pine, Chinese fir	Evaluation of various roof stitching beams in flexure.
[87] 2017	MT	China	E, -, N	MR	LC, LRC	Chinese Fir	Considers beam to long and short columns in a roof structure through experiment and (ANSYS) FEM.
Bracket Systems							
[2] 2021	OBS	Tibet	E, A, -	MR	MN	Mongolian Scotch pine	Experimental and numerical evaluation of bracket where parallel beams of increasing length are stacked and pinned.
[77] 2001	OBS, NC	Japan	E, -, -	SR	ST	Japanese Cypress Column, Zelkova bracket, Japanese pine other	Evaluation of seismic performance with two steel dampers types.
[78] 2001	OBS, NC	Japan	E, -, N	SR	ST	Japanese Cypress Column, Zelkova bracket, Japanese pine other	Experimental and global (NASTRAN) FEM compares buildings with To-Kyou brackets.
[73] 2008	FS, DGB, MT	Taiwan	E, -, N	MR[1] PO[2]	MN[1] MN[2]	200-year-old aged European pine	Global (Algor) FEM of Dei-dou frame structure based on experimental evaluations.
[72] 2018	DGB	China	E, -, -	SR	CY, RC	African Rosewood	Considers various dou-gong brackets, including single and double, concentrically and eccentrical loading.
[74] 2020	DGB	China	-, A, -	SR	ST	Korean Pine	Presents fast nonlinear analytical seismic evaluation of traditional frame buildings with Dou-gon(g) brackets.
[76] 2020	DGB	China	E, -, -	SR	LRC	Dahurian larch	Identifies features dou-gong types impacting bearing capacity, rotational stiffness, and energy dissipation.
[16] 2008	DGB, O	China	Case Study of oldest surviving Chinese pagoda.				
[120] 2016	DGB, O	China	Case Study of Chinese heritage pavilion structural evaluation with restoration and retrofitting recommendations.				
[121] 2020	DGB, MT, O	China	Proposed framework for safety-state evaluation of aged Chinese timber structures -with a focus on Dou-gong brackets and post and beam mortise tenon.				
***Notation [1, 2, 3] signifies which load protocol belongs to which test if more than one exists**							

Table 2 : Key to carpentry connection descriptions annotations found in Table 1

Carpentry Connection Type	Other Connection Information	Load Applied	Load Protocol	Evaluation Type
MT = Mortise Tenon	WD = With Dowels (or pegs)	C = Compressive	MN = Monotonic	E = Experimental
DMT = Dovetail Mortise Tenon	WoD = Without Dowels (or pegs)	T = Tensile	QS = Quasi-static	A = Analytical (Theoretical equation or building code)
TT = Through Tenon	KY = Keyed	MR = Moment rotation	LC = Low-cyclic	
SSJ = Single Step Joint	NC = Nuki Continuous	PO = Pull-out	LRC = Low-reverse-cyclic	
DSJ = Double Step Joint	NB = Nuki Butted	3PB = 3-point bending	CY = Cyclic	N = Numerical (FEM)
RSJ = Reverse Step Joint	NG = Nageshi	4PB = 4-point bending	RC = Reverse-cyclic	
BM = Birds Mouth Step Joint	FP = Frame and Panel	SR = Seismic response	ST = Shake table	

LJ = Lap Joint SC = Scarf Joint SP = Splice Joint DGB = Dou-Gong Bracket System	FS = Frame Structure TP = Tapered Tenon SN = Saddle Notch OBS = Other Bracket Systems	O = Other	O = Other	

5.2.2 Finite element modelling of wood-wood joints

In recent decades, finite element modelling (FEM) has become a useful tool in numerical modelling of wood-wood joints. If considering only experimental testing, obtaining stress distributions within a joint is almost impossible [61]. In some cases, like for step, scarf, or splice joints, strain data captured on the side of a member presumes the stresses occurring inside the joint. For these types of joints, digital image correlation (DIC) is better suited for tracking stress distributions and crack propagation for FEM validation [83], however, the review herein found that DIC is rarely used in timber connection testing. Since strain data captured from the interface of adjacent members of a complex joint is regarded as impossible, other methods of model validation are necessary. Consequentially, failure criteria in FE models are of the highest importance because failure modes seen in experimental testing are caused by the internal forces of joint, and recreation of the failure mode is based on a joint's internal force distribution, therefore the failure mode can validate a finite element model [32]. This is particularly important in development of adequate nonlinear finite element analysis, which is necessary to understand regions of (i) high-stress where the material is non-elastic and (ii) areas of stress gradients that transition from elastic and non-elastic behavior [61]. In short, a key advantage in finite element modelling of wood-wood connections is the designer's increased understanding of connection behavior at the member interfaces where real data is unobtainable due to the connection's reliance on interlocking and friction.

A key challenge in finite element modelling of wood-wood joints is using a material model that can best reflect the behavior of the material. For example, Šobra and Fajman (2013) modelled a keyed vertical splice skew in 4-point bending using ATENA, software generally used for composite and concrete materials where a nonlinear cementitious material was used and orthotropy can only be achieved by introducing smeared reinforcement, but the authors were unable to achieve results similar to experimental testing with the software limitations [23]. Many material models, like those found in Section 5.1, exist within FEM software and can be used in FEM software such as ANSYS or ABAQUS, though poor material model choices for expected actions can produce inaccurate model connection behavior results. Due to the complexity of the anisotropic material properties of timber, wood-wood joints are almost always simplified, and one common approach is to use an orthotropic material model with transversal isotropy [13,58,65,83,119] which has the additional benefit of gains in computational costs with relation to more complex material models [83]. Complex models can be created from scratch by writing a User Material Code, or UMAT code, which can incorporate the material properties and failure criteria of the user and is read by the finite element analysis software.

In addition to material model selection, many other parameters can impact the outcome of an analysis, such as element selection, mesh concentration, node numbers, degree of freedom, friction coefficients, contact surfaces, etc. In ANSYS, models often used the SOLID185 [65] and SOLID186 [32] elements as well as PLANE42 elements [83,119], and in ABAQUS C3D8R elements are often used [42,59,81]. For improved results in failure regions, the "h" method can be applied by densifying mesh at areas of interest [83,119]. Within the literature reviewed herein, mesh sizing is not always disclosed though it was visible that this method was common and, in many cases, it was stated that areas of interest employed a mesh of 1 mm [13,81,83,89,119]. When selecting a friction coefficient, Villar-García *et al.* (2018) found references validating values ranging between 0.3-0.5 [83], however, when Karolak (2021) compared many friction coefficient values between 0.2 and 0.5 the results were not noticeably impacted in this case, despite that the generally recommended value for wood-wood interfaces is 0.5 [89]. All models included in this review take friction coefficient values between 0.2 - 0.6, and authors commonly note that Coulomb friction model is considered.

The most common software seen in finite element modelling of historic wood-wood joints within the previous 10 years is ANSYS. All finite element models discussed herein are validated with experimental testing. Jeong *et al.* (2013) observed the load-carrying capacity of variations of traditional Korean dovetail connections to validate a reasonably predictive FEM to optimize connection geometry where model inputs applied transverse isotropic material assumptions and a governing failure criterion of capacity of shear and tension perpendicular to the grain [58]. Jeong *et al.* (2013b) considered parameterized geometries of a *sagae* connection in FEM to maximize moment carrying capacity using Solid185 library to make the 3 structural elements, where longitudinal, transverse, and shear material properties are defined and failure criterion considers gain parallel tensile and shear forces, of which the difference between model and testing outcomes is 3.33% and geometry recommendations are provided [65]. Chun *et al.* (2017) used FEM to supplement a limited number of moment-rotation tests of two mortise-tenon type connections found in Chinese post-and-lintel roof constructions using a contact force-based nonlinear analysis that considers elastic-plastic orthotropic material model and uses generalized hill yield criteria to achieve relative agreement with test data [87]. Aira *et al.* (2016)

used external surface strain gauge data of 42 specimens of 8 geometries tested in bending to validate a plane stress linear static study in FEM of tenoned scarf joints where the material model was orthotropic with transversal isotropy, and material parameters, contact parameters, and mesh are discussed and result in the evaluation of three failure modes for which verification equations are proposed based on the finite element analysis [119]. Villar-García *et al.* (2018) evaluated 4 reverse step joint scenarios in experimental compression tests and FEM, of which the linear elastic model incorporates a two-dimensional elastic-orthotropy with averaged radial and tangential mechanical properties are averaged and material properties, elements, contact parameters, and mesh are rigorously justified in the literature, and although Tsai-Wu failure criterion is the most commonly used it was found to be conservative [83]. Kunecky *et al.* (2018) used scaled experimental bending testing of scarf joints to validate FEM where the model aim was to obtain (i) forces acting on vulnerable areas and (ii) stiffness variation [32]. This was achieved with a model using quadratic solid structural elements SOLID186 and linear orthotropic material properties were considered for spruce members and oak dowels, while interfaces were modelled with asymmetric contact using Augmented Lagrange contact algorithm and peak values were reduced by reducing the stiffness parameter FKN, as the interface forces were of concern [32]. Karolak (2021) used Workbench 16.0 for a FEM with an orthotropic and elastic material model to compare bolt configurations of retrofitted scarf and splice joints in 4-point bending to FEM to compare the stiffness of different joints and recommended retrofitting with the best stiffness [89].

Another common software used to model wood-wood connections is ABAQUS, and these models are also always validated with experimental testing. Guan *et al.* (2008a) carried out initial stress state evaluation of *Nuki* beams with varying wedge sizes [61] and Guan *et al.* (2008b) evaluated the beams under racking conditions [63] as a two-part study. In both cases, 3D nonlinear FE models were made with orthotropic elastic material model in tension and elasto-perfect plasticity in compression where wedge considers strain hardening [61,63]. The initial stress state of four wedge embedment models had acceptable agreement with strain gauge data [61], and pre-failure modelling to evaluate joint behavior and local failure modes from racking was achieved reasonably well in modelling though post-failure modelling was not possible without damage mechanism models, and overall findings indicate that joint stiffness and not wedge size effect ultimate racking [63]. Sangree and Schafer (2009) created 3D linear elastic FE models of scarf joints where material models are variations of isotropic, transversely isotropic, and orthotropic materials, and models predict contact and stiffness with respect to gaps, as well as stresses relative to grain angle [122]. Koch *et al.* (2013) used FEM to make models of tapered tenon joints with cohesive zone models (CZM) to increase understanding failure modes I, II, and III of lamella separation where the transversal isotropic material model considers elastic material parameters and the modelling methods are discussed in depth, though the authors believe that the discrepancies with experimental failure capacities must be explained by the influence of friction coefficients [13]. Parisi and Cordié (2010) made a FEM of double step joints in compression and moment rotation, where Hill type elastoplastic materials are considered and outer transversal elements are simplified as an orthotropic material which was comparable to test outcomes including realistic plastic crushing regions with moderate friction coefficients and disassembly of the connection with low friction coefficients [17]. Chen *et al.* (2016) modelled dovetail mortise tenon beams in moment rotation with two different column assemblies and considered an orthotropic material model with elasto-brittle properties in tension and elasto-plastic Hill type properties in compression where contact modelling is discussed and results are comparable to experiments, though initial stiffness and non-linear response in pre-peak moment exhibit some discrepancies [42]. Xie *et al.* (2021) outlines the process of implementing a finite element analysis to validate a contact model of a mortise and tenon connection with 7144 elements that are uniformly spaced at 30 mm without concentration zones to accurately predict failure modes seen in reverse cyclic testing [59]. Verbist *et al.* (2020) built on past work to build a FEM in combination with experimental testing of specimens within a geometric parametric study that considers Hammock Shape Shear Stress Distribution (HSSSD) with a Cohesive Zone Model (CZM) at the upper to lower sections interface of the notched beam to find a reducer coefficient to be used to improve design of shear crack resistance of single step joints [81]. The linear elastic orthotropic material model with mode I, II, and III fracture conditions are considered at the cohesive zone, and the authors present empirical equations based on the results of the analysis [81]. Otherwise, Amaruddin *et al.* (2013) used LUSAS 14.7 to model a mortise and tenon with square dowels using a linear elastic orthotropic material model on the mortise, tenon and dowels to accurately reflect the failure modes seen experimental testing, despite some limitations to number of nodes used across all mesh elements [55].

As one can see, there is very little literature published annually that discusses finite element modelling of wood-wood connections, however, many utilize the same approaches. Additionally, models must be validated with experimental results, and in rare cases a model can be validated by strain gauge data, but most models rely on aligned outcomes with failure modes or capacities, not necessarily both. There are no cases where digital image correlation

(DIC) is used for FEM validation, and no cases where state-of-the-art strain gauges like optic fiber sensors are used to capture connection interface strain data for FEM validation.

5.2.3 Structural evaluation of buildings with carpentry connections

As connection deterioration may critically affect joint stiffness, global functionality, or increase failure risk, structural evaluation of heritage buildings is important part of preservation. Within the literature, some global models presented in the literature have used Algor [73], SAP2000 [123], or SCIA Engineer [32,41] to evaluate dynamic loading, particularly eastern frames under seismic loading or western truss roofs under wind loading. In addition to these studies, some practical frameworks for structural evaluations have also been proposed throughout the literature. Qiao *et al.* (2016) documents the process of evaluating, restoring, and retrofitting the *Feiyan* Pavilion in China, which is over 500 years old and maintains most of its original components, including three levels supported by *Yong-ding* columns and outrigger beams, complete with a top-level double-deck eaves under a gable-and-hip roof [120]. The assessment methods and classifications of damage are discussed, then retrofitting solutions to local failures are nominated on a case by case basis that relates to either beam, column, or bracket (connection) failures [120]. Huan *et al.* (2020) developed a safety-state evaluation system for ancient Chinese timber architecture where (1) foundation, (2) plinth, (3) timber frame, (4) enclosing wall, and (5) roof are evaluated based the structural entropy weight method and the *Niaoqiangsanchu* building in the Forbidden City is used as a case study [121]. When evaluating *Dou-gong* joints, the damage caused by decay, biological attack, and deterioration are all considered using the index 'section damage' and a standard is applied for degree of damage, while in safety grading of simple mortise-tenon joints, cracking and dislodgement are also considered [121]. Though this evaluation is applied only to ancient Chinese building, it is relevant to all connection integrity evaluations. For those specifically interested in seismic vulnerability assessment, Shabani *et al.* (2021) focus on heritage timber buildings in European regions most at risk as well as evaluation and retrofitting methods, particularly by discussing experimental and finite element modelling approaches in log, post and beam, and frame structures [124]. It is unclear how evaluation methods applied to heritage buildings might be useful in monitoring new structures as they age, as this topic is rarely mentioned in literature. Contrarily, in anticipation of increased construction of large timber buildings, preventative recommendations are surfacing. To complement the growing popularity but extensive inherent complications of large-scale building with timber, Voulpoitis *et al.* (2021) propose a framework to evaluate structural robustness of tall timber buildings that considers both global and local factors including connections, connectors, and materials [15]. This kind of literature is increasingly popular for typical tall timber buildings, but lacks attention to concerns in wood-wood carpentry detailing in tall buildings.

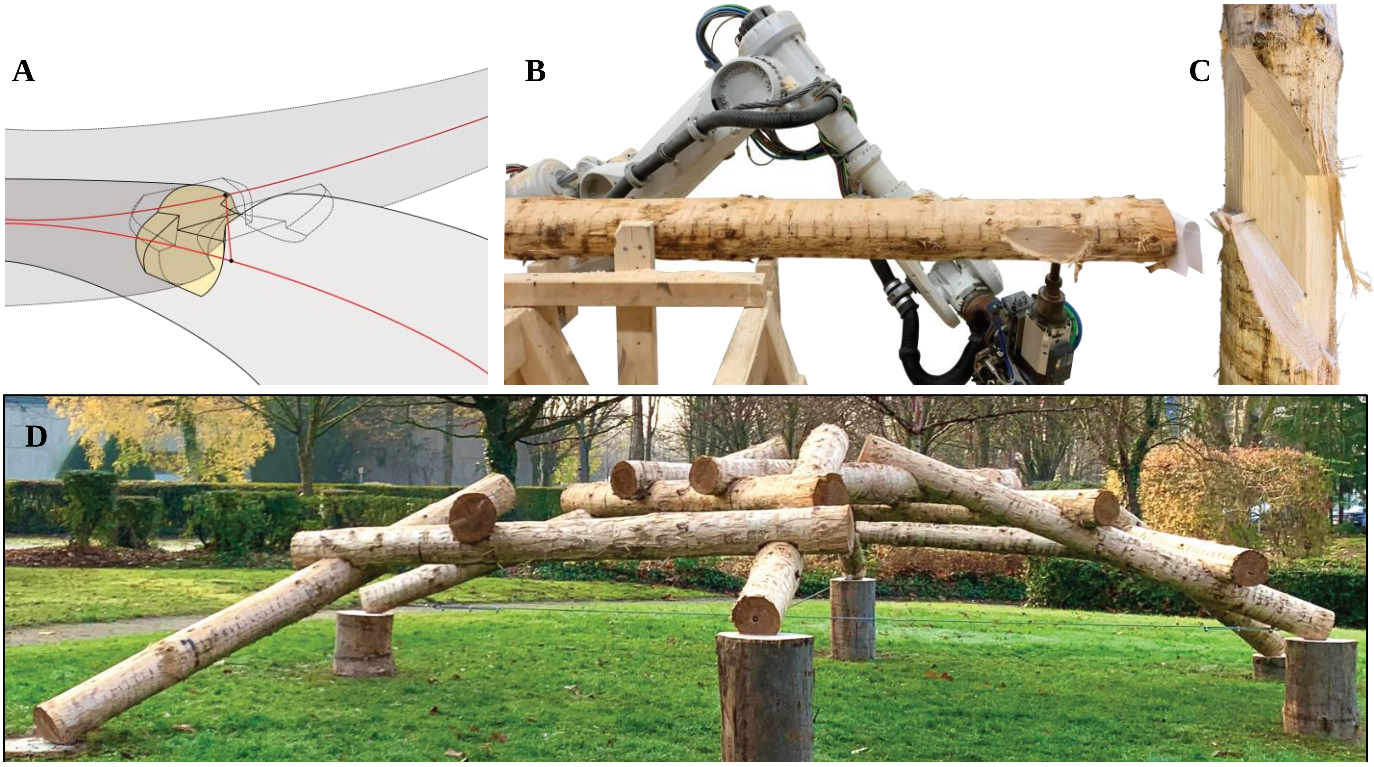


Figure 13: Production and assembly of a wood-wood connected reciprocal structure milled by a robotic arm [30]. The structural elements overlay (A) gives a reference for robotic milling instruction where connections are made with robotic subtractive fabrication of a joint in a whole timber log structural element using an ABB IRB 6700 robotic arm (B). The finished subtractive connections (C) are interlocked in the finished structure (D). For videos of design and production of the reciprocal structure, see here and here.

6 Contemporary wood-wood carpentry

6.1 Digitizing carpentry

Carpentry has changed drastically throughout the recent centuries. In Europe, timber frames are common for practical reasons, such as high affordability in northern Europe and good seismic response in southern Europe [33]. In the 21st century, modern tall timber buildings are already increasing in popularity and technology in this area is seeing unprecedented evolution [15]. Most tall timber buildings are using steel connectors. This is because though it is not necessary to use metal fasteners to make a sound timber joint, conventionally it is generally understood that metal fasteners are easier since traditional carpentry requires a great amount of time and skill [51]. This is becoming decreasingly true with technological advancements.

Historically, carpenters have hand-shaped joints according to local traditions. A key roadblock to creating carpentry connections is often cited as either lack of access to a skilled craftsperson or the great cost associated with labor of shaping each connection when compared to the cost of steel brackets or alternative building methods. New technological advances make these reasons obsolete. Modern tools like computer numerically controlled (CNC) milling machines facilitate the production of economic and efficient connections with high precision cutting, which has renewed interest in traditional connections [13,48]. Consequentially, selection of wood-wood carpentry is increasing since the requirement of highly skilled workers is no longer an obstacle with the use of milling machines [119]. For the first time in history opportunities exist in computer-controlled milling to industrialize carpentry joints [17]. Furthermore, computerized milling provides the ability to produce various geometries of connections [58]. The reoccurring problem is that designs considering digitized fabrication cannot be fully realized until design and fabrication guidelines exist in building codes [48]. Regardless of the lack of guidelines, many researchers already use CNC milling machines to fabricate test specimens for research regarding traditional connections [13,65,66,79,91]. For example, most experimental campaigns discussed in this review were machined and not hand crafted.

In a similar fashion, researchers who are looking to modernize carpentry are now able to design and fabricate complex connections with robotic assistance. Robots are being used to both collect information about the geometries of available materials and to reshape the materials to suit nominated connection geometries which allows non-uniform structural elements to act within a global structure. Vestartas (2021) and Geno *et al.* (2021) both exemplify how structural elements of non-uniform geometries can be digitized and shaped with robotic assistance (see Figure 13 and Figure 14) to create structures with materials that would traditionally be discarded or reshaped [31,125]. Rogeau *et al.* (2020) focus on applications of fiducial markers and offer a framework for robotic insertion of the timber joints in leu of other automated assembly techniques which might not accommodate variations in geometrical tolerances [126]. Rogeau *et al.* (2022) then consider design constraints realized throughout hands-on trials of robotic assembly of through-tenon timber joints and submit design recommendations to facilitate accurate assembly, notably balancing compromises between manufacturing tolerances such that joint dimensions retain rigidity while facilitating ease of assembly [127]. Further to these ideas, the efficiency offered by material digitization in fabrication facilitates a method of design for manufacture and assembly (DFMA) that would simplify both design and assembly through planning alongside supply and manufacturing capabilities with considerations for cost-effectiveness [35]. Technological methods applied to sustainable materials selection through digitization, global design using digitized materials, and manufacturing with respect to individual geometries are emerging to present a new field where the focus combines reduction of waste, decreased processing of materials, increased efficiency of production, and reduced costs in sustainable construction.

Prefabricated structural elements with shaped connections is reoccurring in the literature, whether by subtractive fabrication with robotic arms or milling machines. It appears to be the next natural step in fast, affordable, and sustainable timber buildings. The modern revival of carpentry has been born from the desire to preserve cultural heritage and the modern attraction to carpentry has been born from the potential to meet sustainability demands, but the industry is once again changing. Carpentry is now at the threshold of a technological revolution where for the first time the creation of structures with wood-wood carpentry connections is no longer solely an art or craft, but also a science where advanced technology in design and manufacturing is only held back by limitations in design guidelines.

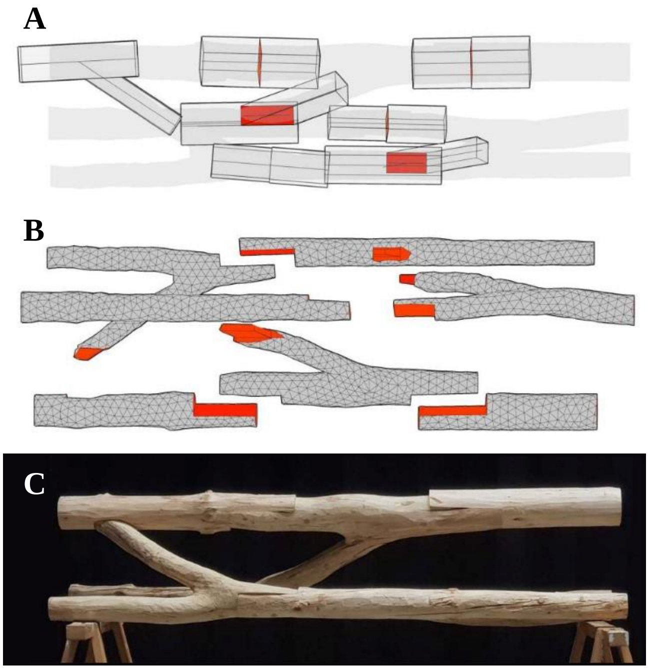


Figure 14: Framework of Joinery Solver applied to an irregular geometry element tree trunk truss where (A) Elements are grouped, (B) reference mesh is applied to irregular tree trunks, and the fabricated elements are seen assembled (C) [125].

6.2 Modern research and development of contemporary wood-wood connections

Modern research in the area of timber building connections, like most timber research, mainly focuses on practical ways to connect elements in Mass Timber (MT) buildings with steel connectors, and therefore has been excluded from this study, however, some research is focused on modern wood-wood or *all-wood* connectors. In this spirit, some research has reinvented plate and bolt concepts familiar to steel connections often referred to as “all-wood” connections, which increase sustainability and lower costs by eliminating steel. Chang *et al.* (2011) proposed a new all-wood beam-beam connection method using oak pins and plywood slot-in plates in heavy timber structures [128].

Kromoser *et al.* (2021) used robotics in the fabrication of trusses with timber structural elements and all-wood joints of plywood and wooden pegs to meet sustainability objectives while maintaining low costs [129]. In industry, rectangular sections with pinned plate connections remain the norm, but very recent innovations are stepping away from conventions and are reinventing wood-wood connections to economically address sustainability.

Despite the increased mechanical material advantages of whole and round timber, it is rarely seen in construction and even less so with carpentry connections, though the topic is gaining interest. In a state-of-the-art review of whole timber construction, Bukauskas *et al.* (2019) identified connections as a problem area due to the difficulty of connecting round elements [130]. Bukauskas *et al.* (2019) also noted that 'subtractive fabrication' involving digitization and reshaping with robotic arms or milling machines is an increasingly popular technique, yet the focus on joint types remained mostly on conventional connections [130]. Following this review, Geno *et al.* (2021) outlined the parametric design of roundwood reciprocal structures using a case study of robotically-shaped, wood-wood, end-notched connections which were substantively fabricated on trunks of unique geometries within a whole-scale reciprocal structure (Figure 13) where the unique connection was designed specifically to respond to geometric variations from moisture content changes [31]. Vestartas and Weinand (2020a) outlined the process of employing laser scanning and robotic arms for fabrication of elements from raw round wood by detailing the framework in the context of the Swiss timber and manufacturing industry and presented a prototype with subtractive fabrication connections made via the methodology therein [131]. Vestartas and Weinand (2020b) later presented the methodology developed with cooperation of industry partners to adapt geometrical scans and global spatial assignment of elements to a joinery solver that generates robotically shaped connections into round wood elements for the fabrication of whole timber structures [132]. In the doctoral thesis of Vestartas (2021), a complete and open-access framework is presented for design and fabrication of roundwood structures with wood-wood connections on structural elements of natural geometry, like crooked or forked logs (Figure 14), by using a Joinery Solver [125]. Research involving natural-form whole timber structures with digitization to aid in design and robotic machining in subtractive fabrication of wood-wood connections is only in its infancy.

Modern wood-wood connections are not only limited to whole structural elements, for example, components of shell-element structures made of plates with machine-cut, through-tenon fingers as joiners have emerged. Rezaei Rad *et al.* (2019) describes a modern experimental investigation into tensile performance of plate elements with through-tenon joints to hold LVL in place exhibits how plate elements with steel free connections can be applied [133]. Nguyen *et al.* (2019) introduced a framework which incorporates design and structural analysis of free-form structures made of plates edged with interlocking wood-wood though-tenons to form the structure shell [134]. Rezaei Rad *et al.* (2021) used the same case study structure as Nguyen *et al.* (2019) and automated a framework for design of digitally fabricated free-form shell structures with plates connected by interlocking through tenons [135]. Gamerro *et al.* (2020a) then proposed an analytical method for evaluating through-tenon connections in assemblies made from materials and connections accessible in industry [136]. Additionally, Gamerro *et al.* (2020b) compared Eurocode 5 design equations to experimental compressive and shear connection testing of three types of digitally fabricated, modern mortise tenons made from five different engineered timber products with a connection design suitable for prefabrication of residential structural elements with joints for in-plane loading [137]. Gamerro (2020) also presented a doctoral thesis that included the development of standardized structural plate elements made from shell-elements with digitally fabricated through tenon connections, of which the framework included construction systems, design guidance, and mechanical characterization [138]. While this area differs from a traditional sense of architectural carpentry connections, it introduces modern aspects of wood-wood joinery to architecture and engineering.

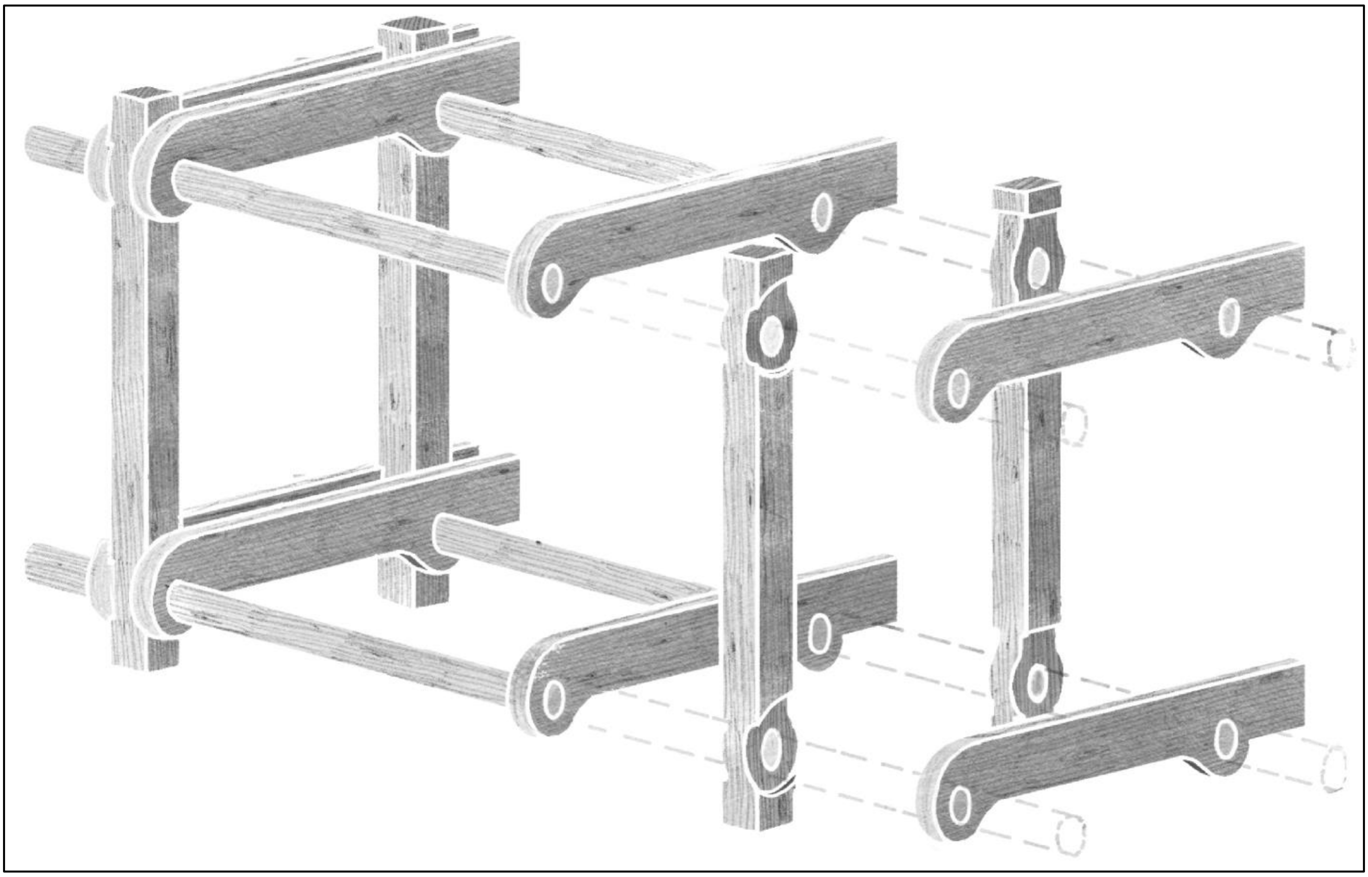


Figure 15: Four beam-to-column connections in an outer section of the Tamedia office building, Zurich, Switzerland, with assembled connection details (left) and exploded details (right). Drawing by Eva Kévers-Leclercq.

6.3 Commercial construction case studies

In an architectural review of modern steel-free carpentry, it was noted by Arlet (2021) that the number of projects discovered where architects incorporated timber carpentry connections was so high that they could not all be included, so the author resolved to include only 24 examples designed across four celebrated timber architects [3]. Among those examples most connections are not structural, however, some noteworthy examples stand out, including the Wooden Bridge Museum and coffee house designed by Kengo Kuma, featuring Chinese *dou-gong* joints which are also mostly ornamental [3]. Contrarily, Japanese Architect Shigeru Ban is well known for his work with timber and carpentry [139], among which the most relevant to modern steel-free structural connections is the Tamedia Office Building (Figure 15) in Zurich, Switzerland, which is a tall timber building made of engineered timber and Japanese inspired steel-free connections [3]. In addition to these examples, Fang (2020) notes in a review of modern carpentry that tensile tenon joinery was incorporated in the Writers Theatre (Glencoe, Il, USA), but comments that, due to lack of building codes, examples of carpentry connections in commercial buildings around the world are largely “boutique architecture” and not structural [10]. Since then, the Heartwood is an 8-storey residential mass timber building that is the first 2-hour fire rated building with all-wood doweled joints to be built as a type IV-C class building in North America, and is to be completed in 2023 [46]. Otherwise, mid-rise and high-rise structures with wood-wood connections are almost unheard-of while they remain relatively rare in low-rise. Likewise, there are many regions that still use carpentry connections in housing and small buildings, though few examples exist in commercial buildings.

7 Conclusion

This work has discussed wood-wood connections, from the ancient connections preserved through cultural heritage, to the developing innovations born recently from technological advancements and necessity. Elements joined with interlocking components have held structures erect for millennia, but challenges to material availability, access to skilled workers, and technological advancements shifted construction norms away from timber during the 20th century. Today, very few new structures honor traditional techniques. The majority of interest in timber carpentry is related to deterioration of heritage buildings and consequential restoration, retrofitting and preservation. In recent decades, concerns about climate change have forced industries to reconsider practices with excessive carbon emissions, including construction. Timber was quickly identified as a potential option for carbon storage, and in a logical string of thought many researchers have called for increased use of timber and carpentry. While it is undeniable that timber might aid in lowering CO_2 emissions from materials, there are outstanding logistical roadblocks to incorporating more timber and carpentry into construction, like the need for comprehensive systems and increased building code guidance.

The perceived advantages and disadvantages of timber carpentry are extensively discussed herein. Throughout an exhaustive review of academic literature related to wood-wood carpentry, global trends have been highlighted in relation to connection classification, typologies common to various localities, and evaluation methods, including experimental, analytical, and numerical approaches. Various theoretical evaluation methods are discussed in a short history of the origins and evolution of analytical approaches. The drawbacks of classical and modern analytical approaches are highlighted, and the limitations and benefits of numerical modelling is explained. Finally, developments in modern approaches to carpentry connections are reviewed. Throughout this investigation, some knowledge gaps have been identified:

(i) Lifecycle systems: Wood has not been widely popular for many decades, and the systems for growing, harvesting, coordinating, allocating, designing, fabricating, installing, maintaining, reusing, recycling, and disposing of materials are insufficient to facilitate or manage an increase in the use of timber without increased planning.

(ii) *Material issues*: Further research related to fire resistance, geometrical moisture response, degradation, and impact of defects is necessary in design guidelines for wider use of carpentry.

(iii) Experimental data collection: Evaluation methods largely focus on capacities and moment rotation but it is difficult to accurately validate analytical and finite element models (FEM) without the ability to capture experimental strain data, especially at complex interfaces between multiple elements in a connection. Digital image correlation (DIC) to evaluate strain on *external* surfaces is uncommon and *internal* interface strain data is difficult to capture. It is unclear to what extent technology could aid in model validation, particularly as data collection from inside a connection was only seen on one occasion in the literature. Future research could benefit from incorporating state of the art technologies such as distributed fiber optic sensors which can be installed at connection interfaces that were previously considered inaccessible to capture high resolution strain measurements with high accuracy, therefore quantifying mechanical behaviors that are otherwise considered to be indeterminate.

(iv) *Design Guidelines:* When timber became unpopular, research and consequential design guidelines for timber were neglected. As the benefits of timber in countering climate change are now well-known, interest in timber is increasing though it cannot be meaningfully used in many regions without sufficient design guidelines.

(v) Language barriers in studies of local typologies: While writing the literature review herein it became clear that extensive research on local typologies exists, but many works are published in the local language. The consequences are increased accessibility for the local populations and relative isolation of information between countries, cultures, and languages. Reviews comprised of compilations of localized research could be beneficial to the global community.

There has never been a time in history when knowledge of various wood-wood carpentry connections was more abundant. There is also no other time when knowledge was more accessible, when transfer of knowledge was easier, or when technology was more accommodating to facilitating fabrication. Additionally, there has never been a time with greater drivers to choose carbon-conscious materials. Despite this, the resources, technology, knowledge, and solutions presented are largely underutilized. While there are many in industry, academia, and the general public that support a shift toward greater use of carpentry, the frameworks and guidelines that might facilitate both wood-wood connection detail nomination and wider timber use are underdeveloped. Experts in both academia and industry agree on the potential for timber to meet sustainability and economic objectives simultaneously, but the frameworks required in supply chains, design guidance, technical specialization, and maintenance are incomplete. Though the methodology of implementation lacks modes of facilitation, researchers continue to work to improve traditional connections and mechanical predictability. Furthermore, many innovations are moving forward to incorporate sustainable connections into construction practices with the anticipation of eventual adoption of wide-spread design guidelines that adequately address detailing. Overall, many issues must be addressed before wood-wood carpentry connections can be used with confidence on a greater scale, but sustainability objectives continue to fuel persistence in exploring traditional techniques as a solution to modern problems.

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9 CRediT author statement

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