Automated Code Compliance for Structural Timber Design with Building Information Modelling

By

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DECLARATION

I hereby declare that the present thesis is of my own composition and that it contains no material previously submitted for the award of any other degree. The research work reported in this thesis has been conducted by myself, except where due acknowledgement is made in the text.

Andrew Livingstone

ABSTRACT

Title: **Automated Code Compliance for Structural Timber Design with Building Information Modelling**

This work set out to identify and overcome the barriers to specifying structural timber within the UK. In the first instance, a survey of practising structural engineers was conducted, and in combination with a review of the context via available literature, the objectives for the main body of the work were formulated. The solutions identified to address these barriers are in two forms: the first was to create code-compliant calculation tools for timber connections and the second approach presents a system for the automation of structural timber design as part of a Building Information Modelling (BIM) approach. The mathematical process of multi-dimensional data fitting is introduced in order to create automatic code compliance tools in BIM. This process is used to simplify the complex engineering calculations into a single equation that can be implemented into current BIM software engineering packages. BIM-based tools can contribute to addressing some of the challenges faced by structural engineering practitioners with respect to the design and detailing of timber structural systems, given the range of available timber products and enhanced levels of design complexities. From an industry perspective, it is envisaged that the work presented here can support structural engineers who want to incorporate timber in their projects but are finding the level of technical expertise required a significant barrier.

Author: **Andrew Livingstone**

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This thesis is dedicated to the memory of my sister Christine

TERMINOLOGY & ACRONYMS:

ACC: Automated Code Compliance

AEC: Architecture, Engineering and Construction

BS: British Standards

BIM: Building Information Modelling

SP-BIM: Single Platform BIM

MP-BIM: Multi-Platform BIM

CC: Code Compliance

CLT: Cross-laminated timber

EC5: Eurocode 5 - Design of timber structures.

ENU: Edinburgh Napier University

GUI: Graphical User Interface

MDDF: Multi-dimensional data fitting

ACADEMIC OUTPUTS FROM THIS DOCTORAL WORK

International Conference Publications

1) The Case for Mass Customisation of Structural Timber Design. Structures Congress. 2015. Portland Oregon: ASCE. Andrew Livingstone, Jesus Menendez, Kenneth Leitch, Robert Hairstans

2) A BIM Platform for Offsite Timber Construction eCAADe 2015. Vienna 2015. p. 597-604 Panagiotis Patlakas, Andrew Livingstone, Robert Hairstans ISBN: 9789491207082

   http://ecaade.org/testsite/wp-content/uploads/2016/08/eCAADe2015_volume1_lowres.pdf

3) Multi-Dimensional Data Fitting for the structural design of a simple timber connection. World Conference on Timber Engineering, Vienna 2016. Andrew Livingstone, Panagiotis Patlakas, Mark Milne, Sean Smith, Robert Hairstans

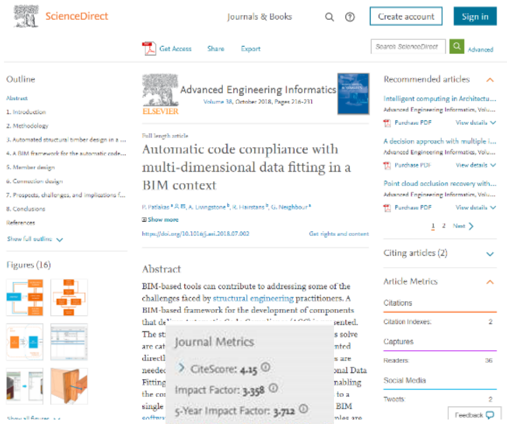
4) Automated code compliance checking to EC5 in BIM for structural timber connections. CompWood, Vienna 2017 ISBN 978-3-903024-49-6 Andrew Livingstone, Panagiotis Patlakas, Sean Smith, Robert Hairstans

5) An Automated Code Compliance system within a BIM environment *eCAADe 2017.* Rome Italy ISBN 978-94-91207-12-9 Panagiotis Patlakas, Andrew Livingstone, Sean Smith, Robert Hairstans

6) Design of a long span Belfast truss using UK home-grown timber, ECCM-ECFD 2018, Glasgow UK D. Johnstone, R. Hairstans, A. Livingstone

Journal publication

Published paper with Advanced Engineering informatics, ‘Automatic Code Compliance with Multi-Dimensional Data Fitting in a BIM context’, volume 38 page 216 – 231, (impact factor 3.4) (See Figure a).



Published Automated structural timber calculations

Automated structural calculations, published within the Tekla Tedds framework. There are approximately sixteen thousand users on the network within the UK. See 0.

1) Main to side member

2) Multi-member

3) Tension splice

4) Axial withdrawal

CPD Events

CPD iStructe 20th September 2016
Timber Connections Design theory to Eurocode 5, including a demonstration of how to undertake value engineering

CONTENTS

Chapter 1. Introduction

1.1. Timber in construction

Within structural engineering, the sub-specialism of structural timber design presents a particular case. As a structural material, timber has many well-established advantages which are: it has an excellent strength-to-weight ratio, it is durable and can be easily shaped and repaired [2]. It is typically considered aesthetically pleasing; possibly the only structural material where it is left exposed as an aesthetic feature and not as a cost-saving measure [3]. Finally, it is inherently sustainable as it provides good insulation properties [2] and offers a low carbon alternative to more conventional construction materials. The procurement of construction projects has a significant impact on the environment, the initial impact of a building on the environment results from the energy and other products consumed in its construction. Thereafter, the building and continues to affect the environment directly and indirectly throughout its operation, maintenance, refurbishment and final deconstruction.

On average, trees absorb the equivalent of a tonne of CO_2 for every cubic metre's growth [4]; this compares to nearly 0.9 tonnes of CO_2 emitted for every 1 tonne of cement produced [5] and an average of 1.8 tonnes of CO_2 emitted for every 1 tonne of steel produced [6]. Most importantly, timber is indefinitely renewable and extremely durable if used properly. It is ideal for the construction and prefabrication of buildings and building components [7] and a key component of initiatives that aim to modernize construction [8]. Carbon sequestration is typically defined as the capture and long-term storage of atmospheric carbon dioxide to help mitigate or defer global warming, this is illustrated within Figure 1-1. A comment for the European Commission DG Enterprise stated that 'For every cubic meter of wood used instead of other building materials equates to around 0.8 tonne of CO_2 saved from the atmosphere.' [9]

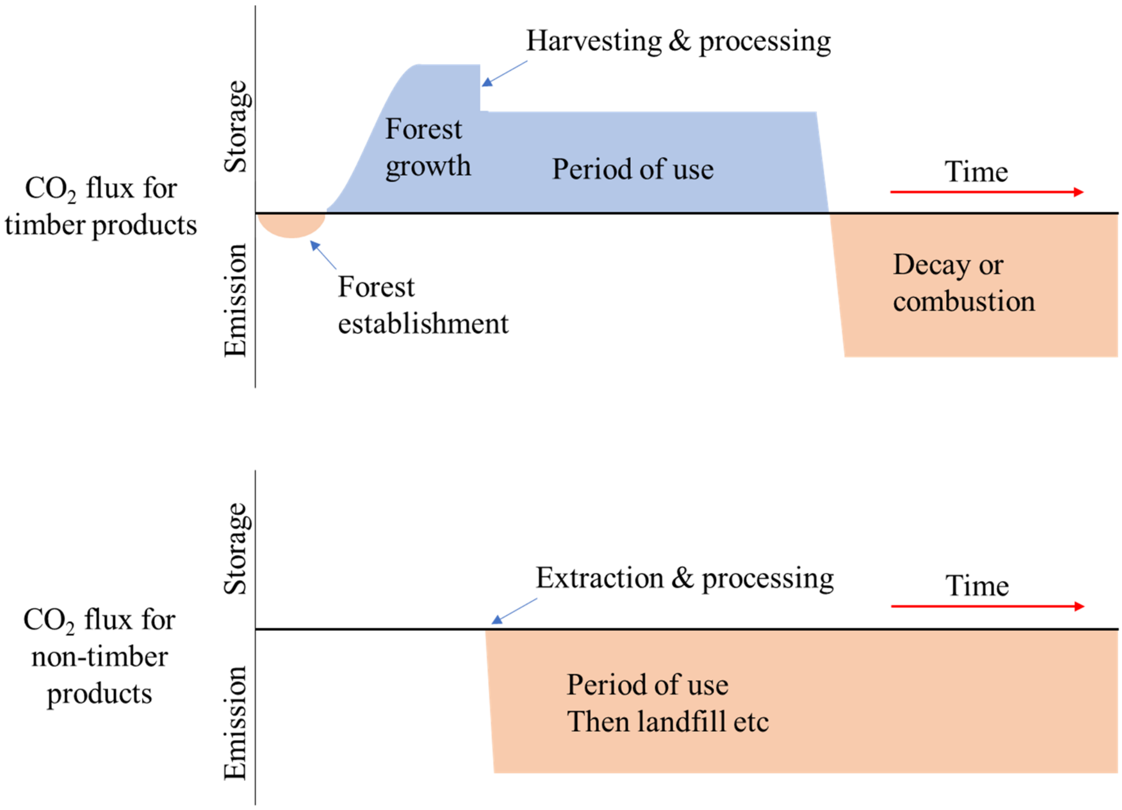


Figure 1-1 CO2 flux for timber and non-timber products

These advantages of timber use within construction have been recognised by producers and manufacturers and in recent years there have been considerable advances in the development of new timber products for structural uses. One of the main new products, Cross Laminated Timber (CLT), is seeing increases in both volume and distribution on a global scale [10]. In combination with other structural systems. such as Glued Laminated Timber (Glulam) CLT provides possibilities for tall, or even very tall, wooden buildings [11]. Such systems can contribute significantly to the reduction of the greenhouse gas emissions of buildings [12] and thus support the sustainability agenda.

Within a number of developed nations, timber construction is the dominant construction method for residential construction, accounting for over 70% of all new starts in climates as diverse as Australia and Norway. Table 1-1 gives a detailed breakdown of the residential market and corresponding application of timber construction in different countries, based on statistical data from [13-24].

Table 1-1 Snapshot of current global timber residential market share

Country	Population (in millions)	Housing stock (in millions)	Annual housing starts (000's)	Timber Structures market share (%)
Australia	23.5	7.1	190.8	90 %+
Canada	35.3	13.3	122.3	90 %+
Ireland	4.7	2.0	10.0	30%
Japan	127.3	60.6	1.4	39%
Norway	5.1	2.5	27.0	90 %+
Sweden	9.6	4.5	26.8	90 %+
USA	316.1	117.5	1009.0	90 %+
UK	64.1	25.7	149.0	22.8%

For the UK perspective the timber market is diverse but the supply chain still heavily relies on the key sectors of new housebuilding and home improvement for both the public and private sector [25].

1.2. How design is fundamental for optimisation

The main purpose of structural design is to provide a technical and efficient system to design a structure. This structure must resist and transmit the actions and deformations applied to it throughout its construction and working life, according to [26-29]. As pointed out by Rosenblueth [27], “Optimisation should consider not only the initial cost of the structure, but also: the benefits to be derived from the structure whilst in services; the present values of maintenance, damage and failure costs; and the probabilities that the structure might suffer damage or failure as a function of time.” Therefore, design criteria must take into account the probability that the structure will undergo acceptable levels of damage and the probability of structural failure, often referred to as probabilities of failure. A quotation from Gallagher in 1974 [26], “in contrast to analysis technology, optimal structural design technology had not yet enjoyed the protected or expected acceptance in practical design, and that it is difficult to ascertain the full range of considerations responsible for the slow acceptance of the available design technologies in the computational aspects of practical design.” This statement is still valid today; for example, the draft Eurocodes were published in the early 1990s, and their introduction has taken considerably longer than was envisaged [30]. Esteva and Rosenblueth in 1980 [28] make the following statements, “Engineering design is rooted in society’s need to optimise. It implies considering alternative lines of action, assessing their consequences, and making the best choice”. Then Esteva concludes, “Achievements of the foregoing objectives requires much more than the dimensioning of structural members for given internal forces. It implies explicit consideration of those objectives and of the problems related to a non-linear structurel response and behaviour of materials, members and connections when subjected to several cycles of high-load reversals. It implies as well the identification of serviceability conditions and the formulation of acceptance criteria with respect to them."

From Esteva and Rosenblueth's publication [28] it is clear that to gain achievements of design optimisation the engineer requires more than independent structural analysis, studies of mechanical behaviour of a structure and dimensioning of structural members for the given internal forces. It requires both a clear understanding of the roles of each of the above aspects and the overall grasp of their intimate relationship in each phase of the total design process [29]. This is where having an understanding of structural connections is vital.

1.3. Structural analysis and design

The three main objectives for design criteria are safety, performance of function and economy as described by Biggs [31]. Safety is the most important objective, as ultimate structural failure has the potential for loss of life and always involves financial losses. With this in mind, no structure can be described as 100% safe; there is always a non-zero probability of failure due to human errors during the design or construction phase; even accidental or environmental conditions may play a role. Different degrees of safety are dependent upon the nature of the structure and consequences of failure. For the intended use of a structure to be satisfied, the inhabitants or the users must have a degree of confidence over the robustness of the structure. This can be often undermined, even if the building has not suffered structural collapse but instead suffers from visible deflections, cracks in the walls / ceilings or even excessive vibrations.

The design philosophy or approach is to define the complete process, including the design criteria, in terms of the numerical analysis and design phase of the total process. Over the years there have been a number of different philosophies and design approaches, which are described within Bertero's paper [32].

Design philosophies include

- Linear elastic philosophy:
- Plastic design philosophy:
- Limit state design (LSD) philosophy: ultimate limit state (ULS) and serviceability limit state (SLS)
- Performance-based building design philosophy

Linear elastic design philosophy assumes that the design strength is calculated and restrained to yield a limit, under which the material follows Hooke's law. Structures designed by the elastic method will have considerable redundancy strength beyond that of the elastic yield. The negative feature of this design philosophy is that a ductile member will not have the redundancy quantified or utilised in a clear and detailed manner [33].

From as early as 1914, research was being conducted into quantifying the redundancy within the linear elastic philosophy [34]. This research was then further refined until the early 1950s when Horne, Greenberg and Prager presented a foundation for the new theory of plasticity [35, 36]. The plastic design philosophy will design the structure based on the collapse loads as opposed to properly factored service working loads. This approach is based on the assumption that the structural members have sufficient ductility to allow a large increase in strain beyond the yield point without any increase in stress.

Elastic-Plastic behaviour and the stress-strain relationship can be visualised by the typical stress-strain diagram, see Figure 1-2.

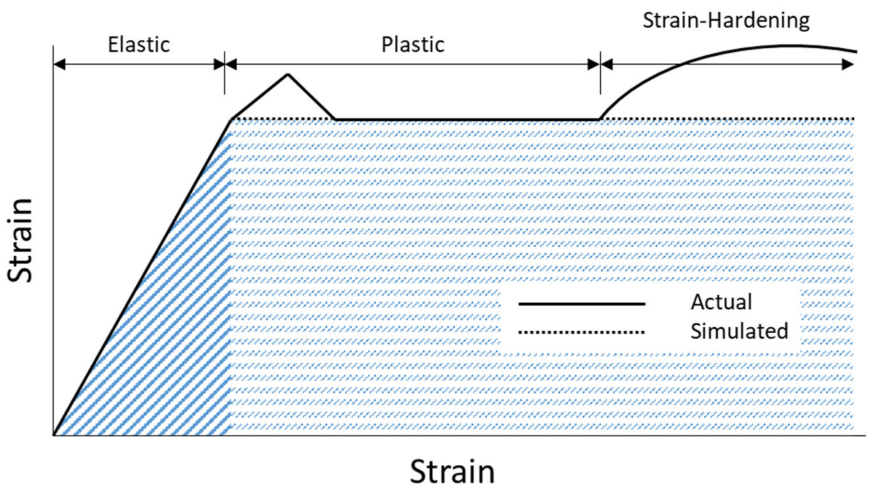


Figure 1-2 Typical Stress-Strain Diagram of Structural Steel

The withdrawn structural design guidance document from 1973 CP-112 [37] and the more recently withdrawn / non-maintained code of practice BS-5268-2 [38] are based upon permissible stress design philosophy. This approach states that stresses developed within the structure as a result of service loads not exceeding the elastic limit have factors of safety applied.

The performance-based building design (PBBD) approach is in essence, the practice of operating in terms of the end results rather than the systems or means [39]. Heidkamp defines it as: a structure shall be designed in such a way that it will function in a reliable manner and within an economical way to attain the required performance [40]. These statements do not say anything about the ways and means of building, e.g. types of material, thickness, dimensions and size of building components or methods of assembly, but instead clearly state the required end results. For further information on performance-based building design please see Appendix A.

The principle of limit state design is to define limits beyond which a structure no longer performs its designed function satisfactorily. The design methodology uses two terms: Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

Failure of ULS can be described as a condition of a structure beyond which it no longer fulfils one or more of the relevant design criteria and will most likely lead to collapse or an unsafe environment. These are defined as:

- equilibrium (EQU): loss of static equilibrium
- strength (STR): internal failure or excessive deformations
- geotechnical (GEO): failure or excessive deformation of the ground where the strengths of the soil or rock are significant in providing resistance
- fatigue (FAT): fatigue failure of the structure or structural members

For the Serviceability Limit States (SLS), failure is defined as:

- Deformations affecting the cosmetic appearance or even the perceived safety of the structure, or indeed hindering the function or the comfort within the structure
- Vibrations that may cause discomfort or reduce the usability of the structure
- Damage that is affecting the cosmetic appearance, reducing the durability of the construction materials

The limit state design philosophy is adopted fully within the Eurocode framework.

1.4. The Aims of this Thesis

In order to fulfil the AEC sector's needs and to support the sustainability agenda, the more that the built environment is constructed out of wood fibre rather than less environmental alternatives, the better. The move from the withdrawn permissible stress design philosophy to the limit state design parametric approach will allow for better design optimisation. Bearing this in mind, the aims of this thesis are:

a) To ease the specification of structural timber within the AEC sector, in order to increase the UK market share for structural timber.
b) To aid the transition from BS 5268-6.1 to EC5
c) To create user-friendly and time-efficient timber structural calculations that will allow design optimisation.

These aims are addressed throughout the body of this work.

1.5. The Objectives of this Thesis

From the Thinking outside the box report by Harker [41] and the finding of the literature review [Chapter 2], it was identified that additional evidence and detail was required to categorise the barriers to Eurocode 5 (EC5) adoption and subsequent structural timber specification. A digital online survey was designed for this purpose [Chapter 4]. The findings from the industry research survey were used to inform the objectives which in turn help to flesh out the main aims of this research.

Objectives:

1. To do an industry survey of structural engineers that gives further clarity in identifying barriers for timber specification.

Reduce barriers for timber specification and connection design by:

2. To create and deliver educational material of current research, for the purpose of increasing the level of knowledge of structural timber for both university students and practising engineers.
3. To reduce the complexity of EC5 through automation of timber connections:
    a. Identification of gaps and shortcomings within the existing automated structural timber connections calculations.
    b. Identification of the ideal software platform that can offer the best route for impact.

c. Creation of automated structural timber connection calculations within the identified platform.

4. To Create case studies demonstrating the advantages of parametric methodology within EC5 timber connections.
5. To Create case studies demonstrating the benefits of a transition to EC5 through the ability of optimisation.
6. To identify and utilising routes for current research to be implemented into the AEC sector.
7. To develop a proof of concept for BIM integration using multi-dimensional data fitting.

1.6. The Structure of the Thesis

The work presented within this thesis has been categorised and summarised into the nine chapters. The interrelationship between each stage of the work has been highlighted in Figure 3-2. The content of each chapter is summarised as follows:

Chapter 2 Literature review

> This section looks at the current structural timber sector and its needs around standardisation and mass-customisation. It touches upon the Eurocode methodology for the calculation of timber connections. It then concludes with a discussion on automated design within a digital environment, including multi-dimensional data fitting as a tool.

Chapter 3 Methodology

> This thesis used a wide range of methodologies including an industry survey, code compliance software creation, case studies and multi-dimensional data fitting as a tool for creating BIM-ready equations.

Chapter 4 Barriers to structural timber use: a survey

> The knowledge gap is identified and addressed, giving further evidence and detail for the AEC sector reluctance to transition to Eurocode 5: Design of timber structures.

Chapter 5 Code Compliance timber connection tools

This section discusses the creation of the Code Compliance tools. Two timber connection calculations were created and released to the public. The usage data shows a high level of uptake among UK engineers.

Chapter 6 Case studies

The case studies shown here demonstrate the creation timeline for the newly Code Compliance timber connection calculations described above. In addition, the case studies highlight the power of this software in specifying timber for the most complex use cases.

Chapter 7 BIM-ready equations

This section discusses the creation and implementation of the BIM-ready equations, highlighting all of the steps taken, including the important role of the Code Compliance tools. The implementation of the BIM-ready equations results in an Automated Code Compliance timber connections calculations, within a BIM environment.

Chapter 8 Conclusions

This section brings together the body of work by identifying how each of the objectives have been addressed.

Chapter 9 Future work

The thesis is rounded off by highlighting the possible ways that this work can be expanded upon in the future

Chapter 2. Literature Review

2.1. The current structural timber sector

2.1.1. Fragmented sector

For simplicity, the timber market supply-chain structure can be subdivided into five levels, from harvesting/importing to retailing, see Figure 2-1 for the list. Research conducted for the timber merchants market report UK 2011-2015 identifies that most companies cross over into more than one of the levels of this structure. For example some timber merchants now have manufacturing capabilities for engineered wood products [25].

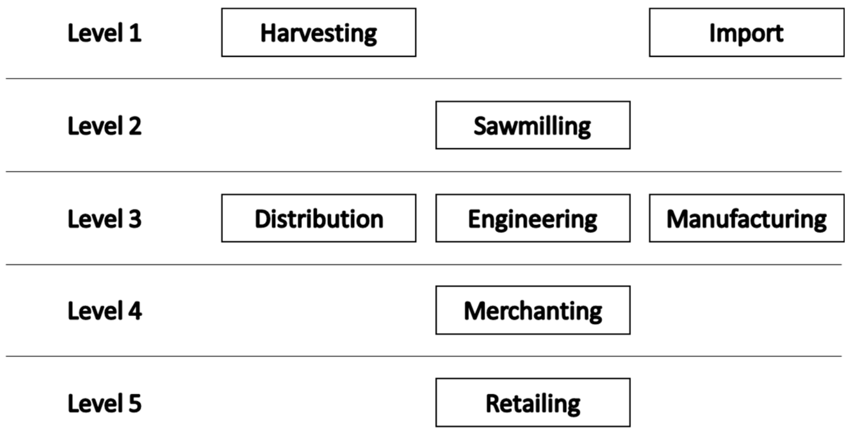


Figure 2-1 Levels in the timber supply chain

The timber sector is highly fragmented throughout the five levels of the supply chain, from resource ownership to retail [25, 41, 42]. There are some larger national merchants' firms but the majority of the sector operates on a more local or regional basis, with the smaller firms accounting for around 50% of the market share. This is illustrated within Figure 2-2.

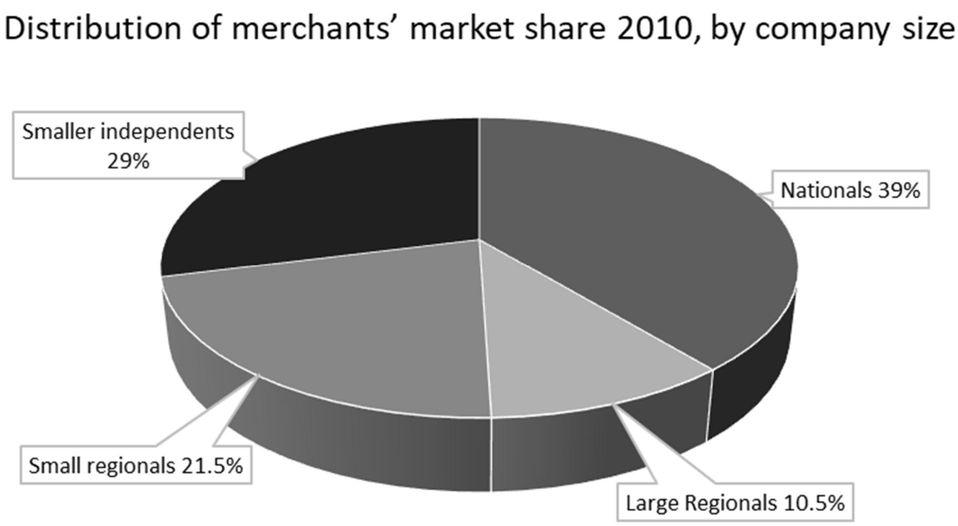


Figure 2-2 Distribution of timber merchants' market share 2010, by company size, information from [25]

There are several significant suppliers of timber frame housing, i.e. Robertson, Stewart Milne, etc., but it is estimated that there are approximately 100 suppliers in total. The fragmentation of the timber sector is also apparent within the customer base for timber merchants, see Figure 2-3.

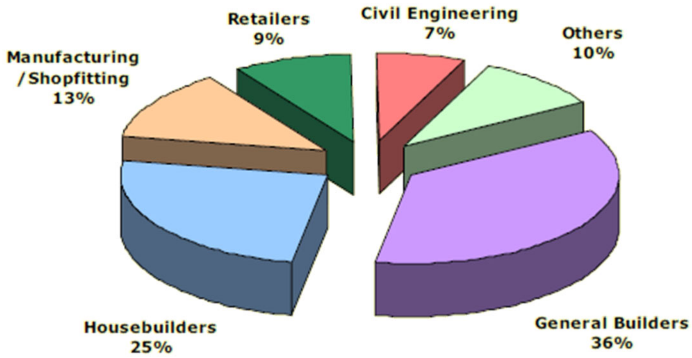


Figure 2-3 Timber Merchants sales distribution by customer group 2010 – by volume m^3, from [25]

2.1.2. Need for improved levels of standardisation and supply chain integration

A definition of standardisation for the purposes of this research is the extensive use of common products, systems or processes which bring numerous benefits: lower construction costs, regularity, interface precision, lower maintenance costs and more scope for recycling, This has been summarised from Gibb [43]. From the finding of 'The Shaping the future of construction report' [44], there are misconceptions of standardisation within the AEC sector about construction quality, lack of personalisation in design and final cost. This acts as a barrier for the use of standardisation techniques and components. These concerns with standardisation have already been addressed within other manufacturing industries, but for mitigation within the construction context, options are available. For example to mitigate against the limited customisation and additional risk of committing to a particular supplier, it is proposed by Renz [44] to develop industry-wide standards on component dimensions and connections that have the added ability of mass-customisation options.

Research work undertaken by Dubois [45] identifies that it is increasingly common for firms to collaborate with their supply chain network as a means to improve company performance because of network effect. For example, firms adapt to one another in terms of technical solutions, logistics or administrative routines. But they then continue to say that within the AEC sector this network effect is less than usual. The reasons for, "the absence of adaptation are found to be the current focus on the efficiency of individual projects and the competitive tendering procedures used. It is concluded that these characteristics are having a hampering effect on both efficiency and innovation in the industry today".

Despite the inherent benefits of using timber for construction, timber use is relatively limited. This can be attributed to a range of factors, from the supply chain to the inherent physical and mechanical complexity in the engineering modelling of a natural material such as wood. The major issue appears to be the lack of knowledge amongst the technical community and thus an ensuing lack of confidence in its performance. It is telling that this lack of expertise seems to appear globally, in countries with widely different markets, building traditions and levels of technical expertise [46-51]. As such, timber's potential remains underutilized; this is particularly unfortunate today, where the technological advances of the past two decades have expanded significantly what can be achieved with structural timber systems.

The AEC sector is often accused of being resistant to innovation and the introduction of new concepts [52]. This coupled with the fragmentation of the timber sector means that significant improvements in mass-customised standardisation and supply chain integration are necessary.

2.1.3. Mass customised approach

Supply chain integration can be improved and simplified by taking a mass customisation approach to product design. In many respects, the non-timber construction sector within the UK is several steps ahead of their timber counterpart, with better implementation of Building Information Modelling, Mass customisation and Design for manufacture and assembly. This is primarily a consequence of the fragmentation of the structural timber supply chain. There is a major disparity in investment into research between the steel/concrete and the timber sector. This disparity and fragmented supply chain results in the following shortfalls within the UK timber industry [41]:

- The quality and accessibility of data to support modern wood building solutions and their associated design processes.
- Established standardised design and detailing and communication of best practice.
- Effective dissemination of academic research to practising structural engineers.

2.2. Mass customisation approach for the timber AEC sector

For the timber construction industry to compete against the steel and concrete sector, the timber construction industry must engage with Modern Methods of Construction (MMC), which can be described as methods that improve both products and processes, as defined within the barker review [53]. This review also outlines the barriers to MMC. From the Architecture, Engineering and Construction (AEC) sector perspective, greater uptake of technology is considered fundamental. One of the keys to this improvement is having a Mass Customised approach (MC) [54], which is simply defined as delivering a customised requirement at an industrial scale using standardised components and construction methods [55, 56]. An early example of MC within the AEC Japanese sector is discussed within a Planning review document by Kotler [57], in which he gives credit to Davis for coining the seeming oxymoron MC in Davis' book Future Perfect [58].

MMC & MC will predominantly involve various levels of offsite construction and design for manufacture and assembly (DfMA), which is a process where the focus is on the ease of manufacture efficiency and the onsite assembly. This will help deliver competitive pricing and lead times [59]. This is a large undertaking. Figure 2-4 shows a simplified version of the various systems, sub-systems and components that may be used within a house construction. This research will be focused on timber connections using dowel type fixings, i.e. nailed, screwed and bolted, as it has been identified that the connections within a timber superstructure are the critical points. This is further explored within Section 2.4. Mass customisation permits the use of many different types of components and configurations to suit the project in hand. Engineers need good quality information in an accessible form to maximise the advantages of mass customisation.

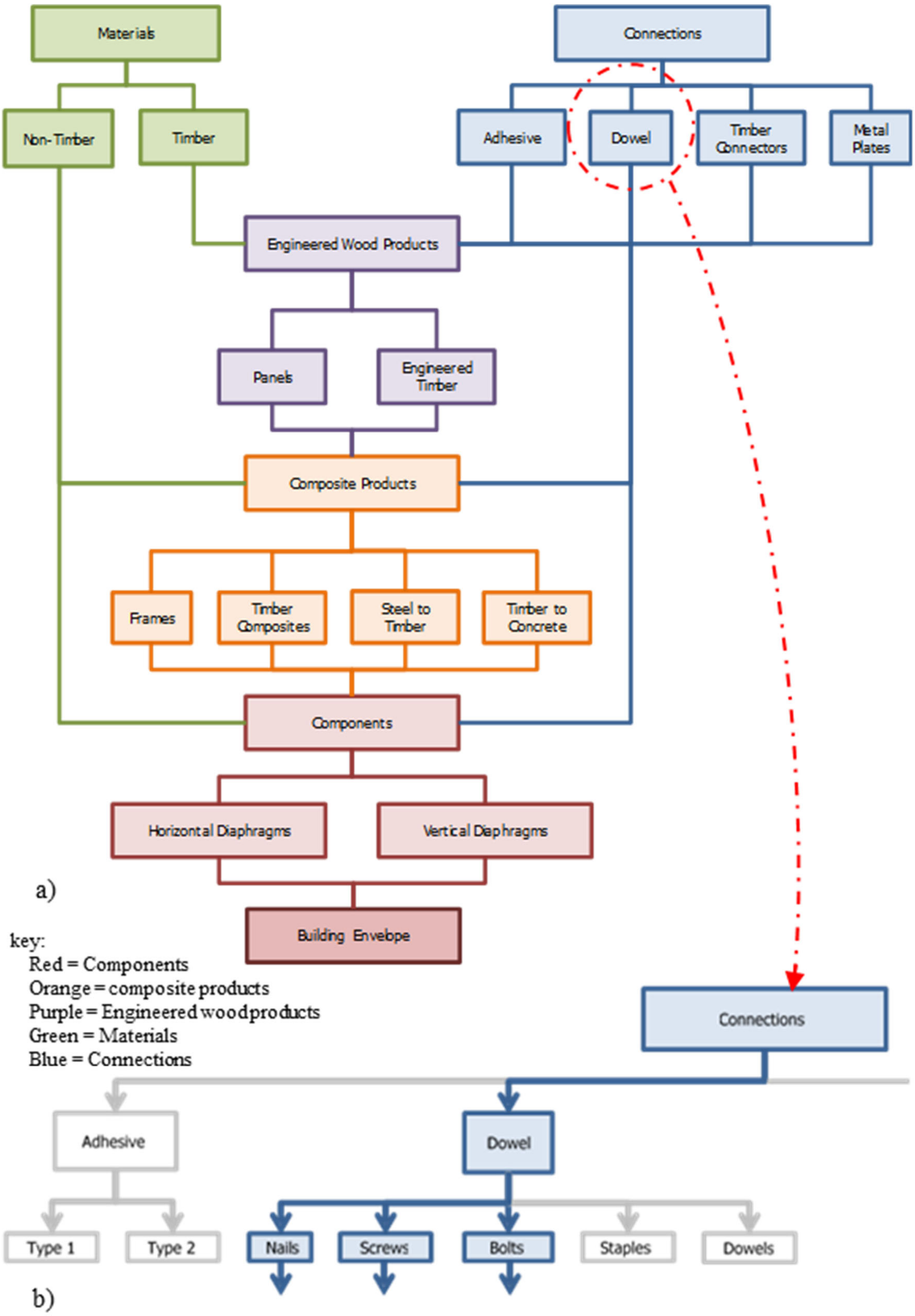


Figure 2-4 Simplified view of the various systems, sub-systems and components that may be used within a house construction (a). The focus is on the nail screw and bolt connections (b)

2.2.1. There are three capabilities that make Mass Customisation work.

For an organisation to succeed in implementing an MC business approach, Fabrizio Salvador stated that three fundamental organisational capacities have to be developed [60].

These are defined as:

- Solution space development:
    - The mass producer seeks to fulfil the central needs of their customer base while offering a limited number of standardised products.
    - Any business intending to adopt mass customisation will have to be able to understand what the individual needs of its customers are and then identify the product specifications on which customers' needs diverge the most. With the addition of boundary conditions, this provides the scope for defining the proposed offerings. (Description summarised from [61].)
- Robust process design:
    - Being able to offer flexibility in design without impeding on the business's operations and supply chain [56].
- Choice navigation:
    - Customer frustration can be mitigated by simplifying the complexity of choice and the selection method. Otherwise, there is a risk of the "paradox of choice". This is where being exposed to too many choices can be overwhelming, reducing the customer value rather than increasing it [62].

MC can reduce the barrier of timber connection calculation by creating a limited range of pre-approved connections. However, as part of the robust process design capability identified by Salvador, the main barrier to MC in timber engineering is still the connection design. Often it is the connection design that is the limiting factor as opposed to the member design, this is explored further in Chapter 2.4. This process can be simplified through the automation of design solutions.

2.3. Eurocodes

One of the aims of the introduction of Eurocodes was to harmonise the technical specifications of structures so that the limit state design methodology is the same for any material. The framework for how the Eurocodes are structured can be observed in Figure

2-5. The core codes provide the principles and requirements for reliability, safety, serviceability and durability of structures, actions on structures, geotechnical design principles and design considerations for seismic events. The material codes provide the design and detailing rules for all types of building and civil engineering structures and also for the primary construction materials. For a complete set of Eurocodes listed, see 0. Note that every national standards body may produce its own national annexe for each part of the Eurocode, providing nationally determined parameters.

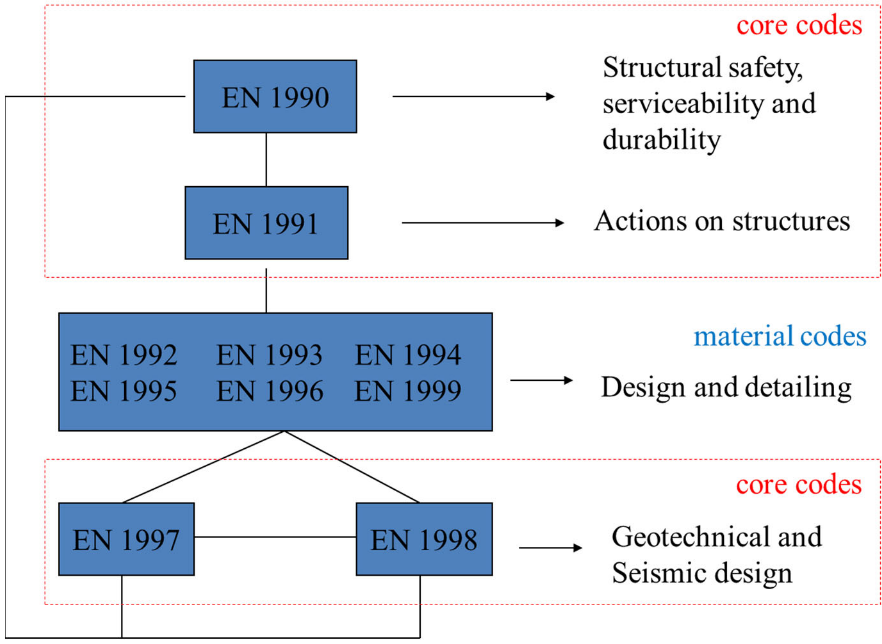


Figure 2-5 European structural code of practice

2.3.1. Precursors to Eurocode 5

The British standards institution first published guidance documents for a Code of Practice for structural use of timber in 1952 CP 112. This was based upon permissible stress design. Amendments to the 1952 version came in 1967 with the introduction of strength classes as a means of simplifying the specification of structural timber. In 1971 the United Kingdom was in a transition period of converting from imperial to the metric unit system and as a result CP 112:1971, the so-called 'metric unit' was published. The CP 112: 1971 metric unit revision was eventually withdrawn in February of 1988.

The British standards institution rewrote the codes of practice and in August 1984 published BS 5268-2, structural use of timber. This was a code of practice for permissible stress design, materials and workmanship. The first edition contained grade stresses that use the fifth percentile lower exclusion values of strength, in contrast to CP 112 which contained first percentile values. Further editions and subsequent revisions were released and are summarised below:

- Second edition 1988; Information for species of tropical hardwood, grades of plywood and other structural boards.
- Third edition July 1991; Amends and adds to the contents of the 1988 edition.
- Fourth edition August 1996; Incorporates some of the European Committee for Standardization (CEN) standards on materials, to ease the specification and supply of materials during the period of coexistence of BS 5268 and Eurocode 5 [63].
- Fifth edition March 2002, with an amendment on 31st December 2007; Technical changes only.

The UK transitioned from the British Standards to Eurocodes with the introduction of BS EN 1995-1-1 Eurocode 5: Design of timber structures – Part 1-1 General – common rules and rules for buildings. This was first published 15th December 2004, using the Limit State Design yield moment design model introduced by [64-66]. This standard then had a number of amendments: 31st July 2006; 31st January 2009; 31st May 2014, which were implementations of CEN amendments.

2.3.2. The UK's transition to Eurocode 5

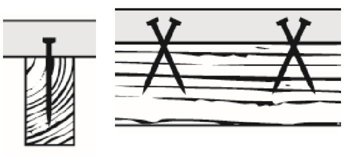
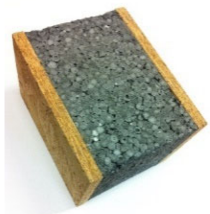
The draft Eurocodes were published in the early 1990s; their introduction has taken considerably longer than was envisaged, as reported by Brooker from BSI in 2015 [30]. A report by BDO on behalf of the Timber-Trade-Federation [41] starts to identify the barriers for EC5 adoption. The priorities identified are summarised as:

- Professional education and continuing professional development around modern wood building solutions.
- Continuing to develop and communicate the positive environmental benefits of modern wood building solutions, to counter the high degree of activity in this area by the steel and concrete lobbies.
- Low quality and accessibility of data to support design processes for modern wood buildings solutions, including addressing the supply chain control over information.
- Increased publicity around positive case studies for modern wood buildings solutions with lots of examples of good detailing for modern wood buildings solutions.

As little work has been conducted, it is seen that further and more detailed, evidence for the barriers of EC5 adoption will be required. This is explored further in future chapters.

The move from the withdrawn permissible stress design philosophy (BS) to the limit state design approach (EC5) allows for any of the variables used within the calculation to be changed. This permits greater freedom in design and product optimisation. This is in contrast to the method adopted within the withdrawn British Standards of lookup table of predefined solutions. This simplifies the use of new engineered timber solutions such as:

- **Timber Concrete Composite** combines timber and concrete, utilising the complementary properties of each material, see Figure 2-6a.
- **Structural Insulated Panels** consist of an insulating layer of rigid core sandwiched between two layers of structural board, see Figure 2-6b.
- **Engineered Joists** sandwich the web between the top and bottom flanges, creating an "I" shape. The flanges can be made from LVL (laminated veneer lumber) or solid wood, see Figure 2-6c.
- **Cross Laminated Timber (CLT)** is an engineered wood product consisting of a number of layers of wood glued at alternating angles to one another, providing a structural two-way spanning timber panel that can be used to form walls, roof and floor panels, see Figure 2-6d.
- **Dowel Laminated Timber** is fabricated from softwood timber posts connected with hardwood timber dowels. It can have a nailed or interlocking variant and is also referred to as brettstapel, see Figure 2-6e.
- **Glued Laminated Timber** is made by glueing pieces of timber together to make larger sections. It is a way of manufacturing timber elements that cannot be easily sourced in solid timber, due to the large size or unusual shape, see Figure 2-6f.


a: Timber Concrete Composite *b: Structural Insulated Panel* *c: Engineered Joist*



d: Cross Laminated Timber *e: Dowel Laminated Timber* *f: Glued Laminated Timber*

Figure 2-6: Examples of engineered timber solutions

At the time of writing, structural engineers working with timber within the UK are still going through a period of transition from the withdrawn non-maintained Permissible Stress design within BS 5268-2 by Limit State Design, Eurocode 5. This transition brings timber design in line with other materials such as steel and concrete. Eurocode 5 contains only the essential rules and general formulae for design. It is an analytical approach that benefits from a transparent method of calculation. This allows users to change variables in order to achieve an efficient structure. It also allows for empirically validated strength values for both the material and the fasteners, which is directly applicable to the implementation of mass customisation (optimisation of standard components). The methods of calculation of timber connections used in these codes are discussed further in the next section.

2.4. Structural Appraisal of connections using dowel type fasteners

An interesting way of looking at structural engineering is described by Thomas McLain: "a structure is a constructed assembly of joints separated by members" [67]. That is to say, when designing a structure the joints are generally the critical factor of any engineered structure. The strength of the connectors in the joint will normally dictate the strength of the structure; their stiffness will greatly influence its overall behaviour and member sizes will generally be determined by the numbers and physical characteristics of the connector rather than by the strength requirements of the member material [2]. Research work conducted by Foliente on timber buildings identifies that it is often the inadequate connection design that is the primary cause of damage after the structure is exposed to extreme wind or earthquake events [68].

Key points:

- Joints are crucial points in many timber structures because they can determine the overall strength and performance of that structure.
- The length of structural timber is generally shorter than the required spans and as a result splicing or composite structures (e.g. trusses) must be used.

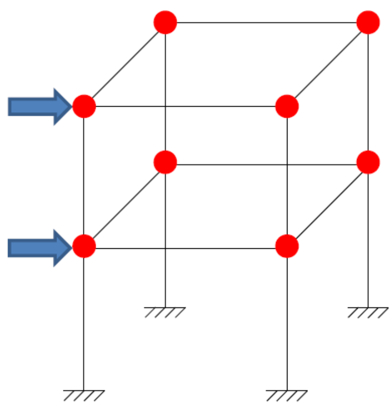


Figure 2-7 Joints and system

- Forces between members are most often transferred through lap joints, either by adhesives (glues) or by dowel-type fixings (nails, bolts, screws, dowels or nail plates).

The calculation method for connection resistance within EC5 uses the Johansen's equations for dowel-type joints.

2.4.1. General theory and assumptions

Johansen first published his theory for the lateral load-carrying capacity of dowel type fastener timber connections in 1941 [64] in Danish. He published a shortened English version in 1949 [65], and this was then progressed by Möller [69]. His theory was based on the assumption that the connector and the timber (or wood-based material) being connected will behave as essentially rigid plastic materials in accordance with the strength-displacement relationships. So when the dowel or timber deforms, the lateral load-bearing capacity remains unchanged. Figure 2-8 shows how this assumption (black line) compares with the actual timber behaviour (green dashed lines). An additional assumption is made that failure is not caused as a result of insufficient spacing between the fixings and the end distances, and the minimum fixing spacing, edge and end distances for these assumptions to be valid can be found in EC5. The experiments for the 1949 paper consisted of a bolted connection. The assumed behaviour composed of two effects:

- Dowel effect of the bolt: which is influenced by yielding of the dowel and timber embedment.
- Tension effect of the bolt: resistance of friction and tension between the surfaces.

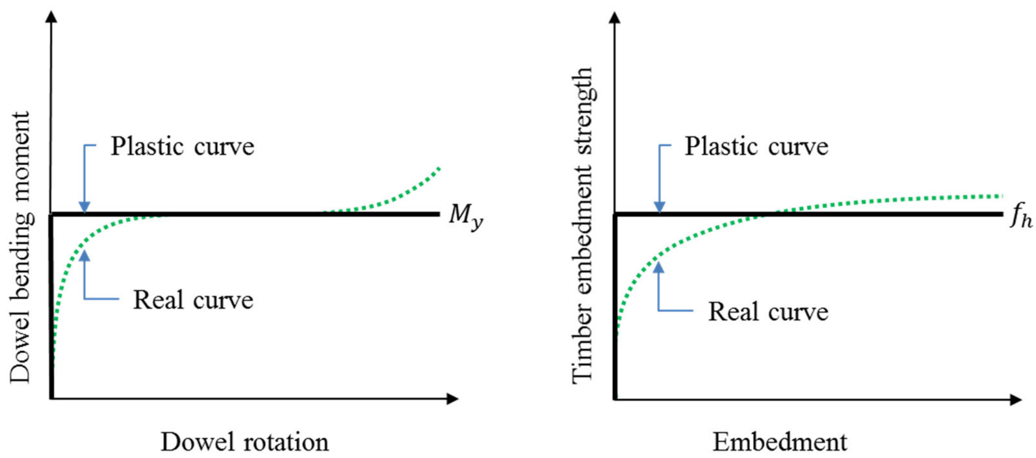


Figure 2-8 Strength-strain relationships used for dowel connections [70]

He assumed no axial load occurs, so no friction between the bolt and timber members were considered. Over the years other researchers have carried out tests to validate these equations [69-75]. By minimising the effects of friction between the members and contributions from the rope effect, a good similarity was found between the experimental tests and calculated values.

2.4.2. Development of Johansen's equations

In Johansen's 1941 paper [64], he stated that tests would continue with nailed joints, which could essentially be dealt with from the same point of view as joints already investigated. Unfortunately, these tests, conducted over the period 1940 to 1945, were postponed indefinitely as he turned his attention onto other materials such as steel and concrete. His work was later expanded upon by Meyer [76]. However, it was not until the late 1970's that Larsen (with permission from Johansen) carried out the missing tests. Larsen's paper [66] concludes that Johansen's theory for the dowelled joints based on the theory of plasticity is also suitable for the determination of the load-carrying capacity of nailed timber joint connections. Larsen demonstrated that the load-carrying capacity of the joints is about 20% greater than Johansen's predictive theory. This is attributed to axial force and the resulting friction between the timber members created when the joint begins to deform, see Figure 2-9. Additionally, the head causes restraining of the nail that may be greater than axial withdrawal in the point side resistance, see Figure 2-10. Therefore, in order to increase the accuracy of the Johansen's yield theory being used within EC5, two correction terms were introduced:

- the axial force in the fastener, based on the work of Larsen [66] and Hilson [77].
- The friction term, as proposed by Hansen [78].

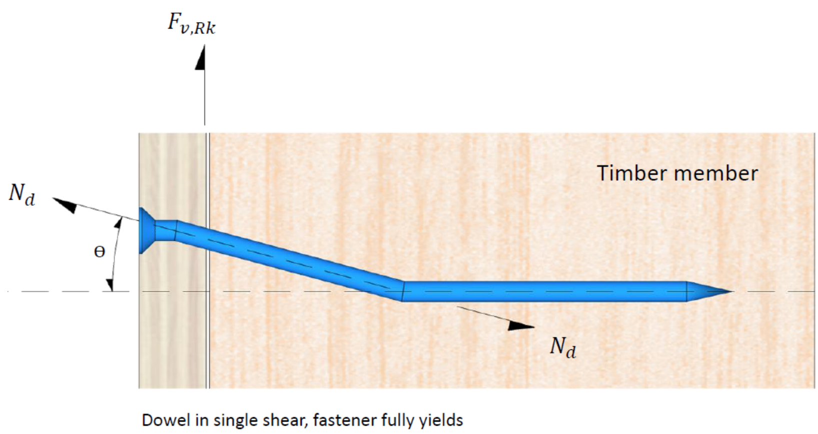


Figure 2-9 Nail deformation

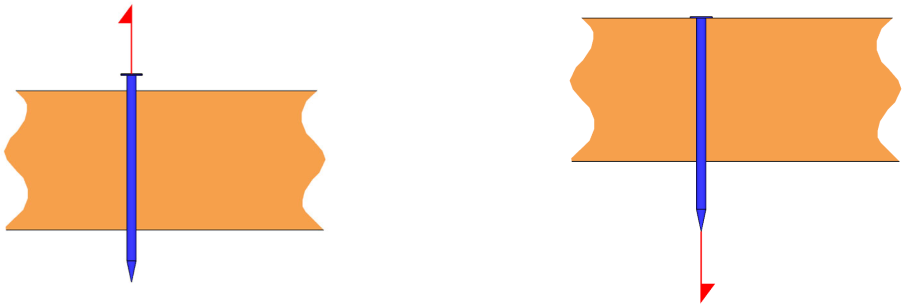


Figure 2-10: Nail withdrawal and pull through

Larsen's work also investigated the effect of nails placed tangentially to annular rings, see Figure 2-11. Even with all the additional work and development into the yield moment design model by other authors, these equations are still referred to as the Johansen's equations see Figure D- 11 Johansen's timber to timber single shear equations

and Figure D- 12 Johansen's timber to timber double shear equations

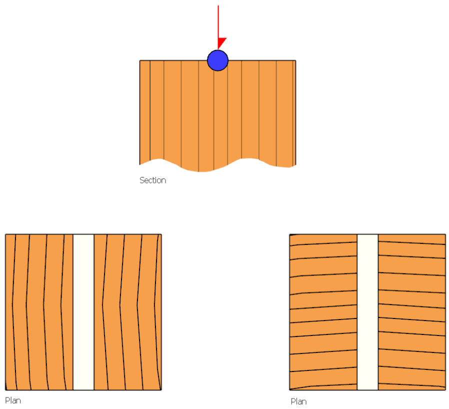
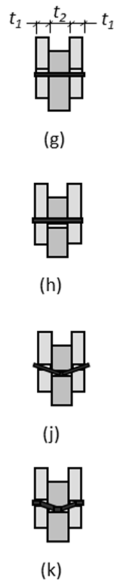


Figure 2-11: Regular and tangential placement

$$F_{v.Rk} = \min \begin{cases} F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d & \text{(a)} \\ F_{v,Rk} = f_{h,2,k} \cdot t_2 \cdot d & \text{(b)} \\ F_{v,Rk} = \dfrac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right] + \dfrac{F_{ax,Rk}}{4} & \text{(c)} \\ F_{v,Rk} = 1.05\dfrac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & \text{(d)} \\ F_{v,Rk} = 1.05\dfrac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_2^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & \text{(e)} \\ F_{v,Rk} = 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} & \text{(f)} \end{cases}$$

Figure 2-12 Johansen's timber to timber single shear equations



$$F_{v,Rk} = \min \begin{cases} F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d & \text{(g)} \\ F_{v,Rk} = 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d & \text{(h)} \\ F_{v,Rk} = 1.05 \dfrac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^{2} \cdot d}} - \beta \right] + \dfrac{F_{ax,Rk}}{4} & \text{(j)} \\ F_{v,Rk} = 1.15 \sqrt{\dfrac{2\beta}{1+\beta}} \sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} & \text{(k)} \end{cases}$$

Figure 2-13 Johansen's timber to timber double shear equations

The resulting equations used in EC5 rely upon three main parameters of influence for the load-carrying capacity and behaviour of joints with dowel type fasteners, which are:

1. the bending capacity of the dowel or yield moment.
2. the embedding strength of the timber or wood-based material.
3. the withdrawal strength of the dowel.

Please see 0 and D for further explanation and context. As a result, the current calculation method is not straightforward and does not lead itself to hand calculations on account of the complex and repetitive nature of the developed calculations, see 0 for an example of connection calculations.

2.4.3. Johansen's equations, state of the art

Further development of Johansen's equations still continues, by identifying weakness, proposing amendments or new equations which only leads to improving accuracy. Some of the more recent proposed amendments are summarised here.

As a result of work conducted by Jockwer, Steiger and Frangi [79] in 2015, a lower design value for the material parameter used within EC5 design equations for connections loaded perpendicular to grain has been proposed.

Blass [1] brings attention to the advantages of using inclined self-tapping screws with continual threads as illustrated in Figure 2-14. This leads to an increased stiffness with increasing angle. Also, a new equation is presented for lateral load-carrying capacity of inclined screws with continual threads. The equation is for failure mode 'f', see

************** **Error! Reference source not found.Error! Reference source not found.Error! Reference source not found.Error! Reference source not found.**, and this applies to single shear timber to timber connections with two plastic hinges forming.

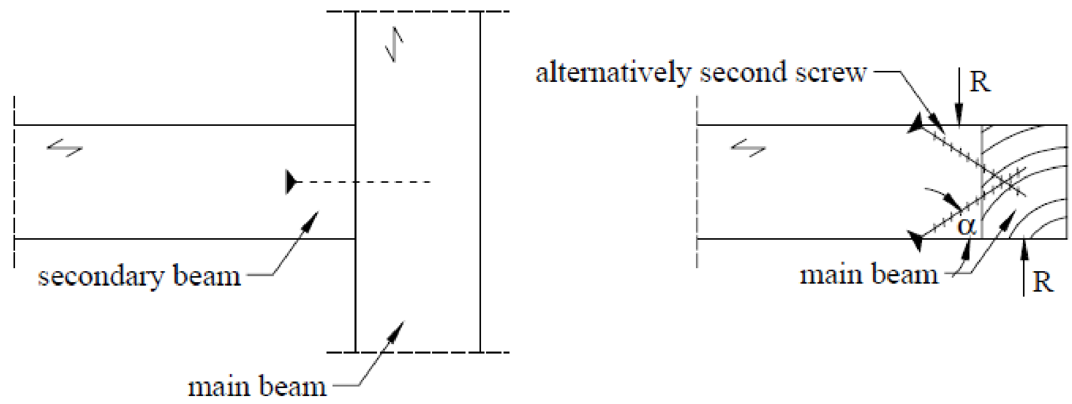


Figure 2-14 Main to side member timber connection [80]

Predominantly self-tapping screws are used in solid timber and laminated timber products, with the main functions of load transfer between elements or reinforcement of timber members. The withdrawal properties have been analysed by a number of authors, primarily focusing on strength. A report by Blass in 2006 [81] sets the foundation for the theory where Pirnbacher [82] defines the basic parameters, and then Frese 2010 [83] proposes equations for calculating the withdrawal capacity of the self-tapping screws. This process was further advanced when Hübner 2013 [84] introduced hardwood to the equations. In 2012 the behaviour of self-tapping screws into the end grain of the wood, which is currently not considered within EC5, was investigated by Ellingsbo [85]. They did, however, find that the EC5 expression for axial withdrawal of the point side thread shows a good general agreement with characteristic values obtained in experimental tests. Ellingsbo comments that for longer effective lengths overestimation of the failure mode could occur, so further tests are required. The main body of the work presented by [81-85] focuses on withdrawal strength rather than stiffness, with the investigation primarily on solid timber. Therefore these models are only applicable for timber products tested, and using these models on other timber products may cause unreliable results. An example of this is the complexity of Cross Laminated Timber (CLT) with thin lamellas in different directions, see Figure 2-15.

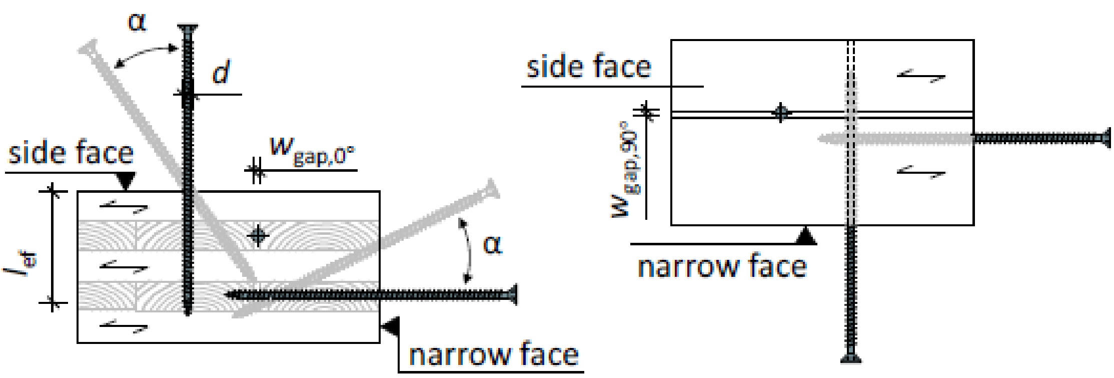


Figure 2-15 Essential influencing parameters of axially loaded screw connections in CLT. from [86]

Uibel and Blass [87] in 2007 developed a reduction factor for the possibility of a screw intersecting any gaps and intersecting zones of two layers with different access to grain angles. In 2015 Ringhofer, Brandner and Schickhofer [86] provided an alternative to the reduction factor of Uibel and Blass by introducing a new universal approach for calculating the withdrawal properties of self-tapping screws within laminated timber products. Blass and Colling 2015 [88] reviewed the findings from seven different research studies based upon 1588 laboratory tests. For dowel connections with a double plastic hinge failure mode, an additional effect of slenderness is identified with a modified equation for M_y slenderness proposed:

$$\mathrm{M_y} = \frac{\mathrm{f_{y,ef}} \cdot \mathrm{d}^3}{6} \tag{2-1}$$

$$\mathrm{f_{y,ef}} = \begin{cases} \dfrac{0.9 \cdot (\mathrm{f_y} + \mathrm{f_u})}{2} & \text{for } \mathrm{f_u} < 450 \text{ MPa} \\ 0.9 \cdot \mathrm{f_u} & \text{for } \mathrm{f_u} > 450 \text{ MPa} \end{cases} \tag{2-2}$$

where:

d is the dowel diameter,

f_y is the fastener yield strength

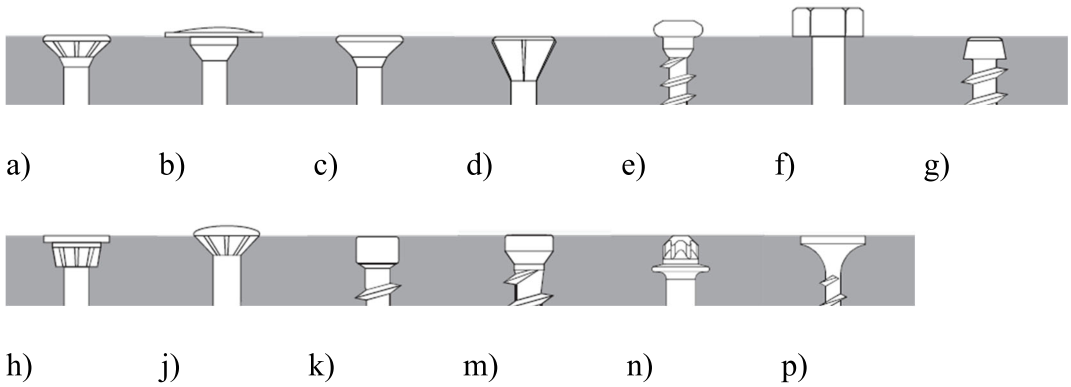
f_u is the fastener tensile strength.

This can increase the load-carrying capacity of these connections in the region of 25% in comparison to the current EC5 model.

The Johansen’s equations have come a long way as research is ongoing. The more the research continues the more complex the calculations become. One of the current research focusses is in increasing the functionality of modern timber screws.

2.5. The complexity of the modern timber screws

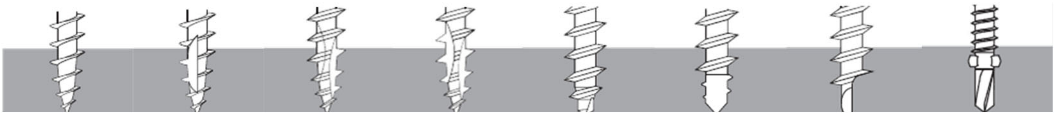
Barander states that “CLT and self-tapping screws have strongly dominated the latest developments in timber engineering” [89], which along with the environmental impact has helped to increase the market share of timber structures within the building sector of European countries like Austria, Germany and United Kingdom [90-92]. As a result, wood screws have become highly specialised and technical, and there are many different designs and parameters that manufacturers can modify to adjust the screw performance and capabilities. Some of these are head diameter and functional design, shank diameter, the ratio of outer thread diameter to the inner diameter, thread pitch, effective embedment depth, effective diameter, smooth shank penetration into the member containing the point of the screw and different design requirements depending upon the effective diameter. These parameters all can vary the uses of a fixing. An example of this can be seen in the Rothoblass screws for wood catalogue [93]. Here are some of their products: Figure 2-16 for screw head variations; **Error! Reference source not found.** for screw point variations; Figure 2-18 for screw thread variations.



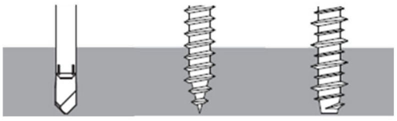
Key for Screw head options, Figure 2-16

- a) Countersunk Smooth
- b) Countersunk with Ribs
- c) Countersunk 60°
- d) Cone-Shaped
- e) Under-head Ribs
- f) Convex
- g) Bugle
- h) Bush
- j) Cylindrical
- k) SFS Cylindrical
- m) Round
- n) Hexagonal
- p) Wide

Figure 2-16 Screw Head, images from [93]



a) b) c) d) e) f) g) h)

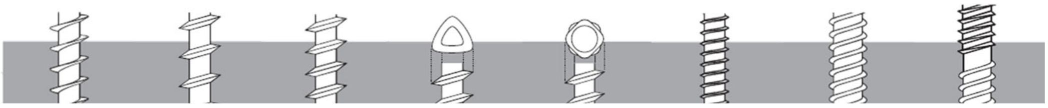


j) k) m)

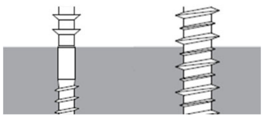
Key for Screw point options, **Error! Reference source not found.**

a) Sharp
b) Sharp 1 cut
c) Sharp 2 cut
d) Sharp 4 cut
e) Bleeder
f) Drilling
g) Hook
h) Wood - Steel
j) Steel
k) Classic wood
m) Cement

Figure 2-17 Screw points, images from [93]



a) b) c) d) e) f) g) h)



j) k)

Key for Screw thread options, Figure 2-18

a) Asymmetric
b) Regular fast
c) Regular slow
d) Tilobular
e) Quadlobular
f) Fine, for steel
g) Classic wood
h) Bugle
h) Metric + Wood
j) Double spacer
k) Hi-Low

Figure 2-18 Screw threads, images from [93]

A list of material and coating available is given as follows,

- Carbon steel + Galvanic zinc coating
- Carbon steel + Duro-coat
- Carbon steel + Organic zinc coating
- Carbon steel + Revodip
- Stainless steel; AISI 410 / martensitic
- Stainless steel; AISI 304/A2
- Stainless steel; AISI 316/A4
- Bi-metal stainless steel + carbon steel; External use
- Phosphate steel; Drywall

All the information about Rothoblass screws for wood catalogue is quoted from [93]

The complexity of modern screw fixings introduces new challenges for the structural engineer. The British Standards design approach which incorporates look-up tables is not adequately equipped to account for these advanced fixing variations. The Eurocode design method does allow for the connection effect of such fixing variations to be taken into consideration. These calculations, however, are laborious and not easily accessible for an engineer often working within tight time constraints.

Possible ways to overcome this barrier are by the adoption of the parametric calculations used within the Eurocodes in either a Code-Compliance calculation tool or an Automated Code-Compliance tool within a BIM environment. Both of these approaches by the very nature of the parametric method allow for the mass customised methodology (MC).

2.6. Automated design

2.6.1. Software / ICT

Continued development of new information and computer technology in recent years has allowed and is in part driving a fundamental transformation of the AEC sector. More powerful computer hardware and sophisticated software tools are allowing for all construction disciplines to interact together at all stages of the structure's life cycle, from design to demolition.

2.6.2. Building Information Modelling (BIM)

Within the realm of the Architecture, Engineering and Construction (AEC) sector, over the last two decades, the term Building Information Modelling (BIM) has had many different definitions by a number of authors. Perhaps the most common definition is by the National Building Information Model Standard Project Committee, according to which BIM is "a digital representation of physical and functional characteristics of a facility" [94]. The core idea of BIM as an information resource used together by several AEC professionals is common [95] and goes back to the first years of research in AEC computing [96]. More recently, the advances in computing power and software development and the adoption of software by all AEC disciplines have led many to view BIM as a process [97] or activity. Sometimes people differentiate between the Building Information *Model* (the resource) and Building Information *Modelling* (the process or activity) [98]. The promise of BIM is both significant and wide-ranging. One of the standard textbooks on the topic [98] lists a number of benefits covering the entire lifecycle process, from pre- and post-construction to design and fabrication. This benefits almost everyone within the construction process. This ranges from standard activities such as architectural design [99, 100] and cost estimation [101, 102] to more specialised aspects such as energy consumption [103] and labour productivity [104].

The UK has attracted a great deal of attention by being the first country to introduce a mandatory implementation of BIM level 2. For England and Wales, this began in April 2016 and for Scotland April 2017.

Software vendors of the main structural engineering packages provide some level of BIM integration, mostly via Input/Output (I/O) tools with BIM-oriented architectural packages such as Autodesk Revit and the generic Industry Foundation Classes (IFC) [105, 106]. However, there are numerous issues with those: as early as 2008, practising structural engineers were reporting considerable scepticism with vendors' promises of "seamless" links between packages [107]. The fact that interoperability is a major point of discussion in the review by Volk *et al* [108] suggests that it will remain a major concern in the foreseeable future. In addition, there is the recurrent issue of the complexity and steep learning curve inherent in BIM-oriented software, as well as the need to maintain parallel BIM infrastructures for different clients [107]. The return of investment (ROI) of BIM, especially for micro, small and medium enterprises (SME) has long been a point of contention [109]. In structural engineering practice, this can be even more demanding, as structural engineers already must master complex structural analysis and design packages for completing the core tasks of their work. As such, any additional software package must demonstrate a considerable return to justify the overheads in time and resources.

The structural timber design process

Structural design is one of the core areas of the building design process and can pose unique challenges to the BIM process. The structural engineer interacts not only with the architect but also with a range of other engineering specialities. This can be either directly or via the architect, the project manager, or the contractor, depending on the nature and organisation of the project. A simplified schematic representation of the interactions (and thus data exchanges) between the architectural designer and some of the engineering disciplines is given in Figure 2-19.

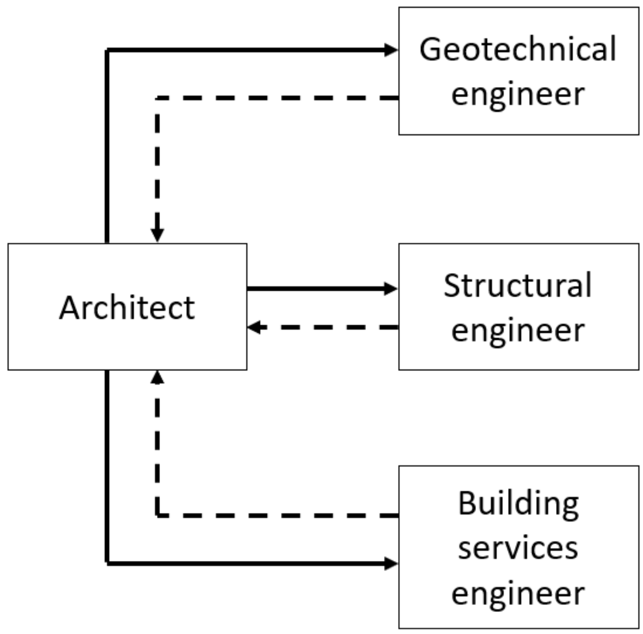


Figure 2-19 Schematic representation of interactions and data exchange between architect and some engineering disciplines

Even within the structural engineering domain, a range of processes take place. Those might be done in-house in different types of software or subcontracted to specialists. Thus, more interactions and data exchanges take place. A simplified schematic representation of those for the case of timber structures is given in Figure 2-20, while more extended versions exist for case-specific problems [110].

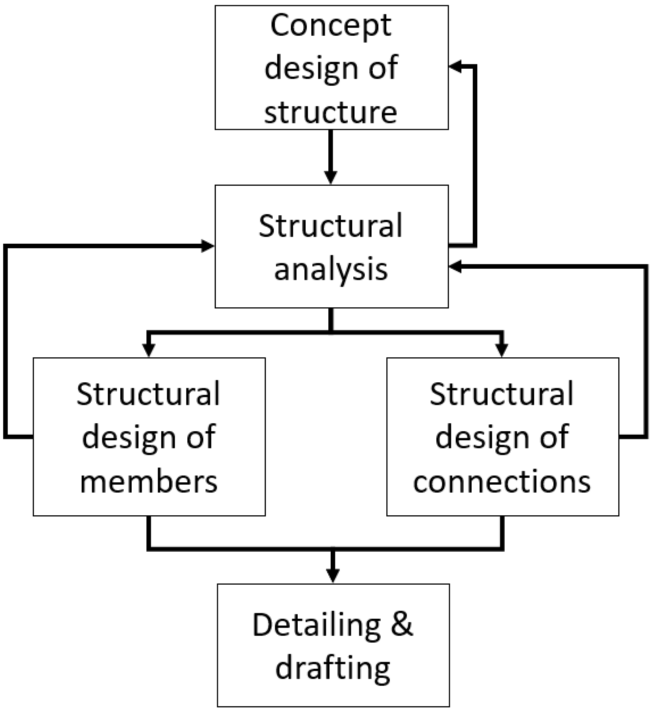


Figure 2-20 Schematic representation of the different stages of the structural design of a typical timber structure

From the activities described in Figure 2-20, the structural design of members and especially connections, pose particular technical challenges. It is telling that timber design is either omitted altogether from the mainstream structural engineering packages with BIM support [105, 111] or, when addressed, it is without the more technically challenging aspects, such as connections [112]. The latter is usually the province of specialised software applications, often lacking effective BIM integration support [113]. The automation of the structural design of such elements would offer significant benefits, addressing some of the challenges mentioned in the introduction. The integration of this automation in a BIM context would also allow these benefits to be combined with the well-established benefits of BIM [98] and thus contribute further to the take-up of timber by the wider AEC industry. However, the complexity inherent in the activities presented in Figure 2-19 and Figure 2-20, together with the aforementioned interoperability issues often present in BIM, means that any automation needs to take place within a well-defined BIM framework, in order to fulfil its potential.

Approaches in BIM frameworks

As the definition of BIM is quite broad, it is useful to attempt to position the intended outcomes within the continuum of BIM possibilities. An often-employed scale for this range is the BIM levels or stages. As defined by Succar [114], stage 1 represents object-based modelling, stage 2 represents collaborative working, utilising at least one collaborative model, while stage 3 aspires to interdisciplinary nD models with network-based integration and synchronous exchange of model and document data. Understandably, Succar recognises that Stage 3 would require fundamental changes in the modus operandi of the entire AEC industry, as well as significant maturity in network & software technologies. Though Succar was writing in 2009, a description of the same concept (referred to as "Level 3") in the National Building Specification website, originally written in 2014 but not updated at the time of writing of this research, refers to it as the "holy grail" [115].

Existing approaches can be said to belong to one of two categories, defined here as "single platform BIM" (SP-BIM) and "multi-platform BIM" (MP-BIM). Single platform BIM relies on the utilisation of either a single piece of BIM software or a small range of BIM-compatible software applications, typically from the same vendor or as part of the same suite. This ensures I/O consistency and thus minimizes the development overheads. Other software packages might be used for non-BIM operations and, overall, SP-BIM is closer to Stage/Level 2. By contrast, multi-platform BIM allows the use of a wide range of BIM-enabled software packages, targeted at different disciplines and developed by different vendors. Data I/O might be done via the Industry Foundation Class (IFC) or via middleware developed within the context of a professional project or research project. Conceptually, this is closer to Stage/Level 3. A brief summary of the two definitions is given in Table 2-1.

Table 2-1 Single/multi-platform BIM description

Term	BIM functionality	BIM Level
Single-platform BIM (SP-BIM)	Concentrated in a single application, or a small number of applications with verified interoperability (typically from a single vendor) External hardware, datasets, and calculations processes are handled via customized I/O tools.	2
Multi-platform BIM (MP-BIM)	Allows the use of multiple BIM software packages, from many, or any vendor. This includes all, or most, hardware, datasets, and calculation processes.	3

The relatively straightforward implementation of this classification means that it has been utilised for a wide range of AEC tasks and problems. Wang *et al* developed a framework for enabling facilities managers to engage in the design of a building [116] via SP-BIM, while McArthur suggested a framework for the operations phase, where the main interaction is between a facilities management and a BIM system [117]. Song *et al* developed a structural BIM framework for construction planning and scheduling and implemented it via Open Cascade, a generic 3D modelling C++ kernel [118], using IFC and Microsoft Excel files as data I/O mechanisms. Porwal and Hewange proposed a BIM-based framework for public-private partnering in public construction projects [119] utilising the Autodesk suite of products to ensure compatibility.

Similar approaches are also applicable to more specialised requirements. Choi *et al* [120] as well as Chavada *et al* [121] have applied it to the problem of work-space planning and management; in both cases, a single BIM environment was utilised to enable nD modelling and resolve conflicts. Addressing more technical issues, Kim *et al* developed a framework that enables the dimensional and surface quality assessment of precast concrete elements [122]. A key part of this work involved primary data collection via laser scanning. The focus of the framework was not BIM per se, but I/O interoperability with a BIM system. Thus, IFC was used as the data format and a single BIM platform was utilised. Park *et al* produced a framework to link augmented reality with a BIM to facilitate defect identification and management [123]. In this, the mock-ups suggest that the envisaged implementation would be via an SP-BIM.

An earlier review by Cervosek [124] found that effective integration largely worked only within "tightly coupled" solutions, i.e. when the software developed has invested significantly in data I/O; typically this is the case only within applications from a single vendor, while users that attempt data I/O between applications from different vendors are faced with loss of information which often results in significant time loss for manual data input and remodelling.

Researchers working on multi-disciplinary problems have typically had to engage more with an MP-BIM approach. Singh *et al* developed a theoretical framework which, crucially, addresses server issues and thus computing/software requirements of a more technical nature [125]. Working with a range of different software packages and platforms, they identified an extensive set of technical requirements, ranging from model organisation and data security issues to the various aspects of administrative, training and legal support required. It is interesting that these challenges are raised for a case that concerns only architectural design, hydraulics and lighting, while the emphasis is on the visualisation and not detailed calculations. As such, even this extensive and highly detailed framework is unlikely to be able to satisfy the considerable computational demands of the structural engineering aspects of design.

Similar limitations can be identified in other work: Ding *et al* developed a framework intended to provide computable nD [97]. The work is comprehensive and provides important pointers to areas for further BIM research. The computation described, however, is highly unlikely to satisfy the requirements of state-of-the-art of structural design. Similarly, frameworks suggested by Lu and Olofsson [126] and Kadolsky *et al* [127] deal with the immediate problem of compatibility between heterogeneous data and integration of different knowledge domains but provide a limited scope for advanced computational applications.

The direct integration of computationally demanding approaches such as Finite Element Analysis (FEA) in an MP-BIM environment appears unlikely to be achievable in the foreseeable future [128]. It is characteristic that Volk *et al* [108] identified only one structural analysis-related innovative BIM process in their review. In this, Lee *et al* [129] rely on heuristics in order to satisfy the structural safety requirements they set, even when effectively using an approach closer to SP-BIM. BIM is challenging and there are a number of issues that need to be addressed in order for its full potential to be realised.

2.6.3. Error risk and the need for peer-review of automated calculations

Currently, spreadsheets or other calculation software such as Mathcad are common within organisations; with the move towards limit state design within the Eurocodes, this will only increase. However, a widely cited study conducted by Panko [130] estimates that about 5% of all formulas within operational spreadsheets contain errors, while a further study by Powell calculates this figure to be nearer 1% [131]. An additional study in 2009 by Powell set out to look at the measure of impact as the result of errors within 25 operational financial spreadsheets from 5 organisations. They identified 117 confirmed errors that were categorised into six types, see Table 2-2. From this total, 70 errors impacted the results, with a range of severity. Of these errors, seven exceeded $10 million [132]. This highlights the need for a high level of peer-review.

Table 2-2 Errors found were categorised into six types from [132]

Name of errors	Description
logic errors	where the incorrect formula was used
reference errors	incorrect variable reference or cell reference
hardcoding numbers	hardcoding numbers in a formula
copy/paste error	incorrect formula due to copy paste error
data input error	incorrect input data used by the user
optimisation error	variables or cells left blank which affect the results

As spreadsheets grow in complexity and are passed between users, errors are more likely to be made. One famous example of this is a widely cited paper from Harvard on the relationship between public debt and growth [133]. This paper was rebutted by an undergraduate doing a homework assignment when he turned up several significant errors in the original spreadsheet [134]. The original authors had mistakenly not selected several countries, had missed information out and had incorrectly weighted their averaging. Before the paper was disproved it had been influential in policy-making decisions in several countries and been cited by 1330 other academic papers. This example highlights both the ease of making errors in spreadsheets and the profound impact these errors can have.

2.6.4. Automated Code Compliance: Definition classification and history

Automated code compliance (ACC) is a topic that is currently much discussed within the BIM community [135-137]. Within the building construction industry, building standards and structural design codes are in place to provide health and safety in the form of structural stability, reliability, quality of materials and workmanship. Currently, the code compliance checking process is predominantly performed manually. Several studies have identified that the process of checking building designs against building codes is time-consuming and prone to error [138, 139]. These problems are predominately a result of repetitive design iterations and modifications. The manual certification processes of the building codes carried out by certifying authorities and the increasing complexity of building regulations / structural design codes [136, 140]. Therefore, the code compliance checking for a new building can be a significant cause of delays and cost increases within the design stage. Therefore, political and public interest has contributed to an increased demand for using modern digital tools to optimise the construction process [141]. The implementation of BIM within the AEC sector is opening new possibilities to make building processes more efficient [142-144]. In the future, we may find that ACC enabled BIM models may play a key role in obtaining approval from strategy bodies [145].

For structural and civil engineering, there are a number of three-dimensional BIM enabled structural analysis software packages which can perform automated code compliance checking for the sizing of structural timber members but lack the sophistication for calculating timber connections.

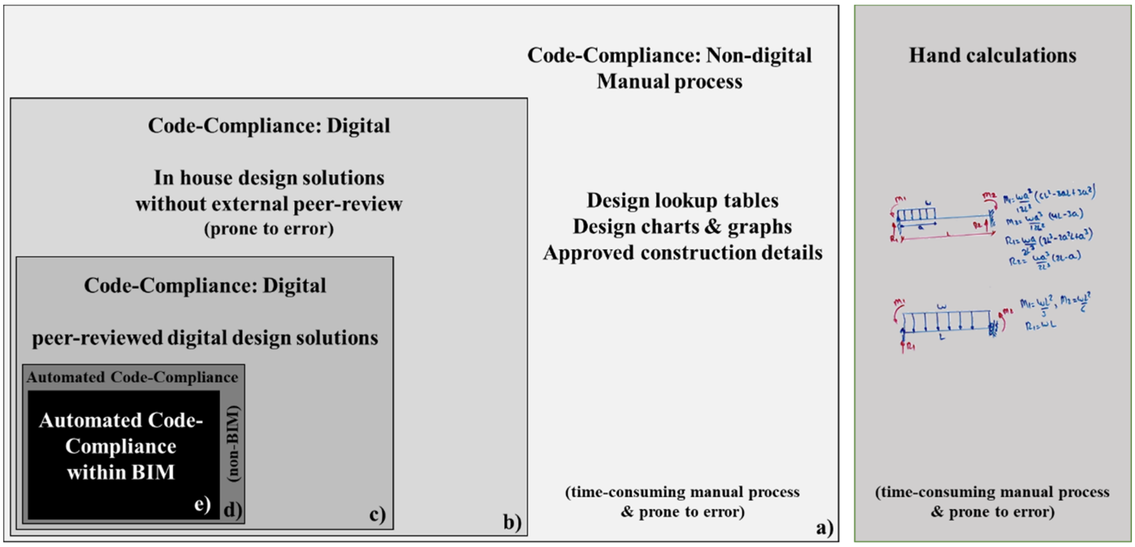


Figure 2-21 Classifications of Code-compliance, showing all the current posable systems of code-compliance

Table 2-3 Classifications of Code-compliance

a	Code-Compliance: Non-digital manual process	CC-nonDig
b	Code-Compliance: In-house design solutions	CC-inhouse
c	**Code-Compliance: Digital peer-reviewed design solutions**	**CC**
d	Automated Code-Compliance: non-BIM enabled	ACC-nonBIM
e	**Automated Code-Compliance: BIM enabled**	**ACC**

It may be helpful to introduce the concept of classifications of Code-Compliance, as illustrated within Figure 2-21 and Table 2-3. At one end we have “CC-nonDig”: non-digital design solutions that rely on such thing as lookup tables, design charts/graphs and the use of approved construction details amongst other non-digital solutions. At the other end, “ACC”: an automated design system that is BIM enabled. For the remainder of this research, the thesis will focus on CC and ACC approaches.

Automated Code Compliance is not a new topic within the BIM community. Fenves in the 1960s published work on decision table formulation [146, 147] which created a foundation for ACC. However, the main applications of ACC so far have focused on issues such as the design of the building envelope from an environmental perspective [136, 148] . Compliance uses different methods of approach: Rule or text-based interpretation using a logical approach and subsets of, i.e. simple logical approach [137, 149, 150], predicate [151, 152], deontology [153], ontological [154, 155], object-based approach [149, 156-163], programmatic aspects [135] or machine learning [164]. It appears that the current literature on structural design ACC within BIM is of a design concept nature rather than industry application [128, 163, 165-175]. Ismail used a quote from the book 'Object-Oriented Methods: A foundation' [176], to justify the statement that 'The variety of different techniques will always continue to develop in this field, the challenge is how to select and integrate these approaches' [177]. Implementing "Smart" BIM objects used within this research draws upon the examples of "Intelligent" objects presented by Sacks in 2002 [178] and was first developed in 1998 [179]. The programming was conducted using AutoLISP++ for use within AutoCAD. Another name that has been used is semantic enrichment, as a catchall term for adding smart information into a BIM building model [180, 181].

Some structural aspects of ACC are directly implementable in BIM platforms. A typical example is the lookup tables. Before the advent of computers structural engineering practice relied heavily on such tables for identification of aspects such as appropriate cross-section sizes, etc. While the advance of computational design has reduced their importance, they are still popular in professional reference guides [182] and manufacturers' literature [183]. These normally deal with single-variable problems. For example, for a given type of timber joist and given structural loading and support conditions, the engineer can find the necessary joist spacings in a lookup table. The implementation of such single-variable ACC aspects in BIM is both possible and with significant benefits. However currently it appears underutilized by timber manufacturers, who instead prefer to offer either detailed analysis software [184] or in-house design services [185]. Many other aspects are much more complex and computationally intensive. Contemporary structural codes and standards are developed under the assumption that they will be interpreted and applied by highly qualified and experienced practitioners. They incorporate simplified or generalised versions of the scientific state-of-the-art, which rests on models developed and refined over decades (structural design) or centuries (structural analysis). Software developed to support engineers that work with these codes relies on complex algorithms. From a computer science perspective, ACC is often unfeasible with existing approaches as, for multi-variable optimisation problems, recursion issues appear, i.e. calculation functions would need to refer to themselves. Thus, the current state-of-the-art relies on the interoperability between a BIM platform and various structural analysis and design software applications, with the issues of performance and data I/O described in the introduction.

Over the years there have been several attempts in developing a method for automated code compliance checking, a timeline is illustrated within Figure 2-22.

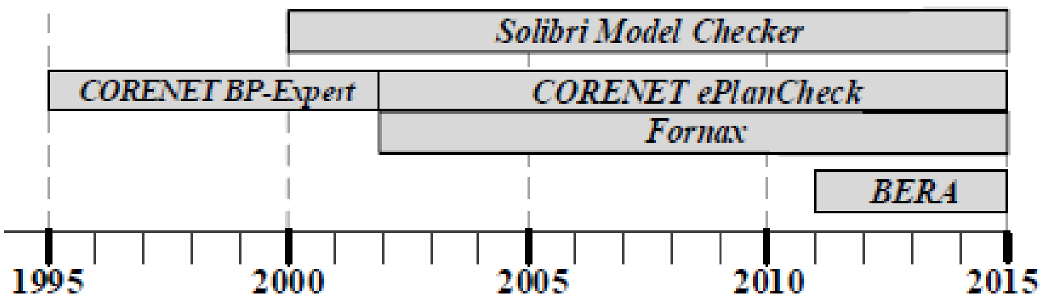


Figure 2-22 Timeline of ACC approaches, form [141] and inspired by [160]

The first authority to deliver a working system was a Singaporean building construction authority with the introduction of the platform, 'CORENET' which stands for construction and real estate network. It was not until 2002 when CORENT ePlanCheck was introduced that the platform had any capability of ACC, the functionalities of which are confined to building control barrier-free access and fire safety [186]. Within the CORENET platform a separate commercial module 'Fornax' has hardcoded checking routines which are not transparent and are therefore referred to as a black-box method (See Figure 2-23). Singaporean legislation and heavy government backing enabled uptake of the ACC CORENET platform.

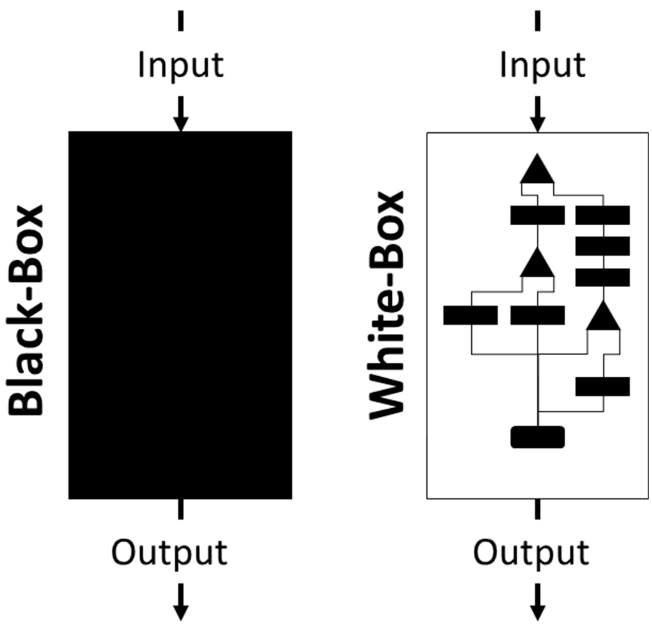


Figure 2-23 Black-box method, inspired by [187]

Solibri Model Checker is commercial software that relies upon imported IFC BIM models. It then operates in a black-box fashion, with an inability to allow custom rules without cooperation with Solibri. Building environment rules and analysis language (BERA) introduces a language-based code compliance checking system that is transparent for the user but unfortunately lacks the generality within the logic base programming language.

The three methods described above, although realising a level of ACC, still demonstrate a lot of inadequacies. Therefore Preidel and Borrmann 2015 [141] developed a visual code checking language, which they describe as "formal language with a visual syntax and visual semantics" (see Figure 2-24).

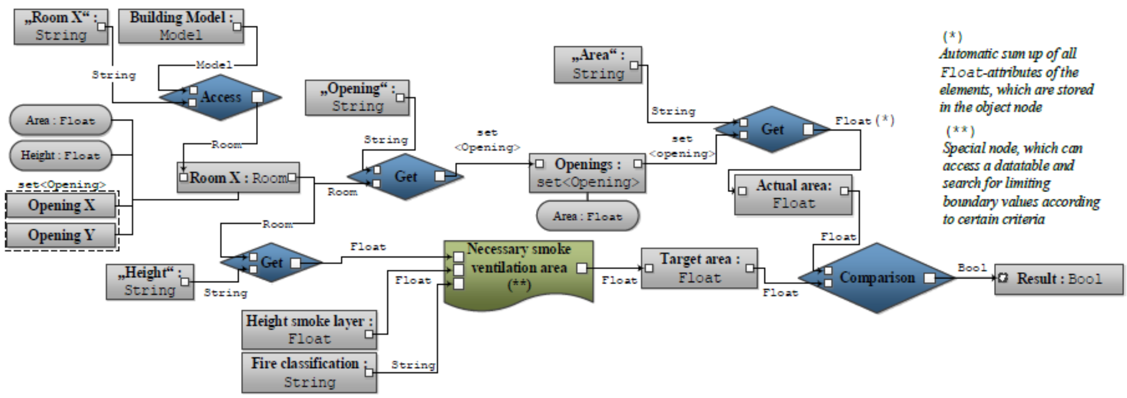


Figure 2-24 VCCL graph describing the central regulations of DIN 18232-2:2007-11, with image from [141]

In conclusion, all the ACC systems presented here have a primary function and design which is based around the architectural building requirements rather than structural design and analysis, which will be required when calculating timber connection design. So, a different approach must be taken.

2.6.5. Review of existing Code-Compliance timber connection software

The thinking outside the box report [41] highlighted the lack of accessible high-quality data to support modern wood building solutions, including addressing the supply chain control over information. This demonstrates that the lack of automated timber connection calculations becomes a barrier to engineers specifying timber within construction projects. Since the report was published in 2013 there has been some progress in developing software tools. The main timber connection software tools currently available are discussed below, together with their advantages and disadvantages. The intention was to identify a potential software platform that is or could fulfil the current needs of UK based structural engineers working in practice. Additionally, an ideal platform would have the ability to run batch processing of data or have an Application Interface (API) ability.

This is a non-exhaustive list of timber calculation tools, all performing timber connection calculations to EC5 for connection such as main to side member, multiple member, tension splice and axial withdrawal. Some may additionally possess the ability to perform more complex calculation configurations, such as moment connections.

Trada timber connections

Developed by Timber Research and Development Association (Trada) which is based in the UK, the calculation tool is for the sole use of their members. The timber connections online tool was first released in May 2010.

The automated calculation tool is limited in its functionality in terms of:

- The output will not show the detailed calculation output, making it difficult to verify the calculation by hand. The software uses pre-calculated datasets for fixing resistance values.
- Connection design options are limited, and the only real option is a tension connection with limited customisation options.
- Customisation loading options: the members only have the option to transfer axial loading, without the option for external loading acting on members
- Customisation of fixings: the fixing information is from a limited data set without the option for fixing variable customisation and ring shank nails are omitted by only showing smooth fixing options.
- The online software tool is only available to Trada members

Example of a calculation output can be seen in Appendix E.

Teretron, The Rope Effect Ltd

Teretron [188] is a product produced by 'The Rope Effect Ltd', released in 2015. Among other features the software calculates the lateral and axial capacity of fasteners, connections under tension and moment-resisting connections. This is an open design platform that allows for more flexibility as the users are not locked into using only one manufacture's fixings. The reporting facility shows the full detailed results including the relevant Eurocodes references.

Example of a calculation output can be seen in Appendix E.

Dluble: RF-/ Joints timber

Dluble RFEM and RSTAB with add-on modules for Timber structures is a German based software house with a strong presence in mainland Europe, but currently without an authorized reseller within the UK. This information was correct on 17/04/2019.

The current two add-on modules are:

- Timber to Timber [189]: joints where timber members are directly connected to each other by means of slant screws.
- Steel to Timber [190]: joints where timber members are indirectly connected to each other by means of steel plates.

Combined, the two add-on modules can be used to design a number of timber connection configurations. Although the software is calculating the results rather than using calculated data from a lookup table, the reported results are still in a black-box format, without the ability to follow the calculation method for validation purposes.

Example of a calculation output can be seen in Appendix E.

Master EC5 Timber Connections, BIMware

Master EC5 Timber connections [191] was first published in 2011 and designed for verification of load-carrying capacity of bolted splice connections for timber, wood-based and steel elements. Connection configurations are limited to connection with bolts only, and therefore, designs using screw or nails are not considered. It was created in Poland and currently has no UK distributor, limiting its uptake in the UK. Despite the vendor's name, this calculation tool is not BIM enabled.

Example of a calculation output can be seen in Appendix E.

Proprietary Software

Proprietary software packages include:

- HECO calculation software (HCS)
- My project, by Rothoblass

Both allow their users to design timber connections using only the fixings supplied by their respective vendor, which is the main limiting factor for this type of software. They both come with a user-friendly interface offering the design engineer a number of connection designs to start from. However, the geometry options are limited, which reduces the end functionality. It is the author's understanding that both vendors' software use pre-calculated datasets for fixing resistance values and for this reason the timber properties are limited to a selection from a list without the ability to enter user defined values. The output will not show the detailed calculation output, and only the results are shown without reference to the EC5 equations.

Examples of calculation outputs can be seen in Appendix E for both vendor options.

Tekla TEDDS, 2014, Using BS 5268-2:2002

The Tekla TEDDS platform has timber connection design calculations, which are calculated to BS 5268-2:2002, a withdrawn and non-maintained standard. The scope of these calculations are limited to checking the design of a simple bolted, nailed, screwed or toothed-plate, timber-to-timber or timber-to-steel connection consisting of two members.

The assumptions and limitations are as follows:

- Connections are assumed to consist of a timber main member and timber or steel connected member inclined at an angle α relative to the main member.
- A load F is applied to the connection through the connected member.
- In bolted and toothed-plate connections, it is assumed that grade 4.6 steel bolts will be used.
- In nailed connections, nails may be either plain wire, square twist shank or annular ring shank type. Nail holes may be specified as pre-drilled if required.
- In toothed-plate connections, toothed-plate connectors may be square or round and either single or double sided.
- In bolted, nailed and toothed-plate connections, both the main member and the connected member may consist of multiple similar elements. Where this is the case the individual elements are assumed to be positioned alternately through the connection.
- In screwed connections, both the main member and the connected member consist of a single element. The main member is pointside and the connected member is headside.
- The calculations do not attempt to check the geometry of the connection; users should satisfy themselves that the specified arrangement of connectors will actually fit.

As this example only conducts the calculations to the withdrawn standards and is in itself fairly limited in functionality.

Automated Timber Connection Summary

As discussed above, there are software tools currently available to perform automated timber connection calculations. It is not clear, however, if the needs of the UK structural engineer to confidently design with timber are met. The current capabilities of each software are summarised in

Table 2-4 and it is plain that there is no one piece of software that can perform all of the tasks identified within

Table 2-4.

Table 2-4 Comparison of timber connection calculation software tools

	1. Trada timber connections	2. Teretron, The Rope Effect Ltd	3. Dluble: RF-/Joints timber	4. Master EC5 Timber Connections, BIMware	5. Proprietary software: HECO, Rothoblass	6. Tekla TEDDS, 2014, Using BS 5268-2:2002
Connection configurations						
Axial capacity, i.e. Timber cladding	☆	★	☆	☆	★	☆
Lateral capacity, i.e. Splice connections	★	★	★	★	★	★
Custom multiple member conection (more that 3 members)	☆	☆	★	☆	★	½★
Fixing into end-grain timber	☆	☆	☆	☆	★	☆
Bending resistant and semi-rigid connections	☆	★	★	★	½★	☆
Individual member design						
User customisation of mechanical properties	☆	☆	★	★	☆	☆
Fixings						
Customisation of fixings	☆	★	★	½★	☆	☆
Nail smooth	★	★	★	☆	☆	★
Nail ring shank	☆	★	★	☆	☆	★
Screw	★	★	★	☆	★	★
Coachscrew	★	★	★	☆	☆	☆
Bolt	★	★	★	★	☆	★
Report output						
Showing summary results	★	★	★	★	★	☆
Showing full results	☆	★	☆	★	☆	☆
Showing full calculation references	☆	★	☆	★	☆	☆
Showing full calculation equations	☆	☆	☆	★	☆	☆

Only the Tekla TEDDS calculations have ability to use an API for batch processing of calculations. Because this version is calculating to the withdrawn British Standards, it is omitted from the search for a suitable existing calculation tool. The majority of these tools lack transparency in the equations used, to allow confidence and easier checking. Some of these tools are produced outside the UK and not all of them can use the UK National Annexe, which causes barriers to use within the UK. Not all of them were available at the start of this PhD and all bar one has been updated and modified over the course of this

PhD. It is unclear if these software tools are widely used by structural engineers in the UK. To satisfy the 'Thinking outside the box' report [41] we need corroboration of whether this report's statements are still valid and information on what software engineers are currently using.

2.7. Multi-Dimensional Data Fitting for Multi-Variable ACC

Currently, there is no existing BIM platform capable of performing automated code compliance calculations for timber connections. One potential method for implementing these complex calculations into a BIM environment would be to simplify the equations using Multi-Dimensional Data Fitting (MDDF). MDDF refers to the mathematical process that allows the fitting of datasets with an arbitrary number (n) of dimensions. Data fitting in one or two dimensions is commonly used in a range of fields. The simplest form of data fitting (with $n = 1$) is the common one-dimensional (1-D) curve fitting, where a mathematical equation is derived from a series of data points. Fitting with $n = 2$ it is typically referred to as surface fitting where a mathematical surface is generated so as to pass through or close a 2-D dataset. MDDF is the generalisation of this process to n-D, allowing the derivation of multi-variable algebraic expressions from extremely large datasets. As a mathematical method, it is neither new nor obscure: it is widely used in a variety of scientific fields to study topics such as gene expression [192, 193] and population synthesis [194]. Significant efforts have been made over the past three decades to improve and enhance the various aspects of the technique from a mathematical and computing perspective [195-197].

Despite its considerable potential and wide applicability, MDDF has not been widely used within the structural engineering field. One obstacle is the high cost of structural engineering experiments, which practically mean that the datasets are very small. A secondary potential reason is that MDDF outputs, while highly useful to predict behaviour, might not necessarily provide such useful insights into the physical behaviour of a system compared to analytical models.

However, MDDF can be particularly useful for the purposes of ACC. It allows the substitution of a complex, multi-equation structural calculation algorithm with a single equation. This single equation might be extensive, but it is significantly lighter in implementation from a computational perspective. More importantly, it allows multi-variable problems to be solved simultaneously and thus enables ACC features to be integrated into a "Smart" BIM component.

The programming of these "Smart" BIM components draws from an appropriate knowledge base. The development of such a knowledge base presents two challenges: firstly, identifying or developing suitable datasets that allow the application of MDDF techniques to derive the single-equation output; secondly, the application of the MDDF technique itself, which can be mathematically demanding. The following chapters describe how these challenges were addressed in this project.

2.8. Summary

From the findings of the literature review, it was identified that the transition to Eurocode 5 for structural timber design is taking considerably longer than was first envisaged as reported by Brooker from BSI in 2015 [30]. A report by Harker in 2013 [41] starts to identify the barriers for EC5 adoption. The review identified that:

- There is a lack of knowledge of timber engineering design to EC5.
- There is a lack of high-quality accessible data to support modern timber solutions, including addressing the supply chain control over information.
- There is a need for increased publicity and case studies of positive modern wood buildings solutions.

It was identified that further evidence and more detailed findings were required in order to ascertain the UK AEC sector perceptions of structural timber design to Eurocode 5.

The connections within a timber structure are often the critical factor in the design, as the strength of the structure is dictated by the strength of the connectors. Member sizes are often determined by the number of fixings used rather than by the strength of the member material itself. In addition, the fixing stiffness has an influence on the overall behaviour of the structure. The complexity of calculating the timber connections to EC5 is such that it is prohibitive – creating a barrier for specifying structural timber. Timber connection design CC or ACC can reduce this barrier, simplifying the connection design process. CC or ACC that has been through a rigorous peer-review process can provide a reduction in calculation errors, which in turn provides for a safer project. CC or ACC is time efficient and therefore provides ease of design optimisation and helps engineers specify timber confidently.

BIM is an ideal environment for the implementation of ACC, as BIM is now mandatory within the UK for government procured projects and as developers and clients start to realise the benefits offered by BIM. This, in turn, will drive the implementation of BIM for non-BIM mandatory projects. BIM is here to stay. BIM provides the central repository for information relating to the project and it makes sense to have output from ACC within it. As BIM ICT further develops structural engineers and designers will be using a singular BIM model and dataset for all of the multidisciplinary project team members. Implementing ACC for timber connection design into BIM will allow the designing engineer real-time feedback on the state of all of the timber connections, as structural changes are made to the model. BIM can allow for MC by providing flexibility and good choice navigation.

This vision of BIM is not currently a reality, so this thesis works towards implementing timber connections CC and ACC into the AEC sector.

Chapter 3. Methodology

3.1. Introduction

Much work has been done to improve the accuracy of timber connection calculations in recent years. However, more work needs to be done to improve the accessibility of these calculations. Modern EC5 timber connection calculations are overly complex for the average structural engineer and are only likely to become more complex as new fixings are developed. This could be improved by CC software and ACC, especially if integrated within a BIM platform. This thesis presents the research work undertaken to implement this, using MDDF and other software solutions to make modern timber connection calculations more accessible and time-efficient.

The development of such a BIM framework, which covers all the intended aspects, requires a sequenced approach and the utilisation of techniques from other fields, such as Multi-Dimensional Data Fitting (MDDF).

The work began with a survey of the current approach in structural timber design, in order to identify the role of the structural engineer within the timber design process, the interaction and data exchange with other disciplines and the key design problems that a structural engineer is expected to solve. The study continued by analysing the current approaches in existing BIM frameworks, in order to identify possible methods to increase time efficiency and industrial applicability. The results of the study were used to formulate the aims and objectives outlined in section 0.

3.2. Survey methodology

The survey was carried out with ethical due-diligence, following best practice principles of anonymity of participants. Advance clear information was provided, outlining the intended data use and providing participants with an opportunity to review and amend their responses.

The survey was disseminated to Structural Engineers via the following means:

1. Direct email sent to members of the Timber Engineering Network. All recipients of this email were based in Scotland.

2. Open online survey made available via LinkedIn, and shared onto the London IStructE group.

The Institution of Structural Engineers (IStructE) is a professional body for structural engineers based in the United Kingdom, as of the 1st August 2020 the UK LinkedIn group has over 52 thousand followers. This provided a good opportunity to directly target UK based structural engineers for this survey.

For the purposes of survey question validation, a selected list of the above mentioned Timber Engineering Network and selected Edinburgh Napier university staff were requested for feed back on the survey questions and formatting. Ensuring that the questions presented, nor the method of response was leading. Please see the list of questions used within the survey in Table 4-1.

3.3. Validation and Verification definitions

The definitions of validation and verification within the PMBOK guide [198], have been adapted for the context of this thesis research work:

- Verification checks whether the calculation tools are in compliance with EC5.
- Validation checks whether the correct functionality has been created.
- Verification is usually an internal process, whereas Validation is external.
- Verification usually takes place before Validation.

3.4. Creation of new TEDDS code compliance timber connection calculations

The specification for the new code compliance timber connection calculation software was drafted and agreed upon before commencement of work. The programming interface for the Trimble Tekla TEDDS platform is a bespoke coding language using Microsoft Word as its editor, please see the scope document within Section 0-1.

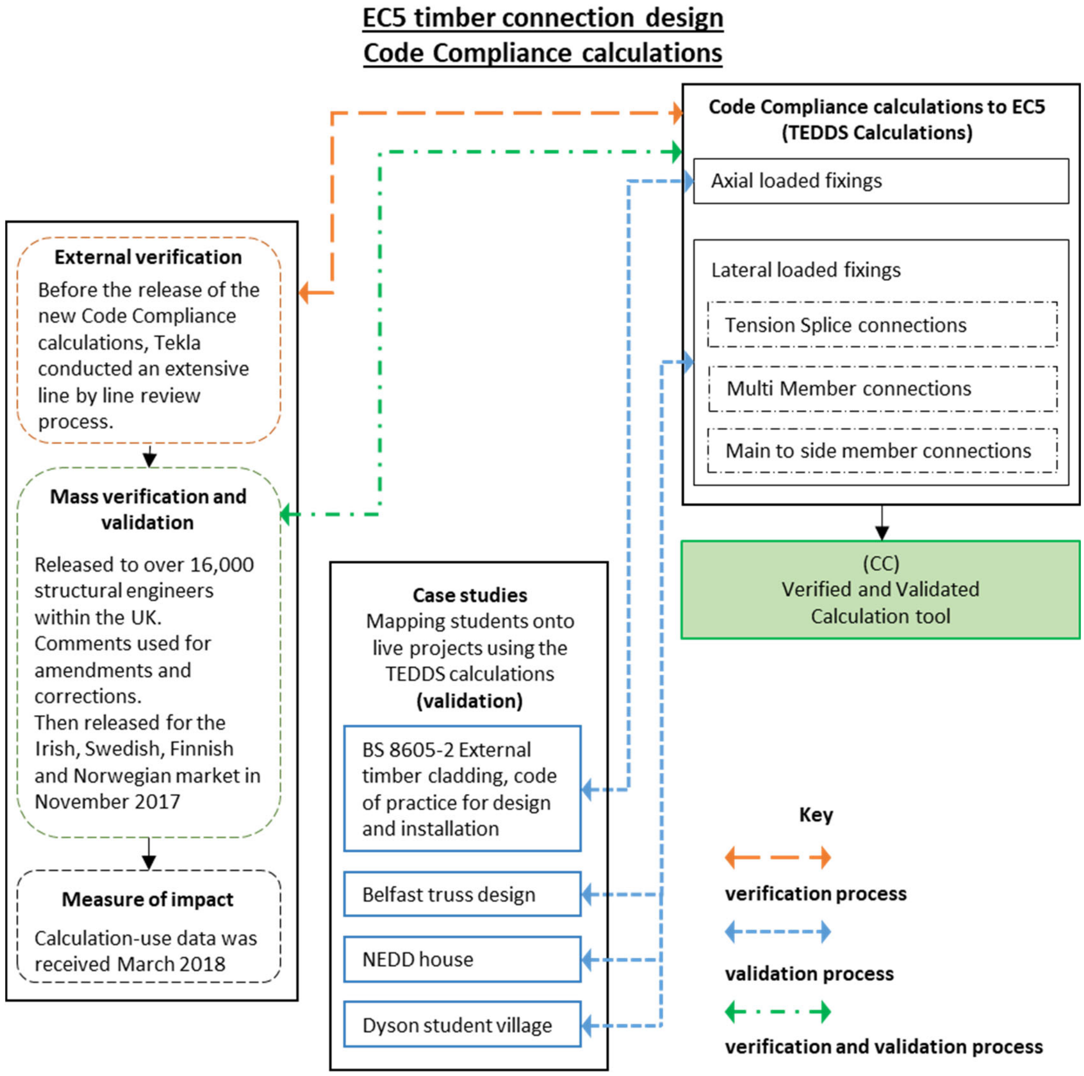


Figure 3-1 Steps taken to create, verify and validate the timber connection calculation tool

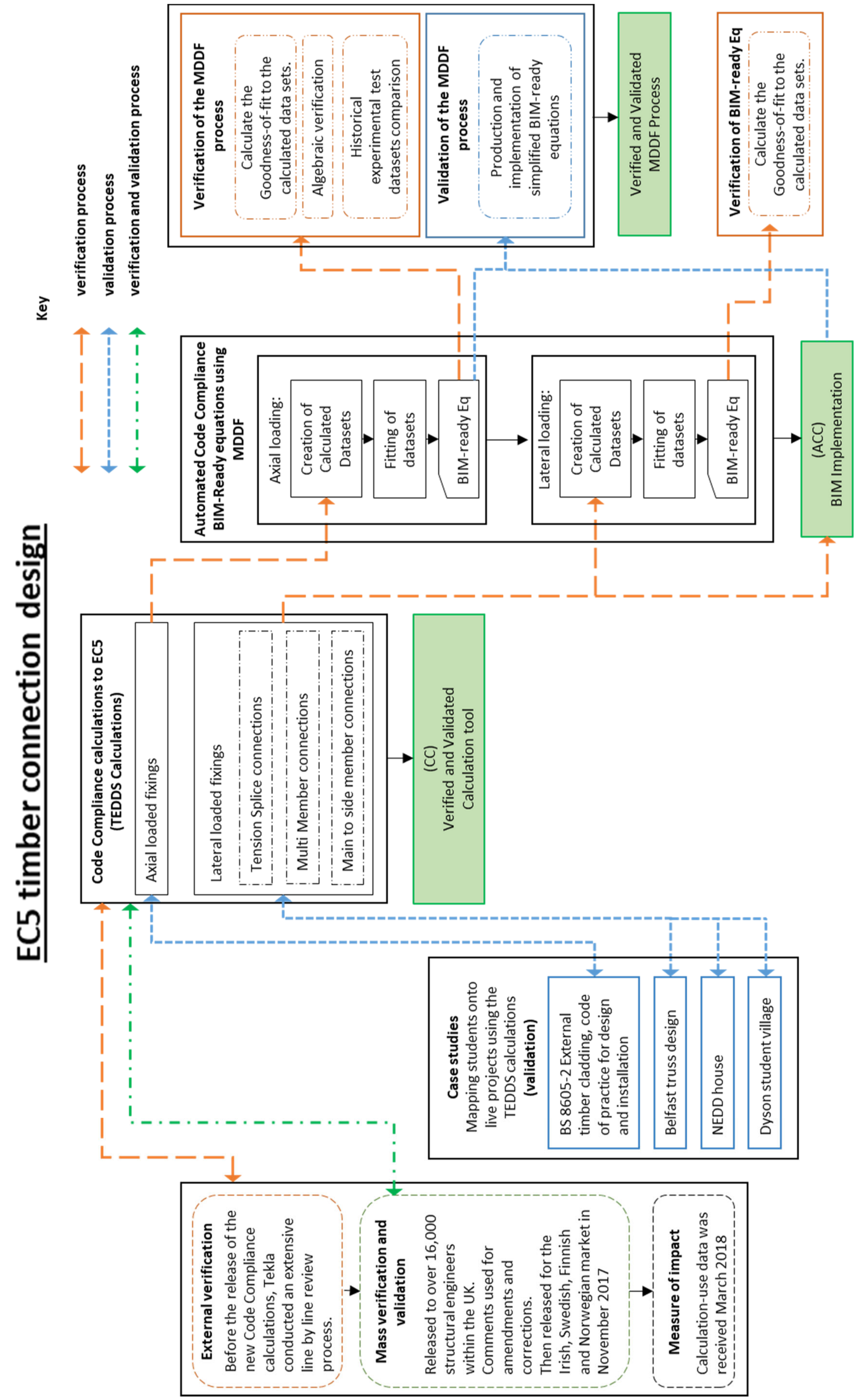


Figure 3-2 Verification and validation process overview of this thesis

Internal verification of the software was conducted to satisfy that the correct results were being produced. Tekla performed additional verification for the same purpose. Mapping students onto four live projects as case studies where the calculation tools were used, served as validation of the usability and functionality. Then additionally there was a mass verification and validation conducted by the large user group, see 0-1 for a list of revisions that were conducted primarily as a result of this mass verification and validation process, as illustrated in Figure 3-1. This approach was also used to identify routes for additional academic research to be implemented into industry practice. For example the bespoke strength class grade C16+ for homegrown timber was implemented into the Tekla TEDDS platform across all of the timber calculations within the calculation library.

3.5. Creation of BIM-ready equations

The creation of the BIM-ready equations required a number of steps. As with the steps for creating the code compliance calculation tools, the axial loading was tackled first, and this provided a good proof of concept for the Multi-Dimensional Data Fitting (MDDF) process. The first step involved identifying the relevant variables and their boundary conditions required and from this the datasets were created using MatLab programming. The data sets underwent random point verification using the verified and validated code compliance calculation tool on the Tekla TEDDS platform. The verified dataset was then fitted using the MDDF process with a bespoke MatLab program which was created within this research. The new BIM-ready equation then was subjected to goodness-of-fit and algebraic verification. Production and implementation into a BIM environment of the simplified BIM-ready equations provided the validation of the MDDF process, see Figure 3-2.

3.5.1. Mathematical fitting

A framework at a conceptual level was developed. This allowed for the identification of the key design problems that need to be automated. The framework identified two general types of problems that a structural engineer has to solve:

- Simple ("single-variable") problems, for which the majority of the structural calculations are either undertaken by the manufacturer or solutions can be easily found in the technical literature.
- Complex ("multi-variable") problems, for which a high level of technical knowledge and computational complexity is required.

The first category of such problems can be automated relatively simply and the integration in the framework in a straightforward process. An example of this can be found within one of the conference papers this study produced [199]. In this paper a smart BIM library component selects the correct structural specification of floor joist depending upon the span conditions, utilising a lookup table.

The second category of problems are significantly more demanding and their complexity partially explains why the automation found in structural timber design is limited. In order to address these obstacles, the work draws on the mathematical technique of Multi-Dimensional Data Fitting. An environment was developed within existing mathematical software, which enables complex problems to be solved. The thesis presents both the process and the capabilities of the software environment, in order to solve a complex multi-variable problem. While the initial concept behind the framework and the particular applications presented here, are focused on automating structural timber design problems, it is found that this can be generalised to address a number of similar problems and, theoretically, automate the building design process to a considerable extent. The conceptual framework is capable of being applied to numerous other problems.

In closing, the limitations of the approach are acknowledged, as well as its applicability and implications for building design.

While there are a number of software platforms available for fitting nonlinear multidimensional data, the particularities of this project meant that none provided the required functionality. As such, a customised MDDF platform was developed, based on the MATLAB computing environment, utilising the inbuilt nonlinear least-squares solver 'lsqcurvefit' [200]. This function is effectively a least-squares estimator, based on the Levenberg-Marquardt algorithm (LM) and trust-region-reflective algorithm methods [201-203]. The first iteration of an LM algorithm can be traced back to 1944 [204], with subsequent improvements in the 1960s [205] and 1970s [206], with further refinement of the goodness-of-fit published in 1980 [207]. The MDDF platform developed for the purposes of this work uses the 'lsqcurvefit' function over multiple dimensions. In addition, it includes a Graphical User Interface (GUI) allowing visual inspection of data with full user ability to interrogate any of the n number of dimensions.

An overview of the fitting procedure for different numbers of dimensions is presented in Figure 3-4, where τ is the number of data points to be fitted, i is the number of iterations, or points, along each dimension, and d is the number of dimensions.

The platform interface presents the user with a number of options for fitting the data. One, two, or all dimensions can be fit at once (Figure 3-3), while a numerical indicator of goodness-of-fit (for 100% and 95% of the data) is provided. The GUI allows visual inspection of the data against the fit, thus providing the user with a greater level of confidence (Figure 3-3 and Figure 3-5).

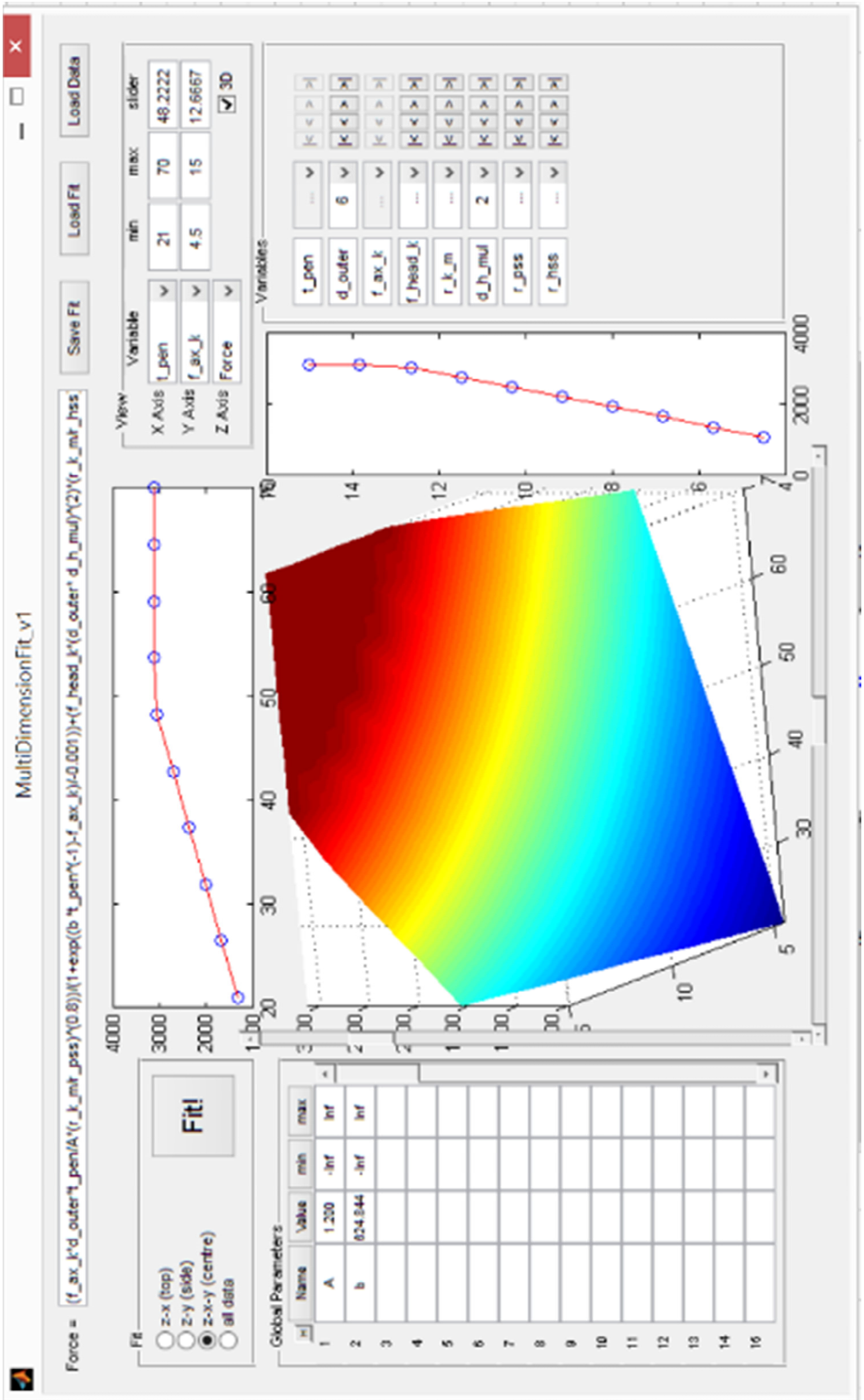


Figure 3-3 Graphical user interface of the fitting software

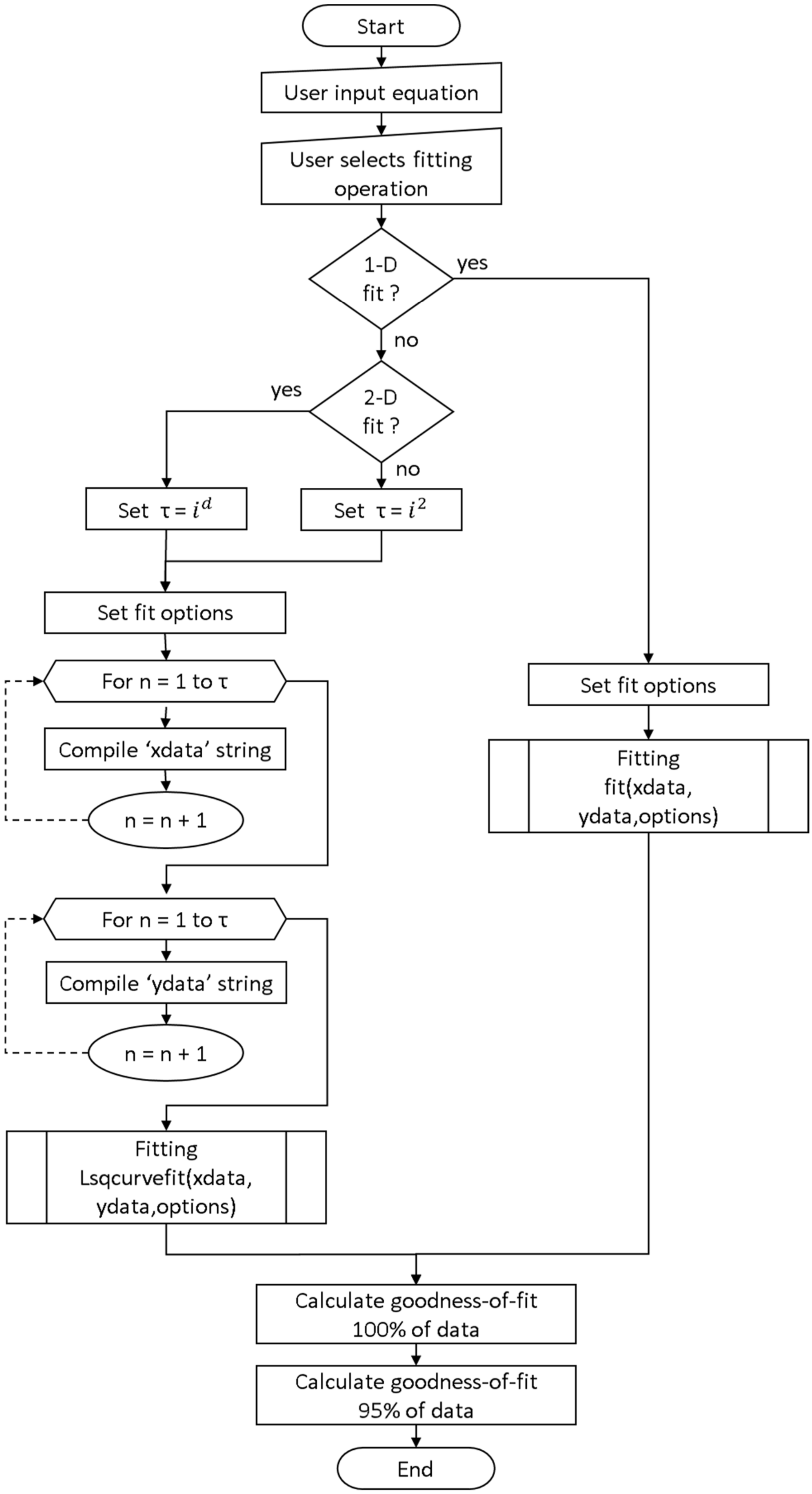


Figure 3-4 Overview of the fitting procedure within the new fitting software

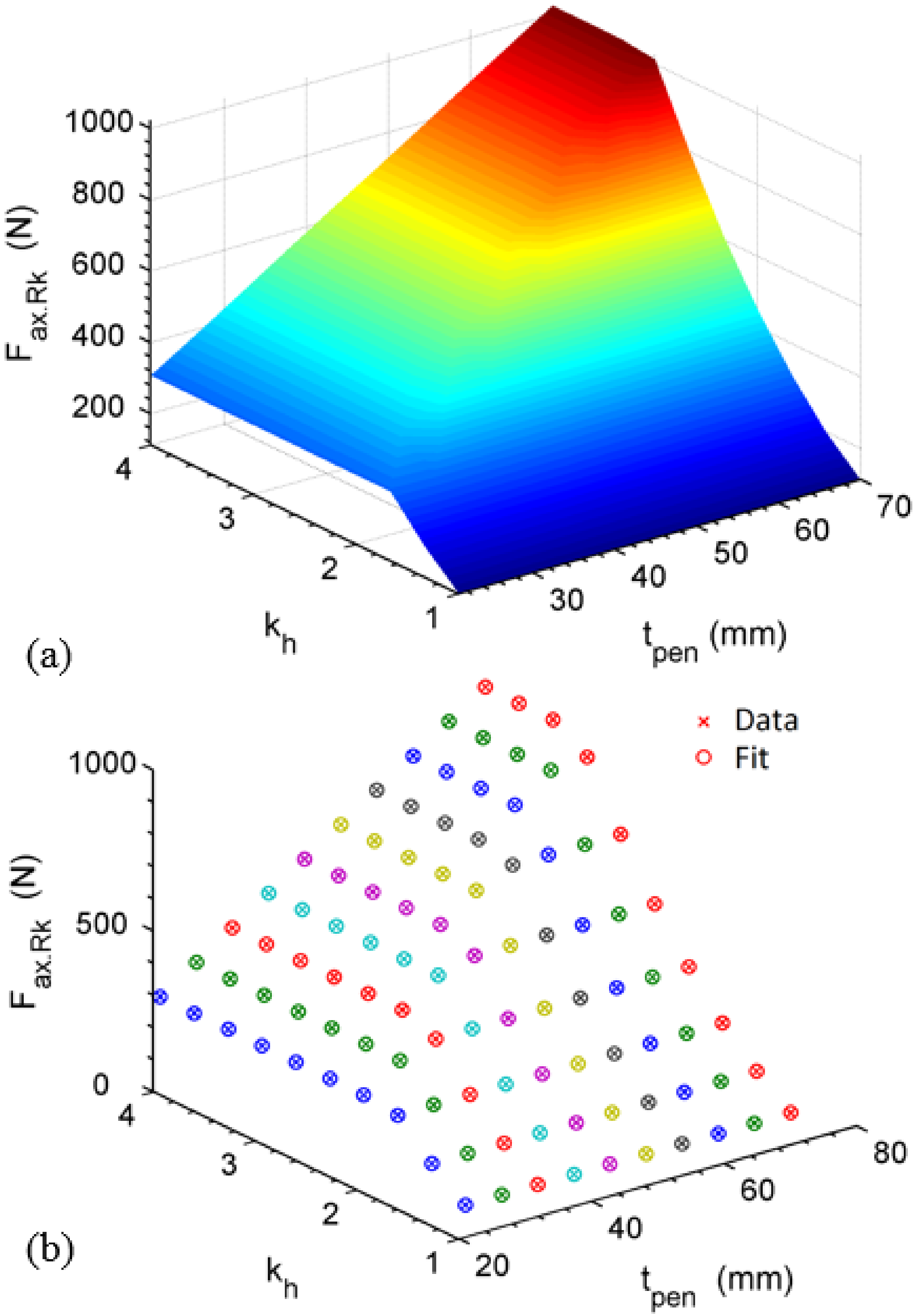


Figure 3-5 Data and fit comparison, for two dimensions

3.6. Scope

Naturally, the work presented in this thesis does not deal with ACC conclusively, and nor does it present a complete ready-to-market commercial-level system. Instead, it is intended to act as a proof-of-concept, demonstrating the feasibility of delivering effective ACC within an SP-BIM system with the current state-of-the-art in software and hardware, as well as highlighting the potential of the ACC concept more generally and the implications it has for changing building design as we currently understand it.

3.7. Summary

This thesis used a range of different methods to fulfil its aims. The software and the method of creation for the BIM-ready equations have been designed, verified and validated over the course of this research work. This process has formed the bulk of the project. The code compliance timber connection calculation tools are currently implemented and being used by the AEC sector, see Section 5.4 for the usage data. Various students have been trained to use the code compliance tools on complex engineering challenges, see Chapter 6, demonstrating that suitable software can empower inexperienced engineers to confidently calculate timber structures. The BIM-ready equations have been demonstrated within a 3D model, see Section 7.4.2, that is capable of showing the potential of Automated Code Compliance for timber connections within a BIM environment.

Chapter 4. Barriers to Structural Timber Use: A Survey

4.1. Identifying barriers

The barriers to Eurocode 5 (EC5) adoption and the specification of structural timber within the UK AEC sector have been discussed within the literature review. 2013 Thinking outside the Box report [41] identified a number of priorities that need to be overcome, as listed below:

- Professional education and continuing professional development around modern wood building solutions.
- Continuing to develop and communicate the positive environmental benefits of modern wood building solutions, to counter the high degree of activity in this area by the steel and concrete lobbies.
- Low quality and accessibility of data to support design processes for modern wood building solutions, including addressing the supply chain control over information.
- Increased publicity around positive case studies for modern wood buildings solutions with lots of examples of good modern wood buildings solution detailing.

These findings are in agreement with research work conducted by Schmidt & Griffin, where they discussed the barriers to the design and use of cross-laminated timber structures in high-rise multi-family housing from the context of the United States [49] published in 2013. Additionally, Barker's report [53] in 2003 states that "greater uptake of technology is considered fundamental, to overcome the barriers to MC".

As the "Thinking outside the box" report was published in 2013, it was decided that a further study was justified to corroborate these findings and add further depth and knowledge. In order to ascertain the barriers preventing the use and application of structural timber within the UK construction sector, a survey was developed based on the findings of this report [41]. It was an online questionnaire which surveyed 76 structural engineers working within the UK.

4.2. Survey

The full report of the survey can be found in Appendix I.

A total of 149 responses were generated from the survey, 76 of these were from IP address based in the UK, 19 from those based in Europe and 56 from the rest of the world. Only the results from the UK and Europe are reported in this study. A breakdown of the location of the UK respondents is given in **Error! Reference source not found.**.

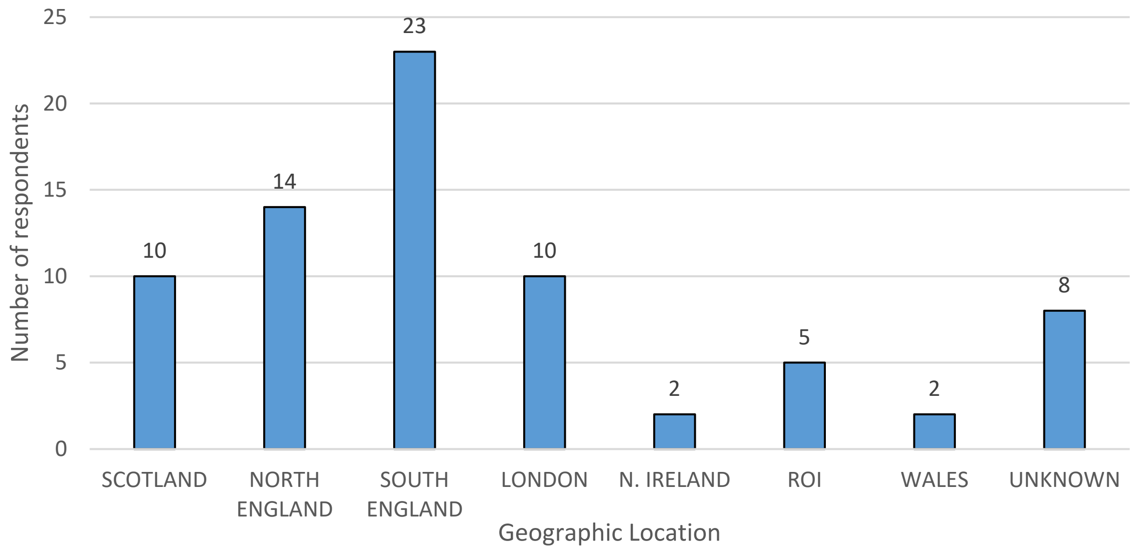


Figure 4-1 Number of UK respondents relative to geographical location within the UK

Note that respondents whose location is given as "unknown" are those which have withheld there IP address. Whilst it is known that they have responded from somewhere in the UK there exact geographic location cannot be identified with certainty.

The survey received 76 respondents from within the UK, and that the population of the UK IstructE LinkedIn group is approximately fifty-two thousand, it is found that the margin of error for the survey is at approximately 11%. This has been calculated using equation I- 1.

For the purposes of this research a margin of error of around 11% is an adequate representation for the purpose of this study.

$$Sample\ size = \frac{\frac{z^2 \cdot p(1-p)}{e^2}}{1 + (\frac{z^2 \cdot p(1-p)}{e^2 \cdot N})} \qquad \text{(I- 1)}$$

where:

N = Population size: = 52,000

e = Margin of error: ≈ 0.11 (11%)

z = Confidence level 95%: z-Score = 1.96

p = Percentage value: 0.5 (this is the default value as this value returns the worst case)

It should be acknowledged that the results of the questionnaire may be skewed due to the nature of the sample surveyed. Respondents were selected from and existing list of COCIS contacts meaning that they are already predisposed to specify timber.

Additionally, the questionnaires title may have had the effect of pre-selecting those engineers responding via LinkedIn i.e those who are already using timber are more likely to respond to a questionnaire relating to timber, than those who do not use the material. For this purpose, some of the responses have been compared by selecting sub groups of the respondents, for example: by Geographic location, High timber use respondents, Low timber use respondents, respondents currently using EC5 and respondents from mainland Europe.

Please see the list of questions used within the survey in Table 4-1.

Table 4-1 Survey questions

User location was only used if opted in by the user.

	Questions	Response options	
Q1	Approximately what percentage of your work is undertaken using Timber, Steel or Concrete?	Scale from 0 to 100%	Automated error correction
	Do you agree or disagree with the following statements, (Q2 to Q4)		
Q2	Knowledge of timber engineering within professional teams is lacking	Strongly Agree Agree Neutral Disagree Strongly Disagree	
Q3	Perceptions of timber often overrule reality. This means that the idea of using modern wood building solutions can often be stifled in the early stages of design		
Q4	The lack of centrally available 'tables' similar to those widely promoted by the concrete and steel industry, means that timber is seen as a riskier choice for designers		
Q5	Are you using Eurocode 5, Design of timber structures?	Yes or No	
Q6	What would facilitate the use of Eurocode 5?	Software, CPD, Supporting service, Robust structural details, Undergraduate education	plus user comment
Q7	What Structural Software do you use?		user comment, only
Q8	Would you be more likely to specify timber and timber related products if the required technical information was freely accessible?	Strongly Agree Agree Neutral Disagree Strongly Disagree	
Q9	Give one example of a timber structural detail that you would like to have standardised information for		user comment, only
Q10	Are you using BIM, and if so to what level	Yes / No	plus user comment

Despite many advantages and positive perceptions of the material, outwith the low-rise housing market, it is seldom viewed as a viable alternative to steel or concrete. The survey reports have identified the issues that prevent modern wood building solutions from being considered on an equal footing as more commonly adopted approaches.

The survey findings are summarised below:

- On average, approximately 33% of the work undertaken by respondents involved the specification of structural timber. Regional variations were found to exist, and this value was found to increase to approximately 50% in Scotland. This has been primarily attributed to the prevalence of the timber platform method of construction in the high volume, low-cost domestic housing sector.
- The survey showed a good level of agreement with “Thinking Outside the Box” [41], indicating that in general a poor level of knowledge of timber and its applications exists and that this is an obstacle to its specification.
- 33% of respondents indicated that they were using Eurocode 5 [63]. It was found that the code was perceived as generally not fit for purpose, overly complex and that it did not offer any advantage over the standard which it replaced. This indicated that the adoption of Eurocode 5 [63] would only come as a result of its use being made a mandatory requirement – it is unlikely that engineers would make the switch to it by choice.
- A wide range of software platforms was shown to be employed by engineers for the purposes of structural design. The majority (57%) of those surveyed used TEKLA Tedds [208] software, showing this to be the platform adopted as standard throughout the UK industry for design and specification. See Figure 4-2 for a breakdown of all software platforms used.
- 40% of respondents agreed that a lack of information relating to timber product and performance was a barrier to the specification of the material.
- A need for the standardisation of details has been identified – in particular, those relating to commonly specified connections.
- 30% of respondents indicated that their design procedures are adopted as part of a wider BIM framework, although further analysis showed that most did this at a low level.

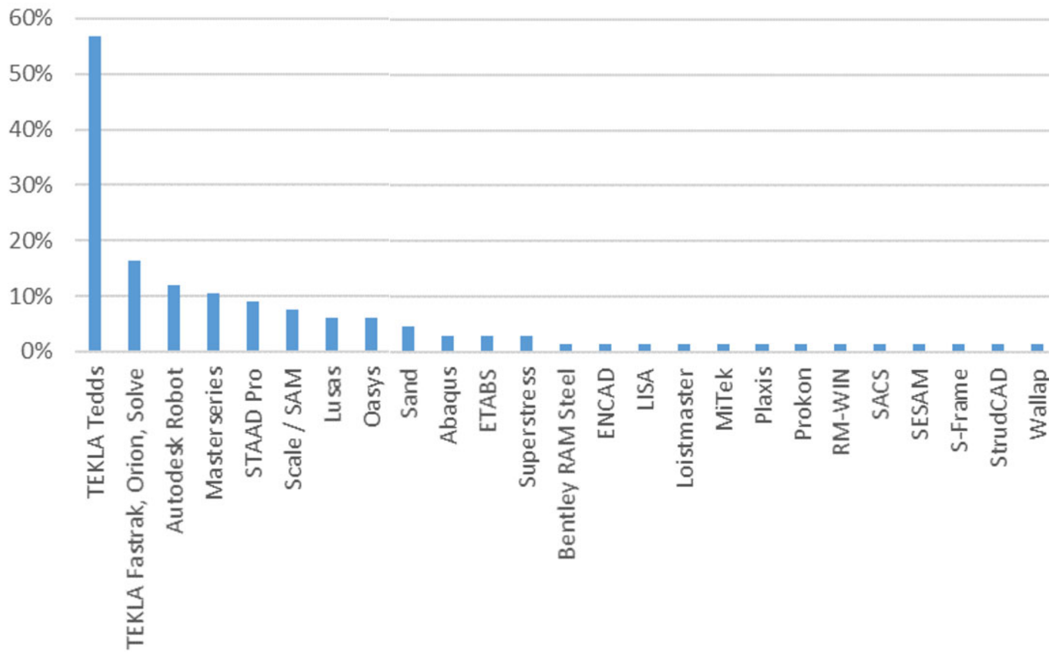


Figure 4-2 Software platforms used by UK based structural engineers, from survey

The potential implications of these responses are as follows:

- Generally speaking, the conclusions of the Thinking Outside of the Box report align with the opinions of those surveyed giving additional validation to the report. The predominant result from these questions is that there is a perceived lack of knowledge amongst design teams although the parameters of the question do not allow for the type of knowledge which is lacking to be determined.
- The uptake of EC5 in the UK is shown to be limited, even amongst respondents who are pre-disposed towards the use of timber. The value of developmental work related to EC5 may be reduced as a result of the fact that results indicate that the majority of the Engineers are likely to still be using the superseded BS 5268 standard.
- It can be surmised from these results that all options presented are valid means of increasing the uptake of EC5. The work being undertaken By COCIS offers a route to address most of the issues highlighted
- CSC (Tekla) Tedds software has been shown to be the most commonly utilised structural software platform and therefore the one providing the most effective route to practicing engineers.
- The results provide evidence that a lack of technical information is one of the barriers to the specification of home-grown timber.

- Due to the prevalence of connection and their critical nature in timber structures it is to be expected that the most requested standardised details requested are in relation to these.
- It would appear that those involved in the Timber Engineering industry could be better positioned to take advantage of BIM. In general, the sector has not engaged with BIM – this could be attributable too:
    - A general unawareness or lack of understanding as to what BIM actually involves or entails
    - The nature of projects which utilise timber – timber is not typically used in the large scale projects where BIM may be adopted.
    - Engineers who specialise in timber tend to be small practices or "one man bands" i.e parties who are not set up to implement a BIM strategy
- As well as indicating a general lack of uptake of BIM, those who are actually implementing it appear to be doing so at the very lowest level. Responses show little evidence of further development of BIM beyond the levels attainable by default through the use of AutoCAD or Revit.

Follow on actions resulting from these key findings are listed as follows

- Further surveys should be extended to consider the wider Engineering community and in particular parties who do not currently design with timber but can see the value of doing so. The information provided by such Engineers would be of great use to removing the barriers preventing the specification of structural timber.
- Utilise findings in conjunction with Thinking Outside of the Box report in support of future funding applications. Quantify the nature of the knowledge which is perceived to be lacking and make best actions to address this shortfall.
- This question should be retained as part of further questionnaires in order to demonstrate any increase in the use of EC5. It should also be followed by a further question asking if the increase in the uptake of EC5 can be attributed to our work with Tedds.

Issue	Solution
EC5 not user friendly	Through automating the calculation process and introducing easy to use user interfaces EC5 can be rendered user friendly
Cost associated with EC5	Economic cost of EC5 not addressable through the work of COCIS but through developing calculation on an existing software platform the need to purchase a physical hard copy is reduced
EC5 Unfit for purpose	Being an approved and reviewed standard EC5 has been declared to be fit for purpose. It is postulated that the view that it is unfit for purpose is as a result of its perceived level of complexity – particularly in relation to the more user friendly BS 5268. Automation of calculation process via Tedds will address this issue.
Offers no feasible advantages to existing British Standards	The advantage of EC5 are its analytical basis and the fact that it offers the user the ability to input their own variables. The ability to do this can be illustrated via CSC (Tekla) Tedds.

- Continue to undertake development activities based on CSC (Tekla) Tedds software.
- Identify routes which can be used to increase the flow of technical information to practicing Structural Engineers.
- Development of streamlined methods for the calculation of timber connections and associated details. Further refinement of exact nature of details required.
- Undertaken further research/ questionnaires to determine if BIM is relevant to timber engineering and also to identify if the creation of a BIM framework for timber will promotes its use amongst larger organisations that may be adopting BIM in earnest.

In many respects, the non-timber construction sector within the UK is several steps ahead of its timber counterpart, with better implementation of building information modelling, mass customisation and design for manufacture and assembly. This is primarily a consequence of the fragmentation of the structural timber supply chain. There is a major disparity in investment into research between the steel/concrete and the timber sector. This disparity and fragmented supply chain results in the following shortfalls within the UK timber industry:

- The quality and accessibility of data to support modern wood building solutions and their associated design processes.
- Established standardised design and detailing and communication of best practice.
- Effective dissemination of academic research to practising structural engineers.

4.3. Limitations of existing software tools

The literature review investigated the currently existing software tools available for calculating timber connections using metal dowel-type fasteners, but the options are all limited in some way. They either lack transparency with the calculated data or do not allow for user-defined variables, for example only allowing for proprietary fixings, or they do not offer the freedom to design anything other than predefined connection configurations.

An example of connections that are out with the capability of all the reviewed existing software options:

- Vertical baton connected to a wall stud when there is either a void or insulation between the members, see Figure 4-3;
- Main to side member connection when the main member is not aligned with the top of the side member, see Figure 4-4.

It is correct that with a good working knowledge of connection design to EC5 an engineer can adapt the output from Teretron by The Rope Effect Ltd to calculate the two examples identified. But this undermines the original purpose of the software tool.

From the finding of the survey conducted within this research, it is clear that none of the identified existing automated timber connection software solutions were being used by the respondents. A full list of the software and web links used by the survey respondents are listed within Appendix J, which is sorted by software functionality.

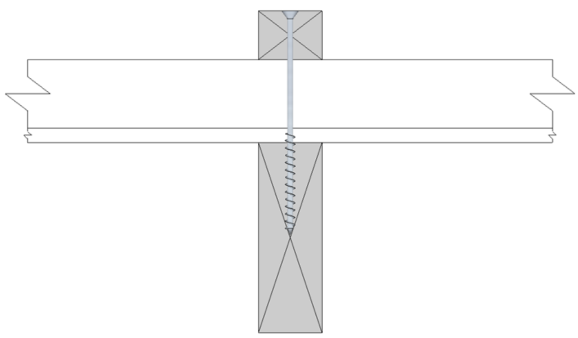


Figure 4-3 Connection for vertical wall baton

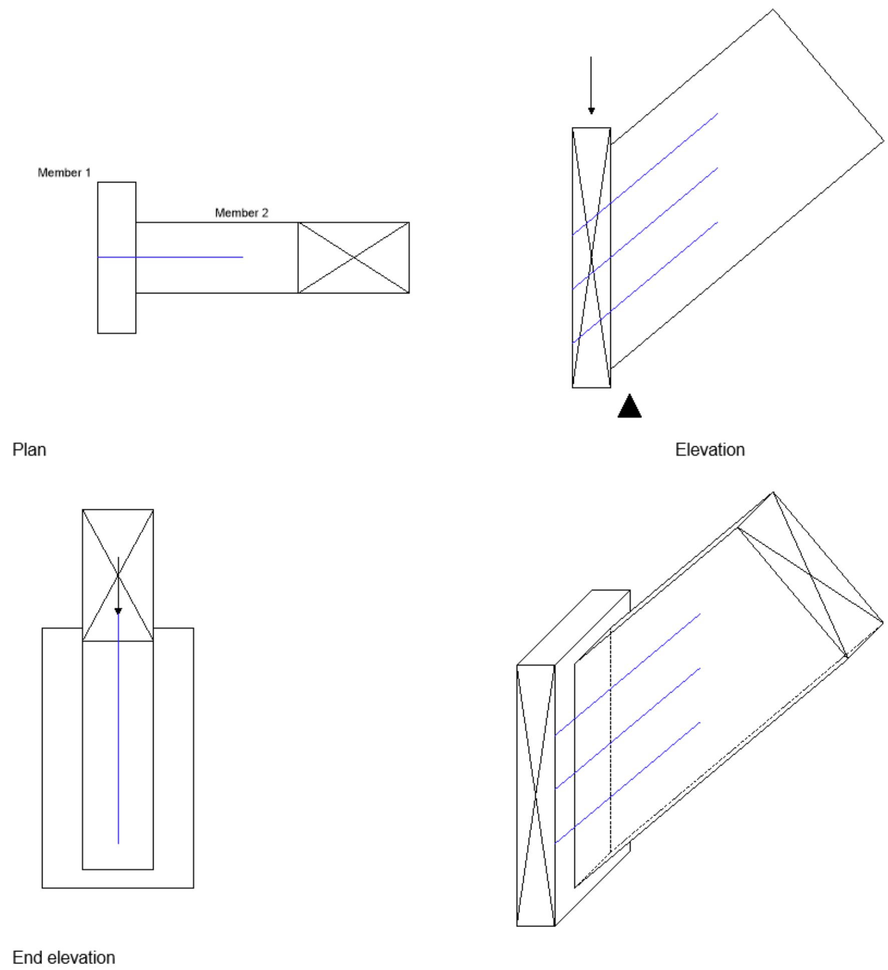


Figure 4-4 Connection for main to side member

4.4. Identify the route to impact for automated calculations

4.4.1. Streamlining academic research onto the desks of structural engineers

The functionality of Eurocode 5 [63] and the Eurocodes, in general, are driven by the requirement for empirically validated strength values within the calculation process. This lends itself to greater freedom of product specification and can be exploited to facilitate the inclusion of academic research data onto the desks of structural engineers, in a mass customised approach.

4.4.2. Justification for software platform TEKLA Tedds

The use and application of structural timber engineering research findings within the Architecture, Engineering and Construction - AEC sector is currently limited. The AEC sector is fragmented in relation to the application of structural timber. Currently, there is a lack of available mechanisms capable of demonstrating overall technical compatibility of new timber solutions from a holistic perspective, while conforming to current and future building codes. This is a real problem, as building methods must change through research exploitation in order to reduce the environmental impact and achieve UK Government's residential building targets. One solution for this would be the development of software tools to allow engineers to take advantage of the latest research findings in their routine structural calculations.

TEKLA Tedds [208] software has been shown via the UK based survey to be the most commonly utilised structural software platform within the UK. It is therefore currently the most effective way to condense the latest research findings into the hands of practising engineers.

The Centre for Offsite Construction + Innovative Structures at Edinburgh Napier University has a track record of embedding research into practice via TEKLA Tedds calculations [209-212]. This mass customisable engineering approach using the industry-standard software demonstrates a mechanism for streamlining research into practice.

4.5. Summary

The finding from the survey has been used to formulise the objectives and aims of this thesis, which are written in full within Chapter 1. In addition, the survey satisfied two of the objectives:

1. To do an industry survey of structural engineers that gives further clarity in identifying barriers for timber specification.

Reduce barriers for timber specification and connection design by:

6. To identify and utilising routes for current research to be implemented into the AEC sector.

The survey findings identified that there are regional variations in the some questions, as structural timber design is more common within the Scottish AEC sector. In general there was a poor level of knowledge of timber and its applications, a lack of information available for timber engineered products but a need for the standardisation of details – in particular, those relating to commonly specified timber connections. This presents an obstacle to structural timber and engineered timber products specifications. **Fulfilling objective 1.**

Eurocode 5 was found to be perceived as generally not fit for purpose, overly complex and not to offer any advantage over the standard which it replaced and that adoption would only come as a result of its use being made a mandatory requirement. The survey also identified that 57% of respondents use TEKLA Tedds software, making this the UK platform of choice for structural engineers and the potential for a clear pathway for delivering research findings into the hands of practising engineers. **This goes part way in fulfilling objective 6.**

From an industry perspective, it is envisaged that the work presented here can support AEC practitioners who want to incorporate timber in their projects but are finding the level of technical expertise required a significant barrier.

Connection design is one of the most challenging aspects of timber design in general, and Eurocode 5 in particular. The combination of timber's anisotropicity and the complexity of the scientific state-of-the-art mean that even basic connections demand highly detailed calculations. In the UK context, the difference in design philosophy between Eurocode 5 [213] and the previous British Standard [214], with Limit State Design as opposed to Permissible Stress Design, poses an additional barrier for practitioners [215]. The design of a structural timber connection with metal dowel-type fasteners was chosen as a model problem to study in this thesis due to this complexity.

Chapter 5. Code Compliance Timber Connection Tools

5.1. Introduction

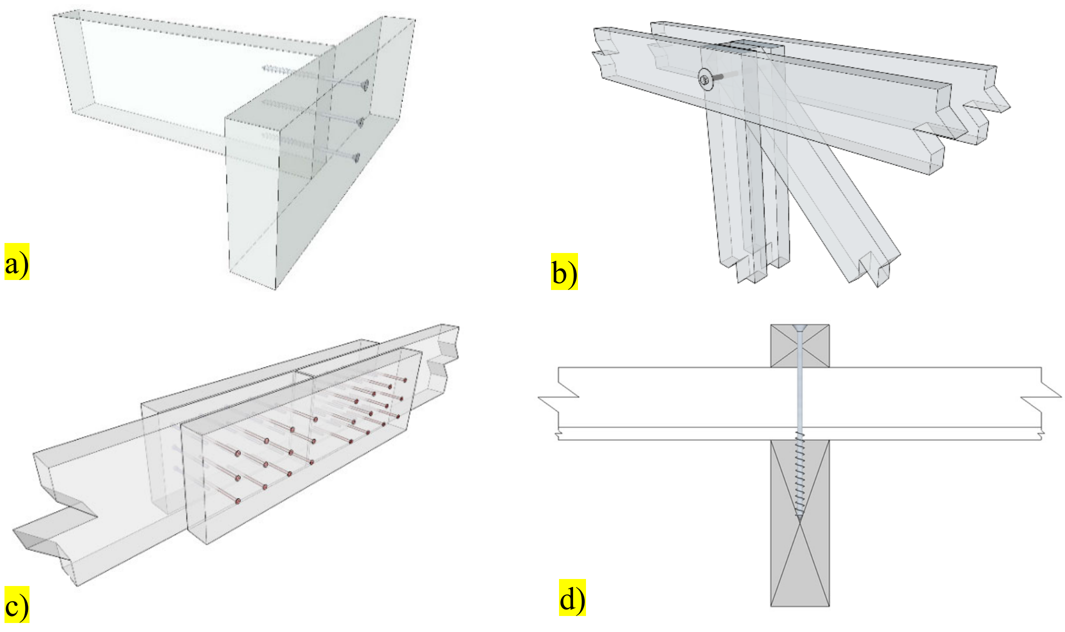
One of the main objectives set out within this research is reducing the complexity of EC5 through automation of timber connections. As discussed in Chapter 4.4.2 the choice of platform for this work was Tekla Tedds, that currently has a user network of over sixteen thousand structural engineers within the UK.

This chapter describes the functionality of the calculations, the assumptions and the limitations. The newly created code-compliance calculation tools will not be able to calculate all possible variations. However, they have been designed to cover the vast majority of timber connections that a structural engineer will encounter within a day to day project. Four calculation tools were developed to fulfil this need.

5.2. Scope, Assumptions and limitations

This is a simplified method of analysis for load-carrying capacity within timber-to-timber and steel-to-timber connections.

- This calculation determines the load-carrying capacity of timber connections using metal dowel-type fasteners.
- In accordance with EN 1995 and the National Annexes for the UK, Ireland, Finland, Sweden, Norway or the recommended Eurocode values.



key: a) Main to side member, b)Multi-member, c) Tension splice, d) Axial withdrawal

Figure 5-1 Connection type examples

All the timber in the connections shall have a minimum strength class of C16, in accordance with BS EN 338. If solid timber is used for the connection, material shall be individually graded and marked.

The load duration of the applied action is defined in the European standard BS EN 1995-1-1:2004+A1, or the corresponding UK National Annexe.

The strength equations used within these calculations assume that the shear and tensile strengths of fasteners will always exceed the capacity of the connection. If, however, there is a need to calculate the shear and tensile strengths of the fastener, it should be carried out in accordance with the requirements of BS EN1993-1-1.

In order to verify the ultimate and serviceability limit states, each design effect has to be checked and for each effect, the largest value caused by the relevant combination of actions must be used.

Timber should be pre-drilled when:

- the characteristic density of the timber is greater than 500 kg/m^3;
- the diameter of the fastener exceeds 6 mm;
- the timber thickness is less than that defined in Eq.(8.18) in EC5-1-1

5.2.1. Axially loaded fixings

This calculation consists of a point and headside members and can be used to calculate the withdrawal capacity of a fixing. A void can be specified to represent a physical void or materials that are not to be calculated for withdrawal capacity, for example PIR insulation boards on a warm deck roof system.

The headside member has a minimum thickness of 16 mm but recommends a minimum of 19mm, which represents the minimum recommendations for timber cladding by BMTRADA 'External timber cladding 3rd edition'.

The calculation accepts the design axial load only.

5.2.2. Tension splice

This connection consists of two categories of members: main members and splice members. The main members are restricted to being timber where the splice members can be timber or steel, and the properties can be individually specified. The fixings can be nails, screws or bolts and the minimum splice member length is automatically calculated based upon fixing end distance spacings.

The main members have a minimum timber thickness of 20 mm. This connection style only permits tensile loading. All loadings are entered as design loading only. For connections using nails or screws, fixings will be from both sides and acting in single shear. For connections using bolts, the connection is in double shear.

5.2.3. Main to side member connection

This connection consists of two members, one of which is connected into the end grain. The calculation allows for fully specifying member properties, dimensions for both members and the second member can be rotated in two directions in order to calculate penetration lengths, spacings and the maximum number of allowable fasteners. If a connection uses slant fasteners then all fasteners must be in a similar fixing angle. Nails other than smooth nails, as defined in EN 14592, may be used in structures other than secondary structures, an example given within EC5 8.3.1.2 (4) of a secondary structure is a fascia board nailed to rafters. They should only be laterally loaded with at least three nails per connection and not exposed to service class three conditions. For nailed non-secondary structures, the fastener should be parallel to the grain without the ability of fixing rotation. For slant nailing, there should be at least two slant nails in a connection.

For screwed connections with axial and lateral loading, the minimum spacings, end and edge distances follow these rules:

- screws with a diameter or effective diameter of less than or equal to 6 mm, use table 8.2 8.6 in EC5-1-1.
- screws with a diameter greater than 6 mm - use table 8.4 and table 8.6 from EC5.

5.2.4. Multiple member connection

This connection consists of a minimum of two members and a maximum of five. The calculation allows for fully specifying member properties, dimensions for all members and each member can be rotated in one direction. Axial and external actions can be applied to all except the final member.

In multiple shear plane connections, the resistance of each shear plane should be determined by assuming that each shear plane is a part of a series of three-member connections. To be able to combine the resistance from individual shear planes and a multiple shear plane connection, the governing failure modes of the fasteners in the respective shear planes should be compatible with each other. For example, they should not consist of combinations of failure modes (a), (b), (g) and (h) from EC5 Figure 8.2 with the other failure modes.

For an explanation of the calculation method used for determining the angle of the shear plane and design force in the shear plane, please refer to “Design of Structural Timber to Eurocode 5” by McKenzie and Zhang, published by Palgrave Macmillan [216]. Connections in single shear have the option of using nails, screws or bolts. Connections in double shear must specify bolts. For connections using nails or screws the minimum number of fixings must be two.

For screwed connections the minimum spacings and end and edge distances follow these rules:

- screws with a diameter or effective diameter of less than or equal to 6 mm, use Table 8.2 in EC5-1-1.
- screws with a diameter greater than 6 mm, use Table 8.4 in EC5-1-1.

5.3. Challenges overcome

There were a large number of challenges to be overcome. Here just one challenge is highlighted: the need for geometric measurements. A simple example of this is the pointside thread penetration calculation in the timber cladding connection in Figure 5-2. There are additional considerations and cases that need to be identified in order to finalise the pointside calculation. Figure 5-3 lists the three-dimensional geometric measurements required to determine the correct rule to use. This need for three-dimensional measurements is also illustrated in a more complex case using the main to side member calculation with the screw fixings inserted at an angle in Appendix K-4.

The software platform TEDDS does not have a native three-dimensional environment to supply these geometric measurements. This was overcome by building a three-dimensional environment using vector mathematics. This is illustrated in Figure 5-4 where each numbered point was assigned three-dimensional coordinates that are governed by geometric rules and equations. This not only gives the required measurements but also gives the additional advantage of a three-dimensional image, as can be seen throughout 0.

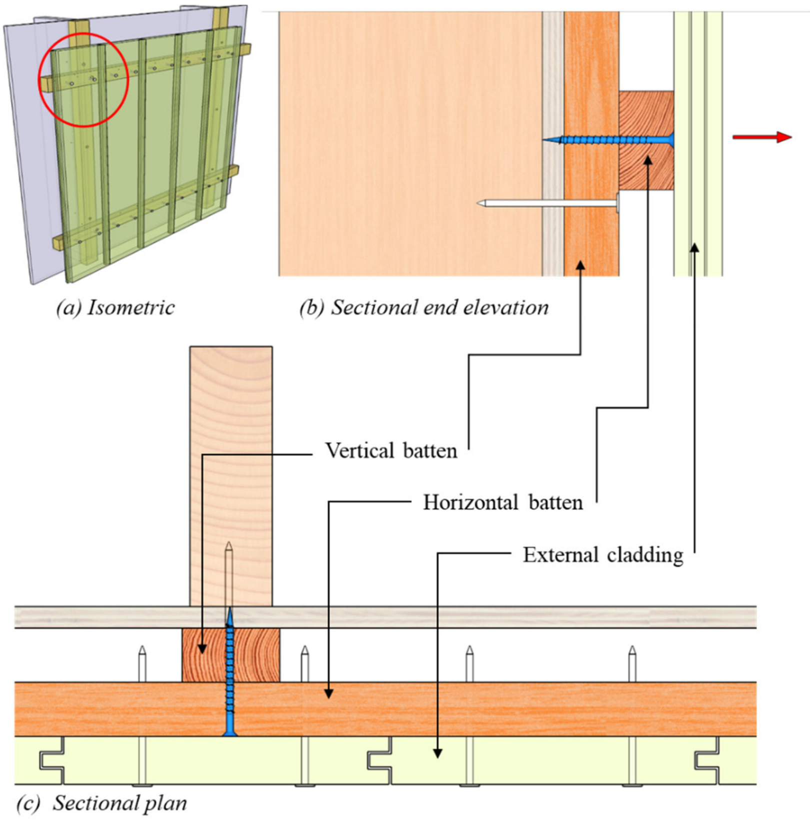
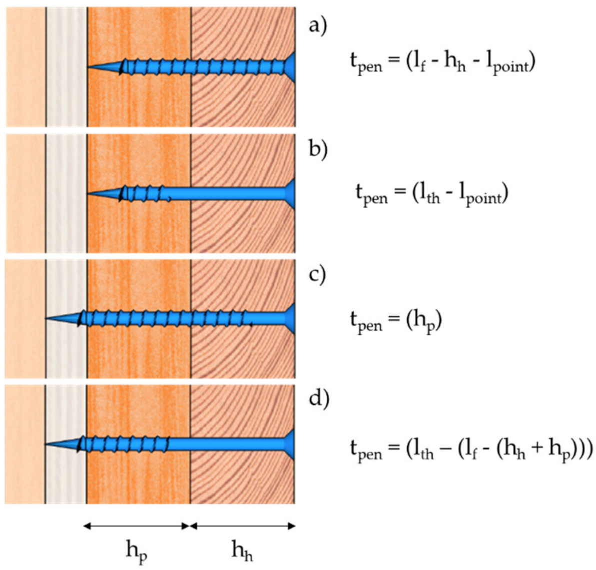


Figure 5-2 Timber external cladding connection



where

t_{pen}	pointside thread penetration
l_f	total length of fixing
l_{point}	point length
h_h	height of headside member
h_p	height of pointside member
l_{th}	total thread length, including the point length

Figure 5-3 Screw pointside thread penetration rules

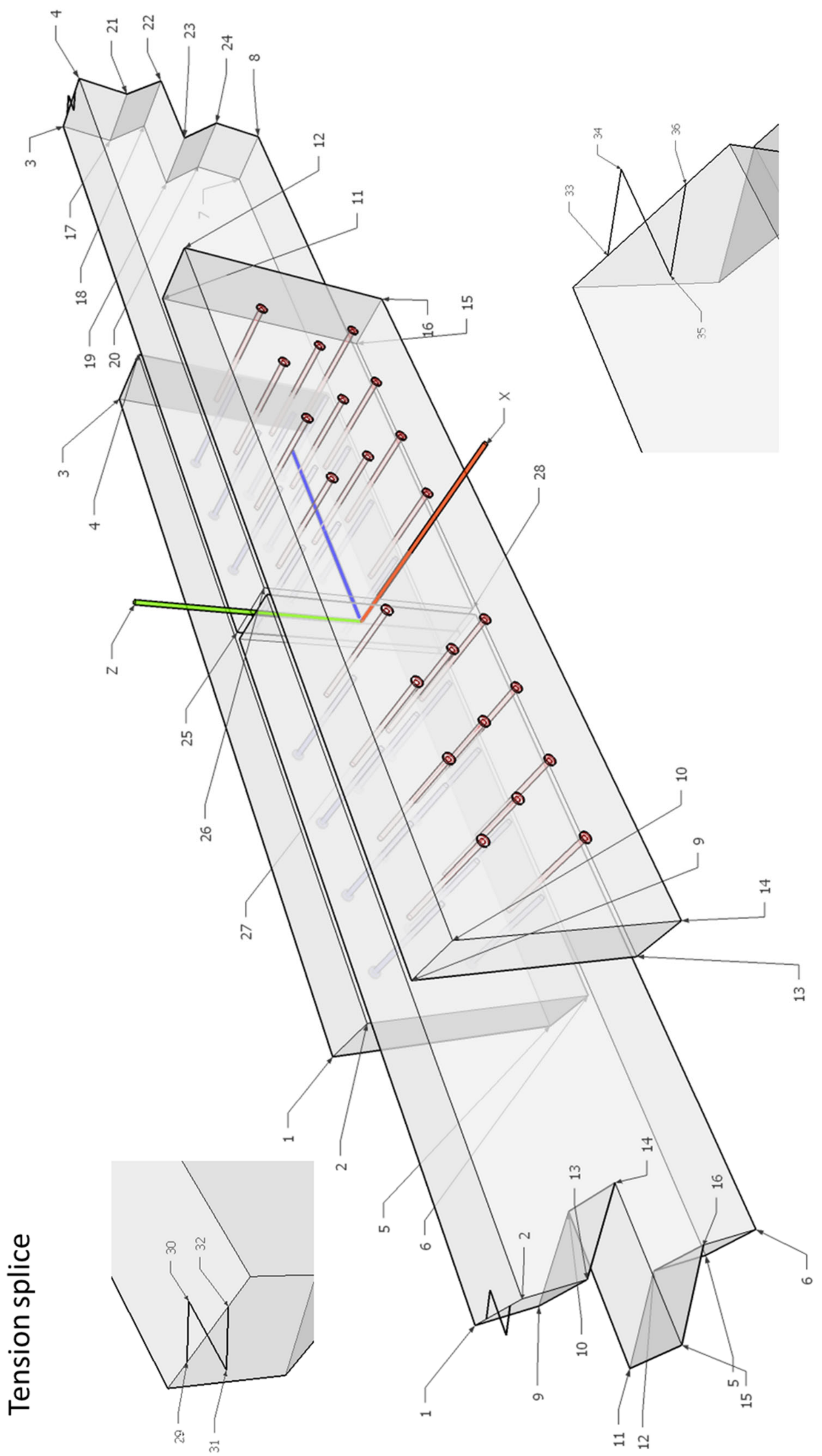


Figure 5-4 Tension splice 3D points

5.4. Usage data from Tekla

For a measure of the impact, the Tekla Tedd's usage data was obtained in March 2018. Note that all statistics are recorded for a subset of the user base, users have to opt-in for Tekla to record their data. The current subset of users that are opted-in and using the UK content calculation library amount to 5,000+ active Tekla Tedds users, per month. For comparison purposes, we see that 42.7% use the timber beam calculations, including 19.2% using the timber to Eurocodes and 23.5% using timber to the non-maintained British Standards. The newly created code-compliance timber connection calculations to EC5 are used by 10.3% of the subset group of UK Tekla Tedds users per month. The code-compliance calculations have led to teaching opportunities, both to university students and to IStructE members. In addition, it has led to several case studies, which are discussed more fully below. Note that on the 9th November 2017 the calculations were amended by Tekla to include the Irish, Swedish, Finnish and Norwegian National Annexes, see 0-1.

In addition Trimble created an educational video demonstrating the connections (Figure 5-5). Link: https://www.youtube.com/watch?v=WACc4yI0bB0&t=88s

Figure 5-5 Youtube demonstration of connection calculations, screen captures

5.5. Summary

The newly created Code Compliance timber connection calculation tools have been both verified and validated and are now published and widely used by UK structural engineers. These calculation tools provide confidence by reducing the complexity for the user, while providing an simple to follow calculation and output. This is fulfilling Objective number 3 of this research work.

Chapter 6. Case Studies

The first project that is discussed here is for the new British Standard for a code of practice for the design and installation of external timber cladding – Part 2. The calculations behind this document are in line with the calculation method within EC5; this served as a good starting project as this is the simplest design calculation process. The work involved the creation of data sets that are included in the new BS 8605-2 External timber cladding – Part 2: Code of practice for design and installation. The second project of the Belfast truss used the splice connection tool that calculated the shear connection strength for the connections. This was an obvious progression, moving on from the simple axial loading connections of the cladding case study project. This case study went on to demonstrate effective project optimisation and resulted in additional publicity of other work carried out by Edinburgh Napier University. The third project which was the Nedd house served as an ideal testbed for proofing and further development of the software. This project resulted in additional functionality enhancements to the system. The fourth project, for the Dyson student village, presented a challenging design problem which was solved by a design iteration process made possible by the speed and flexibility of the newly created code-compliance connection software. The final design solution was then verified by physical testing on a full size prototype within a lab setting.

All four case study projects had students conducting the work using the newly created calculation tools, to allow for validation of the usability and functionality.

6.1. BS 8605-2 External timber cladding – Part 2: Code of practice for design and installation [217]

6.1.1. Introduction

This project was to write a new British Standard code of practice for the design and installation of external timber cladding, based upon the connection calculation methodology of the connection design from EC5. The primary calculation used for this case study was the axial withdrawal tool, which is the simplest calculation process. By mapping a student onto this project this served as a validation of the usability and functionality for the user interface and the results output for the calculation. The student intern that was mapped onto this project was given training on the use of the newly created code compliance calculation tools for the timber connections and was supervised by myself. The project was to condense all the timber connection calculations needed into usable design steps, lookup tables and recommendations that:

- offer guidance on the material and design issues affecting external timber cladding assemblies;
- give competent building designers, who are not structural engineers, the means to confidently design exterior timber cladding assemblies on buildings where the risk of wind damage is relatively low;
- provide criteria for structural engineers designing external timber cladding assemblies for buildings where the risk of wind damage is relatively high, or who are verifying the work of others;
- provide a suite of generic construction details addressing performance issues that might affect an external timber cladding assembly;
- offer guidance for cladding installers.

6.1.2. Build-up types of external timber cladding

The number of configurations possible have been drawn down into the following nine sub system build-up types.

Build-ups without insulation

1. Horizontal external timber cladding on vertical battens, see Figure 6-1;
2. Vertical external timber cladding on horizontal battens, see Figure 6-2;
3. Vertical external timber cladding on two-layer battens, see Figure 6-3;
4. Vertical external timber cladding panel with horizontal battens with a 45-degree cut, see Figure 6-4;
5. Plywood cladding on vertical battens.

Build-ups with insulation include:

6. Horizontal external timber cladding and battens on rigid insulation, see Figure 6-5;
7. Horizontal external timber cladding and battens on secondary battens with flexible insulation include, see Figure 6-6;
8. Horizontal external timber cladding on vertical battens fixed on concrete wall with angle brackets;
9. Wider horizontal and vertical timber claddings with wider boards and 2no of fixings instead of 1no.

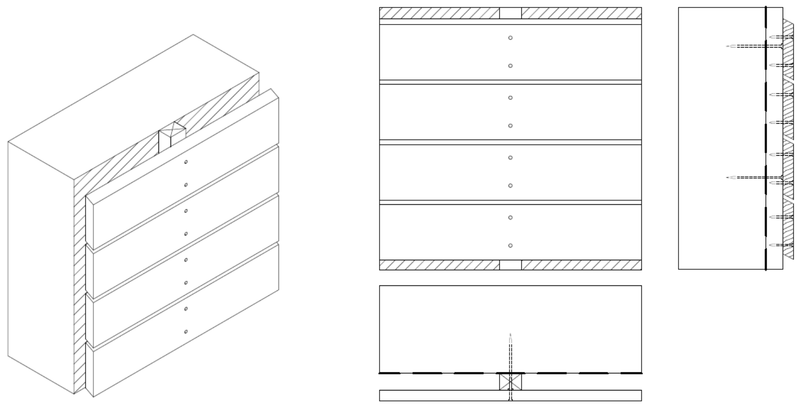


Figure 6-1 Horizontal external timber cladding on vertical battens

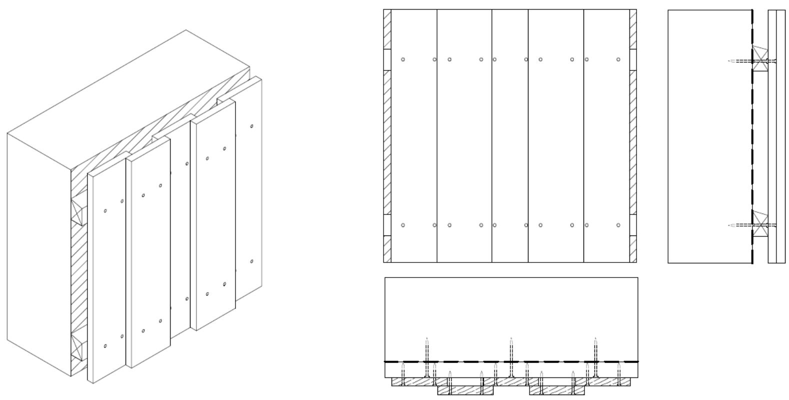


Figure 6-2 Vertical external timber cladding on horizontal battens

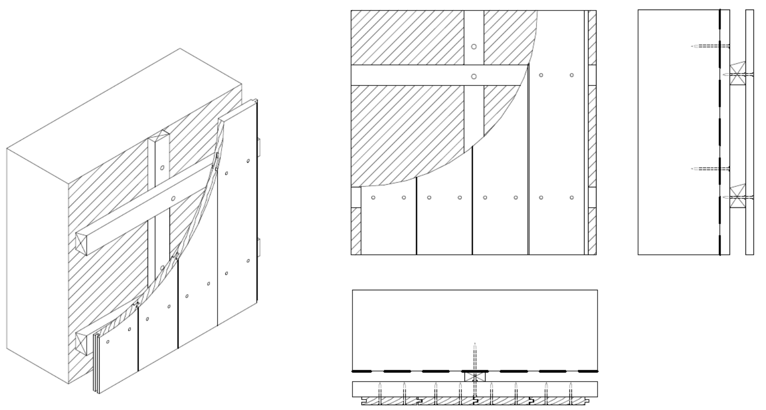


Figure 6-3 Vertical external timber cladding on two-layer battens

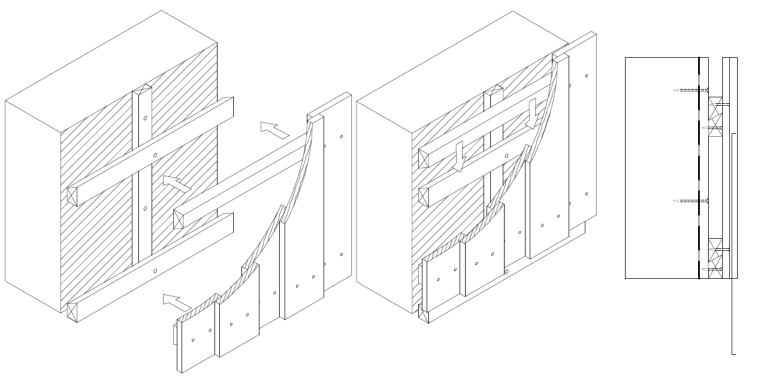


Figure 6-4 Vertical external timber cladding panel with horizontal battens with a 45deg cut

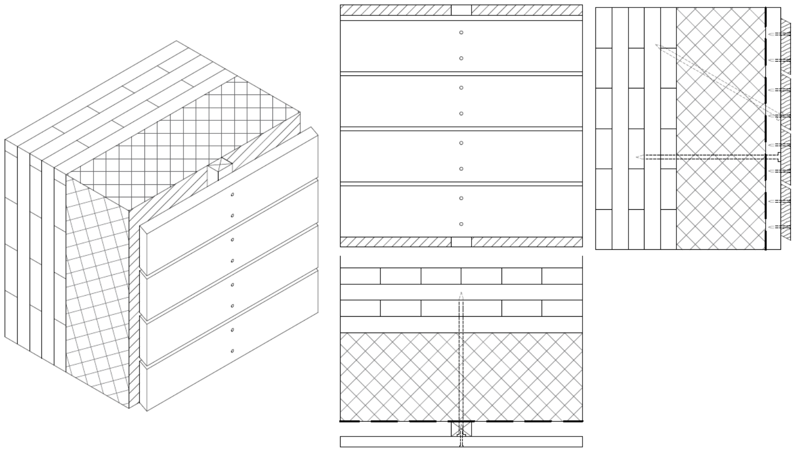


Figure 6-5 Horizontal external timber cladding and battens on rigid insulation

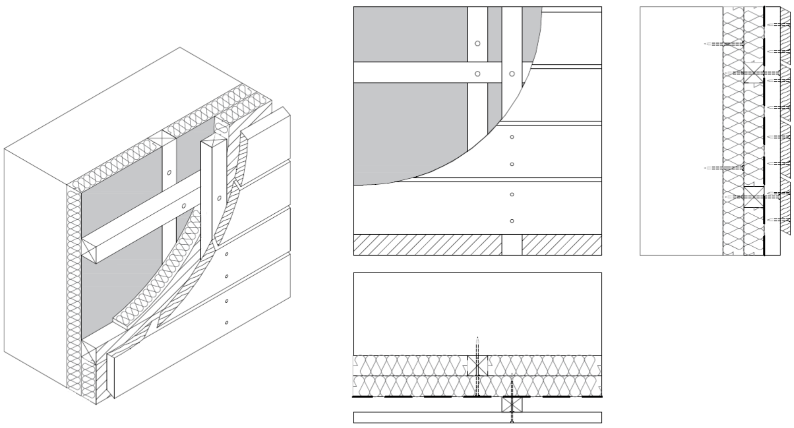


Figure 6-6 Horizontal external timber cladding and battens on secondary battens with flexible insulation

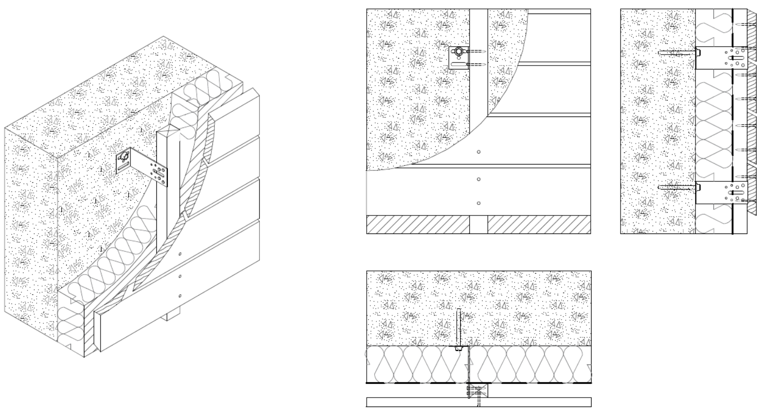


Figure 6-7 Horizontal external timber cladding on vertical battens fixed on concrete wall with angle brackets

6.1.3. Summary

This project took the newly created code-compliance connection design software for the speed and proven reliability of results and used it for the creation of data sets that are included into the new BS 8605-2 External timber cladding – Part 2: Code of practice for design and installation [217].

This case study fulfilled objectives 2, 3 and 6

Reduce barriers for timber specification and connection design by:

2. To create and deliver educational material of current research, for the purpose of increasing the level of knowledge of structural timber for both university students and practising engineers;
3. To reduce the complexity of EC5 through automation of timber connections;
6. To identify and utilising routes for current research to be implemented into the AEC sector.

The British Standard that has been created, (at the time of writing is still to be published) is intended for use by designers and contractors. It gives recommendations for the design and installation of external timber cladding assemblies in the UK. This is taking a very complex subject of timber connections and simplifying it down into a usable step by step approach that is more accessible, **fulfilling objective 2 & 3.** This body of work has been enhanced by the use of the newly created code-compliance connection calculations, which is directly delivering the results of research into the hands of the AEC sector, **fulfilling objective 6** and reducing the barriers for timber specification.

6.2. Case study: Belfast Truss, the potential of home-grown UK timber

6.2.1. Introduction

This case study was undertaken by a Masters student under my supervision, using the newly created code compliance software on the Trimble TEDDS platform, see Appendix I. The case study resulted in a Master's dissertation by Dale Johnstone and an international conference paper [218], and it is summarised below as it has relevance to the thesis.

Carbon Dynamic approached Edinburgh Napier University with a challenge to design a roof system capable of delivering a clear span of 30m for industrial building projects, utilising UK Home-grown British spruce (WPCS). The Belfast Truss design was identified for historical and aesthetic reasons, see Figure 6-8.

Within the investigation, it was identified that rather than the member design, the connection design was the limiting factor for this Belfast Truss system. As the timber density is the critical timer property that has an impact over connection design this presented an ideal opportunity to demonstrate the advantages of home-grown timber. The connections were designed using the newly created calculation software as discussed in Chapter 5. This research investigates the advantages of designing this structural roofing system, using the recently defined bespoke strength class C16+. Due to the greater characteristic strength and density values, this better fits the properties of UK grown spruce.

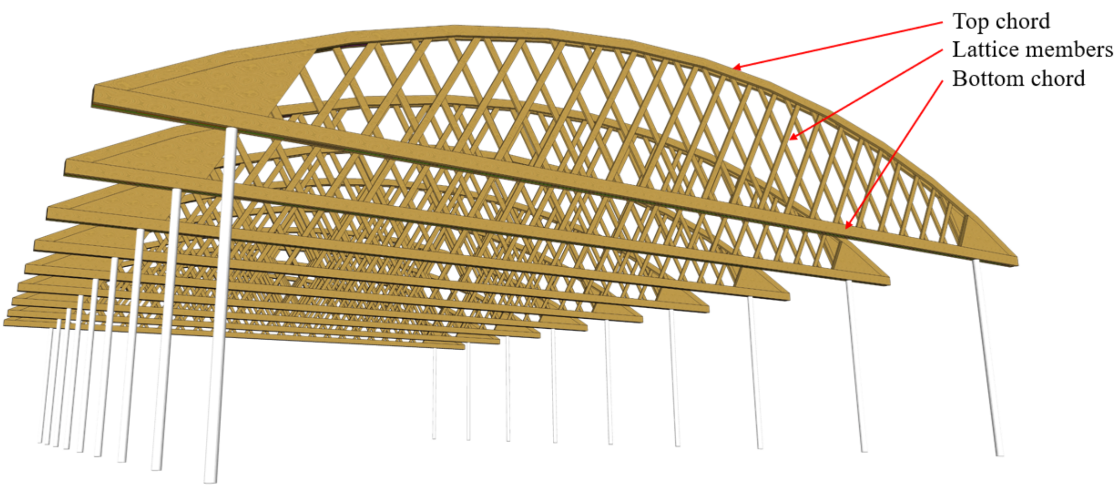


Figure 6-8 Belfast Truss design

6.2.2. Belfast Truss designs

An image of a standard Belfast Truss can be seen in Figure 6-8, with a bowed top chord and a flat bottom chord with lattice bracing connecting both cords. There are a number of variations and definitions of this design with conflicting opinions. However, the Belfast

Truss design is a very efficient system for long span truss design in an account of the shape and corresponding bending moment distribution [219], see Figure 6-10.

The first Belfast Truss system is documented in 1866 when McTear & Co. Promotions described the system as a 'durable, cheap and handsome roof for felt'[219].

Figure 6-9 Belfast Trusses at Esgair Timber (photo courtesy of Barratt Associates)

Comparative analysis for C16 as defined within EN 338 [220] and the newly defined home-grown C16+ for British spruce (WPCS) [221-223], were undertaken and optimised for each timber design class specification.

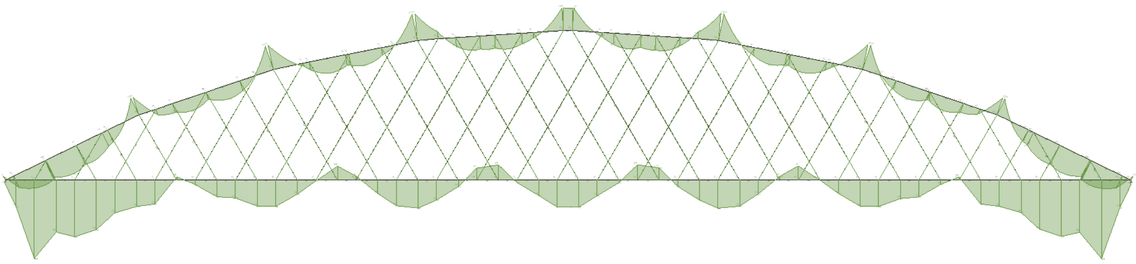


Figure 6-10 Bending moment example diagram

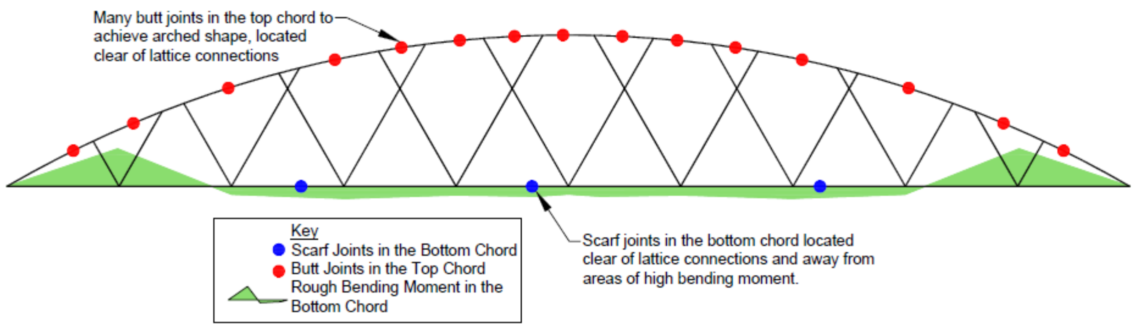


Figure 6-11 Appropriately located butt and scarf joints

There were a number of design challenges to be overcome in order for this system to work, and one example is of the connections within the top and bottom chords, as illustrated within Figure 6-11, Figure 6-12 and Figure 6-13. This is in addition to the connections of the lattice members to the top and bottom cords.

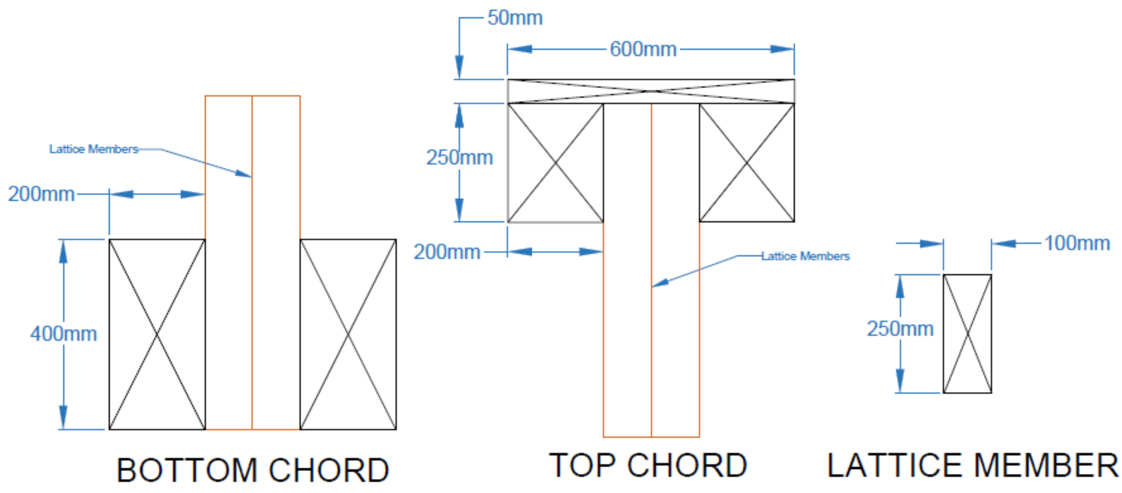


Figure 6-12 Cross-section of truss members, before final optimisation with respect to timber density

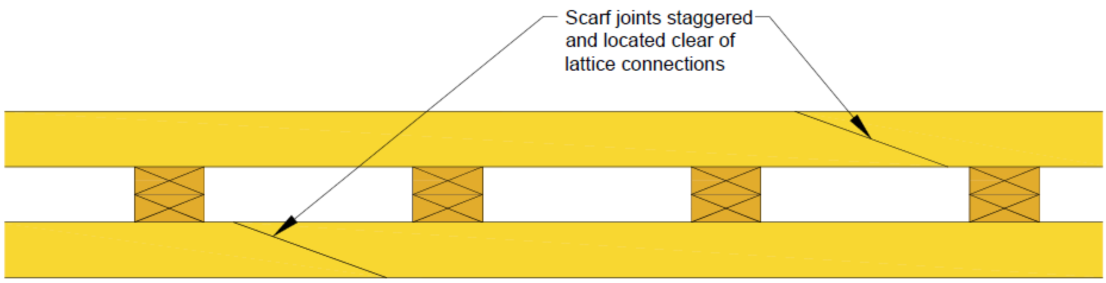


Figure 6-13 Plan view of bottom chord, Scarf joint

6.2.3. UK Home-Grown Timber

At present 52% of the wood fibre based panels (i.e. OSB) and 38% of dimensional timber are from home-grown sources. However, it is predicted that the production of timber in the UK is to increase by 50% by 2025 [223]. As the UK timber industry increases, it will be preferable that as a nation we find ways of utilising the home-grown resource better and reduce the net import of structural timber into the UK. From the latest Forestry Commission forecasts, we can see the current spread of species within the UK, see Figure 6-14 and Table 6-1 [224, 225]. For the remainder of this case study, we will be focusing on the utilisation of the largest proportion of homegrown timber - British spruce (WPCS).

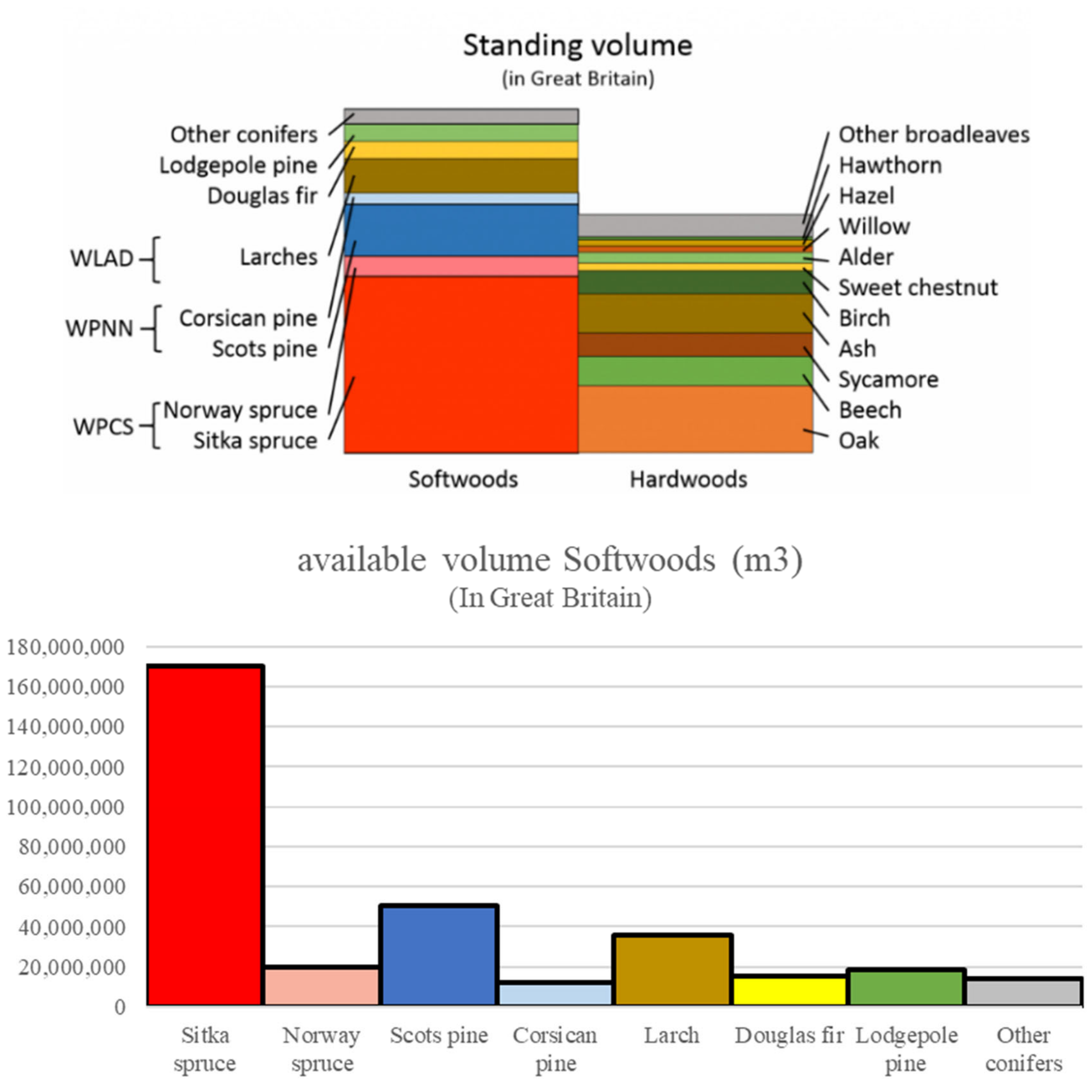


Figure 6-14 Standing and Available over bark volume of home-grown wood (in Great Britain) [224-226]

Table 6-1 Standing coniferous timber volume (overbark standing) by principal species (in Great Britain) [224-226]

		FE/FLS/NRW	Private sector	Total
		$000m^3$	$000m^3$	$000m^3$
	All conifers	124,575	211,167	335,742
WPCS	Sitka spruce	70,766	99,247	170,012
WPCS	Norway spruce	6,678	13,067	19,745
WPNN	Scots pine	12,930	37,732	50,662
WPNN	Corsican pine	6,309	5,648	11,958
WLAD	Larch	9,235	26,887	36,122
	Douglas fir	5,232	9,823	15,055
	Lodgepole pine	9,798	8,486	18,285
	Other conifers	3,627	10,547	14,174

where: FE/FLS/NRW = Forestry England, Forestry and Land Scotland, Natural Resources Wales

Home-grown timber is often seen as inferior to imported timber because a relatively small amount of the UK spruce can be graded to C24 and the industry grades it to C16 or rejects as routine. However, with perfect grading machines, the same material can achieve the results shown in Table 6-2.

The reason that British spruce cannot currently achieve high yields of C24 is that not enough of it can reach the requirement for strength, stiffness and density. The main limiting property is stiffness, rather than strength and density. The timber grading is determined by three "grade determining properties" (strength, stiffness and density). All three properties must be higher than the limiting values, so the strength class is actually a minimum strength class, two of the values may indeed be greater than that of the strength class that the timber is grated to, and the strength class grades are well defined for the bulk of the timber species used within construction [227].

Table 6-2 UK Home-grown timber C24 grading potential

Percentage achieving C24 grade	Timber species
~30%	British spruce
~75%	UK larch
~90%	UK IE Douglas fir

Work undertaken by Edinburgh Napier University has identified that the grading scale as set out within BS EN 338 [220] is not necessarily the best fit for some of the UK home-grown timber species. For example, if a timber sample has been assigned a strength class graded based upon the stiffness being the limiting factor, then there is potential that the bending strength and the density will be much higher than those declared. This is often the case with UK timber [221]. It is commonly stated that UK home-grown timber grows too quickly. This implies that the timber has low density. However, studies at Edinburgh Napier University on one of the most popular UK timbers, Sitka spruce, have shown that the density is the least limiting factor of the timber. It is correct that it achieves saw log size with a short rotation; but the drawback to this is not low density but low stiffness due to the large ratio of juvenile wood [221].

Within the UK, British spruce (WPCS) which is mainly Sitka and Norway spruce is graded on a pass or fail process to C16, but as shown in Figure 6-15 the density and bending strength are much higher than limits set within BS EN 338; and the limiting factor is the stiffness. UK timber producers and sawmills have been looking into marketing a bespoke strength class C16+ strength class, which with greater characteristic strength and density values, better fits the properties of UK grown spruce. Table 6-3 demonstrates the uplift from the standard C16 and the C16+ strength class requirements. The declared values for the C16+ come from [222].

Material properties - Bespoke strength class C16+

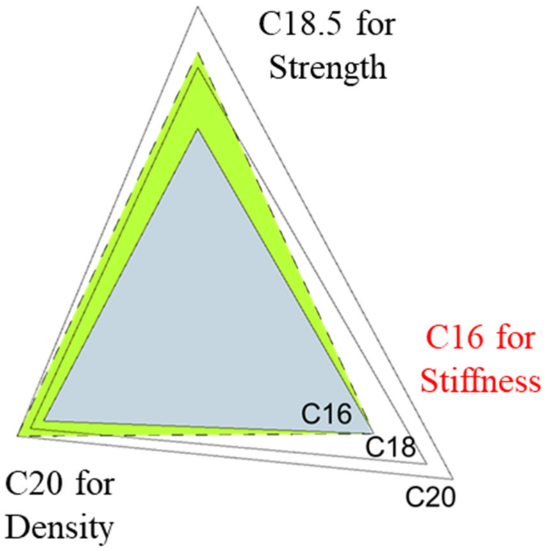


Figure 6-15 The characteristic properties of C16+ Comparison

Table 6-3 Some strength class requirements [222]

This information is know imbedded into Tekla TEDDS software platform. With a user base of over 16,000 user within the UK.	Class	C16 EN 338	C16+	C16+ Uplift
Strength properties in N/mm^2				
Bending	$f_{m,k}$	16	18.5	15.6 %
Tension parallel to grain	$f_{t,0,k}$	8.5	10.4	22.4 %
Tension perpendicular to grain	$f_{t,90,k}$	0.4	0.4	0.0 %
Compression parallel to grain	$f_{c,0,k}$	17	18	5.9 %
Compression perpendicular to grain	$f_{c,90,k}$	2.2	2.3	4.5 %
Shear	$f_{v,k}$	3.2	3.5	9.4 %
Stiffness properties in kN/mm^2				
Mean modulus of elasticity parallel to grain	$E_{0,mean}$	8.0	8.0	0.0 %
Char. modulus of elasticity parallel to grain	$E_{0,k}$	5.4	5.4	0.0 %
Mean modulus of elasticity perpendicular to grain	$E_{90,mean}$	0.27	0.27	0.0 %
Mean shear modulus	G_{mean}	0.50	0.50	0.0 %
Density in kg/m^3				
Char. Density	ρ_k	310	330	6.5 %
Mean density	ρ_{mean}	370	400	8.1 %

6.2.4. Timber connections and the effect of timber density

In order to calculate and optimise the structural timber connection designs, this case study utilised the newly created code-compliance structural timber connection calculation software, to see an output example please see Appendix K. The Belfast Truss design by its nature contains many distinct timber connections. The EC5 calculations for these are complex and time-consuming to do by hand. By utilising the code-compliance calculations connection properties can not only be calculated in an efficient manner, and they can be optimised in a way that would not otherwise be feasible.

The equations used in EC5 rely upon three main parameters of influence for the load-carrying capacity and behaviour of joints with dowel type fasteners, which are:

1. The bending capacity of the dowel or yield moment;
2. The withdrawal strength of the dowel;
3. The embedding strength of the timber or wood-based material.

Note: the timber density is the only timber property that is used in the calculation of lateral load-carrying capacity.

6.2.5. Discussion and Findings

Home-grown timber is often perceived as inferior to its imported equivalent but from the results of this case study, it has been demonstrated that home-grown timber can satisfy all the Eurocode 5 structural checks for this Belfast Truss design. When designing the connection, the only timber property that the calculation takes into account is the density of the timber. The new created C16+ grade of UK home-grown timber has a greater density than that of standard graded C16.

When designing this Belfast Truss example the effect of the density properties of C16+ was compared to standard C16. It was observed that the calculated minimum size of the timber members and the fixings can be significantly reduced. For illustration, the calculated size of the bottom timber chords and the size and number of bolts in the connection were able to be reduced by 34% and 47% respectively, see

Table 6-4, **Error! Reference source not found.** and **Error! Reference source not found.**. When repeating the calculations for this reduced design using the standard C16 grade, the work identifies failure in shear capacity. This proves that utilising the greater density characteristics of the C16+ graded home-grown timber can provide stronger connections with fewer fasteners, thus allowing section sizes of the members to be reduced. This was made possible by the Tedds code-compliance calculations allowing for repetitive design optimisation.

$$Timber\ section\ savings = \left(1 - \frac{350mm \ \times 150mm}{400mm \ \times 200mm}\right) = 0.34375 \ \rightarrow 34.3\% \qquad (6\text{-}1)$$

$$Metal\ fixing\ savings = \left(1 - \frac{6 \times \pi \times \left(\frac{16mm}{2}\right)^2}{9 \times \pi \times \left(\frac{18mm}{2}\right)^2}\right) = 0.47325 \ \rightarrow 47.3\% \qquad (6\text{-}2)$$

Table 6-4 Connection Design for C16 and C16+ timber members

Timber	Density ρ_k kg/m^3	Shear Capacity Utilisation	Splitting Capacity Utilisation	Connection
C16	310	0.843	0.809	9 no. M18 bolts, 400x200mm chord
C16+	330	0.803	0.809	9 no. M18 bolts, 400x200mm chord
C16	310	1.037	0.998	6 no. M16 bolts, 350x150mm chord
C16+	330	0.977	0.998	6 no. M16 bolts, 350x150mm chord

Impact of Findings: this example of a 30m clear span Belfast Truss roof system using UK home-grown timber graded to C16+ as opposed to standard C16 grade timber generated savings of:

- Timber section dimensions savings of 34.4%
- Metal fixings (bolts) savings of 47.3%

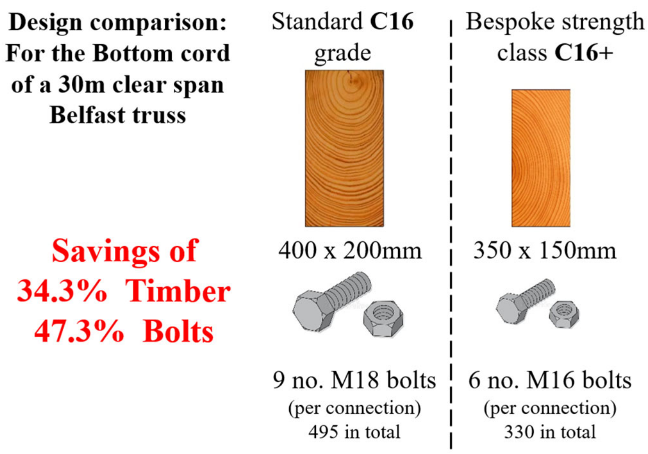


Figure 6-16 Design comparison for Belfast Truss,

These savings are considerable and would make a large difference to the feasibility and cost of using homegrown timber to construct Belfast Trusses. This has been published in the 6th European Conference on Computational Mechanics [218] and more details can be found in that publication. This case study demonstrates three points. Firstly code-compliance calculations can allow complex timber designs to be modelled and optimised with ease. Secondly, that accurately grading timber can make large differences in construction design. Finally, it demonstrates that UK homegrown timber is more able to be used in structural design than was previously thought.

As a result of the work within this case study, the strength class requirements for the bespoke strength class C16+ have implemented into all of the timber design code-compliance calculations within Tekla tedds library, see Figure 6-17. The Tekla Tedds software platform has over 16,00 users within the UK.

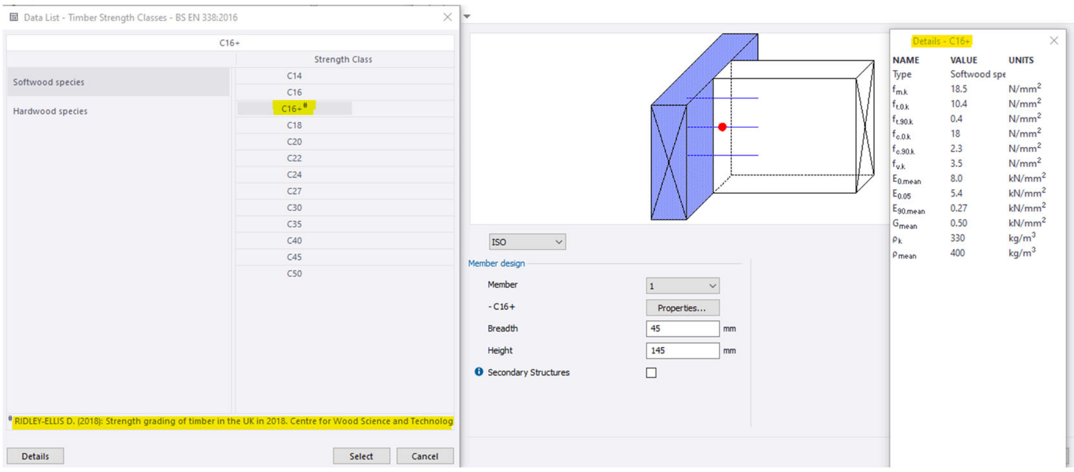


Figure 6-17 Implementation of C16+ into Tedds

6.2.6. Summary

This case study fulfilled objectives 4, 5 and 6

Reduce barriers for timber specification and connection design by:

4. To Create case studies demonstrating the advantages of parametric methodology within EC5 timber connections;
5. To Create case studies demonstrating the benefits of a transition to EC5 through the ability of optimisation;
6. To identify and utilising routes for current research to be implemented into the AEC sector.

This project was ideal for demonstrating a computational timber connection calculation software previously created within this research. This software allowed for parametric optimisation of the Belfast Truss design that was used to compare the advantages between the standard C16 grade timber from EN 338 and the new bespoke strength C16+ home-grown timber. This was only possible as a result of the parametric methodology adopted within the Eurocodes and in particular EC5, **fulfilling objectives 4 and 5.**

The work within this case study was published within an international conference paper [218]. It also resulted in having the bespoke strength class of C16+ home-grown timber, which is an output of research from Edinburgh Napier University being implemented into the library of calculations for Tekla Tedds platform that has a network of over 16 thousand users within the UK, **fulfilling objective 6.**

6.3. Case study: Challenging bespoke mass timber house design

Figure 6-18 Nedd, Site location, photos from Channel 4

Carbon dynamic was commissioned on a design and build contract for a unique house. The site has a very limiting set of conditions, being remote and with a dramatic landscape, situated high on the side of an isolated cliff overlooking Loch Nedd, with views overlooked by mountains, as can be seen in Figure 6-18. With unpredictable weather and an inaccessible location, the site is only accessible by the means of an 8-mile narrow single-track B-road. The site is a mix of bog and exposed bedrock. Perhaps this is why the project was the first in a new series on channel 4 “Impossible builds: Charlie Luxton and Aidan Keane meet ambitious families who are building innovative bespoke homes in some of the UK's most remote and challenging locations”.

Location:	Nedd, Scottish Highlands, IV27 4NN
Year:	2017
Partners:	Carbon Dynamic Edinburgh Napier University Design Engineering workshop

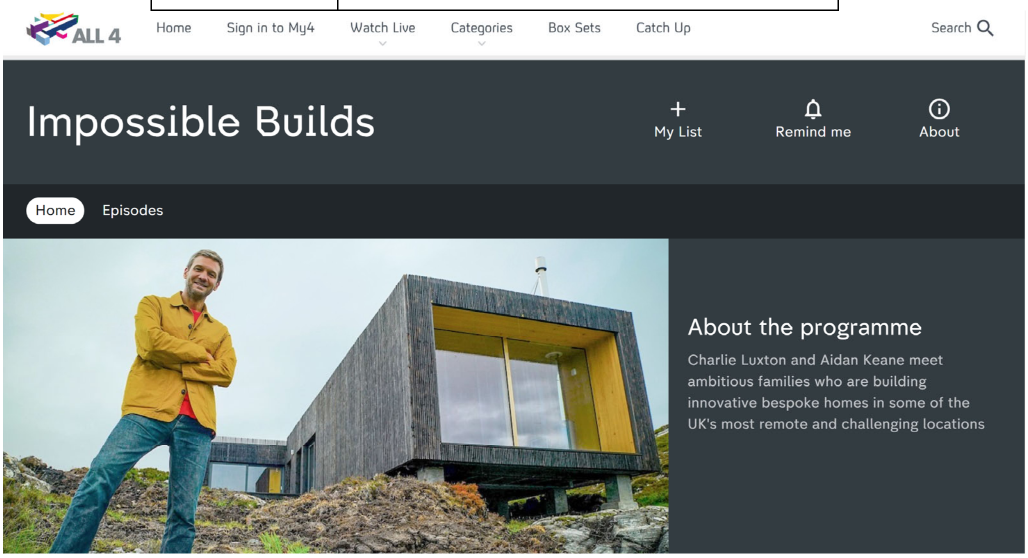


Figure 6-19 Chanel 4 link for the Nedd project https://www.channel4.com/programmes/impossible-builds

6.3.1. Challenging test case

The Nedd house project was the first live project using the newly created code-compliance timber connection calculations, providing a testbed for real-world functionality testing. As a result, improvements were made that extended functionality and user experience. Some bug fixes and improvements were incorporated, for example, adding the ability to calculate connection strength with CLT connections when the screw is connected into the edge face of CLT.

Because of the site limitations, the house was conceived as an offsite constructed volumetric design, as illustrated in Figure 6-20, Figure 6-21 and Figure 6-22. For accessibility down the 8-mile narrow single-track road, the modular units needed a narrow width. The superstructure is sitting elevated above the ground on top of the foundations that are a mix of pads into the bog and piers embedded into bedrock.

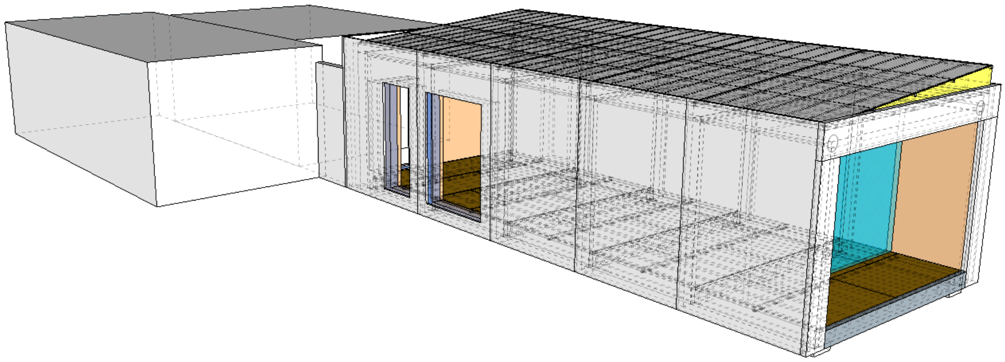


Figure 6-20 Nedd house 3D drawing

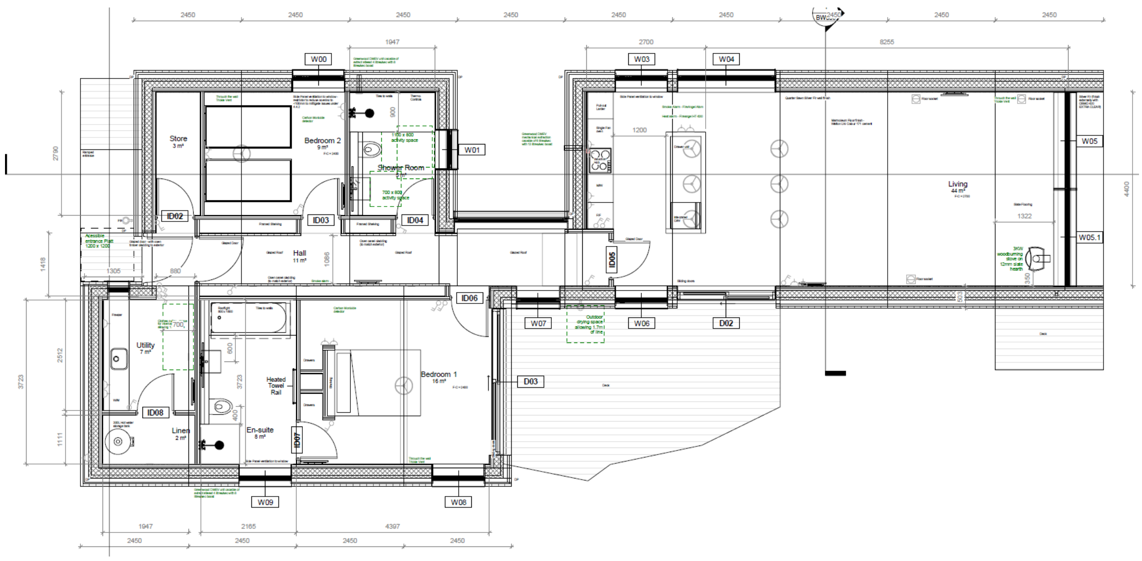


Figure 6-21 Nedd house plan

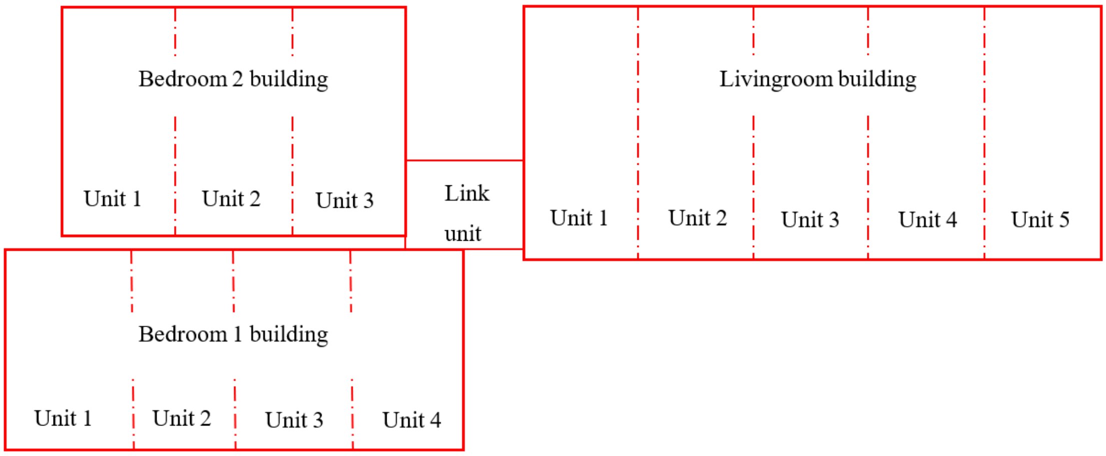


Figure 6-22 Nedd house unit number plan

The structure itself is simple single-story construction using glue-laminated timber panels (GLT) for the walls, cross-laminated timber (CLT) for the roof and a timber frame floor system. The structure also includes two portal frames for picture frame windows, as can be seen in Figure 6-23 and 6-25.

Figure 6-23 Completed project photo, Nedd house, photo from Channel 4

A student was given training on the newly created code compliance calculation tools and was then mapped onto this project with supervision. All the timber connections and the full superstructure of the build were calculated. In addition, the structural engineers' registration (SER) certification was prepared and submitted, which was used by Carbon

Dynamic to secure the building warrant approval. The fact that an inexperienced engineer could be trained to tackle such a demanding project shows both the user-friendliness of the software and its applicability and power.

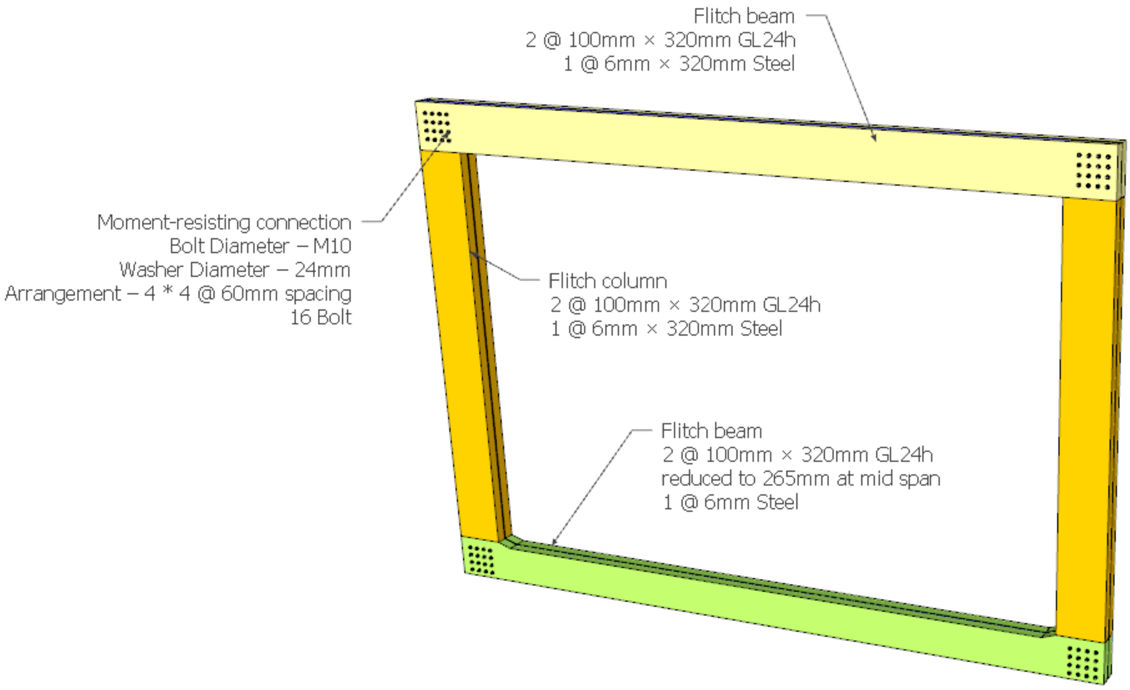


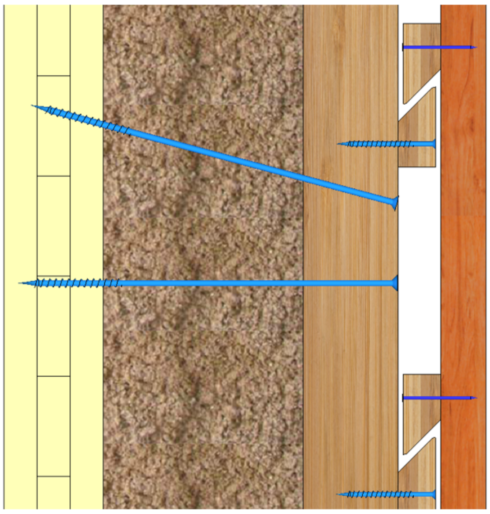
Figure 6-24 Connection details (summary) for a portal frame (picture frame window), Nedd house

6.3.2. Research focus – software testing and optimisation

At this time, the code-compliance timber connection tools were in the process of being written. The first connection calculation had been released to the public, but other calculations used were not yet released. Using the software on a live project was a good test of the software's capabilities and usability. As a result, many changes were implemented and improvements were made.

The main limiting factor for this project was the timber connections. One of the critical calculations was the external timber cladding connection. The project had a very high wind load, due to its location. As a result, the connection had to be both particularly robust and easy to install onsite. Figure 6-25 shows the final design.

External cladding, drawing



Cladding 45mm × 30mm with a 10mm gap
3.1 × 75mm Ring shanked nail

Horizontal batten 38mm × 83mm C16, 45° cut @ 400mm ctr
pre-drilled 6.0 mm / 3.6 mm x 100 mm screw
Eurotec-945634

Vertical batten 95mm × 48mm C16 @ 600mm ctr.
pre-drilled 8.0 mm / 5.1 mm x 380 mm screws
Eurotec-945748

External cladding, Cladding to H. batten connection

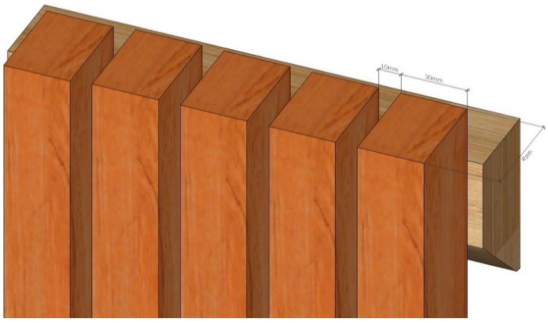
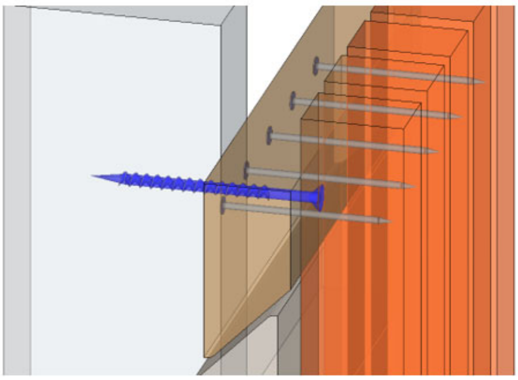


Figure 16 Cladding iso

Cladding boards 25mm × 45mm

$E_d = \gamma_{Q.EQU} \times (25mm \times 400mm \times 1.67\ kPa) =$ **0.025** kN; wind suction
See Cladding to H. batten, connection calculation.

Pass - 3.1 × 75mm Ring shanked nails Utilisation 0.156



Note: the cladding panel will have fixings to prevent uplift and to allow removal for maintenance. The fixings will only be resisting uplift, therefore only required at the corners of the panel.

Figure 6-25 Cladding Connection; top three: connection design, bottom: completed connection

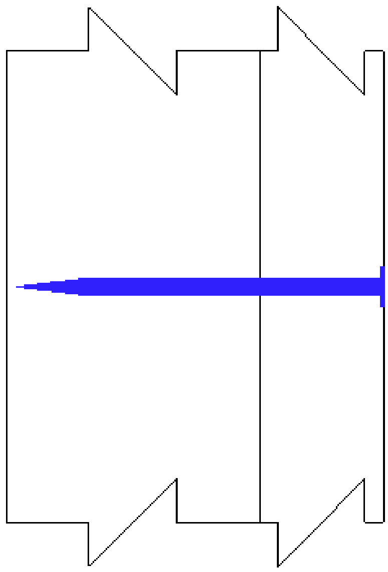
Other connection examples can be found in Figure 6-26 that shows plywood to roof joist connection summary output from Tedds.

TIMBER CONNECTION DESIGN ROOF PLY TO JOISTS (EN1995)

ROOF PLY TO JOISTS

Timber connection design in accordance with EC5, the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.1.00



Section

Actions

Load-duration **Instantaneous** Modification factors k_{mod} = **1.10**

Design axial action $F_{ax.Ed}$ = **0.19** kN

Withdrawal resistance

Characteristic values of the withdrawal and pull-through strengths

Pointside withdraw, User entered $f_{ax.k.ps} = f_{ax.k.pss}$ = **4.50** N/mm^2

Headside pull-through, User entered $f_{head.k.hs} = f_{head.k.hss}$ = **5.00** N/mm^2

Characteristic withdrawal capacity - eq 8.23 $F_{ax.Rk} = \min(f_{ax.k.ps} \times$
$d_f \times t_{pen}, f_{head.k.hs} \times d_{h.f}^2)$ = **0.243** kN

Design value of axial withdrawal capacity $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.206** kN

Load utilisation factor ut_load = $F_{ax.Ed} / F_{ax.Rd}$ = **0.938**

PASS - Design axial withdrawal capacity exceeds the design axial action

Fixing Spacing

Allowable minimum nail spacings from table 8.2

Minimum edge / end distances.

a3,c Distance between fixing and unloaded end

a4,c Distance between fixing and unloaded edge

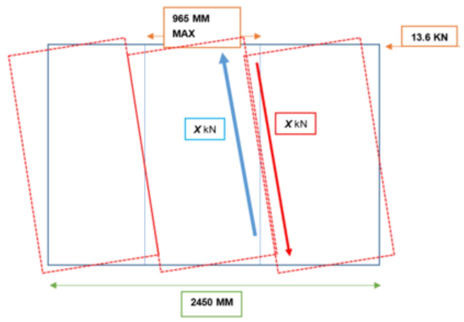
	a3,c	a4,c
Point side member	21.7 mm	9.3 mm
Head side member	21.7 mm	9.3 mm

Figure 6-26 Roof Connection Design

Connecting two GLT panels together for racking wall resistance

Please note that the GLT panels are 965mm wide, and are used to construct the external wall panels of the units. Providing the racking resistance.

In connecting the GLT wall panel together, a lateral shear load will occur on the connection, illustrated below.



Width, b	b = 965 mm
Height, h	h = 3300 mm
Angle, θ	$\theta = \mathrm{Atan}(b / h)$ = **16.300**
Wind Force, F_{wind}	F_{wind} = 13.6 kN
Horizontal Force Component, F_h	$F_h = \sin(\theta) * F_{wind}$ = **3.8** kN
Vertical Force Component, F_v	$F_v = \cos(\theta) * F_{wind}$ = **13.1** kN

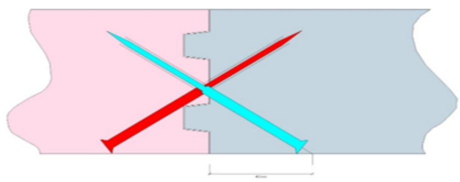


Figure 6-27 Connecting two GLT panels together for racking wall resistance

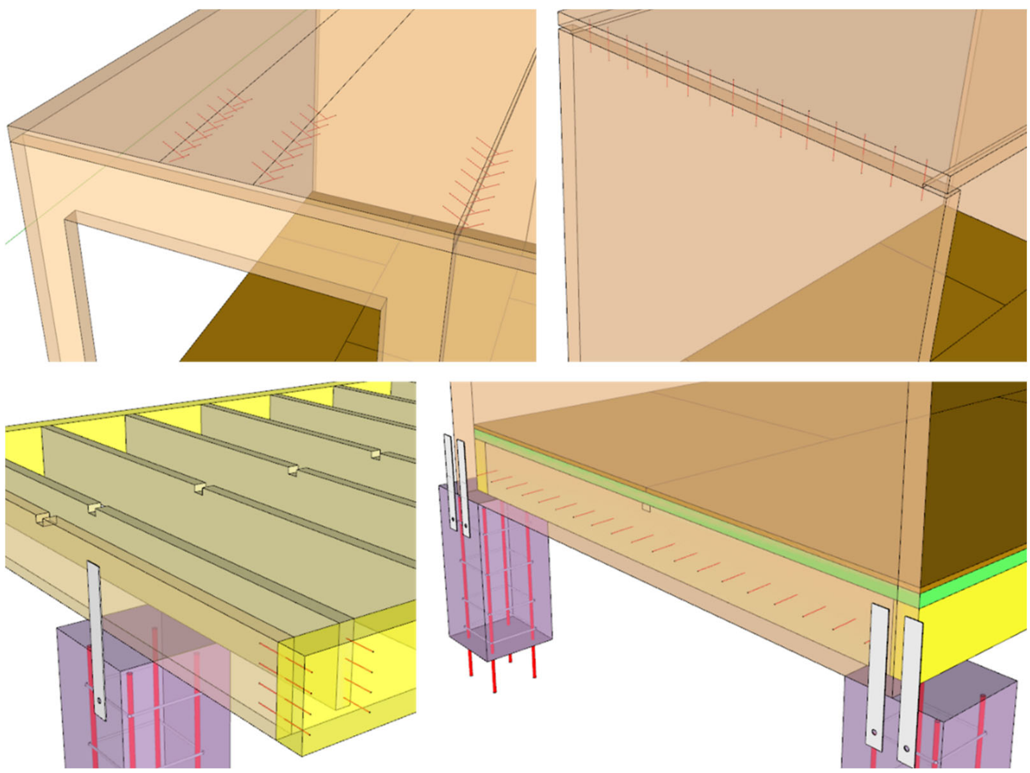


Figure 6-28 3D drawings: Floor joist, Wall panel, roof connections

6.3.3. Summary

This case study fulfilled objectives 2 and 3c:

Reduce barriers for timber specification and connection design by:

2. To create and deliver educational material of current research, for the purpose of increasing the level of knowledge of structural timber for both university students and practising engineers;
3. To reduce the complexity of EC5 through automation of timber connections:
   c. Creation of a robust code-compliance structural timber connection calculations within the identified platform.

This high-profile project that was in the spotlight for a one-hour factual documentary television program, highlighted the benefits of designing using timber offsite design for manufacture and assembly (DFMA) process and by giving an Edinburgh Napier University student the opportunity to gain industry experience on a live project such as this**, fulfilling objective 2.**

This case study project was invaluable for the development proofing of the created code-compliance calculations**, fulfilling objective 3c.**

Figure 6-29 Nedd, Offsite manufacture, upper: in factory, lower: onsite

6.4. Case study: Dyson Student Village

Figure 6-30 Dyson case study: Site photo #1[1]

The Dyson Institute of Engineering and Technology student village was Architecturally designed by WilkinsonEyre and the design and build contract was awarded to Carbon Dynamic for off-site manufacture and assembly – see Figure 6-31 and Figure 6-32. The site can accommodate up to 50 students together with visiting staff and includes a communal space with a library, café, bar and screening room.

Figure 6-31 Carbon Dynamic offsite manufacturing facility

Figure 6-32 Dyson case study: On-site installation of units

Location:	Malmesbury, Wiltshire
Year:	2018
Partners:	• WilkinsonEyre • Matt Stevenson: Carbon Dynamic • Edinburgh Napier University • Design Engineering workshop • Binder Holz • Stora Enso

[1] Photo by Peter Landers, http://www.architecturetoday.co.uk/stack-effect-2/

The development was constructed using volumetric CLT units, entirely manufactured offsite and rapidly assembled on site. Edinburgh Napier University's Centre for Offsite Construction and Innovative Structures (COCIS) participated in the project and carried out structural design and testing. This was an ideal project to test the software developed as part of this thesis and highlight its strengths and potential. The author trained a student and together all the connections in the build were calculated. In addition, the author and student were involved in the structural design process and completed full-scale lab testing of a prototype building.

The student village is constituted of 78 volumetric units, variously assembled to form 19 clusters up to 3-storey high (Figure 6-34, Figure 6-35 and Figure 6-36). Each cluster includes a shared kitchen and laundry and an entry area with reception and storage. Each pod has its own access, either directly from the garden or by earth ramps and stairs. The accommodations include a toilet and shower room, an open-plan bedroom area and work/living space. The units were delivered to site fully-fitted with bespoke furniture and built-in storage (Figure 6-33).

Figure 6-33 Dyson unit internal photo

The buildings are constituted 92% of natural materials and each unit is provided with triple-glazed windows and natural ventilation. Carbon Dynamic took a fabric-first approach and applied Passivhaus principles to design. "A Passivhaus is a building in which thermal comfort can be achieved solely by post-heating or post-cooling the fresh air flow required for good indoor air quality, without the need for additional recirculation of air [228]"; heat sources are, therefore, the sun, occupants and household appliances; additional heat can be supplied up to 10W per square metre [229].

Figure 6-34 Dyson case study: Site photo #2[2]

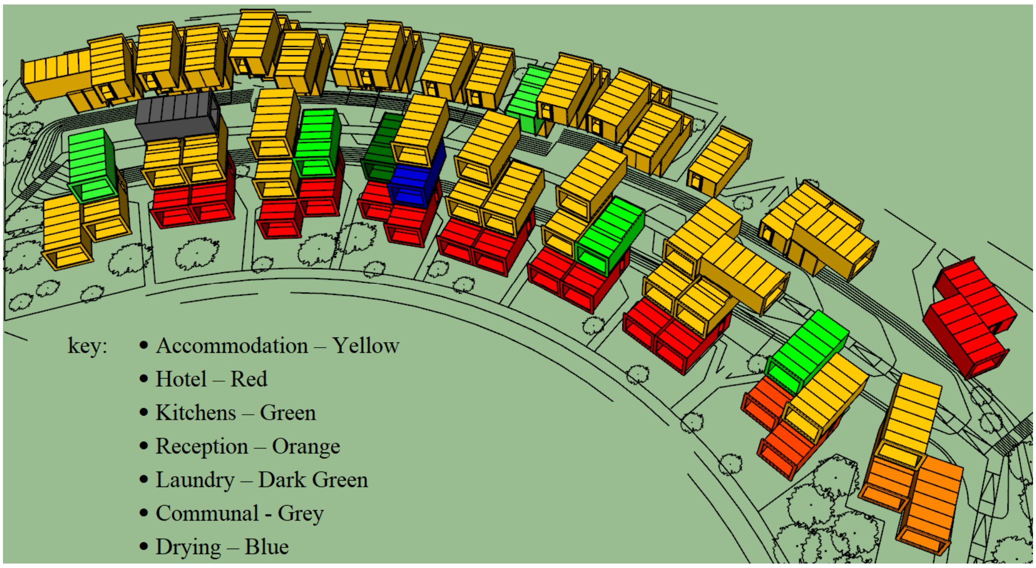


Figure 6-35 Dyson Village Overview

Figure 6-36 Dyson case study: Site photo #3[3]

[2] Photo by Peter Landers, http://www.architecturetoday.co.uk/stack-effect-2/
[3] Photo by Peter Landers, http://www.architecturetoday.co.uk/stack-effect-2/

6.4.1. Timber system

For this project, we designed, manufactured and assembled volumetric units made of Cross Laminated Timber (CLT) imported from Europe. The modules are 7.2m x 4.2m x 2.9m and can be assembled cantilevered to up to 3 meters. The project was developed using two sets of prototypes for the client, a design prototype (Figure 6-37) and a structural prototype (Figure 6-39 and Figure 6-41). The design prototype included interior finishes, furniture, electrical access points and cladding and was used for the client's review. It was also useful to finalise the module's weight and exact size. A technical prototype was built and used to assess the structural properties of the modules and verify the assembly process, the connection to the ground and the accuracy of the system. Due to standard load width restrictions, the external aluminium cladding was installed onsite in a dedicated buffer zone. A key element for the successful delivery of the project was the early design and engagement with the supply chain. Every aspect of the modules, from CLT connections to the position of power sockets, had to be finalised before manufacturing could start. In addition, although the modules look identical, there are small variations in their structural and design properties. To manage the variation between the modules, the student used a design interface matrix to control the balance between replicability and customisation.

Figure 6-37 Dyson: design prototype photo by Matt Stevenson

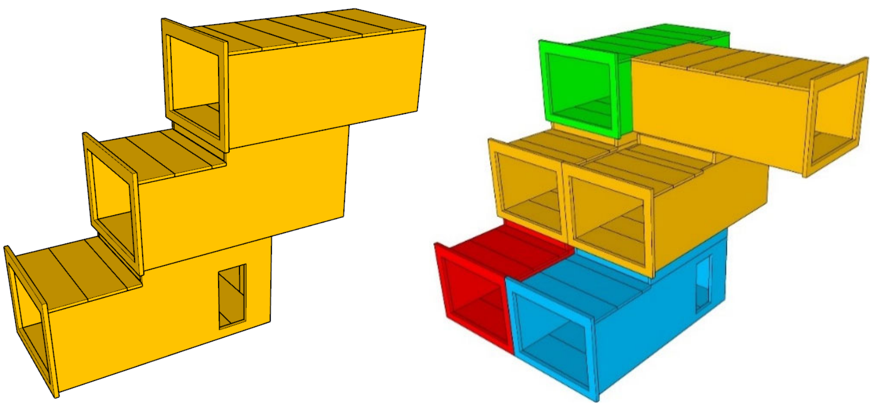


Figure 6-38 Sketchup models of Dyson units and cluster configurations

6.4.2. Research focus – connection design

The presence of cantilevered modules required the study of specific engineering solutions, especially for the connections between units. A new connection was designed for the Dyson Student Village, which consists of a steel plate bolted to the modules from the inside, see Figure 6-39, Figure 6-40 and Figure 6-42.

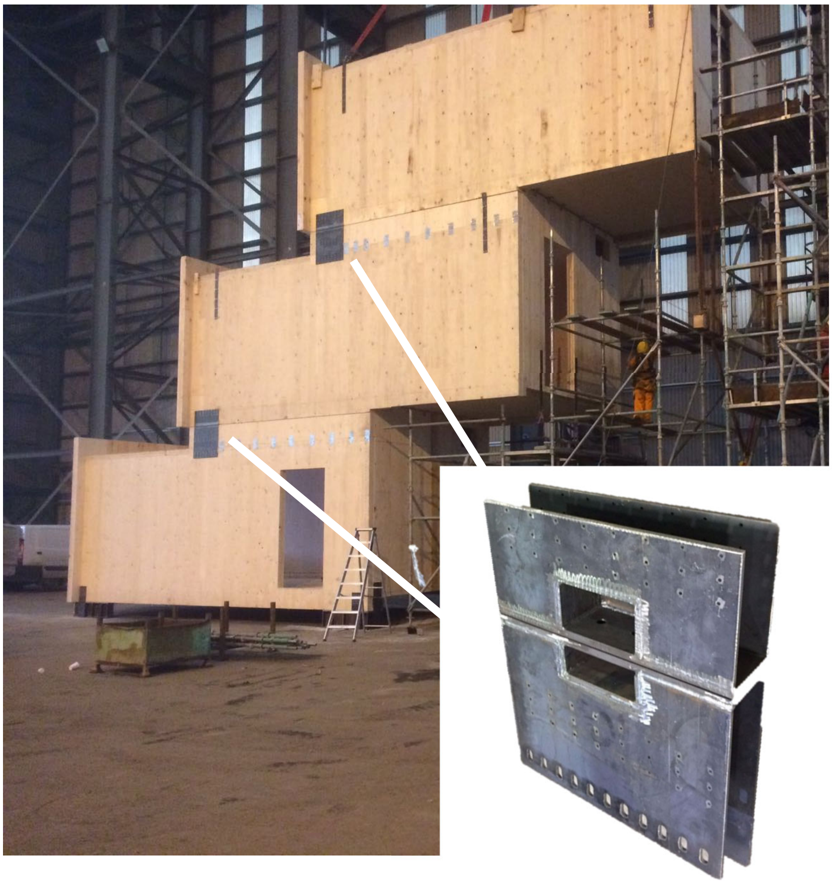


Figure 6-39 Dyson unit connection Detail

All the connections in the building were calculated using the code-compliance timber connection software discussed in this thesis. The critical connection discussed above carries a large load due to the cantilever design of the buildings. This connection design went through many iterations of feasibility checking and optimisation. As part of this process, the code-compliance calculations were heavily utilised to speed up the design process and make the discussion of so many different options possible. The final steel to timber connection was designed and it was decided to conduct a laboratory test to confirm the accuracy of the calculations.

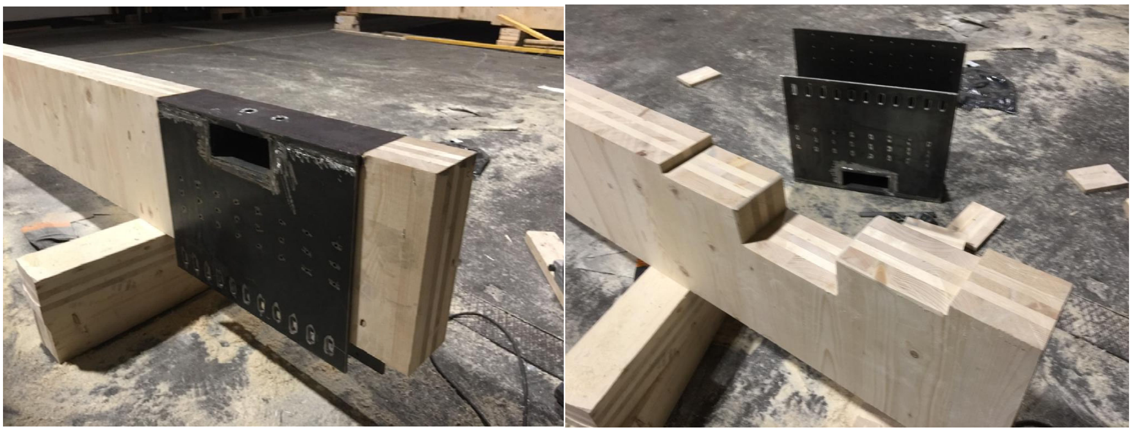


Figure 6-40 Dyson unit connection construction

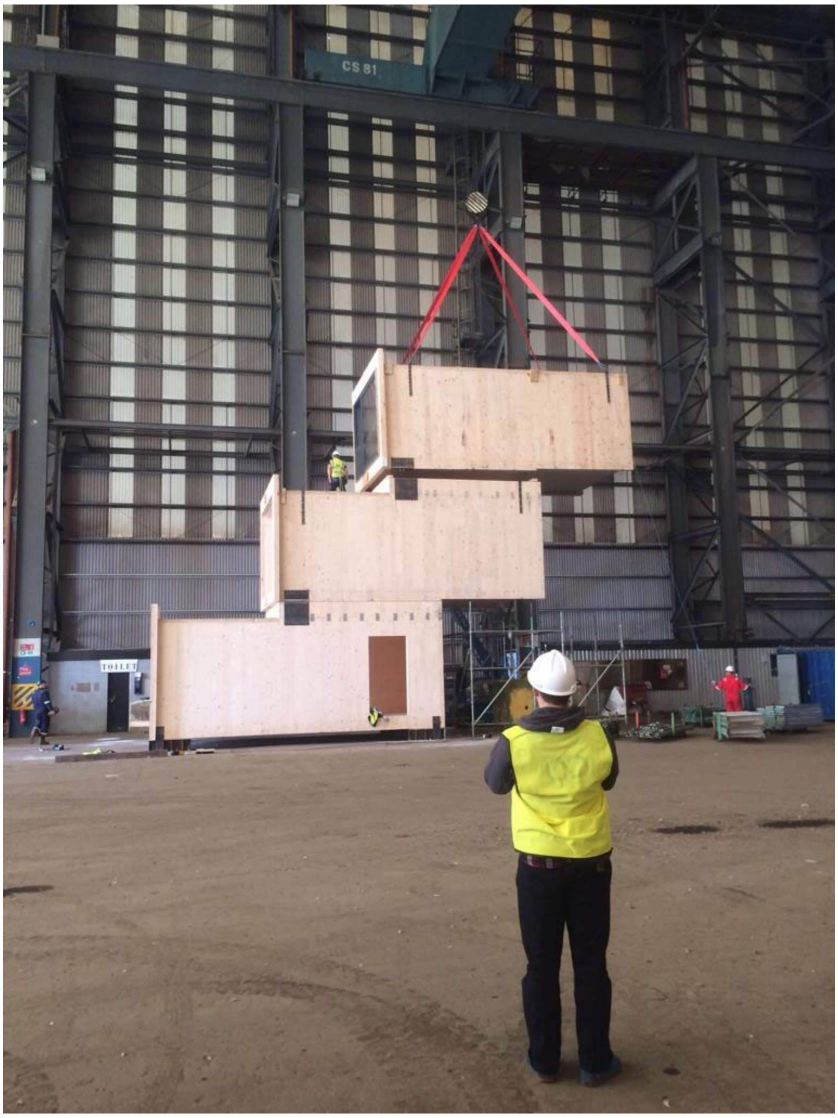


Figure 6-41 Dyson: Three-story static load testing

Figure 6-42 Dyson unit connection, internal view

6.4.3. Research focus – structural performance

Due to the cantilevered design (Figure 6-38 and Figure 6-41), there are strong forces on the timber structure and also the connection designed. The construction of the technical prototype was particularly useful, given that it allowed verifying the predictability and accuracy of the new connections.

The prototype was also used to carry out tests on the static load, vibration and acoustic properties of the structure. In particular, the deflection of the stacked modules was calculated and then tested under a static load applied by a testing rig.

The calculation method was based upon BS EN 380:1993 (Timber structures - Test methods - General principles for static load testing) and considered as total displacement, which is the combination of elements deflection and connections slip. This was calculated using the output of the code-compliance calculation with additional hand calculations.

Symbols

- F load, in newtons
- G permanent load self-weight, in newtons
- Q characteristic value of variable load, in newtons
- T loading time, in seconds
- T_r recovery time, in seconds

The principle of these test methods involves applying a stated regime of loading to a timber structure over a stated period of time, observing the corresponding deformations and reporting the test results. This test consists of three volumetric CLT buildings stacked up to form one complex.

The accuracy of the load and deflection measured was within ± 3%. The test loading was both applied and resisted in a manner approximating to the actual service conditions. Eccentricities, other than those necessary to simulate service conditions, were avoided at points of loading and reaction and care was taken to ensure that no inadvertent restraints were present.

Preparation

To determine the moisture content and density of the CLT at the time of the test, samples were taken and tested at a later date. Also the environmental conditions of temperature and relative humidity existing during the test was measured.

Basic loading procedure

The basic loading procedure consists of the procedural steps (0–10) described in Table 6-5. A diagrammatic representation of the loading procedure is given in Figure 6-43.

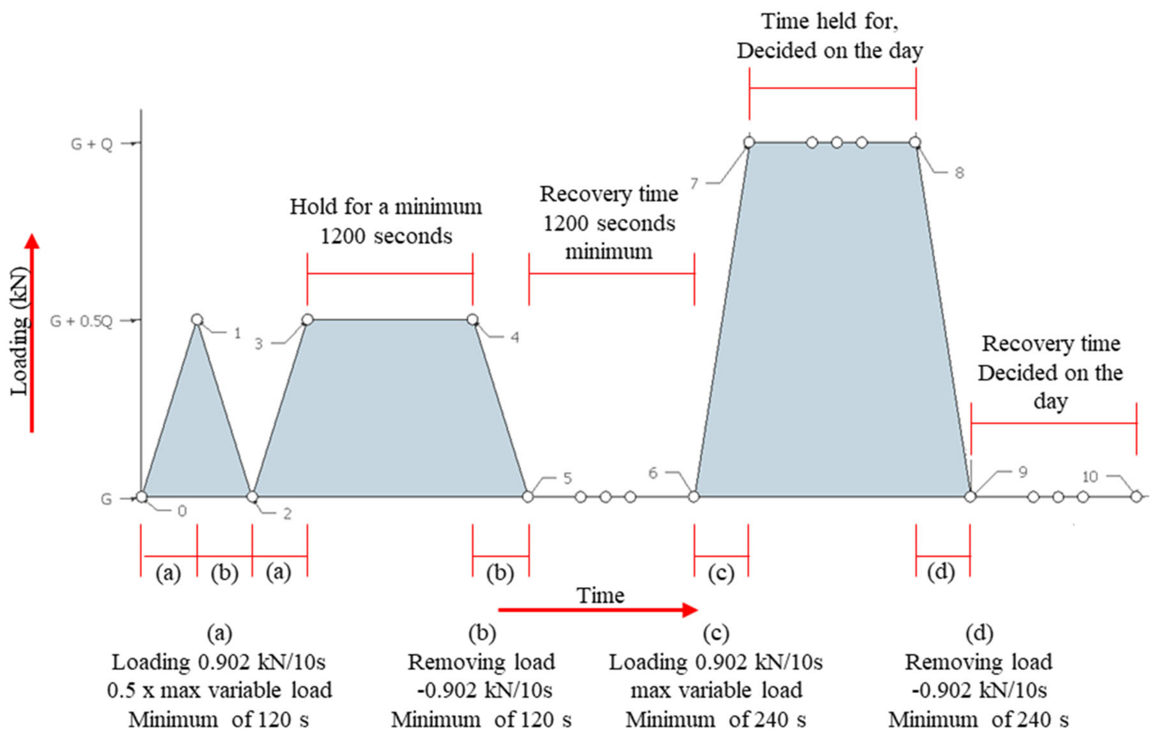


Figure 6-43 Dyson: Schematic loading procedure

Table 6-5 Dyson test loading procedure

Procedural Step	Loading Procedure	Time, in Seconds	Load rate, kg/10s	Load rate, kN/10s
0	Only G, F = 0	0	0	0
0 - 1	Apply F = 0.5Q	≥ 120	92	0.902
1 - 2	Remove F = 0.5Q	≥ 120	-92	-0.902
2 - 3	Apply F = 0.5Q	≥ 120	92	0.902
3 - 4	Maintain F = 0.5Q	≥ 1200 (20min)	0	
4 - 5	Remove F = 0.5Q	≥ 120	-92	-0.902
5 - 6	Recovery Time	≥ 1200 (20min)	0	0
6 - 7	Apply F = Q	≥ 240	92	0.902
7 - 8	Maintain F = Q	T	Decided on the day	
8 - 9	Remove F = Q	≥ 240	-92	-0.902
9 - 10	Recovery Time	T_r	Decided on the day	
• The maximum loading rate shall not exceed 0.25 Q per 60 s. • Load rate is subject to change given weight of loading apparatus.				

Test load application arrangement

The decision to apply the test load as a line load at the cantilever edge rather than a UDL over the length of the cantilever were to have the ability to apply and remove the load in a repeatable/uniform manner, within the time frame set out within BS EN 380:1993.

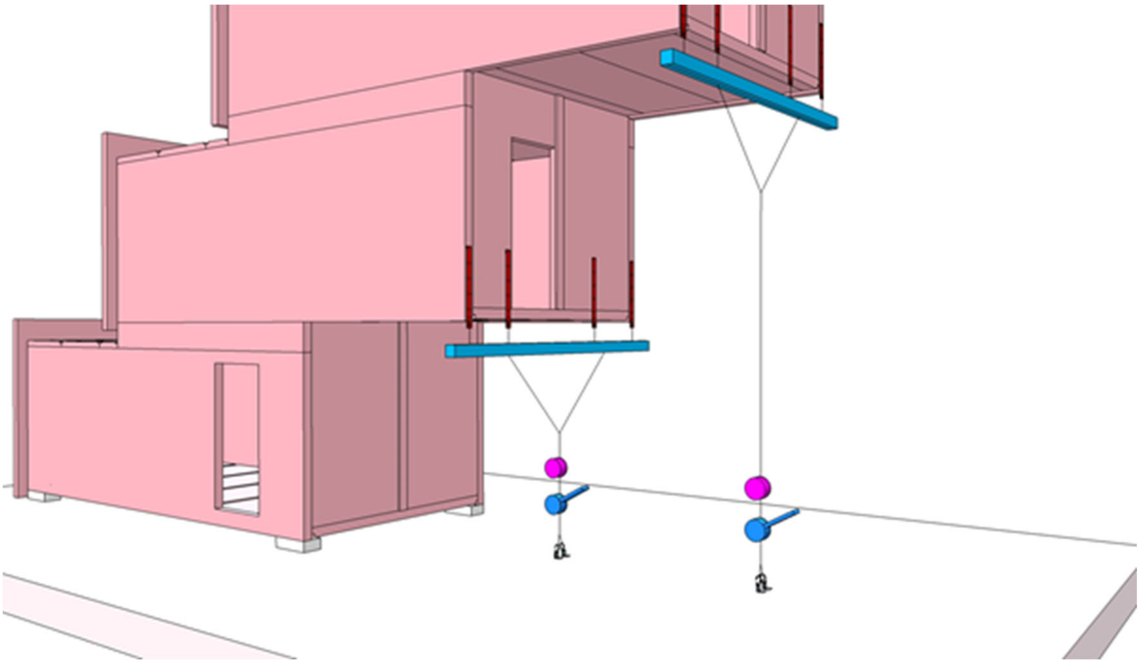


Figure 6-44 Dyson: Method of load application

There are two identical systems identified within Figure 6-44, which consist of four standard lifting straps, spreader bar, load cell and a remote-controlled electric winch.

The test loading was based on a 2 kN/m^2 floor loading; also the snow loading comes from the Dyson village prototype calculation report Section 4.2.1 with a load of 0.75 kN/m^2.

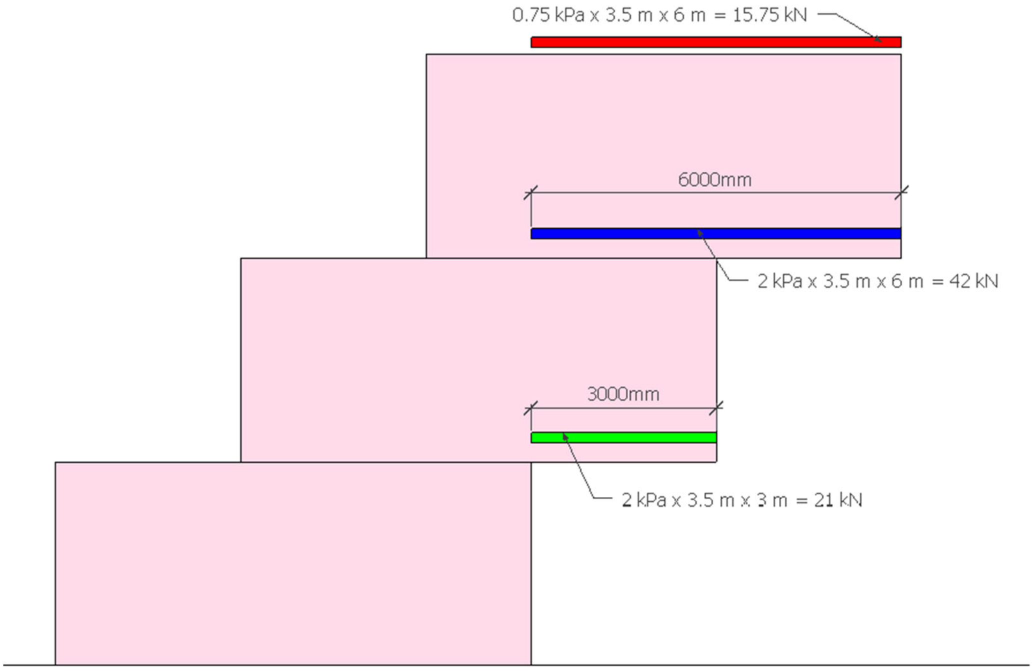


Figure 6-45 Dyson: Calculated load combination

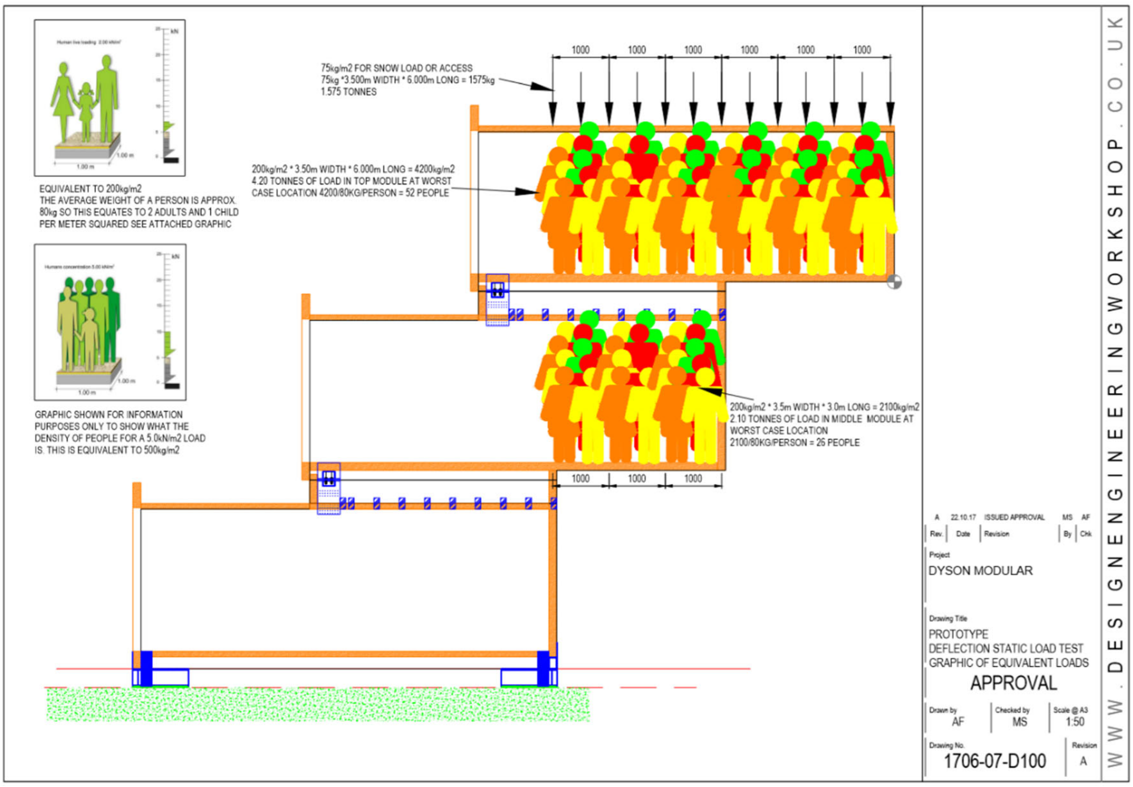


Figure 6-46 Dyson: Deflection static load test equivalent loads, (Image from DEWS Glasgow)

Method of calculating the equivalent point load for the test.

Max deflection for a point load on a cantilever beam; $\delta_{max.p} = (P \times L^3) / (3 \times EI)$

Max deflection for a UDL load on a cantilever beam; $\delta_{max.\omega} = (w \times L^4) / (8 \times EI)$

$(P \times L^3) / (3 \times EI) = (w \times L^4) / (8 \times EI)$ so $(8 \times P) / (3 \times w \times L)$

Equivalent point load = $w \times L \times 3/8 = P$

where

L	length of cantilever (m)
w	UDL load, $= kN/m^2 \times m$
P	point load (kN)

Variable loadings, Q

First floor:

Floor loading, $2\ kN/m^2 \times 3.5\ m \times 3\ m \times 0.375 =$	**7.875 kN**

Second floor:

Floor loading, $2\ kN/m^2 \times 3.5\ m \times 6\ m \times 0.375 =$	15.750 kN
Snow loading, $0.75\ kN/m^2 \times 3.5\ m \times 6\ m \times 0.375 =$	5.906 kN
Total, Second floor	**21.656 kN**

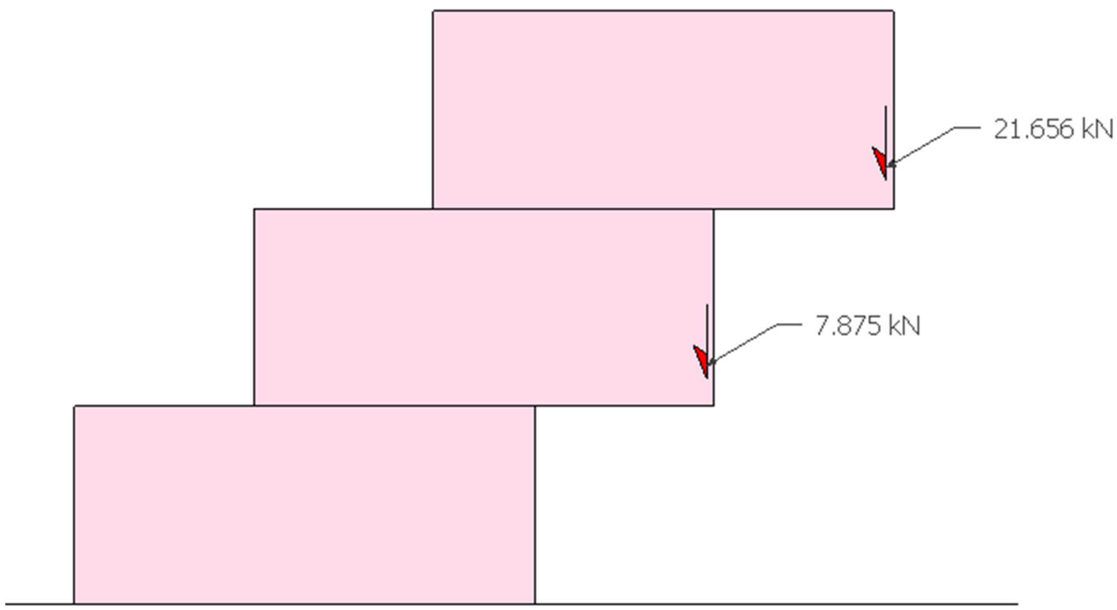


Figure 6-47 Dyson: Transposed Q loads into point loads

Recording of deflections.

The prototype building was constructed with all structural elements, see Figure 6-41. Cladding and internal fixings were not constructed, as they have a negligible structural impact. The prototype was fitted with seven dial gauges, as shown in Figure 6-48 and Figure 6-49. These provided a recording of the deformations of the prototype under the loads applied.

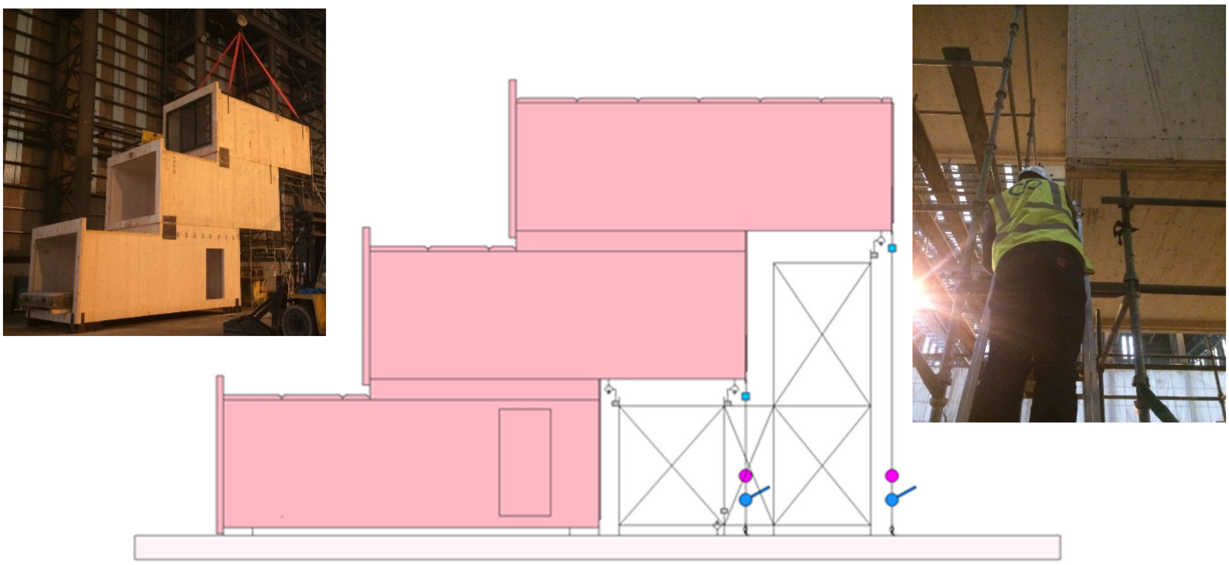
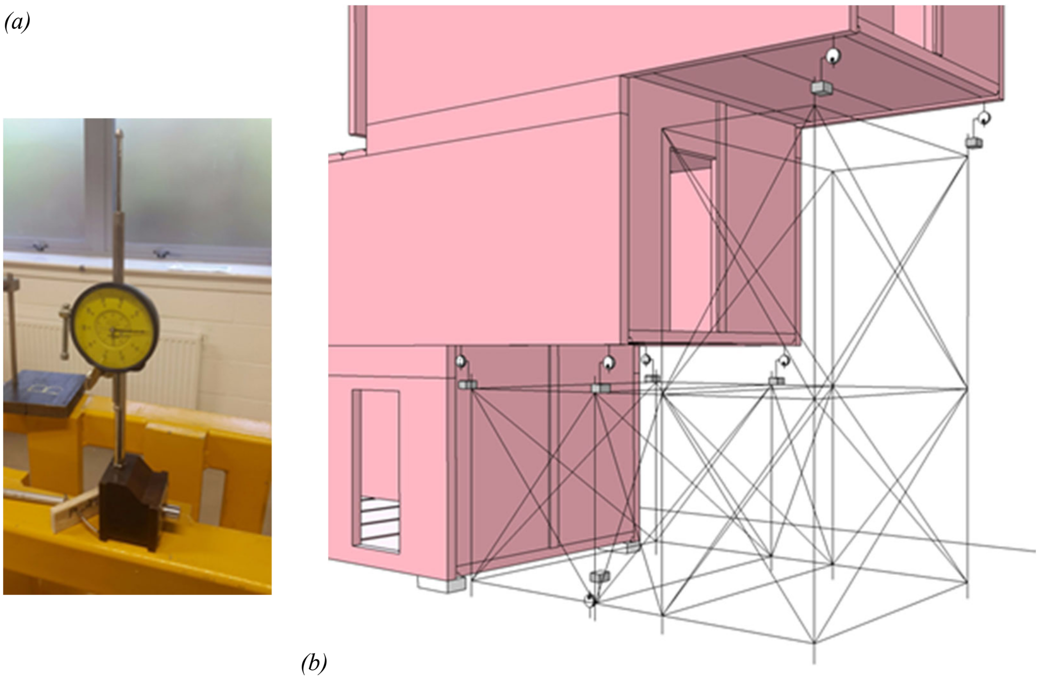


Figure 6-48 Dyson: Test Layout and Construction

(a)



(b)

Figure 6-49 Dyson: (a) Mechanical dial gauge (b) Dial gauge arrangement

According to the calculation the total displacement of the modules under static load was expected to be 11.3 mm as shown in Figure 6-50; displacement result given by the test was better than predicted, with only 8.3 mm.

When the loads were released, the system went back to its original state, demonstrating both the robustness and the flexibility of the engineered volumetric system.

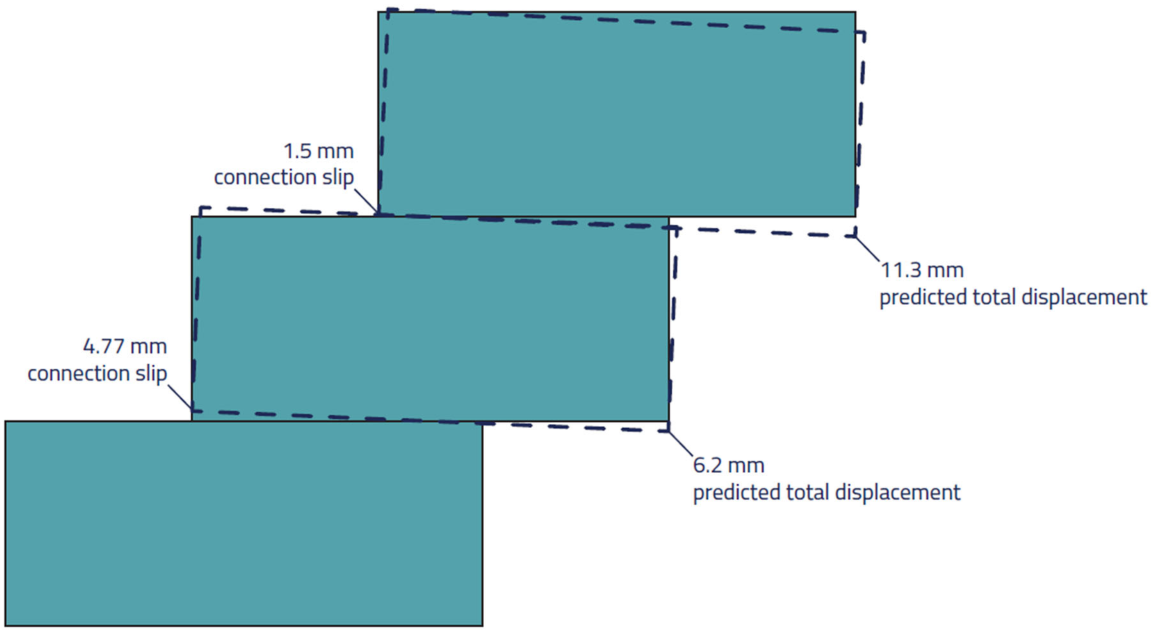


Figure 6-50 Dyson: Calculated deflection under static loading

6.4.4. Summary

This case study fulfilled objectives 2, 3c and 4:

Reduce barriers for timber specification and connection design by:

2. To create and deliver educational material of current research, for the purpose of increasing the level of knowledge of structural timber for both university students and practising engineers;
3. To reduce the complexity of EC5 through automation of timber connections:
   c. Creation of a robust code-compliance structural timber connection calculations within the identified platform;
4. To Create case studies demonstrating the advantages of parametric methodology within EC5 timber connections.

This prestigious project makes for a good case study, as a result of the challenging architectural design, which resulted with larger than normal actions to be resisted at the unit to unit connections and the ground floor rear gable shear wall. The final connection solutions were as a result of an iterative process, to do with a combination of structural design and design for manufacture and assembly + disassembly (DfMA+D) requirements, which are only possible as a result of the parametric approach of EC5, **fulfilling objective 4.** The project had a high level of external scrutiny over the structural design and calculations as a part of the building warrant and insurance process, much of which revolved around the connections of the CLT to CLT and the DfMA+D unit to unit connections. The level of scrutiny was very thorough: collectively the design team responded to over six hundred comments which resulted in zero system or structural changes to the project. This is a good statement to the reliability of the newly created code-compliance connection calculation software, **fulfilling objective 3c**.

This project case study has been used in several external CPD events, including Zero Waste Scotland; Construction Scotland Innovation Centre; IStrucrE. It was used within university teaching activities. The project is also featured within the new Trimble Technology Lab with Edinburgh Napier University, see Figure 6-51, **fulfilling objective 2.**

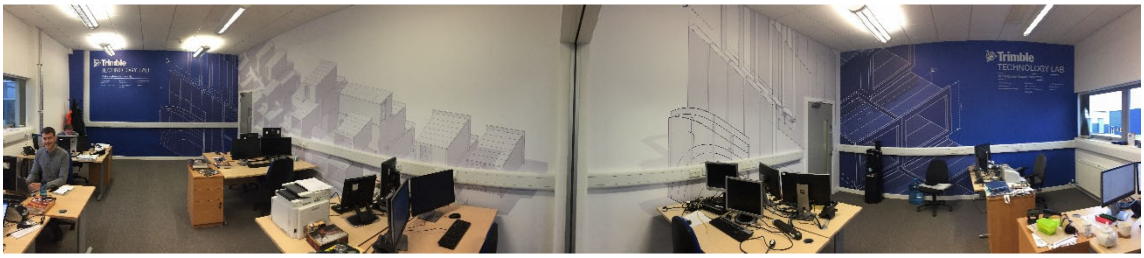


Figure 6-51 Trimble Technology Lab, art work: installed at Seven Hills campus

Chapter 7. BIM-Ready Equation

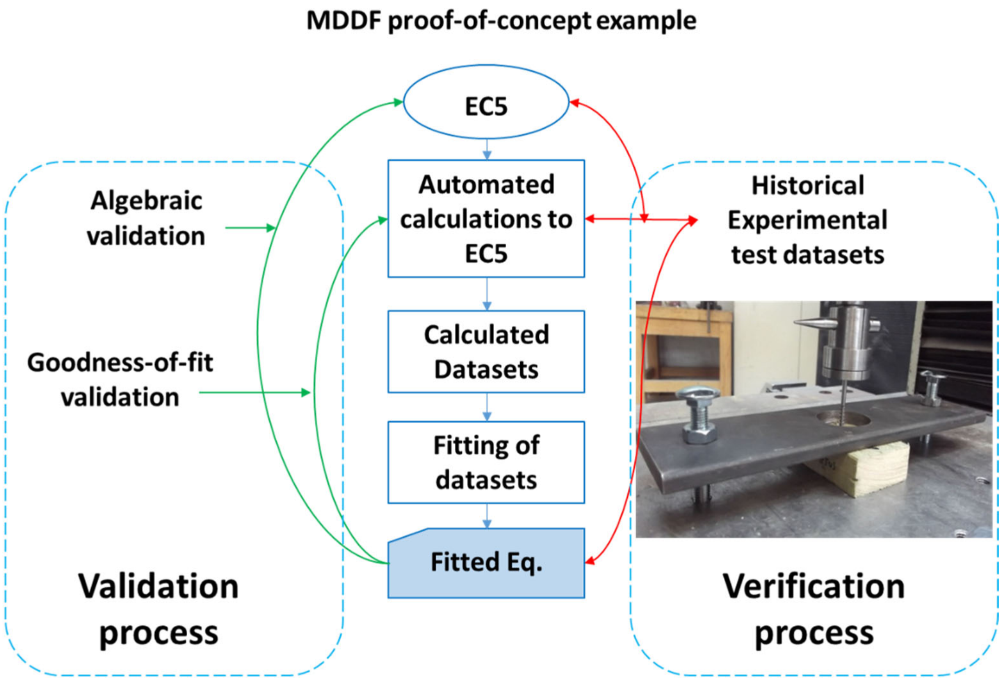
7.1. Introduction

The current level of mathematical and logical function maturity within the main BIM authoring software platforms presents a challenge for the implementation of complex iterative calculation steps to be coded into a native BIM environment.

The nature of the Johansen's equations makes it inherently difficult to program in an environment where a minimum command is not present. Coding solutions would be both lengthy and computationally demanding or require a call out to external software. Finding a mathematical solution is preferable, as a singular project will contain many connections that will all need to be evaluated simultaneously, so computational power is at a premium.

7.2. MDDF proof of concept, Axial loading of fasteners

To create equations that can be natively programmed into BIM, multidimensional data fitting (MDDF) was chosen as a tool. In order to evaluate the complexity and effectiveness of the MDDF approach, a proof-of-concept example was required. Such an example should not be unnecessarily complex, but it should demonstrate effectively the process of multidimensional fitting. Figure 7-1 shows a visual representation of the methodology adopted for the proof of concept example.



where:
Green = Validation process
Red = verification process

Figure 7-1 Proof of concept methodology

A typical timber-to-timber connection with metal dowel-type fasteners subjected to axial loading was selected (Figure 7-2). While it is not a particularly complex problem, it is very common in practice and reflective of the design process of more elaborate connections. The standard adopted for the structural design was Eurocode 5 (EC5) [213], generally considered the state-of-the-art structural timber design code internationally.

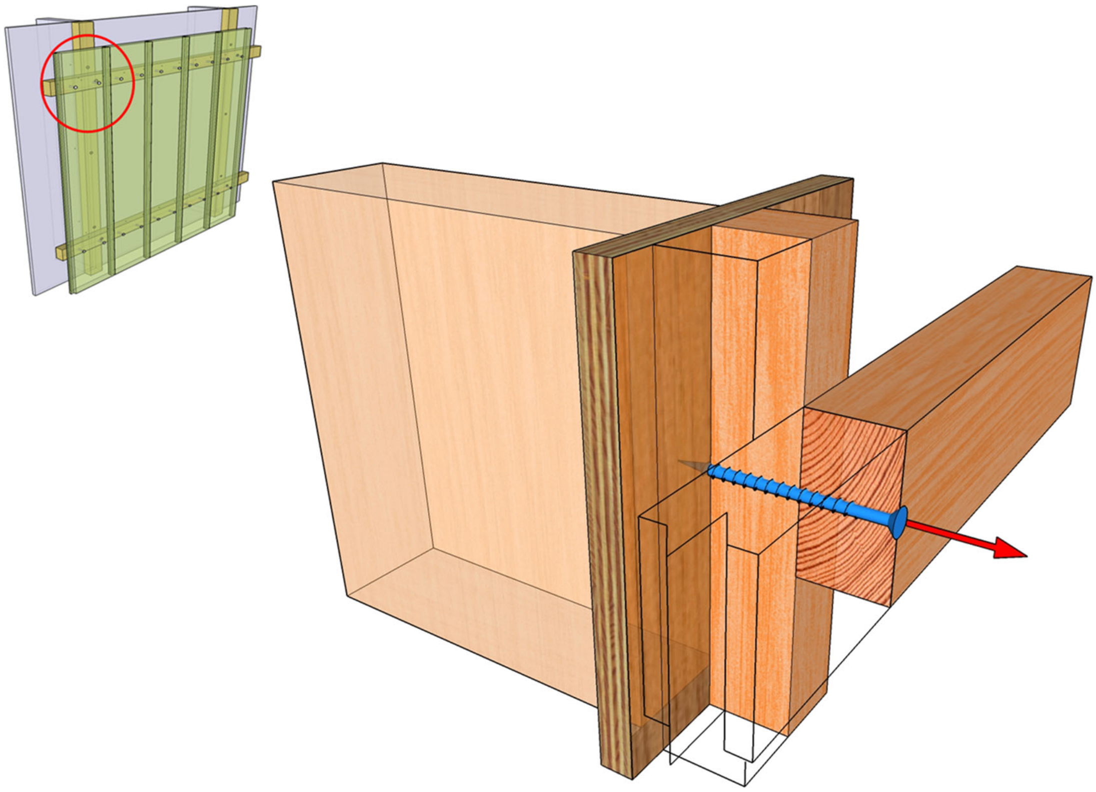


Figure 7-2 Timber-to-timber connection with fastener under axial loading

In such a design case, the key consideration is the calculation of the characteristic withdrawal capacity of the fastener, in this case a screw. Due to the particularities of this connection, the screw is in tension and therefore buckling failure is not possible. In addition, there are no steel plates hence some failure modes, such as failure along the circumference of the group of screws, tensile failure of the screw and tear-off failure of the screw head, were not taken into account. The relevant failure modes that must be taken into account are:

- Withdrawal failure of the threaded part of the screw (point side), $F_{ax,point,Rk}$
- Pull-through failure of the screw head (head side), $F_{head,Rk}$

These are calculated from Equations (8.40a) and (8.40b) of Eurocode 5 respectively [213].

The total characteristic withdrawal capacity for screws, $F_{ax.Rk}$, can, therefore, be calculated from

$$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{head.Rk}) \qquad (7\text{-}1)$$

where:

$$F_{ax.point.Rk} = \frac{1}{1.2} \cdot f_{ax.k} \cdot d_o \cdot t_{pen} \cdot \left(\frac{\rho_{k.m}}{\rho_{pss}}\right)^{0.8} \qquad (7\text{-}2)$$

$$F_{head.Rk} = f_{h.k} \cdot d_o^{\,2} \cdot k_h^{\,2} \cdot \left(\frac{\rho_{k.m}}{\rho_{hss}}\right)^{0.8} \qquad (7\text{-}3)$$

This capacity is affected by eight different variables: the thread point side penetration t_{pen}; the screw head and outer thread diameters, d_h and d_0 respectively; the pointside withdrawal strength $f_{ax.k}$; the headside pull-through strength $f_{h.k}$; the characteristic density of the timber member $\rho_{k,m}$ and the associated densities for the two strengths, ρ_{hss} and ρ_{pss} respectively.

While look-up tables can be developed for specific types of components, this is not possible for a generalised case that covers all possible combinations of materials and screws. In order to develop the required BIM-implementable ACC database, a different approach was needed.

7.2.1. Application of MDDF in the example

The application of MDDF in this connection example requires the creation of large datasets, developed within MATLAB using nested loops of the automated design code calculations. These are then used for the extraction of fitted equations, using the MATLAB-based environment described above and the outputs can then be introduced in a BIM object. The process is summarised in Figure 7-3.

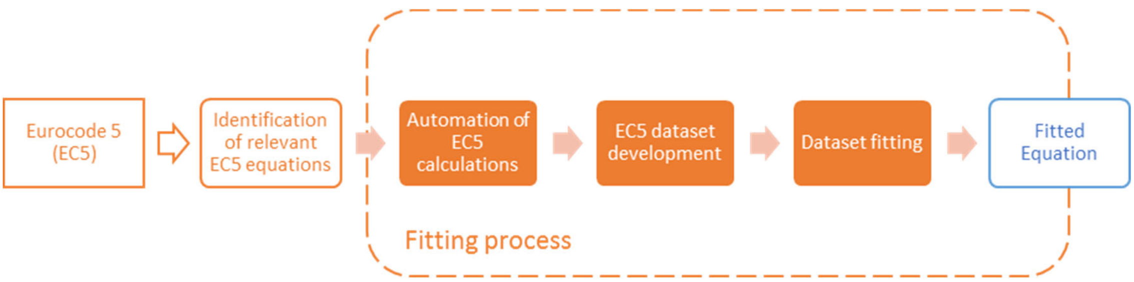


Figure 7-3 Fitting Process Overview

The dataset developed for the axial loading example was based on eight variables with ten iterations each, thus resulting in 10^8 data points, which provided a large enough dataset for the fitting process. The variables and their boundary conditions are given in Table 7-1.

Table 7-1 Boundary conditions of the axial withdrawal loading dataset

		Minimum	Maximum
Thread point side penetration	t_{pen}	$21\ mm$	$70\ mm$
Screw outer thread diameter	d_o	$3.5\ mm$	$6\ mm$
Head factor (ratio) $k_h = d_h / d_0$	k_h	1	4
Pointside withdrawal strength	$f_{ax.k}$	$4.5\ N/mm^2$	$15\ N/mm^2$
Head side pull-through	$f_{h.k}$	$4.5\ N/mm^2$	$15\ N/mm^2$
Member timber density	ρ_{k_m}	$290\ kg/m^3$	$460\ kg/m^3$
Associated density for f_{h_k}	ρ_{hss}	$290\ kg/m^3$	$460\ kg/m^3$
Associated density for f_{ax_k}	ρ_{pss}	$290\ kg/m^3$	$460\ kg/m^3$

As Equation (7-1) demonstrates, the dataset is made up of two separate intersecting surfaces, and therefore by using the intersection of the two surfaces the dataset can be quantified using an equation. For this purpose, a sigmoid function is used as a form of a step function. The basic form of a sigmoid function can be seen in Equation (7-4) and Figure 7-4.

$$y = \frac{1}{1 + e^{-\left(\frac{x+\varepsilon}{\omega}\right)}} \tag{7-4}$$

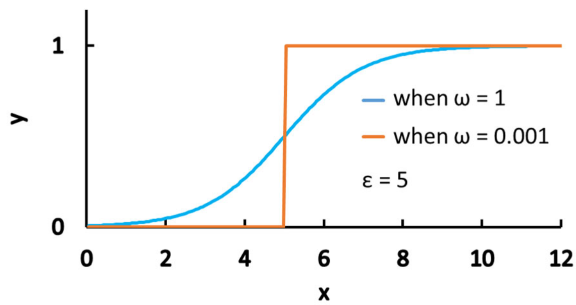


Figure 7-4 Sigmoid function example

The generic surface intersection fitting follows the process described in Figure 7-5.

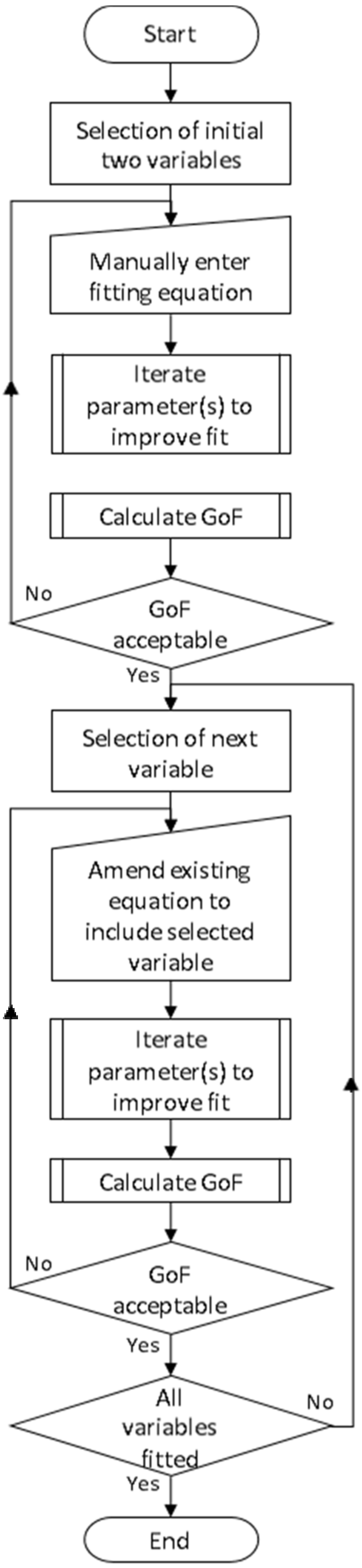


Figure 7-5 Surface fitting process

The application of the methodology in this axial loading example is described below.

Step 1: The process requires the selection of two suitable variables where a clear intersection of the two surfaces can be observed while keeping the rest fixed. Here, the penetration length t_{pen} and the pointside withdrawal strength $f_{ax,k}$ have been selected. The user is required to enter an initial equation that describes the relationship of the surface intersection into the fitting software, with the appropriate fitting parameters and starting points (Figure 7-6.a). The software runs iterations until the best fit of the parameter is found, coinciding with the intersection of the surface planes. If the fitted equation does not have the required level of fit or Goodness of Fit (GoF), either a more appropriate initial value for the fitting parameter is required, or the initial equation needs to be amended. For this example, the resulting fitted equation is:

$$f(n_2) = 624.844 \cdot {t_{pen}}^{-1} - f_{ax.k} \tag{7-5}$$

Step 2: The next variable selected is the headside pull-through strength $f_{h,k}$ .This dataset, including both t_{pen} and $f_{ax,k}$ results in surfaces with a different relationship to Step 1 (Figure 7-6.b). The observation of the shape of the dataset can be useful to provide a starting point for the variable that will be input into the fitting equation. The software then attempts to fit the equation, as per Step 1. For this example, the resulting fitted equation is:

$$f(n_3) = 41.656 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k} \tag{7-6}$$

Steps 3 to 5 repeat the same process as Step 2, progressively including all variables, until finally, the resulting fitted equation includes the complete dataset. The shapes of the datasets on each step can be seen in Figure 7-6.c to Figure 7-6.e.

From step 3, add in the fixing head multiplier k_h

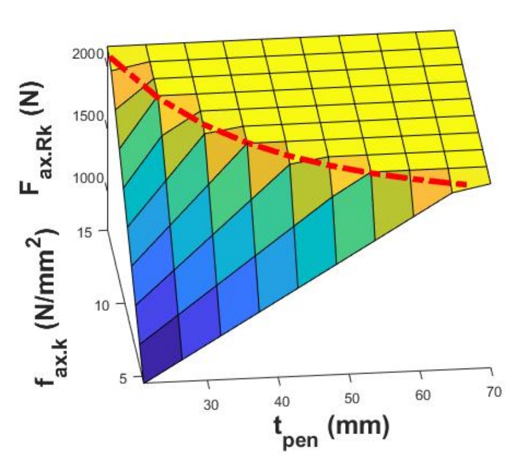
$$f(n_4) = 10.414 \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k} \tag{7-7}$$

From step 4, add in the crew outer thread diameter d_o

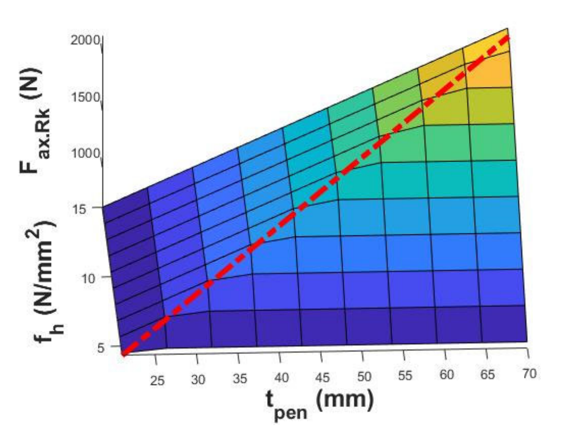
$$f(n_5) = 1.736 \cdot d_o \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k} \tag{7-8}$$

From step 5, add in the two remaining variables ρ_{pss} and ρ_{hss} associated densities headside and pointside.

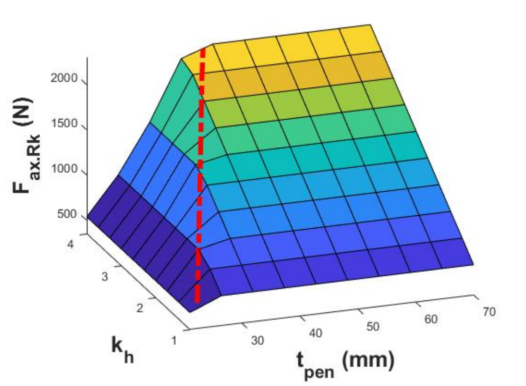
$$f(n_7) = 1.196 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot d_o \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k} \tag{7-9}$$



a. $f(s_1) = p \cdot {t_{pen}}^{-1} - f_{ax.k}$

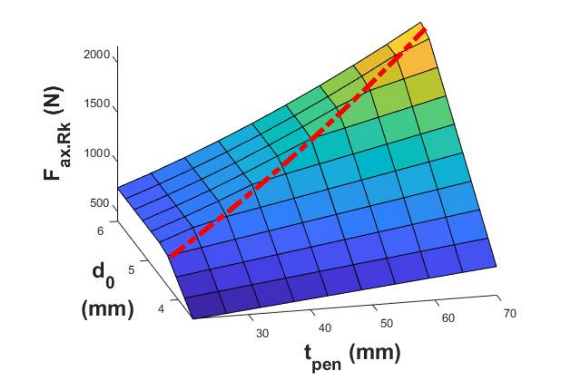


b. $f(s_2) = p \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k}$



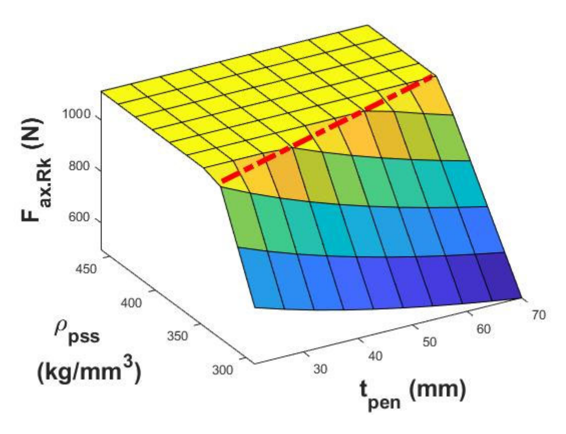
c.

$$f(s_3) = p \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k}$$



d.

$$f(s_4) = p \cdot d_o \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k}$$



e. $f(s_5) = p \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot d_o \cdot {k_h}^2 \cdot f_{h.k} \cdot {t_{pen}}^{-1} - f_{ax.k}$

Figure 7-6 Surface fitting process steps for the axial loading example

Equation (7-10) describes the final fitted surface intersection, where the algebraic validation is the point of intersection when $F_{ax.point.Rk} = F_{head.Rk}$ and the resulting multiplication factors have been rounded up towards safety:

$$f(n) \;=\; 1.196 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot \; d_o \; \cdot k_h{}^2 \cdot f_{h.k} \; \cdot t_{pen}{}^{-1} - f_{ax.k} \quad (7\text{-}10)$$

7.2.2. Validation and verification

Before the fitting equation can be used, it should be both validated and verified. The validation process happens in two steps: inspecting the goodness of fit of the fitted equation and algebraically checking the fitted equation.

7.2.3. Goodness of fit validation

The goodness-of-fit (GoF) of a dataset in relation to a fitted equation describes how well the fit describes the data. It is a summary of the discrepancies between the two. The goodness-of-fit is calculated as follows:

$$GoF_{100} = \frac{max|data_i - fit_i|}{\frac{\Sigma \, data_i}{N}} \cdot 100 \quad (7\text{-}11)$$

$$GoF_{95} = \frac{2 \cdot STDEV(data_i - fit_i)}{\frac{\Sigma \, data_i}{N}} \cdot 100 \quad (7\text{-}12)$$

where:

GoF_{100} is the goodness of fit expressed as a percentage for all of the data;

GoF_{95} is the goodness of fit expressed as a percentage of 95% of the data, useful when fitting experimental datasets;

STDEV is the standard deviation.

The goodness-of-fit values describe how close the data points lie to the fitted model, as a percentage of the data values. Therefore the lower the percentage value, the more accurate the fit is. This particular example has GoF values of: $GoF_{100} = 0.00000013\%$ and $GoF_{95} = 0.00000000012\%$ with a maximum residual of 0.000138 kN. Therefore, for this example, we can state that with a high degree of certainty that the model is a good fit. See Appendix F for a screenshot showing the GoF results.

In addition to a numerical GoF, which can be narrowly focused on particular aspects of the data and then compress that information into a singular number, it is also good practice to visually inspect the graphical representation of the data and the fit, see Figure 3-3.

7.2.4. Algebraic validation

A visual inspection of the original EC5 equation shows it is made up of two surfaces that intersect. We know that this intersection lies at when $F_{ax.point.Rk} = F_{head.Rk}$. Thus, we can rearrange the equations to determine where this point will occur in all dimensions. This can be used to check that the fitting software has converged on the correct solution.

$$F_{ax.point.Rk} = F_{head.Rk} \tag{7-13}$$

$$\frac{1}{1.2} \cdot f_{ax.k} \cdot d_o \cdot t_{pen} \cdot \left(\frac{\rho_{k.m}}{\rho_{pss}}\right)^{0.8} = f_{h.k} \cdot d_o{}^2 \cdot k_h{}^2 \cdot \left(\frac{\rho_{k.m}}{\rho_{hss}}\right)^{0.8} \tag{7-14}$$

$$f_{ax.k} = \frac{f_{h.k} \cdot d_o{}^2 \cdot k_h{}^2 \cdot \left(\frac{\rho_{k.m}}{\rho_{hss}}\right)^{0.8}}{d_o \cdot t_{pen} \cdot \left(\frac{\rho_{k.m}}{\rho_{pss}}\right)^{0.8} \cdot \frac{1}{1.2}} \tag{7-15}$$

$$f_{ax.k} = \frac{f_{h.k} \cdot d_o \cdot k_h{}^2 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot 1.2}{t_{pen}} \tag{7-16}$$

$$0 = 1.2 \cdot f_{h.k} \cdot d_o \cdot k_h{}^2 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot t_{pen}{}^{-1} - f_{ax.k} \tag{7-17}$$

When comparing this to result in Equation (7-17) to the fitted Equation (7-10), there is an acceptable margin of error, which is as a result of the resolution of the data set. Thus, in order to increase the reliability of the fitted equations larger data sets can help. This will come at a computational power cost, however, that is out with the boundary conditions of this research.

7.2.5. Combining the fitted intersection equation

Combining the fitted intersection equation with the sigmoid form Equation (7-4), a final BIM-ready equation can be derived, which accurately describes the complete 8D dataset Equation (7-18)

$$F_{ax.Rk} = \frac{f_{ax.k} \cdot d_o \cdot \frac{t_{pen}}{1.2} \cdot \left(\frac{\rho_{k.m}}{\rho_{pss}}\right)^{0.8}}{1 + e^{\left(\frac{1.2 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot d_o \cdot k_h{}^2 \cdot f_{h.k} \cdot t_{pen}{}^{-1} - f_{ax.k}}{-0.001}\right)}} + \frac{f_{head.k} \cdot (d_o \cdot k_h)^2 \cdot \left(\frac{\rho_{k.m}}{\rho_{hss}}\right)^{0.8}}{1 + e^{\left(\frac{1.2 \cdot \left(\frac{\rho_{pss}}{\rho_{hss}}\right)^{0.8} \cdot d_o \cdot k_h{}^2 \cdot f_{h.k} \cdot t_{pen}{}^{-1} - f_{ax.k}}{0.001}\right)}} \quad (7\text{-}18)$$

7.2.6. Experimental verification

In order for this fitting process to be verified, it should be compared against experimentally obtained data. A series of tests on withdrawal perpendicular to the grain were carried out at Edinburgh Napier University in 2014, in accordance with BS EN 1382:1999 [230] (Figure 7-7). Five types of screws (

Table 7-2) were tested, with forty tests carried out on each type. In order to test if pilot holes affected the results, ten additional tests were carried out with no pilot hole. This gave a total sample size of 210. The moisture content of the battens at the time of testing was 16.0%.

Figure 7-7: Screw pull-out test set-up

Table 7-2: Overview of screws tested

Screw description	Nominal outer diameter mm	Nominal length mm

Heco-Fix-plus	4.5	80
Rothoblass, SCI A2	5	80
Spax, Stainless steel 7755F	5	80
Simpson Strong-Tie, item: S08300DB1E	4.2	76
Generic, Stainless steel	5	75

The F_{max} results were normalised with respect to the outer diameter of thread and the point side penetration.

The fitting equitation requires the characteristic pointside withdrawal strength f_{ax_k} and the associated characteristic timber density ρ_{pss}. These were calculated from the data as:

$$f_{ax\_k} = 19.5\ N/mm^2$$

$$\rho_{pss} = 455.8\ kg/m^3$$

Figure 7-8 shows the laboratory test results when normalised against screw dimensions, showing the withdrawal capacity against timber density. The test data has been normalised for an outer thread diameter of 4.5 mm and an effective thread penetration of 25 mm, excluding the point of the screw. The thread penetration is determined by the thickness of the timber in the test setup. The blue points represent normalised data points, the black dashed line is the EC5 equations and the red line indicates the fitted equation, when $f_{ax.k}$ was calculated from $f_{ax.k} = \frac{F_{max.k}}{d_s \cdot t_{pen}}$

where

$F_{max.k}$ is the characteristic values of the test data after additional normalisation against timber density;

d_s is diameter of the smooth plain part of the screw.

The goodness-of-fit can be described by how far the data points lie from the fitted curve. In this case, 95% of the points lay within 29.9% of the fitted equation. As can be seen from the figure, the variation in the data points were large and this 29.9% is within the statistical noise of the data. In conclusion, the output of the fitted equation shows good agreement with the test data, thus verifying the fitting process.

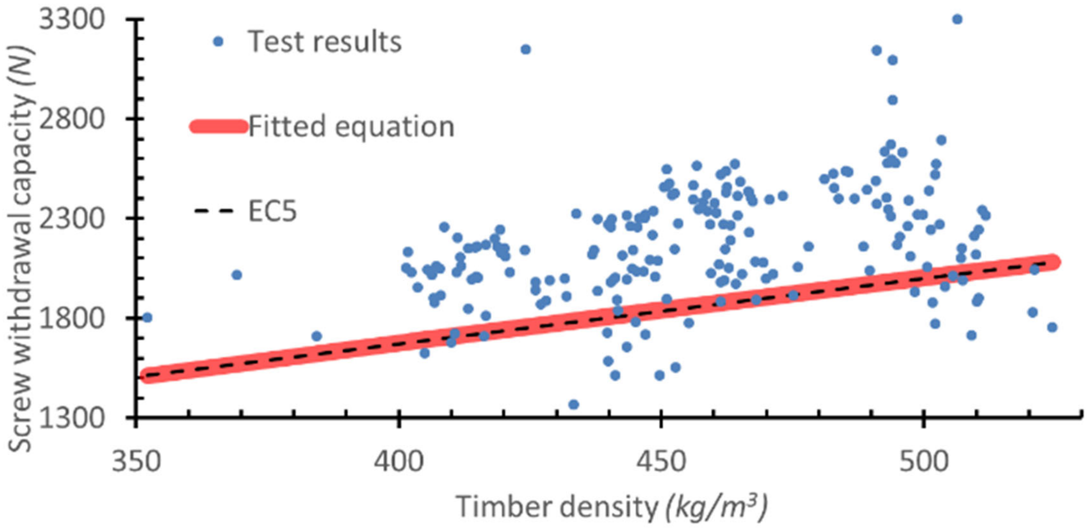


Figure 7-8: Screw withdrawal capacity: Test data comparison

7.2.7. Computational comparison from the MDDF proof of concept

For this axially loaded example the resulting computational loading for the fitted equation is 32.0% of the original. Although this is a relatively simple example of fitting it demonstrates that the newly created BIM-ready equations run computationally approximately 3 times faster than the original equations. In addition, this can be run natively within the Revit or SketchUp pro platform, where the original equations present challenges that would need to be overcome.

Please note that the computational speed comparison tests were conducted within a MATLAB environment where the computational comparison can be controlled. The original equations will be challenging to implement into a native BIM environment and if at all possible, they will likely run significantly slower than in the controlled speed test. The BIM-ready equations, on the other hand, are fairly straightforward to implement.

7.2.8. Summary on the MDDF proof of concept

The final fitted equation was validated by a goodness-of-fit and also algebraically and verified experimentally according to standard structural engineering practice, and this content has been published by Livingstone [231].

7.3. Lateral loading of fasteners

Now that the approach of converting multidimensional equations into a single equation that can be implemented into a BIM environment has been realised, we can shift focus to a more complex timber connection. To calculate the most common timber connections, the lateral loading capacity must be evaluated in combination with the axial loading capacity equation explored above. This section deals with the lateral load-carrying capacity of metal dowel-type fasteners for timber-to-timber and panel-to-timber connections in single shear. An example can be seen in Figure 7-9. The EC5 calculation format uses the Johansen's equations, as described within the literature review. This uses the parametric approach of identifying the predicted failure mode and then the resulting fixing lateral loading resistance, see Appendix D for more details. The multiple failure modes of this approach result in a complex system of relationships between the resulting failure modes. This needs to be understood before data fitting can begin.

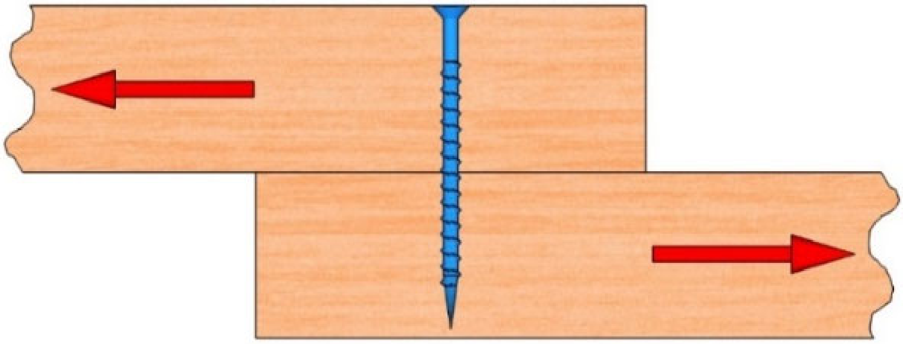


Figure 7-9 Lateral load connection example

7.3.1. Creation of data set

In a similar approach to the previous axial loading proof of concept example, the dataset required for the lateral loading connections needs eight variables. For the data set created it was decided that each variable would have ten iterations, which equates to 10^8 data points, each point requiring eight bytes of storage, and the dataset is therefore 0.8 gigabytes in size. For explanation, if we were to use twenty iterations for each variable the resulting data set will require 204 GB of storage. Within this example, no form of batch processing was used, but merely operated within the confines of my computer hardware limitations. For future work dataset creation and manipulation need not be limited as various mathematical methods and overlapping batch processing can be applied. The dataset was calculated using the Johansen's equations (Equation (7-19)) found in EC5. The selected boundary conditions for the eight variables used are listed within Table 7-3. Please note that the boundary conditions have been chosen to best represent the majority of lateral loading fixings used within structural timber construction.

Table 7-3 Boundary conditions of the lateral loading dataset variables

		Minimum	*Maximum*
Characteristic Withdrawal capacity	$F_{ax,Rk}$	0.1 kN	6 kN
Fastener profile	$r_{f.mod}$	1.15	2
Characteristic timber embedment, headside member	$f_{h,k,1}$	15 N/mm^2	30 N/mm^2
Characteristic timber embedment, pointside member	$f_{h,k,2}$	15 N/mm^2	30 N/mm^2

Thickness headside member	t_1	20 mm	50 mm
Shaft penetration length in pointside member	t_2	20 mm	100 mm
Effective screw diameter	d	2 mm	8 mm
Tensile strength of fastener	$f_{u,f}$	400 N/mm^2	1000 N/mm^2

Verification of this dataset was undertaken in the same manner as the proof of concept dataset. Randomised datapoints were compared with the relevant calculations on the Tedds platform (0.0, where code compliance calculations have been through a process of verification internally by Tekla and will externally be the user network. More details can be seen in 0.1.

See 0.1 for the example code for the data set creation for lateral loaded connections. The created dataset contains the returned force value along with the identification for the failure mode for each data point.

$$F_{v.Rk} = \min \begin{cases} F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d & \text{(a)} \\ F_{v,Rk} = f_{h,2,k} \cdot t_2 \cdot d & \text{(b)} \\ F_{v,Rk} = \dfrac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right] + \dfrac{F_{ax,Rk}}{4} & \text{(c)} \\ F_{v,Rk} = 1.05\dfrac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & \text{(d)} \\ F_{v,Rk} = 1.05\dfrac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_2^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} & \text{(e)} \\ F_{v,Rk} = 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} & \text{(f)} \end{cases} \tag{7-19}$$

7.3.2. Inspecting and fitting of the data set

Using the graphical user interface (GUI) described within Section 3.5 (see Figure 3-3) for visual inspection of the failure modes within the dataset, it can be seen that multiple failure modes intersect (Figure 7-10 and Figure 7-11). In each figure, two variables are varied on the X and Y axes and all the others are fixed, with the values stated above the figures. Both figures are visualising the same data set, just from the viewpoint of different variables. The colour of the surface indicates the failure mode.

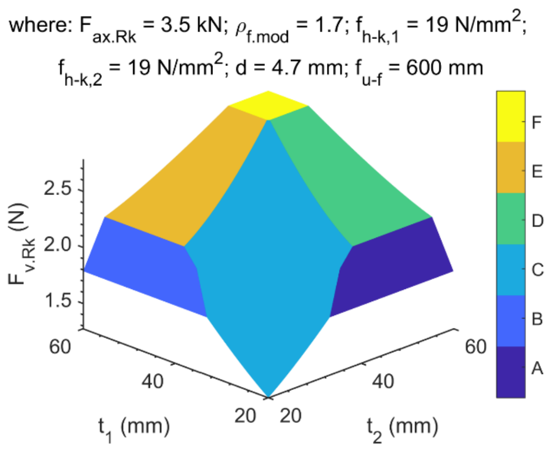


Figure 7-10 Lateral load data set: t_1 vs t_2

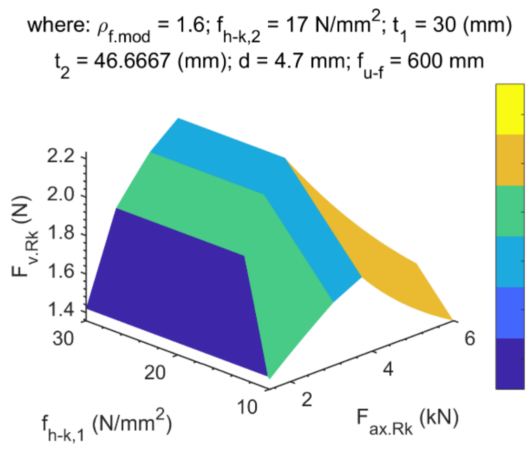


Figure 7-11 Lateral load data set: $f_{h-k,1}$ vs $F_{ax.Rk}$

Note: these figures were created using a greater resolution of dataset for the axes variables.

In Figure 7-10 the dataset is visualised in the form of headside member thickness vs the pointside penetration, while Figure 7-11 shows characteristic embedment strength in the headside member vs the characteristic withdrawal capacity. It was observed that all the failure modes intersect each other at some line and point in the multidimensional dataset, except for failure modes “B” and “D”.

In order to get the best possible fit for the data it was decided to fit just the intersection of the surfaces, as used in the proof of concept example. Therefore, multiple sigmoid functions are used to represent the multiple failure mode intersections. The resulting equation can be seen in Equation (7-20) below. This method of multidimensional data fitting generally results in a good level of fit, or this example of the goodness-of-fit (GoF) for the lateral loaded fitted equation has GoF values of: $GoF_{100} = 0.00012\,\%$ and $GoF_{95} = 0.0000015\,\%$ with a maximum residual of 0.000002178 kN. This demonstrates a high degree of certainty that the model is a good fit, providing certainty of accuracy. See 0.2) showing a screenshot for the resulting GoF values and the code used for calculating this.

At first glance, the new BIM-ready Equation (7-20) is larger than the original that it is intended to replace Equation (7-19), but the advantage of the newly created equation is that it can be easily installed into a BIM environment, where the original presents a difficult challenge for implementation. Within most BIM environments there is a lack of logical function or mathematical operators available, as discussed in Section 7.1. The other advantage is that the computational load of the equation is lower.

For this example the resulting computational loading for the fitted equation is 43.5% of that of the original, running over twice the speed for the EC5 lateral loading equation for single shear. See 0.3 and G.4 showing a screenshot for the lateral load optimisation percentage comparison and the code used for calculating this.

Please note that, as in the proof of concept, the computational speed comparison tests were conducted within a MATLAB, as the original equations are challenging to implement into a BIM environment.

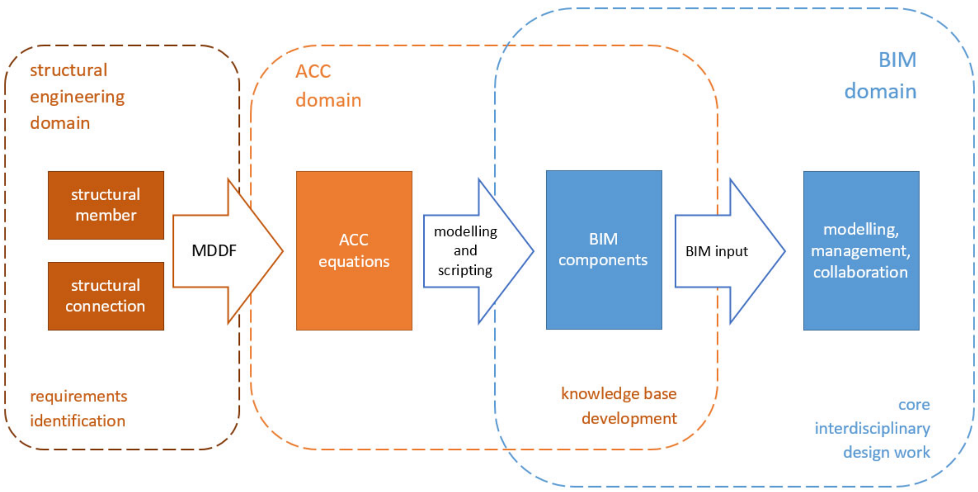
$$
\begin{aligned}
F_{v.Rk} = & \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_A-\mathrm{FM}_B}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_A-\mathrm{FM}_C}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_A-\mathrm{FM}_D}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_A-\mathrm{FM}_E}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_A-\mathrm{FM}_F}{0.001}\right)}}\right) \cdot \mathrm{FM}_A\right] \\
& + \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_B-\mathrm{FM}_A}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_B-\mathrm{FM}_C}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_B-\mathrm{FM}_D}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_B-\mathrm{FM}_E}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_B-\mathrm{FM}_F}{0.001}\right)}}\right) \cdot \mathrm{FM}_B\right] \\
& + \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_C-\mathrm{FM}_A}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_C-\mathrm{FM}_B}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_C-\mathrm{FM}_D}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_C-\mathrm{FM}_E}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_C-\mathrm{FM}_F}{0.001}\right)}}\right) \cdot FM_C\right] \\
& + \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_D-\mathrm{FM}_A}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_D-\mathrm{FM}_B}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_D-\mathrm{FM}_C}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_D-\mathrm{FM}_E}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_D-\mathrm{FM}_F}{0.001}\right)}}\right) \cdot FM_D\right] \\
& + \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_E-\mathrm{FM}_A}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_E-\mathrm{FM}_B}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_E-\mathrm{FM}_C}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_E-\mathrm{FM}_D}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_E-\mathrm{FM}_F}{0.001}\right)}}\right) \cdot FM_E\right] \\
& + \left[\left(\frac{1}{1+e^{\left(\frac{\mathrm{FM}_F-\mathrm{FM}_A}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_F-\mathrm{FM}_B}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_F-\mathrm{FM}_C}{0.001}\right)}} \right.\right. \\
& \left.\left. \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_F-\mathrm{FM}_D}{0.001}\right)}} \cdot \frac{1}{1+e^{\left(\frac{\mathrm{FM}_F-\mathrm{FM}_E}{0.001}\right)}}\right) \cdot FM_F\right]
\end{aligned}
\tag{7-20}
$$

7.4. Implementation into BIM

7.4.1. Interoperability with BIM

The analysis of existing research frameworks demonstrates that the effective automation of structural analysis and design computations in BIM cannot be achieved via a multi-platform (MP)-BIM approach, at least with the current state-of-the-art in hardware and software. It introduces an extensive set of technical challenges and the benefit to cost ratio is not large enough. As described in the literature review, the current practice of structural analysis and design software is to treat structural calculations within BIM as a data I/O issue as opposed to core functionality. Thus it appears reasonable to adopt a single platform (SP)-BIM approach, addressing the structural computational aspects indirectly, as separate knowledge domains.

The framework developed for the purposes of this work assigns an SP-BIM system as the core interdisciplinary modelling and management domain. Individual components are analysed and designed from a structural engineering perspective to achieve Automatic Code Compliance (ACC). This requires them to satisfy the structural design requirements according to the respective national code or standard. The results of the ACC analysis are programmed into BIM components. These BIM components are input in the core BIM platform and are available to designers. A schematic representation of the process for the development of the framework is given in Figure 7-12.



where

MDDF = Multi-Dimensional Data Fitting

ACC equations = fitted equations that describe a defined data set

BIM components = digital three-dimensional components that contain smart information

Figure 7-12 BIM integration, Schematic representation of the framework development process

When design decisions are made that affect the structural performance and thus the code compliance of BIM components, the components respond in real-time to the design decisions. Typically, this is done in one of the following three ways:

Way 1, by adjusting themselves automatically, so as to achieve code compliance. For example, a beam might change cross-section, or the joist spacings of a floor might change. This enables ACC, while maintaining the design intention (e.g. dimensions of a floor).

Way 2, by providing limiting values the designer is protected from going beyond the code. For example, if a beam cannot support the type of loading beyond a certain span, the respective BIM component will be limited to be designed up to a certain span.

Way 3, providing immediate straightforward feedback on the structural performance of a component allows the designer to identify if the component is fit for purpose. For example, a connection can identify that, with the given materials and geometry, it can withstand typical loads for residential buildings, but not for commercial.

As a result of this process and assuming only ACC BIM components have been used, the entire design is code-compliant, without having to resort to costly and inaccurate I/O from structural engineering software. The SP-BIM approach allows for focussing the BIM aspects of the work where the technology performs best, namely 3D modelling, information management and interdisciplinary collaboration without engaging with the complexities of MP-BIM, which is arguably not mature enough for effective use in contemporary professional practice, at least in its full envisaged Level 3 breadth.

Naturally, the success of the framework described in the previous subsection rests on the development of a suitable ACC knowledge base, the outputs of which are utilised to program the respective BIM components. It is important, therefore, to identify the types of ACC problems that can arise, so appropriate examples can be developed in order to demonstrate the feasibility of the approach.

7.4.2. Practical application of fitted equations into smart BIM components

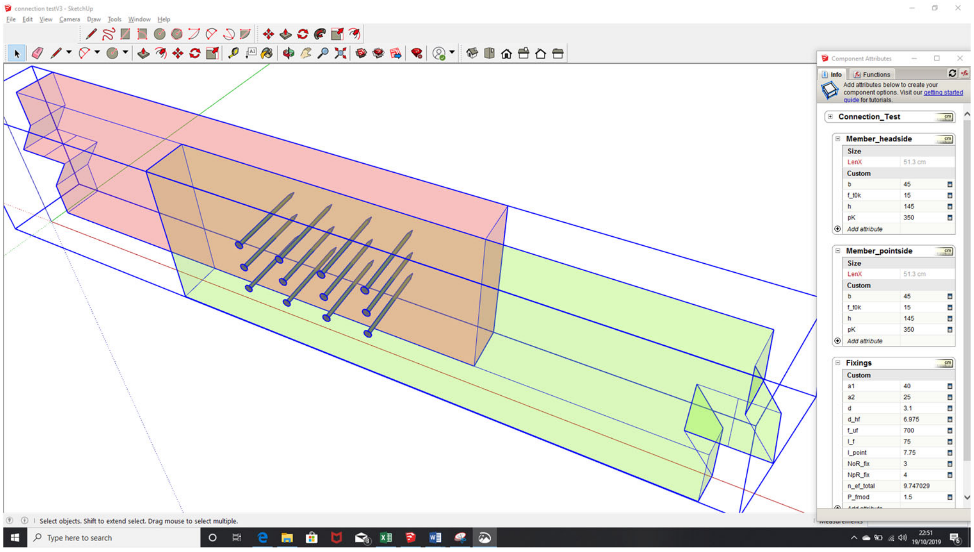


Figure 7-13 Smart BIM component example: Tension splice connection

In order to demonstrate the functionality of the BIM-ready equations for both the axial and lateral loading equations, an example of a tension splice connection is used, as seen in Figure 7-13.

There were a number of problems that were overcome while coding the BIM-ready equation into the BIM environment. The mathematical functionality of Revit or Trimble SketchUp has limits. The character β that represents the embedment ratio is not available so it was replaced with ‘SS’, likewise ϒ was replaced with ‘Lam’. The format of the sigmoid function also needed amending, and the original sigmoid equation results in an extremely large number on the denominator as a result of the exponential form of the equation. This throws an error as the number is too large for the software platform to handle. The solution for this is to change the form of the sigmoid to an equivalent using tanh as opposed to the exponential function. An example of a simple one step sigmoid can be seen in Equation (7-21).

$$\left(\frac{1}{2}+\frac{1}{2}\tanh\left(sw(fm_b-fm_a)\right)\right) \tag{7-21}$$

where:

sw is the inverse sigmoid width, the larger the value the steeper (narrower) the sigmoid;

fm_i is the calculated value of the failure mode, (i), as found in Equation (7-19).

Within this example, a smart BIM component is comprised of three sub-components: headside member, pointside member and a group of fixings. Each contains a number of variables relating to that sub-component. These details can be seen in Figure 7-14.

The main component itself contains a number of variables that are calculated from the sub-components variables, and an example of this can be found within Figure 7-15. Please see Appendix H for the code used within the smart BIM component's implementation for the BIM-ready equation.

The resulting smart BIM components that have the BIM-ready equations are not a resulting BLACK-BOX solution but can indeed identify the failure mode with the results, as seen in Figure 7-15 and Figure 7-16. By identifying the failure mode in this way, it allows for ease of checking.

The real demonstration of the BIM enabled functionality can be seen when the user amends any of the parametric variables. For demonstration purposes, in Figure 7-16 the characteristic density for the headside member has been changed from 350 kg/m^3 to 450 kg/m^3. The resulting actions are automatically calculated reporting the findings, and this is visually demonstrated within Figure 7-15 and Figure 7-16. The results have been verified using the newly created Code Compliance automated calculations within the Trimble Tekla TEDDS platform.

7.5. Summary

In this chapter new equations were created that accurately calculate the connection strength for timber connections. These equations are designed to be natively implementable into existing BIM environments. Currently, the implementation of the existing EC5 equations presents barriers for programming. Even if these barriers were overcome, the existing EC5 equations are far more computationally demanding that the new equations presented here.

The final BIM-ready equations perform in a manner that is best described as a grey box. A black-box approach will not allow a user to follow the design steps, and this approach allows the user to see the results at each step, but will not display the equations used. This is visible within Figure 7-14, Figure 7-15 and Figure 7-16

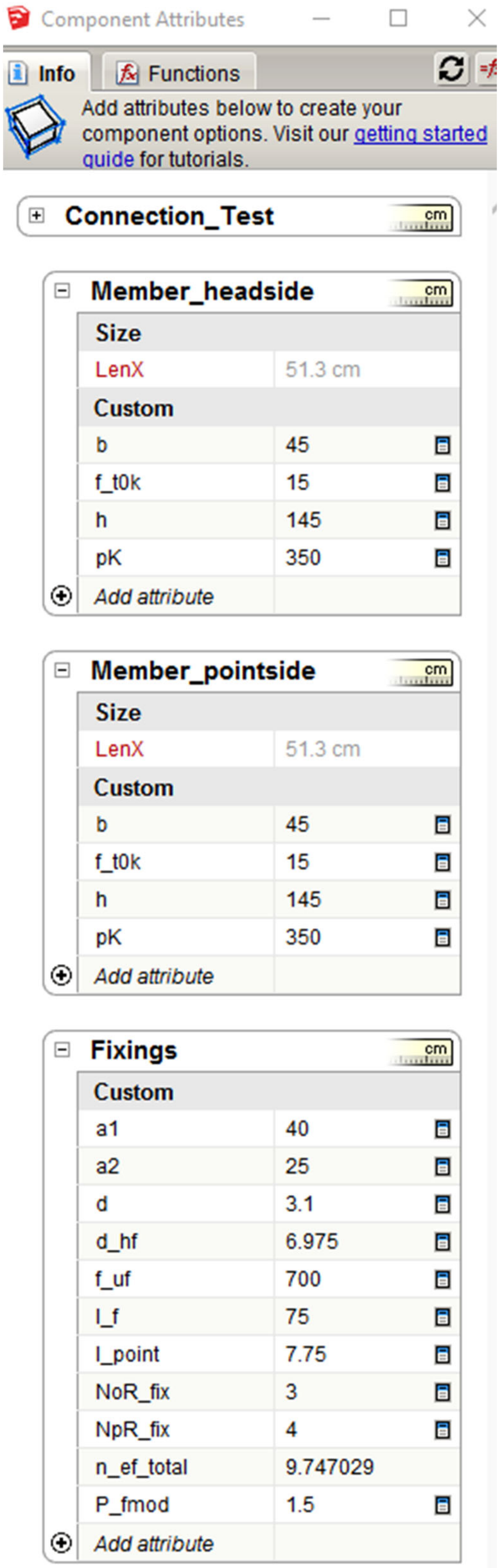


Figure 7-14 Smart BIM Component, sub components details

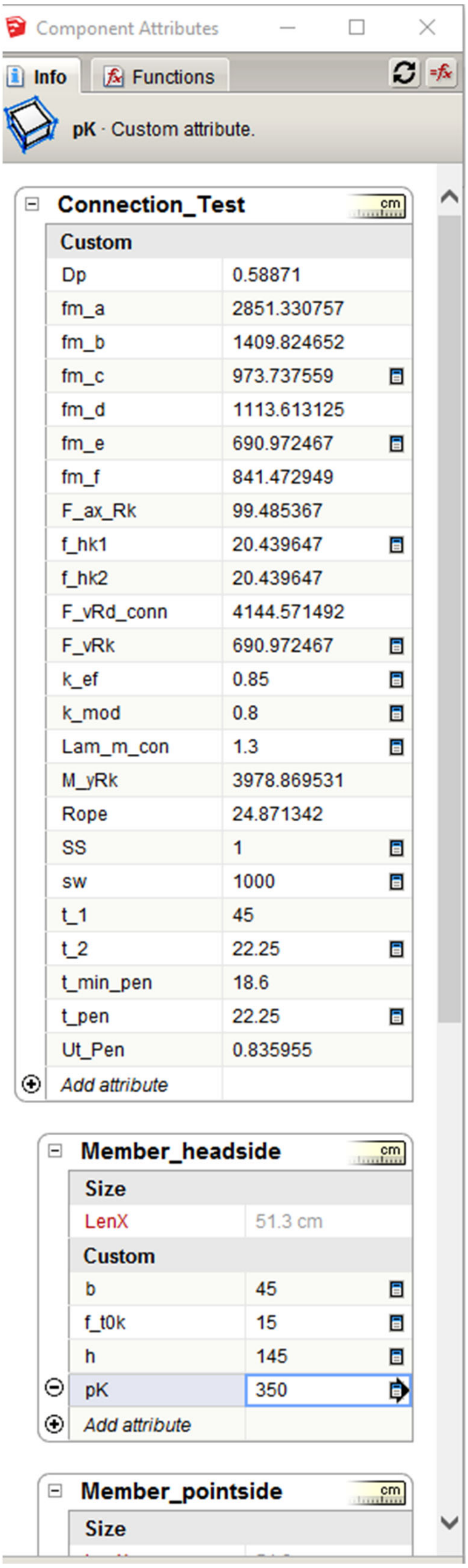


Figure 7-15 Smart BIM component variables

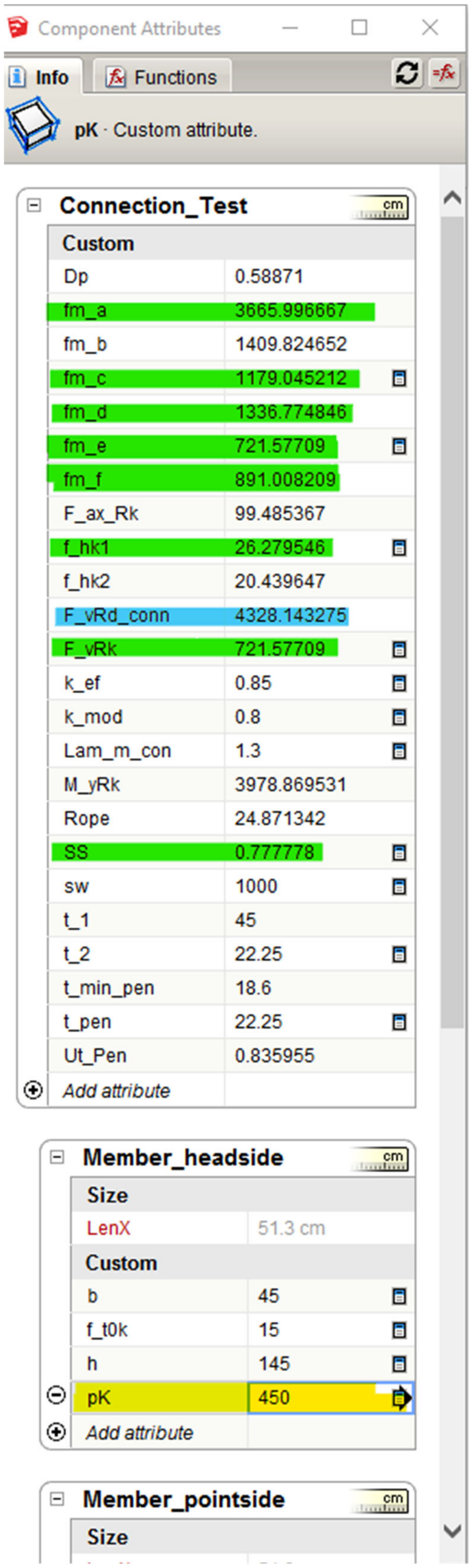


Figure 7-16 Smart BIM component amended headside density

Chapter 8. Conclusions

In Chapter 1 the aims and objectives of this thesis were set out. The subsequent chapters explained how these aims and objectives were met through a range of different techniques and research activities. Here the objectives are restated, together with a brief summary of how they were successfully fulfilled.

The first objective was "To do an industry survey of structural engineers that gives further clarity in identifying barriers for timber specification". This was accomplished with an AEC sector review survey discussed in Chapter 4 with 76 respondents within the UK. The purpose of this survey was to corroborate and expand upon the findings from the thinking outside the box report by Harker [41], which identifies the barriers for structural timber specification within the AEC sector. The results of this survey helped to define the parameters of this research.

The second objective was "To create and deliver educational material of current research, for the purpose of increasing the level of knowledge of structural timber for both university students and practising engineers". This has been achieved in a number of ways. The first is by delivering the newly created Code Compliance timber connection calculation software on the Tekla TEDDS platform. There are approximately sixteen thousand users on the network within the UK, see Section 5.4 for the usage data and 0 for examples of the output of the calculations. Sections of the work presented in this thesis have been used in conference and journal publications. Finally, this objective has been fulfilled in a range of ways by the case studies described in Chapter 6.

The third objective was "To reduce the complexity of EC5 through automation of timber connections". This has been accomplished by the publication of the newly created code compliance calculation software discussed in Chapter 5. In addition, this software was used in the creation of data sets for the new British Standard "BS 8605-2 External timber cladding – Part 2: Code of practice for design and installation" [217], discussed in Section 6.1. Two of the case studies detailed in Chapter 6 were additionally valuable for the development and proofing of the code compliance calculation software. Finally, the use of MDDF to reduce the computational complexity of connection calculations will significantly increase the ease of specifying timber connections so it can be natively implemented in a BIM environment, as discussed in Chapter 7,

The fourth objective was "To Create case studies demonstrating the advantages of parametric methodology within EC5 timber connections". This has been demonstrated within two of the case studies. In the Belfast Truss study (Section 6.2) a structural design was compared between standard C16 grade and C16+ homegrown bespoke grade timber. This is not feasible if the design was undertaken without a parametric approach, i.e. using BS 5268. In a similar fashion, the Dyson student village project case study (Section 6.4) required an iterative approach of design optimisation that was only possible as a result of the parametric approach of EC5.

The fifth objective was "To Create case studies demonstrating the benefits of a transition to EC5 through the ability of optimisation". The Belfast Truss case study (Section 6.2) demonstrates that the connections required for this design are feasible. BS 5268 lacks the ability for an optimised design that was required for the 30m clear span. The iterative optimisation not only created a working solution but also provided a reduction in timber section size and the number of fixings required.

The sixth objective was "To identify and utilising routes for current research to be implemented into the AEC sector". As a result of one of the case studies where the advantages of home-grown timber were highlighted after discussion (Section 6.2), it was agreed that this bespoke C16+ strength class was to be included in the Tekla Tedds platform for all of the timber automated calculations within the library. This new strength class is the result of research work undertaken by Ridley-Ellis [222] from ENU. The Tekla Tedds platform has a user network of over 16 thousand users within the UK. In addition, the BS 8605-2 timber cladding case study (Section 6.1) directly delivers the findings of research into the hands of the AEC sector, which simplifies the specification of external timber cladding, in turn removing barriers for the AEC sector. An additional route has been identified: the use of BIM-ready equations can allow the implementation of complex problems into a BIM environment and thus to a wider audience.

The seventh objective was "To develop a proof of concept for BIM integration using multi-dimensional data fitting". This was accomplished for both axial withdrawal and lateral loading of fasteners (Chapter 7). Both of these connection calculations were successfully fitted and implemented into a BIM environment.

In conclusion, by fulfilling all of the seven objectives set out this achieves the aims set out, to ease the specification of structural timber within the AEC sector, in order to increase the UK market share for structural timber and to aid the transition from BS 5268-6.1 to EC5.

BIM-based tools can contribute to addressing some of the challenges faced by structural engineering practitioners. A BIM-based framework for the development of components that deliver Automatic Code Compliance (ACC) is presented. The structural design problems that such components solve are categorised as simple, where ACC can be implemented directly, or complex, where more advanced approaches are needed. The mathematical process of Multi-Dimensional Data Fitting (MDDF) is introduced to address the latter issue, enabling the compression of complex engineering calculations to a single equation that can be easily implemented into a BIM software engineering package while offering computational efficiency.

Analysis of both of the newly created BIM-ready equations demonstrates a high degree of certainty that the model is a good fit, providing certainty of accuracy. Please see the Table 8-1 below.

Table 8-1 The goodness-of-fit values for BIM-ready equations

	GoF_{100} (%)	GoF_{95} (%)	Max residual (kN)	More details (Section)
Axial loading	0.00000013	0.00000000012	0.000138	7.2.2
Lateral loading	0.00012	0.0000015	0.00000278	7.3.2

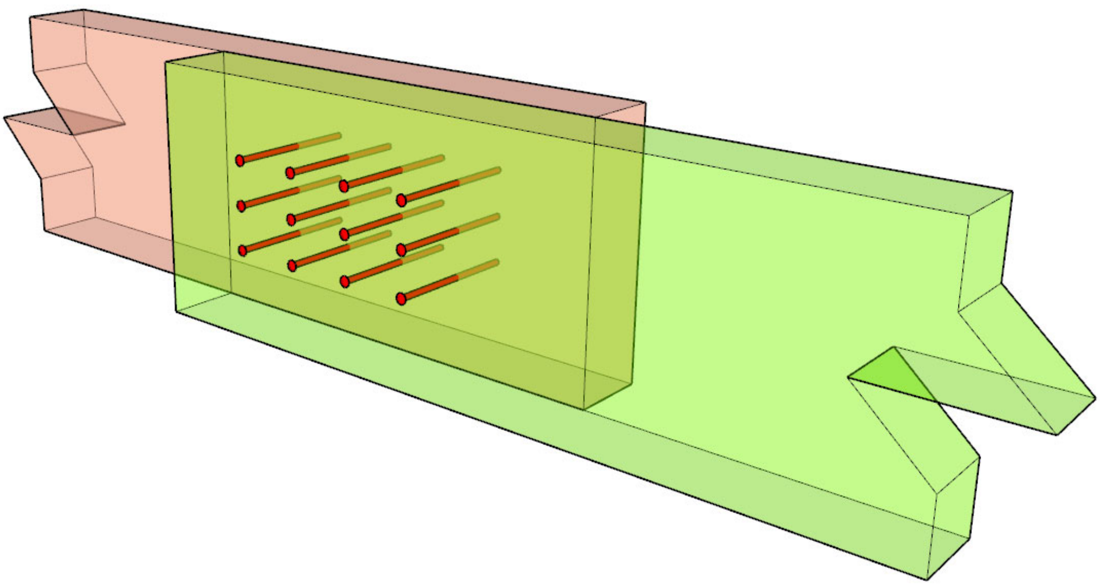


Figure 8-1 Tension splice connection

As the final step, a tension splice connection was demonstrated by the implementation of the newly created BIM-ready equations into a smart BIM object. In this object, changing any of the variables results in an automatic recalculation, thus fulfilling the seventh objective.

Chapter 9. Future Work

One of the main reasons for this work is to make the calculating of timber connections more accessible to the standard structural engineer that may or may not be familiar with timber design to EC5. Figure 9-1 shows the output from a generic three-bed semidetached house BIM model. The model itself contains much information, for this example all of the timber connections were designed using the newly created Code Compliance calculations that were created as part of this research work, and then the results were manually entered into the BIM model. The accessibility and speed for optimisation of the new CC calculation software were apparent. However if the BIM-ready equations were implemented into a standard smart BIM component that can be used in all of the connections within a BIM model, this would allow the connection design to be carried out within the model directly, allowing for true ACC in terms of the connection design. This would also eliminate error created by manual data transfer into and out of the BIM model and speed up the process by several orders of magnitude.

The MDDF approach has been demonstrated here by the creation of BIM-ready equations that calculate the timber-to-timber and panel-to-timber connections designed to EC5. There are still steel-to-timber connections to be considered in the future.

The work presented within this thesis was concentrated around the metal dowel type timber connections, this process of MDDF has potential for incorporating other elements/functions of the construction process into a BIM enabled automated code compliance. Looking at Figure 2-4, it can bee see that there are many other types of connections types, i.e. Adhesive, Timber connectors and metal plates.

<table>
<tr><th rowspan="3">Dead load [kN/m^2]</th><th rowspan="3">Variable load [kN/m^2]</th><th colspan="5">UK CLT - Consisting of C16 grade only</th><th colspan="5">UK CLT - Consisting of C16 and C24 grade</th></tr>
<tr><th>90-3h</th><th>140-5h</th><th>180-5h</th><th>240-7h</th><th>280-7h</th><th>90-3c</th><th>140-5c</th><th>180-5c</th><th>240-7c</th><th>280-7c</th></tr>
<tr><th colspan="5">Span (m)</th><th colspan="5">Span (m)</th></tr>
<tr><td rowspan="5">1</td><td>1.5</td><td>3.0 (d)</td><td>4.5 (v)</td><td rowspan="3">5.1 (v)</td><td rowspan="4">5.8 (v)</td><td rowspan="5">7.0 (v)</td><td>3.3 (d)</td><td rowspan="2">4.7 (v)</td><td rowspan="3">5.3 (v)</td><td rowspan="4">6.2 (v)</td><td rowspan="5">7.5 (v)</td></tr>
<tr><td>2</td><td>2.9 (d)</td><td>4.4 (d)</td><td>3.2 (d)</td></tr>
<tr><td>2.5</td><td>2.8 (d)</td><td>4.2 (d)</td><td>3.1 (d)</td><td>4.5 (d)</td></tr>
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<tr><td rowspan="5">1.5</td><td>1.5</td><td>2.8 (d)</td><td>4.2 (v)</td><td rowspan="4">4.8 (v)</td><td rowspan="4">5.5 (v)</td><td rowspan="5">6.6 (v)</td><td>3.1 (d)</td><td rowspan="2">4.4 (v)</td><td rowspan="4">5.0 (v)</td><td rowspan="4">5.9 (v)</td><td rowspan="5">7.1 (v)</td></tr>
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<tr><td rowspan="5">2</td><td>1.5</td><td>2.7 (d)</td><td rowspan="2">3.9 (v)</td><td rowspan="4">4.6 (v)</td><td rowspan="4">5.2 (v)</td><td rowspan="5">6.3 (v)</td><td>2.9 (d)</td><td rowspan="3">4.1 (v)</td><td rowspan="4">4.7 (v)</td><td rowspan="4">5.6 (v)</td><td rowspan="5">6.8 (v)</td></tr>
<tr><td>2</td><td>2.6 (d)</td><td>2.8 (d)</td></tr>
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<tr><td>3</td><td>2.4 (d)</td><td>3.7 (d)</td><td>2.7 (d)</td><td>4.0 (d)</td></tr>
<tr><td>4</td><td>2.3 (d)</td><td>3.5 (d)</td><td>4.4 (d)</td><td>5.1 (d)</td><td>2.5 (d)</td><td>3.8 (d)</td><td>4.6 (d)</td><td>5.5 (d)</td></tr>
</table>

Table 9-1 Floor slab span table for home grown Cross Laminated Timber panels

Another example is that of creating a fully BIM enabled ACC tool that calculates the maximum floor slab span for home grown Cross Laminated Timber panels see Table 9-1.

On a wider note, the MDDF method can be used to create solutions for many other scenarios, whether the data is from test data or from calculated data as it was in this thesis. It can be used to fit any data set and turn raw data into singular equations, which then allow for easy implementation into a native BIM environment, for automated code compliance functionality.

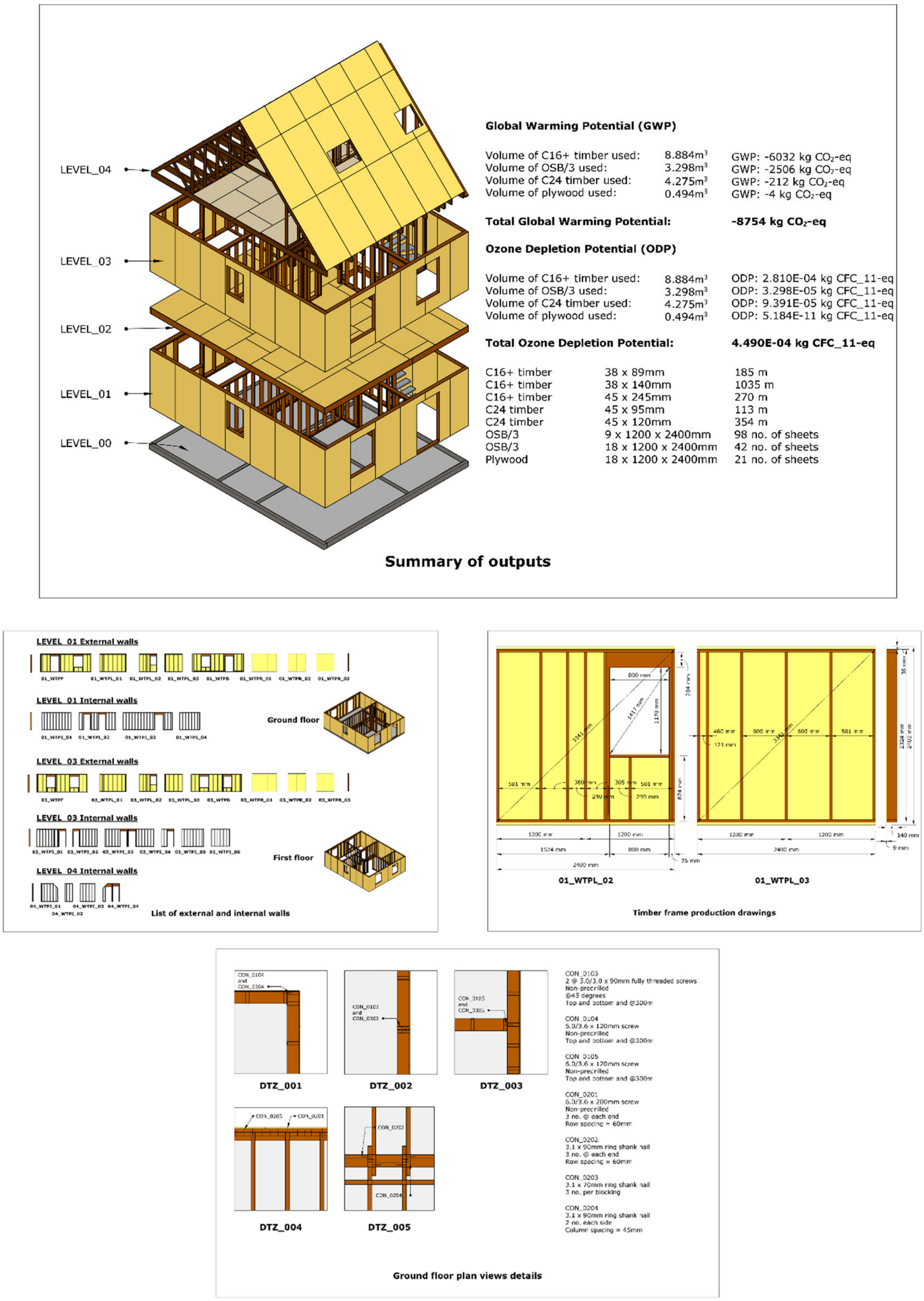

Figure 9-1 BIM model output, Generic three-bed semidetached house

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Appendices to thesis

Appendix A. Performance-based building design

The performance-based building design (PBBD) approach is in essence, the practice of operating in terms of the end results rather than the systems or means [39]. Heidkamp defines it as: A structure shall be designed in such a way that it will function in a reliable manner and within an economical way to attain the required performance [40]. These statements do not say anything about the ways and means of building, e.g. types of material, thickness, dimensions, and size of building components or methods of assembly, but instead clearly states the required end results.

Performance-based building regulatory systems are implemented within both the Scottish and the English/Welsh building regulation systems.

Implementation for England and Wales: a bill to adopt a performance-based system was presented to parliament in 1983 then by 1985 the majority of the previous rules were removed, and the new system was implemented. The obligatory guidelines for means of escape stood retained until 1991, to allow time for training of both building control and fire brigade staff. The full system of performance-based regulations was in force within England and Wales since 1991. In May 2005 Scottish building standards introduced performance-based design, in response to the European Commission's Construction Products Directive, which was brought into UK law through the Construction Products Regulations 1991 [232].

Performance-based design and Eurocodes: Heidkamp and Papaioannou comment, "a consistent realization of a design concept requires the consideration of probabilistic approaches and ultimately leads to a reliability-based design, and that this approach conforms well to the basic design concept of the Eurocodes directive" [40]. Significant progress on advanced algorithms and increased computational power have and will make full probabilistic procedures feasible and practical engineering applications.

Implementation of performance-based building codes within New Zealand: the NZBC facilitates three different methods to demonstrate compliance: [145]

- 'Acceptable solution' which accompanies prescriptive requirements
- 'verification method' compliance is demonstrated by prescribed computational and design methods
- 'alternative solution' compliance by means of proven and peer-reviewed engineering design, which can involve mathematical computation, simulations and appropriate laboratory tests.

Appendix B. EN Eurocode Parts

EN 1990	**Eurocode: Basis of structural design**
EN 1991	**Eurocode 1: Actions on structures**
EN 1991-1-1	Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings
EN 1991-1-2	Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire
EN 1991-1-3	Eurocode 1: Actions on structures - Part 1-3: General actions - Snow loads
EN 1991-1-4	Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions
EN 1991-1-5	Eurocode 1: Actions on structures - Part 1-5: General actions - Thermal actions
EN 1991-1-6	Eurocode 1: Actions on structures - Part 1-6: General actions - Actions during execution
EN 1991-1-7	Eurocode 1: Actions on structures - Part 1-7: General actions - Accidental actions
EN 1991-2	Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges
EN 1991-3	Eurocode 1: Actions on structures - Part 3: Actions induced by cranes and machinery
EN 1991-4	Eurocode 1: Actions on structures - Part 4: Silos and tanks
EN 1992	**Eurocode 2: Design of concrete structures**
EN 1992-1-1	Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings
EN 1992-1-2	Eurocode 2: Design of concrete structures - Part 1-2: General rules - Structural fire design
EN 1992-2	Eurocode 2: Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules
EN 1992-3	Eurocode 2: Design of concrete structures - Part 3: Liquid retaining and containment structures
EN 1993	**Eurocode 3: Design of steel structures**
EN 1993-1-1	Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings
EN 1993-1-2	Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design
EN 1993-1-3	Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting
EN 1993-1-4	Eurocode 3: Design of steel structures - Part 1-4: General rules - Supplementary rules for stainless steels
EN 1993-1-5	Eurocode 3: Design of steel structures - Part 1-5: General rules - Plated structural elements
EN 1993-1-6	Eurocode 3: Design of steel structures - Part 1-6: Strength and stability of shell structures
EN 1993-1-7	Eurocode 3: Design of steel structures - Part 1-7: Strength and stability of planar plated structures subject to out of plane loading
EN 1993-1-8	Eurocode 3: Design of steel structures - Part 1-8: Design of joints
EN 1993-1-9	Eurocode 3: Design of steel structures - Part 1-9: Fatigue
EN 1993-1-10	Eurocode 3: Design of steel structures - Part 1-10: Material toughness and through-thickness properties
EN 1993-1-11	Eurocode 3: Design of steel structures - Part 1-11: Design of structures with tension components
EN 1993-1-12	Eurocode 3: Design of steel structures - Part 1-12: General - High strength steels
EN 1993-2	Eurocode 3: Design of steel structures - Part 2: Steel bridges
EN 1993-3-1	Eurocode 3: Design of steel structures - Part 3-1: Towers, masts and chimneys – Towers and masts

EN 1993-3-2	Eurocode 3: Design of steel structures - Part 3-2: Towers, masts and chimneys – Chimneys
EN 1993-4-1	Eurocode 3: Design of steel structures - Part 4-1: Silos
EN 1993-4-2	Eurocode 3: Design of steel structures - Part 4-2: Tanks
EN 1993-4-3	Eurocode 3: Design of steel structures - Part 4-3: Pipelines
EN 1993-5:	Eurocode 3: Design of steel structures - Part 5: Piling
EN 1993-6:	Eurocode 3: Design of steel structures - Part 6: Crane supporting structures
EN 1994	**Eurocode 4: Design of composite steel and concrete structures**
EN 1994-1-1	Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings
EN 1994-1-2	Eurocode 4: Design of composite steel and concrete structures – Part 1-2: General rules - Structural fire design
EN 1994-2	Eurocode 4: Design of composite steel and concrete structures –Part 2: General rules and rules for bridges
EN 1995	**Eurocode 5: Design of timber structures**
EN 1995-1-1	Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1995-1-2	Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design
EN 1995-2	Eurocode 5: Design of timber structures - Part 2: Bridges
EN 1996	**Eurocode 6: Design of masonry structures**
EN 1996-1-1	Eurocode 6: Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures
EN 1996-1-2	Eurocode 6: Design of masonry structures - Part 1-2: General rules - Structural fire design
EN 1996-2	Eurocode 6: Design of masonry structures - Part 2: Design considerations, selection of materials and execution of masonry
EN 1996-3	Eurocode 6: Design of masonry structures - Part 3: Simplified calculation methods for unreinforced masonry structures
EN 1997	**Eurocode 7: Geotechnical design**
EN 1997-1	Eurocode 7: Geotechnical design - Part 1: General rules
EN 1997-2	Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing
EN 1998	**Eurocode 8: Design of structures for earthquake resistance**
EN 1998-1	Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings
EN 1998-2	Eurocode 8: Design of structures for earthquake resistance – Part 2: Bridges
EN 1998-3	Eurocode 8: Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings
EN 1998-4	Eurocode 8: Design of structures for earthquake resistance – Part 4: Silos, tanks and pipelines
EN 1998-5	Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects
EN 1998-6	Eurocode 8: Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys
EN 1999	**Eurocode 9: Design of aluminium structures**
EN 1999-1-1	Eurocode 9: Design of aluminium structures - Part 1-1: General structural rules
EN 1999-1-2	Eurocode 9: Design of aluminium structures - Part 1-2: Structural fire design
EN 1999-1-3	Eurocode 9: Design of aluminium structures - Part 1-3: Structures susceptible to fatigue
EN 1999-1-4	Eurocode 9: Design of aluminium structures - Part 1-4: Cold-formed structural sheeting
EN 1999-1-5	Eurocode 9: Design of aluminium structures - Part 1-5: Shell structures

Appendix C. Structural timber engineering for context

Timber material properties and grading

The definition of a tree can be summarised as a land plant that is normally tall, and living for more than a couple of years. For perennials, (i.e. oak, pine, coconut, bamboo etc.) the trunk remains from year to year, in contrast the herbaceous perennials (for example a banana tree) the trunk dies back each year. Generally speaking, wood is often put into either one of two categories, hardwood or softwood. These definitions have got nothing to do with the qualities of the harvested wood itself. For example, balsa wood is one of the least dense but is technically classed as a hardwood. In a similar manner, the wood of the yew tree which is classified as a softwood, and has a density higher than most hardwoods, including several types of oak. Classifying a wood is either hard or soft is entirely dependent upon the seeds produced by the tree. Hardwoods are classified if the seeds produced are encapsulated by either a shell or fruit. But if tree produces seeds that are exposed to the elements when they fall from the tree then the timber harvested from it will be classified as a softwood. The technical term given to any softwood is Gymnosperms which means naked seed, and for hardwoods the term is Angiosperms which translates to enclosed seeds. Although the terms hardwood and softwood and in no way related to the toughness of a given piece of wood, it can be seen that hardwoods generally have a higher density than softwoods.

Structural timber as a material is defined as non-homogeneous, that is to say that the material is not uniform and will have irregularities. For example, the growth rings in most cases can be easily identified, as the early wood that has grown earlier in the season contrasts to the latewood grown later in the season. Under a microscope, it can be observed that the early wood cells are wide with thin walls, where the cells of the latewood are narrow with thick walls. The extent of variability is such that two boards cut from the same tree can have very different mechanical and physical properties. Contributing factors to the variability within timber can be down to a number of different factors [227, 233] some of which are summarised as:

- knots, size and position
- slope of grain
- rate of growth, ring width
- wane, insufficient wood at the corner or on the edge
- fissures, lengthwise splits
- resin pockets and bark pockets

- distortion

- ratio of early and late wood
- timber density
- presence of reaction wood
- other damage

On a microscopic level, there are other influencing factors such as, the ratio of the molecules that make up the cell wall. The different types of molecules are: cellulose which provides tension; lignin provides compression; hemicelluloses which links the cellulose and lignin together giving flexibility; extractives and water. All of these factors are influenced from the sawmill processing and drying the timber, the management of the forest and climate conditions while the tree grew, and species of tree [227].

Within Europe there are three key properties used for determining the structural grade of timber, these are strength, stiffness and density. These can either be tested in bending or in tension, within laboratory tests conducted in accordance with [234, 235]. For standard construction timber the strength grade classifications are based on bending and are defined within [236], the grades are either prefixed with 'C' for softwoods and 'D' for hardwoods. For timber products where tension is the governing factor for example, the manufacture of glue laminated products the strength grade classifications are based on tension, and are defined within [237], which have a prefix of 'T'.

Timber member design

The basic requirements for designing timber structures and member in accordance with [238], will be fulfilled when a limit state design encompassing partial factors methods using [239] for actions and combinations, along with the supplementary provisions outlined in [63].

For timber member design the engineer has to identify whether the member is subject to flexure, axle or a combination of both actions. As the design requirements differ, which are summarised in Table C-1.

Table C-1 Main design requirements for design to EC5

For flexural members	For axially loaded members
• Static equilibrium	• Static equilibrium
• Bending stress & lateral torsional instability	• Axial stress & lateral instability
• Shear stress	• Deflection
• Bearing stress	
• Torsion stress	
• Deflection	
• Vibration	

Timber connection overview

An interesting way of looking at structural engineering is described by Thomas McLain: "a structure is a constructed assembly of joints separated by members" [67]. That is to say that the joints are generally the critical factor in the design of the structure. The strength of the connectors in the joint will normally dictate the strength of the structure; their stiffness will greatly influence its overall behaviour and member sizes will generally be determined by the numbers and physical characteristics of the connector rather than by the strength requirements of the member material.

Key points:

- Joints are crucial points in many timber structures because they can determine the overall strength and performance of that structure.
- The length of structural timber is generally shorter than the required spans and as a result splicing or composite structures (e.g. trusses) must be used.
- Forces between members are most often transferred through lap joints, either by adhesives (glues) or by dowel-type fixings (nails, bolts, screws, dowels or nail plates).

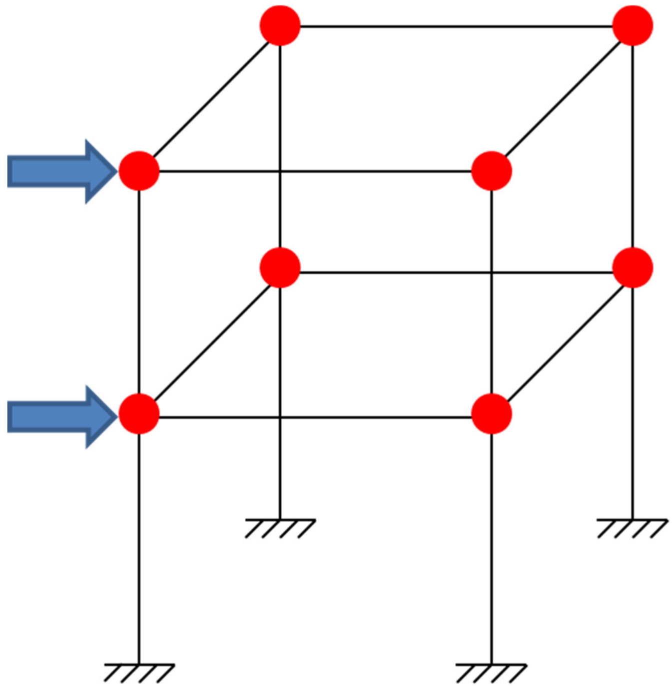


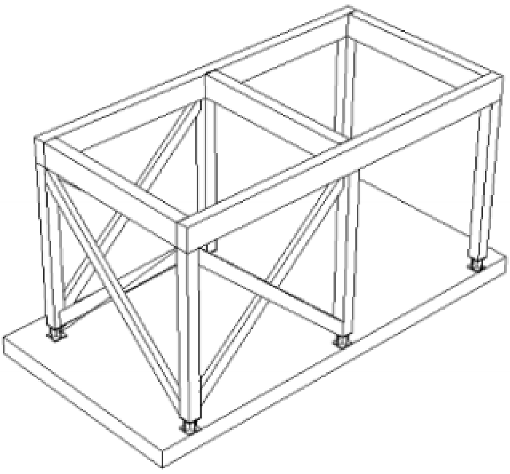
Figure C- 1 Joints and system

Examples of connections in systems

There are a number of different ways that the connection can work within a system. Commonly used pinned-connections can be found in a simple post and beam timber system, see Figure C- 2 Pinned connections, Post and beam system [227]. Moment resistance or semi-rigid connections see Figure C- 3 Moment-resisting frames, [227]. showing a moment-resisting frame, an example can be found in the John Hope Gateway Biodiversity Centre, Edinburgh. Where a moment resistance connection within a stiff roof diaphragm see Figure C- 4 John Hope Gateway Biodiversity Centre, Edinburgh, this transmits the lateral loads to the concrete walls and cores rather than to the slender steel rod columns. Figure C- 5 Sibelius Hall, Lahti, Finland, https://www.sibeliustalo.fi/en/sibelius-hall

shows a tree truss supported by columns, the compressive forces are transmitted to the: we're the connection method, and in Figure C- 6 Scottish parliament, debating chamber

. a truss system where compressive forces are transmitted from the timber web elements to the steel chords, which are intention, via the connection method.



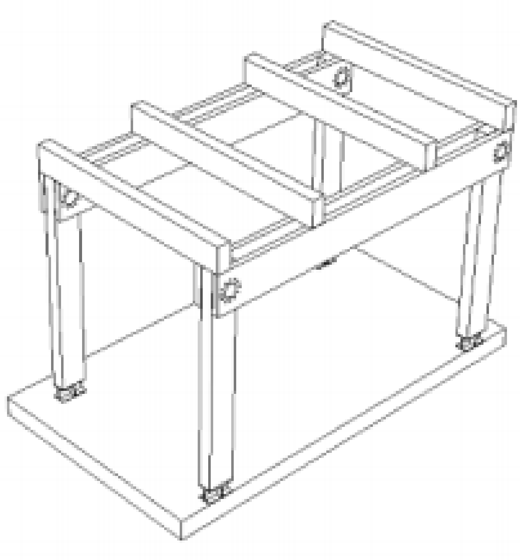


Figure C- 2 Pinned connections, Post and beam system [227]

Figure C- 3 Moment-resisting frames, [227]

Figure C- 4 John Hope Gateway Biodiversity Centre, Edinburgh

Figure C- 5 Sibelius Hall, Lahti, Finland, https://www.sibeliustalo.fi/en/sibelius-hall

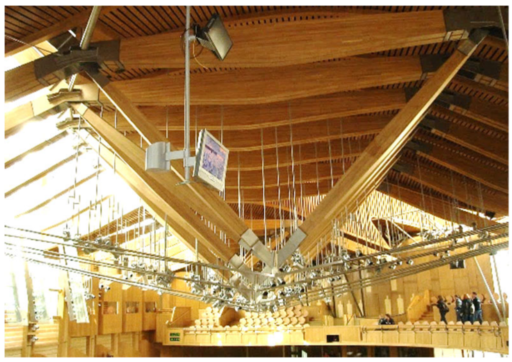


Figure C- 6 Scottish parliament, debating chamber

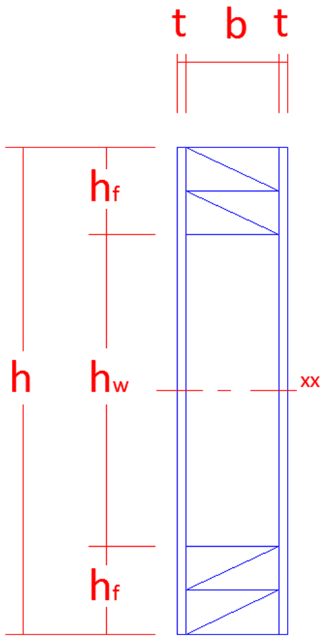


Figure C- 7 Plywood box beam

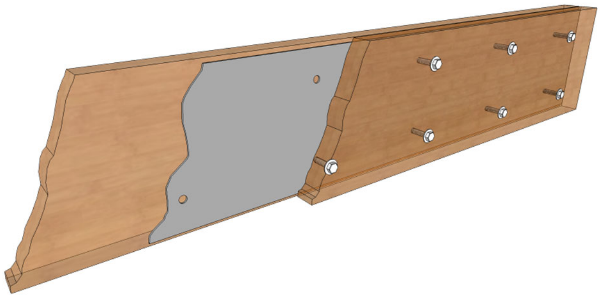


Figure C- 8 Flitch beam

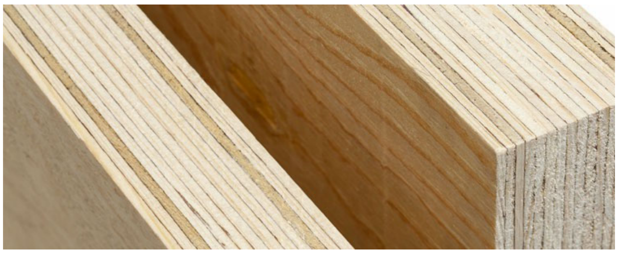


Figure C- 9 Laminated veneer lumber, www.metsawood.com

Figure C- 10 Glue laminated timber beams

Increasing spans through connections

There are a range of options to increase the span by the inclusion of connections. There are multiple different styles of timber trusses and long span timber beams, features and advantages of some are detailed within Table C-2. Figure C- 11 Roof truss examples

(a-c) show some different styles of roof truss where connections have been used to combine timber elements of different lengths to achieve longer spans.

a) b) c)

Figure C- 11 Roof truss examples

There are a number of different styles of timber beams possible, four of the more common beams are listed within Table C-2, which also highlights their differences / advantages. Even although there are some similarities within these timber beams construction / manufacturing process are quite different from each other. The plywood box beams are manufactured by nailing and gluing plywood sheets to horizontal flanges and vertical stiffeners Figure C- 7 Plywood box beam

. Flitch beams are commonly constructed by bolting or nailing together two or more pieces of timber with a metal plate in between Figure C- 8 Flitch beam

. Laminated veneer lumber beams (LVL) is an offsite manufacturing process of laminating thin sheets of timber together to form a solid section Figure C- 9 Laminated veneer lumber, www.metsawood.com. Glue laminated timber beams are also manufactured offsite, by gluing together timber to form solid timber sections Figure C-10 Glue laminated timber beams

.

Table C-2 Truss and beam comparisons

	Suitable for offsite manufacture	Suitable for fabricated on site	Lightweight	Created from existing in-situ joists or beams	Dimensional stability	Uniform	Standard stock readily available	Available in customised design	Notes:
Roor truss	✓	✓	✓					✓	
Plywood box beams	✓	✓	✓						It is easy to include a pre-camber when manufactured offsite
Flitch beams	✓	✓		✓					They require less depth than wood only beams of the same strength
Laminated veneer lumber beams	✓				✓	✓	✓		High compression strength, depending on the percentage of veneers laid crossways.
Glue laminated timber beams	✓				✓		✓	✓	Well suited for decorative portal frame structures

Connection types

Forces between structural members must be transferred, and this can be done in a manner of fashions. The traditional timber carpentry joints can look pleasing to the eye see Figure C- 12 Carpentry joint, http://www.dytimberframing.com

Figure C- 12 Carpentry joint, http://www.dytimberframing.com

, and also function incredibly well when used in the correct setting. However, the carpenter creating the timber connections requires a high skill level, and creating the connection is very time consuming. The other options are glued joints; dowel type connectors such as nails, screws, bolts and dowels and connection plates. All of these options are outlined in more detail below.

Glued joints

To form a glued joint, see Figure C- 13 Glued I joist section

high levels of quality assurance are required in the manufacturing process to ensure a strong bond is created. In choosing an appropriate adhesive the following things have to be taken into account: the method of application, the required speed for curing, will heating is needed or available, and the cost. The cost of the connection not only depends on the manufacturing process but also on the adhesive itself and its rate of spread.

Figure C- 13 Glued I joist section

Correctly made bonded joints on timber surfaces parallel to the grain have the same properties as the wood. Likewise, joints between panel materials and timber and panel materials have the same properties as the weakest of the materials. This is the basis for glued laminated timber (glu-lam), and for built-up members such as box-and I-beams.

Structural glued joints are often stiffer, and can be more pleasing to the eye when compared with mechanical type fasteners. They can be more suitable for a corrosive environment, and when made with thermosetting resins provide better fire resistance than that of metal dowel-type fasteners. For example, when a nail is exposed to fire it begins to heat up, which then chars the wood along the full length of the fastener, reducing the fire resistance time. The main disadvantages of using structural glued connections is that you require strict quality control, therefore it is better achieved within an offsite factory environment. Glued connections are unsuitable when there are significant loads perpendicular to the glue plane. The connection strength is heavily dependent upon angle of grain, and can be unsuitable for connecting different types of material together, or where there is a fluctuating moisture content. Some commonly used glues, together with their properties, are outlined in Table C-3.

Table C-3 Adhesive comparison table

Adhesive	Application	Setting process and cure time	Advantages / Disadvantages
Thermo-Plastic			
Polyvinyl Acetate, Catalyzed Polyvinyl Acetate (PVA)	interior but some special formulations are waterproof	non-reactive, 40 minutes at room temperature	easy to work with
Hot Melts	Interior, high speed production lines	non-reactive, sets by cooling	grips on contact when hot
Thermo and Room Temperature Set			
Resorcinol formaldehyde (RF)	fully exterior, laminating, finger jointing, wood jointing	reactive, sets in 2 minutes with heat and 6 hours at room temperature	waterproof, high cost, marine-plywood
Phenol-resorcinol formaldehyde (PRF)			waterproof
Phenol formaldehyde (PF)	fully exterior, plywood, some particalboard		
Thermo-Set			
Melamine formaldehyde (MF)	semi-exterior and Interior, plywood, particleboard, formwork panels. (not often used alone in the UK)	reactive, sets with heat in 2 minutes and 30 minutes to 12 hours at room temperature	moisture resistant, low cost
Melamine urea formaldehyde (MUF)	semi-exterior and Interior, laminating, plywood, particleboard, finger jointing		
Urea formaldehyde (UF)	interior, plywood, particleboard, wood jointing, bent laminations	10 to 12 hours to cure. There are liquid catalysts that will allow the resin to cure in 20 minutes	easy to work, withsomewat gap filling, moisture resistant, foundry sand molds
Isocyanates and Polyurethanes (Most Polyurethane are thermo-set but thermoplastic are available)	isocyanates fully exterior, polyurethane semi-exterior and moist interior where temperature does not exceed 50°, laminating	reactive, one component sets with heat in 2 minutes, from to 2 to 60 minutes at room temperature for two-part resins	ability to set in high moisture conditions, suitable for multiple martials, 100% solid, good gap filling properties, low glue spread rate, expensive
Catalyst			
Epoxy resins	semi-exterior and Interior	reactive, hardens between 2 - 60 min gains full strength in 24 hours	structural repairs, suitable for multiple martials, timber end-jointing, waterproof, good gap filling properties
(1)	An elevated temperature is required to cure PF, MF and MUF adhesives.		
(2)	PVA (polyvinyl acetate) adhesives should not be used for structural purposes, but in certain limited circumstances PVAc (cross linked PVA adhesives) may be acceptable.		

The contents of this table are from a number of sources. [240-244]

Dowel type connectors

Wooden dowels and pegs have been used in woodworking for many centuries. Joints with dowels are used in timber construction to transmit high forces, and are an economical type of joint which are easy to produce. Dowels are circular rods of timber Figure C- 14 Timber dowel double shear test

, steel, or carbon-reinforced plastics which have a minimum diameter of 6mm and a maximum of 30 mm. The dowels are driven into identically or marginally undersized holes. These holes must either be drilled through all members in one operation or made using computer numerical control machines (CNC).

Figure C- 14 Timber dowel double shear test

Nails

Nails are the most commonly used fasteners in timber construction (see Figure C- 15 Timber frame racking panels

) and are available in a variety of lengths, cross-sectional areas and surface treatments Figure C- 16 Coil of nails

. The most common type of nail is the smooth steel wire nail which has a circular cross-section and is cut from wire coil having a minimum tensile strength of 600N/mm^2. They are available in a standard range of diameters up to a maximum of 8mm and can be plain or treated against corrosion, for example, by galvanising. Nails may be driven in by hand or by use of a nail gun. When nails are to be driven into dense timbers there is a danger that excessive splitting will occur. Methods of avoiding splitting are blunting the pointed end of the nail so that it cuts through the timber fibres rather than separating them or to pre-drill a hole in the timber less than 80% of the nail diameter. Pre-drilling is not normally carried out on timbers with a lower characteristic density of 500kg/m^3. As well as having smooth nails there are also threaded / grooved nails as defined by [245] which have a greater resistance to axle withdrawal.

Advantages of pre-drilling nails:

- The lateral load carrying capacity of the nail is increased.
- The spacing between the nails, and the distances between the nails and the end and edge of the timber may be reduced thus producing more compact joints.
- Less slip occurs in the finished joints.

Disadvantages:

- Labour intensive and as a result expensive.
- Reduces the cross-sectional area of the member.

Figure C- 15 Timber frame racking panels

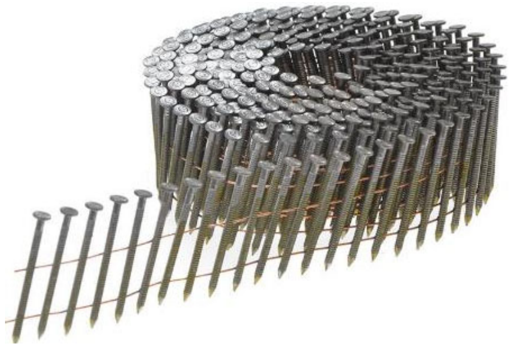


Figure C- 16 Coil of nails

Screws

Wood screws (see Figure C- 17 Common Wood screws

) are especially suitable for steel-to-timber and panel to timber joints, but they can also be used for timber-to-timber joints. Such screwed joints are normally designed as single shear joints. Screws are inserted by turning and this can be done either by hand or by power actuated tool depending on the situation. The main advantage a screw has over a nail is its additional axial withdrawal capacity.

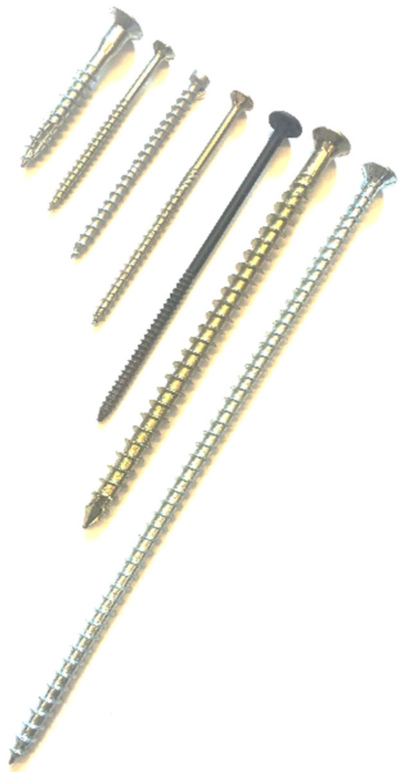


Figure C- 17 Common Wood screws

Bolts

Bolt connections offer a higher lateral load carrying capacity than nails or screws. Standard bolts are made from carbon or alloy steels and come in a variety of different bolt classes, which are categorised by yield strength and ultimate tensile strength see

Table C-4. These connections are generally easily fabricated; the bolts are inserted into oversized predrilled holes. The diameter of the oversized holes in timber should be no greater than 1 mm larger than the diameter of the bolt, or 2 mm larger for steel plate. For the purposes of EC5 calculations bolts can range from 6 mm to 30 mm in diameter. They are used with washers that have a side length of about 3d and thickness of 0.3d, where d is the bolt diameter. A bolted connection will be tightened on application so that the members of the connection fit closely together. If necessary bolts will be required to be re-tightened when the timber has reached equilibrium moisture content.

Another type of bolt is a lag screw which has a sharp end and coarse threads designed to penetrate and grip wood fibre see Figure C- 18 Photograph of standard bolt and lag screw

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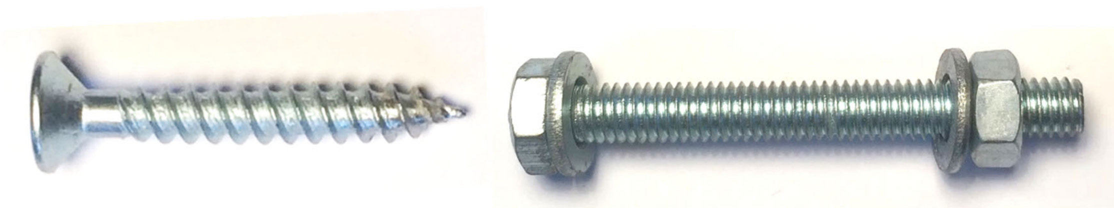


Figure C- 18 Photograph of standard bolt and lag screw

Table C-4 Yield strength f_yb and ultimate tensile strength f_ub for bolts [234]

Bolt Class	f_{yb} N/mm^2	f_{ub} N/mm^2
4.6	240	400
4.8	320	400
5.6	300	500
5.8	400	500
6.8	480	600
8.8	640	800
10.9	900	1000

Timber connectors

Bolted joints subject to lateral loading can be strengthened significantly by the addition of connectors in the joint surface, as they enlarge the wood contact area over which the load is distributed. They are mainly used to transfer loads in heavy timber or glulam members such as roof trusses. They are not usually protectively coated and need to be galvanized when used with wood treated with preservative or in wet service conditions. Specification and installation of the bolt is important as it clamps the joint together so that the connector acts effectively.

This style of connector has been available for more than a hundred years. The first patient dates back to 1889 [246]. Today there are three main categories available which are:

- Ring and split-ring connectors are for timber to timber only and are installed in pre-cut grooves.
- Shear plate connectors are for timber to timber or timber to steel and are installed in pre-cut grooves.
- Toothed-plate connectors are for timber to timber or timber to steel and are pressed into the timber.

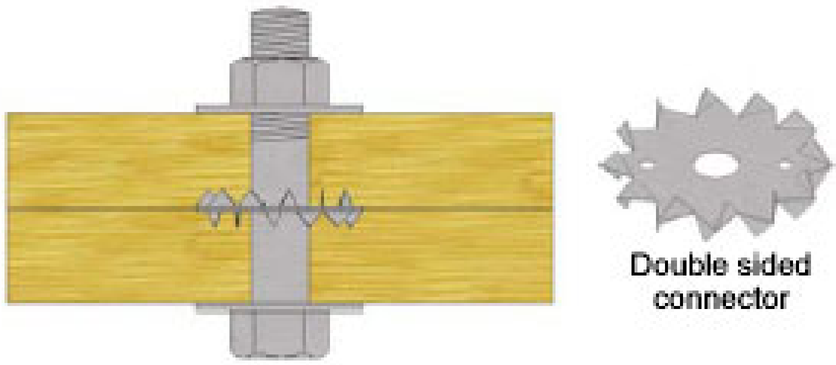


Figure C- 19 Timber connectors, Toothed plate connector www.cullen-bp.co.uk

Connection plates

The definition of punched metal plates fastener comes from BS EN 1075: "Timber Structures – Joints made of punched metal fasteners" as "a fastener made of metal plate having integral projections punched out in one direction and bent perpendicular to the base of the plate, being used to join two or more pieces of timber of the same thickness in the same plane" see Figure C- 20. The punched metal plate fastener was invented in Florida in 1952, which then revolutionised the timber truss industry [247]. The metal used is generally galvanised or stainless steel plate, of thicknesses varying from 0.9mm to 2.5mm.

The key advantages are the low cost, ease of installation, their higher structural efficiency and rotational stiffness.

The limiting strength of a punched metal plate is determined by one of two criteria:

- Its anchorage (gripping) capacity in any of the jointed members.
- Its net sectional steel capacity at any of the interfaces.

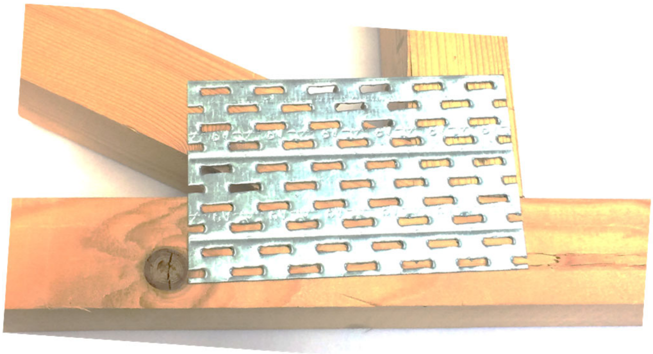


Figure C- 20 Punched metal plate connector

Dimensional nailing plates are made of light-gauge mild steel cut and folded to shape and pre-punched with holes for specified nails see Figure C- 21. The most common kinds are:

- Angle brackets
- Joist hangers
- Truss clips

Figure C- 21 Three-dimension nailing plates

Specification of connections

When designing a connection, the specification of the fixing will depend on a range of factors:

- The nature of the forces being applied and their magnitude.
- The practicality and/or manufacturability.
- The aesthetics required.
- The environmental conditions.
- The cost.

It is also important to consider how the whole system is to function, and this will depend not only on the load-carrying capacity of connection but also on the load-deformation characteristics of the connection. If the system being designed is statically indeterminate then the load deformation is influenced by the load deformation of the members and slip in the joints. Slip in the joints is often the largest contributor and can therefore be an important criterion in specification.

Also, important in design is the concept of connections acting together. Nails, screws and bolts can be used together in a joint as they have similar ductile behaviour. However, because the tolerance required in the bolt predrilled holes leads to higher initial slip, bolts should not be considered to be acting together with other mechanical fasteners.

Structural systems

To create a structural system out of timber, multiple structural members or components are connected together. There are four main categories of structural systems:

Panelised systems, otherwise called two-dimensional construction systems, can take the form of wall panels, floor or roof castes etc (Figure C- 22).

Volumetric systems take the form of three-dimensional units manufactured in a factory then delivered in a near completed state to the construction site. These units can be fully finished complete with fixtures and fittings, or be a module within a larger building or complex. Volumetric systems are often assembled from combinations of panelised systems (Figure C- 23).

Sub-assemblies and components are items that do not meet either of the above criteria, for example: roof trusses, doors, windows. These are usually manufactured within a factory environment (Figure C- 24).

Hybrid systems are a combination of more than one system, for example a combination of volumetric and panelised system (Figure C- 25).

These systems generally lend themselves to Design for manufacture and assembly (DfMA), off-site manufacture (OSM) which can enable mass-customisation (MC).

Figure C- 22 Panelised wall system. from CCG

Figure C- 23 Volumetric modular system. From SiBCAS Ltd

Figure C- 24 Sub-assembly system, Belfast truss [247]

Figure C- 25 Hybrid system

Appendix D. Connection design

Parameter s of influence

The equations used in EC5 rely upon three main parameters of influence for the load carrying capacity and behaviour of joints with dowel type fasteners, which are:

1. The bending capacity of the dowel or yield moment.
2. The embedding strength of the timber or wood-based material.
3. The withdrawal strength of the dowel.

These three parameters are discussed in the following three sections:

Bending capacity / yield moment

Bending capacity or yield moment is theoretically the maximum bending moment that the dowel type fastener can resist before going into a plastic deformation. Figure D- 1 and Figure D- 2 shows a metal dowel connection that has been tested into plastic deformation. The possible strength increase due to the plastic deformation is disregarded.

$M_{y.Rk} = 0.3\ f_u\ d^{2.6}$ for round nails EC5 eq- 8.14

Which is

$M_{y.Rk} = 0.45\ f_u\ d^{2.6}$ for square and grooved nails

where:

$M_{y.Rk}$ is the characteristic value for the yield moment

d is the nail diameter

f_u is the tensile strength of the wire

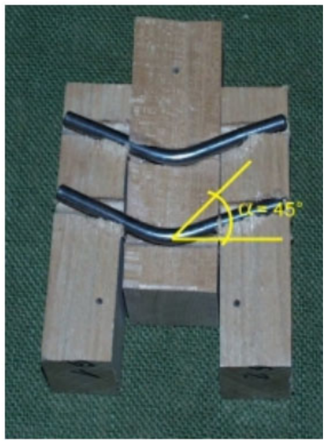


Figure D- 1 Metal dowel double shear test, Blass [1]

Figure D- 2 Screw dowel double shear test

Embedment strength

The Johansen's theory assumes that timber is a rigid plastic material, research into embedment strength started as early as 1954, unfortunately no standardised methods or procedures were in place and it wasn't until the late 1980's that development of a standardised test method and procedure to characterise the embedment strength of timber fasteners was created by [248] see Figure D- 3. This was then adopted by the European committee developing the harmonised design code and test methods, and into the british standards in 1993 [249] and 2007 [250].

The embedment strength of the timber, or wood-based product, f_h, is the average compressive strength at maximum load under the action of a stiff straight dowel. According to BS EN 383 [250].

$$f_h = \frac{F_{max}}{d \cdot t} \quad (BS\ EN\ 383\ eq.2)$$

where:

F_{max} is maximum load of the test, or the load at which a 5mm deformation occurred

d is the Fastener diameter

t is the thickness of the timber

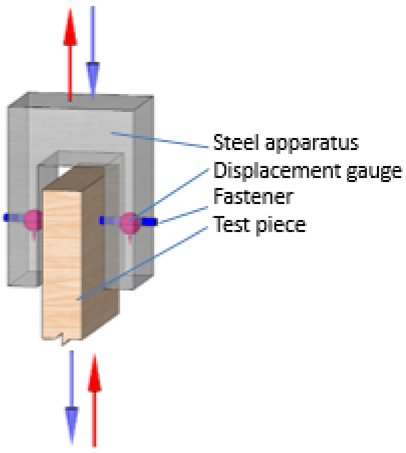


Figure D- 3 European embedment test setup

At a similar time to the European test methods was being developed, over in America, research by [251] was developing a different approach for measuring embedment strength, which has been adopted by the American and Canadian standards. The advantage of this approach, is that it eliminates bending of the fastener during testing, see Figure D- 4. There are also negatives of this approach, which makes it difficult to measure the embedment stiffness / deformations [252].

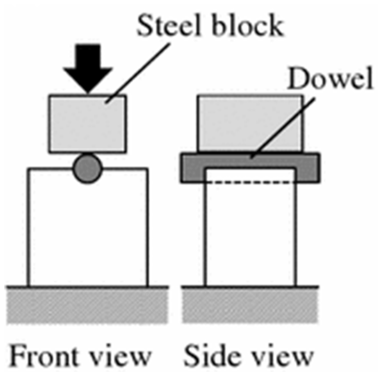


Figure D- 4 American embedment test setup, image from [253]

From experimental tests the following parameters influence the embedment strength of timber: [253, 254]

- Density: embedment strength increases in a linear manner with respect to timber density.
- Moisture content: as this increases the bending strength decreases, and this is independent of timber species and dowel diameter.
- Diameter of the fixing or the predrilled hole: the embedment strength decreases with increasing fastener diameter.

- Reinforcement of timber perpendicular to grain: for example, installing self-tapping wood screws either side of a bolted connection will increase embedment strength within that timber member.
- Friction between the fixing and timber will increase embedment strength.

Embedment strength for nails as defined by EC5

For nailed panel-to-timber connections the embedment strengths are as defined by EC5, and summarised in Table D-1.

Table D-1 Characteristic embedment strength of nails for EC5

Timber based product	**Nail limitations**	**Characteristic embedment strength, $f_{h,0,k}$ (N/mm²)**
LVL and timber	Nails with diameter up to 8mm	$0.082\rho_k \cdot d^{-0.3}$ without predrilled holes & $0.082(1-0.01\cdot d)\rho_k$ with predrilled holes
Plywood	Head diameter of at least 2d	$0.11\rho_k \cdot d^{-0.3}$
Hardboard in accordance with EN 662-2		$30 \cdot d^{-0.3} \cdot t^{0.6}$
Particle board and OSB		$65 \cdot d^{-0.7} \cdot t^{0.1}$

where:

ρ_k is the characteristic density of the timber in kg/m³,

d is the diameter of the nail in mm, and

t is the panel thickness in mm.

Ratio of characteristic embedment strengths

To simplify the strength equations, the **ratio** of the characteristic embedment strength of member 2, $f_{h,2,k}$, to the characteristic embedment strength of member 1, $f_{h,1,k}$, is derived and written as:

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \qquad EC5\ eq.8.8$$

where:

$f_{h,1,k}$ Characteristic embedment strength of timber, in headside member

$f_{h,2,k}$ Characteristic embedment strength of timber, in pointside member

Characteristic withdrawal capacity of nails

Characteristic withdrawal capacity within EC5 for nails comes from the minimum of the pointside axle withdrawal and the headside pull-through.

For nails other than smooth nails, as defined in EN 14592

$$F_{ax.Rk} = min\begin{cases} f_{ax.k} \cdot d \cdot t_{pen} \\ f_{head.k} \cdot d_h^2 \end{cases}$$

EC5 eq. 8.23

For smooth nails

$$F_{ax.Rk} = min\begin{cases} f_{ax.k} \cdot d \cdot t_{pen} \\ f_{ax.k} \cdot d \cdot t + f_{head.k} \cdot d_h^2 \end{cases}$$

EC5 eq. 8.24

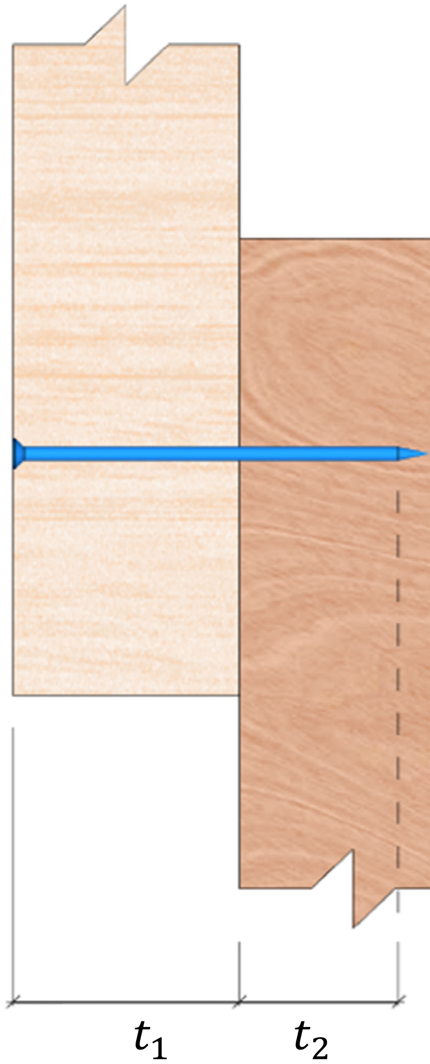


Figure D- 5 Pint-side and head-side of connection fixing

$f_{ax.k}$	is the characteristic point-side withdrawal strength
$f_{head.k}$	is the characteristic head-side pull-through strength
d	is the nail diameter
t_{pen}	is the point-side penetration length or the length of the threaded part, excluding the point length, in the point side member
t	is the thickness of the head side member
d_h	is the nail head diameter

The characteristic withdrawal strength $f_{ax.k}$ and $f_{head.k}$ should be determined by test, unless specified in the following.

For smooth nails with a point-side penetration of at least $12 \cdot d$.

$f_{ax.k} = 20 \cdot 10^{-6} \cdot \rho_k^2$ EC5 eq. 8.25

$f_{head.k} = 70 \cdot 10^{-6} \cdot \rho_k^2$ EC5 eq. 8.26

where:

ρ_k is the characteristic timber density in kg/m^3.

Once the characteristic withdrawal strengths are identified, either from the manufacturers test data, or from the equations above, you may still be required to multiply by a reduction factor depending upon the penetration depth.

From EC5 8.3.2(7) reduction factors

For smooth nails, t_{pen} should be at least $8 \cdot d$

when $t_{pen} < 12 \cdot d$ the withdrawal capacity is multiplied by $\frac{t_{pen}}{4 \cdot d - 2}$

For threaded nails, t_{pen} should be at least $6 \cdot d$

when $t_{pen} < 8 \cdot d$ the withdrawal capacity is multiplied by $\frac{t_{pen}}{2 \cdot d - 3}$

Johansen's equations: Failure mode calculation format

The calculation approach used within EC5 is based upon Johansen's general theory, and predicts the method of failure for the timber-to-timber and steel-to-timber connections using dowel type fixings.

This calculates each possible failure modes in turn, and so identifies the failure mode with the lowest resistance.

The general format for these equations can be summarised as: the result from the Johansen's yield load, multiplied by any effect from friction, plus the rope effect.

$$F_{v.Rk} = friction\,factor \quad * \quad johansen\,yield\,load \quad + \quad Rope\,effect$$

friction effects

There are two types of friction effect that can arise between the two timber members in a connection:

The first will develop if the members are in direct contact when assembled (see ***Error! Reference source not found.***). This friction will be eliminated either if there is no direct contact on assembly or if there is shrinkage of the timber or wood products in service. As a result of this it is conservatively not considered in EC5 (friction factor = 1).

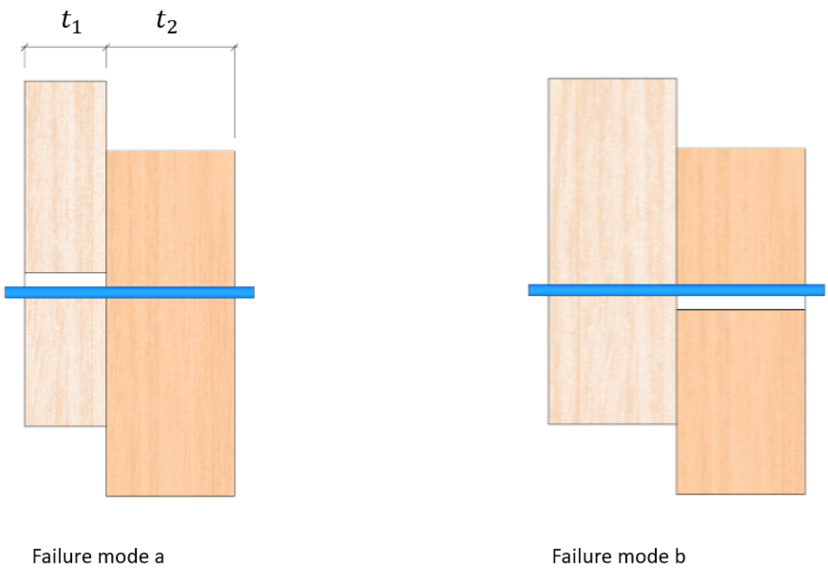


Figure D- 6 Connection, friction not considered

The other will arise when the fasteners yield, pulling the members together as the fasteners deform (see Figure D- 7). This type of friction will always arise in failure modes that include yielding of the fasteners and has been included in the EC5 equations relating to such modes. This effect is termed the "rope effect".

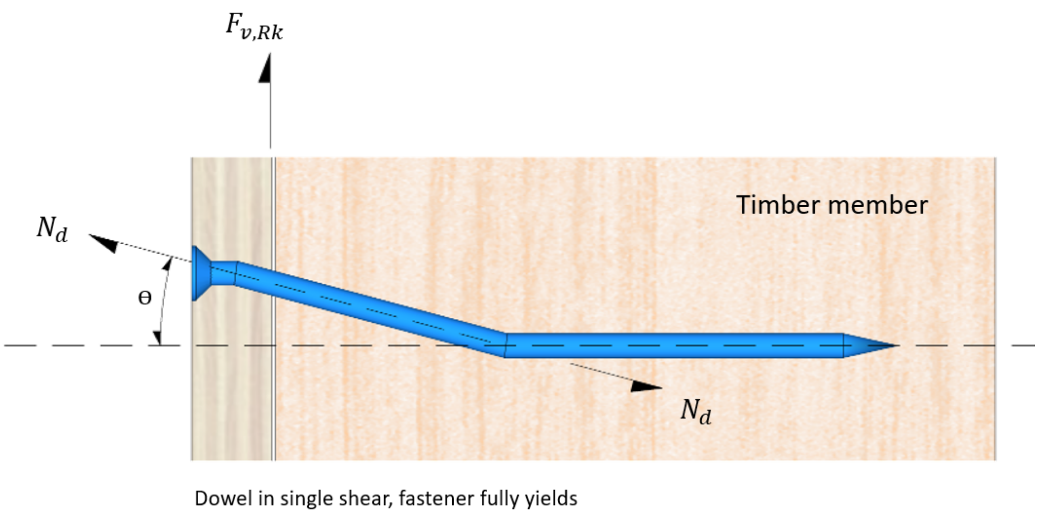


Figure D- 7 Connection friction effect, fastener yields

When the dowel type fastener does not yield we assume that there is no fiction factor, but as the fastener begins to yield then the fiction factor should be included. The friction factor to be used is outlined in Table D-2.

Table D-2 Friction factors

friction factor		failure modes (see section X)
1.05 (+5%)	fastener partially yields	(d) (e) (j)
1.15 (+15%)	fastener fully yields	(f) (k)

Rope effect

The rope effect is the contribution to the lateral load carrying capacity of the point side withdrawal of dowel type fastener.

Which can be written as: $Rope\ effect = \frac{F_{ax.Rk}}{4}$

Please note: If the axial withdrawal capacity of the fastener is not known then the rope effect should be considered as zero.

limiting factor

there is a limiting factor to the rope effect, depending upon the profile of the fixing the maximum uplift percentage limit is altered. As described below.

$$F_{v.Rk} = min\begin{pmatrix} X + Rope\ effect \\ X \cdot LP\% \end{pmatrix}$$

$$X = friction\ factor \cdot johansen\ yield\ load$$

$$Limiting\ percentage\ (Lp) = \begin{Bmatrix} \text{15\% Round nails} \\ \text{25\% Square nails} \\ \text{50\% Other nails} \\ \text{100\% Screws} \\ \text{25\% Bolts} \\ \text{0\% Dowels} \end{Bmatrix}$$

member notation

For the member thickness within EC5 connection calculation the thicknesses are identified as either t_1 or t_2. This is also true for multiple shear plane connections which is explained in more detail later on.

For single shear connections:

t_1 is the fixing head-side member thickness;

t_2 is the fixing point-side penetration;

where 'fixing head-side material thickness' is the thickness of the member containing the fixing head and 'fixing point-side thickness' is the distance that the pointed end of the nail penetrates into a member, minus the point length.

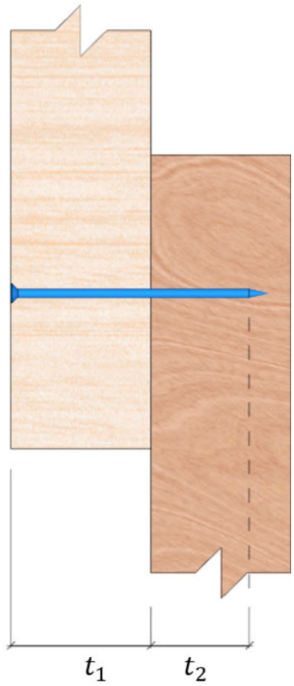


Figure D- 8 Single shear member notation

For double shear connections:

t_1 is the minimum of the fixing head-side member thickness and the fixing point-side penetration minus the point length.

t_2 is the central member thickness for a connection.

Note: In a three-member connection, nails may overlap in the central member provided: $(t - t_2) > 4 \cdot d$

where d is the diameter of fixing.

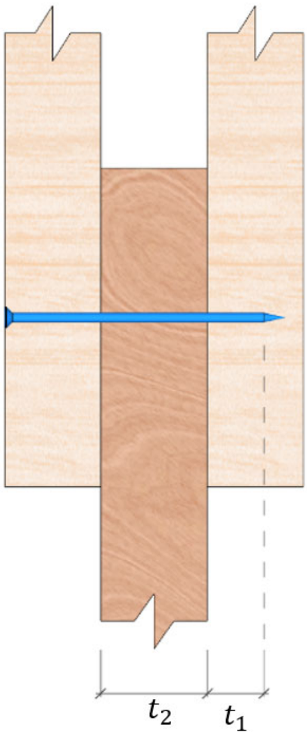


Figure D- 9 Double shear member notation

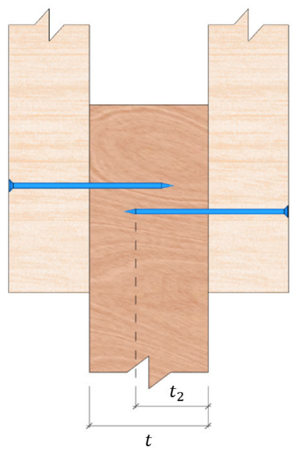
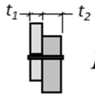


Figure D- 10 Nail overlap

single shear failure modes

$F_{v.Rk} = \min$

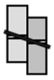


$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d$$

(a)

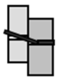
$$F_{v,Rk} = f_{h,2,k} \cdot t_2 \cdot d$$

(b)



$$F_{v,Rk} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1 + \frac{t_2}{t_1}\right)\right] + \frac{F_{ax,Rk}}{4}$$

(c)



$$F_{v,Rk} = 1.05\frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \frac{F_{ax,Rk}}{4}$$

(d)

$$F_{v,Rk} = 1.05\frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_2^2 \cdot d}} - \beta\right] + \frac{F_{ax,Rk}}{4}$$

(e)

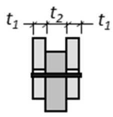
$$F_{v,Rk} = 1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$$

(f)

Figure D- 11 Johansen's timber to timber single shear equations

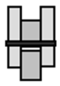
double shear failure modes

$F_{v.Rk} = \min$



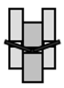
$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d$$

(g)



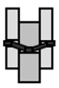
$$F_{v,Rk} = 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d$$

(h)



$$F_{v,Rk} = 1.05\frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \frac{F_{ax,Rk}}{4}$$

(j)



$$F_{v,Rk} = 1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$$

(k)

Figure D- 12 Johansen's timber to timber double shear equations

Other design considerations

Splitting capacity

When a lateral load is applied at an angle to the grain there is a potential for splitting. The splitting capacity is satisfied when:

$$\frac{F_{la.sp} \cdot Sin(\alpha)}{F_{90.Rd}} < 1$$

where:

$F_{90.Rk} = 14\, b\, w \sqrt{\frac{h_e}{1-\frac{h_e}{h}}}$ is the characteristic splitting capacity

$F_{90.Rd} = k_{mod} \cdot \frac{F_{90.Rk}}{\gamma_{M.connection}}$ is the design splitting capacity

$w = \max \begin{cases} \left(\frac{w_{pl}}{100}\right)^{0.35} \\ 1 \end{cases}$ when using punched metal plate fasteners

$w = 1$ for all other fasteners

and:

$F_{la.sp}$ is the design force in the shear plane

w is a modification factor

h_e is the loaded edge distance to the centre of the most distant fastener or to the edge of the punched metal plate fastener

h is the timber member height

b is the member thickness

w_{pl} is the width of the punched metal plate fastener parallel to the grain

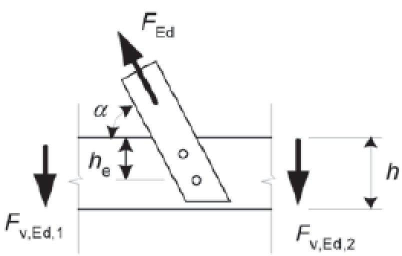


Figure D- 13 Splitting capacity

Multiple fastener connections loaded laterally

The total load carrying capacity of the joint will be the combined ultimate loads of the fasteners. However, this would only be the case if all the respective single fasteners reached their ultimate loads at the same time as the whole connection failed. In fact, the ultimate load carrying capacity of the connection is smaller than the sum of the single fastener ultimate loads and this is known as "group effect".

For one row of fasteners parallel to the grain direction, the effective characteristic load-carrying capacity parallel to the row, $F_{v.ef.Rk}$, should be taken as:

$$F_{v.ef.Rk} = n_{ef.row} \, F_{v.Rk}$$

where:

$n_{ef.row}$ is the effective number of fasteners in line parallel to the grain.

$F_{v.Rk}$ is the characteristic load-carrying capacity of each fastener parallel to the grain.

Serviceability limit state: Joint slip

The design of any structural timber project engineer will have to combine the global analysis along with the local analysis of the connection. The principal factor is the joint behaviour that has an effect on the distribution of forces and overall deformation of the structure. The load slope can be calculated in accordance with EC5, or determined from test results for the chosen connections in accordance with EN 26891 [255].

In contrast with rigidly glued joints, mechanical fasteners exhibit large deformations that must be considered by the designer.

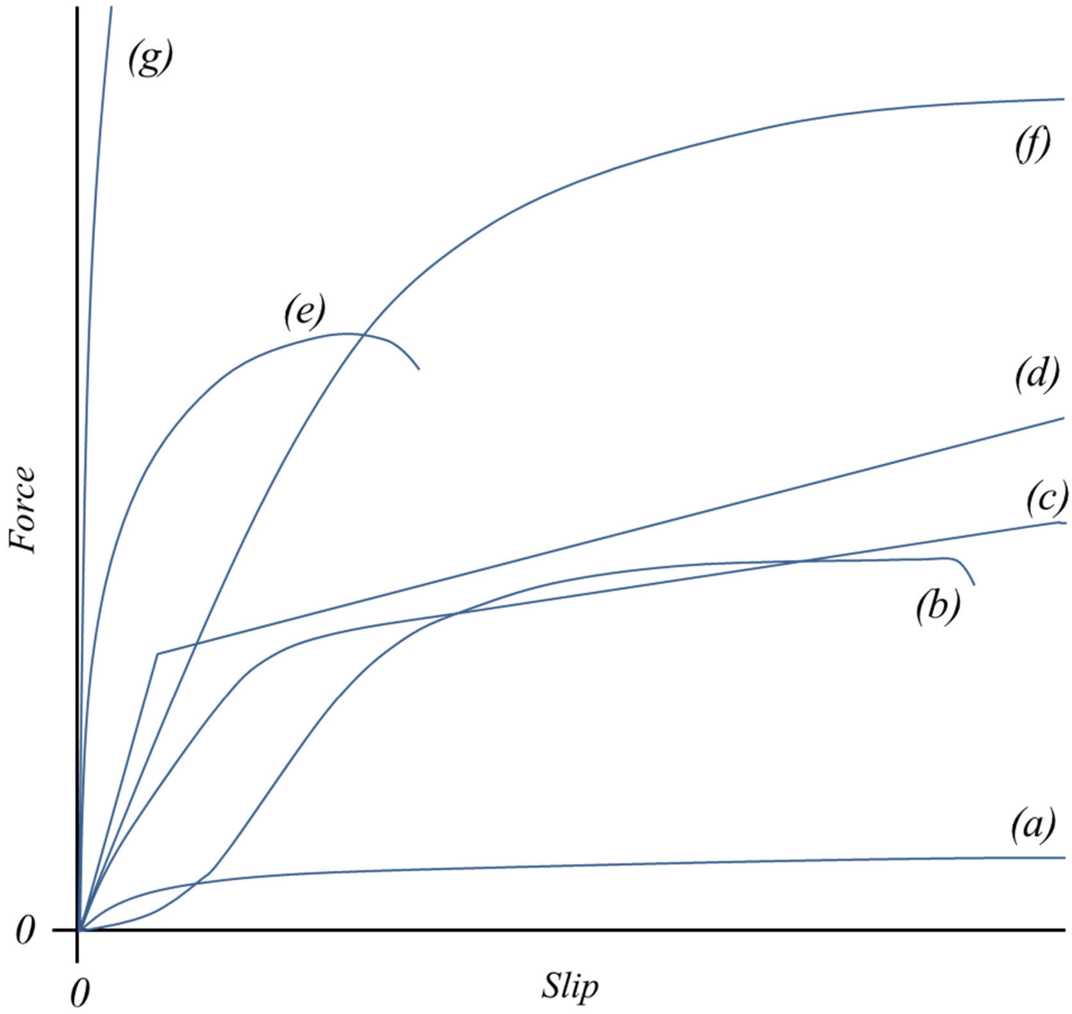


Figure D- 14 Load slip comparison, This graft was recreated from [245]

where:

a. Nail; b. Bolt; c. Dowel; d. Double side toothed-plate; e. Punched plate; f. Split-ring; g. Glued joint.

Note from the graph: the initial slip of the bolted connection is the result of the oversized holes.

Calculating the load slip for dowel-type fasteners in accordance with EC5.

The final deformation of joint, a conservative approach; $ul_{fin} = F_{v.Ed} / K_{ser.fin.c}$

where:

K_{ser} is the slip modulus from Table D-3, or determined from test results

$K_{ser.fin}$ is the final mean value of slip / lateral stiffness per fixing.

$K_{ser.fin.c}$ is the Lateral stiffness per connection

TNF_c	is the total number of fixings in the connection
$k_{def.m}$	is the deformation factor, from Table D-4, (ED5 table 3.2)
$k_{def.c}$	is the deformation factor for each connection
$F_{v.Ed}$	is the design action

$$K_{ser.fin} = K_{ser} / (1 + k_{def.c})$$

$$K_{ser.fin.c} = K_{ser.fin} \cdot TNF_c$$

$$k_{def.c} = 2 \cdot \sqrt{k_{def.m1} \cdot k_{def.m2}}$$

Load slip is a function of the mean density of timber and the diameter of the fixing.

Table D-3 Values of Kser for fasteners in timber-to-timber and wood-based panel-to-timber connections (EC5 Table 7.1)

Fastener type	K_{ser} (N/mm)
Nails without predrilling Small wood screws ($d \leq$ 6mm) without predrilling	$\rho_m^{1.5} d^{0.8}/30$
Wood screws with predrilling Bolts. (Clearance to be added separately.) Dowels	$\rho_m^{1.5} d/23$
Split ring, shear plate and Toothed-plate type C10 and C11 connectors	$\rho_m d/2$
Toothed-plate connectors: type C 1 to C9	$1.5\ \rho_m\ d / 4$

Notes:

ρ_m = mean density of timber (see Tables 3.14 to 3.18)

d = diameter of round nail or side length of square nail, nominal diameter of screw, diameter of bolt or dowel, or nominal diameter or side length of a timber connector (see BS EN 13271[#6.13]).

For bolts the clearance ($d_{hole} - d_{bolt}$) should be added to the calculated slip.

If the mean densities $\rho_{m,1}$ and $\rho_{m,2}$ of two connected wood-based members differ then $\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}}$.

For steel-to-timber or concrete-to-timber connections use ρ_m for the timber member and multiply K_{ser} by 2.

Table D-4 Values of k_def for timber and wood-based materials

Material	Service class		
	1	2	3
Solid timber, Glued laminated timber, LVL	0.60	0.80	2.00
Plywood	0.80	1.00	2.50
OSB/2	2.25	-	-
OSB/3, OSB/4	1.50	2.25	-

Appendix E. Example outputs of existing timber connection software

Trada

timberconnectionsPro EC5 edition:

Timber-Timber - Smooth Nail

TRADA, Stocking Lane, Hughenden Valley, High Wycombe, HP14 4ND
e: information@trada.co.uk w: www.trada.co.uk

Company Details:

Project:		**Project Ref:**	
Client:		**Client Contact:**	
Version:		**Report No:**	267893706

Created by:	TRADA member	**Checked:**		**Approved:**	
Date:	10th January 2018	**Date:**		**Date:**	

Design Data

Service Class **- 1**

Fastener **- Smooth Nail**

Type: Round **Diameter:** 3.35 mm **Head diameter:** 10 mm **Length:** 74 mm **Tensile strength:** 600 N/mm^2

Members	**headside - Timber**	**pointside - Timber**
Thickness:	47 mm	47 mm
Strength class:	C24	C24
Angle of load to grain:	0°	0°
Pre-drilled:	no	no
Density:	350 kg/m^3	350 kg/m^3
Yield strength:	n/a	n/a
$f_{c,90,k}$	2.50 N/mm^2	2.50 N/mm^2
Embedment strength:	19.97 N/mm^2	19.97 N/mm^2
k_{90}	1.40025	1.40025

Factors

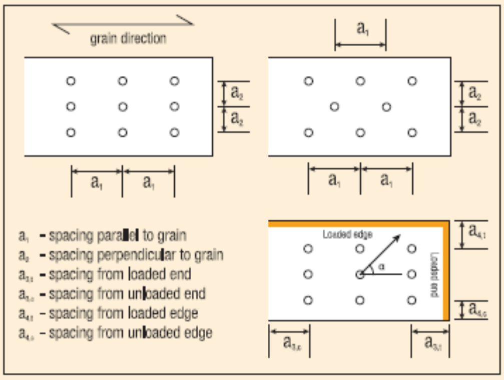
$k_{mod,perm}$ = 0.60	$k_{mod,st}$ = 0.90	$M_{y,R,k}$ = 4172.43 Nmm	**Axial withdrawal characteristic resistance:** = 2.45 N/mm^2
$k_{mod,lt}$ = 0.70	$k_{mod,inst}$ = 1.10	$F_{ax,R,k}$ = 3.31 N	**Head pull through characteristic resistance:** = 8.57 N/mm^2
$k_{mod,mt}$ = 0.80	γ_M = 1.30	**Rope-effect:** = 15%	

Results

Fastener capacities

Permanent:	362 N
Long term:	422 N
Medium term:	483 N
Short term:	544 N
Instantaneous:	665 N

Spacings	**headside**	**pointside**
a_1	34 mm	34 mm
a_2	17 mm	17 mm
$a_{3,t}$	51 mm	51 mm
$a_{3,c}$	34 mm	34 mm
$a_{4,t}$	17 mm	17 mm
$a_{4,c}$	17 mm	17 mm



EC5 timberConnections V1 incorporating amendments in 2010 and 2013 © TRADA Technology Ltd (Now Exova (UK) Ltd). 2010

Whilst every effort is made to ensure the accuracy of the information, given that every design varies, Exova (UK) Ltd. cannot accept liability for loss or damage arising from the information supplied. Please see terms and conditions of use.

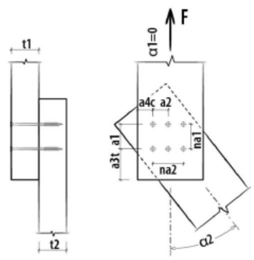
Rothoblaas

myProject calculation software by rothoblaas

PROJECT INFORMATION

Date	:	11/01/2018
Project	:	
Client	:	
Project address	:	
Edited by	:	
Joint	:	
Notes	:	
Code of calculation	:	EN1995:2014 (EU)

SHEAR CONNECTION WITH SCREWS (Timber-to-timber connection / single shear plane)



- Screw type HBS - Countersunk head screw 4x70 mm - (code HBS470)
- Number of screws: 2 x 2 = 4 pcs

CE marking according to ETA 11/0030

CALCULATION DATA

Timber-to-timber connection / single shear plane

Service class	cl	=	2
Main load duration	tq	=	medium
kmod factor	kmod	=	0.80
Connection safety factor	γM	=	1.30
Nominal diameter/thread screw	d1	=	4.0 mm
Shank diameter	ds	=	2.8 mm
Inner core diameter	d2	=	2.6 mm
Head diameter	dk	=	8.0 mm
Screw length	Lv	=	70 mm
Thread length	Lf	=	40 mm
Wood thickness element 1	t1	=	45 mm
Angle element 1	α1	=	0.00°
Wood quality element 1		=	Glulam GL24h (homogeneous)
Wood thickness element 2	t2	=	45 mm
Angle element 2	α2	=	0.00°
Wood quality element 2		=	Glulam GL24h (homogeneous)
Number of elements parallel to the grain	nf	=	2
Distance of elements parallel to the grain	a1	=	40 mm
Number of elements perpendicular to the grain	nc	=	2
Distance of elements perpendicular to the grain	a2	=	40 mm
Action of shear design	Fvd	=	0.00 KN

NOTES

Before the construction, all calculation must be verified and approved by the responsible designer
Mechanical resistance values and geometry refer to product certification
Verification of timber elements resistance must be realized apart.

CALCULATION RESULTS

INPUT DATA:			
Service class	cl	=	2
Duration of main load	tq	=	medium
kmod factor	kmod	=	0.80
Safety factor of connection	γM	=	1.3
Timber type element t1		=	GL24h
Timber volumetric mass	ρk	=	385 Kg/m³
Timber type element t2		=	GL24h
Timber volumetric mass	ρk	=	385 Kg/m³
Steel safety factor	γMa	=	1.25
Thickness element 1	t1	=	45 mm
Thickness element 2	t2	=	45 mm
Angle element 1	α1	=	0.00 °
Angle element 2	α2	=	0.00 °
Number of rows screws	na1	=	2
Distance of rows	a1	=	40 mm
Number of columns screws	na2	=	2
Distance of columns	a2	=	40 mm
SCREW DATA			
HBS - Countersunk head screw 4x70			
Shank diameter of screw	dg	=	2.8 mm
Thread diameter of screw	df	=	4.0 mm
Inner core diameter of screw	dn	=	2.6 mm
Conventional diameter according to EN1995:2014	def=df	=	4.0 mm
Thread length of screw	lf	=	40 mm
Length of screw	lh	=	70 mm
Insertion angle (screw - grain)	β	=	90.00 °
Without pre-drilling hole		=	
Not staggered		=	
Screw head diameter	dh	=	8.0 mm
RESULTS:			
Penetration depth element 1	Lp1	=	45 mm
Penetration depth element 2	Lp2	=	25 mm
Steel ultimate tensile strength	ftens,k	=	5000 N
Effective withdrawal thread length (tip side)		=	25 mm
Withdrawal thread resistance (tip side)	Fax,rk	=	1263 N
Thread length (head side)		=	15 mm
Withdrawal thread resistance (head side)	Fax,rk	=	758 N
Characteristic headside pull-through strength	Fhead,rk	=	725 N
Effective resistance head side	max Fax,rk	=	758 N
Characteristic embedment strength element 1	Fh,1,k	=	20.83 N/mm²
Characteristic embedment strength element 2	Fh,2,k	=	20.83 N/mm²
Yield strength steel	Myk	=	3033 Nmm
Effective number of screws parallel to grain element 1	nef	=	1.80
Effective number of screws parallel to grain element 2	nef	=	1.80
Effective number of screws parallel to grain	nef	=	1.80
MINIMUM DISTANCES element 1 (wood):			
Parallel to grain	a1	=	40 mm
Perpendicular to grain	a2	=	20 mm
From unloaded end (// grain)	a3c	=	40 mm
From loaded end (// grain)	a3t	=	60 mm
From unloaded edge (perp. grain)	a4c	=	20 mm
From loaded edge (perp. grain)	a4t	=	20 mm
MINIMUM DISTANCES element 2 (wood):			
Parallel to grain	a1	=	40 mm
Perpendicular to grain	a2	=	20 mm
From unloaded end (// grain)	a3c	=	40 mm
From loaded end (// grain)	a3t	=	60 mm
From unloaded edge (perp. grain)	a4c	=	20 mm
From loaded edge (perp. grain)	a4t	=	20 mm
VALUES OF RESISTANCE:			
Number of shear planes	nT	=	1
Withdrawal contribution determined with Johansen	Fax,Rk/4	=	0.17 KN

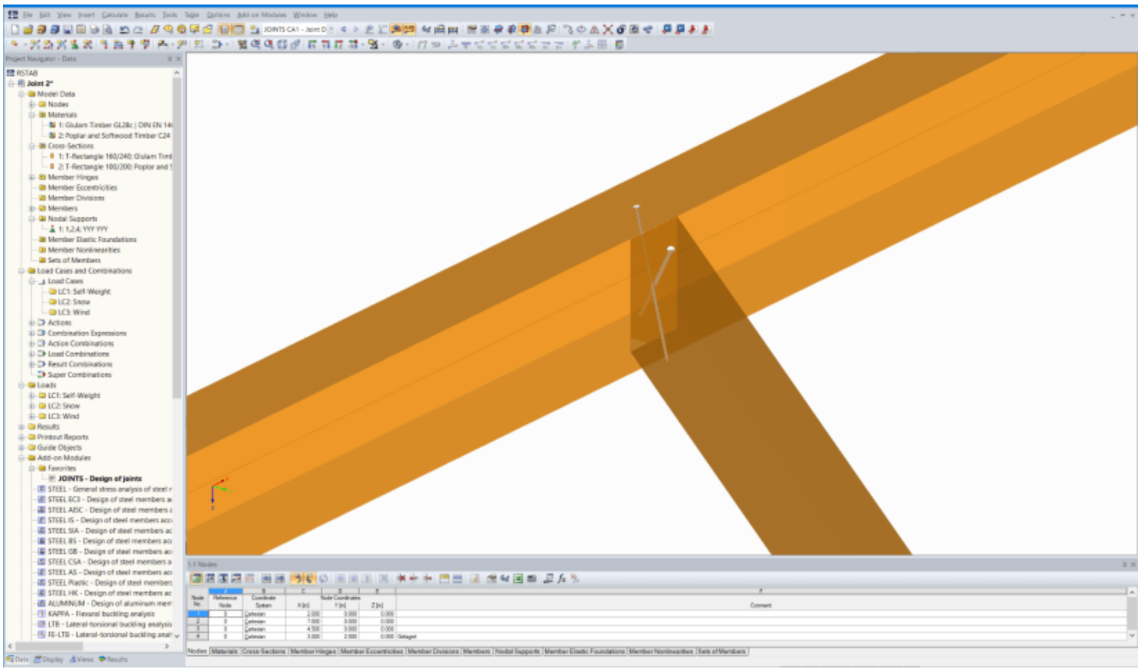
Shear characteristic resistance mode a (element t1)	Fv,Rk	=	3.75 KN
Shear characteristic resistance mode b (element t2)	Fv,Rk	=	2.08 KN
Shear characteristic resistance mode c (element t2)	Fv,Rk	=	1.46 KN
Shear characteristic resistance mode d (element t1)	Fv,Rk	=	1.55 KN
Shear characteristic resistance mode e (element t2)	Fv,Rk	=	1.02 KN
Shear characteristic resistance mode f (element t2)	Fv,Rk	=	0.98 KN
Shear characteristic resistance screws for shear plane (element t2)	Fv,Rk	=	0.98 KN
Shear characteristic resistance screws			0.98 KN
Shear design resistance screws for shear plane	Fv,Rd	=	0.60 KN
Shear design resistance screws			0.60 KN
Shear design resistance of single screws with effective number and withdrawal contribution			0.55 KN
Global shear design resistance of whole connection			2.22 KN
Effective withdrawal number			3.48
Withdrawal characteristic resistance of single fastener			0.76 KN
Withdrawal characteristic resistance of whole connection			2.64 KN
Withdrawal design resistance of whole connection			1.62 KN
Single fastener displacement for shear plane			1.50 KN/mm
Global shear design resistance of whole connection	**Fv,rd,tot**	**=**	**2.22 KN**
Withdrawal design resistance of whole connection	**Faxdtot,ef**	**=**	**1.62 KN**
Single fastener displacement for shear plane	**Kser**	**=**	**1.50 KN/mm**

Dlubal

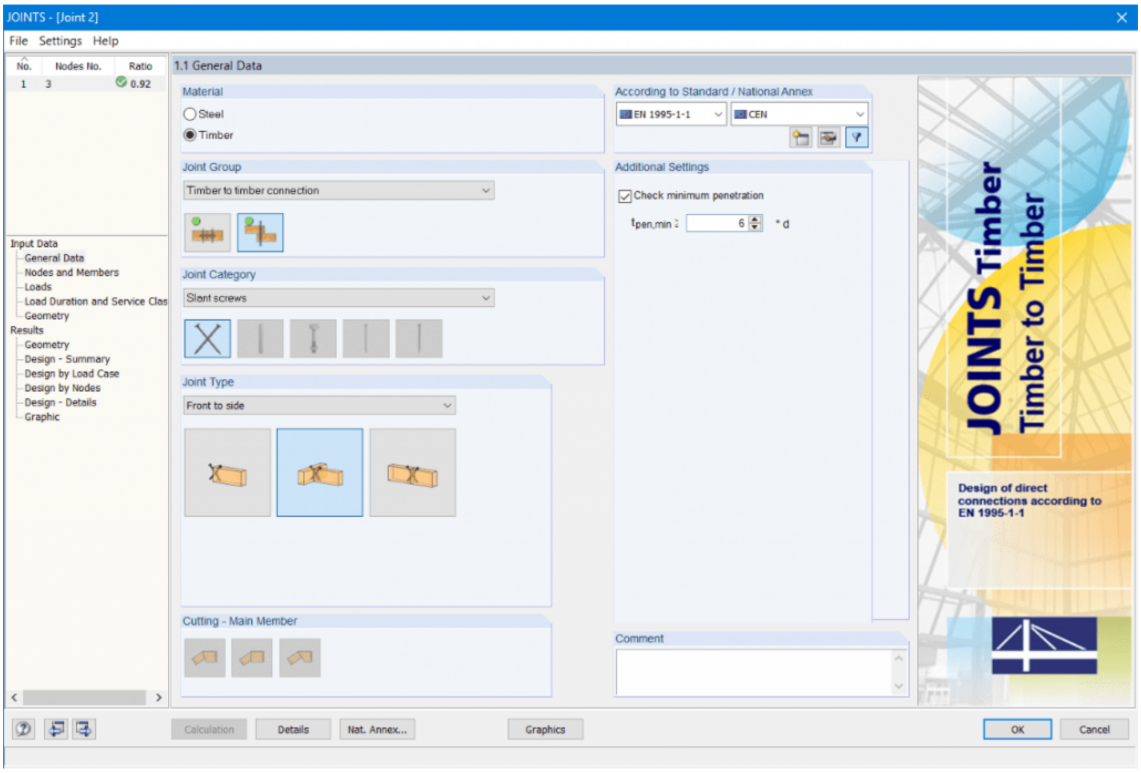
Example output from the Dlubal timber to timber connection calculation add-in. the Date 23rd October 2017

https://bimsolutions.lv/rf-joints-timber-timber-timber-add-module-rfem-rstab/

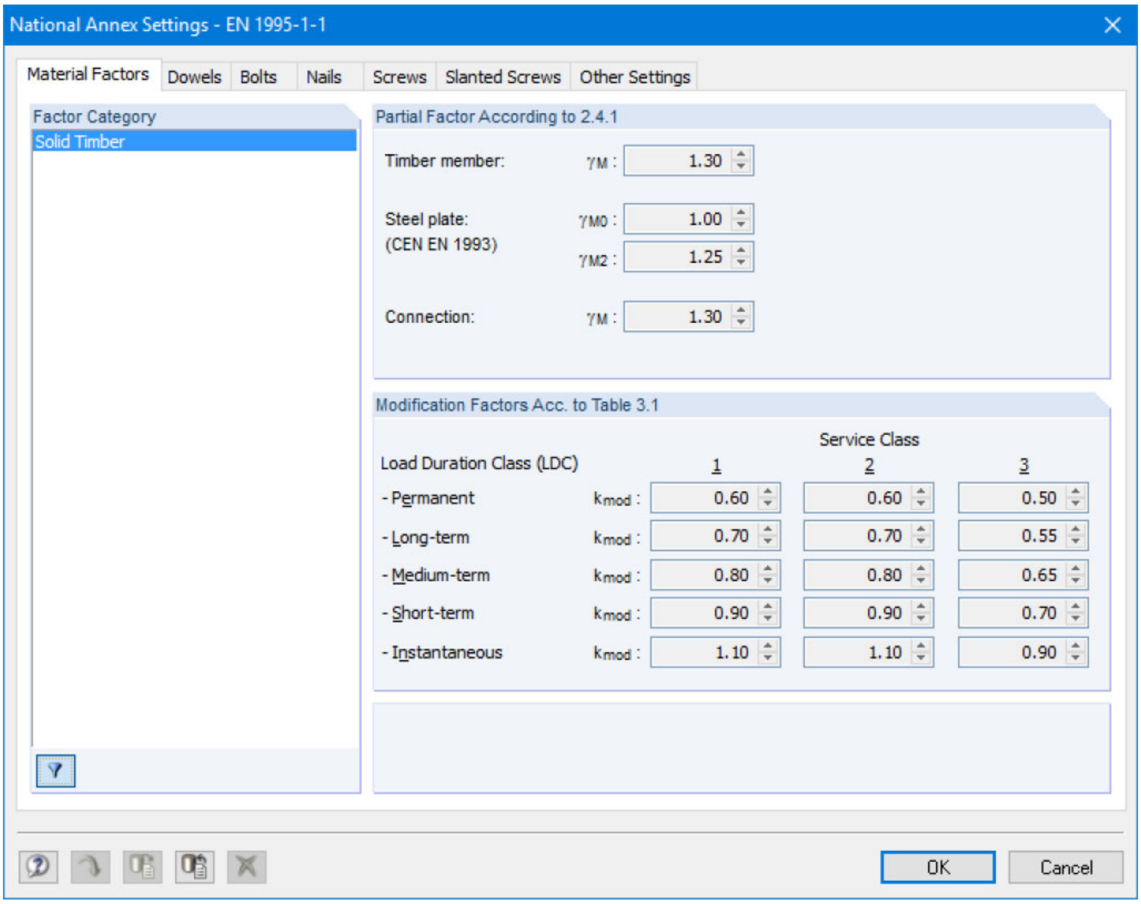
RF-/JOINTS Timber – Timber to Timber Add-on Module for RFEM/RSTAB



Input

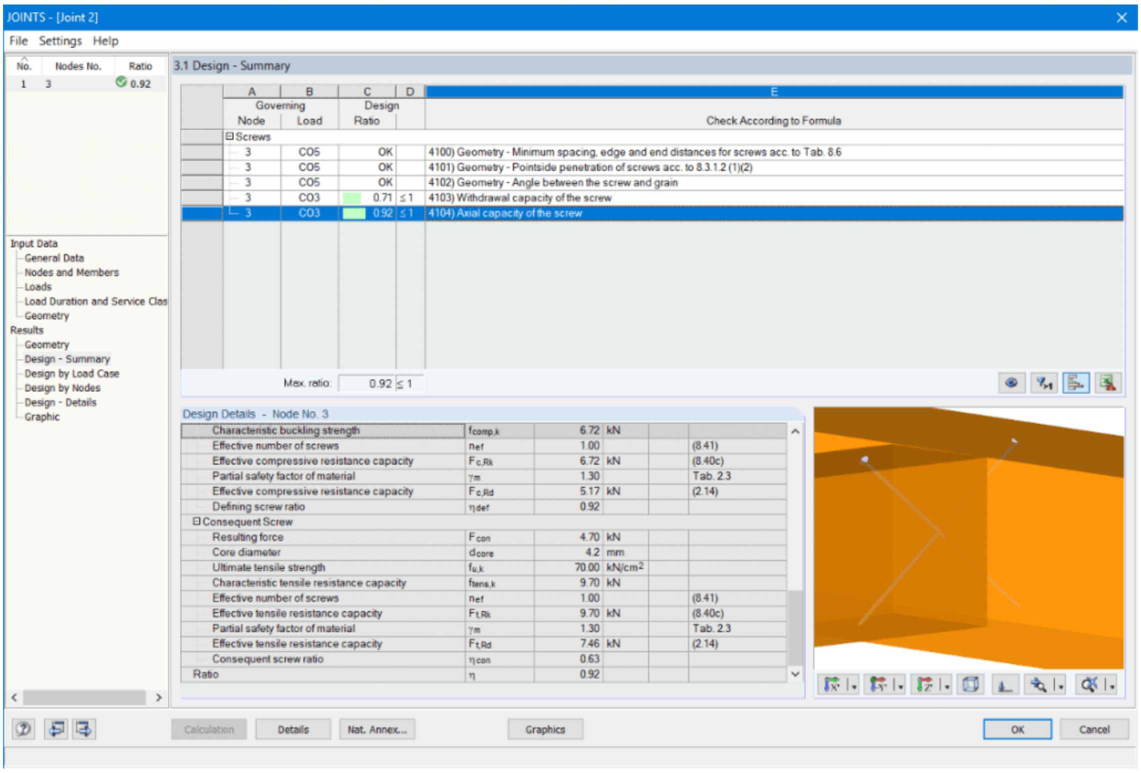


Design



National Annex Settings - EN 1995-1-1
Material Factors
Dowels
Bolts
Nails
Screws
Slanted Screws
Other Settings
Factor Category
Solid Timber
Partial Factor According to 2.4.1
Timber member: γM : 1.30
Steel plate: (CEN EN 1993) γM0 : 1.00 γM2 : 1.25
Connection: γM : 1.30
Modification Factors Acc. to Table 3.1
Service Class
Load Duration Class (LDC) 1 2 3
- Permanent kmod : 0.60 0.60 0.50
- Long-term kmod : 0.70 0.70 0.55
- Medium-term kmod : 0.80 0.80 0.65
- Short-term kmod : 0.90 0.90 0.70
- Instantaneous kmod : 1.10 1.10 0.90
OK
Cancel

Results

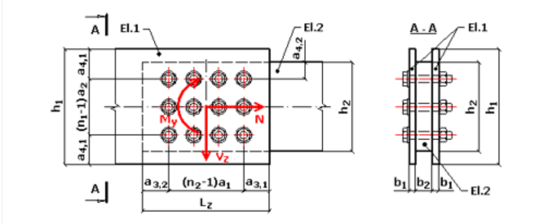


JOINTS - [Joint 2]
File Settings Help
No. Nodes No. Ratio
1 3 0.92
3.1 Design - Summary
Governing Node Load
Design Ratio
Check According to Formula
Screws
3 CO5 OK 4100) Geometry - Minimum spacing, edge and end distances for screws acc. to Tab. 8.6
3 CO5 OK 4101) Geometry - Pointside penetration of screws acc. to 8.3.1.2 (1)(2)
3 CO5 OK 4102) Geometry - Angle between the screw and grain
3 CO3 0.71 ≤ 1 4103) Withdrawal capacity of the screw
3 CO3 0.92 ≤ 1 4104) Axial capacity of the screw
Max. ratio: 0.92 ≤ 1
Input Data
General Data
Nodes and Members
Loads
Load Duration and Service Clas
Geometry
Results
Geometry
Design - Summary
Design by Load Case
Design by Nodes
Design - Details
Graphic
Design Details - Node No. 3
Characteristic buckling strength 6.72 kN
Effective number of screws 1.00 (8.41)
Effective compressive resistance capacity 6.72 kN (8.40c)
Partial safety factor of material 1.30 Tab. 2.3
Effective compressive resistance capacity 5.17 kN (2.14)
Defining screw ratio 0.92
Consequent Screw
Resulting force 4.70 kN
Core diameter 4.2 mm
Ultimate tensile strength 70.00 kN/cm2
Characteristic tensile resistance capacity 9.70 kN
Effective number of screws 1.00 (8.41)
Effective tensile resistance capacity 9.70 kN (8.40c)
Partial safety factor of material 1.30 Tab. 2.3
Effective tensile resistance capacity 7.46 kN (2.14)
Consequent screw ratio 0.63
Ratio 0.92
Calculation
Details
Nat. Annex...
Graphics
OK
Cancel

BIMware

Connection type: Splice connection, timber - timber	Office: Napier cocis	Author: Andrew Livingstone
Standard: EN 1995-1-1	Date: 4/18/2019	Project: Thesis
Connection name:	For:	

Scheme (parameter view):



Data:

Internal forces (design value):				
axial force	$N =$	1.00	[kN]	
shear force	$V_z =$	0.00	[kN]	
bending moment	$M_y =$	0.00	[kN·m]	
Working conditions:				
load-duration class	medium-term			[p.2.3.1.2]
service class	2			[p.2.3.1.3]
Modification factors:				
working conditions factor for connection	$k_{mod} =$	0.80	[-]	[tab.3.1]
length factor for El.i, (i=1,2)	$k_{l,i} = min\left[\left(\frac{3000}{l_{d,i}}\right)^{\frac{s_i}{2}}; 1{,}1\right]$		for LVL	[eq.3.4]
	$k_{l.1} =$	1.00	[-]	
	$k_{l.2} =$	1.00	[-]	
width factor for El.i, (i=1,2)	$k_{h,i} = min\left[\left(\frac{150}{b_i}\right)^{0.2}; 1{,}3\right]$		for solid timber	[eq.3.1]
	$k_{h,i} = min\left[\left(\frac{600}{b_i}\right)^{0.1}; 1{,}1\right]$		for glued laminated timber	[eq.3.2]
	$k_{h.1} =$	1.30	[-]	
	$k_{h.2} =$	1.30	[-]	

Elements:				
element El.1 (timber)	**Softwood C16 EN 338**			
timber class	C16			
thickness	$b_1 =$	38.0	[mm]	
depth	$h_1 =$	145.0	[mm]	
length	$l_{d.1} =$	1000.0	[mm]	
branche number	$l_1 =$	1	[-]	
characteristic wood density	$\rho_{k.1} =$	310.00	[kg/m3]	
characteristic tensile strength along the grain	$f_{t0.k.1} =$	10.00	[MPa]	
characteristic compressive strength perpendicular to grain	$f_{c90.k.1} =$	2.20	[MPa]	
characteristic shear strength	$f_{vk.1} =$	3.20	[MPa]	
size effect exponent	$s_1 =$	0.00	[-]	
element El.2 (timber)	**Softwood C16 EN 338**			
timber class	C16			
thickness	$b_2 =$	38.0	[mm]	
depth	$h_2 =$	145.0	[mm]	
length	$l_{d.2} =$	1000.0	[mm]	
branche number	$l_2 =$	1	[-]	
characteristic wood density	$\rho_{k.2} =$	310.00	[kg/m3]	
characteristic tensile strength along the grain	$f_{t0.k.2} =$	10.00	[MPa]	
characteristic compressive strength perpendicular to grain	$f_{c90.k.2} =$	2.20	[MPa]	
characteristic shear strength	$f_{vk.2} =$	3.20	[MPa]	
size effect exponent	$s_2 =$	0.00	[-]	

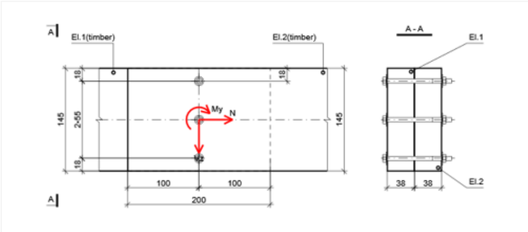
Fasteners:				
bolts	**M6, class 8.8**			
bolt diameter	$d =$	6.0	[mm]	
minimum bolt length	$l_{min} =$	90.0	[mm]	
bolt length	$l =$	90.0	[mm]	
area of bolt in tension	$A_{b.t} =$	20.10	[m2]	
washer area	$A_w =$	346.00	[m2]	
characteristic tensile strength of bolt	$f_{uk} =$	800.00	[MPa]	
shear plane number	$n_{sp} =$	1	[-]	
Partial safety factors:				
material properties partial factor for El.1	$\gamma_{M.d.1} =$	1.30	[-]	[tab.2.3]
material properties partial factor for El.2	$\gamma_{M.d.2} =$	1.30	[-]	[tab.2.3]
material properties partial factor for connection	$\gamma_{M.conn} =$	1.30	[-]	[tab.2.3]
material properties partial factor for bolt	$\gamma_{M.s} =$	1.10	[-]	[tab.2.3]
Connection parameters:				
fastener pattern method	rectangular			
lap length	$L_z =$	200.0	[mm]	
headside member thickness	$t_1 =$	38.0	[mm]	
central member thickness	$t_2 =$	38.0	[mm]	

Minimum values of fastener spacing, edge and end distances:

Minimum values of fastener spacing, edge and end distances in El.1:				[tab.8.4]
min. fastener spacing parallel to grain	$\min a_1 =$	30.0	[mm]	
min. fastener spacing perpendicular to grain	$\min a_2 =$	24.0	[mm]	
min. end distance	$\min a_{3.1} =$	80.0	[mm]	
min. edge distance	$\min a_{4.1} =$	18.0	[mm]	
max. fastener spacing parallel to grain	$\max a_1 =$	40.0	[mm]	
max. fastener spacing perpendicular to grain	$\max a_2 =$	54.5	[mm]	
Minimum values of fastener edge and end distances in El.2:				[tab.8.4]
minimum value of fastener end distance	$\min a_{3.2} =$	80.0	[mm]	
minimum value of fastener edge distance	$\min a_{4.2} =$	18.0	[mm]	

Maximum fastener **M6** number in the connection - **10** bolts, in **2** rows perpendicular to grain and **5** rows parallel to grain.

Fastener arrangement:



Fastener number:						
fastener row number (parallel to grain)	$\min n_1 =$	1	$\le n_1 =$	3	$\le \max n_1 =$	5
fastener row number (perpendicular to the grain)	$\min n_2 =$	1	$\le n_2 =$	1	$\le \max n_2 =$	2
total fastener number in the connection	$\min n =$	1	$\le n =$	3	$\le \max n =$	10

Fastener spacing and distances [mm]:

fastener spacing perpendicular to the grain	$\min a_2 =$	24.0	$\leq a_2 =$	**54.5**	$\leq \max a_2 =$	54.5
fastener end distance for El.1	$\min a_{3,1} =$	80.0	$\leq a_{3,1} =$	**100.0**		
fastener edge distance for El.1	$\min a_{4,1} =$	18.0	$\leq a_{4,1} =$	**18.0**		
fastener end distance for El.2	$\min a_{3,2} =$	80.0	$\leq a_{3,2} =$	**100.0**		
fastener edge distance for El.2	$\min a_{4,2} =$	18.0	$\leq a_{4,2} =$	**18.0**		

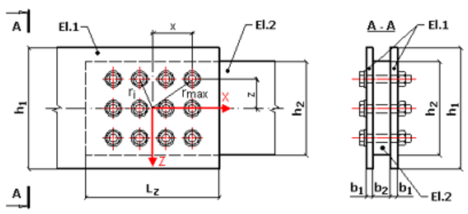
<u>Connection code check:</u>

<u>Bolt load-carrying capacity check:</u>

Calculation of additional bending moment due to bolt gravity center displacement:

bolt gravity center displacement along the X axis		**0.0**	[mm]
bolt gravity center displacement along the Z axis	$e_z =$	**0.0**	[mm]
additional bending moment $M_{y,ad}$	$M_{y,ad} = -N \cdot e_z - V_z \cdot e_x =$	**0.00**	[kN·m]

Calculation of resultant force acting on maximum loaded bolt (per shear plane):



horizontal force acting on each bolt due to horizontal load **N**	$F_{h,d} = \frac{N}{n_{sp} \cdot n} =$	**0.33**	[kN]
vertical force acting on each bolt due to vertical load V_z	$F_{v,d} = \frac{V_z}{n_{sp} \cdot n} =$	**0.00**	[kN]
force acting on each bolt due to moment M_y		**0.00**	[kN]
radial distance from max. loaded bolt to the centre of rotation	$r_{max} = \sqrt{x^2+z^2} =$	**54.5**	[mm]
radial distance from each bolt –i to the centre of rotation	$r_i = \sqrt{x_i^2+z_i^2}$		[mm]
sum of the squares of the radial distances	$\sum_{i=1}^{n} r_i^2 =$	**5940.50**	[m2]
resultant force acting on maximum loaded bolt	$F_d = \sqrt{\left(F_{v,d} + F_{m,d,max} \cdot \frac{x}{r_{max}}\right)^2 + \left(F_{h,d} + F_{m,d,max} \cdot \frac{z}{r_{max}}\right)^2}$		
	$F_d =$	**0.33**	[kN]
angle of Fd to the grain in the timber			
	$\alpha_{Fd} =$	**0.00**	[-]

alculation of bolt load-carrying capacity:

Embedment strength of the timber:				[p.8.5]
characteristic embedment strength parallel to the grain in timber member -i (i=1,2)	$f_{h,0,i,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot d) \cdot \rho_{k,i}$			[eq.8.32]
characteristic embedment strength in timber member -i (i=1,2)	$f_{h,i,k} = \frac{f_{h,0,i,k}}{k_{i,90} \cdot \sin^2 \alpha_{Fd} + \cos^2 \alpha_{Fd}}$			[eq.8.31]
embedment factor for timber member -i, (i=1,2)	$k_{i,90} = 1{,}35 + 0{,}015 \cdot d$	*for softwood*		[eq.8.33]
	$k_{i,90} = 1{,}30 + 0{,}015 \cdot d$	*for LVL*		[eq.8.33]
	$k_{i,90} = 0{,}90 + 0{,}015 \cdot d$	*for hardwood*		[eq.8.33]
embedment factor for timber member with thickness t_1	$k_{1,90} =$	**1.44**	[-]	
embedment factor for timber member with thickness t_2	$k_{2,90} =$	**1.44**	[-]	
characteristic embedment strength parallel to the grain in timber member with thickness t_1	$f_{h,0,1,k} =$	**23.89**	[MPa]	
characteristic embedment strength in timber member with thickness t_1	$f_{h,1,k} =$	**23.89**	[MPa]	
characteristic embedment strength parallel to the grain in timber member with thickness t_2	$f_{h,0,2,k} =$	**23.89**	[MPa]	
characteristic embedment strength in timber member with thickness t_2		**23.89**	[MPa]	
ratio of the embedment strength of member 2 to member 1	$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} =$	**1.00**	[-]	[eq.8.8]
characteristic yield moment of a bolt	$M_{y,Rk} = 0{,}3 \cdot f_{u,k} \cdot d^{2,6}$			[eq.8.30]
	$M_{y,Rk} =$	**0.03**	[kN·m]	

Calculation of bolt withdrawal resistance:				
tensile strength of the bolt	$F_{1,ax,Rk} = f_{u,k} \cdot A_{b,t}$			
	$F_{1,ax,Rk} =$	**16.08**	[kN]	
bearing capacity of the washer				[p.8.5.2(2)]
	$F_{2,ax,Rk} =$	**2.28**	[kN]	
characteristic axial withdrawal capacity of the bolt	$F_{ax,Rk} = \min(F_{1,ax,Rk} ; F_{2,ax,Rk})$			[p.8.5.2(1)]
	$F_{ax,Rk} =$	**2.28**	[kN]	

The contribution to the load-carrying capacity due to the rope effect: [p.8.2.2(2)]

$$if\ \frac{\frac{F_{ax,Rk}}{4}}{F_{v,Rk,i} - \frac{F_{ax,Rk}}{4}} \cdot 100 > 25\% \quad then\ F_{v,Rk,i} = 1{,}25 \cdot \left(F_{v,Rk,i} - \frac{F_{ax,Rk}}{4}\right) ; i = (c,d,e,f)$$

Calculation of the bolt load-carrying capacity: [p.8.2.2]

failure mode *a*	$F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d =$	**5.45**	[kN]	[eq.8.6.a]
failure mode *b*	$F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d =$	**5.45**	[kN]	[eq.8.6.b]

failure mode *c*

$$F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta^3 \cdot \left(\frac{t_2}{t_1}\right)^2} - \beta \cdot \left(1 + \frac{t_2}{t_1}\right)\right] + \frac{F_{ax,Rk}}{4} \quad [eq.8.6.c]$$

$F_{v,Rk,c} =$ **2.82** [kN]

failure mode *d*

$$F_{v,Rk,d} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right] + \frac{F_{ax,Rk}}{4} \quad [eq.8.6.d]$$

$F_{v,Rk,d} =$ **3.12** [kN]

failure mode *e*

$$F_{v,Rk,e} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2 \cdot \beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (1+2 \cdot \beta) \cdot M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta\right] + \frac{F_{ax,Rk}}{4} \quad [eq.8.6.e]$$

$F_{v,Rk,e} =$ **3.12** [kN]

failure mode ***f***

$$F_{v,Rk,f} = 1{,}15 \cdot \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \quad [eq.8.6.f]$$

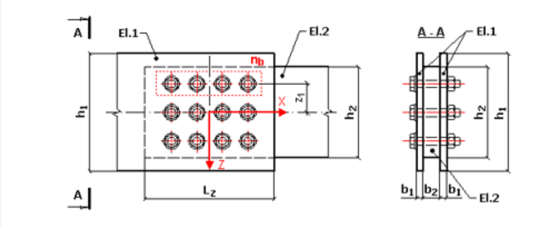
$F_{v,Rk,f} =$ **3.67** [kN]

characteristic load-carrying capacity per shear plane per bolt	$F_{v,Rk} = \min F_{v,Rk,i} =$	**2.82**	[kN]	[p.8.2.2(1)]
design load-carrying capacity per shear plane per bolt	$F_{v,Rd} = \frac{k_{mod} \cdot F_{v,Rk}}{\gamma_{M,conn}} =$	**1.74**	[kN]	[eq.2.17]

Bolt load-carrying capacity condition (for F_d force acting at the α_{Fd} angle to the grain):

$F_d =$	0.33	[kN]	$< F_{v,Rd} =$	1.74	[kN]	*VERIFIED*
efficiency ratio			$w_1 = \frac{F_d}{F_{v,Rd}} =$	0.19	[-]	

Force component checks in a row of bolts parallel to the grain:



vertical distance to extreme bolt row	$z_1 =$	54.50	[mm]
design force component per bolt per shear plane parallel to the grain in each of the bolts in the row	$F_{d,H,max} = \frac{(M_y + M_{y,ad}) \cdot z_1}{n_{sp} \cdot \sum_{i=1}^{n} r_i^2} + \frac{N}{n_{sp} \cdot n} =$	0.33	[kN]

Calculation of the bolt load-carrying capacity, parallel to the grain:

characteristic load-carrying capacity per shear plane per bolt, parallel to the grain	$F_{v,Rk,H} =$	2.82	[kN]
number of bolts per shear plane in the external row		1	[-]
effective number of bolts per shear plane in the row	$n_{ef} = min\left(n_b ; n_b^{0.9} \cdot \sqrt[4]{\frac{a_1}{13 \cdot d}}\right) =$	1.00	[-]
design load-carrying capacity per shear plane per bolt, parallel to the grain	$F_{v,Rd,H} = \frac{n_{ef}}{n_b} \cdot \frac{k_{mod}}{\gamma_{M,conn}} \cdot F_{v,Rk,H} =$	1.74	[kN]

Bolt load-carrying capacity condition (for $F_{d,H,max}$ force parallel to the grain):

$F_{d,H,max} =$	0.33	[kN]	< $F_{v,Rd,H}$ =	1.74	[kN]	VERIFIED
efficiency ratio		$w_2 = \frac{F_{d,H,max}}{F_{v,Rd,H}} =$		0.19	[-]	

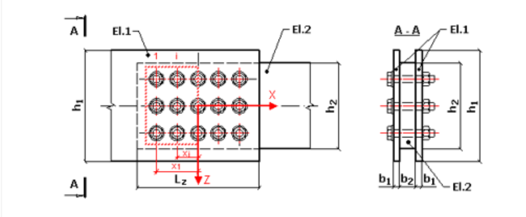
<u>Element El.1 load-carrying capacity check:</u>

Tensile stress along the grain check:

El.1 cross-section net area	$A_{n,1} =$	4712.00	[m2]
tensile stress along the grain	$\sigma_{t,0,d,1} = \frac{N}{A_{n,1}} =$	0.21	[MPa]
design tensile strength along the grain	$f_{t,0,d,1} = \frac{k_{mod} \cdot k_{h,1} \cdot k_{l,1} \cdot f_{t,0,k,1}}{\gamma_{M,d,1}} =$	6.15	[MPa]

$\sigma_{t,0,d,1} =$	0.21	[MPa]	< $f_{t,0,d,1}$ =	6.15	[MPa]	VERIFIED
efficiency ratio		$w_3 = \frac{\sigma_{t,0,d,1}}{f_{t,0,d,1}} =$		0.03	[-]	

Shear stress check:



Maximum shear force in a bolt line(perpendicular to the grain) calculation:

shear force in the external bolt line	$F_{v,Vd,1} = n_{r,1} \cdot \left(\frac{n_{sp} \cdot F_{m,d,max}}{r_{max}} \cdot \lvert x_1 \rvert - \frac{1}{n} \cdot V_z\right)$		
bolt number in the external bolt line	$n_{r,1} =$	3	[-]
external bolt line distance from fastener centroid	$x_1 =$	0.00	[mm]
	$F_{v,Vd,1} =$	0.00	[kN]
shear force in the bolt line - i; (i=2÷k)	$F_{v,Vd,i} = F_{v,Vd,(i-1)} + n_{r,i} \cdot \left(\frac{n_{sp} \cdot F_{m,d,max}}{r_{max}} \cdot \lvert x_i \rvert - \frac{1}{n} \cdot V_z\right)$		
	$n_{r,i}$	*- bolt number in the bolt line -i; (i=2÷k)*	
	x_i	*- bolt line -i distance from fastener centroid; (i=2÷k)*	
bolt line number in the shear zone	$k =$	1	[-]
maximum shear force in a bolt line	$F_{v,Vd,max} = max(F_{v,Vd,i}) =$	0.00	[kN]

Maximum shear stress calculation:

shear stress within the connection area	$\tau_{c,d,1} = \frac{3}{2} \cdot \frac{F_{v,Vd,max}}{l_1 \cdot b_1 \cdot [h_1 - n_{r,max} \cdot (d+1)]}$		
bolt number in the line with $F_{v,Vd,max}$	$n_{r,max} =$	3	[-]
	$\tau_{c,d,1} =$	0.00	[MPa]
shear stress in the beam at the connection	$\tau_{b,d,1} = \frac{3}{2} \cdot \frac{V_z}{l_1 \cdot b_1 \cdot h_1} =$	0.00	[MPa]
design shear stress	$\tau_{d,1} = max(\tau_{c,d,1} ; \tau_{b,d,1}) =$	0.00	[MPa]
design shear strength	$f_{v,d,1} = \frac{k_{mod} \cdot f_{v,k,1}}{\gamma_{M,d,1}} =$	1.97	[MPa]

$\tau_{d,1} =$	0.00	[MPa]	< $f_{v,d,1}$ =	1.97	[MPa]	VERIFIED
efficiency ratio				0.00	[-]	

<u>Element El.2 load-carrying capacity check:</u>

Tensile stress along the grain check:

El.2 cross-section net area	$A_{n,2} =$	4712.00	[m2]
tensile stress along the grain	$\sigma_{t,0,d,2} = \frac{N}{A_{n,2}} =$	0.21	[MPa]
design tensile strength along the grain	$f_{t,0,d,2} = \frac{k_{mod} \cdot k_{h,2} \cdot k_{l,2} \cdot f_{t,0,k,2}}{\gamma_{M,d,2}} =$	6.15	[MPa]

$\sigma_{t,0,d,2} =$	0.21	[MPa]	< $f_{t,0,d,2}$ =	6.15	[MPa]	VERIFIED
efficiency ratio		$w_5 = \frac{\sigma_{t,0,d,2}}{f_{t,0,d,2}} =$		0.03	[-]	

Shear stress check:

Maximum shear stress calculation:

shear stress within the connection area	$\tau_{c,d,2} = \frac{3}{2} \cdot \frac{F_{v,Vd,max}}{l_2 \cdot b_2 \cdot [h_2 - n_{r,max} \cdot (d+1)]}$		
bolt number in the line with $F_{v,Vd,max}$	$n_{r,max} =$	3	[-]
	$\tau_{c,d,2} =$	0.00	[MPa]
shear stress in the beam at the connection	$\tau_{b,d,2} = \frac{3}{2} \cdot \frac{V_z}{l_2 \cdot b_2 \cdot h_2} =$	0.00	[MPa]
design shear stress	$\tau_{d,2} = max(\tau_{c,d,2} ; \tau_{b,d,2}) =$	0.00	[MPa]
design shear strength	$f_{v,d,2} = \frac{k_{mod} \cdot f_{v,k,2}}{\gamma_{M,d,2}} =$	1.97	[MPa]

$\tau_{d,2} =$	0.00	[MPa]	< $f_{v,d,2}$ =	1.97	[MPa]	VERIFIED
efficiency ratio		$w_6 = \frac{\tau_{d,2}}{f_{v,d,2}} =$		0.00	[-]	

1.11. Truss connections

1.11.1. Lateral Load-carrying capacity of connections (EC5 EN1995-1-1:2009, §8)

Connection bolts and connection plates
Selected bolts of diameter d=4.0 mm. Metal plates of thickness t=2.0 mm.
Yield strength for plate steel fy=240 N/mm². Net plate area (minus holes) Anet=(0.75)·b·t

Cross section properties
Thickness of timber d=60.0 mm, thickness of steel plate t=2.0 mm
Bolt properties (EC5 §8.5.1)
Bolt diameter d=4.0mm, washer with diameter >= 12.0mm and thickness >=1.2mm.
Distance between bolts (EC5 Table 8.4)
as most unfavourable is chosen a1=7d=7x4.0=28 mm, a2=4d=16 mm

Characteristic value for yield moment (EC5 §8.5.1.1)
Myrk=0.30fuk·d^2.6=0.30x400x4.0^2.6=4411 Nmm (fuk=400N/mm²) (EN1995-1-1 Eq.8.30)

Characteristic value of embedment strength (EC5 §8.3.1.1)
fhk=0.082(1-0.01d)ρk=29.13N/mm², (ρk=370kg/m³,d=4.0mm) (EN1995-1-1 Eq.8.32)

Permanent action
Capacity of laterally loaded bolts -Double shear connection (EC5 §8.2.3)
t2=60.0 mm, thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.12(j), 8.12(k))
0.50fhk·t2·d=3.496 kN
1.15√[2Myrk·fhk·d]=1.166 kN
Lateral load-carrying capacity of bolt Rd=2Kmod·Fvrk/γM=2x0.60x1.166/1.30=1.076 kN

Medium-term action
Capacity of laterally loaded bolts -Double shear connection (EC5 §8.2.3)
t2=60.0 mm, thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.12(j), 8.12(k))
0.50fhk·t2·d=3.496 kN
1.15√[2Myrk·fhk·d]=1.166 kN
Lateral load-carrying capacity of bolt Rd=2Kmod·Fvrk/γM=2x0.80x1.166/1.30=1.435 kN

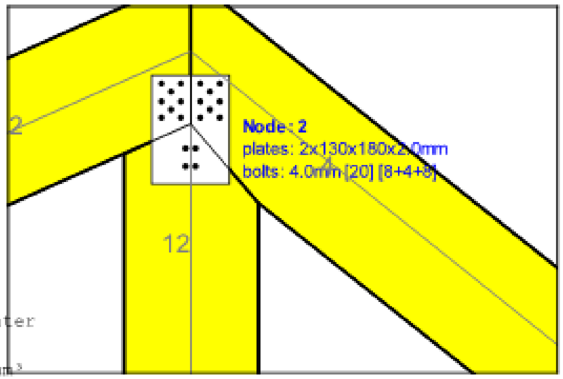
Short-term action
Capacity of laterally loaded bolts -Double shear connection (EC5 §8.2.3)
t2=60.0 mm, thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.12(j), 8.12(k))
0.50fhk·t2·d=3.496 kN
1.15√[2Myrk·fhk·d]=1.166 kN
Lateral load-carrying capacity of bolt Rd=2Kmod·Fvrk/γM=2x0.90x1.166/1.30=1.614 kN

Assumptions for the design of bolted connections
The design of connections is based on plastic analysis. The forces at the bolts are all reaching the same limit value. The metal plate capacity is based on plastic section modulus. The compressive design force is reduced to 0.50xFd

1.11.2. Ultimate limit state

Design of bolted connection at node : 2 (EC5 EN1995-1-1:2009, §8.5)
Connection with double (2) metal plates on the two faces of the truss.

Connection check of element 2, with elements 4 and 12, at node 2



Fastener characteristics:
Two(2) metal 2.0 mm plates with dimensions
BxH=130mmx180mm, and thickness 2.0mm
Bolts with diameter d=4.0mm,
8 bolts on each of the connected elements
Distance between bolts a1=28 mm, a2=16 mm
Yield strength for plate steel fy=240 N/mm²
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner bolt Fn=Fa/n+Ma/Wp
n: number of bolts, a: bolt section area
A=nxa: total area of bolts
r: distance of corner bolt from connection center
Wp: section modulus of connection
n= 8, (nef=1.30n), A=101mm², r=37mm, Wp =3407mm³
σ and σd plate normal and bearing stress N/mm²

Forces at node 2 ,from elements 4, 12, at the center of the joint F(force) M(moment)
Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.G	Permanent	0.60	1.946	-0.029	0.348	<	1.076
2	γg.G+γq.Q1	Short-term	0.90	8.041	-0.110	1.406	<	1.614
3	γg.G+γq.Q2	Short-term	0.90	5.277	-0.069	0.912	<	1.614
4	γg.G+γq.Q3	Short-term	0.90	7.758	-0.110	1.370	<	1.614
5	γg.G+γq.Q4	Short-term	0.90	2.814	-0.047	0.519	<	1.614
6	γg.G+γq.Q5	Short-term	0.90	1.946	-0.029	0.348	<	1.614
7	γg.G+γq.Qf	Medium-term	0.80	-0.770	-0.019	0.062	<	1.435
8	γg.G+γq.Qi	Short-term	0.90	3.921	-0.054	0.687	<	1.614
9	γg.G+γq.Q1+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	7.200	-0.112	1.303	<	1.614
10	γg.G+γq.Q1+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	6.767	-0.103	1.218	<	1.614
11	γg.G+γq.Q2+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	4.439	-0.071	0.810	<	1.614
12	γg.G+γq.Q2+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	4.004	-0.063	0.725	<	1.614
13	γg.G+γq.Q3+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	6.930	-0.112	1.269	<	1.614
14	γg.G+γq.Q3+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	6.495	-0.103	1.183	<	1.614
15	γg.G+γq.Q4+γq.ψo.Q1+γq.ψo.Qf	Short-term	0.90	5.214	-0.088	0.967	<	1.614
16	γg.G+γq.Q4+γq.ψo.Q2+γq.ψo.Qf	Short-term	0.90	3.567	-0.064	0.672	<	1.614
17	γg.G+γq.Q4+γq.ψo.Q3+γq.ψo.Qf	Short-term	0.90	5.058	-0.089	0.946	<	1.614
18	γg.G+γq.Q5+γq.ψo.Q1+γq.ψo.Qf	Short-term	0.90	4.342	-0.071	0.796	<	1.614
19	γg.G+γq.Q5+γq.ψo.Q2+γq.ψo.Qf	Short-term	0.90	2.691	-0.046	0.501	<	1.614
20	γg.G+γq.Q5+γq.ψo.Q3+γq.ψo.Qf	Short-term	0.90	4.183	-0.071	0.775	<	1.614
21	γg.G+γq.Qf+γq.ψo.Q1+γq.ψo.Q4	Short-term	0.90	4.262	-0.077	0.803	<	1.614
22	γg.G+γq.Qf+γq.ψo.Q1+γq.ψo.Q5	Short-term	0.90	3.824	-0.068	0.717	<	1.614
23	γg.G+γq.Qf+γq.ψo.Q2+γq.ψo.Q4	Short-term	0.90	2.636	-0.052	0.509	<	1.614
24	γg.G+γq.Qf+γq.ψo.Q2+γq.ψo.Q5	Short-term	0.90	2.198	-0.044	0.424	<	1.614
25	γg.G+γq.Qf+γq.ψo.Q3+γq.ψo.Q4	Short-term	0.90	4.112	-0.077	0.783	<	1.614
26	γg.G+γq.Qf+γq.ψo.Q3+γq.ψo.Q5	Short-term	0.90	3.674	-0.068	0.697	<	1.614
27	γg.G+γq.Qi+γq.ψo.Q1+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	6.738	-0.105	1.220	<	1.614
28	γg.G+γq.Qi+γq.ψo.Q1+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	6.305	-0.096	1.135	<	1.614
29	γg.G+γq.Qi+γq.ψo.Q2+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	5.081	-0.080	0.924	<	1.614
30	γg.G+γq.Qi+γq.ψo.Q2+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	4.647	-0.072	0.839	<	1.614
31	γg.G+γq.Qi+γq.ψo.Q3+γq.ψo.Q4+γq.ψo.Qf	Short-term	0.90	6.575	-0.105	1.199	<	1.614
32	γg.G+γq.Qi+γq.ψo.Q3+γq.ψo.Q5+γq.ψo.Qf	Short-term	0.90	6.141	-0.096	1.114	<	1.614

Teretron

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Date of publication 2016

Components	
Name	Teretron 2016 Sample Project
Location	London, UK
Client	
Associates	
Other	

Component	
Name	Fastener lateral capacity (2)
Folder	Connections
Type	Fastener lateral capacity
Floor	
Other	

FASTENER LATERAL CAPACITY

Fastener lateral capacity

EN 1995-1-1:2004+A2:2014

Members

Single shear connection

Material Type 1 : Timber

Timber class : C16

Width [$\mathbf{b}_1$] = 45 mm

Angle [$\boldsymbol{\varphi}_1$] = 0 °

Characteristic density [$\boldsymbol{\rho}_{k,1}$] = 310 kg/m^3

Material Type 2 : Timber

Timber class : C16

Width [$\mathbf{b}_2$] = 45 mm

Angle [$\boldsymbol{\varphi}_2$] = 90 °

Characteristic density [$\boldsymbol{\rho}_{k,2}$] = 310 kg/m^3

Timber-to-timber connection in single shear

Total screw length [**l**] = 75 mm

Penetration depth [$\mathbf{t}_1$] = 45 mm

Penetration depth [$\mathbf{t}_2$] = 30 mm

Clause 8.7.1(3)

Effective diameter [$\mathbf{d}_{ef}$] = 2.695 mm

Characteristic yield moment [$\mathbf{M}_{y,Rk}$] = 2,716 N·mm

Without predrilling

Maximum contribution of rope effect [$(F_{ax,Rk} / 4)_{max}$] = 100 %

Head diameter [$\mathbf{d}_h$] = 6.8 mm

Outer thread diameter [**d**] = 3.6 mm

Inner thread diameter [$\mathbf{d}_1$] = 2.45 mm

Total screw length [**l**] = 75 mm

Length of the threaded part [$\mathbf{l}_g$] = 45 mm

Penetation length of the threaded part [$\mathbf{l}_{ef}$] = 45 mm

Total width [**b**] = 90 mm

Characteristic density [$\rho_{k,min}$] = 310 kg/m^3

Associated density [ρ_a] = 400 kg/m^3

Characteristic pointside withdrawal strength [$f_{ax,k}$] = 13.16 MPa

Equation (8.40a) Characteristic withdrawal capacity of shank [$F_{shank,ax,Rk}$] = 1,738.65 N

Characteristic headside pull-through strength [$f_{head,k}$] = 20.56 MPa

Equation (8.40b) Characteristic pull-through resistance [$F_{head,ax,Rk}$] = 775.322 N

Clause 8.7.2(1) Characteristic axial withdrawal capacity [$F_{ax,Rk}$] = 775.322 N

Design action

Design force angle = 0 °

Design modifications to the load-carrying capacity

Number of shear planes [n_{sp}] = 1

Table 3.1 Factor [k_{mod}] = 1.1

Table 2.3 Partial factor [γ_M] = 1.3

Fastener lateral capacity

Embedment strengths

Characteristic embedment strength parallel to the grain [$f_{h,0,1,k}$] = 18.88 MPa

Characteristic embedment strength parallel to the grain [$\mathbf{f}_{h,0,2,k}$] = 18.88 MPa

Characteristic lateral load-carrying capacity per shear plane

Load-carrying capacity values

Load-carrying capacity value [$\mathbf{F}_{v,Rk,a}$] = 2,289.703 N
Load-carrying capacity value [$\mathbf{F}_{v,Rk,b}$] = 1,526.469 N
Load-carrying capacity value [$\mathbf{F}_{v,Rk,c}$] = 1,011.037 N
Load-carrying capacity value [$\mathbf{F}_{v,Rk,d}$] = 1,057.394 N
Load-carrying capacity value [$\mathbf{F}_{v,Rk,e}$] = 819.265 N
Load-carrying capacity value [$\mathbf{F}_{v,Rk,f}$] = 798.421 N

Characteristic lateral load-carrying capacity per shear plane [$\mathbf{F}_{v,Rk}$] = 798.421 N

Design lateral load-carrying capacity

Design lateral load-carrying capacity [$\mathbf{F}_{v,Rd}$] = 675.587 N

OTHER CHECKS

Other checks

Pointside penetration requirements

Pointside penetration length [$\mathbf{t}_{pen}$] = 30 mm

Clause 8.7.2(3) Required pointside penetration length [$\mathbf{t}_{pen,\ min}$] = 21.6 mm

Pointside penetration length check: **PASS**

For timber without predrilling

Clause 8.3.1.1(2) Maximum allowed diameter/side [$\mathbf{d}_{max}$] = 6 mm

Diameter/Side [$\mathbf{d}$] = 2.695 mm

Clause 8.3.1.1(2) Maximum allowed density [$\boldsymbol{\rho}_{k,max}$] = 500 kg/m^3

Clause 8.3.1.2(6) Minimum allowed thickness [$\mathbf{b}_{1,min}$] = 18.865 mm

Width [$\mathbf{b}_1$] = 45 mm

Clause 8.3.1.1(2) Maximum allowed density [$\boldsymbol{\rho}_{k,max}$] = 500 kg/m^3

Characteristic density [$\boldsymbol{\rho}_{k,2}$] = 310 kg/m^3

Clause 8.3.1.2(6) Minimum allowed thickness [$\mathbf{b}_{2,min}$] = 18.865 mm

Width [$\mathbf{b}_2$] = 45 mm

Predrilling requirements check: **PASS**

SPACINGS AND DISTANCES

Minimum allowed spacings and distances

Member 1

Timber class : C16

Characteristic density [$\boldsymbol{\rho}_{k,1}$] = 310 kg/m^3

	Outer thread diameter [$\mathbf{d}$] = 3.6 mm
	Angle between force and member 1 [θ_1] = 0 °
Table 8.2	Column spacing [$a_{1,min}$] = 36 mm
Table 8.2	Row spacing [$a_{2,min}$] = 18 mm
Table 8.2	Distance from loaded end [$a_{3,t,min}$] = 54 mm
Table 8.2	Distance from unloaded end [$a_{3,c,min}$] = 36 mm
Table 8.2	Distance from loaded edge [$a_{4,t,min}$] = 18 mm
Table 8.2	Distance from unloaded edge [$a_{4,c,min}$] = 18 mm
	Member 2
	Timber class : C16
	Characteristic density [$\rho_{k,2}$] = 310 kg/m^3
	Outer thread diameter [$\mathbf{d}$] = 3.6 mm
	Angle between force and member 2 [θ_2] = 90 °
Table 8.2	Column spacing [$a_{1,min}$] = 18 mm
Table 8.2	Row spacing [$a_{2,min}$] = 18 mm
Table 8.2	Distance from loaded end [$a_{3,t,min}$] = 36 mm
Table 8.2	Distance from unloaded end [$a_{3,c,min}$] = 36 mm
Table 8.2	Distance from loaded edge [$a_{4,t,min}$] = 25.2 mm
Table 8.2	Distance from unloaded edge [$a_{4,c,min}$] = 18 mm

References

Timber Data

Solid timber data according to:

EN 338: 2009, "Structural Timber. Strength Classes"

EN 1990:2002 Eurocode 0, Basic of Structural Design

Elastic analysis according to:

EN 1990:2002 and EN 1991:2002

EN 1995:2004+A2:2014 Eurocode 5, Section 1-1, General - Common rules and rules for buildings

Partial factor γ_M for properties
and strengths of materials according to:
EN 1995-1-1:2004+A2:2014,
Section 2, Table 2.3

Modification factor k_{mod} taking into account the effect
of duration of load and moisture content according to:
EN 1995-1-1:2004+A2:2014,
Section 2, Clause 2.4

The structural analysis for the calculation of forces and moments must conform to:
EN 1995-1-1:2004+A2:2014,
Section 5

Connections with metal fasteners according to:
EN 1995-1-1:2004+A2:2014, Section 8

Calculation of lateral load-carrying capacity
of metal dowel-type fasteners according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Clause 8.2

The effect of the threaded part of the screw
shall be taken into account in determining
the load-carrying capacity, by using an
effective diameter d^{ef} according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Subclauses 8.7.1(1) , 8.7.1(2) and 8.7.1(3)
Calculation of characteristic yield moment
of screws according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Subclauses 8.7.1(4) , 8.7.1(5) ,
8.3.1.1(4) and 8.5.1.1(1)
Calculation of characteristic embedment strength
for screws according to:

EN 1995-1-1:2004+A2:2014,
Section 8, Subclauses 8.7.1(4) , 8.7.1(5) , 8.3.1.1(5) ,
8.3.1.1(6) , 8.3.1.3(3) , 8.5.1.1(2) and 8.5.1.2(1)
Calculation of effective number of screws
arranged in a row parallel to the grain, according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Subclauses 8.7.1(4) , 8.7.1(5) ,
8.3.1.1(8) and 8.5.1.1(4)
Maximum contribution of the rope effect
to the characteristic load-carrying capacity
as a percentage of the Johansen part:
100% for screws, according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Subclause 8.2.2(2)
Recommended mimimum spacings, edge distances
and end distances for bolts in a grid arrangement,
according to:
EN 1995-1-1:2004+A2:2014,
Section 8, Subclauses 8.7.1(4) , 8.7.1(5) , 8.3.1.2(5) ,
8.3.1.3(2) , 8.3.1.4(1) and 8.5.1.1(3)

The design must conform to the requirements of:
EN 1995-1-1:2004+A2:2014, Section 10

Appendix F.   Axial loading data fitting GoF screen shot

```
Command Window
Gof_max =

   1.3039e-07


Gof_95 =

   1.2204e-10


Max_Resid =

   1.3881e-04
```

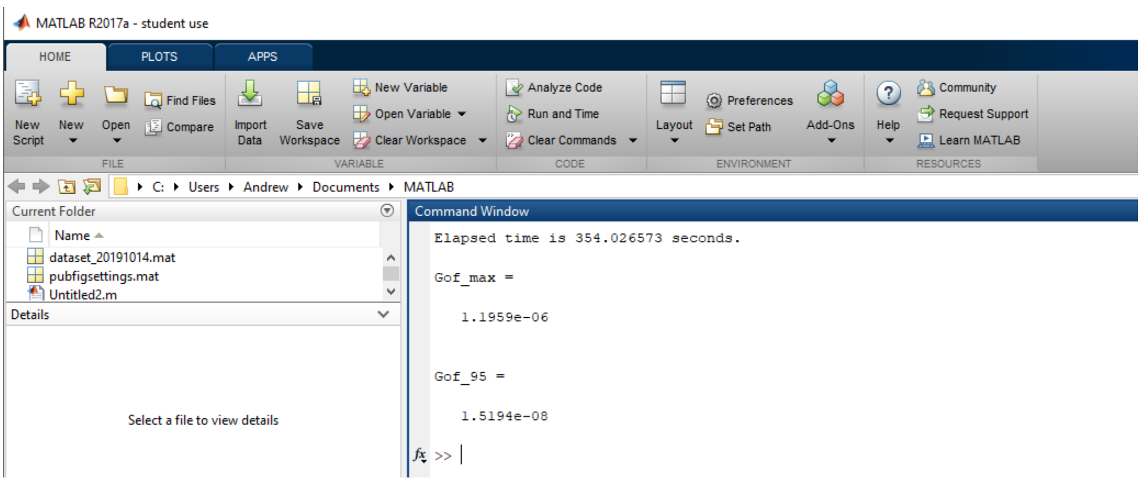
Appendix G. Data fitting laterally loaded connections

1. Data set creation and calculating the Goodness-of-fit: Code

```
clear
clc
int = 10;
SW = 0.00001;

Fax_Rk_var1 = linspace(1.5,5,int);
Pf_mod_var2 = linspace(1.5,2,int);
fh_k_1_var3 = linspace(10,30,int);
fh_k_2_var4 = linspace(10,30,int);
t1_var5 = linspace(20,50,int);
t2_var6 = linspace(20,100,int);
d_var7 = linspace(3.1,8,int);
fu_f_var8 = linspace(400,1000,int);

data = zeros(length(Fax_Rk_var1),length(Pf_mod_var2), length(fh_k_1_var3),
length(fh_k_2_var4), length(t1_var5),
length(t2_var6),length(d_var7),length(fu_f_var8));
dataf = data;
fitdat=data;
EC5Ans = zeros(length(Fax_Rk_var1),length(Pf_mod_var2), length(fh_k_1_var3),
length(fh_k_2_var4), length(t1_var5),
length(t2_var6),length(d_var7),length(fu_f_var8));
EC5AnsList = zeros(length(Fax_Rk_var1),length(Pf_mod_var2),
length(fh_k_1_var3), length(fh_k_2_var4), length(t1_var5),
length(t2_var6),length(d_var7),length(fu_f_var8));
tic
for i = 1 : length(Fax_Rk_var1)
    for j = 1 : length(Pf_mod_var2)
        for k = 1 : length(fh_k_1_var3)
            for l = 1 : length(fh_k_2_var4)
                for m = 1 : length(t1_var5)
                    for n = 1 : length(t2_var6)
                        for p = 1 : length(d_var7)
                            for q = 1 : length(fu_f_var8)

    Be = fh_k_2_var4(l) / fh_k_1_var3(k) ;
    Rope = Fax_Rk_var1(i) / 4;
    t21 = t2_var6(n)/t1_var5(m);
    f_m_a = (fh_k_1_var3(k) * t1_var5(m) * d_var7(p)) / 1000;
    f_m_b = (fh_k_2_var4(l) * t2_var6(n) * d_var7(p)) / 1000;

    f_m_c_jyt = f_m_a / (1 + Be) * (sqrt(Be + 2 *
Be^(2)*(1+(t21)+t21^(2))+Be^(3)*(t21^(2)))-Be*(1+t21));
    f_m_c = min((f_m_c_jyt + Rope), (Pf_mod_var2(j) * f_m_c_jyt));

    f_m_d_jyt = (1.05 * f_m_a / (2 + Be)) * (sqrt(2 * Be*
(1+Be)+((4*Be*(2+Be)*  (0.45 * fu_f_var8(q) *
d_var7(p)^(2.6)))/(fh_k_1_var3(k)*t1_var5(m)^(2)*d_var7(p))))-Be);
    f_m_d = min((f_m_d_jyt + Rope), (Pf_mod_var2(j) * f_m_d_jyt));

    f_m_e_jyt = (1.05 * (fh_k_2_var4(l) * t2_var6(n) *d_var7(p)) /
(1+2*Be))*(sqrt(2*Be^(2)*(1+Be)+((4*Be*(1+2*Be)*  (0.45 * fu_f_var8(q) *
d_var7(p)^(2.6)))/(fh_k_1_var3(k) * t2_var6(n)^(2) *d_var7(p))))-Be) / 1000;
    f_m_e = min((f_m_e_jyt + Rope), (Pf_mod_var2(j) * f_m_e_jyt));

    f_m_f_jyt = 1.15 * sqrt((2*Be) / (1+Be)) * sqrt((2*  (0.45 * fu_f_var8(q)
* d_var7(p)^(2.6)) * fh_k_1_var3(k) * d_var7(p))) / 1000;
    f_m_f = min((f_m_f_jyt + Rope), (Pf_mod_var2(j) * f_m_f_jyt));

    [force,failure] = min([f_m_a  f_m_b f_m_c f_m_d f_m_e f_m_f]);
    data(i,j,k,l,m,n,p,q) = force;
    dataf(i,j,k,l,m,n,p,q) = failure;

    EC5Ans = min([f_m_a f_m_b f_m_c f_m_d f_m_e f_m_f]);
```

```
    AnsA = ((1 / (1 + exp((f_m_a - f_m_b)/SW)))*(1 / (1 + exp((f_m_a -
f_m_c)/SW))) * (1 / (1 + exp((f_m_a - f_m_d)/SW))) * (1 / (1 + exp((f_m_a -
f_m_e)/SW))) * (1 / (1 + exp((f_m_a - f_m_f)/SW))) * f_m_a);
    AnsB = ((1 / (1 + exp((f_m_b - f_m_a)/SW)))*(1 / (1 + exp((f_m_b -
f_m_c)/SW))) * (1 / (1 + exp((f_m_b - f_m_e)/SW))) * (1 / (1 + exp((f_m_b -
f_m_f)/SW))) * f_m_b);
    AnsC = ((1 / (1 + exp((f_m_c - f_m_a)/SW)))*(1 / (1 + exp((f_m_c -
f_m_b)/SW))) * (1 / (1 + exp((f_m_c - f_m_d)/SW))) * (1 / (1 + exp((f_m_c -
f_m_e)/SW))) * (1 / (1 + exp((f_m_c - f_m_f)/SW))) * f_m_c);

    AnsD = ((1 / (1 + exp((f_m_d - f_m_a)/SW)))*(1 / (1 + exp((f_m_d -
f_m_c)/SW))) * (1 / (1 + exp((f_m_d - f_m_e)/SW))) * (1 / (1 + exp((f_m_d -
f_m_f)/SW))) * f_m_d);
    AnsE = ((1 / (1 + exp((f_m_e - f_m_a)/SW)))*(1 / (1 + exp((f_m_e -
f_m_b)/SW))) * (1 / (1 + exp((f_m_e - f_m_c)/SW))) * (1 / (1 + exp((f_m_e -
f_m_d)/SW))) * (1 / (1 + exp((f_m_e - f_m_f)/SW))) * f_m_e);
    AnsF = ((1 / (1 + exp((f_m_f - f_m_a)/SW)))*(1 / (1 + exp((f_m_f -
f_m_b)/SW))) * (1 / (1 + exp((f_m_f - f_m_c)/SW))) * (1 / (1 + exp((f_m_f -
f_m_d)/SW))) * (1 / (1 + exp((f_m_f - f_m_e)/SW))) * f_m_f);

    Anser = AnsA + AnsB + AnsC + AnsD + AnsE + AnsF;

    fitdat(i,j,k,l,m,n,p,q) = Anser;

                        end
                    end
                end
            end
        end
    end
    end
end
toc

    Resid = fitdat - data;
    Gof_max = max(abs(Resid(:)))/ mean(data(:))
    Gof_95 = (2*std(Resid(:))) / mean(data(:))
```

2. Screen shot for the Goodness-of-fit

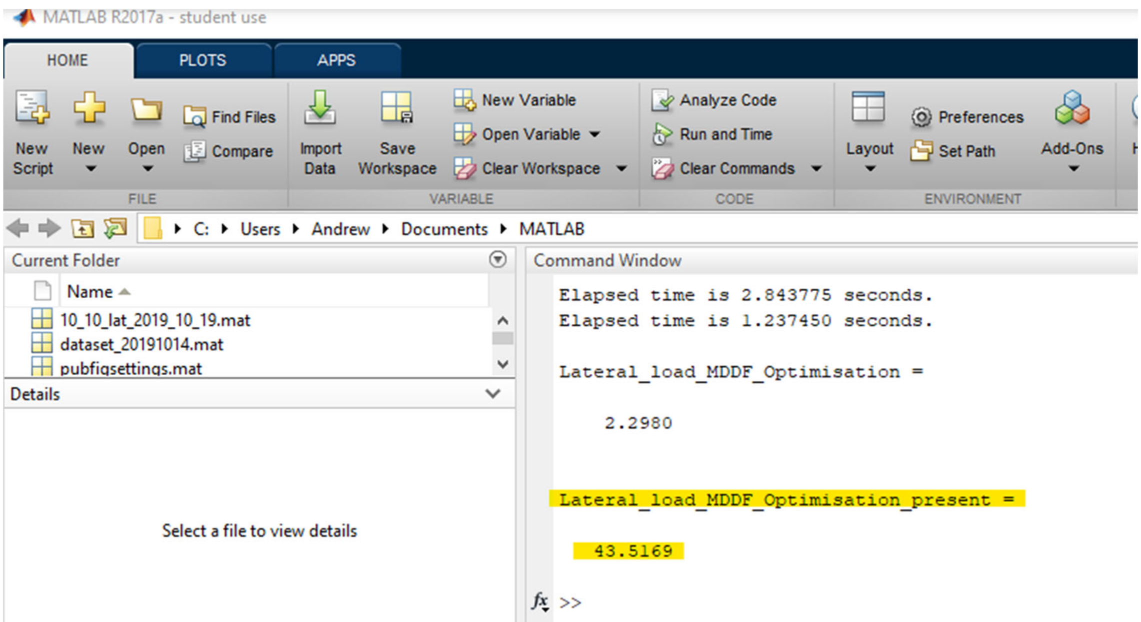


3. Computational comparison, Code

```
clear
clc
int = 8;
Fax_Rk_var1 = 3.4;  Pf_mod_var2 = 2; fh_k_1_var3 = 23.3; fh_k_2_var4 = 23.3;
t1_var5 = 20; t2_var6 = 45;  d_var7 = 4.7; fu_f_var8 = 600;

t_pen = 40; d = 6; k_h = 2.5; f_ax_k = 6; f_h_k = 6; ro_km = 310; ro_pss =
350; ro_hss = 350;
A_1 = 1.2 * (ro_pss / ro_hss)^(0.8) * d * k_h * k_h * f_h_k * t_pen^(-1) -
f_ax_k;

Be = fh_k_2_var4 / fh_k_1_var3 ;
Rope = Fax_Rk_var1 / 4;
t21 = t2_var6 / t1_var5;


loop = 100000;
tic
for i = 1 : loop
    F_ax_point_Rk = f_ax_k * d * (t_pen / 1.2) * (ro_km / ro_pss)^(0.8);
    F_head_Rk = f_h_k * (d * k_h) * (d * k_h) * (ro_km / ro_hss)^(0.8);
    Fax_Rk_var1 = min([F_ax_point_Rk F_head_Rk]);
    Rope = Fax_Rk_var1 / 4;
    t21 = t2_var6 / t1_var5;
    f_m_a = ((fh_k_1_var3 * t1_var5 * d_var7) / 1000);
    f_m_b = (fh_k_2_var4 * t2_var6 * d_var7) / 1000;

    f_m_c_jyt = ((fh_k_1_var3 * t1_var5 * d_var7)) / (1 + Be) * (sqrt(Be + 2 *
Be^(2)*(1+(t21)+t21^(2))+Be^(3)*(t21^(2)))-Be*(1+t21))+ Rope;
    f_m_c = min((f_m_c_jyt + Rope), (Pf_mod_var2 * f_m_c_jyt));

    f_m_d_jyt = (1.05 * ((fh_k_1_var3 * t1_var5 * d_var7)) / (2 + Be)) *
(sqrt(2 * Be* (1+Be)+((4*Be*(2+Be)*  (0.45 * fu_f_var8 *
d_var7^(2.6)))/(fh_k_1_var3*t1_var5^(2)*d_var7)))-Be)+ Rope;
    f_m_d = min((f_m_d_jyt + Rope), (Pf_mod_var2 * f_m_d_jyt));

    f_m_e_jyt = ((1.05 * (fh_k_2_var4 * t2_var6 *d_var7) /
(1+2*Be))*(sqrt(2*Be^(2)*(1+Be)+((4*Be*(1+2*Be)*  (0.45 * fu_f_var8 *
d_var7^(2.6)))/(fh_k_1_var3 * t2_var6^(2) *d_var7)))-Be) / 1000) + Rope;
    f_m_e = min((f_m_e_jyt + Rope), (Pf_mod_var2 * f_m_e_jyt));

    f_m_f_jyt = (1.15 * sqrt((2*Be) / (1+Be)) * sqrt((2*  (0.45 * fu_f_var8 *
d_var7^(2.6)) * fh_k_1_var3 * d_var7)) / 1000) + Rope;
    f_m_f = min((f_m_f_jyt + Rope), (Pf_mod_var2 * f_m_f_jyt));

    F_M = min([f_m_a f_m_b f_m_c f_m_d f_m_e f_m_f]);
end
toc
elapsedTime_1 = toc;
tic



for i = 1 : loop
    Fax_Rk_var1 = (f_ax_k * d * (t_pen / 1.2) * (ro_km / ro_pss)^(0.8) ) / (1
+ exp(A_1 / -0.001)) + (f_h_k * (d * k_h)* (d * k_h) * (ro_km / ro_hss)^(0.8))
/ (1 + exp(A_1 / 0.001));
    Rope = Fax_Rk_var1 / 4;
    f_m_a = (fh_k_1_var3 * t1_var5 * d_var7) / 1000;
    f_m_b = (fh_k_2_var4 * t2_var6 * d_var7) / 1000;
    f_m_c = ((fh_k_1_var3 * t1_var5 * d_var7)) / (1 + Be) * (sqrt(Be + 2 *
Be^(2)*(1+(t21)+t21^(2))+Be^(3)*(t21^(2)))-Be*(1+t21))+ Rope;
```

```
    f_m_d = (1.05 * ((fh_k_1_var3 * t1_var5 * d_var7)) / (2 + Be)) * (sqrt(2 *
Be* (1+Be)+((4*Be*(2+Be)*  (0.45 * fu_f_var8 *
d_var7^(2.6)))/(fh_k_1_var3*t1_var5^(2)*d_var7)))-Be)+ Rope;
    f_m_e = ((1.05 * (fh_k_2_var4 * t2_var6 *d_var7) /
(1+2*Be))*(sqrt(2*Be^(2)*(1+Be)+((4*Be*(1+2*Be)*  (0.45 * fu_f_var8 *
d_var7^(2.6)))/(fh_k_1_var3 * t2_var6^(2) *d_var7)))-Be) / 1000) + Rope;
    f_m_f = (1.15 * sqrt((2*Be) / (1+Be)) * sqrt((2*  (0.45 * fu_f_var8 *
d_var7^(2.6)) * fh_k_1_var3 * d_var7)) / 1000) + Rope;
    Anser = (((0.5+0.5*tanh(1000*(f_m_b-f_m_a)))*(0.5+0.5*tanh(1000*(f_m_c-
f_m_a)))*(0.5+0.5*tanh(1000*(f_m_d-f_m_a)))*(0.5+0.5*tanh(1000*(f_m_e-
f_m_a)))*(0.5+0.5*tanh(1000*(f_m_f-f_m_a)))) * f_m_a) +
(((0.5+0.5*tanh(1000*(f_m_a-f_m_b)))* (0.5+0.5*tanh(1000*(f_m_c-f_m_b)))*
(0.5+0.5*tanh(1000*(f_m_e-f_m_b)))* (0.5+0.5*tanh(1000*(f_m_f-f_m_b)))) *
f_m_b) + (((0.5+0.5*tanh(1000*(f_m_a-f_m_c)))* (0.5+0.5*tanh(1000*(f_m_b-
f_m_c)))* (0.5+0.5*tanh(1000*(f_m_d-f_m_c)))* (0.5+0.5*tanh(1000*(f_m_e-
f_m_c)))* (0.5+0.5*tanh(1000*(f_m_f-f_m_c)))) * f_m_c) +
(((0.5+0.5*tanh(1000*(f_m_a-f_m_d)))* (0.5+0.5*tanh(1000*(f_m_c-f_m_d)))*
(0.5+0.5*tanh(1000*(f_m_e-f_m_d)))* (0.5+0.5*tanh(1000*(f_m_f-f_m_d)))) *
f_m_d) + (((0.5+0.5*tanh(1000*(f_m_b-f_m_e)))* (0.5+0.5*tanh(1000*(f_m_c-
f_m_e)))* (0.5+0.5*tanh(1000*(f_m_d-f_m_e)))* (0.5+0.5*tanh(1000*(f_m_e-
f_m_e)))*(0.5+0.5*tanh(1000*(f_m_f-f_m_e)))) * f_m_e) +
(((0.5+0.5*tanh(1000*(f_m_a-f_m_f)))* (0.5+0.5*tanh(1000*(f_m_b-f_m_f)))*
(0.5+0.5*tanh(1000*(f_m_c-f_m_f)))* (0.5+0.5*tanh(1000*(f_m_d-f_m_f)))*
(0.5+0.5*tanh(1000*(f_m_e-f_m_f)))) * f_m_f);
end
toc
elapsedTime_2 = toc;


Lateral_load_MDDF_Optimisation = elapsedTime_1 / elapsedTime_2
Lateral_load_MDDF_Optimisation_present = 100 / elapsedTime_1 * elapsedTime_2
```

4. Screen shot for the Computational comparison



Appendix H. BIM-ready equation: code for the lateral loading

F_v.Rk=((0.5+0.5*tanh(sw*(fm_b-fm_a)))*(0.5+0.5*tanh(sw*(fm_c-fm_a)))*(0.5+0.5*tanh(sw*(fm_d-fm_a)))*(0.5+0.5*tanh(sw*(fm_e-fm_a)))*(0.5+0.5*tanh(sw*(fm_f-fm_a)))*fm_a) + ((0.5+0.5*tanh(sw*(fm_a-fm_b)))*(0.5+0.5*tanh(sw*(fm_c-fm_b)))*(0.5+0.5*tanh(sw*(fm_d-fm_b)))*(0.5+0.5*tanh(sw*(fm_e-fm_b)))*(0.5+0.5*tanh(sw*(fm_f-fm_b)))*fm_b) + ((0.5+0.5*tanh(sw*(fm_a-fm_c)))*(0.5+0.5*tanh(sw*(fm_b-fm_c)))*(0.5+0.5*tanh(sw*(fm_d-fm_c)))*(0.5+0.5*tanh(sw*(fm_e-fm_c)))*(0.5+0.5*tanh(sw*(fm_f-fm_c)))*fm_c) + ((0.5+0.5*tanh(sw*(fm_a-fm_d)))*(0.5+0.5*tanh(sw*(fm_b-fm_d)))*(0.5+0.5*tanh(sw*(fm_c-fm_d)))*(0.5+0.5*tanh(sw*(fm_e-fm_d)))*(0.5+0.5*tanh(sw*(fm_f-fm_d)))*fm_d) + ((0.5+0.5*tanh(sw*(fm_a-fm_e)))*(0.5+0.5*tanh(sw*(fm_b-fm_e)))*(0.5+0.5*tanh(sw*(fm_c-fm_e)))*(0.5+0.5*tanh(sw*(fm_d-fm_e)))*(0.5+0.5*tanh(sw*(fm_f-fm_e)))*fm_e) + ((0.5+0.5*tanh(sw*(fm_a-fm_f)))*(0.5+0.5*tanh(sw*(fm_b-fm_f)))*(0.5+0.5*tanh(sw*(fm_c-fm_f)))*(0.5+0.5*tanh(sw*(fm_d-fm_f)))*(0.5+0.5*tanh(sw*(fm_e-fm_f)))*fm_f)

where:

FM_A = f_hk1*t_1*Fixings!d

FM_B =f_hk2*t_2*Fixings!d

FM_C = (fm_a/(1+SS))*(sqrt(SS+2*power(SS,2)*(1+(t_2/t_1)+power(t_2/t_1,2))+power(ss,3)*power(t_2/t_1,2))-ss*(1+(t_2/t_1)))+(Rope/4)

FM_D = 1.05 * (fm_a/(2+SS))*(sqrt(2*SS*(1+SS)+((4*SS*(2+SS)*M_yRk)/(f_hk1*Fixings!d*power(t_1,2))))-SS)+Rope

FM_E = 1.05*((f_hk1*t_2*Fixings!d)/(1+2*SS))*(sqrt(2*power(SS,2)*(1+SS)+((4*SS*(1+2*SS)*M_yRk)/(f_hk1*Fixings!d*power(t_2,2))))-SS)+Rope

FM_F = 1.15*(sqrt((2*SS)/(1+SS))) * sqrt(2*M_yRk*f_hk1*Fixings!d)+Rope

Appendix I. Full Survey findings

Contained within this section are the results of an online survey developed as a means to identify the barriers detrimental to the specification of timber and wood based products in the UK. The survey was developed based on the work undertaken by the Centre for Offsite Construction and Innovative Structures (COCIS) in combination with the findings of the "Thinking outside the box" report – a collaboration between Swedish Wood, the Timber Trade Federation and the former United Kingdom Timber Frame Association (now rebranded as the Structural Timber Association).

The result of the survey are presented in full as part of this section and are summarised in section 4.1.

Introduction

The Centre for Offsite Construction and Innovative Structures (COCIS) is a research centre within Edinburgh Napier Universities Institute of Sustainable Construction. In order to gain an improved understanding of the obstacles preventing the specification of structural timber and wood based products a survey was undertaken.

The following questions were asked of the respondents:

1. *Approximately what percentage of your percentage of your work is undertaken in Timber, Steel or concrete?*
2. *How strongly do you agree or disagree with the following statements:*
3. *"Knowledge of timber Engineering within professional teams is lacking"*
4. *"Perceptions of timber often overrule reality. This means that the idea of using modern wood building solutions can often be stifled in the early design stages.*
5. *"The lack of centrally available tables similar to those promoted by the concrete and steel industry means that timber is seen as a riskier choice for designers.*
6. *Are you using Eurocode 5, Design of timber structures*
7. *What would facilitate the use of Eurocode 5 ?*
8. *What Structural software do you use?*
9. *Would you be more likely to specify timber and timber related products if the required technical information was freely accessible?*
10. *Give one example of a timber structural detail that you would like to have standardised information for.*
11. *Are you using BIM, and if so to what level?*

Distribution of survey

The survey was developed using SurveyMonkey and was disseminated to Structural Engineers via the following means:

1. Direct email sent to members of the Timber Engineering Network. All recipients of this email were based in Scotland.

2. Open online survey made available via Linkidin

A total of 149 responses were generated from the survey, 76 of these were from IP address based in the UK, 19 from those based in Europe and 56 from the rest of the world. Only the results from the UK and Europe are reported in this study. A breakdown of the location of the UK respondents is given in Figure I- 1.

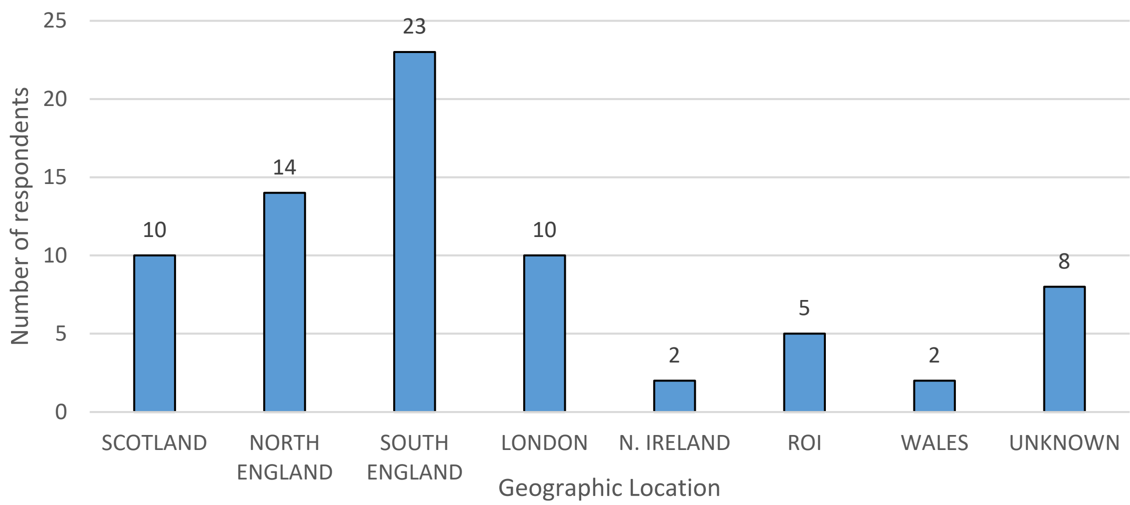


Figure I- 1 Number of UK respondents relative to geographical location within the UK

Note that respondents whose location is given as "unknown" are those which have withheld there IP address. Whilst it is known that they have responded from somewhere in the UK there exact geographic location cannot be identified with certainty.

The nature of the responses to each question and along with their perceived implications are reported in the following sections.

Question 1 - Nature of work undertaken

<u>"Approximately what percentage of your work is undertaken using Timber, Steel or Concrete?"</u>

Purpose of Question: To determine the degree to which timber is currently utilised in the structural design process in comparison to commonly specified materials.

Results: The responses from this question are presented in and Figure I- 2.

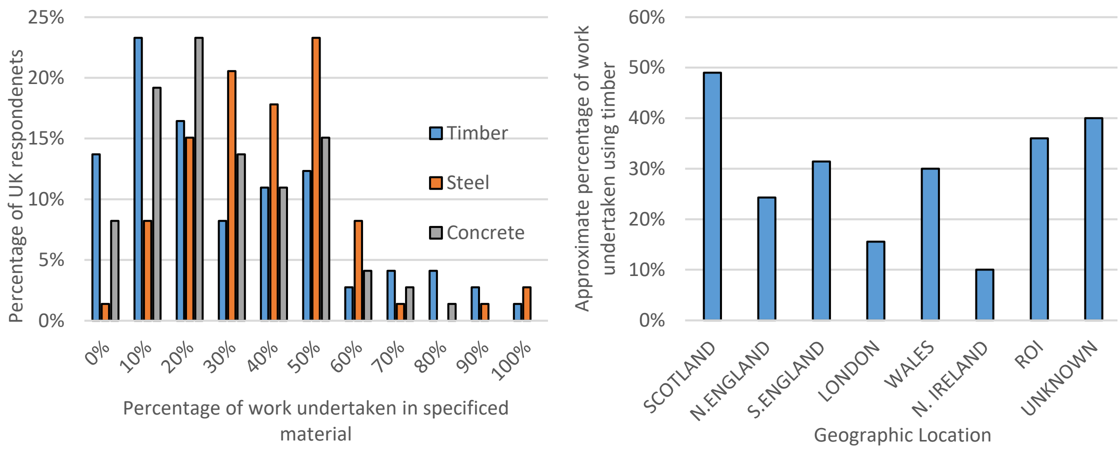


Figure I- 2 Responses to Question 1: "Approximately what percentage of your work is undertaken using timber?"- responses (Left) and average percentage of work undertaken using timber for each geographic region (Right).

Analysis: The results suggest that it is in Scotland that the majority of structural engineers are involved with designs utilising timber – a result in line with the prevalence of the Timber Platform Frame method of construction. Approximately 25-30% of engineers throughout England use regularly specify timber. This figure reduces to 16% in and around central London, possibly as a result of a lack of prevalence of the low-cost high volume domestic housing where timber is most commonly employed in this area.

Only one respondent to the survey (based in Dublin) indicated that they worked exclusively using timber.

It should be acknowledged that the results of the questionnaire may be skewed due to the nature of the sample surveyed. Respondents were selected from and existing list of COCIS contacts meaning that they are already predisposed to specify timber. Additionally, the questionnaires title may have had the effect of pre-selecting those engineers responding via Linkidin i.e those who are already using timber are more likely to respond to a questionnaire relating to timber that those who do not use the material.

On average, it would appear that timber accounts for approximately 30% of the work undertaken by respondents.

Implication: Values presented for the amount of work undertaken involving timber should be viewed as being greater than the overall consensus of practicing Structural Engineers operating within the UK.

Further Action: Further surveys should be extended to consider the wider Engineering community and in particular parties who do not currently design with timber but can see the value of doing so. The information provided by such Engineers would be of great use to removing the barriers preventing the specification of structural timber.

Question 2, 3 and 4 - Agreement with "Thinking outside of the Box " report

"Do you agree or disagree with the following statements:

Q2. Knowledge of timber engineering within professional teams is lacking

Q3.Perceptions of timber often overrule reality. This means that the idea of using modern wood building solutions can often be stifled in the early stages of design

Q4.The lack of centrally available 'tables' similar to those widely promoted by the concrete and steel industry, means that timber is seen as a riskier choice for designers"

Purpose of Question: These questions have been developed based on the key findings of the "Thinking Outside the Box" report. Responses will be used to further validate the findings of this report.

Results: Results to these questions are presented in Figure I- 3.

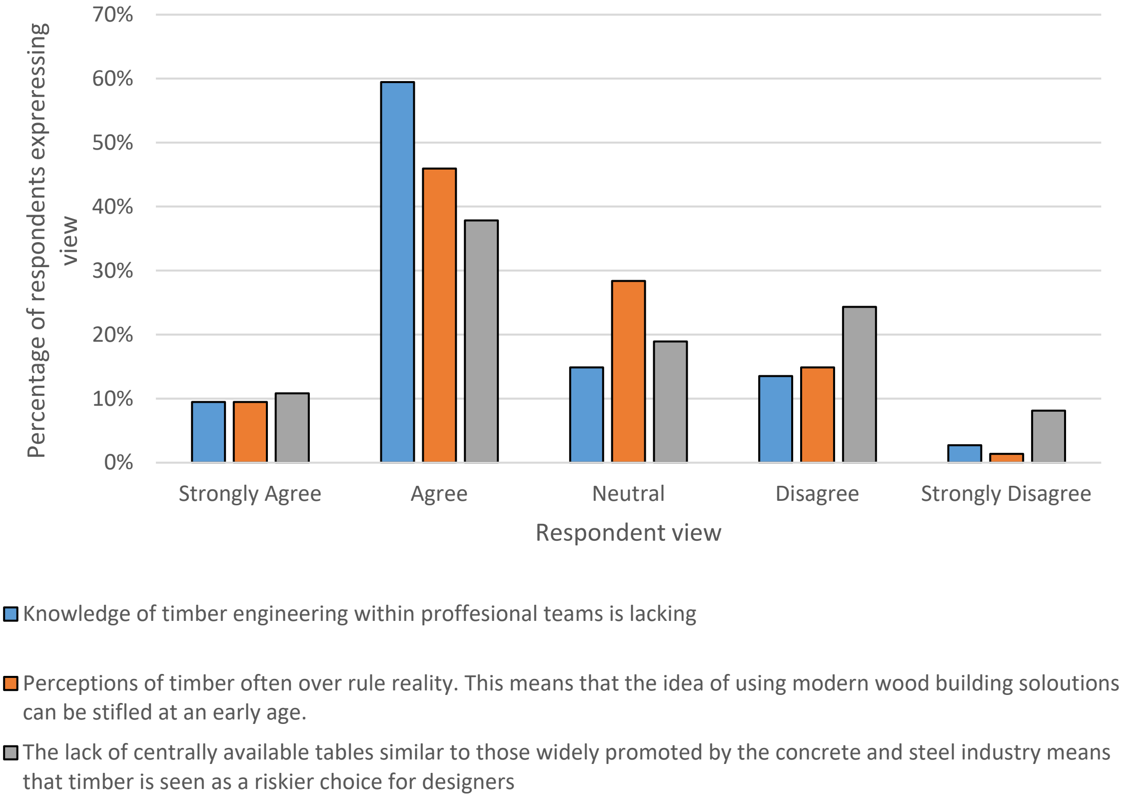


Figure I- 3 Summary of responses to Q2, Q3 and Q4

Analysis: Respondents generally agreed with the findings of the "Thinking Outside of the Box" report with 59% of all respondents agreeing that knowledge of timber engineering amongst professional teams was lacking. This is of interest due to the fact that as already highlighted – the results of the questionnaire are skewed towards those already using the material indicating there may be a lack of knowledge amongst those who already work predominantly with timber.

The greatest disagreement was shown in relation to the lack of centrally available design tables being one of the barriers to specification of timber with 24% of respondents disagreeing with the statement given and 8% strongly disagreeing.

Q3 provoked the greatest degree of neutrality – possibly as a result of the unclear wording or lack of directness (the statement is reproduced as it appears in the Thinking Outside of the Box report)

At the extreme ends of opinion, the number of respondents strongly agreeing with the statements remains consistent at approximately 10% whilst those strongly disagreeing with the given statements is less than this.

Implication: Generally speaking, the conclusions of the Thinking Outside of the Box report align with the opinions of those surveyed thereby giving additional validation to the report.

The predominant result from these questions is that there is a perceived lack of knowledge amongst design teams. The parameters of the question do not allow for the nature of the type of knowledge which is lacking to be determined – this may be relevant to further surveys to be undertaken.

Further Action: Utilise findings of the survey in conjunction with Thinking Outside of the Box report in support of future funding applications. Quantify the nature of the knowledge which is perceived to be lacking and make best actions to address this shortfall.

Question 5 - Use of Eurocode 5

<u>"Are you using Eurocode 5, Design of timber structures?"</u>

Purpose of Question: To determine the uptake of the BS EN 1995-1-1 Eurocode 5 (EC5) document amongst respondents as a means to identifying the most effective route for future developmental work.

Results: Responses to Question 5 are presented in Figure I- 4.

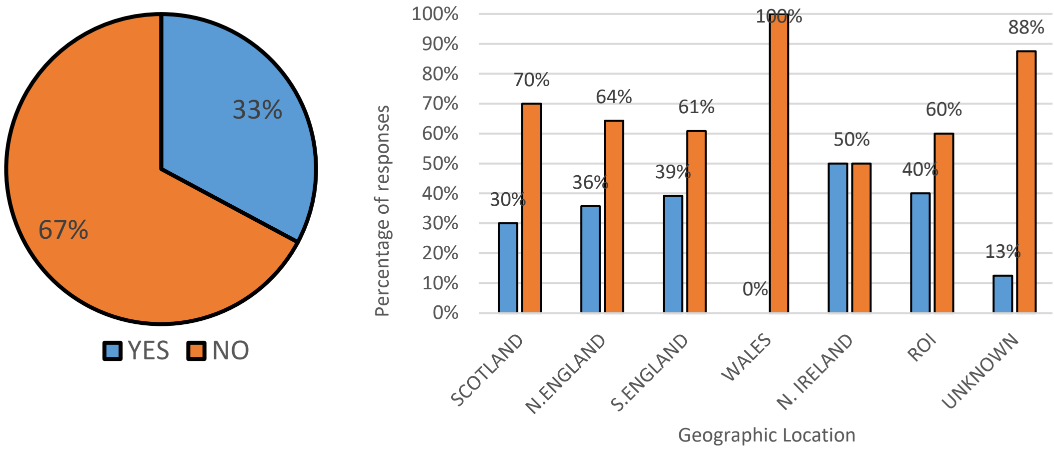


Figure I- 4 Overall responses (Left) and responses for geographic locations in the UK (Right) for Question 5: " Are you using Eurocode 5 Design of Timber Structures?"

Analysis: Approximately 1/3 of respondents state that they are currently using EC5. The remaining 2/3's do not specify what design code they have adopted in its place but it is postulated that BS5268 is still the preferred method of design (for the UK at least). This general trend was shown to exist across the country.

Implication: The uptake of EC5 in the UK is shown to be limited, even amongst respondents who are pre-disposed towards the use of timber. The value of developmental work related to EC5 may be reduced as a result of the fact that results indicate that the majority of the Engineers are likely to still be using the superseded BS 5268 standard.

Further Action: Further work undertaken in relation to EC5 should be aimed at further facilitating the introduction and use of the standard.

Question 6 - Facilitation of Eurocode 5

"What would facilitate the use of Eurocode 5?"

Purpose of Question: To identify areas to target for future programs of work facilitating the introduction of EC5.

Results: The responses to Question 6 have been summarised and presented in Figure I-5. Further to the prescribed answers respondents were also provided with the opportunity to add their own comments to what they saw to be the areas which would facilitate the introduction of EC5. Each of the responses are reproduced directly as part of Table I- 1 . A further in depth analysis of each response and their perceived implications are also provided.

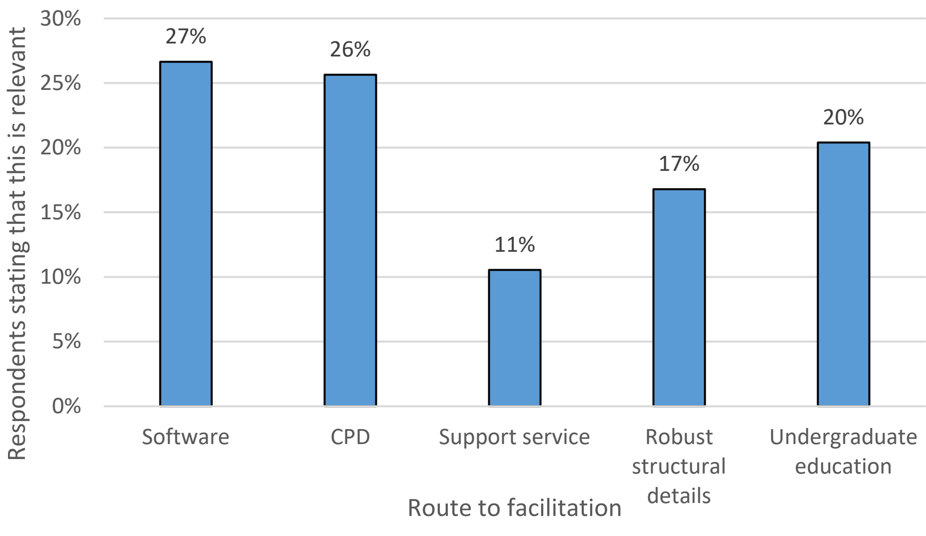


Figure I- 5 Responses to Question 6: "What would facilitate the use of Eurocode 5?"

Analysis: Results demonstrate that software and CPD are highlighted to be of primary importance to the uptake of EC5. It is interesting to note that undergraduate education ranks almost as highly as the two leading responses (*Software* and *CPD*). Due to the fact that respondents can answer positively to all questions, results are less definitive than could be hoped for and conclusive findings are somewhat lacking. However, based on a comparison of the results it would appear that the needs for a support services is not seen to be particularly pressing.

Opinions with respect to EC5 show stagnation with respect to its adoption. Although responses were brief and not developed to a great extent, they give a sense that there is a great deal of inertia which requires to be overcome for EC5 to be accepted. It appears that the change to EC5 must be forced upon the Engineering community otherwise it will quite happily continue to adopt tried and tested methods. Resistance to EC5 appears to be as a result of:

1. **Lack of user friendliness** - The level of departure from existing design methods, the perceived level of complexity in the calculations and its general lack of user friendliness
2. **Associated expense** - The expense associated with switching to a new design method
3. **Unfit for purpose** - Lack of confidence in the method and opinion that EC5 is "not fit for purpose"
4. **Offers little perceivable advantage** - Lack of clarity as to what advantage the is offered by the method

Views of individual engineers with respect to Question 6

Table I- 1 Views of individual engineers with respect to Question 6

Direct Quote	Analysis	Implication/Action
"Good, detailed design guides"	Further need for published information and technical notes	Possibly an inclusion of technical notes as part of Tedds calculation (in line with Eurocodes+ approach)
"the need to design timber structures"	Respondent specified that they 0% of their work involves timber	EC5 may be used if this engineers does begin to undertake work with timber
"Requirement by professional institutions for their members to stay up-to-date with changes in the industry, rather than allowing some of their members to ignore them."	Push required from professional bodies such as IStructE. Indicative of the fact that unless forced to change the industry will continue to adopt established practice	Engineers must be "pushed" into adopting EC5. The perceived advantages of using the new code will not be sufficient on their own to create sufficient "pull" to encourage change.
"Reduction in costs of guides"	Self-explanatory – although it is unclear if this is in reference to the price of the actual codes (EC5 approx. £225) or the price of the guides published in the support of them	EC5 is inherent to the calculations developed as part of Tedds therefore the necessity to purchase the codes is negated – further to this information and guidance could be included as part of commonly adopted design software
"company standard - due to move across to Eurocodes soon"	Respondent currently adopts code other than Eurocode in line with company practice but will move to EC5 soon	A movement already exists amongst larger companies towards switching to EC5

"tradition, experience, precedents"	Unclear – possibly meaning that once the use of the codes is established further that this in itself will promote a further increase in uptake	Engineers are well experienced with the current design standards which they employ – the lack of experience with respect to EC5 is a barrier to its acceptance
"The suitability of timber for the projects I work on (large span bridges)"	Unclear – possibly that it is the limitations of the material itself that prevent the respondent from specifying it and therefore using EC5	Increasing awareness of the capabilities of timber required (timber can/has been used for long span bridges)
"Time"	Unclear – possibly time taken to retrain or to undertake the actual calculation process itself or the time associated with retraining in a new design method	Offers an opportunity to highlight the role of software programs in streamlining the design process
"Racking and Masonry Shielding need to be sorted to bring these into line with BS5268 figures at least."	Specific technical criticism of the design method	There still exists a preference for tried and tested methods
"Certification"	Unclear – possibly that the use of EC5 would increase if it was made mandatory for certification purposes.	Pressure to adopt Eurocode is required from external parties i.e Engineers required to be "pushed" into using it
"Industry Consensus"	If EC5 was to be adopted as industry norm then this would further promote uptake	Industry consensus may be promoted through bodies such as ISTRCUTE and ICE
"Full withdrawal of British stand - But I saw the introduction of Eurocodes as pointless"	Engineers have to be given no alternative but to adopt Eurocode – again this is consistent of the notion of being forced into adopting it	The fact that British Standards have now been superseded needs to be made clearer
"A useful Code of Practice, EC5 is worthless."	Lack of faith in the method	The benefits and usability of the Eurocodes requires to be highlighted

Implication: It can be surmised from these results that all options presented are valid means of increasing the uptake of EC5. The work being undertaken By COCIS offers a route to address most of the issues highlighted

Further Actions:

Table I- 2 Further action identified as a result of responses to Question 6

Issue	Solution
Lack of user friendliness	Through automating the calculation process and introducing easy to use user interfaces EC5 can be rendered user friendly
Associated expense	Economic cost of EC5 not addressable through the work of COCIS but through developing calculation on an existing software platform the need to purchase a physical hard copy is reduced
Unfit for purpose	Being an approved and reviewed standard EC5 has been declared to be fit for purpose. It is postulated that the view that it is unfit for purpose is as a result of its perceived level of complexity – particularly in relation to the more user friendly BS 5268. Automation of calculation process via Tedds will address this issue.
Offers little perceivable advantage	The advantage of EC5 are its analytical basis and the fact that it offers the user the ability to input their own variables. The ability to do this can be illustrated via Tedds.

Question 7 - Structural software

<u>"What Structural Software do you use?"</u>

Purpose of question: To determine the structural software platforms commonly utilised by engineers as a means to identify those which provide the most direct route to the end user.

Results: Responses to the question showed a wide and varied range of software is currently employed by Engineers. The survey has revealed that 38 differing platforms are employed by engineers.

See Figure 4-2

Analysis: 57% of respondents use CSC (Tekla) Tedds software making it by far the most commonly used software platform. A further 6% of respondents used other CSC (Tekla) software platform meaning that CSC (Tekla) software is adopted by 63% of structural engineers in the UK. Bentley's STAAD PRO and Master series software are the second most commonly used amounting to 6 percent of the respondents. Responses make it clear that there are a wide variety of platforms adopted by the respondents to the questionnaire. No obvious trend other than the prevalence of CSC (Tekla) software appears to exist – the majority of software platforms identified being utilised by a single respondent.

Implications: CSC (Tekla) Tedds software has been shown to be the most commonly utilised structural software platform and therefore the one providing the most effective route to practicing engineers.

Further Action: Utilise CSC (Tekla) Tedds as an effective route through which to facilitate the structural timber design process.

Question 8 - Technical information requirements

<u>“Would you be more likely to specify timber and timber related products if the required technical information was freely accessible? “</u>

Purpose of Question: To determine if the lack technical information on timber and timber products is a barrier to its specification.

Results:

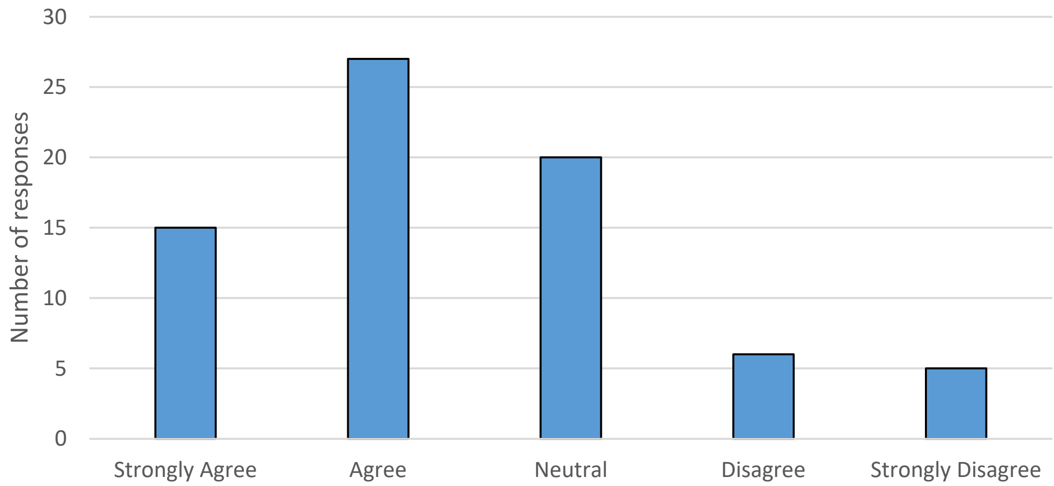


Figure I- 6 Responses to Question 8: “Would you be more likely to specify timber and timber related products if the required technical information was freely available?”

Analysis: Responses were geared towards agreeing with the outlined statement with the majority of respondents (57.5%) either agreeing or strongly agreeing that the free ability of technical information would increase their likely hood of specifying timber for structural purposes. Approximately 1/3 of respondents (27.4%) expressed no definitive opinion whilst 8.2% disagreed and 6.8% strongly disagreed with the statement.

Implication: The results provide evidence – although by no means conclusive - that a lack of technical information is one of the barriers to the specification of home-grown timber.

Further Action: Further utilise existing routes whilst identifying further ones which may be used to increase the flow of technical information to practicing Structural Engineers.

Question 9 - Standardised details

<u>"Give one example of a timber structural detail that you would like to have standardised information for"</u>

Purpose of Question: To determine what technical details engineers would like to see presented in a standardised easily specified format in order that they me be incorporated into ongoing work – namely their incorporation within the Tedds library of calculations.

Results: Responses from those surveyed have been directly reproduced and categorised according to subject area and are presented in

Table I- 3 Responses to Question 9: "Give one example of a timber structural detail that you would like to have standardised information for"

Category	Detail highlighted by response
Members	*"Flitch beams"/ "Columns"/ "Columns"/ "Joists, trusses"*
Connections	*"nailed and bolted joints"/ "Shear connections including nailing and screwing details"/ "Bolted connections"/ "Connections"/ "Typical Connection Details"/ "Moment connection details"/ "Main workstream is in rail bridges and ancillary structures. End connections usually problematic on waybeams."/ "Typical joint details"/ "Connections"/ "Moment connections"/ "Connections generally."/ "Connections. Of all types. Something like the steel green book."/ "Beam to column Connections"*
Wall Details	*"Racking panels anchorage detailing/sole plate fixings."/ "Sole Plate Fixings and Panel Bottom rail fixings to Sole Plates."/ "Internal wall parallel to floor joists where wall is required for racking resistance"/ "Floor to wall detail"/ "pinned fixing for curtain wall including 3-dimensional tolerances adjustment."/ "Tie between wallhead and floor"/ "Timber frame sole plate fixings"*
Other Details	*"Column beam connection"/ "Column bases"/ "Details to prevent disproportionate collapse"/ "standard details for CLT"/ "BRIDGE DECKING"/ "Bolt groupings at joints"/ "C16 lintels and cripple stud load tables."/ "Hip apex joint."/ "Splicing new joist ends to cut out rotten joists."*
Systems and components	*"Portals"/ "Multi-storey medium rise timber frame flats"/ "trusses"/ "SIPS"/ "Racking panels to EC5"/ "trusses"*
Miscellaneous	*"None. Ensuring the competency of the contractor would be better."/ "Timber frame settlement"/ "skeleton"/ "Not applicable"/ "All details are different depending on the structure. Generally we have standard details in house for timber"/ "None"*

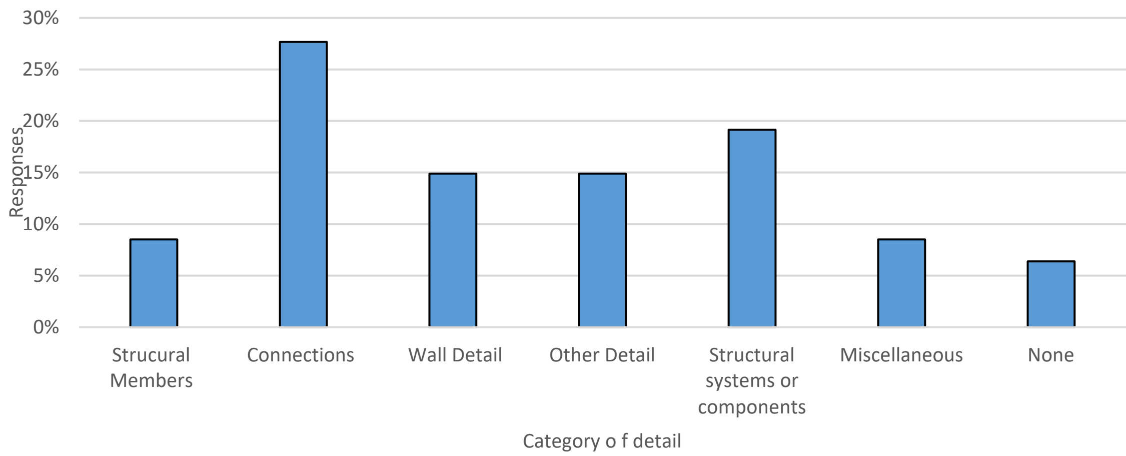


Figure I- 7 Responses to Question 9: "Give one example of a timber structural detail that you would like to have *standardised information for"*

Analysis: Those surveyed indicated that connection and connection orientated details are the most desirable standardised detail. The requirement for standardised details related to walls detailing for racking capacity should also be noted. The responses given could be interpreted to infer that the majority of respondents are involved in the design of domestic housing.

Implication: Due to the prevalence of connections and their critical nature in the design of timber structures it is to be expected that the most requested standardised details are related to this area.

Further Action: Development of streamlined methods for the calculation of timber connections and associated details based on further input from practicing Engineers.

Question 10 - Attitudes to BIM

"Are you using BIM, and if so to what level"

Purpose of Question: To determine the BIM level at which the respondents are operating as a means to assess the future receptiveness to the development of a "Mass Customised approach to Timber Engineering" using the approach

Results: Responses to the question are presented in Figure I- 8 and are presented so as to reflect the two aspects of the question; i) Are you using BIM and ii) if so to what level? The data presented in reference to ii) has been gathered from the respondents who answered "Yes" to i).

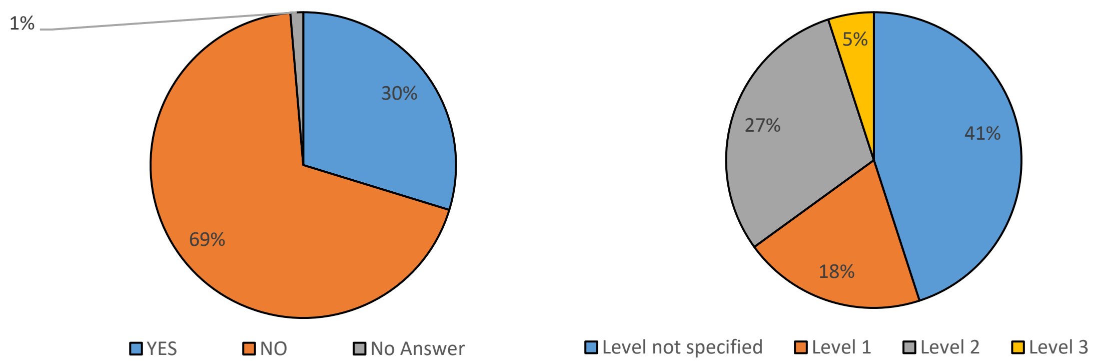


Figure I- 8 Responses to Question 10: "Are you using BIM (Left) and if so to what level (Right)?"

Table I- 4 Additional information given in response to Question 10 is presented as follows:

Details of responses given to Question 10	*"For building services design and detailing only."/ "Technicians in office are beginning to use BIM" /"Infrequent-job specific" /"occasionally" /"Structural steel design and detailing" /"Basic induction course. Looking to adopt Revit for all projects within the company in the near future." /"Timber frame models are bin models as information is taken from them. Citing lists etc. Is not integrated with structural software though." /"10% of projects" /"Early days." /"Drawings" /"I don´t use it directly but work in projects that implement it" /"Aiming to get to LEVEL 2. but currently running at a pseudo level 1.8/1.9. ie not supporting cobie "*

Analysis: It would appear that the vast majority (69%) of those surveyed are not implementing BIM in anyway. Of the 30% who are implementing BIM, it would appear that this is only to a limited degree or level. Of those implementing BIM, only 27% are operating at Level 2 and only 5% (1 respondent) of those surveyed are operating at Level 3. The majority of those indicating that they are using BIM did not indicate to which level they were employing it too and it is postulated that is as a result of BIM being implemented at a minimal level i.e the use of AutoCAD or Revit being taken as BIM compliancy.

Implications: It would appear that those involved in the Timber Engineering industry could be better positioned to take advantage of BIM. In general, the sector has not engaged with BIM – this could be attributable too:

- A general unawareness or lack of understanding as to what BIM actually involves or entails
- The nature of projects which utilise timber – timber is not typically used in the large scale projects where BIM may be adopted.
- Engineers who specialise in timber tend to be small practices or "one man bands" i.e parties who are not set up to implement a BIM strategy

As well as indicating a general lack of uptake of BIM, those who are actually implementing it appear to be doing so at the very lowest level. Responses show little evidence of further development of BIM beyond the levels attainable by default through the use of AutoCAD or Revit.

Further Action: Undertaken further research/ questionnaires to determine if BIM is relevant to timber engineering and also to identify if the creation of a BIM framework for timber will promotes its use amongst larger organisations that may be adopting BIM in earnest.

Further Analysis: Data from "High Timber Use" respondents (≥50%) to survey

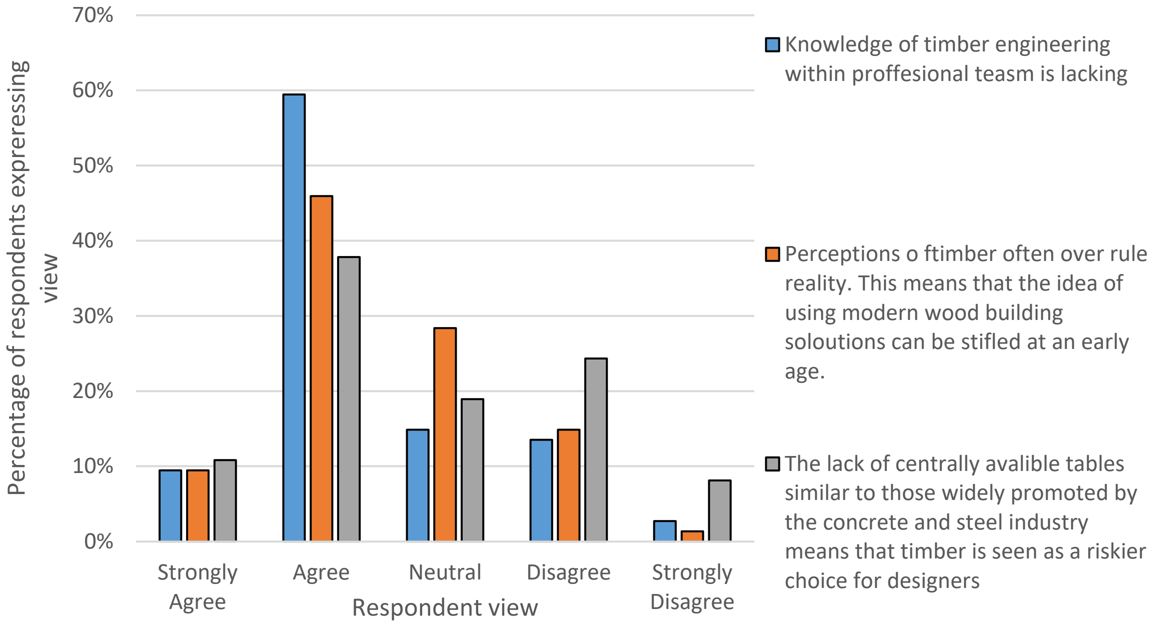
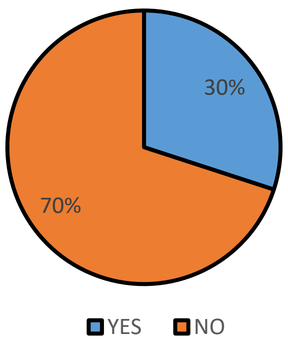


Figure I- 9 Data from "High Timber Use" respondents to Q2, Q3 and Q4



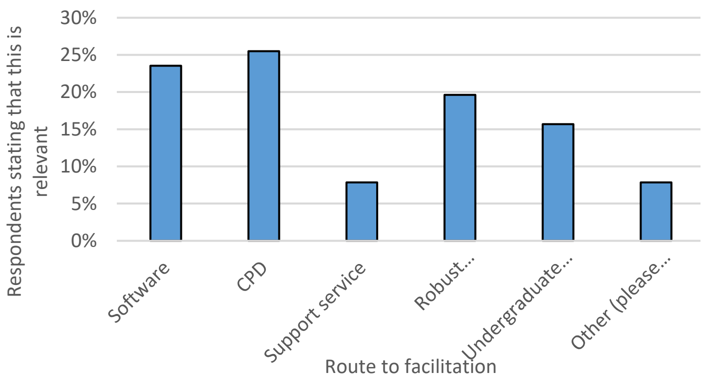
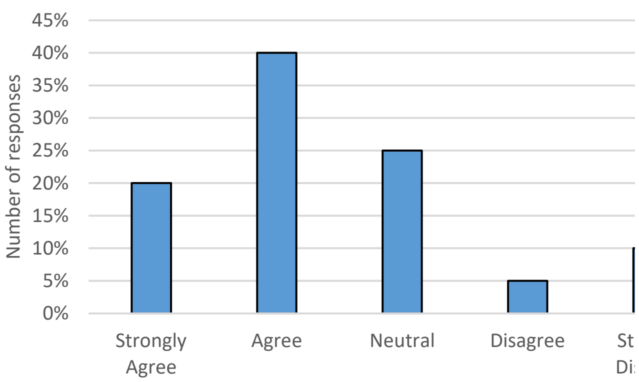


Figure I- 10 Data from "High Timber Use" respondents: Q5 "Are you using EC5?" (left) and Q6 "What would facilitate the use of EC5?" (right)



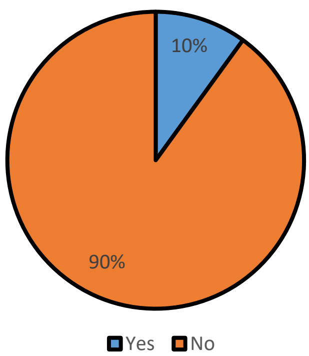


Figure I- 11 Data from "High Timber Use" respondents: Q8 "Would you be more likely to specify timber and timber related products if the required technical information was freely accessible?" (left) and Q10 "Are you using BIM?"

Further Analysis: Data from "Low Timber Use" respondents (≤50%) to survey

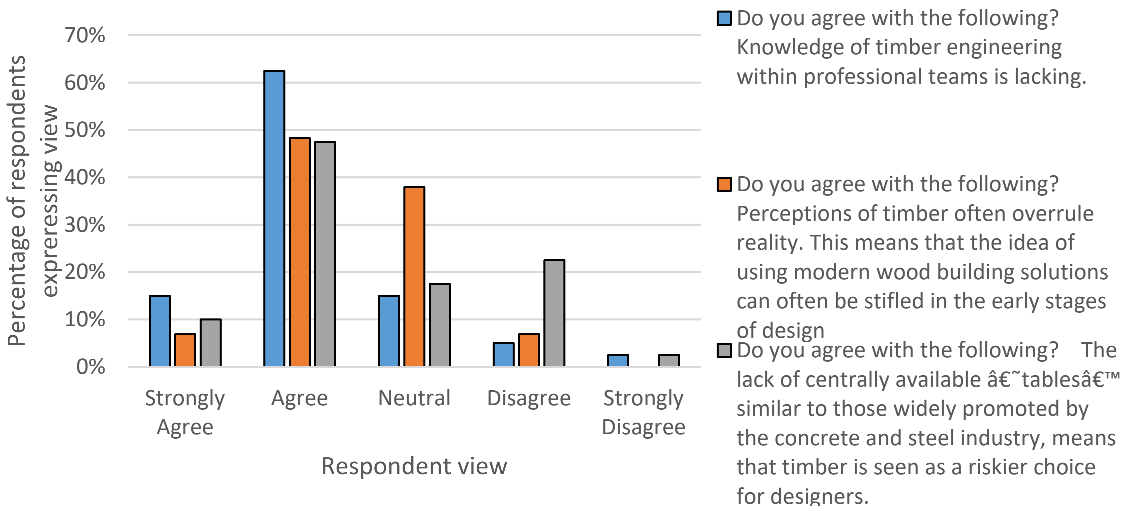


Figure I- 12 Data from "Low Timber Use" respondents to Q2, Q3 and Q4.

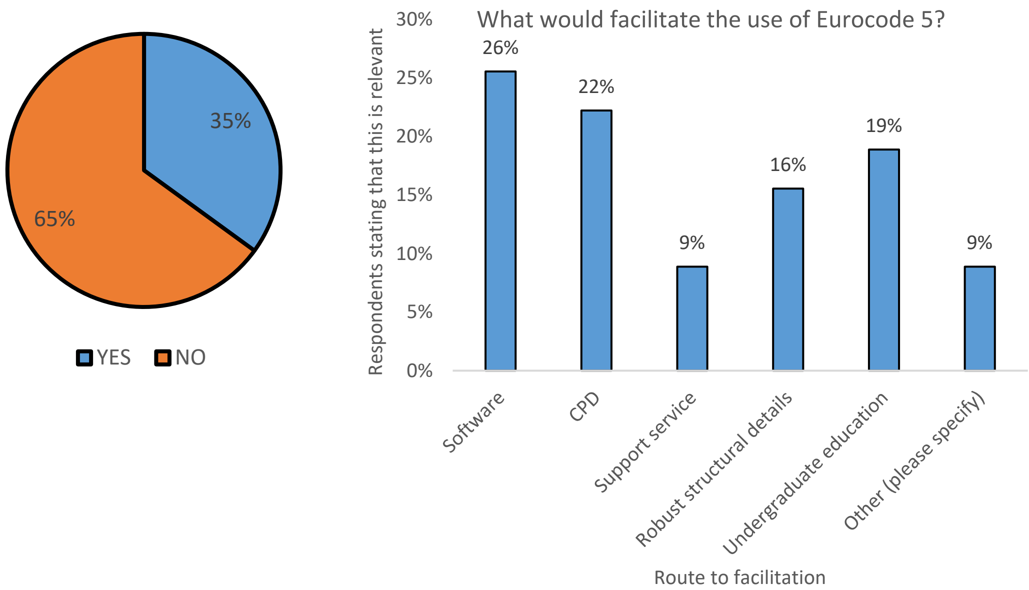
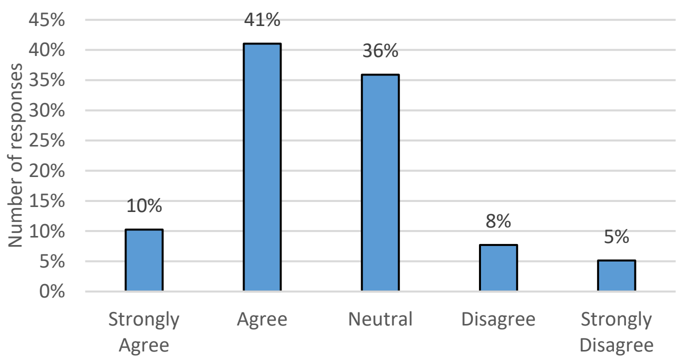


Figure I- 13 Data from "Low Timber Use" respondents: Q5 "Are you using EC5?" (left) and Q6 "What would facilitate the use of EC5?" (right)

Amongst those specifying a high proportion of their work is undertaken using timber, 35% of respondents use Tedds software.



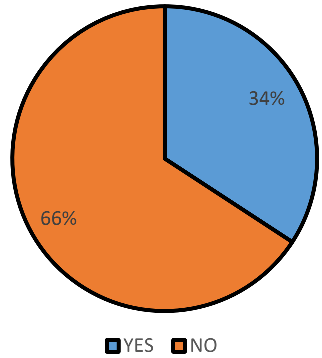


Figure I- 14 Data from "Low Timber Use" respondents: Q8 "Would you be more likely to specify timber and timber related products if the required technical information was freely accessible?" (left) and Q10 "Are you using BIM?"

Further Analysis: Data from respondents using EC5

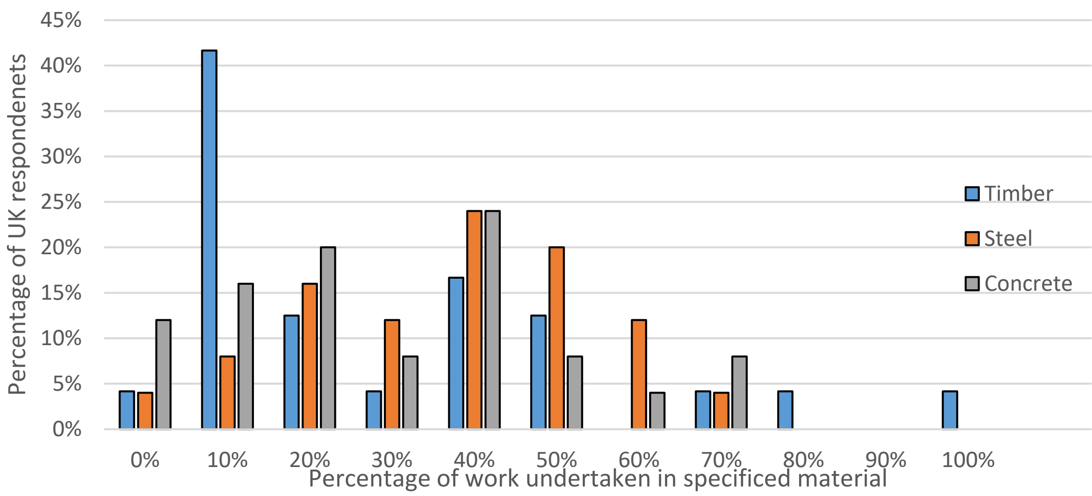


Figure I- 15 Data from respondents using EC5 for Q1

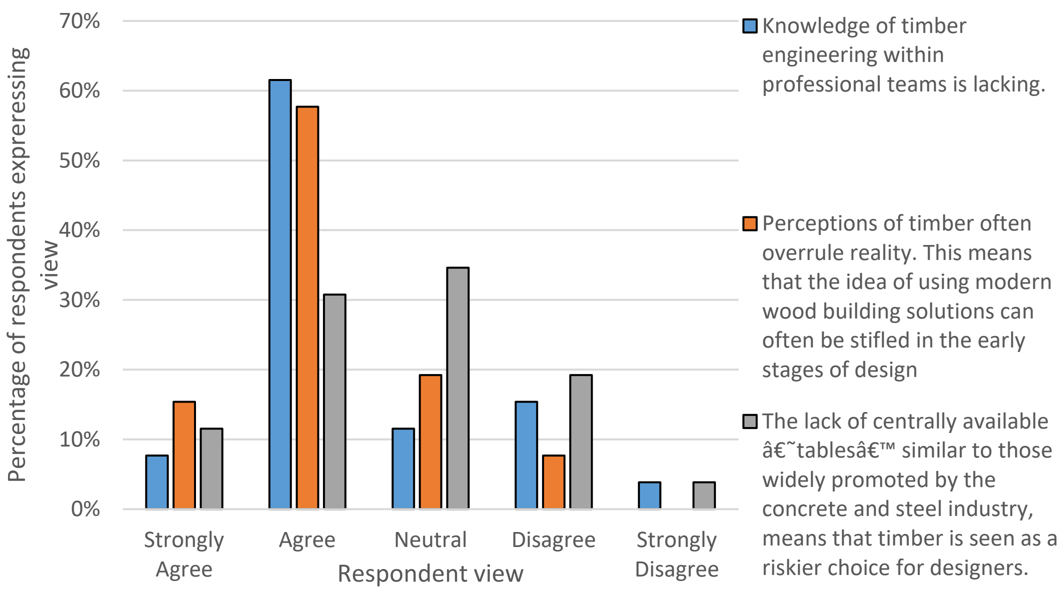
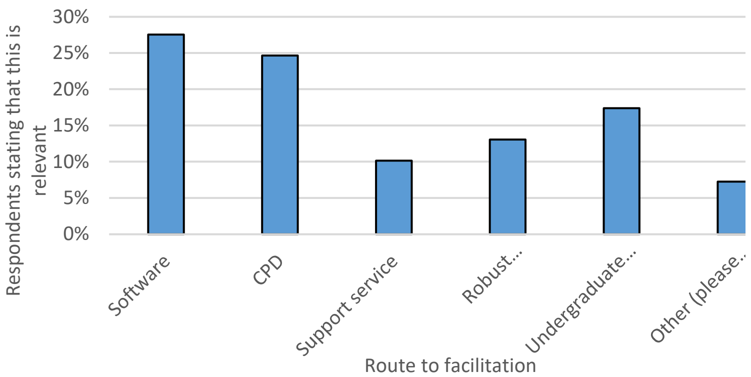


Figure I- 16 Data from respondents using EC5 to Q2, Q3 and Q4.

46% of respondents using EC5 indicated that they use Tedds software.



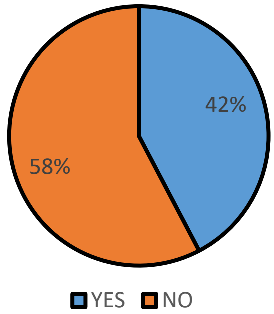


Figure I- 17 Data from respondents using EC5 to Q8 "Would you be more likely to specify timber and timber related products if the required technical information was freely accessible?" (left) and Q10 "Are you using BIM?"

Further Analysis: Responses from Europe

The survey was made available online via Linkidin and was therefore open to international recipients. A total of 19 respondents from mainland Europe completed the questionnaire 6 responses came from Spain, 3 from Germany and Italy, 2 from Romania and a single response from Belgium, Denmark, Greece, Poland and the Netherlands.

A further 53 respondents from around the world replied to the questionnaire, given the diversity of the locations and the nature of the responses this information has not been considered as part of this study.

Results, analysis and conclusions drawn from the European respondents are collectively presented as follows:

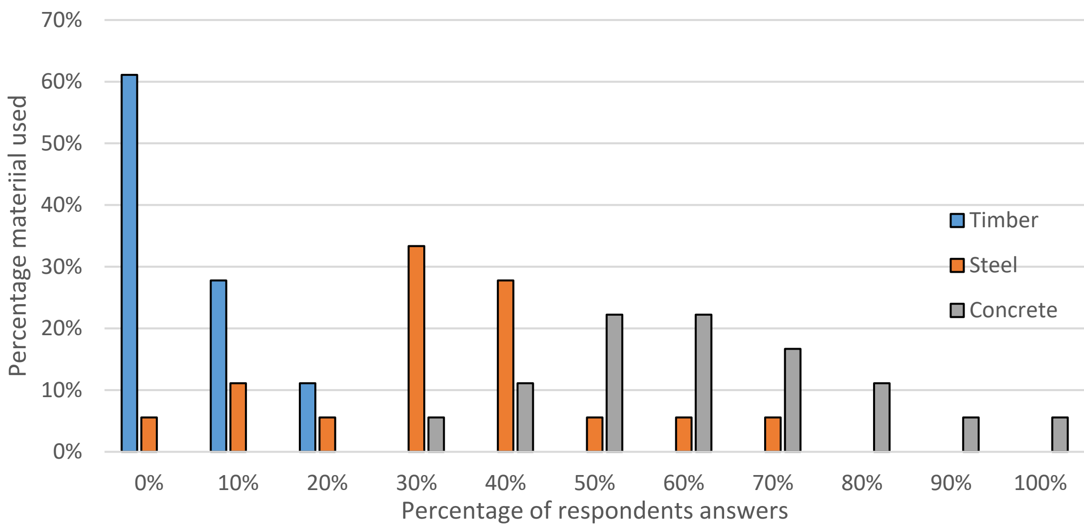


Figure I- 18 European responses to Q1: "Approximately what percentage of your work is undertaken in the following *materials?"*

The percentage break down of those using timber revealed that amongst the respondents it was not widely utilised with over 60% of respondents indicating that they do not undertake any sort of work involving the material. The maximum percentage of work using timber was indicated to be 20%. Concrete would appear to be the predominant material specified by those completing the questionnaire.

View to Questions 2, 3 and 4 show those responding to the questionnaire from mainland Europe agree with the principles of the Thinking Outside of the Box report giving further credence to findings of the report.

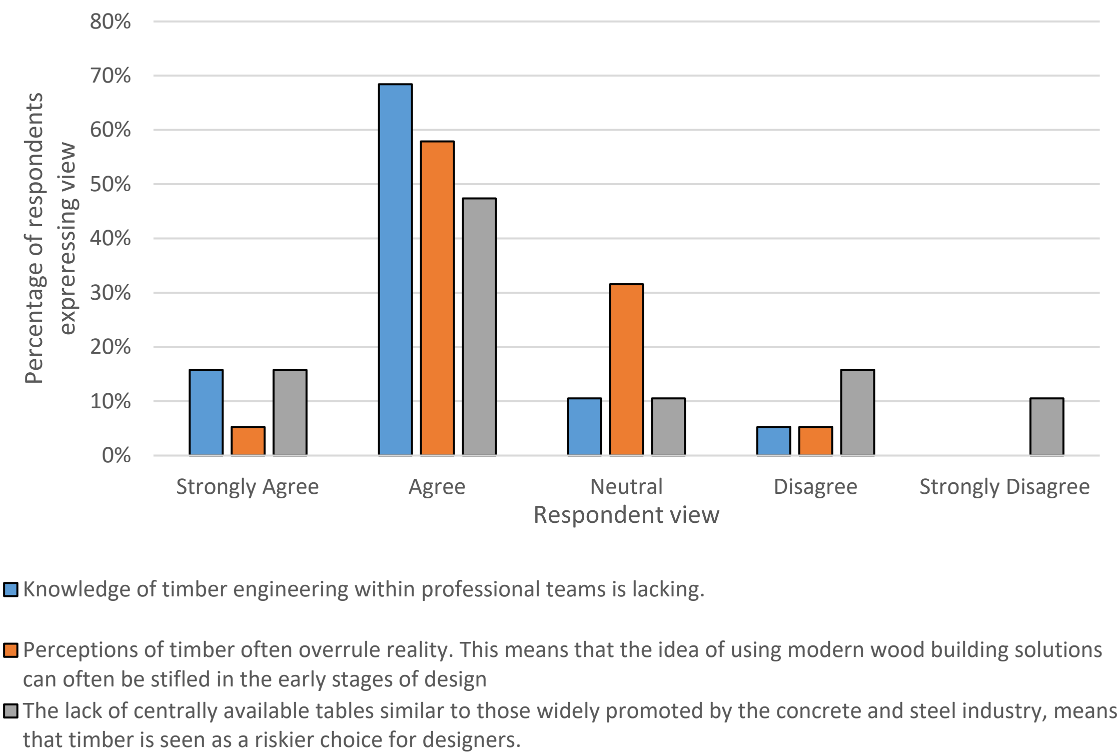


Figure I- 19 European responses to Q2, Q3 and Q4

Despite the fact that a minority of respondents indicated that they undertook work with timber it would appear that those designing with it do so in accordance with Eurocode 5. It is suggested that the 68% indicating that they do not use EC5 do so because they do not undertake any work using timber. Such results would also suggest that the Eurocode suite of design methods have been more widely accepted and are employed to a greater degree across mainland Europe than in the UK.

In terms of measures that could be employed to facilitate the use of EC5 software development and undergraduate training showed a number of positive responses with 53% and 58% respectively.

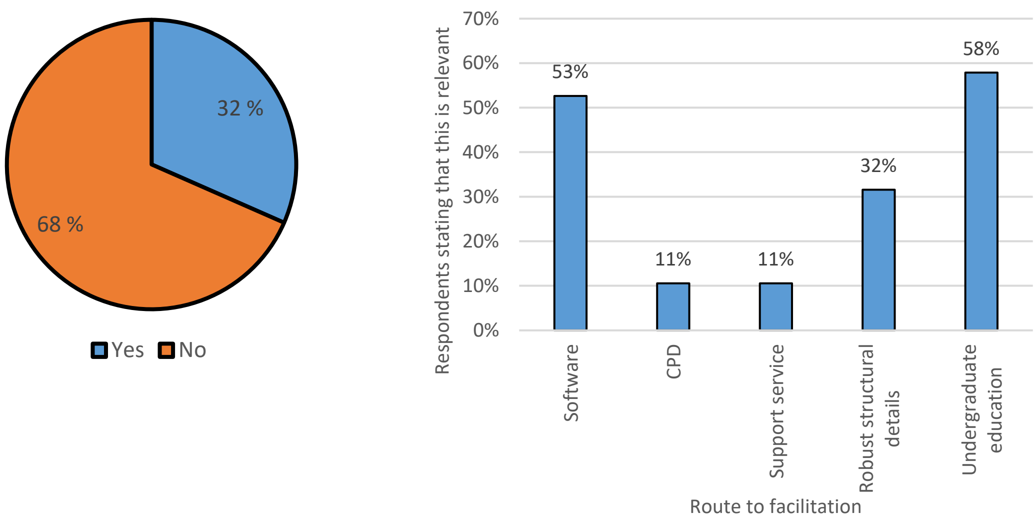


Figure I- 20 European responses to Q5 and Q6: "Are you using Eurocode 5?"(Left) and "What would facilitate the use of Eurocode 5?"(Right)

Examples of details the respondents would like to see included are given in Table I- 5. In line the responses from the UK the requirements for standardised connections details features highly. The nature of the standardised details presented could be taken to indicate that timber is utilised more for the purposes of Civil Engineering and infrastructure projects (i.e bridges) rather than domestic housing as indicated by UK respondents.

Table I- 5 Responses to Question 9 "Give one example of a timber structural detail that you would like to have standardised information for"

Details of responses given to Question 9	*"pedestrian bridge parapet composite joint detail (timber+ steel or concrete) Timber"/ "roof-truss systems"/ "Timber connections" / "unnecessary" / "Base column connection"/ "balustrades, guardrails /"Joints" /"Pedestrian walkway"/ "For instance, the dimensions and material properties of timber beams and not only the woodwork." /"Fixed bolted connections" /"joints details" /"joints"*

Responses to Question 10 would indicate that only 17% of European respondents utilise BIM – as opposed to 30% of UK respondents.

Appendix J. Survey respondents identified software used

List of used software identified by survey respondents

2D/3D Analysis	
Prokon	www.prokon.com/
Tekla: Fastrak, Orion, Solve	www.tekla.com
Autodesk: Robot	www.autodesk.co.uk/
RM-WIN	www.rm-win.pl/
Bentley: Sacs (oil and gas)	www.bentley.com/
Bentley: RAM steel	www.bentley.com/
Bentley: STAD Pro	www.bentley.com/
Fitzroy: Sand	www.fitzroy.com/
SESAM (offshore structures)	www.dnvgl.com/
S-Frame	www.s-frame.com/
Oasys	www.oasys-software.com/
Masterseries	www.masterseries.com/
ETABS	www.csiamerica.com/products/etabs
Graitec: Superstress	www.uk.graitec.com/supersuite/superstress/
Finite element analysis	
LISA (Free)	www.lisafea.com/
Plaxis	www.plaxis.com/
Lusas	www.lusas.com/
Altair: StrudCAD	www.altair.com
3Ds: Abaqus/simulia	www.3ds.com
MEP Design CAD	
ENCAD	www.encad.co/
Retaining wall analysis	
Wallap	www.geosolve.co.uk/
Timber engineering using steel connector products	
MiTek	www.mitek.co.uk/
Calculation pad and template calculations	
Tekla Tedds	www.tekla.com/uk/products/tekla-tedds
Template calculations	
Scale / SAM	https://fitzroy.com/

Appendix K. The Newly created Code Compliance Tedds output

1) Tekla TEDDS timber connections, references and revision history

Calculation references:

- Eurocode 5: Design of timber structures

  Part 1-1:General - Common rules and rules for buildings - EN1995-1-1:2004 + A1:2008 incorporating Corrigendum No.1
- Published Document PD 6693-1 as UK Non-Contradictory Complementary Information to Eurocode 5: Design of timber structures (2012 Publication).
- Boverket mandatory provisions amending the board's mandatory provisions and general recommendations (2011:10) on the application of European design standards (Eurocodes), EKS - BFS 2015:6 EKS 10
- Finnish National Annex NA to SFS EN 1995-1-1
- Irish National Annex NA to IS EN 1992-1-1:2004
- Norwegian National Annex NA to NS EN 1995-1-1:2004/NA:2010 + A1:2013
- UK National Annex NA to BS EN 1995-1-1:2004 + A1:2008 incorporating National Amendment No.1
- Structural timber - Strength classes - EN 338:2016
- Timber structures – requirements for dowel type fasteners - EN 14592, Edition 2012

Revision History

Version	Date	Description
	22nd January 2019	Added link to video demonstration in calculation notes.
1.1.07	30th August 2018	Enhanced the calculation to include options for a summary table and user defined output notes.
1.1.06	7th June 2018	Fixed summary output always calculating "Load utilisation factor" assuming single shear. Modified Show fields in various Calc Items to prevent result from appearing in the progress log.
1.1.05	15th May 2018	Initialise variables used for drawing Id's as temporary variables so that they do not affect automated testing of the examples.
1.1.04	15th December 2017	Fixed issue for the tension splice connection when using steel side members and screws the characteristic axial force capacity was incorrectly calculated.
1.1.03	9th November 2017	Added Irish, Swedish, Finnish and Norwegian national annexes.
1.1.02	1st September 2017	Fixed issue for the nail data lists. Fixed issue for the main to side member connection when using screws, the moment calculation was using the incorrect variable for tensile strength. Fixed issue for the main to side member and tension splice calculation, the withdrawal capacity and pull-through resistance was multiplied by the effective number of fixings.
1.1.01	9th January 2017	Enhanced output options to include an option to omit the fixing spacing table. Enhanced output options to include an option to omit the timber splitting calculation. Fixed main to side member connection not retaining the head diameter of the selected screw. Corrected clause references in output. Corrected multi-member connection transposing the Elevation and End Elevation sketches in the user interface and output.
1.1.00	8th September 2016	Enhanced to include multiple member; tension splice and axially loaded fixing connections. Minor corrections to the main to side member connection.
1.0.00	3rd May 2015	Original version

2) Tedds output: Main to side member - Screwed edge beam example

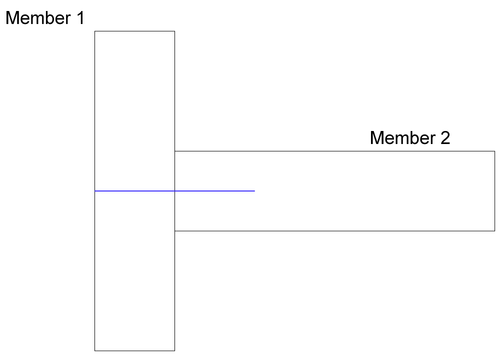
TIMBER CONNECTION DESIGN

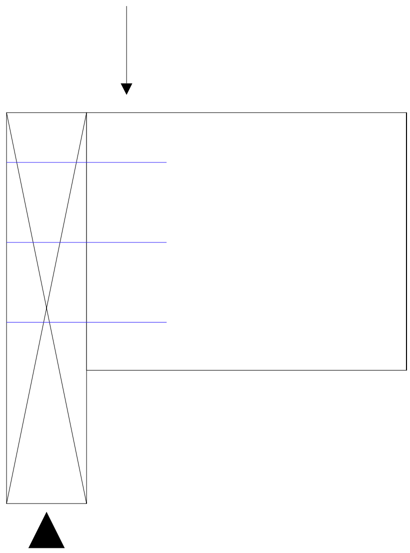
In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.1.07

Design summary

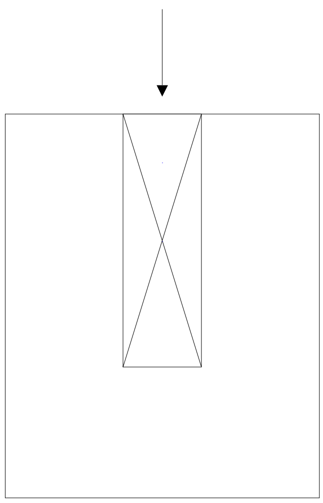
Description	Provided	Required	Utilisation	Result
pointside penetration	21.0 mm	35.5 mm	0.592	PASS
spacing/edge distances			0.875	PASS
load utilisation factor			0.628	PASS

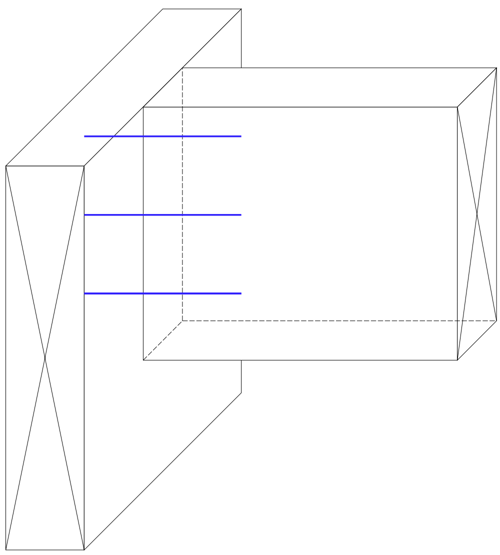




Plan

Elevation





End elevation

Edinburgh Napier University Edinburgh Napier, COCIS Unit 1, 7 Hills Business Park Bankhead Crossway South Edinburgh EH11 4EP	Project				Job Ref.	
	Section				Sheet no./rev. 242	
	Calc. by AL	Date 15 October	Chk'd by	Date	App'd by	Date

Geometric Properties

Member 1

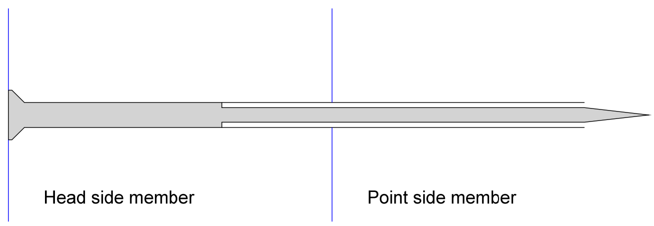
Breadth b_1 = **45** mm
Height h_1 = **220** mm
Cross sectional area A_1 = **9900** mm^2
Strength class C16
Characteristic tension strength parallel to the grain $f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber $\rho_{k.m1}$ = **310** kg/m^3

Member 2

Breadth b_2 = **45** mm
Height h_2 = **145** mm
Cross sectional area A_2 = **6525** mm^2
Rotation about the X-X axis 0 °
Rotation about the Z-Z axis 0 °
Strength class C16
Characteristic tension strength parallel to the grain $f_{t.0.k.m2}$ = **10** N/mm^2
Characteristic density of the timber $\rho_{k.m2}$ = **310** kg/m^3

Screws

Description 3.5 mm / 2.1 mm x 90 mm screw
Number of screws $N_{fixings}$ = **3**
Head diameter $d_{h.f}$ = **6.975** mm
Head length $l_{n.l.f}$ = **0.500** mm
Smooth shank diameter d_f = **3.50** mm
Outer thread diameter $d_{outer.f}$ = **3.50** mm
Inner thread diameter $d_{inner.f}$ = **2.05** mm
Total length l_f = **90.0** mm
Thread length, including the point $l_{th.f}$ = **60.0** mm
Point length $l_{point.f}$ = **9.0** mm
Total screw pointside penetration PsP = **44.50** mm
Tensile strength of each fixing $f_{u.fs}$ = **600** N/mm^2
Counter sunk head



Section of Screw

Smooth shank penetration $(4 \times d_{outer.f}) / (PsP - l_{th.f})$ = **-0.903**
Conditions of 8.7.1(2) are not met, effective diameter calculated in accordance with 8.7.1(3)
Effective screw diameter - cl 8.7.1(3) $d_{ef.f} = d_{inner.f} \times 1.1$ = **2.26** mm
Pointside penetration
non smooth shank - cl 8.7.1(2) $t_{pen} = PsP - l_{point.f}$ = **35** mm
Minimum penetration of fixing in main member $t_{min.pen} = 6 \times d_{outer.f}$ = **21.0** mm
$t_{min.pen} / t_{pen}$ = **0.59**

PASS - Pointside penetration is acceptable

Check to validate that no pre-drilling is acceptable

Characteristic dencity < 500 kg/m³ - cl 8.3.1.1(2) $\rho_{k.m1}$ / 500kg/m³ = **0.62** OK

$\rho_{k.m2}$ / 500kg/m³ = **0.62** OK

Diameter of fixing < 6mm - cl 8.3.1.1(2) d_f / 6mm = **0.58** OK

Timber thickness > t_{min} - cl 8.3.1.2(6)

t_{min} = max (7 × d_f, (13 × d_f - 30mm) × (max($\rho_{k.m1}$, $\rho_{k.m2}$) / 400kg/m³)) = **25** mm OK

PASS - Predrilling is not required

Partial safety factors

Safety factors - EC0 National Annex

Limit state (STR)

Permanent actions γ_G = **1.35**

Variable actions γ_Q = **1.50**

Safety factors – EC5 National Annex

Material factor for timber γ_M = **1.30**

Material factor for connections $\gamma_{M.connection}$ = **1.30**

Actions acting on member 2

Characteristic lateral action

Permanent vertical lateral action $G_{la.v.k}$ = **0.08** kN

Permanent horizontal lateral action $G_{la.h.k}$ = **0.00** kN

$G_{la.k} = \sqrt{(G_{la.v.k}^2 + G_{la.h.k}^2)}$ = **0.08** kN

Variable vertical lateral action $Q_{la.v.k}$ = **0.08** kN

Variable horizontal lateral action $Q_{la.h.k}$ = **0.00** kN

$Q_{la.k} = \sqrt{(Q_{la.v.k}^2 + Q_{la.h.k}^2)}$ = **0.08** kN

Design lateral action, EN1990 - eq 6.10 $F_{la.Ed} = \sqrt{((\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k})^2 + (\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k})^2)}$ = **0.23** kN

Characteristic axial permanent action $G_{ax.k}$ = **0.00** kN

Characteristic axial variable action $Q_{ax.k}$ = **0.00** kN

Design axial action, EN1990 - eq 6.10 $F_{ax.Ed} = \gamma_G \times G_{ax.k} + \gamma_Q \times Q_{ax.k}$ = **0.00** kN

Angle between the force and the grain direction α = 90 - atan(($\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k}$) / ($\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k}$)) = **90**°

Min. angle screw axis - grain direction point side α_{screw} = **0**°

Modification factors – Table 3.1

Service class of timber **1**

Load-duration **Short term**

k_{mod} = **0.90**

Embedment strength in timber - eq 8.15

Characteristic embedment strength, side member $f_{h.k.1}$ = 82 kNm/kg × $\rho_{k.m1}$ × ($d_{ef.f}$ / 1mm)$^{-0.3}$ = **19.92** N/mm²

Characteristic embedment strength, main member $f_{h.k.2}$ = 82 kNm/kg × $\rho_{k.m2}$ × ($d_{ef.f}$ / 1mm)$^{-0.3}$ = **19.92** N/mm²

Yield moment of screw - eq 8.14 $M_{y.Rk}$ = 0.30 mm$^{0.4}$ × $f_{u.fs}$ × $d_{ef.f}^{2.6}$ = **1491** Nmm

Withdrawal resistance

Penetration length of the threaded part $l_{ef.f} = t_{pen}$ = **35.50** mm

Characteristic values of the withdrawal and pull-through strengths

Withdraw capacity, User entered $f_{ax.k.pss}$ = **11.000** N/mm²

Associatated density $\rho_{a.ax.k.pss}$ = **350.00** kg/m³

Effective number of screws - eq 8.41 $n_{ef} = N_{fixings}^{0.9}$ = **2.688**

Withdrawal capacity - eq 8.40a — $F_{ax.point.Rk} = f_{ax.k.pss} \times d_{outer.f} \times l_{ef.f} / (1.2 \times \cos(\alpha_{screw})^2 + \sin(\alpha_{screw})^2) \times (\rho_{k.m2} / \rho_{a.ax.k.pss})^{0.8}$ = **1034** N

Headside pull-through, User entered — $f_{head.k.hss}$ = **9.40** N/mm^2

Associatated density — $\rho_{a.head.k.hss}$ = **350.00** kg/m^3

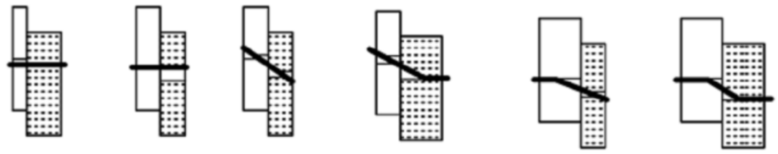
Pull-through resistance - eq 8.40b — $F_{ax.head.Rk} = f_{head.k.hss} \times d_{h.f}^2 \times (\rho_{k.m2} / \rho_{a.head.k.hss})^{0.8}$ = **415.003** N

$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{ax.head.Rk})$ = **0.4150** kN

Design value of axial withdrawal capacity — $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.2873** kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber connections:



a b c d e f

Embedment ratio — $\beta = f_{h.k.2} / f_{h.k.1}$ = **1.00**

Thickness headside member — $t_1 = b_1$ = **45** mm

Penetration length in pointside member — $t_2 = t_{pen}$ = **35** mm

Maximum rope effect contribution - cl. 8.2.2(2) & 8.3.1.2(4)

$P_{f.mod} = 2$

$P_{f.mod} - 1$ = **100** %

Rope = $F_{ax.Rk} / 4$ = **104** N

Failure mode (a) — $f.m.a = f_{h.k.1} \times t_1 \times d_{ef.f}$ = **2.02** kN

Failure mode (b) — $f.m.b = f_{h.k.2} \times t_2 \times d_{ef.f}$ = **1.59** kN

Failure mode (c) — $f.m.c_{Jyt} = f.m.a / (1+\beta) \times (\sqrt{[\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1)+(t_2 / t_1)^2) + \beta^3 \times ((t_2 / t_1)^2)]} - \beta \times (1 + (t_2 / t_1)))$

$f.m.c = \min(f.m.c_{Jyt} + Rope, P_{f.mod} \times f.m.c_{Jyt})$ = **0.86** kN

Failure mode (d) — $f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{[2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_1^2 \times d_{ef.f}))]} - \beta)$

$f.m.d = \min(f.m.d_{Jyt} + Rope, P_{f.mod} \times f.m.d_{Jyt})$ = **0.85** kN

Failure mode (e) — $f.m.e_{Jyt} = (1.05 \times (f_{h.k.1} \times t_2 \times d_{ef.f}) / (1 + 2 \times \beta)) \times (\sqrt{[2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_2^2 \times d_{ef.f}))]} - \beta)$

$f.m.e = \min(f.m.e_{Jyt} + Rope, P_{f.mod} \times f.m.e_{Jyt})$ = **0.71** kN

Failure mode (f) — $f.m.f_{Jyt} = 1.15 \times \sqrt{[(2 \times \beta) / (1 + \beta)]} \times \sqrt{[(2 \times M_{y.Rk} \times f_{h.k.1} \times d_{ef.f})]}$

$f.m.f = \min(f.m.f_{Jyt} + Rope, P_{f.mod} \times f.m.f_{Jyt})$ = **0.52** kN

Characteristic lateral nail shear resistance — **Failure mode (f)**

$F_{v.Rk}$ = **0.52** kN

Design resistance per fixing - cl 8.3.1.2(4) — $F_{v.Rd} = (k_{mod} \times F_{v.Rk}) / \gamma_{M.connection} / 3$ = **0.12** kN

Load utilisation factor — $ut\_load = F_{la.Ed} / (N_{fixings} \times F_{v.Rd})$ = **0.628**

PASS - Design resistance exceeds design load

Splitting capacity of timber

Loaded edge distance — h_e = **118** mm

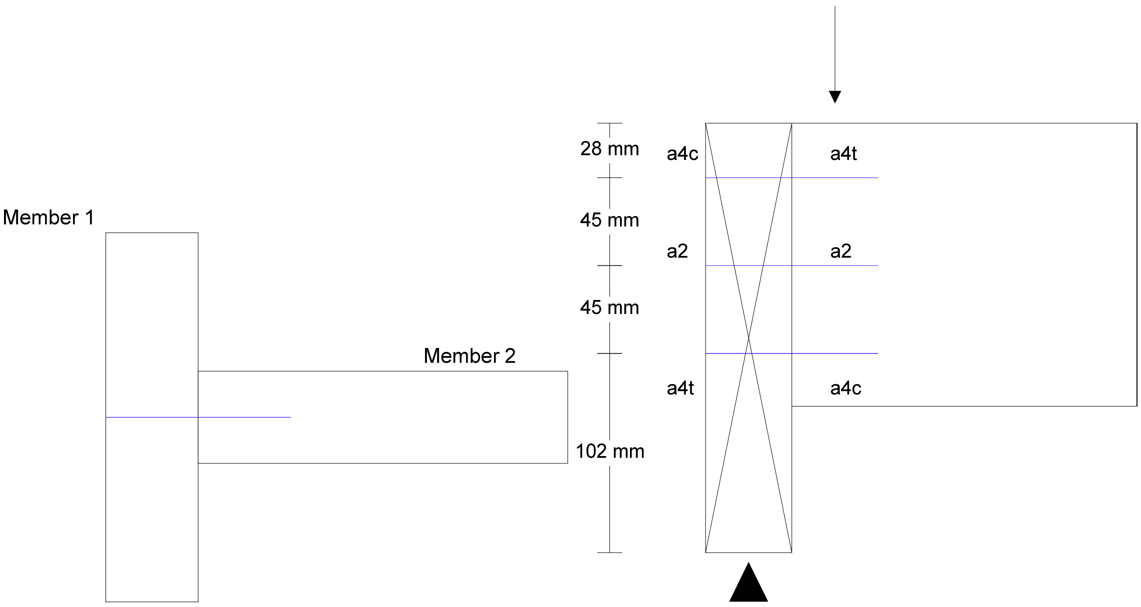
Characteristic splitting capacity - eq 8.4 — $F_{90.Rk} = 14 \times b_1 \times 1\ mm^{-1} \times \sqrt{(h_e \times 1mm^{-1} / (1 - (h_e / h_1)))} \times 1\ N$ = **10.05** kN

Design splitting capacity — $F_{90.Rd} = k_{mod} \times F_{90.Rk} / \gamma_{M.connection}$ = **6.96** kN

$F_{la.Ed} / F_{90.Rd}$ = **0.033**

PASS - Splitting capacity of timber exceeds the design force in member

Spacing



	Allowable minimum	Applied
Member 1		
Spacing of rows perpendicular to grain, a2: 5 × $d_{outer.f}$	17.5 mm	45.0 mm
Distance between fixing and loaded edge, a4t: (5 + 2 × sin α) × $d_{outer.f}$	24.5 mm	102.0 mm
Distance between fixing and unloaded edge, a4c: 5 × $d_{outer.f}$	17.5 mm	28.0 mm
Member 2		
Spacing of rows perpendicular to grain, a2: 5 × $d_{outer.f}$	17.5 mm	45.0 mm
Distance between fixing and loaded edge, a4t: (5 + 2 × sin α) × $d_{outer.f}$	24.5 mm	10000.0 mm
Distance between fixing and unloaded edge, a4c: 5 × $d_{outer.f}$	17.5 mm	10000.0 mm
Minimum breadth of member: 2 × a4c.m2	35.0 mm	45.0 mm

Allowable minimum from table 8.2

PASS - All spacing conditions are met

3) Tedds output: Main to side member - Nailed roof soffit example

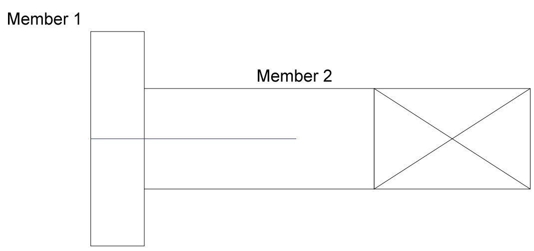
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

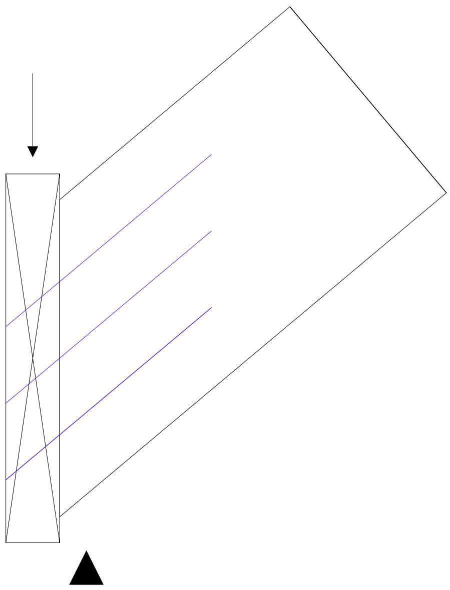
Tedds calculation version 1.1.07

Design summary

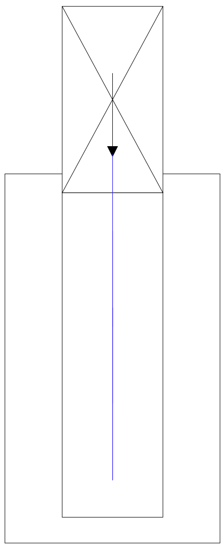
Description	Provided	Required	Utilisation	Result
pointside penetration	36.0 mm	44.6 mm	0.807	PASS
spacing/edge distances			0.933	PASS
load utilisation factor			0.247	PASS

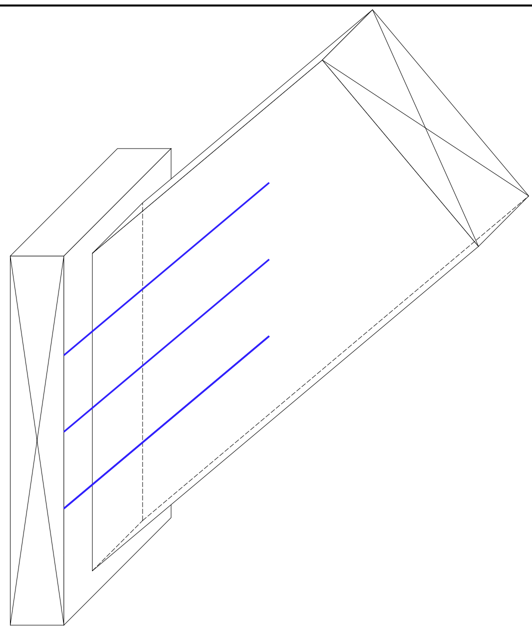


Plan



Elevation





End elevation

This is a Secondary Structure

(Note : An example of a secondary structure is a fascia board nailed to rafters)

<u>Geometric Properties</u>

Member 1

Breadth	b_1 = **32** mm
Height	h_1 = **220** mm
Cross sectional area	A_1 = **7040** mm^2
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m1}$ = **310** kg/m^3

Member 2

Breadth	b_2 = **60** mm
Height	h_2 = **145** mm
Cross sectional area	A_2 = **8700** mm^2
Rotation about the X-X axis	40 °
Rotation about the Z-Z axis	0 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m2}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m2}$ = **310** kg/m^3

Screws

Description	6.0 mm / 3.6 mm x 160 mm screw
Number of screws	$N_{fixings}$ = **3**
Head diameter	$d_{h.f}$ = **6.975** mm
Head length	$l_{n.l.f}$ = **0.500** mm
Smooth shank diameter	d_f = **6.00** mm
Outer thread diameter	$d_{outer.f}$ = **6.00** mm
Inner thread diameter	$d_{inner.f}$ = **3.55** mm
Total length	l_f = **160.0** mm

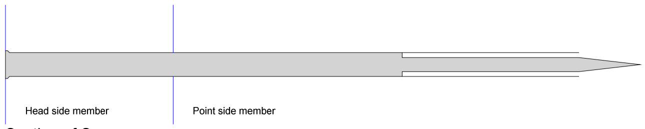
Thread length, including the point $l_{th.f}$ = **60.0** mm

Point length $l_{point.f}$ = **15.4** mm

Total screw pointside penetration PsP = **117.73** mm

Tensile strength of each fixing $f_{u.fs}$ = **600** N/mm^2

Counter sunk head



Section of Screw

Smooth shank penetration - cl 8.7.1(2) $(4 \times d_{outer.f})$ / $(PsP - l_{th.f})$ = **0.416**

Conditions of 8.7.1(2) are met, effective diameter equals smooth shank diameter

Effective screw diameter $d_{ef.f} = d_f$ = **6.00** mm

Pointside penetration, smooth shank - cl 8.7.1(2) $t_{pen} = l_{th.f} - l_{point.f}$ = **45** mm

Minimum penetration of fixing in main member $t_{min.pen} = 6 \times d_{outer.f}$ = **36.0** mm

$t_{min.pen} / t_{pen}$ = **0.81**

PASS - Pointside penetration is acceptable

Check to validate that no pre-drilling is acceptable

Characteristic dencity < 500 kg/m^3 - cl 8.3.1.1(2) $\rho_{k.m1}$ / 500kg/m^3 = **0.62** OK

$\rho_{k.m2}$ / 500kg/m^3 = **0.62** OK

Diameter of fixing < 6mm - cl 8.3.1.1(2) d_f / 6mm = **1.00** Not suitable

Timber thickness > t_{min} - cl 8.3.1.2(6)

t_{min} = max (7 × d_f, (13 × d_f - 30mm) × (max($\rho_{k.m1}$, $\rho_{k.m2}$) / 400kg/m^3)) = **42** mm OK

FAIL - Requires to be predrilled

<u>Partial safety factors</u>

Safety factors - EC0 National Annex

Limit state (STR)

Permanent actions γ_G = **1.35**

Variable actions γ_Q = **1.50**

Safety factors – EC5 National Annex

Material factor for timber γ_M = **1.30**

Material factor for connections $\gamma_{M.connection}$ = **1.30**

Actions acting on member 1

Characteristic lateral action

Permanent vertical lateral action $G_{la.v.k}$ = **0.08** kN

Permanent horizontal lateral action $G_{la.h.k}$ = **0.00** kN

$G_{la.k} = \sqrt{(G_{la.v.k}^2 + G_{la.h.k}^2)}$ = **0.08** kN

Variable vertical lateral action $Q_{la.v.k}$ = **0.08** kN

Variable horizontal lateral action $Q_{la.h.k}$ = **0.00** kN

$Q_{la.k} = \sqrt{(Q_{la.v.k}^2 + Q_{la.h.k}^2)}$ = **0.08** kN

Design lateral action, EN1990 - eq 6.10 $F_{la.Ed} = \sqrt{((\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k})^2 + (\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k})^2)}$ = **0.23** kN

Characteristic axial permanent action $G_{ax.k}$ = **0.00** kN

Characteristic axial variable action $Q_{ax.k}$ = **0.00** kN

Design axial action, EN1990 - eq 6.10 $F_{ax.Ed} = \gamma_G \times G_{ax.k} + \gamma_Q \times Q_{ax.k}$ = **0.00** kN

Angle between the force and the grain direction $\alpha = 90 - \text{atan}((\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k}) / (\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k}))$ = **90**°

Min. angle screw axis - grain direction point side α_{screw} = **0**°

Modification factors – Table 3.1

Service class of timber **2**

Load-duration **Medium term**

k_{mod} = **0.80**

Embedment strength in timber - eq 8.15

Characteristic embedment strength, side member $f_{h.k.1} = 82\ kNm/kg \times \rho_{k.m1} \times (d_{ef.f} / 1mm)^{-0.3}$ = **14.85** N/mm^2

Characteristic embedment strength, main member $f_{h.k.2} = 82\ kNm/kg \times \rho_{k.m2} \times (d_{ef.f} / 1mm)^{-0.3}$ = **14.85** N/mm^2

Yield moment of screw - eq 8.14 $M_{y.Rk} = 0.30\ mm^{0.4} \times f_{u.fs} \times d_{ef.f}^{\ 2.6}$ = **18987** Nmm

Withdrawal resistance

Penetration length of the threaded part $l_{ef.f} = t_{pen}$ = **44.60** mm

Characteristic values of the withdrawal and pull-through strengths

Withdraw capacity, User entered $f_{ax.k.pss}$ = **4.500** N/mm^2

Associatated density $\rho_{a.ax.k.pss}$ = **350.00** kg/m^3

Effective number of screws - eq 8.41 $n_{ef} = N_{fixings}^{0.9}$ = **2.688**

Withdrawal capacity - eq 8.40a $F_{ax.point.Rk} = f_{ax.k.pss} \times d_{outer.f} \times l_{ef.f} / (1.2 \times \cos(\alpha_{screw})^2 + \sin(\alpha_{screw})^2) \times (\rho_{k.m2} / \rho_{a.ax.k.pss})^{0.8}$ = **911** N

Headside pull-through, User entered $f_{head.k.hss}$ = **5.00** N/mm^2

Associatated density $\rho_{a.head.k.hss}$ = **350.00** kg/m^3

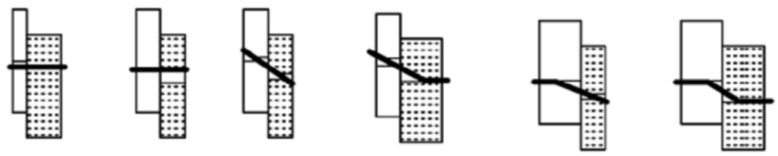
Pull-through resistance - eq 8.40b $F_{ax.head.Rk} = f_{head.k.hss} \times d_{h.f}^2 \times (\rho_{k.m2} / \rho_{a.head.k.hss})^{0.8}$ = **220.746** N

$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{ax.head.Rk})$ = **0.2207** kN

Design value of axial withdrawal capacity $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.1358** kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber connections:



a b c d e f

Embedment ratio $\beta = f_{h.k.2} / f_{h.k.1}$ = **1.00**

Thickness headside member $t_1 = b_1$ = **32** mm

Penetration length in pointside member $t_2 = t_{pen}$ = **45** mm

Maximum rope effect contribution - cl. 8.2.2(2) & 8.3.1.2(4)

$P_{f.mod} = 2$

$P_{f.mod} - 1$ = **100** %

Rope = $F_{ax.Rk} / 4$ = **55** N

Failure mode (a) $f.m.a = f_{h.k.1} \times t_1 \times d_{ef.f}$ = **2.85** kN

Failure mode (b) $f.m.b = f_{h.k.2} \times t_2 \times d_{ef.f}$ = **3.97** kN

Failure mode (c) $f.m.c_{Jyt} = f.m.a / (1+\beta) \times (\sqrt{[\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1)+(t_2 / t_1)^2) + \beta^3 \times ((t_2 / t_1)^2)]} - \beta \times (1 + (t_2 / t_1)))$

$f.m.c = \min(f.m.c_{Jyt} + Rope, P_{f.mod} \times f.m.c_{Jyt})$ = **1.50** kN

Failure mode (d) $f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{[2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_1^2 \times d_{ef.f}))]} - \beta)$

$f.m.d = \min(f.m.d_{Jyt} + Rope, P_{f.mod} \times f.m.d_{Jyt})$ = **1.60** kN

Failure mode (e) — $f_{m.eJyt} = (1.05 \times (f_{h.k.1} \times t_2 \times d_{ef.f}) / (1 + 2 \times \beta)) \times (\sqrt{[2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_2^2 \times d_{ef.f}))]} - \beta)$

$f_{m.e} = \min(f_{m.eJyt} + Rope, P_{f.mod} \times f_{m.eJyt})$ = **1.86** kN

Failure mode (f) — $f_{m.fJyt} = 1.15 \times \sqrt{[(2 \times \beta) / (1 + \beta)]} \times \sqrt{[(2 \times M_{y.Rk} \times f_{h.k.1} \times d_{ef.f})]}$

$f_{m.f} = \min(f_{m.fJyt} + Rope, P_{f.mod} \times f_{m.fJyt})$ = **2.17** kN

Characteristic lateral nail shear resistance — **Failure mode (c)**

$F_{v.Rk}$ = **1.50** kN

Design resistance per fixing - cl 8.3.1.2(4) — $F_{v.Rd} = (k_{mod} \times F_{v.Rk}) / \gamma_{M.connection} / 3$ = **0.31** kN

Load utilisation factor — $ut\_load = F_{la.Ed} / (N_{fixings} \times F_{v.Rd})$ = **0.247**

PASS - Design resistance exceeds design load

Splitting capacity of timber

Loaded edge distance — h_e = **169** mm

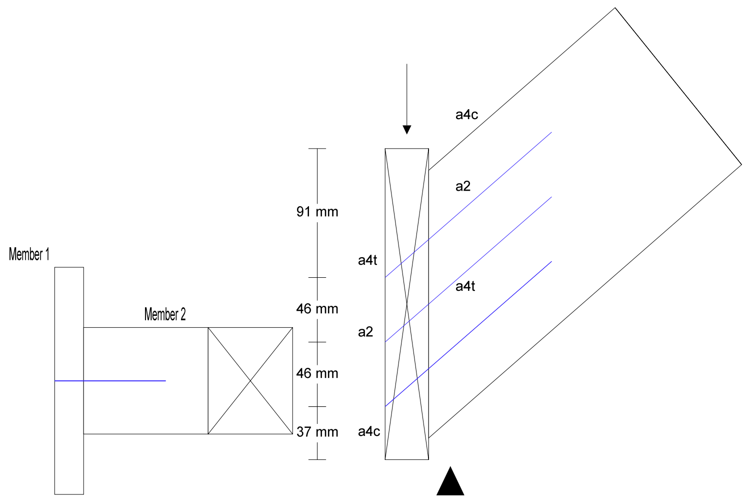
Characteristic splitting capacity - eq 8.4 — $F_{90.Rk} = 14 \times b_1 \times 1\ mm^{-1} \times \sqrt{(h_e \times 1mm^{-1} / (1 - (h_e / h_1)))} \times 1\ N$ = **12.11** kN

Design splitting capacity — $F_{90.Rd} = k_{mod} \times F_{90.Rk} / \gamma_{M.connection}$ = **7.45** kN

$F_{la.Ed} / F_{90.Rd}$ =**0.031**

PASS - Splitting capacity of timber exceeds the design force in member

Spacing



	Allowable minimum	Applied
Member 1		
Spacing of rows perpendicular to grain, a2: $5 \times d_{outer.f}$	30.0 mm	35.0 mm
Distance between fixing and loaded edge, a4t: $(5 + 5 \times \sin \alpha) \times d_{outer.f}$	60.0 mm	64.3 mm
Distance between fixing and unloaded edge, a4c: $5 \times d_{outer.f}$	30.0 mm	37.5 mm
Member 2		
Spacing of rows perpendicular to grain, a2: $5 \times d_{outer.f}$	30.0 mm	35.0 mm
Distance between fixing and loaded edge, a4t: $(5 + 5 \times \sin \alpha) \times d_{outer.f}$	24.0 mm	37.5 mm
Distance between fixing and unloaded edge, a4c: $5 \times d_{outer.f}$	18.0 mm	37.5 mm
Minimum breadth of member: $2 \times$ a4c.m2	36.0 mm	60.0 mm

Allowable minimum from table 8.2

PASS - All spacing conditions are met

Edinburgh Napier University Edinburgh Napier, COCIS Unit 1, 7 Hills Business Park Bankhead Crossway South Edinburgh EH11 4EP	Project				Job Ref.	
	Section				Sheet no./rev. 251	
	Calc. by AL	Date 15 October	Chk'd by	Date	App'd by	Date

4) Tedds output: Main to side member - Ver. Slant screw example

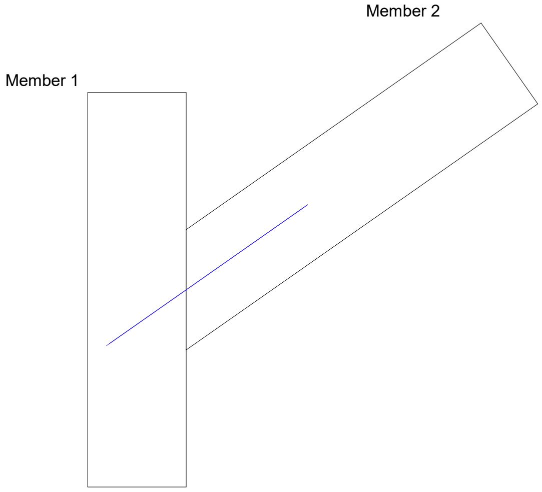
TIMBER CONNECTION DESIGN

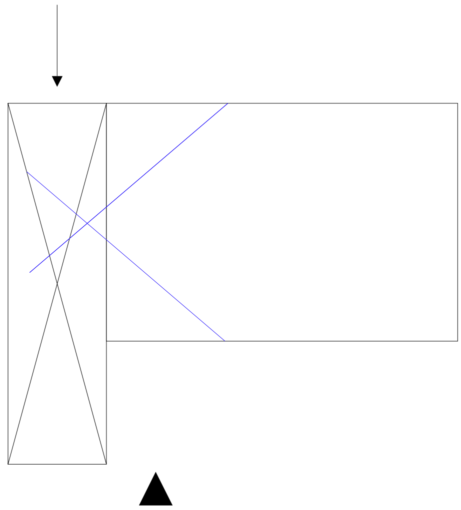
In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.1.07

Design summary

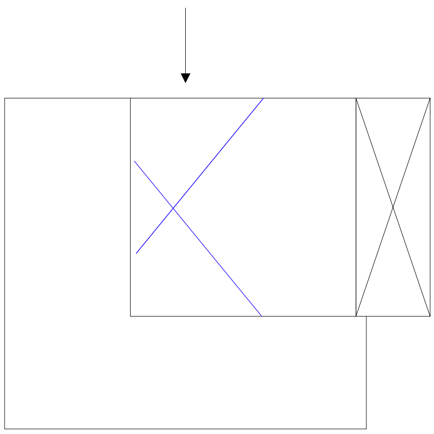
Description	Provided	Required	Utilisation	Result
pointside penetration	36.0 mm	44.6 mm	0.807	PASS
spacing/edge distances			0.681	PASS
load utilisation factor			0.507	PASS

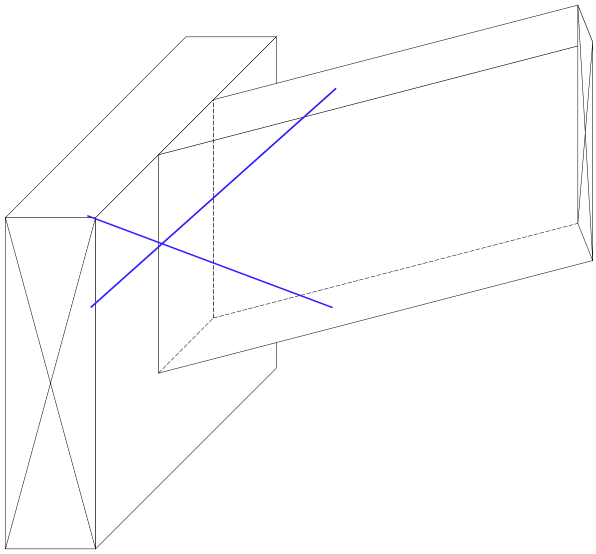




Plan

Elevation





End elevation

Geometric Properties

Member 1

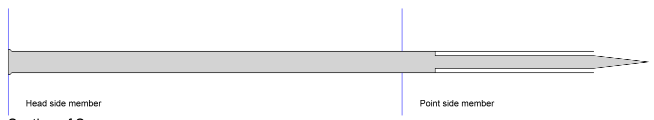
Breadth b_1 = **60** mm
Height h_1 = **220** mm
Cross sectional area A_1 = **13200** mm^2
Strength class C16
Characteristic tension strength parallel to the grain $f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber $\rho_{k.m1}$ = **310** kg/m^3

Member 2

Breadth b_2 = **60** mm
Height h_2 = **145** mm
Cross sectional area A_2 = **8700** mm^2
Rotation about the X-X axis 0 °
Rotation about the Z-Z axis -35 °
Strength class C16
Characteristic tension strength parallel to the grain $f_{t.0.k.m2}$ = **10** N/mm^2
Characteristic density of the timber $\rho_{k.m2}$ = **310** kg/m^3

Screws

Description 6.0 mm / 3.6 mm x 180 mm screw
Number of screws $N_{fixings}$ = **2**
Head diameter $d_{h.f}$ = **6.975** mm
Head length $l_{n.l.f}$ = **0.500** mm
Smooth shank diameter d_f = **6.00** mm
Outer thread diameter $d_{outer.f}$ = **6.00** mm
Inner thread diameter $d_{inner.f}$ = **3.55** mm
Total length l_f = **180.0** mm
Thread length, including the point $l_{th.f}$ = **60.0** mm
Point length $l_{point.f}$ = **15.4** mm
Total screw pointside penetration PsP = **69.32** mm
Tensile strength of each fixing $f_{u.fs}$ = **600** N/mm^2
Counter sunk head



Section of Screw

Smooth shank penetration $(4 \times d_{outer.f}) / (PsP - l_{th.f})$ = **2.575**
Conditions of 8.7.1(2) are not met, effective diameter calculated in accordance with 8.7.1(3)
Effective screw diameter - cl 8.7.1(3) $d_{ef.f} = d_{inner.f} \times 1.1$ = **3.91** mm
Pointside penetration, smooth shank - cl 8.7.1(2) $t_{pen} = l_{th.f} - l_{point.f}$ = **45** mm
Minimum penetration of fixing in main member $t_{min.pen} = 6 \times d_{outer.f}$ = **36.0** mm
$t_{min.pen} / t_{pen}$ = **0.81**

PASS - Pointside penetration is acceptable

Check to validate that no pre-drilling is acceptable

Characteristic dencity < 500 kg/m^3 - cl 8.3.1.1(2) $\rho_{k.m1}$ / 500kg/m^3 = **0.62** OK
$\rho_{k.m2}$ / 500kg/m^3 = **0.62** OK

Diameter of fixing < 6mm - cl 8.3.1.1(2) d_f / 6mm = **1.00** Not suitable

Timber thickness > t_{min} - cl 8.3.1.2(6)

t_{min} = max (7 × d_f, (13 × d_f - 30mm) × (max($\rho_{k.m1}$, $\rho_{k.m2}$) / 400kg/m^3)) = **42** mm OK

FAIL - Requires to be predrilled

<u>Partial safety factors</u>

Safety factors - EC0 National Annex

Limit state (STR)

Permanent actions γ_G = **1.35**

Variable actions γ_Q = **1.50**

Safety factors – EC5 National Annex

Material factor for timber γ_M = **1.30**

Material factor for connections $\gamma_{M.connection}$ = **1.30**

Actions acting on member 1

Characteristic lateral action

Permanent vertical lateral action $G_{la.v.k}$ = **0.08** kN

Permanent horizontal lateral action $G_{la.h.k}$ = **0.00** kN

$G_{la.k} = \sqrt{(G_{la.v.k}^2 + G_{la.h.k}^2)}$ = **0.08** kN

Variable vertical lateral action $Q_{la.v.k}$ = **0.08** kN

Variable horizontal lateral action $Q_{la.h.k}$ = **0.00** kN

$Q_{la.k} = \sqrt{(Q_{la.v.k}^2 + Q_{la.h.k}^2)}$ = **0.08** kN

Design lateral action, EN1990 - eq 6.10 $F_{la.Ed} = \sqrt{((\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k})^2 + (\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k})^2)}$ = **0.23** kN

Characteristic axial permanent action $G_{ax.k}$ = **0.00** kN

Characteristic axial variable action $Q_{ax.k}$ = **0.00** kN

Design axial action, EN1990 - eq 6.10 $F_{ax.Ed} = \gamma_G \times G_{ax.k} + \gamma_Q \times Q_{ax.k}$ = **0.00** kN

Angle between the force and the grain direction α = 90 - atan(($\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k}$) / ($\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k}$)) = **90**°

Min. angle screw axis - grain direction point side α_{screw} = **55**°

Modification factors – Table 3.1

Service class of timber **1**

Load-duration **Medium term**

k_{mod} = **0.80**

Embedment strength in timber - eq 8.15

Characteristic embedment strength, side member $f_{h.k.1}$ = 82 kNm/kg × $\rho_{k.m1}$ × ($d_{ef.f}$ / 1mm)$^{-0.3}$ = **16.89** N/mm^2

Characteristic embedment strength, main member $f_{h.k.2}$ = 82 kNm/kg × $\rho_{k.m2}$ × ($d_{ef.f}$ / 1mm)$^{-0.3}$ = **16.89** N/mm^2

Yield moment of screw - eq 8.14 $M_{y.Rk}$ = 0.30 mm$^{0.4}$ × $f_{u.fs}$ × $d_{ef.f}^{\,2.6}$ = **6216** Nmm

Withdrawal resistance

Penetration length of the threaded part $l_{ef.f} = t_{pen}$ = **44.60** mm

Characteristic values of the withdrawal and pull-through strengths

Withdraw capacity, User entered $f_{ax.k.pss}$ = **4.500** N/mm^2

Associatated density $\rho_{a.ax.k.pss}$ = **350.00** kg/m^3

Effective number of screws - eq 8.41 $n_{ef} = N_{fixings}^{0.9}$ = **1.866**

Withdrawal capacity - eq 8.40a $F_{ax.point.Rk} = f_{ax.k.pss} \times d_{outer.f} \times l_{ef.f} / (1.2 \times \cos(\alpha_{screw})^2 + \sin(\alpha_{screw})^2) \times (\rho_{k.m2} / \rho_{a.ax.k.pss})^{0.8}$ = **1025** N

Headside pull-through, User entered $f_{head.k.hss}$ = **5.00** N/mm^2

Associatated density $\rho_{a.head.k.hss}$ = **350.00** kg/m^3

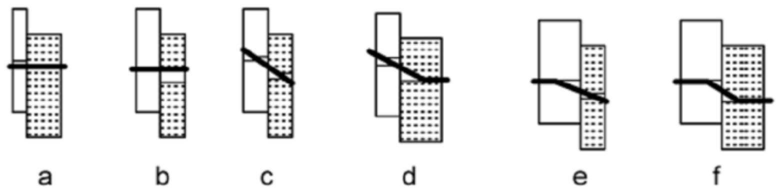
Pull-through resistance - eq 8.40b $F_{ax.head.Rk} = f_{head.k.hss} \times d_{h.f}^2 \times (\rho_{k.m2} / \rho_{a.head.k.hss})^{0.8}$ = **220.746** N

$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{ax.head.Rk}) = \mathbf{0.2207}$ kN

Design value of axial withdrawal capacity $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection} = \mathbf{0.1358}$ kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber connections:



Embedment ratio $\beta = f_{h.k.2} / f_{h.k.1} = \mathbf{1.00}$

Thickness headside member $t_1 = b_1 = \mathbf{60}$ mm

Penetration length in pointside member $t_2 = t_{pen} = \mathbf{45}$ mm

Maximum rope effect contribution - cl. 8.2.2(2) & 8.3.1.2(4)

$P_{f.mod} = 2$

$P_{f.mod} - 1 = \mathbf{100}$ %

Rope = $F_{ax.Rk} / 4 = \mathbf{55}$ N

Failure mode (a) $f.m.a = f_{h.k.1} \times t_1 \times d_{ef.f} = \mathbf{3.96}$ kN

Failure mode (b) $f.m.b = f_{h.k.2} \times t_2 \times d_{ef.f} = \mathbf{2.94}$ kN

Failure mode (c) $f.m.c_{Jyt} = f.m.a / (1+\beta) \times (\sqrt{[\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1) + (t_2 / t_1)^2) + \beta^3 \times ((t_2 / t_1)^2)]} - \beta \times (1 + (t_2 / t_1)))$

$f.m.c = \min(f.m.c_{Jyt} + \text{Rope}, P_{f.mod} \times f.m.c_{Jyt}) = \mathbf{1.51}$ kN

Failure mode (d) $f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{[2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_1^2 \times d_{ef.f}))]} - \beta)$

$f.m.d = \min(f.m.d_{Jyt} + \text{Rope}, P_{f.mod} \times f.m.d_{Jyt}) = \mathbf{1.55}$ kN

Failure mode (e) $f.m.e_{Jyt} = (1.05 \times (f_{h.k.1} \times t_2 \times d_{ef.f}) / (1 + 2 \times \beta)) \times (\sqrt{[2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_2^2 \times d_{ef.f}))]} - \beta)$

$f.m.e = \min(f.m.e_{Jyt} + \text{Rope}, P_{f.mod} \times f.m.e_{Jyt}) = \mathbf{1.23}$ kN

Failure mode (f) $f.m.f_{Jyt} = 1.15 \times \sqrt{[(2 \times \beta) / (1 + \beta)]} \times \sqrt{[(2 \times M_{y.Rk} \times f_{h.k.1} \times d_{ef.f})]}$

$f.m.f = \min(f.m.f_{Jyt} + \text{Rope}, P_{f.mod} \times f.m.f_{Jyt}) = \mathbf{1.10}$ kN

Characteristic lateral nail shear resistance **Failure mode (f)**

$F_{v.Rk} = \mathbf{1.10}$ kN

Design resistance per fixing - cl 8.3.1.2(4) $F_{v.Rd} = (k_{mod} \times F_{v.Rk}) / \gamma_{M.connection} / 3 = \mathbf{0.22}$ kN

Load utilisation factor ut_load = $F_{la.Ed} / (N_{fixings} \times F_{v.Rd}) = \mathbf{0.507}$

PASS - Design resistance exceeds design load

Splitting capacity of timber

Loaded edge distance $h_e = \mathbf{83}$ mm

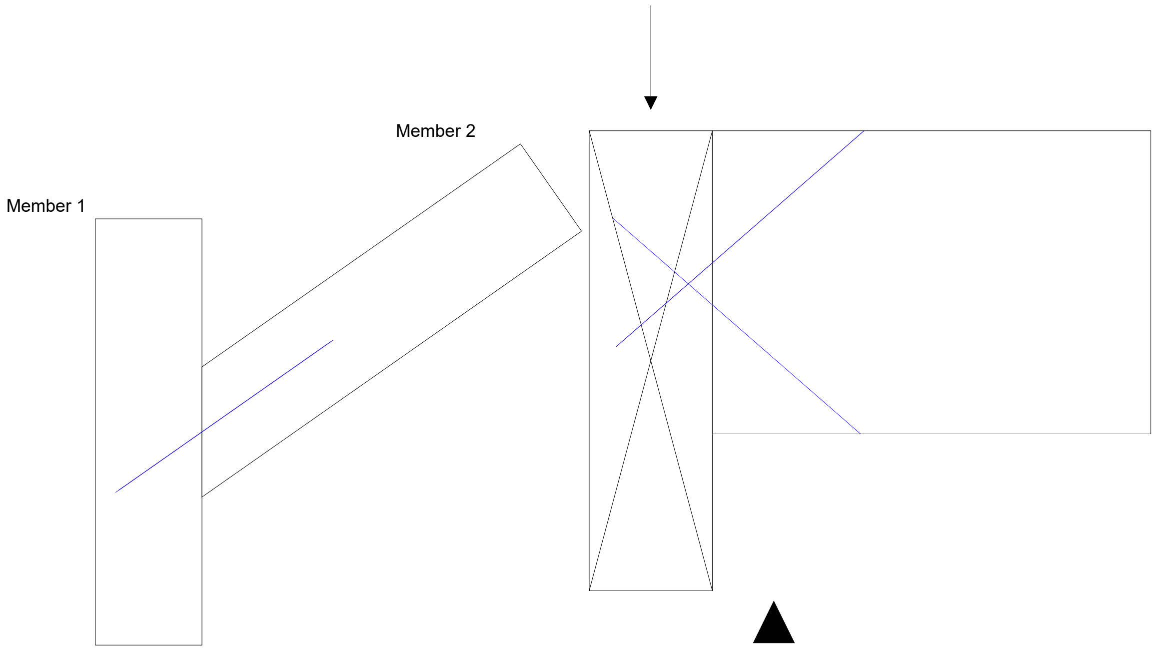
Characteristic splitting capacity - eq 8.4 $F_{90.Rk} = 14 \times b_1 \times 1\ \text{mm}^{-1} \times \sqrt{(h_e \times 1\text{mm}^{-1} / (1 - (h_e / h_1)))} \times 1\ \text{N} = \mathbf{9.72}$ kN

Design splitting capacity $F_{90.Rd} = k_{mod} \times F_{90.Rk} / \gamma_{M.connection} = \mathbf{5.98}$ kN

$F_{la.Ed} / F_{90.Rd} = \mathbf{0.038}$

PASS - Splitting capacity of timber exceeds the design force in member

Spacing



Member 1	**Allowable minimum**	**Applied**
Member 2		
Slant fixings, EC5 8.3.2(10), aslant	60.0 mm	88.1 mm
Minimum breadth of member: 2 × a4c.m2	60.0 mm	60.0 mm
Allowable minimum from table 8.2		

PASS - All spacing conditions are met

5) Tedds output: Main to side member - Hor. Slant screw sxample

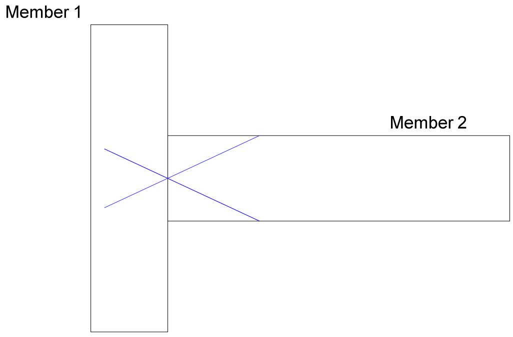
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

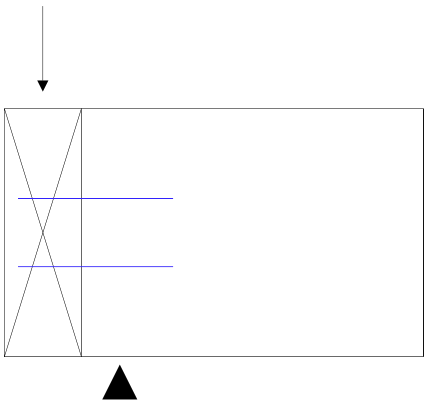
Tedds calculation version 1.1.07

Design summary

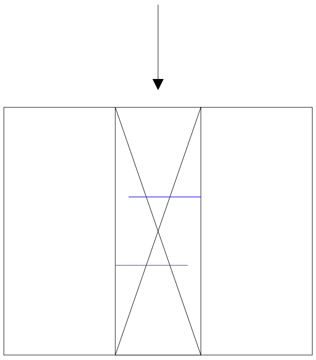
Description	Provided	Required	Utilisation	Result
pointside penetration	30.0 mm	33.3 mm	0.900	PASS
spacing/edge distances			0.952	PASS
load utilisation factor			0.623	PASS

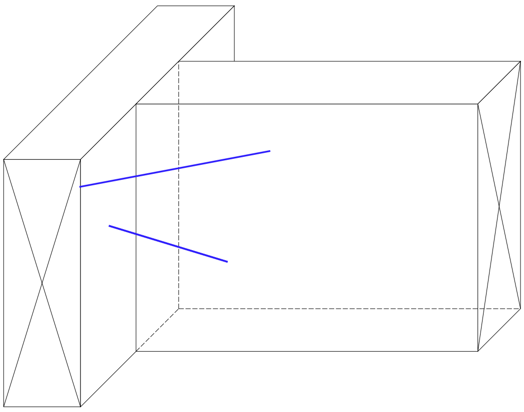


Plan



Elevation





End elevation

Geometric Properties

Member 1

Breadth b_1 = **45** mm

Height h_1 = **145** mm

Cross sectional area A_1 = **6525** mm^2

Strength class C16

Characteristic tension strength parallel to the grain $f_{t.0.k.m1}$ = **10** N/mm^2

Characteristic density of the timber $\rho_{k.m1}$ = **310** kg/m^3

Member 2

Breadth b_2 = **50** mm

Height h_2 = **145** mm

Cross sectional area A_2 = **7250** mm^2

Rotation about the X-X axis 0 °

Rotation about the Z-Z axis 0 °

Strength class C16

Characteristic tension strength parallel to the grain $f_{t.0.k.m2}$ = **10** N/mm^2

Characteristic density of the timber $\rho_{k.m2}$ = **310** kg/m^3

Screws

Description 5.0 mm / 3.0 mm x 100 mm screw

Number of screws $N_{fixings}$ = **2**

Head diameter $d_{h.f}$ = **6.975** mm

Head length $l_{n.l.f}$ = **0.500** mm

Smooth shank diameter d_f = **5.00** mm

Outer thread diameter $d_{outer.f}$ = **5.00** mm

Inner thread diameter $d_{inner.f}$ = **3.00** mm

Total length l_f = **100.0** mm

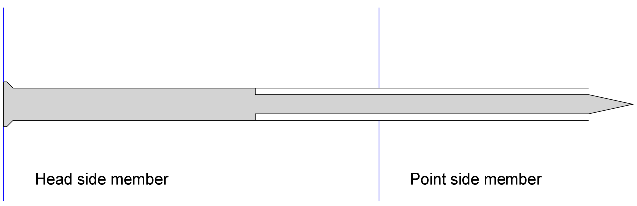
Thread length, including the point $l_{th.f}$ = **60.0** mm

Point length $l_{point.f}$ = **7.0** mm

Total screw pointside penetration PsP = **40.34** mm

Tensile strength of each fixing $f_{u.fs}$ = **600** N/mm^2

Counter sunk head



Section of Screw

Smooth shank penetration $(4 \times d_{outer.f})$ / $(PsP - l_{th.f})$ = **-1.018**

Conditions of 8.7.1(2) are not met, effective diameter calculated in accordance with 8.7.1(3)

Effective screw diameter - cl 8.7.1(3) $d_{ef.f} = d_{inner.f} \times 1.1$ = **3.30** mm

Pointside penetration

non smooth shank - cl 8.7.1(2) $t_{pen} = PsP - l_{point.f}$ = **33** mm

Minimum penetration of fixing in main member $t_{min.pen} = 6 \times d_{outer.f}$ = **30.0** mm

$t_{min.pen} / t_{pen}$ = **0.90**

PASS - Pointside penetration is acceptable

Check to validate that no pre-drilling is acceptable

Characteristic dencity < 500 kg/m^3 - cl 8.3.1.1(2) $\rho_{k.m1}$ / 500kg/m^3 = **0.62** OK

$\rho_{k.m2}$ / 500kg/m^3 = **0.62** OK

Diameter of fixing < 6mm - cl 8.3.1.1(2) d_f / 6mm = **0.83** OK

Timber thickness > t_{min} - cl 8.3.1.2(6)

$t_{min} = \max(7 \times d_f, (13 \times d_f - 30mm) \times (\max(\rho_{k.m1}, \rho_{k.m2}) / 400kg/m^3))$ = **35** mm OK

PASS - Predrilling is not required

<u>Partial safety factors</u>

Safety factors - EC0 National Annex

Limit state (STR)

Permanent actions γ_G = **1.35**

Variable actions γ_Q = **1.50**

Safety factors – EC5 National Annex

Material factor for timber γ_M = **1.30**

Material factor for connections $\gamma_{M.connection}$ = **1.30**

Actions acting on member 1

Characteristic lateral action

Permanent vertical lateral action $G_{la.v.k}$ = **0.08** kN

Permanent horizontal lateral action $G_{la.h.k}$ = **0.00** kN

$G_{la.k} = \sqrt{(G_{la.v.k}^2 + G_{la.h.k}^2)}$ = **0.08** kN

Variable vertical lateral action $Q_{la.v.k}$ = **0.08** kN

Variable horizontal lateral action $Q_{la.h.k}$ = **0.00** kN

$Q_{la.k} = \sqrt{(Q_{la.v.k}^2 + Q_{la.h.k}^2)}$ = **0.08** kN

Design lateral action, EN1990 - eq 6.10 $F_{la.Ed} = \sqrt{((\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k})^2 + (\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k})^2)}$ = **0.23** kN

Characteristic axial permanent action $G_{ax.k}$ = **0.00** kN

Characteristic axial variable action $Q_{ax.k}$ = **0.00** kN

Design axial action, EN1990 - eq 6.10 $F_{ax.Ed} = \gamma_G \times G_{ax.k} + \gamma_Q \times Q_{ax.k}$ = **0.00** kN

Angle between the force and the grain direction $\alpha = 90 - \text{atan}((\gamma_G \times G_{la.h.k} + \gamma_Q \times Q_{la.h.k}) / (\gamma_G \times G_{la.v.k} + \gamma_Q \times Q_{la.v.k}))$ = **90**°

Min. angle screw axis - grain direction point side α_{screw} = **65**°

Modification factors – Table 3.1

Service class of timber **1**

Load-duration **Medium term**

k_{mod} = **0.80**

Embedment strength in timber - eq 8.15

Characteristic embedment strength, side member $f_{h.k.1} = 82\ \text{kNm/kg} \times \rho_{k.m1} \times (d_{ef.f} / 1\text{mm})^{-0.3}$ = **17.77** N/mm^2

Characteristic embedment strength, main member $f_{h.k.2} = 82\ \text{kNm/kg} \times \rho_{k.m2} \times (d_{ef.f} / 1\text{mm})^{-0.3}$ = **17.77** N/mm^2

Yield moment of screw - eq 8.14 $M_{y.Rk} = 0.30\ \text{mm}^{0.4} \times f_{u.fs} \times d_{ef.f}^{\ 2.6}$ = **4012** Nmm

Withdrawal resistance

Penetration length of the threaded part $l_{ef.f} = t_{pen}$ = **33.34** mm

Characteristic values of the withdrawal and pull-through strengths

Withdraw capacity, User entered $f_{ax.k.pss}$ = **11.000** N/mm^2

Associatated density $\rho_{a.ax.k.pss}$ = **350.00** kg/m^3

Effective number of screws - eq 8.41 $n_{ef} = N_{fixings}^{0.9}$ = **1.866**

Withdrawal capacity - eq 8.40a $F_{ax.point.Rk} = f_{ax.k.pss} \times d_{outer.f} \times l_{ef.f} / (1.2 \times \cos(\alpha_{screw})^2 + \sin(\alpha_{screw})^2) \times (\rho_{k.m2} / \rho_{a.ax.k.pss})^{0.8}$ = **1607** N

Headside pull-through, User entered $f_{head.k.hss}$ = **9.40** N/mm^2

Associatated density $\rho_{a.head.k.hss}$ = **350.00** kg/m^3

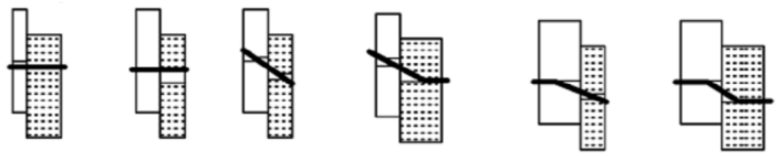
Pull-through resistance - eq 8.40b $F_{ax.head.Rk} = f_{head.k.hss} \times d_{h.f}^2 \times (\rho_{k.m2} / \rho_{a.head.k.hss})^{0.8}$ = **415.003** N

$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{ax.head.Rk})$ = **0.4150** kN

Design value of axial withdrawal capacity $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.2554** kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber connections:



Embedment ratio $\beta = f_{h.k.2} / f_{h.k.1} = \mathbf{1.00}$

Thickness headside member $t_1 = b_1 = \mathbf{45}$ mm

Penetration length in pointside member $t_2 = t_{pen} = \mathbf{33}$ mm

Maximum rope effect contribution - cl. 8.2.2(2) & 8.3.1.2(4)

$P_{f.mod} = 2$

$P_{f.mod} - 1 = \mathbf{100}$ %

Rope = $F_{ax.Rk} / 4 = \mathbf{104}$ N

Failure mode (a) $f.m.a = f_{h.k.1} \times t_1 \times d_{ef.f} = \mathbf{2.64}$ kN

Failure mode (b) $f.m.b = f_{h.k.2} \times t_2 \times d_{ef.f} = \mathbf{1.96}$ kN

Failure mode (c) $f.m.c_{Jyt} = f.m.a / (1+\beta) \times (\sqrt{[\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1)+(t_2 / t_1)^2) + \beta^3 \times ((t_2 / t_1)^2)]} - \beta \times (1 + (t_2 / t_1)))$

$f.m.c = \min(f.m.c_{Jyt} + Rope, P_{f.mod} \times f.m.c_{Jyt}) = \mathbf{1.07}$ kN

Failure mode (d) $f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{[2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_1^2 \times d_{ef.f}))]} - \beta)$

$f.m.d = \min(f.m.d_{Jyt} + Rope, P_{f.mod} \times f.m.d_{Jyt}) = \mathbf{1.12}$ kN

Failure mode (e) $f.m.e_{Jyt} = (1.05 \times (f_{h.k.1} \times t_2 \times d_{ef.f}) / (1 + 2 \times \beta)) \times (\sqrt{[2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.1} \times t_2^2 \times d_{ef.f}))]} - \beta)$

$f.m.e = \min(f.m.e_{Jyt} + Rope, P_{f.mod} \times f.m.e_{Jyt}) = \mathbf{0.91}$ kN

Failure mode (f) $f.m.f_{Jyt} = 1.15 \times \sqrt{[(2 \times \beta) / (1 + \beta)]} \times \sqrt{[(2 \times M_{y.Rk} \times f_{h.k.1} \times d_{ef.f})]}$

$f.m.f = \min(f.m.f_{Jyt} + Rope, P_{f.mod} \times f.m.f_{Jyt}) = \mathbf{0.89}$ kN

Characteristic lateral nail shear resistance **Failure mode (f)**

$F_{v.Rk} = \mathbf{0.89}$ kN

Design resistance per fixing - cl 8.3.1.2(4) $F_{v.Rd} = (k_{mod} \times F_{v.Rk}) / \gamma_{M.connection} / 3 = \mathbf{0.18}$ kN

Load utilisation factor ut_load = $F_{la.Ed} / (N_{fixings} \times F_{v.Rd}) = \mathbf{0.623}$

PASS - Design resistance exceeds design load

Splitting capacity of timber

Loaded edge distance $h_e = \mathbf{93}$ mm

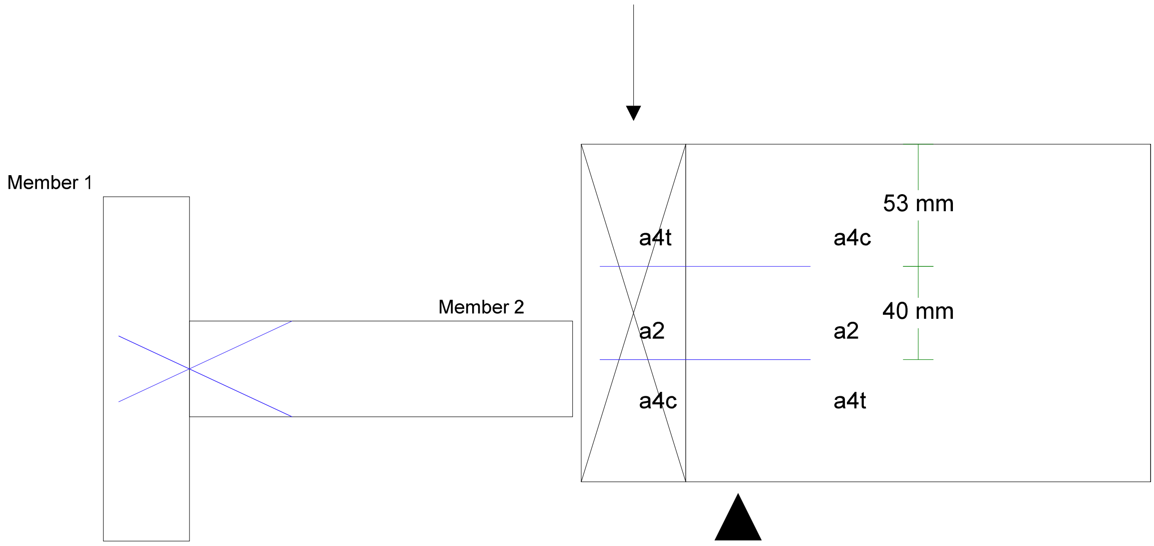
Characteristic splitting capacity - eq 8.4 $F_{90.Rk} = 14 \times b_1 \times 1\ mm^{-1} \times \sqrt{(h_e \times 1mm^{-1} / (1 - (h_e / h_1)))} \times 1\ N = \mathbf{10.07}$ kN

Design splitting capacity $F_{90.Rd} = k_{mod} \times F_{90.Rk} / \gamma_{M.connection} = \mathbf{6.20}$ kN

$F_{la.Ed} / F_{90.Rd} = \mathbf{0.037}$

PASS - Splitting capacity of timber exceeds the design force in member

Spacing



	Allowable minimum	Applied
Member 1		
Spacing of rows perpendicular to grain, a2: $5 \times d_{outer.f}$	25.0 mm	40.0 mm
Distance between fixing and loaded edge, a4t: $(5 + 5 \times \sin \alpha) \times d_{outer.f}$	50.0 mm	52.5 mm
Distance between fixing and unloaded edge, a4c: $5 \times d_{outer.f}$	25.0 mm	52.5 mm
Member 2		
Spacing of rows perpendicular to grain, a2: $5 \times d_{outer.f}$	25.0 mm	40.0 mm
Distance between fixing and loaded edge, a4t: $(5 + 5 \times \sin \alpha) \times d_{outer.f}$	50.0 mm	52.5 mm
Distance between fixing and unloaded edge, a4c: $5 \times d_{outer.f}$	25.0 mm	52.5 mm
Slant fixings, EC5 8.3.2(10), aslant	50.0 mm	53.6 mm
Minimum breadth of member: $2 \times$ a4c.m2	50.0 mm	50.0 mm

Allowable minimum from table 8.2

PASS - All spacing conditions are met

6) Tedds output: Multi Member - Bolts 5 members example

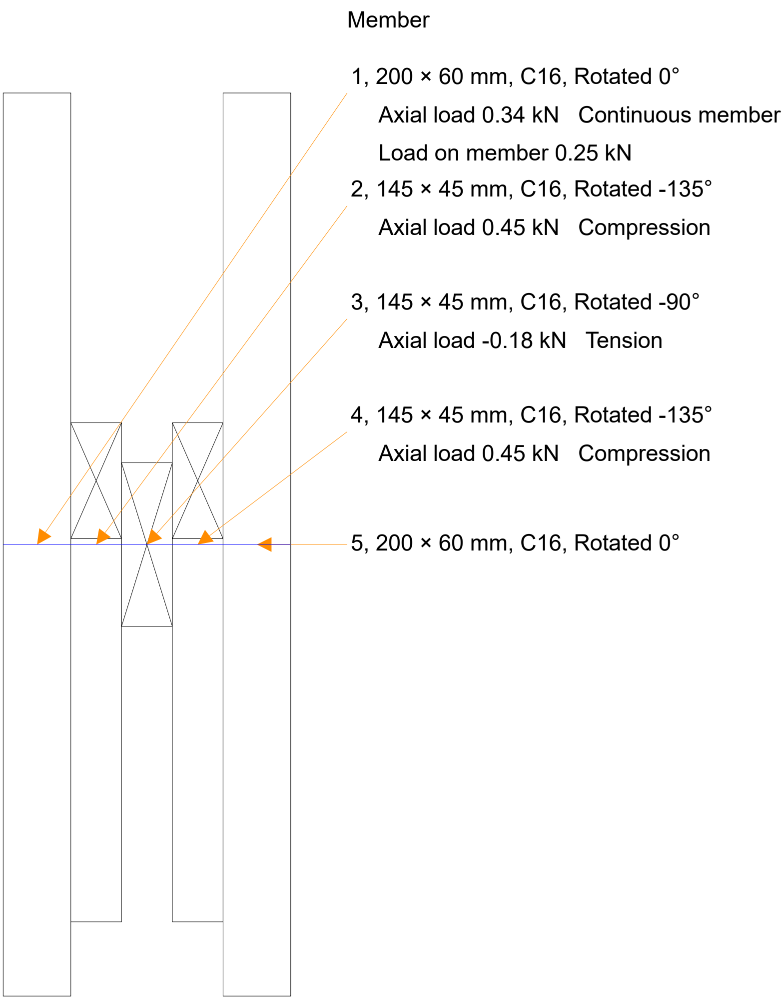
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

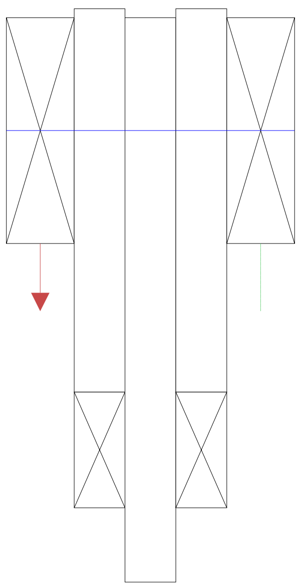
Tedds calculation version 1.1.07

Design summary

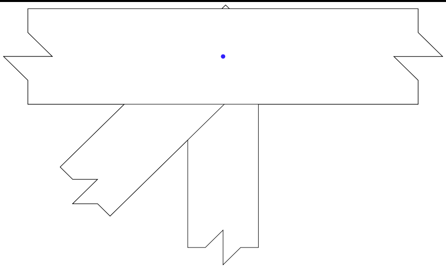
Description	Provided	Required	Utilisation	Result
Edge/end spacing				PASS
Shear plane resistance			0.135	PASS
Splitting capacity of timber			0.034	PASS

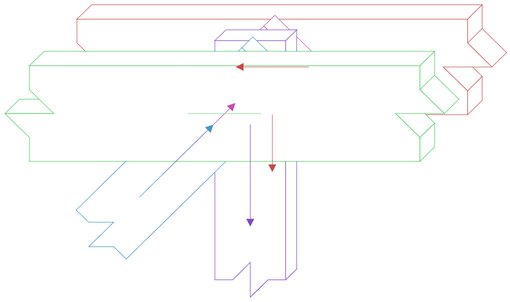


Plan



End elevation





Elevation

This connection is in double shear

Geometry

Bolts

Bolt description	M10 Grade 8.8 bolt
Number of Bolts	$N_{fixings}$ = **1**
Effective number of fixings	n_{ef} = $N_{fixings}$ = **1**
Bolt diameter	d = d_f = **10** mm
Washer diameter required - cl 10.4.3(2)	d_w = 3 × d_f = **30** mm
Washer thickness required - cl 10.4.3(2)	d_{wt} = 0.3 × d_f = **3** mm
Tensile strength of each fixing	$f_{u.f}$ = **800** N/mm^2
Tensile stress area of bolt	$A_{b.t}$ = **58.52** mm^2

Member 1

Member type	Continuous member
Breadth	b_1 = **60** mm
Height	h_1 = **200** mm
Cross sectional area	A_1 = **12000** mm^2
Rotation about the Y-Y axis	0 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m1}$ = **310** kg/m^3

Member 2

Member type	End member
Breadth	b_2 = **45** mm
Height	h_2 = **145** mm
Cross sectional area	A_2 = **6525** mm^2
Rotation about the Y-Y axis	-135 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m2}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m2}$ = **310** kg/m^3

Member 3

Member type	End member
Breadth	b_3 = **45** mm
Height	h_3 = **145** mm
Cross sectional area	A_3 = **6525** mm^2
Rotation about the Y-Y axis	-90 °
Strength class	C16

Characteristic tension strength parallel to the grain $f_{t.0.k.m3}$ = **10** N/mm^2
Characteristic density of the timber $\rho_{k.m3}$ = **310** kg/m^3

Member 4

Member type	End member
Breadth	b_4 = **45** mm
Height	h_4 = **145** mm
Cross sectional area	A_4 = **6525** mm^2
Rotation about the Y-Y axis	-135 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m4}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m4}$ = **310** kg/m^3

Member 5

Member type	Continuous member
Breadth	b_5 = **60** mm
Height	h_5 = **200** mm
Cross sectional area	A_5 = **12000** mm^2
Rotation about the Y-Y axis	0 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m5}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m5}$ = **310** kg/m^3

Partial safety factors

Material factor for connections, table 2.3 $\gamma_{M.connection}$ = **1.30**

Actions

Modification factors – Table 3.1

Service class of timber	**1**	
Load-duration	**Medium term**	
	k_{mod} = **0.80**	
Design action in member 1	$F_{la.1}$ = **0.34** kN	Continuous member
Design action in member 2	$F_{la.2}$ = **0.45** kN	Compression
Design action in member 3	$F_{la.3}$ = **-0.18** kN	Tension
Design action in member 4	$F_{la.4}$ = **0.45** kN	Compression
Design load on member 1	$F_{la.ex1}$ = **0.25** kN	

Calculated output

Angle of shear planes relative to the grain direction in each member

Shear plane	**Member**	**Joint**	**Angle of shear plane**
1	1	1.2.1	$\theta_{1.1.2}$ = **36** °
1	2	1.2.1	$\theta_{1.2.1}$ = **171** °
2	2	2.3.2	$\theta_{2.2.3}$ = **63** °
2	3	2.3.2	$\theta_{2.3.2}$ = **18** °
3	3	3.4.3	$\theta_{3.3.4}$ = **169** °
3	4	3.4.3	$\theta_{3.4.3}$ = **-146** °
4	4	4.5.4	$\theta_{4.4.5}$ = **-10** °
4	5	4.5.4	$\theta_{4.5.4}$ = **-145** °

Design force in shear plane 1 $F_{la.sp1}$ = **0.42** kN
Design force in shear plane 2 $F_{la.sp2}$ = **0.07** kN

Design force in shear plane 3 $F_{la.sp3}$ = **0.11** kN

Design force in shear plane 4 $F_{la.sp4}$ = **0.36** kN

Characteristic embedment strength in timber at angle α to the grain

eq 8.33, softwood $k_{90.m1}$ = 1.35 + 0.015 × (d / 1mm) = **1.50**

eq 8.33, softwood $k_{90.m2}$ = 1.35 + 0.015 × (d / 1mm) = **1.50**

eq 8.33, softwood $k_{90.m3}$ = 1.35 + 0.015 × (d / 1mm) = **1.50**

eq 8.33, softwood $k_{90.m4}$ = 1.35 + 0.015 × (d / 1mm) = **1.50**

eq 8.33, softwood $k_{90.m5}$ = 1.35 + 0.015 × (d / 1mm) = **1.50**

member	Joint	- eq	
1	1.2.1	8.32	$f_{h.0.k.1}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m1}$ =**22.88** N/mm^2
		8.31	$f_{h.k.1.2}$ = $f_{h.0.k.1}$ / ($k_{90.m1}$ × sin($\theta_{1.1.2}$)^2 + cos($\theta_{1.1.2}$)^2) = **19.46** N/mm^2
2	1.2.1	8.32	$f_{h.0.k.2}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m2}$ =**22.88** N/mm^2
		8.31	$f_{h.k.2.1}$ = $f_{h.0.k.2}$ / ($k_{90.m2}$ × sin($\theta_{1.2.1}$)^2 + cos($\theta_{1.2.1}$)^2) = **22.62** N/mm^2
2	2.3.2	8.32	$f_{h.0.k.2}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m2}$ =**22.88** N/mm^2
		8.31	$f_{h.k.2.3}$ = $f_{h.0.k.2}$ / ($k_{90.m2}$ × sin($\theta_{2.2.3}$)^2 + cos($\theta_{2.2.3}$)^2) = **16.40** N/mm^2
3	2.3.2	8.32	$f_{h.0.k.3}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m3}$ =**22.88** N/mm^2
		8.31	$f_{h.k.3.2}$ = $f_{h.0.k.3}$ / ($k_{90.m3}$ × sin($\theta_{2.3.2}$)^2 + cos($\theta_{2.3.2}$)^2) = **21.86** N/mm^2
3	3.4.3	8.32	$f_{h.0.k.3}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m3}$ =**22.88** N/mm^2
		8.31	$f_{h.k.3.4}$ = $f_{h.0.k.3}$ / ($k_{90.m3}$ × sin($\theta_{3.3.4}$)^2 + cos($\theta_{3.3.4}$)^2) = **22.47** N/mm^2
4	3.4.3	8.32	$f_{h.0.k.4}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m4}$ =**22.88** N/mm^2
		8.31	$f_{h.k.4.3}$ = $f_{h.0.k.4}$ / ($k_{90.m4}$ × sin($\theta_{3.4.3}$)^2 + cos($\theta_{3.4.3}$)^2) = **19.79** N/mm^2
4	4.5.4	8.32	$f_{h.0.k.4}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m4}$ =**22.88** N/mm^2
		8.31	$f_{h.k.4.5}$ = $f_{h.0.k.4}$ / ($k_{90.m4}$ × sin($\theta_{4.4.5}$)^2 + cos($\theta_{4.4.5}$)^2) = **22.53** N/mm^2
5	4.5.4	8.32	$f_{h.0.k.5}$ = 82 MN/kg × (1 mm - (0.01 × d)) × $\rho_{k.m5}$ =**22.88** N/mm^2
		8.31	$f_{h.k.5.4}$ = $f_{h.0.k.5}$ / ($k_{90.m5}$ × sin($\theta_{4.5.4}$)^2 + cos($\theta_{4.5.4}$)^2) = **19.67** N/mm^2

Yield moment of fixing - eq 8.30 $M_{y.Rk}$ = 0.30 mm$^{0.4}$ × $f_{u.f}$ × d$^{2.6}$ = **95546** Nmm

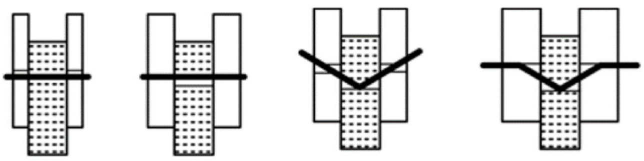
Design strength of the fixing, EN1993-1-8 $F1_{ax.Rd}$ = 0.9 × $f_{u.f}$ × 0.8 × $A_{b.t}$ = **33707.46** N

Design capacity of washer - cl 8.5.2(2) $F2_{ax.Rd}$ = 3 × $f_{c.90.k.m2}$ × (π / 4) × ((d_w)^2 – (d + 1mm)^2) = **4038.05** N

$F_{ax.Rk}$ = min($F1_{ax.Rd}$, $F2_{ax.Rd}$) = **4038.05** N

Lateral load-carrying capacity of connection

Failure modes for timber to timber double shear connections:



g h j k

Maximum rope effect contribution - cl. 8.2.2(2)

cl 8.2.2(2) $P_{f.mod}$ = Profile_{.f} = **1.25**

$P_{f.mod}$ – 1 = **25** %

Rope = $F_{ax.Rk}$ / 4 = **1010** N

Double Shear joint formed from members 1.2.1 for timber to timber joint with a bolt in double Shear, the characteristic lateral resistance per shear plane is the smallest value in - eq 8.7

Embedment ratio $\beta_{2.1}$ = $f_{h.k.2.1}$ / $f_{h.k.1.2}$ = **1.16**

Thickness headside member t_1 = b_1 = **60** mm

Thickness of central member t_2 = b_2 = **45** mm

Failure mode (g) — $f.m.g_1 = f_{h.k.1.2} \times t_1 \times d = \mathbf{11.68}$ kN

Failure mode (h) — $f.m.h_1 = 0.5 \times f_{h.k.2.1} \times t_2 \times d = \mathbf{5.09}$ kN

Failure mode (j) — $f.m.j_{Jyt.1} = 1.05 \times (f.m.g_1 / (2 + \beta_{2.1})) \times (\sqrt{}[2 \times \beta_{2.1} \times (1 + \beta_{2.1}) + ((4 \times \beta_{2.1} \times (2 + \beta_{2.1}) \times M_{y.Rk}) / (f_{h.k.1.2} \times d \times t_1^2))] - \beta_{2.1})$

$f.m.j_1 = \min(f.m.j_{Jyt.1} + Rope, P_{f.mod} \times f.m.j_{Jyt.1}) = \mathbf{6.78}$ kN

Failure mode (k) — $f.m.k_{Jyt.1} = 1.15 \times \sqrt{}((2 \times \beta_{2.1}) / (1 + \beta_{2.1})) \times \sqrt{}(2 \times M_{y.Rk} \times f_{h.k.1.2} \times d)$

$f.m.k_1 = \min(f.m.k_{Jyt.1} + Rope, P_{f.mod} \times f.m.k_{Jyt.1}) = \mathbf{8.28}$ kN

Characteristic lateral shear resistance — **Failure mode (h)**

$F_{v.Rk1} = \mathbf{5.09}$ kN

Design resistance per fixing — $F_{v.Rd1} = (k_{mod} \times F_{v.Rk1}) / \gamma_{M.connection} = \mathbf{3.13}$ kN

$\theta_{1.1.2.Abs} = Abs(\theta_{1.1.2}) = \mathbf{36}$ deg

$\theta_{1.2.1.Abs} = 180 - Abs(\theta_{1.2.1}) = \mathbf{9}$ deg

$\theta_{min.SP1} = \min(\theta_{1.1.2.Abs}, \theta_{1.2.1.Abs}) = \mathbf{9}$ deg

Load utilisation factor — $ut\_load_1 = F_{la.sp1} / ((n_{ef} + (N_{fixings} - n_{ef}) \times (\theta_{min.SP1} / 90)) \times F_{v.Rd1}) = \mathbf{0.135}$

PASS - Design capacity of the shear plane exceeds the design force within shear plane

Double Shear joint formed from members 2.3.2 for timber to timber joint with a bolt in double Shear, the characteristic lateral resistance per shear plane is the smallest value in - eq 8.7

Embedment ratio — $\beta_{3.2} = f_{h.k.3.2} / f_{h.k.2.3} = \mathbf{1.33}$

Thickness headside member — $t_1 = b_2 = \mathbf{45}$ mm

Thickness of central member — $t_2 = b_3 = \mathbf{45}$ mm

Failure mode (g) — $f.m.g_2 = f_{h.k.2.3} \times t_1 \times d = \mathbf{7.38}$ kN

Failure mode (h) — $f.m.h_2 = 0.5 \times f_{h.k.3.2} \times t_2 \times d = \mathbf{4.92}$ kN

Failure mode (j) — $f.m.j_{Jyt.2} = 1.05 \times (f.m.g_2 / (2 + \beta_{3.2})) \times (\sqrt{}[2 \times \beta_{3.2} \times (1 + \beta_{3.2}) + ((4 \times \beta_{3.2} \times (2 + \beta_{3.2}) \times M_{y.Rk}) / (f_{h.k.2.3} \times d \times t_1^2))] - \beta_{3.2})$

$f.m.j_2 = \min(f.m.j_{Jyt.2} + Rope, P_{f.mod} \times f.m.j_{Jyt.2}) = \mathbf{5.74}$ kN

Failure mode (k) — $f.m.k_{Jyt.2} = 1.15 \times \sqrt{}((2 \times \beta_{3.2}) / (1 + \beta_{3.2})) \times \sqrt{}(2 \times M_{y.Rk} \times f_{h.k.2.3} \times d)$

$f.m.k_2 = \min(f.m.k_{Jyt.2} + Rope, P_{f.mod} \times f.m.k_{Jyt.2}) = \mathbf{7.89}$ kN

Characteristic lateral shear resistance — **Failure mode (h)**

$F_{v.Rk2} = \mathbf{4.92}$ kN

Design resistance per fixing — $F_{v.Rd2} = (k_{mod} \times F_{v.Rk2}) / \gamma_{M.connection} = \mathbf{3.03}$ kN

$\theta_{2.2.3.Abs} = Abs(\theta_{2.2.3}) = \mathbf{63}$ deg

$\theta_{2.3.2.Abs} = Abs(\theta_{2.3.2}) = \mathbf{18}$ deg

$\theta_{min.SP2} = \min(\theta_{2.2.3.Abs}, \theta_{2.3.2.Abs}) = \mathbf{18}$ deg

Load utilisation factor — $ut\_load_2 = F_{la.sp2} / ((n_{ef} + (N_{fixings} - n_{ef}) \times (\theta_{min.SP2} / 90)) \times F_{v.Rd2}) = \mathbf{0.024}$

PASS - Design capacity of the shear plane exceeds the design force within shear plane

Double Shear joint formed from members 3.4.3 for timber to timber joint with a bolt in double Shear, the characteristic lateral resistance per shear plane is the smallest value in - eq 8.7

Embedment ratio — $\beta_{4.3} = f_{h.k.4.3} / f_{h.k.3.4} = \mathbf{0.88}$

Thickness headside member — $t_1 = b_3 = \mathbf{45}$ mm

Thickness of central member — $t_2 = b_4 = \mathbf{45}$ mm

Failure mode (g) — $f.m.g_3 = f_{h.k.3.4} \times t_1 \times d = \mathbf{10.11}$ kN

Failure mode (h) — $f.m.h_3 = 0.5 \times f_{h.k.4.3} \times t_2 \times d = \mathbf{4.45}$ kN

Failure mode (j) — $f.m.j_{Jyt.3} = 1.05 \times (f.m.g_3 / (2 + \beta_{4.3})) \times (\sqrt{}[2 \times \beta_{4.3} \times (1 + \beta_{4.3}) + ((4 \times \beta_{4.3} \times (2 + \beta_{4.3}) \times M_{y.Rk}) / (f_{h.k.3.4} \times d \times t_1^2))] - \beta_{4.3})$

$f.m.j_3 = \min(f.m.j_{Jyt.3} + Rope, P_{f.mod} \times f.m.j_{Jyt.3}) = \mathbf{6.36}$ kN

Failure mode (k) — $f.m.k_{Jyt.3} = 1.15 \times \sqrt{}((2 \times \beta_{4.3}) / (1 + \beta_{4.3})) \times \sqrt{}(2 \times M_{y.Rk} \times f_{h.k.3.4} \times d)$

$f.m.k_3 = \min(f.m.k_{Jyt.3} + Rope, P_{f.mod} \times f.m.k_{Jyt.3}) = \mathbf{8.30}$ kN

Characteristic lateral shear resistance — **Failure mode (h)**

$F_{v.Rk3}$ = **4.45** kN

Design resistance per fixing — $F_{v.Rd3} = (k_{mod} \times F_{v.Rk3}) / \gamma_{M.connection}$ = **2.74** kN

$\theta_{3.3.4.Abs} = 180 - Abs(\theta_{3.3.4})$ = **11** deg

$\theta_{3.4.3.Abs} = 180 - Abs(\theta_{3.4.3})$ = **34** deg

$\theta_{min.SP3} = min(\theta_{3.3.4.Abs}, \theta_{3.4.3.Abs})$ = **11** deg

Load utilisation factor — $ut\_load_3 = F_{la.sp3} / ((n_{ef} + (N_{fixings} - n_{ef}) \times (\theta_{min.SP3} / 90)) \times F_{v.Rd3})$ = **0.042**

PASS - Design capacity of the shear plane exceeds the design force within shear plane

Double Shear joint formed from members 4.5.4 for timber to timber joint with a bolt in double Shear, the characteristic lateral resistance per shear plane is the smallest value in - eq 8.7

Embedment ratio — $\beta_{5.4} = f_{h.k.5.4} / f_{h.k.4.5}$ = **0.87**

Thickness headside member — $t_1 = b_4$ = **45** mm

Thickness of central member — $t_2 = b_5$ = **60** mm

Failure mode (g) — $f.m.g_4 = f_{h.k.4.5} \times t_1 \times d$ = **10.14** kN

Failure mode (h) — $f.m.h_4 = 0.5 \times f_{h.k.5.4} \times t_2 \times d$ = **5.90** kN

Failure mode (j) — $f.m.j_{Jyt.4} = 1.05 \times (f.m.g_4 / (2 + \beta_{5.4})) \times (\sqrt{} [2 \times \beta_{5.4} \times (1 + \beta_{5.4}) + ((4 \times \beta_{5.4} \times (2 + \beta_{5.4}) \times M_{y.Rk}) / (f_{h.k.4.5} \times d \times t_1^2))] - \beta_{5.4})$

$f.m.j_4 = min(f.m.j_{Jyt.4} + Rope, P_{f.mod} \times f.m.j_{Jyt.4})$ = **6.36** kN

Failure mode (k) — $f.m.k_{Jyt.4} = 1.15 \times \sqrt{((2 \times \beta_{5.4}) / (1 + \beta_{5.4}))} \times \sqrt{(2 \times M_{y.Rk} \times f_{h.k.4.5} \times d)}$

$f.m.k_4 = min(f.m.k_{Jyt.4} + Rope, P_{f.mod} \times f.m.k_{Jyt.4})$ = **8.29** kN

Characteristic lateral shear resistance — **Failure mode (h)**

$F_{v.Rk4}$ = **5.90** kN

Design resistance per fixing — $F_{v.Rd4} = (k_{mod} \times F_{v.Rk4}) / \gamma_{M.connection}$ = **3.63** kN

$\theta_{4.4.5.Abs} = Abs(\theta_{4.4.5})$ = **10** deg

$\theta_{4.5.4.Abs} = 180 - Abs(\theta_{4.5.4})$ = **35** deg

$\theta_{min.SP4} = min(\theta_{4.4.5.Abs}, \theta_{4.5.4.Abs})$ = **10** deg

Load utilisation factor — $ut\_load_4 = F_{la.sp4} / ((n_{ef} + (N_{fixings} - n_{ef}) \times (\theta_{min.SP4} / 90)) \times F_{v.Rd4})$ = **0.099**

PASS - Design capacity of the shear plane exceeds the design force within shear plane

PASS - Failure modes are compatible with each other

Splitting capacity of timber

Member 1

Loaded edge distance — $h_{e.m1}$ = **100** mm

Characteristic splitting capacity - eq 8.4 — $F_{90.Rk1} = 14\ N/mm \times b_1 \times \sqrt{(Abs(h_{e.m1} \times 1mm^{-1} / (1 - (h_{e.m1} / h_1))))}$ = **11.88** kN

Design splitting capacity — $F_{90.Rd1} = k_{mod} \times F_{90.Rk1} / \gamma_{M.connection}$ = **7.31** kN

Design shear force — $F_{v.Ed1} = F_{la.sp1} \times Sin(\theta_{1.1.2})$ = **0.25** kN

Splitting utilisation — $ut\_Split_1 = Abs(F_{v.Ed1} / F_{90.Rd1})$ = **0.034**

PASS - Splitting capacity of timber exceeds design force in member

Member 2

Loaded edge distance — $h_{e.m2}$ = **73** mm

Characteristic splitting capacity - eq 8.4 — $F_{90.Rk2} = 14\ N/mm \times b_2 \times \sqrt{(Abs(h_{e.m2} \times 1mm^{-1} / (1 - (h_{e.m2} / h_2))))}$ = **7.59** kN

Design splitting capacity — $F_{90.Rd2} = k_{mod} \times F_{90.Rk2} / \gamma_{M.connection}$ = **4.67** kN

Design shear force — $F_{v.Ed2} = max(F_{la.sp1} \times Sin(\theta_{1.2.1}), F_{la.sp2} \times Sin(\theta_{2.2.3}))$ = **0.06** kN

Splitting utilisation — $ut\_Split_2 = Abs(F_{v.Ed2} / F_{90.Rd2})$ = **0.014**

PASS - Splitting capacity of timber exceeds design force in member

Member 3

Loaded edge distance $h_{e.m3}$ = **18125000173** mm

Characteristic splitting capacity - eq 8.4 $F_{90.Rk3}$ = 14 N/mm × b_3 × $\sqrt{}$(Abs($h_{e.m3}$ × 1mm^{-1} / (1 - ($h_{e.m3}$ / h_3)))) = **7.59** kN

Design splitting capacity $F_{90.Rd3}$ = k_{mod} × $F_{90.Rk3}$ / $\gamma_{M.connection}$ = **4.67** kN

Design shear force $F_{v.Ed3}$ = max($F_{la.sp2}$ × Sin($\theta_{2.3.2}$), $F_{la.sp3}$ × Sin($\theta_{3.3.4}$)) = **0.02** kN

Splitting utilisation ut_Split$_3$ = Abs($F_{v.Ed3}$ / $F_{90.Rd3}$) = **0.005**

PASS - Splitting capacity of timber exceeds design force in member

Member 4

Loaded edge distance $h_{e.m4}$ = **73** mm

Characteristic splitting capacity - eq 8.4 $F_{90.Rk4}$ = 14 N/mm × b_4 × $\sqrt{}$(Abs($h_{e.m4}$ × 1mm^{-1} / (1 - ($h_{e.m4}$ / h_4)))) = **7.59** kN

Design splitting capacity $F_{90.Rd4}$ = k_{mod} × $F_{90.Rk4}$ / $\gamma_{M.connection}$ = **4.67** kN

Design shear force $F_{v.Ed4}$ = max($F_{la.sp3}$ × Sin($\theta_{3.4.3}$), $F_{la.sp4}$ × Sin($\theta_{4.4.5}$)) = **-0.06** kN

Splitting utilisation ut_Split$_4$ = Abs($F_{v.Ed4}$ / $F_{90.Rd4}$) = **0.014**

PASS - Splitting capacity of timber exceeds design force in member

Member 5

Loaded edge distance $h_{e.m5}$ = **100** mm

Characteristic splitting capacity - eq 8.4 $F_{90.Rk5}$ = 14 N/mm × b_5 × $\sqrt{}$(Abs($h_{e.m5}$ × 1mm^{-1} / (1 - ($h_{e.m5}$ / h_5)))) = **11.88** kN

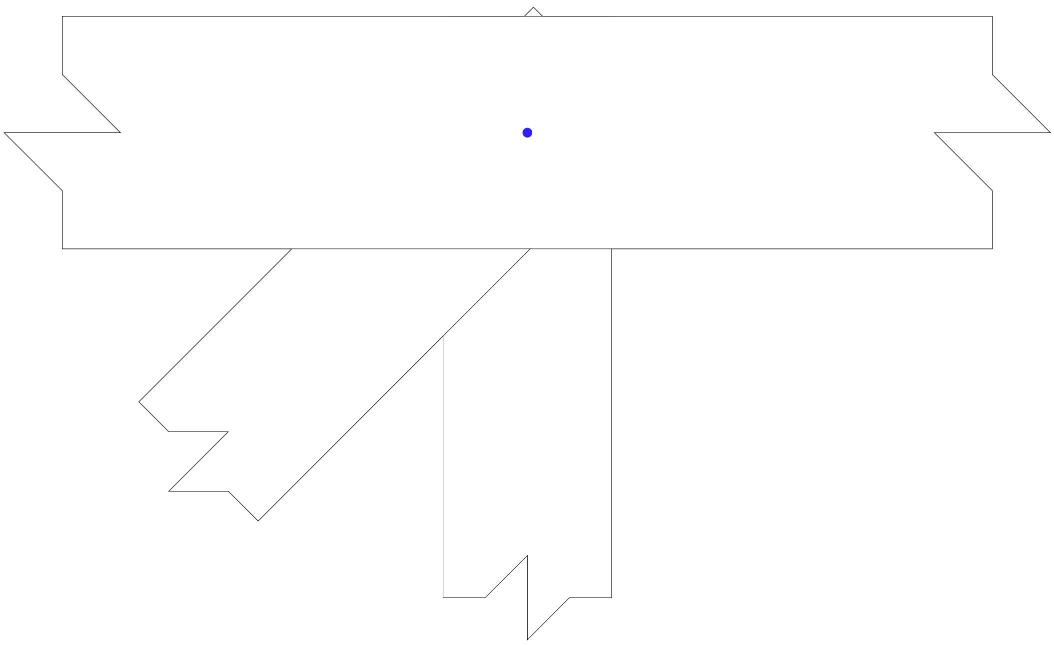
Design splitting capacity $F_{90.Rd5}$ = k_{mod} × $F_{90.Rk5}$ / $\gamma_{M.connection}$ = **7.31** kN

Design shear force $F_{v.Ed5}$ = $F_{la.sp4}$ × Sin($\theta_{4.5.4}$) = **-0.21** kN

Splitting utilisation ut_Split$_5$ = Abs($F_{v.Ed5}$ / $F_{90.Rd5}$) = **0.028**

PASS - Splitting capacity of timber exceeds design force in member

Fixing Spacing



Allowable minimum bolt spacings from table 8.4

Minimum spacings and edge / end distances.

a1 Spacing of fixings within one row parallel to grain,
a2 Spacing of rows perpendicular to grain
a3,c Distance between fixing and unloaded end
a3,t Distance between fixing and loaded end
a4.top Distance between fixing and unloaded/loaded edge, dependent upon shear plane angle
a4,bot Distance between fixing and unloaded/loaded edge, dependent upon shear plane angle

	a1	a2	a3,t	a3.c	a4.top	a4.bot
Member Member 1	48.1 mm	40.0 mm			31.8 mm	30.0 mm
Member Member 2	49.9 mm	40.0 mm	80.0 mm		37.8 mm	30.0 mm
Member Member 3	49.8 mm	40.0 mm	80.0 mm		30.0 mm	30.0 mm
Member Member 4	49.8 mm	40.0 mm	80.0 mm		30.0 mm	31.2 mm
Member Member 5	48.2 mm	40.0 mm			30.0 mm	31.8 mm

PASS - All spacing conditions are met

7) Tedds output: Multi Member - Nailed 2 member example

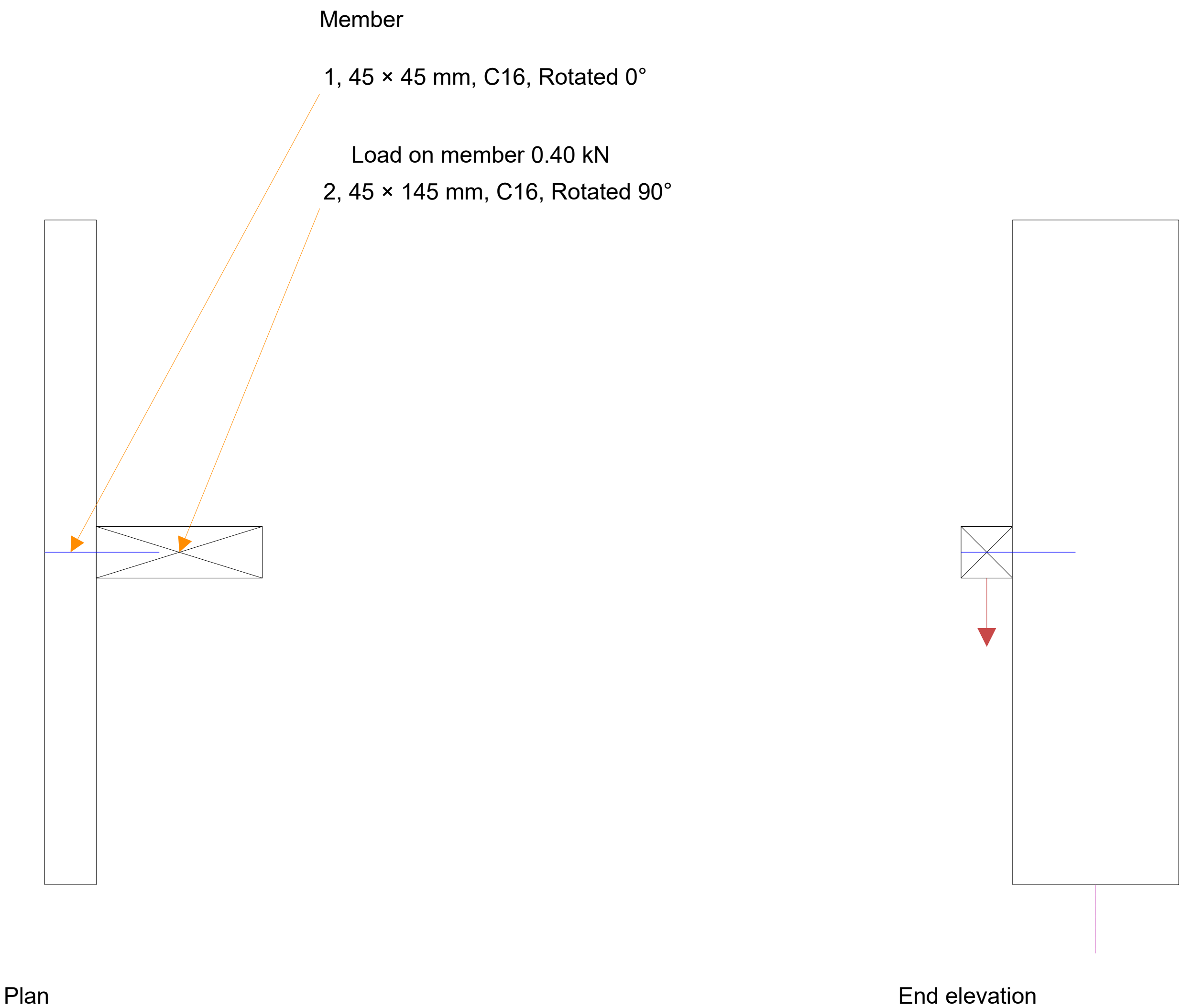
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

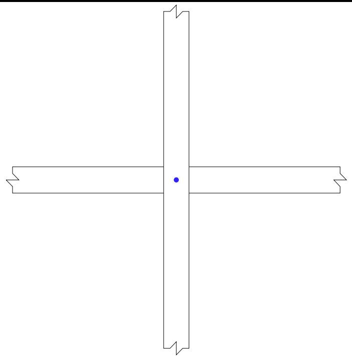
Tedds calculation version 1.1.07

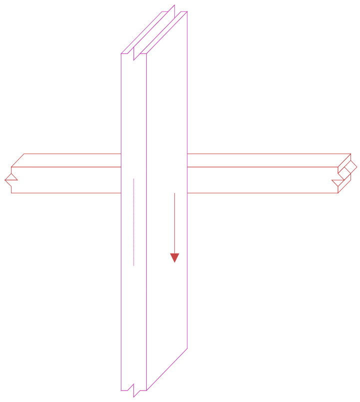
Design summary

Description	Provided	Required	Utilisation	Result
Pointside penetration	18.6 mm	47.3 mm	0.394	PASS
Edge/end spacing				PASS
Shear plane resistance			0.467	PASS
Splitting capacity of timber			0.112	PASS



Edinburgh Napier, COCIS Unit 1, 7 Hills Business Park Bankhead Crossway South Edinburgh EH11 4EP	Project				Job Ref.	
	Section				Sheet no./rev. 270	
	Calc. by AL	Date 15 October	Chk'd by	Date	App'd by	Date





Elevation

<u>This connection is in single shear</u>

Geometry

Nails

Nail description	3.10 mm x 100 mm ring shanked nail
Number of nails	$N_{fixings}$ = **1**
Number of rows	$NoR_{fixings}$ = **1**
Number per row	$NpR_{fixings}$ = **1**
Fixings to both sides	FBS = **1** No
Effective number of fixings - cl 8.3.1.1(8)	$n_{ef} = N_{fixings} \times FBS$ = **1**
Nail diameter	$d = d_f$ = **3.1** mm
Nail head diameter	$d_{h.f}$ = **6.975** mm
Nail length	l_f = **100.0** mm
Nail point length	$l_{point.f}$ = **7.75** mm
Nail pointside penetration	t_{pen} = **47.25** mm
Tensile strength of each fixing	$f_{u.f}$ = **700** N/mm^2
Minimum pointside penetration - cl 8.3.1.2(2)	$t_{min.pen} = 6 \times d$ = **18.6** mm
Pointside penetration	$t_{min.pen} / t_{pen}$ = **0.39**

PASS - Pointside penetration is acceptable

Member 1

Member type	Continuous member
Breadth	b_1 = **45** mm
Height	h_1 = **45** mm
Cross sectional area	A_1 = **2025** mm^2
Rotation about the Y-Y axis	0 °
Strength class	C16
Characteristic tension strength parallel to the grain	$f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m1}$ = **310** kg/m^3

Member 2

Member type	Continuous member
Breadth	b_2 = **145** mm
Height	h_2 = **45** mm
Cross sectional area	A_2 = **6525** mm^2
Rotation about the Y-Y axis	90 °
Strength class	C16

Characteristic tension strength parallel to the grain $f_{t.0.k.m2}$ = **10** N/mm^2

Characteristic density of the timber $\rho_{k.m2}$ = **310** kg/m^3

Partial safety factors

Material factor for connections, table 2.3 $\gamma_{M.connection}$ = **1.30**

Actions

Modification factors – Table 3.1

Service class of timber **2**

Load-duration **Instantaneous**

k_{mod} = **1.10**

Design action in member 1 $F_{la.1}$ = **0.00** kN Continuous member

Design load on member 1 $F_{la.ex1}$ = **0.40** kN

Calculated output

Angle of shear planes relative to the grain direction in each member

Shear plane	**Member**	**Joint**	**Angle of shear plane**
1	1	1.2.1	$\theta_{1.1.2}$ = **90** °
1	2	1.2.1	$\theta_{1.2.1}$ = **0** °

Design force in shear plane 1 $F_{la.sp1}$ = **0.40** kN

Characteristic embedment strength in timber, - eq 8.15

member	Joint	
1	1.2.1	$f_{h.k.1.2}$ = 82 kNm/kg × $\rho_{k.m1}$ × (d_f / 1mm)$^{-0.3}$ =**18.10** N/mm^2
2	1.2.1	$f_{h.k.2.1}$ = 82 kNm/kg × $\rho_{k.m2}$ × (d_f / 1mm)$^{-0.3}$ =**18.10** N/mm^2

Yield moment of fixing - eq 8.14 $M_{y.Rk}$ = 0.45 mm$^{0.4}$ × $f_{u.f}$ × $d_f^{2.6}$ = **5968** Nmm

Withdrawal resistance

cl 8.3.2(7) t_{pen} / d_f = **15.2** Dp = **1.00**

Characteristic values of the withdrawal and pull-through strengths

Pointside withdraw - eq 8.25 $f_{ax.k.ps}$ = 20 (m^5/(kg×s^2)) × $\rho_{k.m2}^2$ × Dp = **1.92** N/mm^2

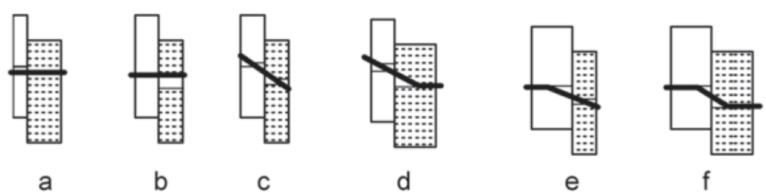
Headside pull-through - eq 8.26 $f_{head.k.hs}$ = 70 (m^5/(kg×s^2)) × $\rho_{k.m1}^2$ = **6.73** N/mm^2

Characteristic withdrawal capacity - eq 8.23 $F_{ax.Rk}$ = min($f_{ax.k.ps}$ × d_f × t_{pen}, $f_{head.k.hs}$ × $d_{h.f}^2$) = **0.282** kN

Design value of axial withdrawal capacity $F_{ax.Rd}$ =(k_{mod}× $F_{ax.Rk}$) / $\gamma_{M.connection}$ = **0.238** kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber single shear connections:



Maximum rope effect contribution - cl. 8.2.2(2)

cl 8.2.2(2) $P_{f.mod}$ = Profile$_{.f}$ = **1.50**

$P_{f.mod}$ – 1 = **50** %

Rope = $F_{ax.Rk}$ / 4 = **70** N

The characteristic lateral resistance per shear plane is the smallest value in equations (8.6)

Embedment ratio β = $f_{h.k.2.1}$ / $f_{h.k.1.2}$ = **1.00**

Thickness headside member t_1 = b_1= **45** mm

Penetration length in pointside member t_2 = t_{pen} = **47** mm

Failure mode (a)	$f.m.a = f_{h.k.1.2} \times t_1 \times d =$ **2.53** kN
Failure mode (b)	$f.m.b = f_{h.k.2.1} \times t_2 \times d =$ **2.65** kN
Failure mode (c)	$f.m.c_{Jyt} = f.m.a / (1 + \beta) \times (\sqrt{} [\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1) + (t_2 / t_1)^2) + \beta^3 \times ((t_2 / t_1)^2)] - \beta \times (1 + (t_2 / t_1)))$
	$f.m.c = \min(f.m.c_{Jyt} + Rope, P_{f.mod} \times f.m.c_{Jyt}) =$ **1.14** kN
Failure mode (d)	$f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{} [2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.1.2} \times t_1^2 \times d))] - \beta)$
	$f.m.d = \min(f.m.d_{Jyt} + Rope, P_{f.mod} \times f.m.d_{Jyt}) =$ **1.09** kN
Failure mode (e)	$f.m.e_{Jyt} = (1.05 \times (f_{h.k.1.2} \times t_2 \times d) / (1 + 2 \times \beta)) \times (\sqrt{} [2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.1.2} \times t_2^2 \times d))] - \beta)$
	$f.m.e = \min(f.m.e_{Jyt} + Rope, P_{f.mod} \times f.m.e_{Jyt}) =$ **1.13** kN
Failure mode (f)	$f.m.f_{Jyt} = 1.15 \times \sqrt{} [(2 \times \beta) / (1 + \beta)] \times \sqrt{} [(2 \times M_{y.Rk} \times f_{h.k.1.2} \times d)]$
	$f.m.f = \min(f.m.f_{Jyt} + Rope, P_{f.mod} \times f.m.f_{Jyt}) =$ **1.01** kN
Characteristic lateral nail shear resistance	**Failure mode (f)**
	$F_{v.Rk1} =$ **1.01** kN
Design resistance per fixing	$F_{v.Rd1} = (k_{mod} \times F_{v.Rk1}) / \gamma_{M.connection} =$ **0.86** kN
	$\theta_{1.1.2.Abs} = Abs(\theta_{1.1.2}) =$ **90** deg
	$\theta_{1.2.1.Abs} = Abs(\theta_{1.2.1}) =$ **0** deg
	$\theta_{min.SP1} = \min(\theta_{1.1.2.Abs},\theta_{1.2.1.Abs}) =$ **0** deg
Load utilisation factor	$ut\_load_1 = F_{la.sp1} / ((n_{ef} + (N_{fixings} - n_{ef}) \times (\theta_{min.SP1} / 90)) \times F_{v.Rd1}) =$ **0.467**

PASS - Design capacity of the shear plane exceeds the design force within shear plane

Splitting capacity of timber

Member 1

Loaded edge distance	$h_{e.m1} =$ **23** mm
Characteristic splitting capacity - eq 8.4	$F_{90.Rk1} = 14\ N/mm \times b_1 \times \sqrt{}(Abs(h_{e.m1} \times 1mm^{-1} / (1 - (h_{e.m1} / h_1)))) =$ **4.23** kN
Design splitting capacity	$F_{90.Rd1} = k_{mod} \times F_{90.Rk1} / \gamma_{M.connection} =$ **3.58** kN
Design shear force	$F_{v.Ed1} = F_{la.sp1} \times Sin(\theta_{1.1.2}) =$ **0.40** kN
Splitting utilisation	$ut\_Split_1 = Abs(F_{v.Ed1} / F_{90.Rd1}) =$ **0.112**

PASS - Splitting capacity of timber exceeds design force in member

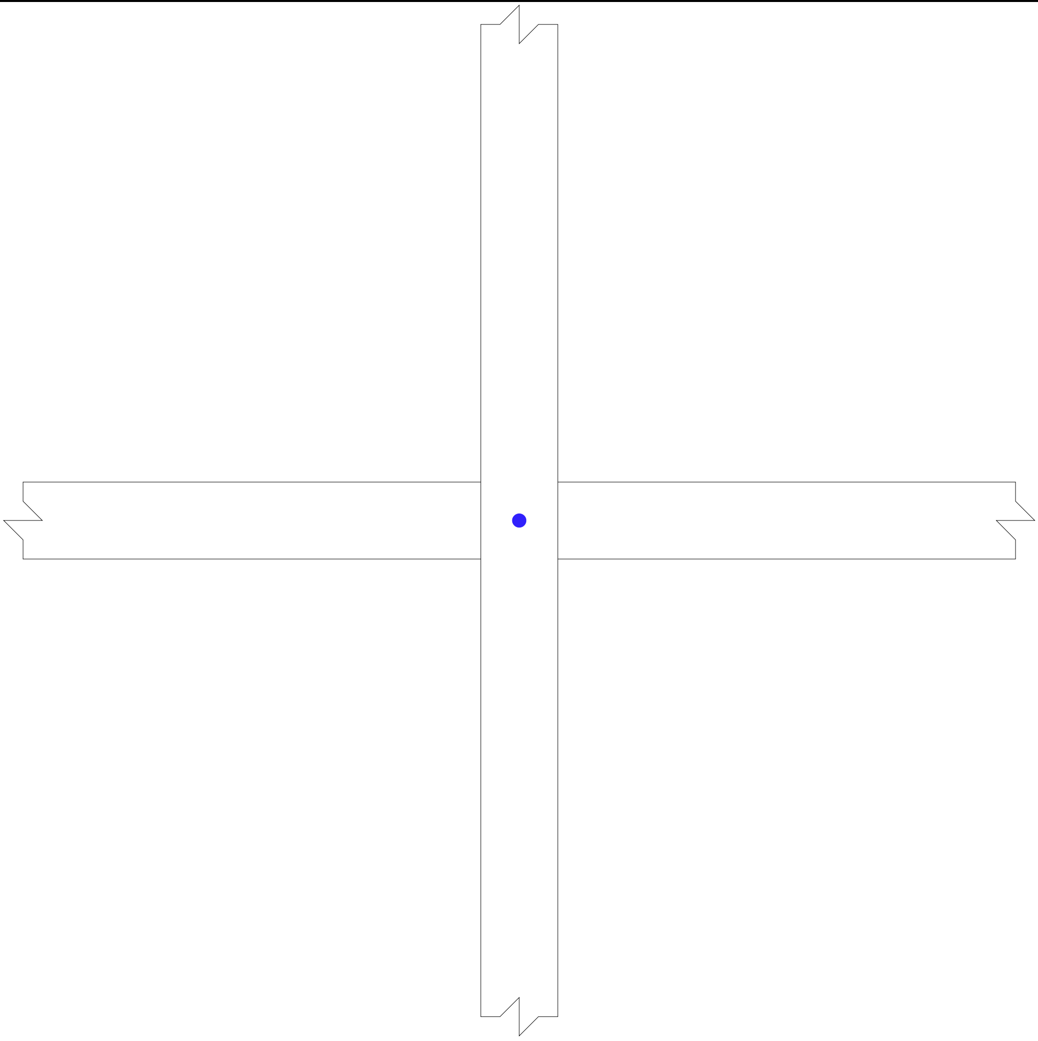
Member 2

Loaded edge distance	$h_{e.m2} =$ **6525000268** mm
Characteristic splitting capacity - eq 8.4	$F_{90.Rk2} = 14\ N/mm \times b_2 \times \sqrt{}(Abs(h_{e.m2} \times 1mm^{-1} / (1 - (h_{e.m2} / h_2)))) =$ **13.62** kN
Design splitting capacity	$F_{90.Rd2} = k_{mod} \times F_{90.Rk2} / \gamma_{M.connection} =$ **11.52** kN
Design shear force	$F_{v.Ed2} = F_{la.sp1} \times Sin(\theta_{1.2.1}) =$ **0.00** kN
Splitting utilisation	$ut\_Split_2 = Abs(F_{v.Ed2} / F_{90.Rd2}) =$ **0.000**

PASS - Splitting capacity of timber exceeds design force in member

Fixing Spacing

	Project				Job Ref.	
Edinburgh Napier, COCIS Unit 1, 7 Hills Business Park Bankhead Crossway South Edinburgh EH11 4EP	Section				Sheet no./rev. 273	
	Calc. by AL	Date 15 October	Chk'd by	Date	App'd by	Date



Allowable minimum nail spacings from table 8.2

Minimum spacings and edge / end distances.

a1 Spacing of fixings within one row parallel to grain,

a2 Spacing of rows perpendicular to grain

a3,c Distance between fixing and unloaded end

a3,t Distance between fixing and loaded end

a4.top Distance between fixing and unloaded/loaded edge, dependent upon shear plane angle

a4,bot Distance between fixing and unloaded/loaded edge, dependent upon shear plane angle

	a1	a2	a3,t	a3.c	a4.top	a4.bot
Member Member 1	15.5 mm	15.5 mm			21.7 mm	15.5 mm
Member Member 2	31.0 mm	15.5 mm			15.5 mm	15.5 mm

PASS - All spacing conditions are met

8) Tedds output: Tension Splice - Ply splice example

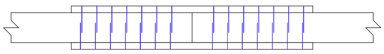
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

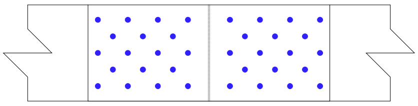
Tedds calculation version 1.1.07

Design summary

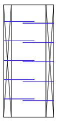
Description	**Provided**	**Required**	**Utilisation**	**Result**
Load utilisation factor	7.220 kN	4.500 kN	0.623	PASS
Pointside penetration	12.6 mm	24.8 mm	0.509	PASS
Column spacing	26.8 mm	45.0 mm	0.595	PASS
Row spacing	12.5 mm	25.0 mm	0.500	PASS
Edge spacing	10.5 mm	22.5 mm	0.467	PASS
Overlap			0.415	PASS



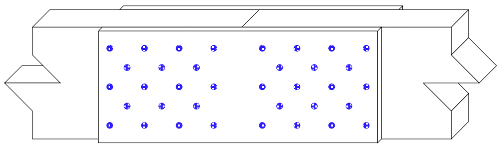
Plan



Elevation



End elevation



This connection is in single shear

Geometry

Nails

Nail description 2.10 mm x 40 mm ring shanked nail

Number of nails $N_{fixings}$ = **18**

Number of rows $NoR_{fixings}$ = **5**

Row spacing a_2 = **25.0** mm

Number per row $NpR_{fixings}$ = **4**

Column spacing a_1 = **45.0** mm

Fixings to both sides FBS = **2** Yes

Nail diameter $d = d_f$ = **2.1** mm

Nail head diameter $d_{h.f}$ = **4.725** mm

Nail length l_f = **40.0** mm

Nail point length $l_{point.f}$ = **5.25** mm

Nail pointside penetration t_{pen} = **24.75** mm

Tensile strength of each fixing $f_{u.f}$ = **700** N/mm^2

Effective number of fixings - cl 8.3.1.1(8) $n_{ef} = (NpR_{fixings}$^1.000 - 0.5) × FBS × $NoR_{fixings}$ = **35.00**

Minimum pointside penetration - cl 8.3.1.2(2) $t_{min.pen} = 6 \times d$ = **12.6** mm

Pointside penetration $t_{min.pen} / t_{pen}$ = **0.51**

PASS - Pointside penetration is acceptable

Edinburgh Napier UNIVERSITY	Project				Job Ref.	
Edinburgh Napier, COCIS Unit 1, 7 Hills Business Park Bankhead Crossway South Edinburgh EH11 4EP	Section				Sheet no./rev. 275	
	Calc. by AL	Date 15 October	Chk'd by	Date	App'd by	Date

Main member

Breadth	b_1 = **45** mm
Height	h_1 = **145** mm
Cross sectional area	A_1 = **6525** mm^2
Strength class	C16
Characteristic tensile strength parallel to the grain	$f_{t.0.k.m1}$ = **10** N/mm^2
Characteristic density of the timber	$\rho_{k.m1}$ = **310** kg/m^3

Plywood splice members

Breadth	b_2 = **10** mm
Height	h_2 = **145** mm
Minimum length	$len_{m2} = 2 \times (a_{3t.m1} + a_{3t.m2} + (NpR_{fixings}-1)\times a_1)$ = **362** mm
Characteristic density of the plywood	$\rho_{k.m2}$ = **500** kg/m^3
Characteristic compression perpendicular	$f_{c.90.k.m2}$ = **1.88** N/mm^2

If you are to allow a gap between the connection members this will need to be added to len_{m2}

Partial safety factors

Material factor for connections, table 2.3	$\gamma_{M.connection}$ = **1.30**

Actions

Modification factors – Table 3.1

Service class of timber	**1**
Load-duration	**Permanent**
	k_{mod} = **0.60**
Design tensile action	$F_{t.Ed}$ = **4.50** kN

Calculated output

Characteristic embedment strength in timber, - eq 8.15

member	Joint	
Main member,	eq 8.15	$f_{h.k.1} = 82\ kNm/kg \times \rho_{k.m1} \times (d / 1mm)^{-0.3}$ =**20.35** N/mm^2
Splice member,	plywood eq 8.20	$f_{h.k.2} = 110\ kNm/kg \times \rho_{k.m2} \times (d / 1mm)^{-0.3}$ =**44.02** N/mm^2
Yield moment of fixing - eq 8.14		$M_{y.Rk} = 0.45\ mm^{0.4} \times f_{u.f} \times d_f^{2.6}$ = **2168** Nmm

Withdrawal resistance

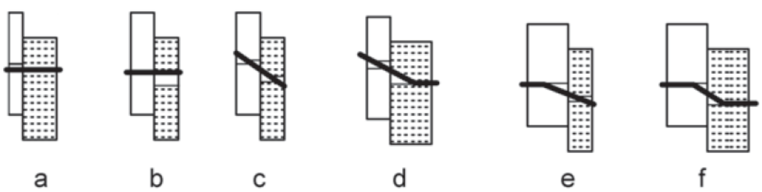
cl 8.3.2(7)	t_{pen} / d_f = **11.8**	Dp = **1.00**

Characteristic values of the withdrawal and pull-through strengths

Pointside withdrawal - eq 8.25	$f_{ax.k.ps} = 20\ (m^5/(kg\times s^2)) \times \rho_{k.m1}^2 \times Dp$ = **1.92** N/mm^2
Headside pull-through - eq 8.26	$f_{head.k.hs} = 70\ (m^5/(kg\times s^2)) \times \rho_{k.m2}^2$ = **17.50** N/mm^2
Characteristic withdrawal capacity - eq 8.23	$F_{ax.Rk} = \min(f_{ax.k.ps} \times d_f \times t_{pen}, f_{head.k.hs} \times d_{h.f}^2)$ = **0.1** kN
Design value of axial withdrawal capacity	$F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.046** kN

Lateral load-carrying capacity of connection

Failure modes for timber to timber single shear connections:



Maximum rope effect contribution - cl. 8.2.2(2)	
cl 8.2.2(2)	$P_{f.mod} = Profile_{.f}$ = **1.50**

$P_{f.mod} - 1 =$ **50** %

Rope = $F_{ax.Rk} / 4 =$ **25** N

The characteristic lateral resistance per shear plane is the smallest value in equations (8.6)

Embedment ratio $\beta = f_{h.k.1} / f_{h.k.2} =$ **0.46**

Thickness headside member $t_1 = b_2 =$ **10** mm

Penetration length in pointside member $t_2 = t_{pen} =$ **25** mm

Failure mode (a) f.m.a = $f_{h.k.2} \times t_1 \times d =$ **0.92** kN

Failure mode (b) f.m.b = $f_{h.k.1} \times t_2 \times d =$ **1.06** kN

Failure mode (c) $f.m.c_{Jyt} = f.m.a / (1 + \beta) \times (\sqrt{[\beta + 2 \times \beta^2 \times (1 + (t_2 / t_1) + (t_2 / t_1)^2) + \beta^3 \times (t_2 / t_1)^2]} - \beta \times (1 + (t_2 / t_1)))$

f.m.c = min($f.m.c_{Jyt}$ + Rope, $P_{f.mod} \times f.m.c_{Jyt}$) = **0.45** kN

Failure mode (d) $f.m.d_{Jyt} = (1.05 \times f.m.a / (2 + \beta)) \times (\sqrt{[2 \times \beta \times (1 + \beta) + ((4 \times \beta \times (2 + \beta) \times M_{y.Rk}) / (f_{h.k.2} \times t_1^2 \times d))]} - \beta)$

f.m.d = min($f.m.d_{Jyt}$ + Rope, $P_{f.mod} \times f.m.d_{Jyt}$) = **0.46** kN

Failure mode (e) $f.m.e_{Jyt} = (1.05 \times (f_{h.k.2} \times t_2 \times d) / (1 + 2 \times \beta)) \times (\sqrt{[2 \times \beta^2 \times (1 + \beta) + ((4 \times \beta \times (1 + 2 \times \beta) \times M_{y.Rk}) / (f_{h.k.2} \times t_2^2 \times d))]} - \beta)$

f.m.e = min($f.m.e_{Jyt}$ + Rope, $P_{f.mod} \times f.m.e_{Jyt}$) = **0.54** kN

Failure mode (f) $f.m.f_{Jyt} = 1.15 \times \sqrt{[(2 \times \beta) / (1 + \beta)]} \times \sqrt{[(2 \times M_{y.Rk} \times f_{h.k.2} \times d)]}$

f.m.f = min($f.m.f_{Jyt}$ + Rope, $P_{f.mod} \times f.m.f_{Jyt}$) = **0.60** kN

Characteristic lateral nail shear resistance **Failure mode (c)**

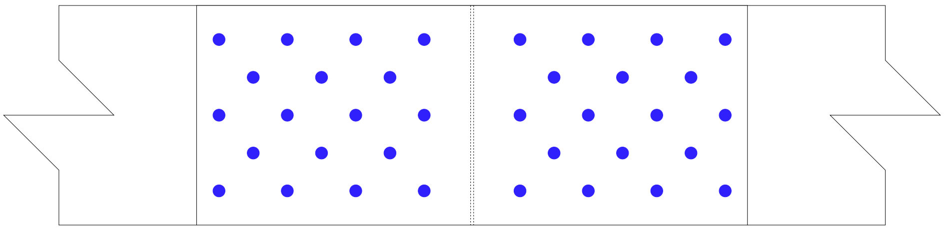
$F_{v.Rk1} =$ **0.45** kN

Design resistance per fixing $F_{v.Rd1} = (k_{mod} \times F_{v.Rk1}) / \gamma_{M.connection} =$ **0.21** kN

Load utilisation factor ut_load = $F_{t.Ed} / (n_{ef} \times F_{v.Rd1}) =$ **0.623**

PASS - Design capacity of the shear plane exceeds the design force within shear plane

Fixing Spacing



Allowable minimum nail spacings from table 8.2

Minimum spacings and edge / end distances.

a1 Spacing of fixings within one row parallel to grain,

a2 Spacing of rows perpendicular to grain

a3,t Distance between fixing and loaded end

a4,c Distance between fixing and unloaded edge

	a1	a2	a3,t	a4,c
Main member	17.9 mm	8.9 mm	31.5 mm	10.5 mm
Splice member	26.8 mm	12.5 mm	14.7 mm	6.3 mm

Applied column spacing $a_1 =$ **45.0** mm

Applied row spacing $a_2 =$ **25.0** mm

Applied edge distance, main member $App_{main} =$ **22.5** mm

Applied edge distance, splice member $App_{splice} =$ **22.5** mm

Minimum length of splice member $len_{m2} =$ **362** mm

PASS - All spacing conditions are met

9) Tedds output:Tension Splice - Steel splice example

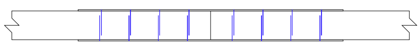
TIMBER CONNECTION DESIGN

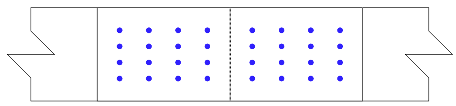
In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.1.07

Design summary

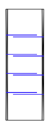
Description	Provided	Required	Utilisation	Result
Load utilisation factor	10.572 kN	10.000 kN	0.946	PASS
Pointside penetration	13.8 mm	32.3 mm	0.428	PASS
Column spacing	16.1 mm	45.0 mm	0.358	PASS
Row spacing	8.1 mm	25.0 mm	0.322	PASS
Edge spacing	11.5 mm	35.0 mm	0.329	PASS
Overlap			0.722	PASS

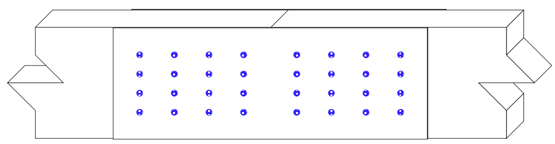




Plan

Elevation





End elevation

This connection is in single shear

Geometry

Nails

Nail description	2.30 mm x 40 mm ring shanked nail
Number of nails	$N_{fixings}$ = **16**
Number of rows	$NoR_{fixings}$ = **4**
Row spacing	a_2 = **25.0** mm
Number per row	$NpR_{fixings}$ = **4**
Column spacing	a_1 = **45.0** mm
Fixings to both sides	FBS = **2** Yes
Nail diameter	$d = d_f$ = **2.3** mm
Nail head diameter	$d_{h.f}$ = **5.175** mm
Nail length	l_f = **40.0** mm
Nail point length	$l_{point.f}$ = **5.75** mm
Nail pointside penetration	t_{pen} = **32.25** mm
Tensile strength of each fixing	$f_{u.f}$ = **700** N/mm^2
Effective number of fixings - cl 8.3.1.1(8)	$n_{ef} = (NpR_{fixings}$^1.000) × FBS × $NoR_{fixings}$ = **32.00**
Minimum pointside penetration - cl 8.3.1.2(2)	$t_{min.pen} = 6 \times d$ = **13.8** mm
Pointside penetration	$t_{min.pen} / t_{pen}$ = **0.43**

PASS - Pointside penetration is acceptable

Main member

Breadth b_1 = **45** mm

Height h_1 = **145** mm

Cross sectional area A_1 = **6525** mm^2

Strength class C16

Characteristic tensile strength parallel to the grain $f_{t.0.k.m1}$ = **10** N/mm^2

Characteristic density of the timber $\rho_{k.m1}$ = **310** kg/m^3

Steel splice members

Breadth b_2 = **2.0** mm

Height h_2 = **145** mm

If you are to allow a gap between the connection members this will need to be added to len_{m2}

Partial safety factors

Material factor for connections, table 2.3 $\gamma_{M.connection}$ = **1.30**

Actions

Modification factors – Table 3.1

Service class of timber **1**

Load-duration **Permanent**

k_{mod} = **0.60**

Design tensile action $F_{t.Ed}$ = **10.00** kN

Calculated output

Characteristic embedment strength in timber, - eq 8.15

member Joint

Main member, eq 8.15 $f_{h.k.1}$ = 82 kNm/kg × $\rho_{k.m1}$ × (d / 1mm)$^{-0.3}$ =**19.80** N/mm^2

Yield moment of fixing - eq 8.14 $M_{y.Rk}$ = 0.45 mm$^{0.4}$ × $f_{u.f}$ × $d_f^{2.6}$ = **2747** Nmm

Withdrawal resistance

cl 8.3.2(7) t_{pen} / d_f = **14.0** Dp = **1.00**

Characteristic values of the withdrawal and pull-through strengths

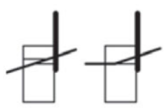
Pointside withdrawal - eq 8.25 $f_{ax.k.ps}$ = 20 (m^5/(kg×s^2)) × $\rho_{k.m1}^2$ × Dp = **1.92** N/mm^2

Characteristic withdrawal capacity - eq 8.23 $F_{ax.Rk}$ = $f_{ax.k.ps}$ × d_f × t_{pen} = **0.1426** kN

Design value of axial withdrawal capacity $F_{ax.Rd}$ =(k_{mod}× $F_{ax.Rk}$) / $\gamma_{M.connection}$ = **0.0658** kN

Lateral load-carrying capacity of connection

Linear interpolation of the failure modes for timber to steel plate single shear connections, when the steel plate thickness is greater that 0.5d and less than d, with the tolerance on the hole diameters being less than $0.1d_{outer.f}$:



a b

Maximum rope effect contribution - cl. 8.2.2(2)

cl 8.2.2(2) $P_{f.mod}$ = $Profile_{.f}$ = **1.50**

$P_{f.mod}$ – 1 = **50** %

Rope = $F_{ax.Rk}$ / 4 = **36** N

The characteristic lateral resistance per shear plane is the smallest value in equations (8.9)

Pointside penetration t_1 = t_{pen} = **32** mm

Failure mode (a) — f.m.a = 0.4 × $f_{h.k.1}$ × t_1 × d = **0.59** kN

Failure mode (b) — f.m.b_{Jyt} = 1.15 × √ [2 × $M_{y.Rk}$ × $f_{h.k.1}$ × d]

f.m.b = min(f.m.b_{Jyt} + Rope, $P_{f.mod}$ × f.m.b_{Jyt}) = **0.61** kN

Characteristic lateral nail shear resistance (8.9) — **Failure mode (a)**

$F_{v.Rk1.ab}$ = **0.59** kN

c d e

The characteristic lateral resistance per shear plane is the smallest value in equations (8.10)

Pointside penetration — t_1 = t_{pen} = **32** mm

Failure mode (c) — f.m.c = $f_{h.k.1}$ × t_1 × d = **1.47** kN

Failure mode (d) — f.m.d_{Jyt} = $f_{h.k.1}$ × t_1 × d × (√[2 + ((4 × $M_{y.Rk}$) / ($f_{h.k.1}$ × d × $(t_1)^2$))] - 1)

f.m.d = min(f.m.d_{Jyt} + Rope, $P_{f.mod}$ × f.m.d_{Jyt}) = **0.76** kN

Failure mode (e) — f.m.e_{Jyt} = 2.3 × √[$M_{y.Rk}$ × $f_{h.k.1}$ × d]

f.m.e = min(f.m.e_{Jyt} + Rope, $P_{f.mod}$ × f.m.e_{Jyt}) = **0.85** kN

Characteristic lateral nail shear resistance (8.10) — **Failure mode (d)**

$F_{v.Rk1.cde}$ = **0.76** kN

Linear interpolation — m_{inter} = ($F_{v.Rk1.cde}$ - $F_{v.Rk1.ab}$) / (d – (0.5 × d)) = **151** kN/m

c_{inter} = $F_{v.Rk1.ab}$ – (m_{inter} × (0.5 × d)) = **414** N

Characteristic lateral nail shear resistance — $F_{v.Rk1}$ = m_{inter} × b_2 + c_{inter} = **0.716** kN

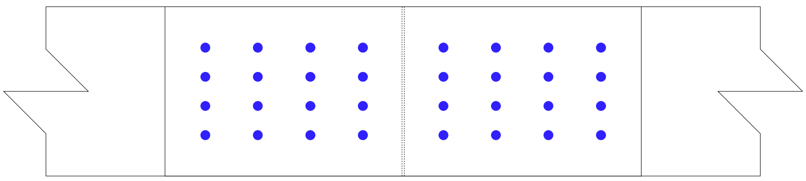
Design resistance per fixing — $F_{v.Rd1}$ = (k_{mod} × $F_{v.Rk1}$) / $\gamma_{M.connection}$ = **0.33** kN

Load utilisation factor — ut_load = $F_{t.Ed}$ / (n_{ef} × $F_{v.Rd1}$) = **0.946**

PASS - Design capacity of the shear plane exceeds the design force within shear plane

The load-carrying capacity of the steel plates should be calculated independently by the user.

Fixing Spacing



Allowable minimum nail spacings from table 8.2

Minimum spacings and edge / end distances.

a1 — Spacing of fixings within one row parallel to grain,

a2 — Spacing of rows perpendicular to grain

a3,t — Distance between fixing and loaded end

a4,c — Distance between fixing and unloaded edge

	a1	a2	a3,t	a4,c
Main member	16.1 mm	8.1 mm	34.5 mm	11.5 mm
Splice member				

Applied column spacing — a_1 = **45.0** mm

Applied row spacing — a_2 = **25.0** mm

Applied edge distance, main member — App_{main} = **35.0** mm

Applied edge distance, splice member — App_{splice} = **35.0** mm

Minimum length of splice member — len_{m2} = **408** mm

PASS - All spacing conditions are met

10) Tedds output: Axially loaded fixing - Roof batten 75mm insulation void example

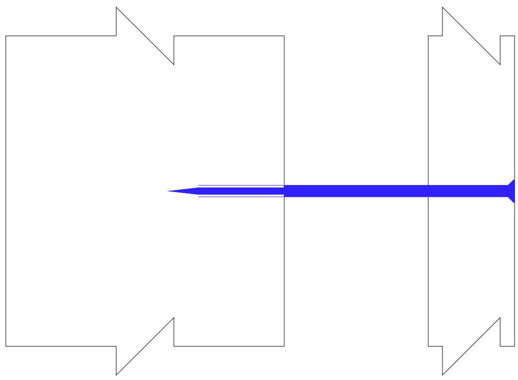
TIMBER CONNECTION DESIGN

In accordance with EC5 and the UK National Annex incorporating National Amendment No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.1.07

Design summary

Description	Provided	Required	Utilisation	Result
Load utilisation factor	0.906 kN	0.800 kN	0.883	PASS
Pointside penetration	36.0 mm	44.6 mm	0.807	PASS

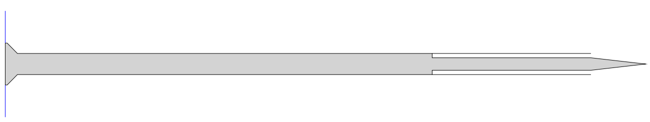


Section

Geometry

Screws

Description	6.0 mm / 3.6 mm x 180 mm screw
Effective number of fixings	$n_{ef.ax}$ = 1 = **1.00**
Head diameter	$d_{h.f}$ = **11.800** mm
Head length	$l_{n.l.f}$ = **0.500** mm
Smooth shank diameter	d_f = **6.00** mm
Outer thread diameter	$d_{outer.f}$ = **6.00** mm
Inner thread diameter	$d_{inner.f}$ = **3.55** mm
Total length	l_f = **180.0** mm
Thread length, including the point	$l_{th.f}$ = **60.0** mm
Point length	$l_{point.f}$ = **15.4** mm
Tensile strength of each fixing	$f_{u.f}$ = **600** N/mm^2
Thread pointside penetration	$t_{pen.th}$ = t_{pen} = **44.60** mm
Minimum pointside penetration - cl 8.7.2(3)	$t_{min.pen}$ = 6 × $d_{outer.f}$ = **36.0** mm
Counter sunk head	



Section of Screw

Pointside penetration $t_{min.pen} / t_{pen.th}$ = **0.81**

PASS - Pointside penetration is acceptable

Point side member

Depth h_1 = **145** mm

Strength class C24

Characteristic tensile strength parallel to the grain $f_{t.0.k.m1}$ = **14** N/mm^2

Characteristic density of the timber $\rho_{k.m1}$ = **350** kg/m^3

Head side member

Depth h_2 = **45** mm

Strength class C24

Characteristic tensile strength parallel to the grain $f_{t.0.k.m2}$ = **14** N/mm^2

Characteristic density of the timber $\rho_{k.m2}$ = **350** kg/m^3

Void between head side and point side member vod = **75.0** mm

Check to validate that no pre-drilling is acceptable

Characteristic dencity < 500 kg/m^3 - cl 8.3.1.1(2) $\rho_{k.m1}$ / 500kg/m^3 = **0.70** OK

$\rho_{k.m2}$ / 500kg/m^3 = **0.70** OK

Diameter of fixing < 6mm - cl 8.3.1.1(2) d_f / 6mm = **1.00** Not suitable

Timber thickness > t_{min} - cl 8.3.1.2(6)

t_{min} = max (7 × d_f, (13 × d_f - 30mm) × (max($\rho_{k.m1}$, $\rho_{k.m2}$) / 400kg/m^3)) = **42** mm OK

FAIL - Requires to be predrilled

<u>Partial safety factors</u>

Material factor for connections, table 2.3 $\gamma_{M.connection}$ = **1.30**

<u>Actions</u>

Modification factors – Table 3.1

Service class of timber **3**

Load-duration **Instantaneous**

k_{mod} = **0.90**

Design axial action $F_{ax.Ed}$ = **0.80** kN

<u>Calculated output</u>

Withdrawal resistance

Penetration length of the threaded part $l_{ef.f} = t_{pen.th}$ = **44.60** mm

Min. angle screw axis - grain direction point side α_{screw} = **90**°

Characteristic values of the withdrawal and pull-through strengths

Characteristic values of the withdrawal and pull-through strengths

Withdraw capacity, User entered $f_{ax.k.pss}$ = **11** N/mm^2

Associatated density $\rho_{a.ax.k.pss}$ = **350** kg/m^3

Withdrawal capacity - eq 8.40a $F_{ax.point.Rk} = f_{ax.k.pss} \times d_{outer.f} \times l_{ef.f} / (1.2 \times \cos(\alpha_{screw})^2 + \sin(\alpha_{screw})^2) \times (\rho_{k.m1} / \rho_{a.ax.k.pss})^{0.8}$ = **2944** N

Headside pull-through, User entered $f_{head.k.hss}$ = **9.4** N/mm^2

Associatated density $\rho_{a.head.k.hss}$ = **350** kg/m^3

Pull-through resistance - eq 8.40b $F_{ax.head.Rk} = f_{head.k.hss} \times d_{h.f}^2 \times (\rho_{k.m2} / \rho_{a.head.k.hss})^{0.8}$ = **1309** N

$F_{ax.Rk} = \min(F_{ax.point.Rk}, F_{ax.head.Rk})$ = **1.309** kN

Design value of axial withdrawal capacity $F_{ax.Rd} = (k_{mod} \times F_{ax.Rk}) / \gamma_{M.connection}$ = **0.906** kN

Load utilisation factor $\text{ut\_load} = F_{ax.Ed} / F_{ax.Rd} = \mathbf{0.883}$

PASS - Design axial withdrawal capacity exceeds the design axial action

Fixing Spacing

Allowable minimum screw spacings for point side member from table 8.6

Allowable minimum screw spacings for head side member from table 8.6

Minimum edge / end distances.

a3,c Distance between fixing and unloaded end

a4,c Distance between fixing and unloaded edge

a1,CG End distance of the centre of gravity of threaded part of the screw in member

a2,CG Edge distance of the centre of gravity of threaded part of the screw in member

	a3,c	a4,c	a1,CG	a2,CG
Point side member			60.0 mm	24.0 mm
Head side member			60.0 mm	24.0 mm