

Behaviour of Hybrid Timber-Steel Beam-to-Column Connections

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I confirm that this submission is my own work. In it, I give references and citations whenever I refer to, describe or quote from the published, or unpublished, work of others.

Vasileios Karagiannis

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Dedications

To my parents

ΑΡΙΣΤΕΙΔΗΣ και ΚΑΤΕΡΙΝΑ

Εάν ταίς γλώσσαις τών ανθρώπων λαλώ καί τών αγγέλων,
αγάπην δε μη έχω γέγονα χαλκός ηχών ή κύμβαλον αλαλάζον.

Paul

Corinthians

Chapter 13, verse 1

Μηδένα πρό τοῦ τέλους μακάριζε
Σόλων

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Abstract

This thesis studies the behaviour of hybrid systems consisting of tubular steel columns and laminated glulam timber beams. The research includes experimental investigations at the material and structural system levels as well as several numerical and assessments. Firstly, an extensive experimental programme is conducted on the determination of the material properties of the glulam. This involve compressive, tensile, shear and bending material tests. In addition, one-dowel connection tests are carried out to examine the interaction between the steel fastener and the timber volume around the fastener hole with the aid of Digital Image Correlation (DIC) techniques. Subsequently, detailed three-dimensional detailed models are constructed and their results are compared with the experimental tests. To account for the material damage, the foundation zone approach was used. This model was shown to be able to correctly simulate the crushing response of wood in the embedding region. The implementation of this region into the model allows an accurate simulation of the damage accumulation process. To this end, new relationships were formulated as part of this thesis, that can be used to estimate the material characterisation as a function of the crushing volume. The proposed model allows for a relatively low dependence on the radius of the foundation volume adopted making it applicable to a wider range of varying geometrical configurations. The validity and accuracy of the proposed modified foundation models were examined against the experimental force-displacement curves, and good agreement was found between the experimental response and the numerical simulations

The second experimental programme is concerned with the performance of timber beam-steel column assemblages. Two timber beam-to-steel column alternatives are examined: a) top and seat angle connection and b) slotted-in T-stub connection with bolts. The configuration of the connections and their set-up are presented, followed by detailed results in the form of figures and tables as well as observations from the tests. The main behavioural patterns are identified and key response characteristics such as stiffness, capacity and failure mechanisms are discussed. Detailed finite element models were created to simulate the experimental timber beam-to-steel column connection tests. The models included advance features such as contact phenomena, bolt pretension and orthotropic material definitions. The foundation zone approach developed in this thesis was implemented. The results of the models were validated against the experimental results and good agreement was found. Additionally, component-based models were also formulated for the prediction of the flexural response of the connection types tested.

These expressions were developed to estimate the stiffness, capacity and ultimate loads of the connections and the results were validated against the experimental and numerical findings. The component models proposed were able to predict the response of the connection accurately including those cases where screws and bottom-wedge angles were involved.

In the concluding part of the thesis, the developed finite element models are employed in a parametric assessment in order to highlight the influence of key geometric and material considerations. The dearth of information on timber-steel hybrid connection of this type, design methodologies must be developed to offer a preliminary assessment on the prediction and evaluation of the key characteristics and hence the results are also compared with simplified analytical expressions. Finally, the last section of the thesis summarised all the findings and numerous possible future research are identified.

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Notations

d :	Fastener diameter
d_{SB} :	Diameter of the standard bolts
d_w :	Bearing capacity of the steel plate inserted into the glulam beam
d_s :	Diameter of the screw
E_1 :	Young's modulus parallel to the direction of the grain
E_2 :	Young's modulus perpendicular to the direction of the grain
$E_{i,Emb}$:	Embedment Young's modulus
σ_b :	Bending strength
σ_{Emb} :	Embedment stress
σ_{FZ} :	Stress of the foundation zone
σ_{11} :	Stress acting parallel to the direction of the grain
σ_{22} :	Stress acting perpendicular to the direction of the grain
σ_{12} :	Shear stress
$\sigma_{11,emb,k}$:	Characteristic embedment strength parallel to the grain
$\sigma_{y,f}$:	Yield stress of the angle
σ_{ub} :	Tensile strength the appropriate bolt grade
$\sigma_{12,k}$:	Characteristic shear strength of glulam
$F_{t,Rd}$:	Tension resistance of the bolts
$F_{ax,Rk}$:	Characteristic withdrawn capacity of the bolt
$F_{v,Rk,conf1}$:	Characteristic load-carrying capacity of a steel-to-timber connection
$F_{v,conf1}$:	Effective design load-carrying capacity
F_{shear} :	Shear capacity of Type 1 connection

$F_{T,Rd}$:	Design resistance of the blind-bolted angle
$F_{m,d,max}$:	Force acting at Bolt-1
$F_{m,d,max,h}$:	Horizontal component of the force acting on Bolt-1
$F_{m,d,max,h,k}$:	Characteristic horizontal component of the force acting on Bolt-1
$F_{d,H,row1}$:	Total effective horizontal force acting on Bolt row-1
F_{shear_limit} :	Capacity of plug shear failure
$F_{screw i}$:	Screw force
R_i :	Strength associated with screw failure
$M_{y,Rk}$:	Characteristic yield moment of the bolt
$M_{p,angle}$:	Yield capacity of Type 1 connection
M_{shear} :	Moment capacity at ultimate associated with brittle shear failure
M_d :	Applied moment
$M_{d,y}$:	Moment capacity of Type 1 connection
$M_{d,PS}$:	Moment capacity for plug shear failure
$M_{y,s}$:	Yield moment of the screw
M_T :	Total moment capacity
$M_{connection}$:	Yield capacity of Type 2 connection
$M_{d,angle}$:	Moment of wedge angle or Type 2 connection
A_t :	Tensile stress area
$n_{b,ef}$:	Effective number of fasteners connecting the angle to the glulam beam in the direction parallel to the grain
n_b :	Number of bolts in the direction parallel to the width of the width
$n_{s,pl}$:	Number of bolt-rows.
n_{row1} :	Number of bolts in the direction parallel to the grain
$n_{row1,ef}$:	Effective number of bolts in the direction parallel to the grain

$\gamma_M :$	Material safety factor
$h_{gl} :$	Height of the glulam beam
$h_{ef} :$	Effective height
$h_A :$	Length of the smaller leg of the angle excluding its thickness
$l_{ef1} :$	Effective length
$l_{ef} :$	Effective depth
$G :$	Shear modulus
$\beta_1, \beta_2, \gamma_1, \gamma_2 :$	Empirical parameters
$w_f :$	Width of the angle
$w_b :$	Width of the glulam beam
$t :$	Width of the glulam beam
$t_f :$	Thickness of the angle
$t_1 :$	Distance parallel to the width of the beam between the side of the beam to the inserted metal plate
$t_{Tstub} :$	Thickness of the metallic plate
$r_{fill} :$	Corner radius of the angle
$r_{max} :$	Position vector of Bolt-1 with respect to the point of rotation
$r_i :$	Distance of fastener of the point of rotation for Type 2 connection
$y_1 :$	Vertical distance between bolt-1 and the centre of rotation
$y_A :$	Distance between point of rotation and $h_A/2$
$D :$	Summation of all fasteners radii $r_i, i \in [1, n]$ squared
$P :$	Ratio between the radius if the foundation zone and diameter of the fastener.
$K_{CF} :$	Bearing stiffness of the column face
$K_{HB} :$	Hollo-bolt axial stiffness

K_t :	Tension stiffness of the angle's horizontal leg
$S_{config1,init}$:	Rotation stiffness of Type 1 connection
$S_{config2}$:	Rotation stiffness of Type 2 connection
K_{ser} :	Slip modulus of each individual fastener
ρ :	Density of the glulam beam.
l :	Length or beam's span.
b :	Width.
h :	Height.

Chapter 1

Introduction

1.1 Background

Hybrid structures have a long history of development and use in civil infrastructure due to the beneficial effects brought about by the synergetic combination of different structural materials with complementary properties. For this purpose, the use of structural timber elements has many advantages, especially in terms of carbon storage potential, economy of construction and lightness. On the other hand, the architectural versatility of framed construction, relative to solid wall type buildings, can be further enhanced by the aesthetic and structural efficiency of tubular columns. Furthermore, the use of tubular columns in combination with timber beams and floors has the advantage of requiring smaller column sections (with respect to full timber moment resisting frames) and a relatively less constrained connection configuration due to the avoidance of the perpendicular-to-grain weaknesses of a wooden column. However, the perceived difficulties in establishing appropriate connections between timber beams and tubular steel columns and the lack of adequate modelling strategies limit the application of this hybrid solution and can lead to the many advantages of hybrid structures incorporating timber elements to go under-exploited.

The importance of connections in framed structures cannot be over-emphasised. They not only constitute the part of the assembly that is most susceptible to initial failure but their behaviour can determine the response of overall structural systems. In fact, it is not possible to make realistic estimations of the structural behaviour of framed buildings without a detailed understanding of their connections [1,2]. In this context, significant research has been recently conducted on practical semi-rigid connection alternatives in steel-only frames with tubular columns [3-6]. This thesis aims to provide practical solutions for connecting glulam timber beams and tubular steel columns and to provide essential information on the interaction between their constituent components as well as tools for their modelling and design.

1.2 Scope and objectives

Although, as noted before, the response of steel-to-steel joints with tubular members is fairly well understood and there is increasingly more information on the design and assessment of blind-bolted connections to tubular columns, a dearth of understanding exists on the interaction between timber beams and steel tubular columns. This lack of understanding is particularly notorious with relation to the flexural response of dowel-type fastening groups and the characterization and prevention of associated brittle failure modes. Similarly, accurate and efficient numerical modelling strategies to simulate the intricate interactions between the wood and the metal fastener during embedment are lacking. To this end, this thesis focuses on proposing and examining the behaviour of hybrid timber beam-to-steel tubular column connections, with emphasis on the estimation and prevention of potentially brittle failure modes. The provision of numerical models and simplified expression for the characterisation of behaviour of such connections is also part of the scope of the present study.

1.3 Outline of the thesis

This thesis encompasses nine chapters. The present chapter provides a general introduction and sets out the purpose and objectives of the research. Chapter 2 presents the literature review on previous and relevant research that has been conducted on timber-to-steel components and sets the context for the contributions of the thesis.

Chapter 3 deals with the experimental characterization of the glulam beams employed in this thesis. It describes the various experimental setups and presents and discusses their corresponding test results. These tests involved compression and tension tests parallel and perpendicular to the grain, shear tests and large-scale four-point bending tests. The results of Chapter 3 inform the experimental work and Finite Element (FE) simulations conducted later in the thesis.

A series of embedment tests are presented in Chapter 4. The experimental setup and test results as well as their implications are carefully examined. The tests conducted involved both parallel to grain and perpendicular to grain embedding of steel dowels of different diameters.

Chapter 5 discusses the numerical analysis of the embedment behaviour of in timber specimens and proposes a numerical approach to deal with the accumulation of damage in the embedding zone using the Finite Element package ABAQUS 6.14 [7]. A full description on the proposed modelling technique is presented together with its validation against the experimental results from Chapter 4.

The experimental response of two types of glulam beam-to-tubular column connections is presented in Chapter 6. The chapter introduces the joint configurations and test set-up employed. From the experimental results, the main behavioural patterns are identified and key response characteristics such as stiffness, capacity and failure mechanism are discussed.

Chapter 7 presents three-dimensional FE models as well as component-based approaches developed to simulate the response of the connections introduced in Chapter 6. A detailed discussion of the modelling methods and their validation is presented in this chapter.

Chapter 8 builds upon the findings presented in Chapter 7 and examines the response of the two studied connection configurations over a wider range of geometric and mechanical parameters beyond those already examined in previous chapters. Attention is placed on the evolution of key design parameters such as rotational stiffness and connection yield moment capacity and the adequacy of the simplified component-based estimations is verified. Finally, the closing chapter summarizes the main conclusions and contribution of the thesis and provides recommendations for future work.

Chapter 2

Literature Review

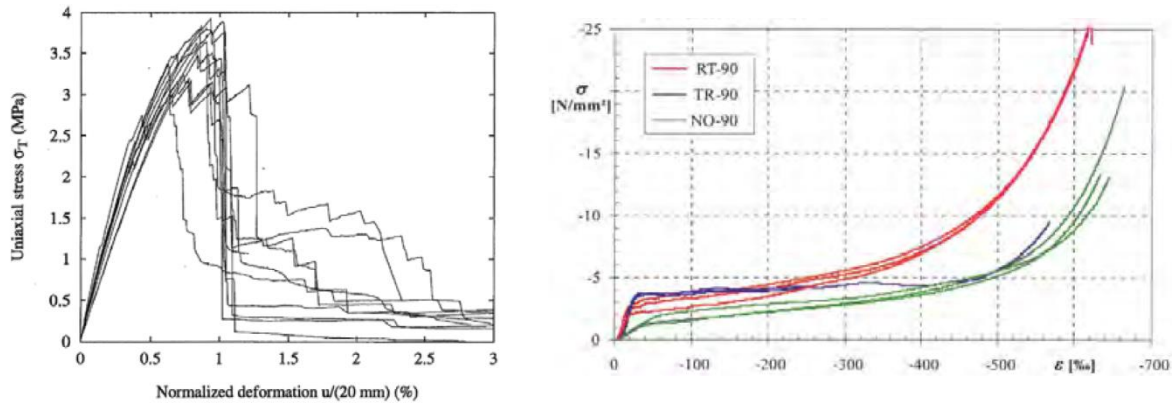
2.1 Introduction

A hybrid structure is one in which two or more distinct materials are utilised synergistically. Such structures have the potential to achieve significant levels of efficiency since the limitations of one material can be offset by the strengths of the other(s). For example, the high strength of steel under tension and the high strength of timber under compression can be used in structures or elements where steel components resist mainly tensile loads whereas timber components and elements resist largely compressive loads. It follows that an in-depth understanding of the overall properties of each constituent material is a prerequisite for the optimal design and construction of such systems. In this context, numerous studies such as [8,9] have examined the response of hybrid systems formed by reinforced concrete beams and steel columns. In contrast, despite their merits, research on the response of timber-steel hybrid building systems is less common. There are several examples that indicate a renewed interest on timber constructions [10-13]. Wood has about two thirds the weight of steel and one sixth the weight of reinforced concrete [14,15,16]. This lightness offers several advantages in the design of large structures. Available investigations include the study of Dickof et al. [17] which proposed to combine the lightness, economy and sustainable characteristics of laminated timber panels with the ductility of steel beam-column elements to create a multi-storey timber-steel framed structure of high lateral stiffness. Likewise, the use of hybrid timber-steel flooring systems formed of Cross-Laminated Timber (CLT) and steel beams has been studied by Okutu et al. [18]. They showed that the use of timber flooring systems results in a significantly lighter structure leading to reduced column sizes and more economical foundations. Recent examples of the implementation of timber-steel hybrid systems include the Earth Science Building of the University of British Columbia in Canada [19] and the Banyan Wharf building in the UK [20] that employ moment resisting and shear wall configurations, respectively. In all cases, an adequate understanding of the structural response requires an explicit consideration of the behaviour of their connections between main structural elements since its behaviour will have a significant impact on the overall performance of the full structure.

This chapter starts by introducing timber as a material highlighting the various benefits that are associated with glued laminated timber. It continues by referring to beam-to-column connection, their importance within framed buildings and by summarizing the various connection classifications. Moreover, a number of timber connection with and without metal fasteners are mentioned and discussed following a relevant literature survey. After addressing the two types of hybrid connection that are the focus of this thesis, the present chapter identifies the different components that compromise a typical beam-to-column connection and briefly refers to their failure modes. Subsequently, steel dowel-type connections between timber and steel are introduced including previous experimental and analytical studies as well as current design guidelines. Alternative modelling approaches for timber connections are also discussed. The chapter closes by addressing the issues of modelling embedment on timber and highlighting specific gaps in current knowledge related to the response of semi-rigid bolted connections between tubular steel columns and timber beams.

2.2 Timber as a structural material

Timber is a complex foam fibre-reinforced composite material with anisotropic properties, i.e. its mechanical properties are directionally dependant. Globally, there are thousands of tree species, each having different growth rates, structural properties and durability levels. The high variability of timber, results in a high scatter of the experimental data associated with its mechanical properties typical of any biological material. An example of experimentally obtained stress-strain curves for tensile tests on *Picea abies* (spruce) specimens is presented in Figure 2.1. Figure 2.1a shows the results obtained by Poulsen [21] where timber specimens were subjected to tensile forces perpendicular to grain, whereas Figure 2.1b shows the curves obtained by Franke [22] for timber cubes, with three different annual ring orientations, compressed perpendicular to grain.



(a) Tension perpendicular to grain [21].

(b) Compression perpendicular to grain [22].

Figure 2.1 – Stress-strain curves for Picea abies specimens loaded perpendicular to grain.

The general properties of wood in general are specified by nine mechanical properties: Young's modulus E , the Poisson's ratio ν and the shear modulus G in each of the three orthotropic directions. Woods hand books [23] and various publications [24-28] can provide valuable information on the mechanical properties of many wood species. Other mechanical properties of wood such as damage index and embedment behaviour are less well characterised. Although expressions exist to correlate those with other wood parameters like density [29] their variation for a full range of practical geometric configurations is still not well understood. In particular, the influence of size effects, load direction and fastener relative dimensions are subject of current study [30, 31, 32].

Many of the advantages of wood as structural material spring from its natural origin, as do many of the challenges associated with its structural engineering use. For instance, as previously mentioned the mechanical strength of wood is different even along the direction of the same grain. Hence the benefits of laminated timber in achieving a greater degree of uniformity in the mechanical properties of timber comparable to that of other construction materials. For this reason, the use of laminated wood in structural members for various forms of construction (bridges, buildings, etc.) is progressively increasing.

Laminated timber products aim to reduce the effect of knots and defects on the overall strength characteristics, they achieve a smaller dispersion in its material properties along the length and section of the structural members and allow for an economical mass production of structural components. There are various types of engineered wood products, such as: laminated strand lumber, parallel strand lumber, laminated veneer lumber and glued laminated timber. Glued

laminated timber, or glulam, is the most versatile and widely used structural laminated wood material and has been employed for the construction of various buildings (libraries, warehouses, schools, etc). Glulam, is made of several timber layers (laminas) glued together with the wood grain parallel to the length of the beam, as shown in Figure 2.2. The thickness of the laminas can vary from 25 to 50mm [14]. Glulam offers many advantages over solid timber such as more uniform distribution of mechanical properties and increased design flexibility. Besides, its natural appearance also makes glulam timber structures more attractive and aesthetically pleasing [15].



Figure 2.2 – Glulam, glued laminated timber beam [14].

An added advantage of glued laminated timber is the possibility of manufacturing elements of larger lengths than those possible with solid wood. This effectively extends the range of applications for timber as a structural material and allows the well-established environmental credentials of wood to be incorporated in an increasing number of versatile structures. A case study based on a roof structure of a new airport in Oslo, Norway [33] compared the Greenhouse gas (GHG) emissions, life-cycle inventory and cost-efficiency of a glulam structure and its steel alternative [33]. The steel option was found to require two to three times more energy and six to twelve times more fossil fuel consumption during its construction than the design incorporating glulam members. Furthermore, the manufacturing of glued laminated timber beams produced approximately five times less GHG emissions than their steel beams counterparts. Also, at the end of their life cycle, timber beams can be burned to substitute fossil fuels, resulting in a substantial reduction of the overall GHG emissions making timber a great source of renewable energy source.

Besides the numerous advantages of using glulam timber structural elements, the laminations in them can also exacerbate the directionality effects and make more notorious the already natural weakness of wood when loaded not along the grain. This fact is recognized in Eurocode

5 [29] through the higher reductions in strength applicable to glulam relative to solid wood for loads applied at an angle to the direction of the fibre.

2.3 Timber Connections

The importance of connections in framed structures cannot be over-emphasised, since it is the part of the assembly most susceptible to failure. Figure 2.3 below shows a schematic of a typical beam-to-column connection. This assembly can be categorised into three groups: beam, column and the connection components.

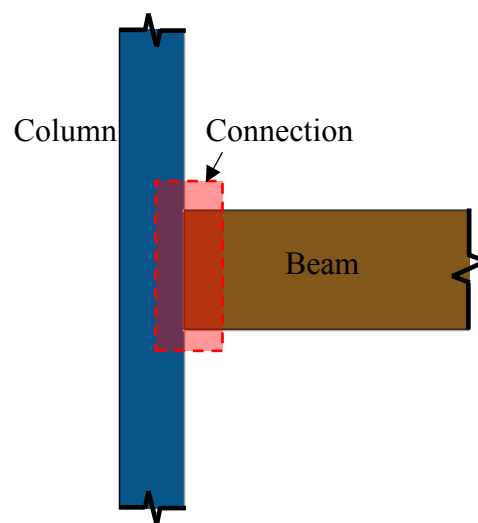


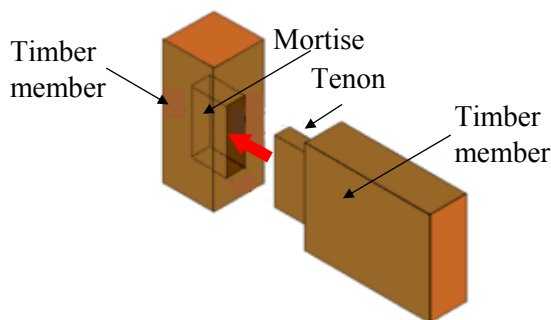
Figure 2.3– Schematic typical beam-to-column connection.

It is not possible to make reliable predictions about the behaviour of assemblies and structures if the behaviour of the connections employed is not well understood [1,2]. Additionally, when designing building frames, it must be ensured that appropriate design strategies were used that allowed stable and reliable behaviour and reliable failure mechanism. A good example of a design strategy is that structural engineers design connections that exhibit ductile response so that the structure can provide early warnings of unexpected failure.

According to current design guidance for steel [34] connections can be classified as rigid, semi-rigid or nominally pinned depending on the ratio between their rotational stiffness and the flexural stiffness of the connected beam. Likewise, depending on the connection strength, in relation to the strength of the beam being connected, connections can be categorized as full-strength, partial-strength or nominally pinned. The use of semi-rigid and/or partial-strength

connections in framed structures requires a careful consideration of the moment-rotation behaviour of the joint and a suitable estimation of its stiffness and capacity.

The two timber-to-timber connections most widely used are: a) mortise and Tenon joints and b) joints with the fasteners. Mortise and Tenon joints is a type of connection that does not use any kind of fastener but instead the timber members are interlocked by shaping them as shown in Figure 2.4a. Presently, such connection method undergoes a resurgence in popularity mainly for upscale residential or commercial construction like the one shown in Figure 2.4b. However, this connection is inefficient, costly and difficult to use in multi-storey buildings for two main reasons. Firstly, timber joinery requires a significant amount of wood fibre to be removed in the area where the joint occurs and thus timber members must larger in size to what it would be actually required. Secondly, design codes do not provide significant specific design guides for this type of connection due to sensitivity to quality of workmanship and material quality. Hence, the design of Mortise and Tenon joints is by and large very conservative.



a) Typical mortise and tenon joint [35]



b) Timber frame [36]

Figure 2.4 – Timber joinery.

The second connection type incorporating metal fasteners represents an efficient way to transfer loads between members. For timber members the most common fasteners are: nails, screws, dowels and bolts. Nails are most commonly used in timber constructions whereas screws are used for steel-to-timber and panel-to-timber joints and they are both available in a range of diameters up to 8 mm. However, such connections are very labour intensive to produce and thus very expensive for buildings. On the other hand, dowels are circular rods with minimum diameters of 6 mm usually made of steel whereas bolts are dowel-type fasteners with heads and nuts. This thesis, focuses mostly on those two latter fasteners, since they are more suitable for integrating timber and steel parts.

Significant research on timber connections with metal fasteners has been carried out in the past. Bolted timber column-to-timber beam connections with slotted-in steel plates are commonly used by timber engineers for their simplicity and sophistication. Smith et al. [37] studied both experimentally and numerically slotted-in steel plates connection using spruce timber under tensile loads. Porteous and Kermani, [38], provide detailed guidance for the analytical calculations of forces acting on individual fasteners. Based on these analytical expressions, Pataklas [39] developed an algorithm that allows designer to quickly assess the strength of the connection under bending moment. However, such algorithm only accounts for bolt-row shear failure along the direction perpendicular to the grain. This is in contradiction with the work conducted by Lam et.al. [40] where the moment-resisting capabilities on bolted timber connections with slotted-in steel plates for Douglas-Fir specimens were evaluated experimentally. It was reported that at high bending stresses, the bolt-row connection failure occurred in the direction parallel to the grain.

All studies mentioned above were carried out on timber-to-timber connections. To date, there has been no research on hybrid bolted steel column-to-timber beam connections, especially when tubular columns are employed. Such connection will provide the utilisation of timber material in high-storey framed buildings. Hence, there is need for experimental, analytical and numerical study on such connections. Although there exists information on the bolt-row connection failure occurred in the direction parallel to the grain for Douglas-Fir, it will be beneficial to study more timber species. Using glulam beams in conjunction with steel columns will avoid the potentially limiting stresses generated perpendicular to the fibres in wooden columns. Besides, smaller column dimensions could be expected by taking advantage of the high strength-to-weight ratio and torsional stiffness of steel tubes. Therefore, the provision of practical solutions for connecting tubular steel columns and glulam beams presents a number of potential benefits that should be analysed.

2.4 Hybrid Steel-Timber Connections

One of the most basic connection types for steel beam-to-column connection is the top and seat angle connections which have been studied extensively in steel [3,4,5]. However, this connection type has not been usually applied to timber beams due to the lack of understanding on the behaviour of the long fasteners (perpendicular to the beam direction) and their

interaction with the timber material. A proposal made as part of the present thesis is shown in Figure 2.5a. In addition, a hybrid slotted-in T-stub connection with bolts, welded to a hollow structural sections (HSS) and illustrated in Figure 2.5b will also be studied. These connections can unlock the benefits of using hybrid timber-steel joints and will be the main focus in this project.

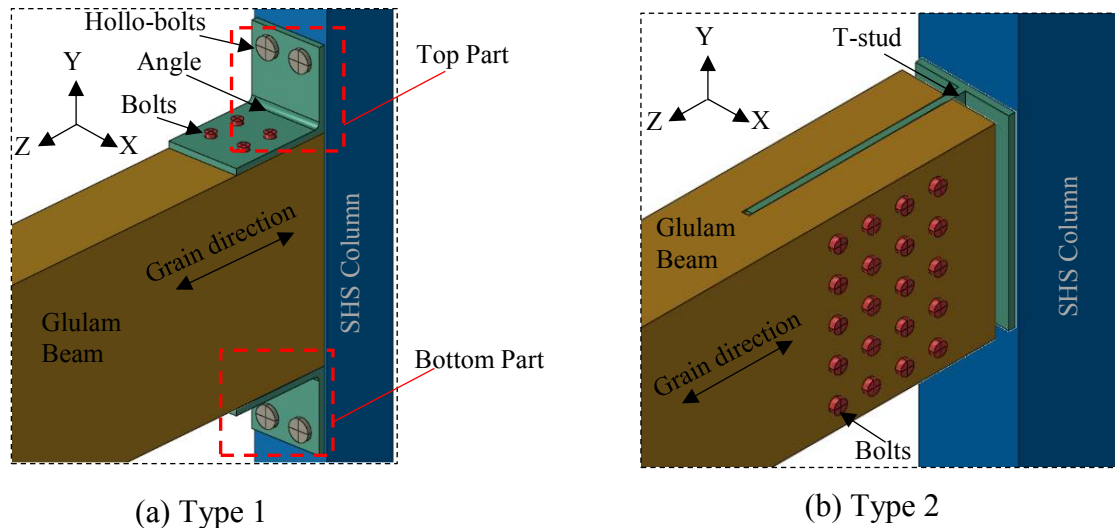


Figure 2.5 – Isometric view of connection details.

When it comes to hybrid timber-steel connections, hollow structural sections (HSS) offer a more practical solution than open-section columns. The use of hollow structural sections (HSS) has experienced significant growth over the last two decades [41]. Research shows that the flexibility offered by HSS opens up new design possibilities for a wide variety of structures including framed buildings. Compared to open-section members (I-beams, tee-beam, etc.), hollow structural sections provide several beneficial features [41], such as structural efficiency and architectural appeal.

Due to the tightening difficulties, proper of HSS sections, bolting has usually been neglected. However, recently a number of blind-bolting systems, that allow single-side tightening, have been developed and represent a viable alternative, [42]. Several publications on blind-bolted moment steel connections can be found in the literature [3,4,5,43]. One of the most versatile blind-bolting systems was introduced by Lindapter Inc through the development of the Hollo-bolt [44]. The working of such a bolt is relatively simple, as depicted in Figure 2.6. It can be appreciated from this figure that as the head is tightened, the cone moves closer towards it

causing the leg of the sleeve to expand until the cone locks the sleeve against the inner surface of the plate. This bolt type will be used to connect the steel angles to the tubular column in the Type 1 connections (Figure 2.5 (a) studied herein.

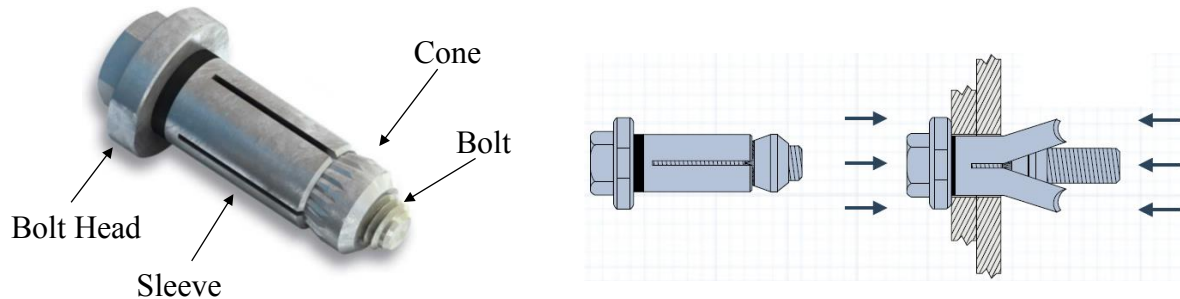


Figure 2.6 – Lindapter Hollo-Bolt [44].

Various researchers have investigated the behaviour of Hollo-bolts for open steel beam to steel column connections, including its application in partial-strength moment-resisting connections. Mourad et al. [45] studied experimentally the behaviour of Hollo-bolted extended end-plate connections under cyclic loading and used frame analysis in order to explore the effect of joint flexibility on the overall frame response when subjected to dynamic loading. Wang et al. [46], performed a numerical comparison of the behaviour of a blind-bolted T-stub connection utilising the Hollo-bolt as well as standard bolts as fasteners. It was found that Hollo-bolted connections are more flexible than standard bolted connections. This phenomenon can be attributed to the ductile deformation of the sleeve. It was also found that as the thickness of the flanges increases, the contribution of the Hollo-bolt to the plastic deformation of the connection becomes more significant. Elghazouli et al. [4], performed a series of experiments in order to investigate the behaviour of Hollo-bolted angle connections under monotonic and cyclic loading. The experimental programme comprised seventeen monotonic and cyclic tests and the influence of several material properties and geometric parameters was assessed. These experiments showed that the grade of the Hollo-bolt, combined with the gauge distance between the Hollo-bolt and the flange of the beam, strongly influence the performance of the connection. Furthermore, the hysteretic response of the connections was stable and their corresponding energy dissipation capacities were adequate. Also, the ductility of the connection was found to be satisfactory. Due to the extensive research already performed on blind-bolted connections, the blind-bolt behaviour is not subject of study in this thesis,

however, the insight gained in previous investigation has been incorporated in the design and numerical modelling of these when required.

To successfully design a beam-column connection, it is imperative to understand how each component interacts with other as well as how it behaves under various loading conditions. To this end, an introduction on the components comprising typical connections and their failure modes are briefly discussed in the following sub-sections.

2.4.1 Steel Column face

The failure modes that can occur in the tension region of Holo-bolted connections are either yielding of the column face or punching failure of the bolt holes as illustrated in Figure 2.7. Various publications have studied those phenomena in detail [3,4,43,47, 48, 49]. On the other hand, the compression forces that are acting on the bottom part of the connection can cause local buckling (dent) on the column face as shown in Figure 2.8 below. Liu et al. [43] estimated the capacity of the column/angle assemblage under compression and offered semi-analytical expressions for their design and calculation.

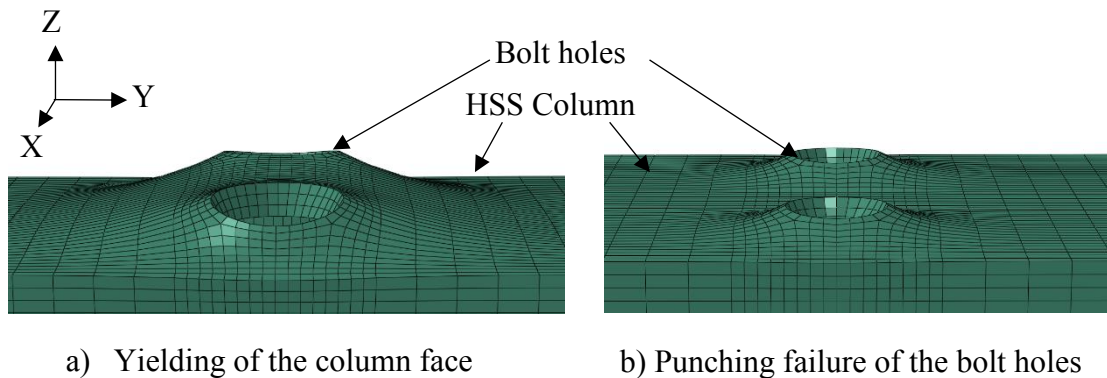


Figure 2.7 – Schematic of failure modes on column face at the top part for Type 1 connection.

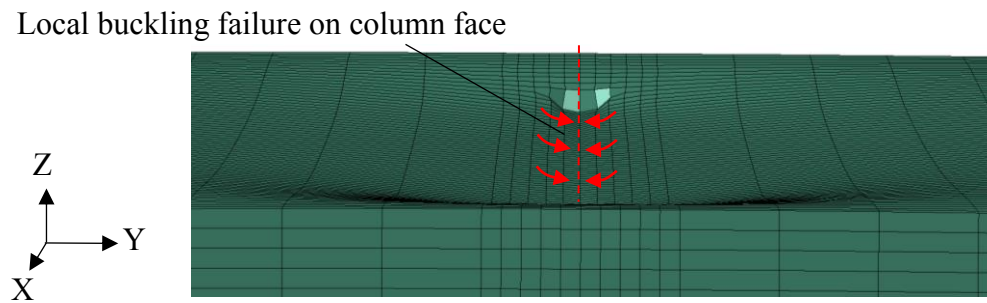


Figure 2.8 – Schematic of Failure mode on column face at the bottom part for Type 1 connection.

Finally, the SHS face on Type 2 connection can undergo local buckling failure along its length due to the simultaneous application of compressive and tensile stresses associated with bending along the fin-plate axes, as shown in Figure 2.9. Significant guidance for the calculation of the column face capacity and stiffness associated with this failure mode is offered by [3,43,49].

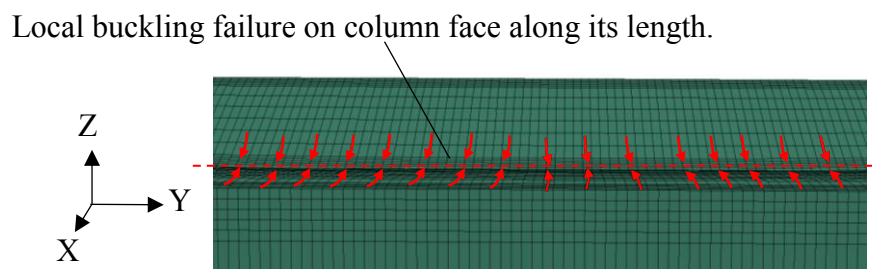


Figure 2.9 – Schematic of Failure mode on column face for Type 2 connection.

2.4.2 Timber Beams

Timber beams are designed to resist flexure, shear and lateral torsions while supporting the floor elements. Figure 2.10 illustrates the failure that can occur on a timber beam while the expressions to determine the limit cases for the above mentioned failure modes, can be obtained from any textbook on mechanics such as [50].

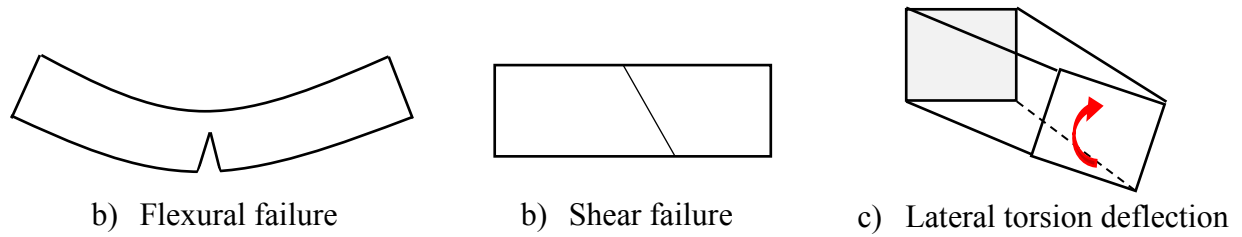


Figure 2.10 – Schematic Failure modes on timber beam.

2.4.3 Connection Region

As previously mentioned there are many ways to fastened a beam into a column. One of the most versatile ways to achieve that is with the use of metal brackets such as angles and T-stubs. Eurocode 3 Part 1-8 Section 6.2.6.6 [34], suggests that the tension design resistance and the failure mode of such metal brackets can be associated with those of an equivalent T-stub flange. In general T-stub flanges and therefore angles, can be subjected to three distinct failure modes under tensile forces: a) complete yielding of the flange, b) bolt failure with yielding of the flange and c) bolt failure, as shown in Figure 2.11 below. Bolted angle connections have been extensively studied experimentally, analytically and numerically due to their practical and economical merits [51-55].

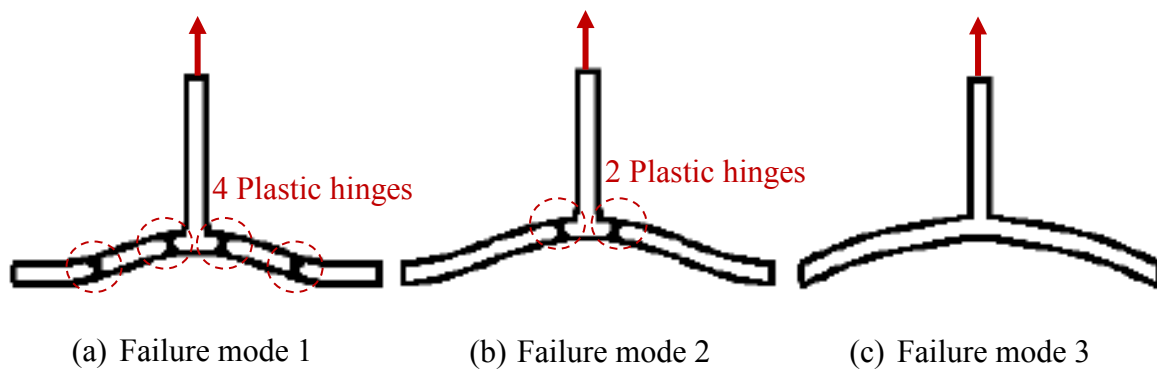


Figure 2.11 – Failure modes of equivalent T-stub under tension

In every bolted beam-column timber connection, the conventional fasteners used must be able to endure the design tension and shear forces as well as to be appropriately placed to avoid block or shear tearing of the steel plate as illustrated in Figure 2.12. The formulas to obtain the tension $F_{t,Rd}$ and shear $F_{v,Rd}$ resistance of the bolts are given in Eurocode 3 part: 1-8 [34].

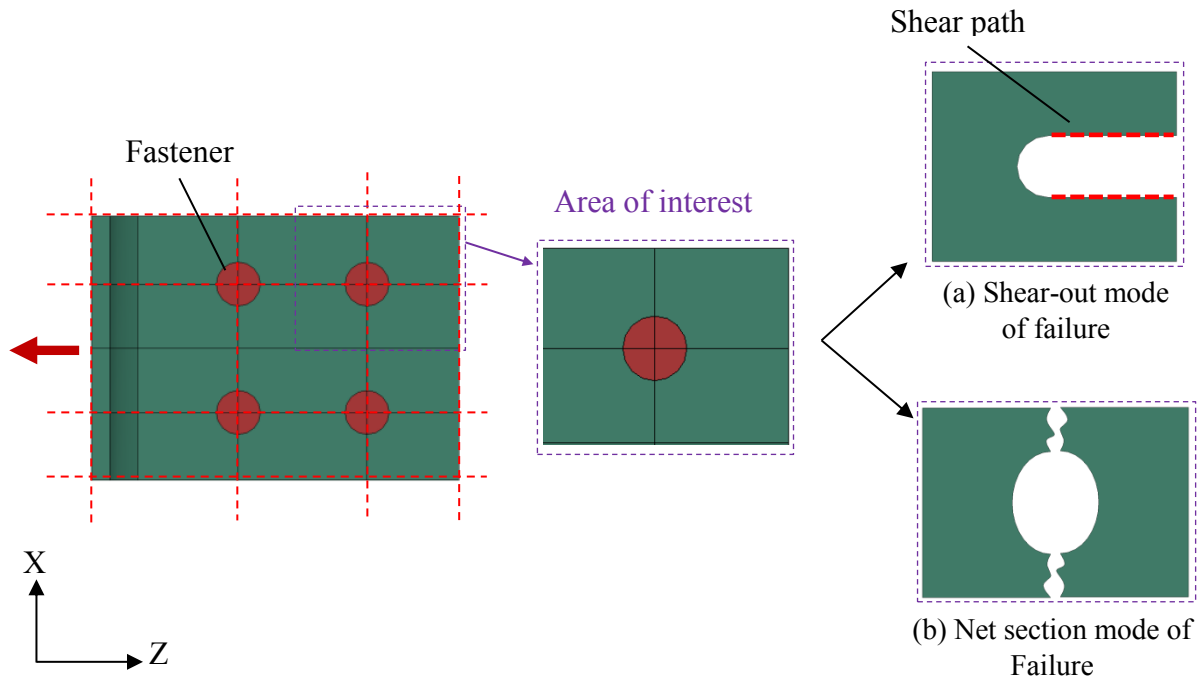


Figure 2.12 – Failure modes of angle flange

The interaction between the steel fastener and the timber components is determinant for the behaviour of fastener-type timber connections. According to Eurocode 5 [29] there are eleven different failure modes, when it comes to timber-steel connections. The expressions for capacity estimations that the code associates with each of the individual failure mechanisms presented in Figure 2.13 (Section 8.2.3 of Eurocode 5[29]) assume a rigid-plastic behaviour of fastener and timber and can be obtained by establishing equilibrium. This approach was first proposed by Johansen in 1949 [56] and is often referred to as the European Yield Model (EYM).

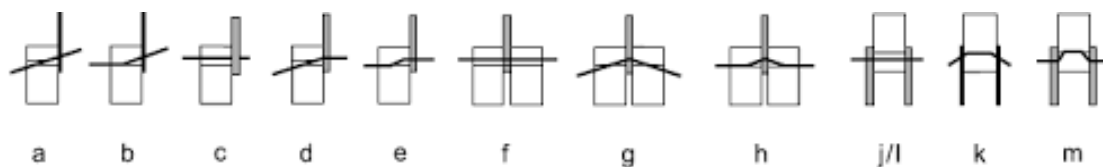


Figure 2.13 – Failure modes of timber-steel connections [29].

The EYM estimations, where applicable, agree well with experiments and the data of test results on timber joints with dowels and bolts as fasteners is extensive (e.g. 57-60). Although, the force-displacement response from such tests can be evaluated to successfully design and

assess timber joints. Therefore, *single-dowel* connections as shown in Figure 2.14, constitute the fundamental building-block for designing timber joints and hence will be the focus of Chapters 4 and 5.

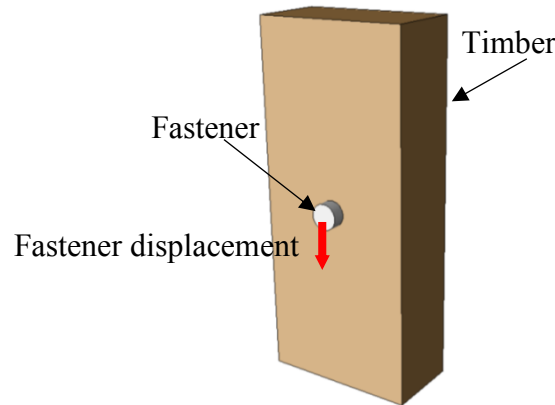


Figure 2.14 – Example of single-dowel timber connection test.

It should be noted that the EYM only deals with the most well-known ductile failure modes of timber connections with steel fasteners subjected to pure tension or compression and no modelling consideration are available for complex load combinations or brittle failures such as splitting, shear or plug-shear fractures. Besides, although the corresponding formulae for each individual failure mode are well established, there is a dearth of information on the implementation of these expressions and analysis on hybrid beam - column connections of the type studied in this thesis. Moreover, the application of the EYM to cases of concurrent flexion and compression as those developed in Type 1 connections (Figure 2.5a) is expected to require modifications that will be introduced in Chapter 7.

2.5 Numerical modelling of timber connections

The complex natural composition of wood makes the modelling of timber components difficult and computationally expensive. Wood can be modelled as cellular material [61, 62] but that is usually restricted to small samples and not fully applicable within a structural engineering context. In order to render timber connection models economical and practical for structural applications, it is very common to treat timber as a solid orthotropic material by defining three main directions [63]. Additionally, the cellulose microstructure of timber generates difficulties

when dealing with the behaviour of timber joints due to the intricate and highly non-linear interactions arising at the interface between fasteners and wood.

Single-dowel connections can undergo four failure modes [64]: a) bearing, b) splitting along the direction of the load application, c) shearing and d) fastener deformation. The most common failure mode for single-dowel connection tests in failure by bearing (Shown in figure 2.15), since in the majority of the cases bearing failure is always generated prior to failures b, c, and d. A few analytical studies have been carried out to predict such failure. Collins [65] made an attempt by using Yamada's strength criterion to predict failure strength and bearing failure modes in composite materials. Xiao et. al [66], developed an analytical tool to predict the bearing failure and response of bolted composite joints. Generally speaking, it is very difficult to undertake an analytical approach on the failure mechanisms of timber joints due to the interactions between the various parameters. For that matter simplified approaches and many assumptions are made to conduct analytical calculations. Therefore, numerical models are usually employed to study timber joints.

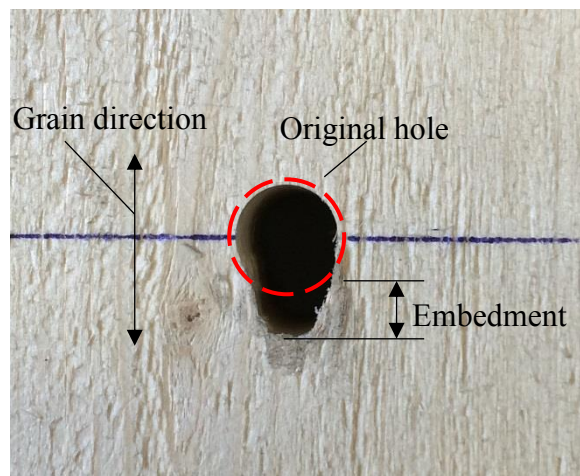


Figure 2.15 – Bearing failure of single-dowel connection tests parallel to the direction of the grain.

Previous research [67-69] has demonstrated that when modelling dowel/bolted timber connection as solid materials, conventional FE strategies that assume standard material constitutive properties cannot accurately represent the full connection response. Hence, more advanced methods are required to deal with the localized nonlinear behaviour of timber such as that arising from embedment effects. Patton-Mallory et al. [24,25], implemented an orthotropic constitutive law for timber (Douglas-fir) in the commercial software ABAQUS [7]

incorporating a tri-linear stress-strain compression response parallel to grain. The proposed model was able to simulate the experimental results but with a marked tendency towards a numerically stiffer response. Importantly, only compression parallel-to-grain was considered. Moses and Prion [26], studied the three-dimensional non-linear behaviour of a single-dowel timber connection test by means of a material model based on orthotropic elasticity and anisotropic plasticity together with the Weibull Weakest Link Theory to account for brittle failure. A good level of agreement between the numerically predicted force-displacement relationships and their corresponding experimental results was achieved. However, the estimation of ultimate loads was found to be over-conservative. Kharouf et al. [27] modelled bolted timber joints by assuming an elastic-plastic orthotropic Hill criterion for wood in compression and a linear-elastic orthotropic criterion for wood in tension. The model was able to reproduce the experimentally observed failure modes and global deformation patterns in one-bolt joints but with a numerical stiffness significantly larger than the experimental values.

In general, the computational implementation of advanced failure criteria, such those reported in the above-cited studies, is highly complex and requires knowledge of advanced programming. Moreover, the difficulties associated with the definition of interactive failure modes and localized non-linear deformations further complicate their adoption. To this end, Foschi [28] proposed an approximate numerical two-dimensional model that takes into account the crushing behaviour of wood around the dowel by incorporating an equivalent circular area centred around the dowel with weakened material properties known as the ‘foundation model’. More recently, Hong [70] extended the ‘foundation’ approach to three-dimensional continuum FE models representing bolted connections in Douglas-fir wood. Nevertheless, the dependence of the equivalent material properties on the volume assumed for the foundation zone was not investigated. However, the foundation zone methods proposed by Foschi [28] and Hong [70] limited the foundation zone to have a radius of 2.8 times the dowel diameter. This fixed value of foundation radius restricted the size of specimens. Additionally, for multi-dowel timber-steel connection this restriction put a limit on the spacing between the fasteners especially for small connections. Therefore, there is a need to improve their method by eliminating the foundation zone radius restriction.

2.6 Connection reinforcement

When high levels of embedment are produced into timber-steel connections where the direction of the load is applied parallel to the direction of the grain, splitting of the specimen throughout its whole height is occurred [71]. This phenomenon is caused due to the generation of tension singularities stresses which overcome the strength of the fibres laying as tangents to the dowel hole. To prevent, thus type of failure Lathuillière et al [72] and Bejtka et al. [73] studied the response of reinforced dowel connection. Fasteners such as screws or nails were inserted through the timber specimen near the connection, at the side where high strengths are generated as shown in Figure 2.16 below. This reinforcement technique overcomes the weaknesses of timber such as reducing or even preventing crack propagations at high loads and switching the behaviour of timber from brittle to ductile. Additionally, this type of reinforcement increases the shear and tensile strength of the timber when load perpendicular to the direction of the grain. This thesis will explore the benefits of screw reinforcement with particular attention to Type 2 (Figure 2.15) connection. Besides, since the mobilization the full connection strength depends on the location of the point of rotation, this thesis will also explore alternative ways of making a reliable modification to the joint point of rotation by means of wedged angles.

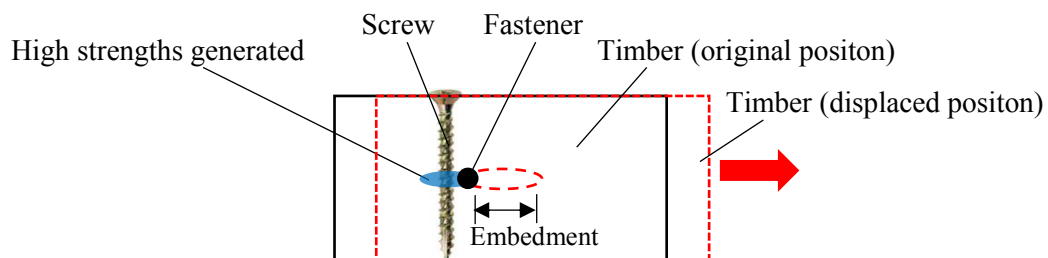


Figure 2.16 – Reinforced single-dowel connection.

2.7 Concluding remarks

The review conducted in this chapter has shown that there is a dearth of information on the response of hybrid connection involving timber glulam beams and steel tubular columns. Such connections have not been previously studied and practical connecting alternatives have not been proposed or verified. As mentioned earlier, this thesis proposes and studies two new hybrid timber beam-to-tubular steel moment connections, namely: (i) top and seat bolted angle

connections with long bolts, and (ii) slotted-in T-stub connections with bolts. The specific characteristics and configurations of these joints will be presented and studied in subsequent chapters of this thesis.

It is customary in every experimental campaign that a comprehensive material testing is performed to characterise the particular timber used in the connections even when information on the nominal material properties for various timber species can be found in the literature. To this end, an experimental program for material testing based on the British Standard EN 408[74] is essential to determine the material properties for the timber used and create the material model for the FE models. Such experimental campaign is described in Chapters 3 and 4.

Experimental testing becomes even more valuable for examining the embedment behaviour of timber since the cellulose microstructure of timber generates difficulties when dealing with the behaviour of timber joints due to the intricate and highly non-linear interactions arising at the interface between fasteners and wood, as explained in this chapter. A well understood embedment behaviour is vital for the correct assessment of the two hybrid bolted connections, mentioned earlier. It springs from the literature survey conducted that there is a dearth of information on the embedment behaviour for Spruce wood of strength class C30. Commonly, an experimental test to characterise the embedment behaviour involves load application on both grain directions with at least two dowel diameters (usually 12 and 16 mm) [63,75,76,77]. This will be tackled in Chapter 4.

On the other hand, the real challenge appears when modelling the embedment behaviour of timber, where conventional FE strategies that assume standard material constitutive properties cannot accurately represent the full connection response. It is clear from the literature review performed that to avoid the generation of highly complex material subroutines that require knowledge of advanced programming, simple, yet accurate ways of incorporating damage modelling are deeply needed. The literature review has also identified the foundation zone approach as viable alternative. However, such models have severe restrictions on the area of the foundation zone that limits their applicability. These issues are further discussed in Chapter 5 where new modelling solutions are offered.

It was also observed in the review that a growing body of knowledge exists on the response of blind-bolted connections to tubular columns in all steel joints. The behaviour of the blind-bolt

insert and HSS column face components is therefore well understood. On the other hand, while a wealth of information is available on the response of dowel-type connections in timber, past studies have focused mainly on pure tensile or compression loads and there is a need to investigate the mechanical response of hybrid connections involving different configuration of fasteners and bolt groups under bending loads, especially in relation to the identification and prevention of brittle failure modes. The research work conducted in this thesis aims to address those needs. In particular, Chapter 6 studies the experimental response of the two connection configurations being proposed while Chapter 7 presents the proposal and validation of numerical and component-based model. The effects of various parameters on the response of the hybrid timber beam-to-tubular column connections are explored in Chapter 8. Finally, Chapter 9 concludes the thesis by stating the main findings together with suggestions for future work.

Chapter 3

Material Characterisation Tests

3.1 Introduction

An experimental characterization of mechanical properties is especially important for a natural and anisotropic material such as wood due to the complicated relationships between stresses and strains along different loading directions and the inherent variability in its response. This chapter presents an extensive experimental study conducted in order to determine the mechanical characteristics of the laminated wood used along this thesis. The tests described in this chapter have been carried out on wood samples of the same material employed in subsequent experimental parts of this thesis (Chapters 4 and 6). Besides, the results of this chapter also inform the numerical models developed in Chapters 5 and 7.

Tests on Scandinavian Spruce glulam specimens subjected to shear, compression and tension loads (parallel and perpendicular to grain) are described and discussed in this chapter. In addition, four-point bending tests on large glulam beams performed to determine the material's bending stress and shear modulus are also presented. All tests were carried out in accordance with European standards EN 408 [74]. Mean values of compressive strength in each of the principal directions as well as their corresponding shear resistance and Young's Moduli were obtained and are presented below. The main behavioural patterns and key response characteristics of Spruce Glulam are discussed on the basis of the results presented.

3.2 Experimental set-up and specimen details

All experiments were conducted at the Structures Laboratory of the Department of Civil and Environmental Engineering of Imperial College London. The wood species used was Scandinavian Spruce (*Picea Abies*), with a mean density, ρ , of approximately 430 kg/m³ and a moisture content of approximately 10 %. The specimens were manufactured from 7.5 m long beams sourced from Glulam Ltd. UK [14]. All specimens were measured, weighted and then stored in a dry environment with mean a temperature of 20 °C and relative humidity of 60 %.

3.2.1 Compression tests

The compressive resistance of the orthotropic timber specimens was examined in both the longitudinal and radial directions with respect to the grain. A total of eight wood cubes of $45 \text{ mm} \times 45 \text{ mm} \times 45 \text{ mm}$ were tested under compression parallel to the grain, whereas nine timber specimens of dimensions $45 \text{ mm} \times 70 \text{ mm} \times 95 \text{ mm}$ were compressed perpendicular to the grain. Figures 3.1 and 3.2, display the dimensions of the tested pieces and the testing apparatus, for compression tests parallel and perpendicular to the grain directions, respectively.

In both test series, the timber specimens were placed in between the steel plates attached to the testing machine. A monotonically increasing load was applied to the specimens at a rate of 1 mm/min . The applied load was measured with an accuracy of $\pm 0.1 \text{ kN}$ by means of the load cell incorporated within the testing apparatus. The tested specimens for each test series, were named as depicted in Figure 3.3.

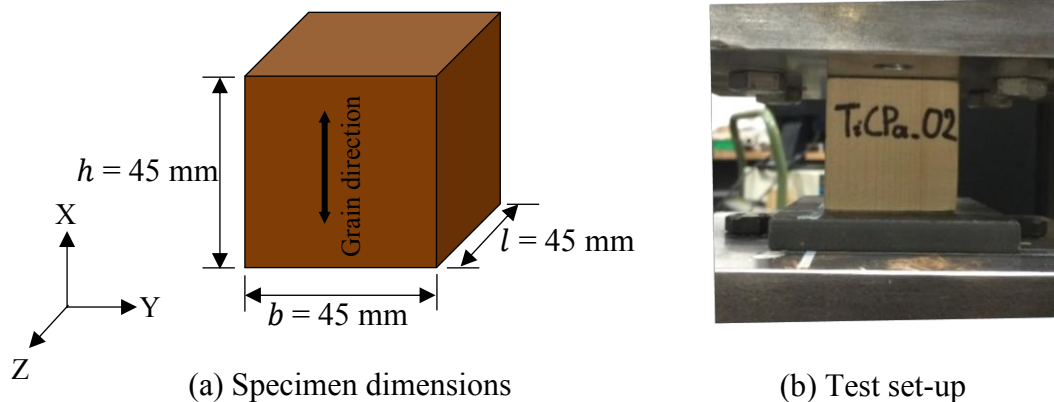


Figure 3.1 – Compression tests parallel to grain.

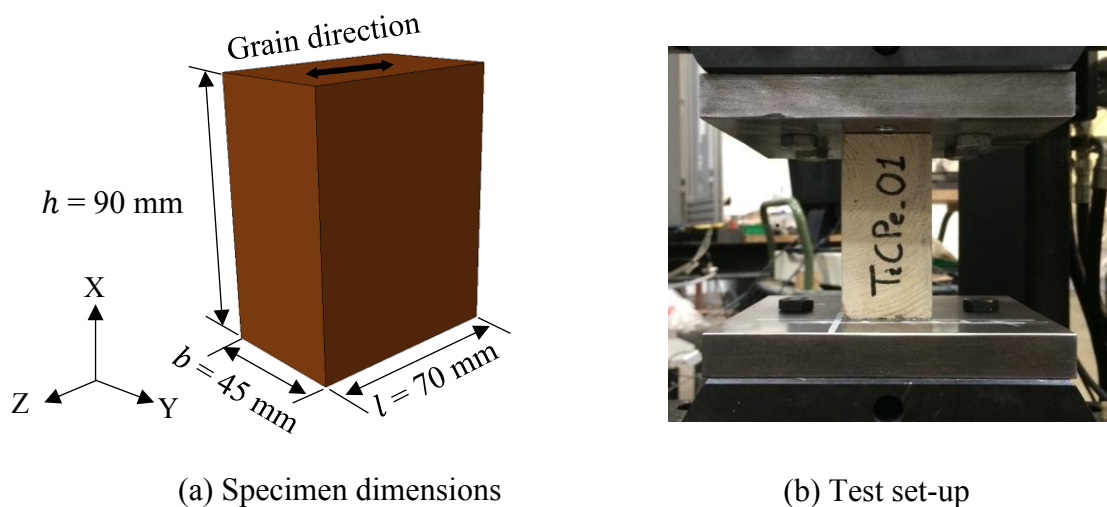


Figure 3.2 – Compression tests perpendicular to grain.

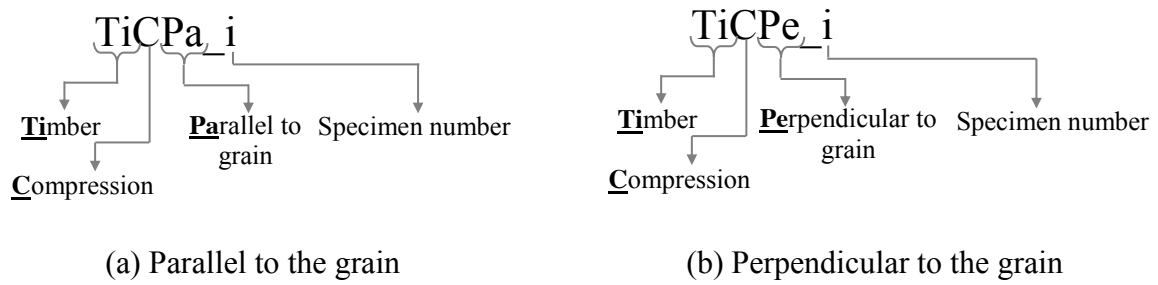


Figure 3.3 – Nomenclature of wood specimens tested in compression.

3.2.2 Tensile tests

Similar to the compression tests, the tensile resistance of timber specimens in both grain directions was also examined. Eight “dog-bone” shaped wood specimens were clamped to an INSTRON machine and tested under tension parallel to the grain direction. On the other hand, four timber specimens of dimensions $44 \text{ mm} \times 70 \text{ mm} \times 180 \text{ mm}$ were glued to the platens of the testing machine and then pulled in the direction perpendicular to the grain. The glue used was a high strength high peel, multi-purpose epoxy adhesive called Redux 810 [78]. Figures 3.4 and 3.5, display the dimensions of the test pieces and the testing apparatus employed for the tensile tests parallel and perpendicular to the grain direction, respectively.

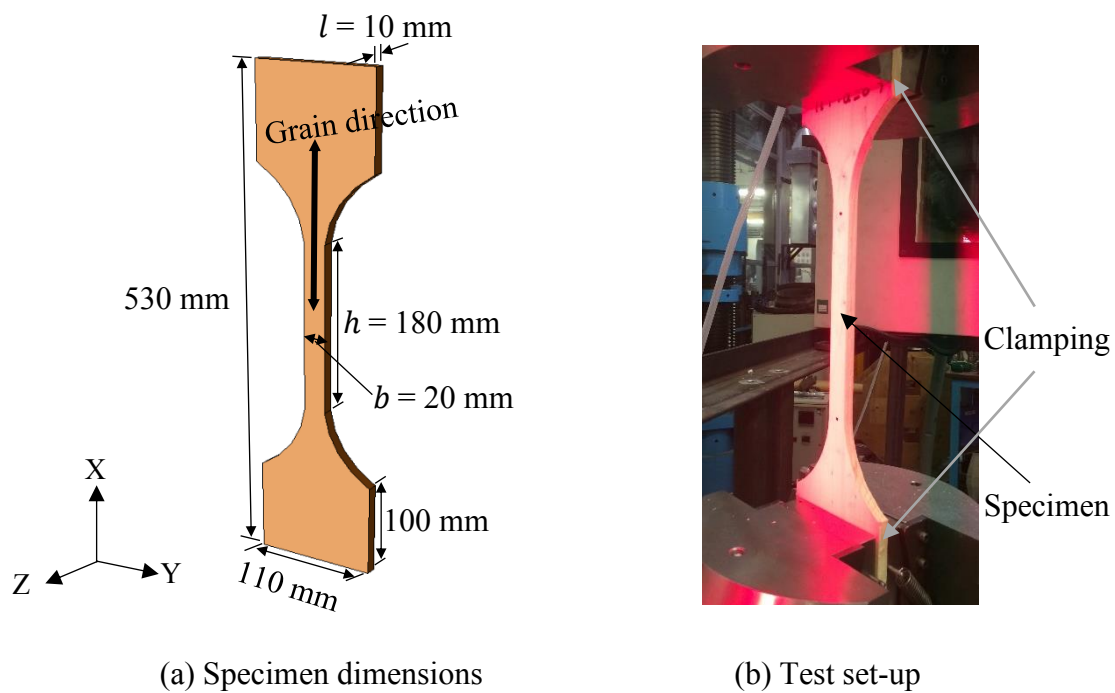


Figure 3.4 – Tensile tests parallel to grain.

In both test series, the applied load was monotonically increased at a rate of 1 mm/min and it was measured with an accuracy of ± 0.1 kN by means of the load cell incorporated within the testing apparatus. Additionally, a video-extensometer was positioned on one side of the specimens to measure the deformation in the direction of load application. Care was taken not to induce bending in the specimens during the application of the tensile load. The specimens were named as depicted in Figure 3.6.

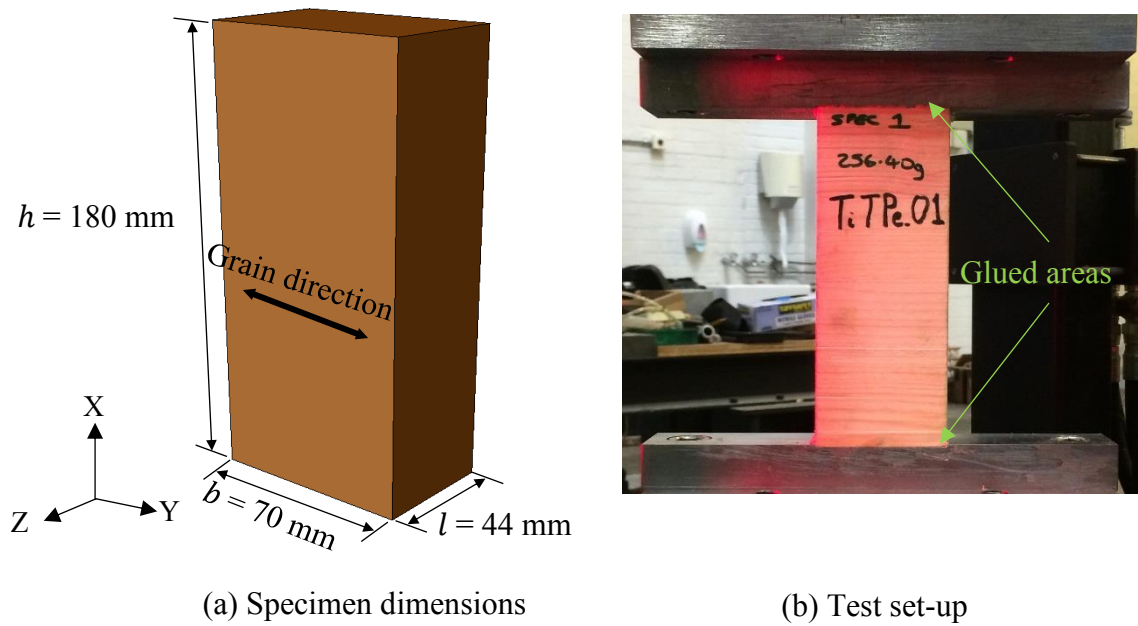


Figure 3.5 – Tensile tests perpendicular to grain.

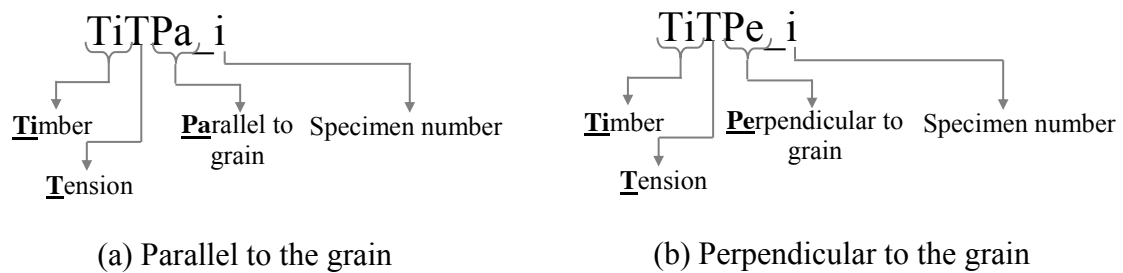


Figure 3.6 – Name of timber specimens under tension.

3.2.3 Shear tests

The characterization of the shear strength of the wood employed in this study followed the European Standard EN408 [74]. To this end, eight specimens of 55 mm $\times$ 32 mm $\times$ 300 mm were manufactured and subjected to shear forces as showed in Figure 3.7. Two 10 mm thick steel plates were glued to each side of the specimen by means of a high strength, high peel

multi-purpose epoxy adhesive called Redux 810 [78]. Since the strength of this glue is significantly higher than that of timber ($46.6 \text{ MPa} > 5.0 \text{ MPa}$), no failure of the glue-line was expected or observed. Based on the specimen dimensions provided by European Standard EN408 [74], all specimens were supported at an angle of 14° with respect to the vertical axis of the actuator. This was done so that the two opposite corners that are in full-contact with the metallic stands, will be vertically aligned as indicated in the norm. A monotonically increasing vertical load, was applied at a constant rate of 0.4 mm/min . Additionally, Figure 3.8 presents the name of the specimens for the shear tests.

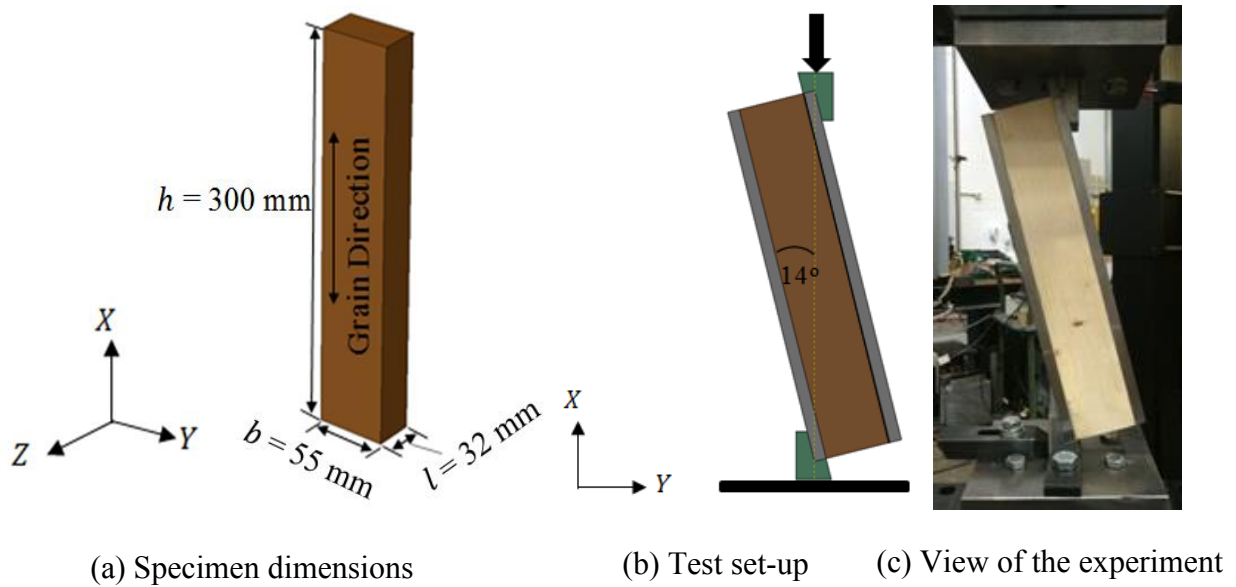


Figure 3.7 – Shear test.

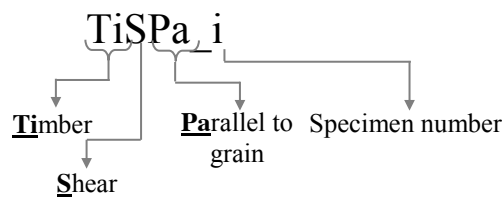


Figure 3.8 – Name of timber specimens under shear parallel to the grain.

3.2.4 Four-point bending tests

The characterization of the bending strength, σ_b , modulus of elasticity, E_1 , and shear modulus, G_{12} , of the wood employed in this study followed the European Standard EN408 [74]. To this

end, a series of four-point bending tests were conducted on glulam beams of length 7500 mm and cross-section of 140 mm $\times$ 405 mm. The strength class of the beams was GL28h, while each C30 strength class lamina had a height of 45 mm. A total of six simply supported beams were loaded at two points until failure. Figure 3.9 depicts the experimental set-up and dimensions. The load F was applied at a rate of 1.0 mm/min.

To prevent lateral buckling, restrains were placed near the supports of the beam, as depicted in the Figure 3.10. Polytetrafluoroethylene (PTFE) sheets were placed between the beam and the restraint to eliminate potential frictional resistance. Importantly, European Standard EN408 [74], specifies a beam length between the two supports such that no lateral torsion buckling can occur and thus there was no need to install lateral restrains between supports. Additionally, two small steel plates were placed between the two loading heads and the beam, to minimize local indentation as shown in Figure 3.11.

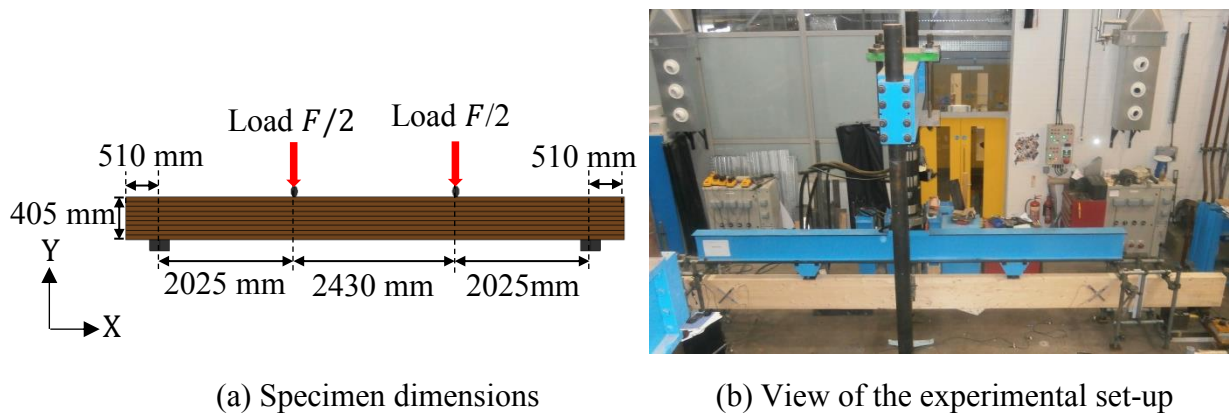


Figure 3.9 – Four-point bending test arrangement.

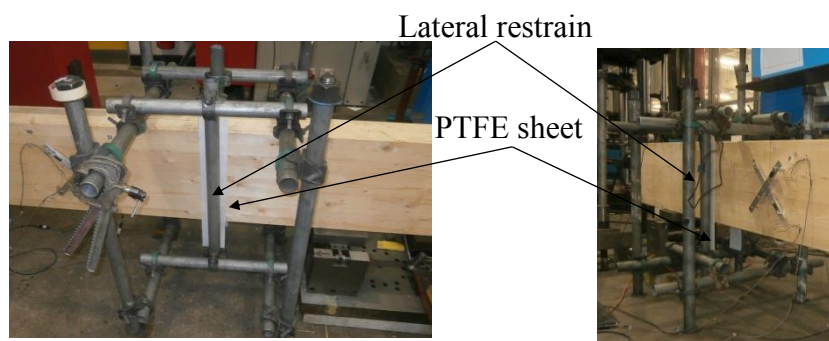


Figure 3.10 – Lateral restrains.

In total, twelve (12) transducers were used to measure the deformations at key locations. To this end, two transducers were employed to measure the central deflection of the beam, another two to measure the rotation of the edge sections of the beam and eight were utilised to monitor localised distortions necessary for the estimation of shear modulus as described later. The accuracy of each the transducer employed is ± 0.01 mm. Figure 3.12 shows the transducers used in the experiments to obtain the rotation and mid-span deflection of the glulam beam.

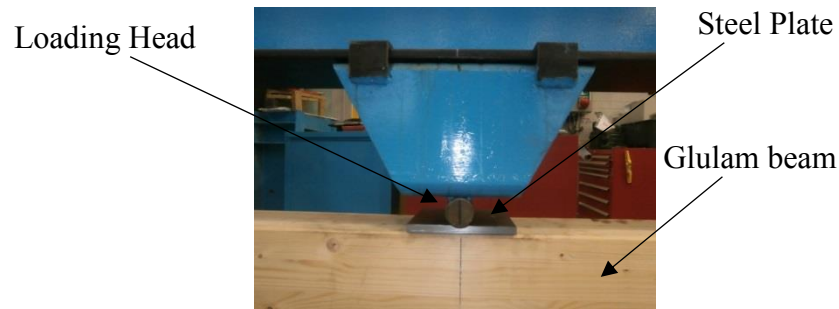
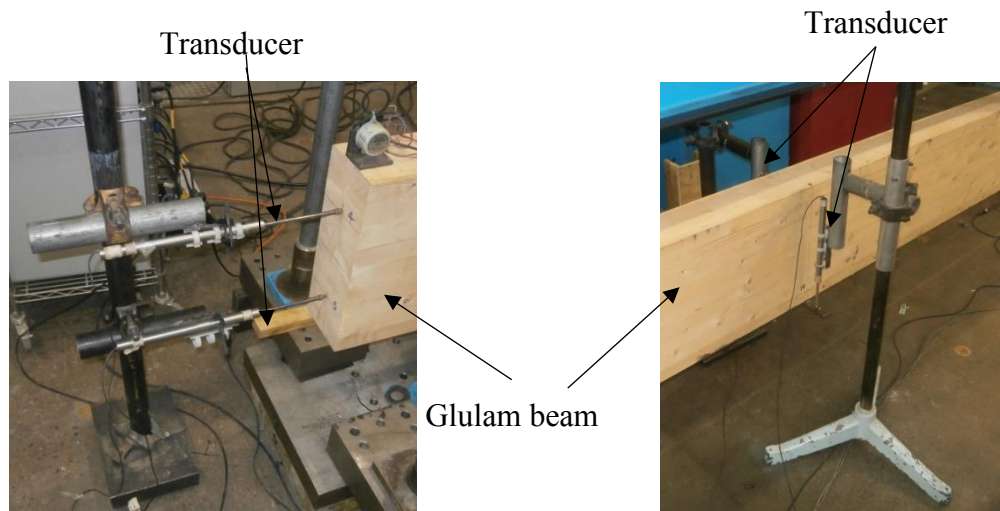


Figure 3.11 – Steel plates between loading heads and glulam beam.



(a) Transducers monitoring the angle of rotation at the beam's edge

(b) Transducers monitoring the mid-span deflection of the beam

Figure 3.12 – Transducers deployment.

3.2.4.1 Determination of Shear Modulus: Method A

The shear-field testing method recommended in the European Standard EN 408 [74] was employed herein in order to determine the shear modulus, G_{12} , of the glulam. Two implementations of this method were implemented. In the first instance, four “cross-shaped”

shear distortion transducers were placed in the areas of constant transverse force at both sides at 510 mm from the supports of the glulam beams, as illustrated in Figure 3.13. Each shear distortion transducer consisted of two rectangular shaped metallic plates mounted into the glulam beam, one on top of the other, in such way that they formed a rotated cross. This configuration is depicted in Figure 3.14 below. The length of each plate was 250 mm. Each plate has one end fixed to the timber beam, while the other end has a socket. Within each socket, a pin is placed to restrict the plates to move only along their length. Finally, a transducer was placed on each plate, which allowed the distortion to be measured (Figure 3.15).

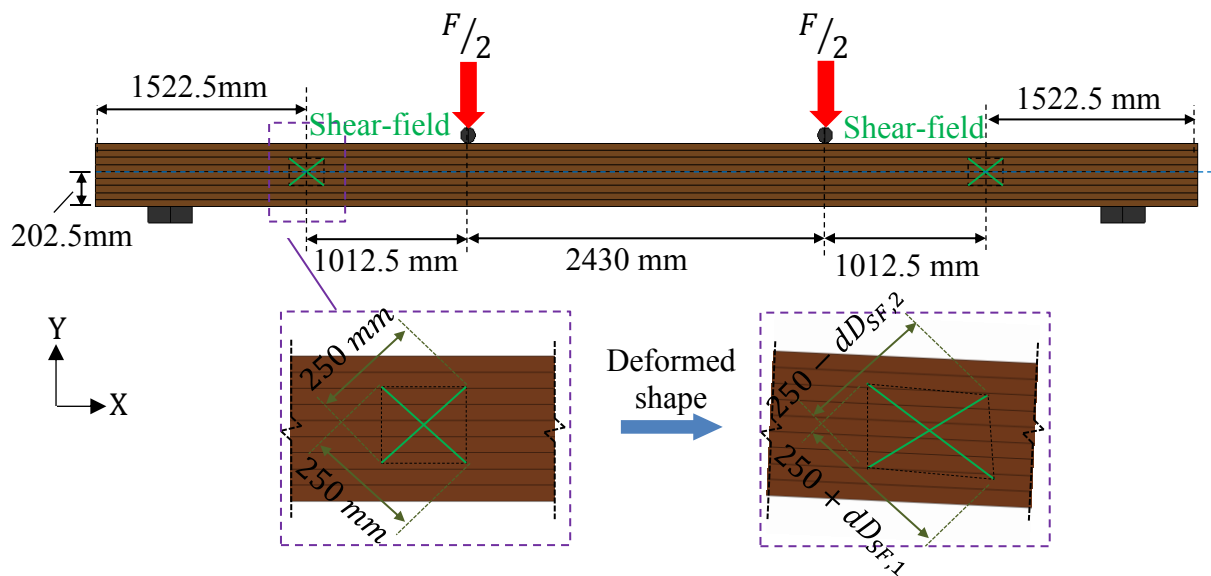


Figure 3.13 – Test arrangement of the shear field test and deformation of the diagonals.

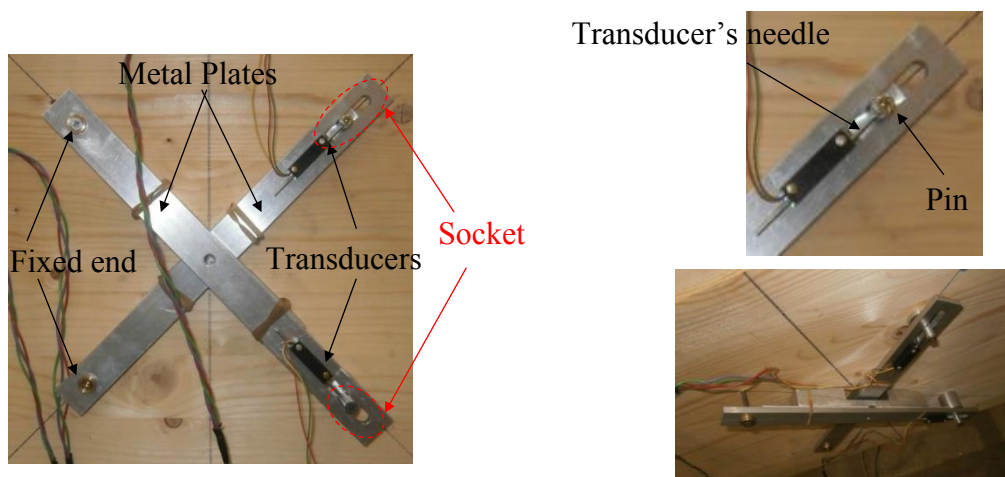


Figure 3.14 – Shear field, experimental configuration.

The shear distortion measuring device described previously configures a square from which the shear strain, γ_{SF} , of the shear-field can be determined as depicted in Figure 3.15. It is worth noting that the displacements of the plates along their corresponding lengths are very small and hence small-angle approximation was used. Additionally, it can be assumed that the length of the metallic plates remained constant throughout the experiment. Figure 3.16 below, shows the undeformed shape and a scaled-up deformed shape of the distorted square.

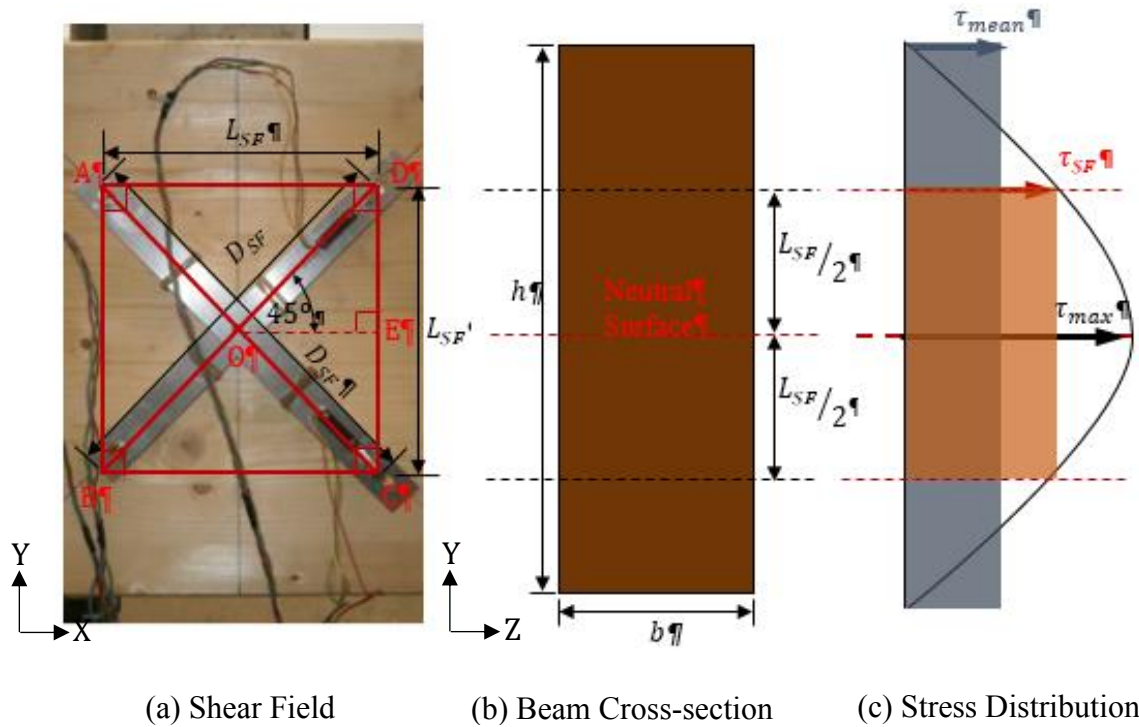


Figure 3.15 – Shear distortion measurement device.

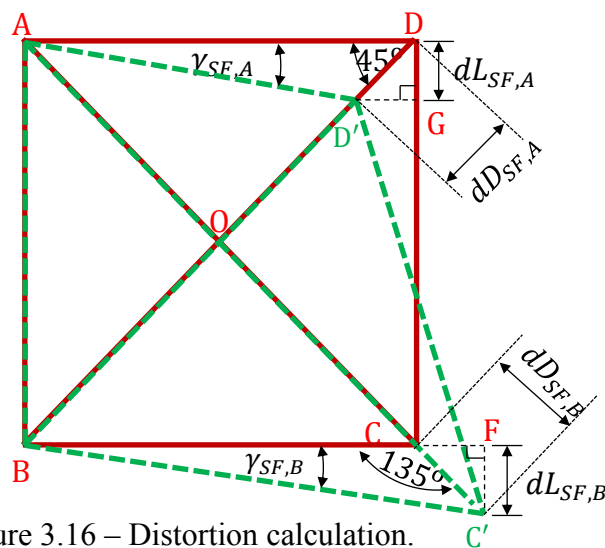


Figure 3.16 – Distortion calculation.

The shear modulus, G_{12} , between two loading intervals, where F_1 and F_2 are the corresponding forces, can be expressed as a function of the cross-shaped device deformations such that:

$$G_{12} = 1 + \frac{1}{3} \frac{bh(h^2 - L_{SF}^2)}{8 \times I} \times \frac{D_{SF}}{h \times b} \times \frac{F_2 - F_1}{(dD_{SF,A} + dD_{SF,B})_2 - (dD_{SF,A} + dD_{SF,B})_1} \quad (3.1)$$

Where, l is the span in bending, b is the width of the beam, h is the height of the beam, F_1 is 10% of the load F , F_2 is 40% of the load F and $(dD_{SF,A} + dD_{SF,B})_i$ is the mean deformation of both diagonals AC and BD on opposite sides of the beam at the given load F_i with $i = 1, 2$. The detailed derivation of the formula and method is presented in Appendix A.

3.2.4.2 Determination of Shear Modulus: Method B

The second implementation of the shear-field test to determine the shear modulus (G_{12}), uses the same principle described above but instead of a “cross-shaped” shear device, it uses digital image capture on forty-nine (49) dots 30 mm apart from each other forming an square grid. These dots were marked as depicted in Figure 3.17.

The square grid was constructed in such a way, that the diagonals AC and BD had equal lengths D_{SF} . A digital SONY camera, placed at a distance 1 m from the specimen, was used to capture images at 30 seconds intervals. The set-up is shown in Figure 3.18. The video analysis and modelling program Tracker [79] was used to obtain the displacements of the four points (A, B, C and D), describing the corners of the grid. The additional dots were used for verification purposes and error quantification.

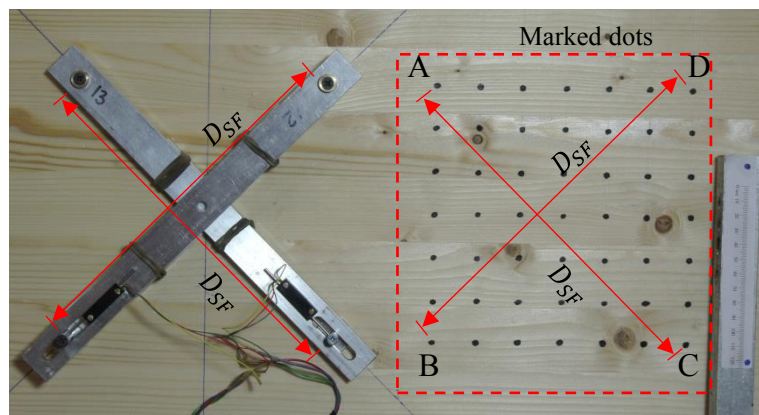


Figure 3.17 – Dots marked on the beam to obtain the Shear modulus.

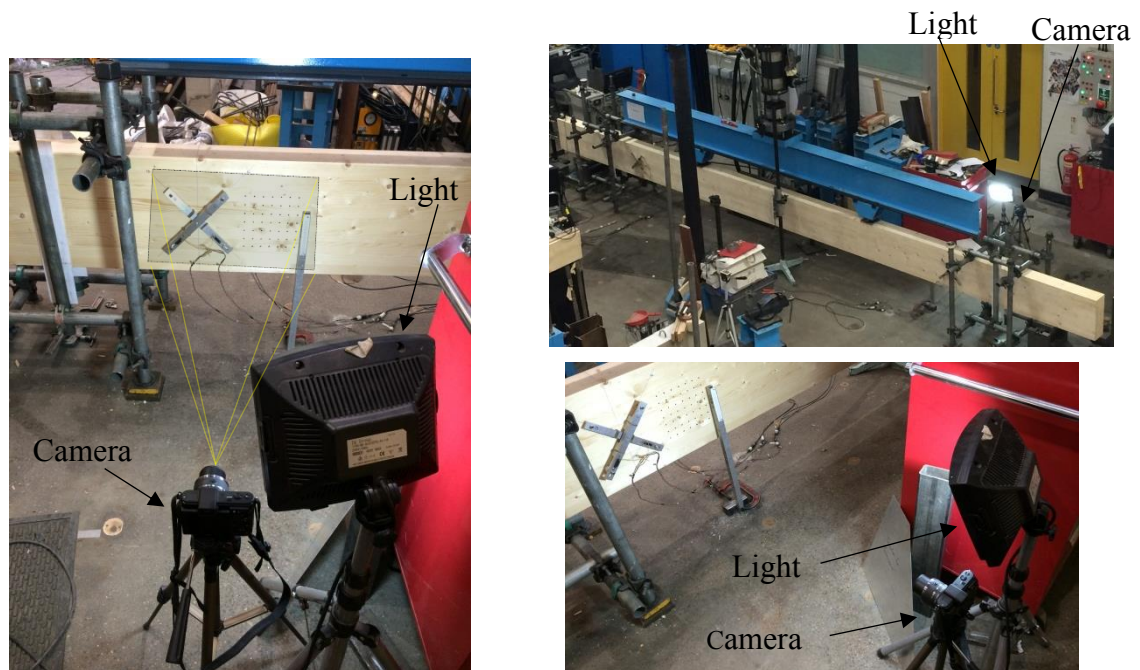


Figure 3.18 – Experimental set-up of the photographic camera used.

Figure 3.19 below, depicts two images processed in Tracker [79]. The image in the left-hand side, shows the positions of the dots at $F = 0 \text{ kN}$ while the image in the right-hand side shows the positions of the dots at $F = F_{max}$.

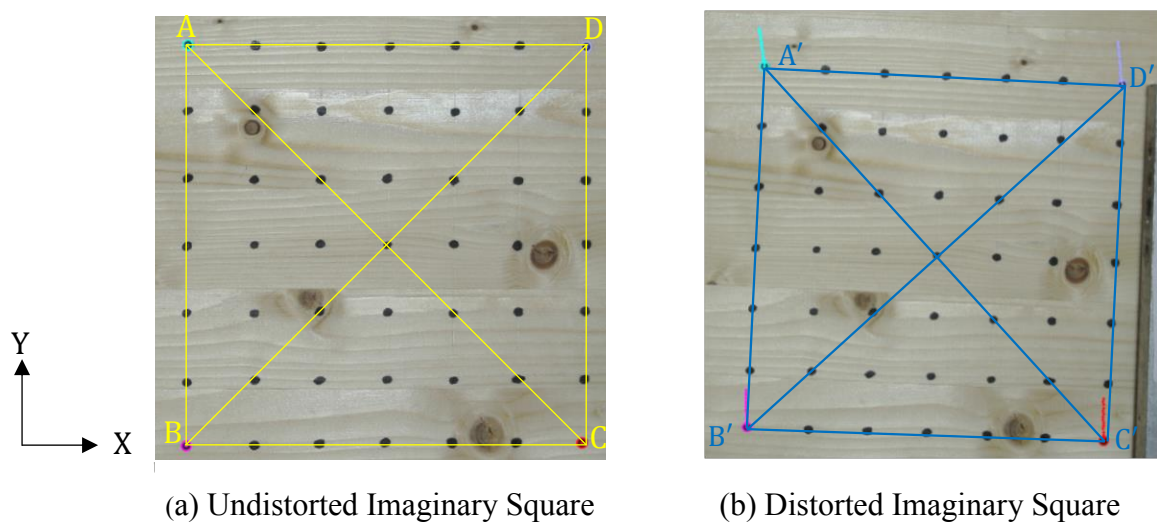


Figure 3.19 – Positions of the dots at $F = 0 \text{ kN}$ and at $F = F_{max}$.

The shear modulus (G_{12}) was estimated from the relative displacement of the diagonals AC and BD. In determining an expression that relates the shear modulus (G_{12}) with the deformation of the two diagonals, it was assumed that the same geometric relationship shown in Figure 3.16 holds and Equation (3.1) was applied. To this end the displacements of the two diagonals $dD_{SF,A}$ and $dD_{SF,B}$ were obtained from Pythagoras Theorem, such that

$$dD_{SF,B} = \sqrt{(x_A - x_C)^2 + (y_A - y_C)^2} \quad (3.2)$$

and

$$dD_{SF,A} = \sqrt{(x_B - x_D)^2 + (y_B - y_D)^2} \quad (3.3)$$

where horizontal (x) and vertical (y) displacements of points A, B, C and D at the two loading intervals F_1 and F_2 were obtained from the software Tracker [79].

3.3 Main results and observations

This section presents and examines the main response parameters obtained from the test series described above. The section is divided into four parts corresponding to the four test types carried out, namely compression, tension, shear and bending.

3.3.1 Compressive resistance

Figures 3.20 and 3.21 present the stress-strain relationships for the compression tests parallel and perpendicular to the grain, respectively. Both figures present the stress-strain behaviour of each individual specimen up to the point of failure, as well as the lower and upper bounds of the test series while Tables 3.1 and 3.2 presents the dimensions and mean response parameters obtained from the compression tests parallel and perpendicular to the grain together with their related average values and coefficients of variation (COV).

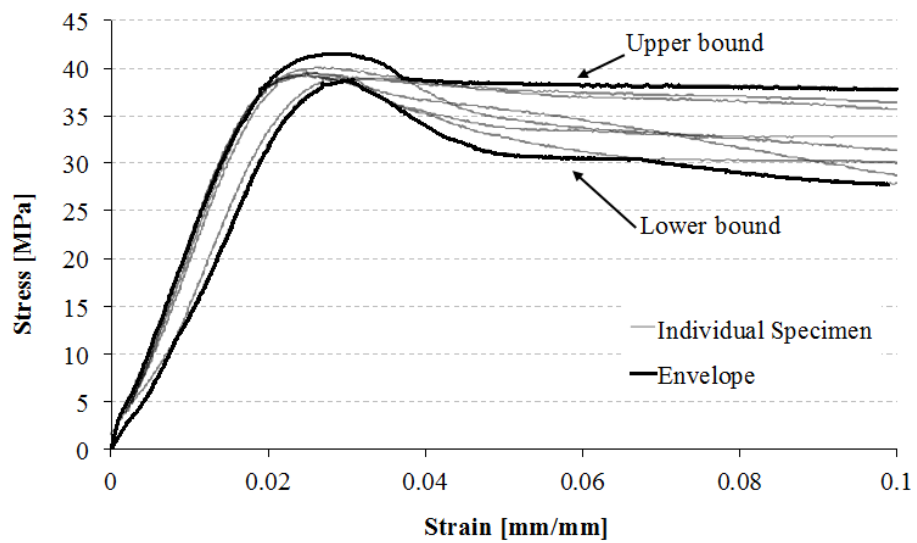


Figure 3.20 – Compression stress-strain relationships for Scandinavian Spruce Glulam parallel to the grain direction

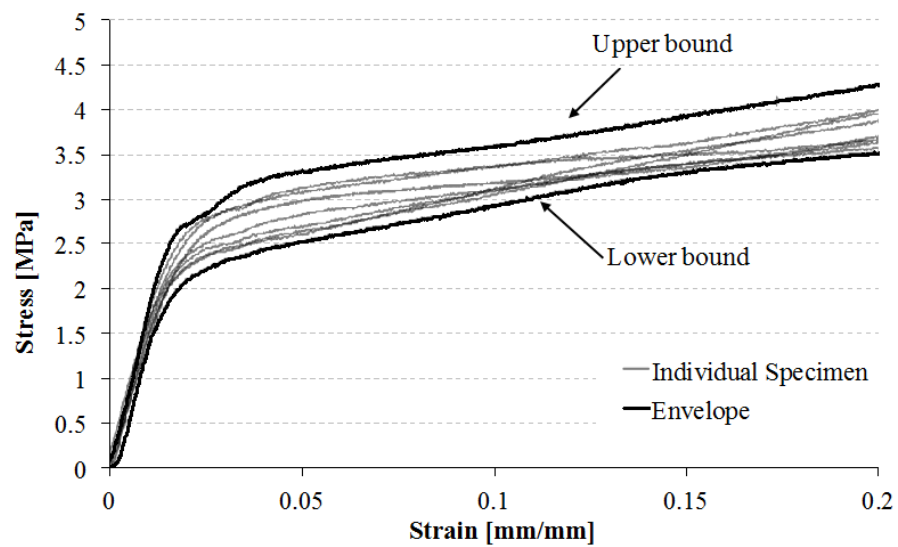


Figure 3.21 – Compression stress-strain relationships for Scandinavian Spruce Glulam perpendicular to the direction of the grain

Specimen	Compressive Area A [mm ²]	Height h [mm]	Compressive strength $\sigma_{11,c}$ [MPa]	Young's modulus $E_{1,c}$ [MPa]
TiCPa_01	2003	45.1	35.2	2336.1
TiCPa_02	2048	45.2	33.6	2020.4
TiCPa_03	2030	45.0	30.0	2168.1
TiCPa_04	2029	45.3	36.2	2189.8
TiCPa_05	2029	45.4	33.4	1828.8
TiCPa_06	2010	45.0	33.7	2266.3
TiCPa_07	2020	45.4	28.9	1396.6
TiCPa_08	2017	45.0	33.2	2200.6
Average	2023	45.2	33.0	2050.8
COV [%]	0.6	0.4	6.9	14.0

Table 3.1 – Dimensions and characteristic parameters for Scandinavian Spruce glulam under compression parallel to grain.

Specimen	Compressive Area A [mm ²]	Height h [mm]	Compressive strength $\sigma_{22,c}$ [MPa]	Young's modulus $E_{2,c}$ [MPa]
TiCPa_01	3085	90.1	3.3	171.9
TiCPa_02	3070	90.0	3.1	210.1
TiCPa_03	3079	90.0	4.0	170.8
TiCPa_04	3077	89.8	3.4	147.3
TiCPa_05	3094	90.1	3.6	184.0
TiCPa_06	3081	90.0	3.9	166.3
TiCPa_07	3089	90.1	4.0	170.8
TiCPa_08	3090	89.7	3.8	166.9
TiCPa_09	3074	89.8	3.6	161.3
Average	3082	89.9	3.6	172.1
COV [%]	0.2	0.2	8.9	9.5

Table 3.2 – Dimensions and characteristic parameters for Scandinavian Spruce glulam under compression perpendicular to grain.

The mean compressive strength parallel ($\sigma_{11,c}$) and perpendicular ($\sigma_{22,c}$) to the grain, reported in Tables 3.1 and 3.2 were obtained by averaging the maximum strength values of each individual response. On the other hand, the Elastic Modulus in compression parallel ($E_{1,c,i}$) and perpendicular ($E_{2,c,i}$) to the grain, for each specimen (i) can be determined as:

$$E_{1,c,i} = \frac{(0.4F_{11,c,max,i} - 0.1F_{11,c,max,i})h_i}{(u_{0.4,i} - u_{0.1,i})A_i} \quad (3.4)$$

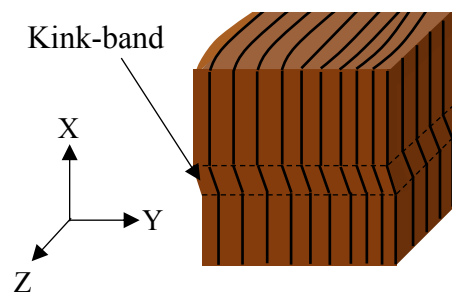
$$E_{2,c,i} = \frac{(0.4F_{22,c,max,i} - 0.1F_{22,c,max,i})h_i}{(u_{0.4,i} - u_{0.1,i})A_i} \quad (3.5)$$

where, $F_{11,c,max,i}$ is the maximum compressive load reached by the timber specimen parallel to the grain, $F_{22,c,max,i}$ is the maximum compressive load reached by the timber sample, perpendicular to the grain, A_i is the area under stress, h_i is the specimen height (see Figures 3.1 and 3.2), and $u_{0.1,i}$ and $u_{0.4,i}$ are deformation increments corresponding to 10 % and 40 % of $F_{11,c,max,i}$ or $F_{22,c,max,i}$, respectively.

Timber specimens undergo four main deformation stages when they are compressed parallel to the grain by less than 15% of their initial height. These are: a) linear-elastic stage, b) incipient kinking stage, c) transient kinking stage and d) steady-state kinking. The incipient kinking stage starts at the end of the elastic stage, where the misaligned fibres located close to the resin canals start to buckle. When the maximum force is reached, the transient kinking phase starts and it is defined as the unification of all the incipient kinking areas in a single kink-band that spreads throughout the entire width of the timber specimen as depicted in Figure 3.22(b). In this phase, the damage caused in the specimens becomes very noticeable. Additionally, this stage is characterised by a decrease in the force, up to the point where it reaches a steady-state. This force plateau is attained when the fibres inside the kink-band are unable to rotate and the specimen has reached the steady-state kinking. When the compression exceeds the 15% strain limit, then the specimens reach a fifth stage which is called “*crushing stage*”. However, this study focuses of the first four deformation stages of the specimens.



(b) Experiment



(a) Schematic

Figure 3.22 – Failure state of timber specimen when compressed parallel-to-grain at 10% strain.

The behaviour of timber under compression perpendicular to the grain can be easily understood by examining it on the microscopic level. As stated in a Chapter 2, timber is a cellular material and its microstructure is very similar to that of a honeycomb. In technical terms, such type of cellular material is called “*closed-cell foam*”. Figure 3.23 shows in a microscopic level, the failure behaviour of timber under compression in the radial direction (perpendicular to the grain). The compression force applied on the timber, causes the cells to collapse. The collapse of the timber cells in the microscopic scale shown in Figure 3.23, propagates into the macroscopic level. This can be seen in Figure 3.24, where the isometric view of a timber specimen at 20% strain is presented, the collapsed timber cells causing the specimen to fold around that section are evident in Figure 3.24.

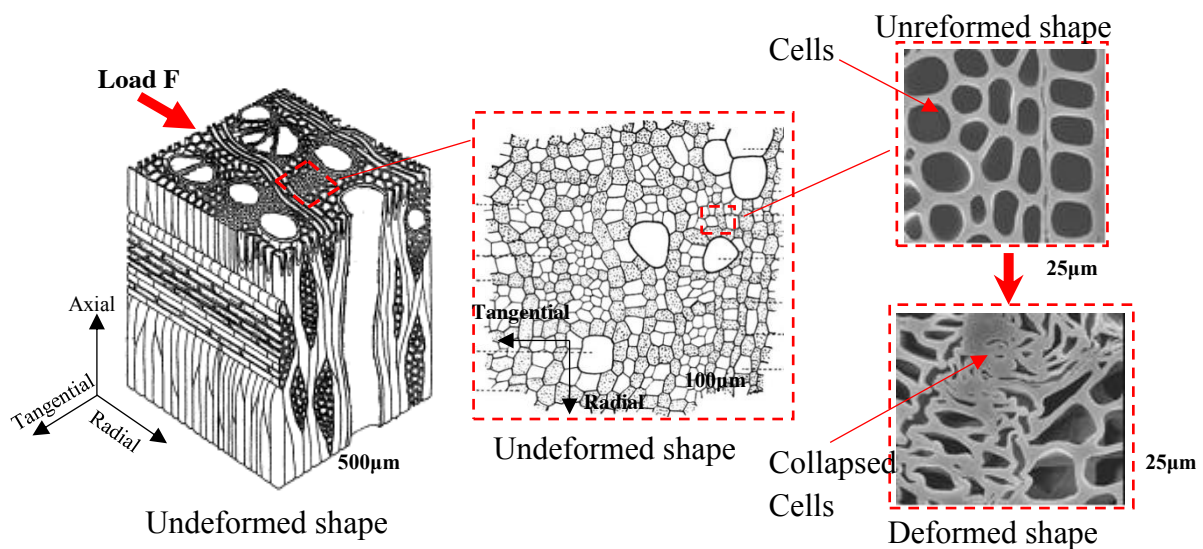


Figure 3.23 – Compression of timber perpendicular to grain in a microscopic level [80].

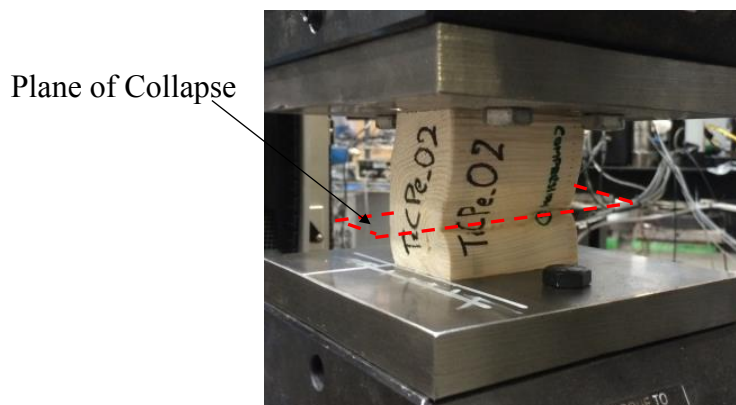


Figure 3.24 – Failure state of timber specimen when compressed perpendicular-to-grain at 20% strain.

3.3.2 Tensile resistance

Figures 3.25 and 3.26 present the stress-strain relationships for the tension tests parallel and perpendicular to the grain, respectively. Similar to the compression tests, both figures display the stress-strain behaviour of each individual specimen up to the point of failure together with the corresponding lower and upper bounds.

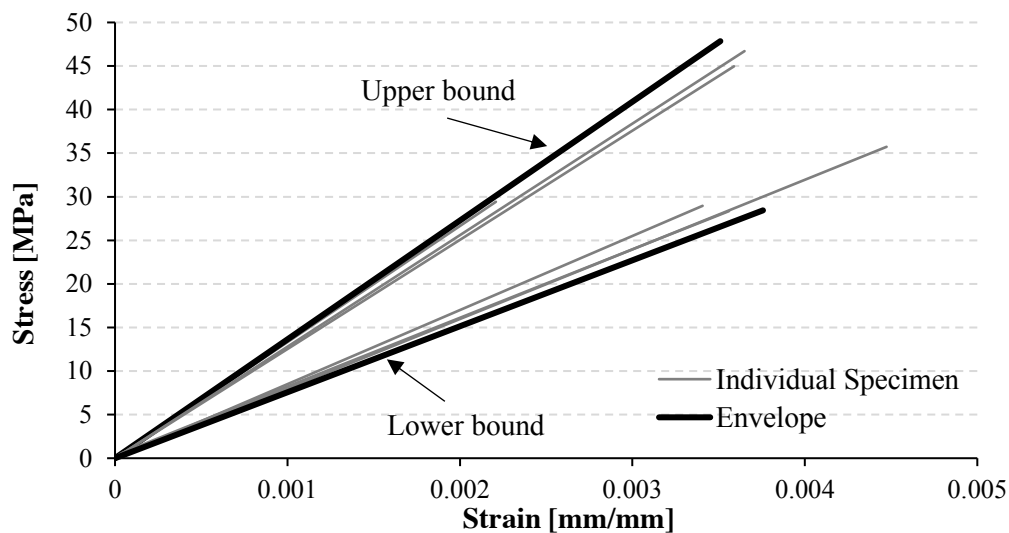


Figure 3.25 – Stress-strain relationships for Scandinavian Spruce Glulam under tension parallel to the grain direction

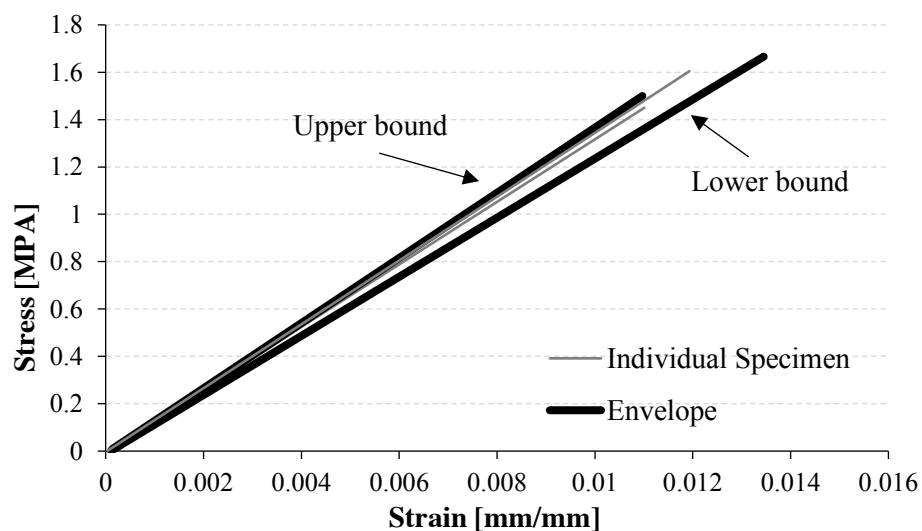


Figure 3.26 – Stress-strain relationships for Scandinavian Spruce Glulam under tension perpendicular to the grain direction

Tables 3.3 and 3.4 present the dimensions and mean response parameters obtained from the tensile tests parallel and perpendicular to the grain together with their related average values and coefficients of variation (COV).

Specimen	Compressive Area A [mm ²]	Height h [mm]	Tensile strength $\sigma_{11,t}$ [MPa]	Young's modulus $E_{1,t}$ [MPa]
TiTPa_01	212	179.9	28.4	7756.4
TiTPa_02	210	179.8	29.4	13578.5
TiTPa_03	210	179.5	47.9	14500.0
TiTPa_04	207	179.6	46.7	13738.2
TiTPa_05	202	179.7	45.0	12488.8
TiTPa_06	203	179.6	28.4	8768.0
TiTPa_07	209	179.3	35.7	8714.6
TiTPa_08	206	179.4	28.4	8049.1
Average	207	179.6	36.2	10949.2
COV [%]	1.4	0.1	22.9	22.3

Table 3.3 – Dimensions and characteristic parameters for Scandinavian Spruce glulam under tension parallel to grain.

Specimen	Compressive Area A [mm ²]	Height h [mm]	Tensile strength $\sigma_{22,t}$ [MPa]	Young's modulus $E_{2,t}$ [MPa]
TiTPa_01	3080	181.0	1.4	181.0
TiTPa_02	3059	178.8	1.5	178.9
TiTPa_03	3094	179.7	1.7	193.2
TiTPa_04	3108	179.9	1.6	171.5
Average	3086	179.9	1.6	181.1
COV [%]	0.1	0.6	5.7	4.3

Table 3.4 – Dimensions and characteristic parameters for Scandinavian Spruce glulam under tension perpendicular to grain.

The mean tensile strength parallel ($\sigma_{11,t}$) and perpendicular ($\sigma_{22,t}$) to the grain, reported in Tables 3.3 and 3.4 were obtained by averaging the maximum strength values of each individual response. On the other hand, the Elastic Modulus in tension parallel ($E_{1,t,i}$) and perpendicular ($E_{2,t,i}$) to the grain, for each specimen (i) was determined as:

$$E_{1,t,i} = \frac{(0.4F_{11,t,max,i} - 0.1F_{11,t,max,i})h_i}{(u_{0.4,i} - u_{0.1,i})A_i} \quad (3.6)$$

$$E_{2,t,i} = \frac{(0.4F_{22,t,max,i} - 0.1F_{22,t,max,i})h_i}{(u_{0.4,i} - u_{0.1,i})A_i} \quad (3.7)$$

where, $F_{11,t,max,i}$ is the maximum tensile load reached by the timber specimen parallel to the grain, $F_{22,t,max,i}$ is the maximum tensile load reached by the timber sample perpendicular to the grain, A_i is the area under stress, h_i is the specimen height (see Figures 3.4 and 3.5), and $u_{0.1,i}$ and $u_{0.4,i}$ are deformation increments corresponding to 10 % and 40 % of $F_{11,t,max,i}$ or $F_{22,t,max,i}$, respectively. As it can be seen in Figures 3.25 and 3.26 when subjected to tension forces, timber behaves in a brittle manner independent to the direction of the grain. However, as expected, timber is stronger in the direction parallel to the grain.

3.3.3 Shear resistance

Similar to the compression and tension tests, Figure 3.27 shows the stress-strain behaviour of each individual specimen up to the point of failure, as well as the lower and upper bounds of the set, while Table 3.5 presents the dimensions and mean response parameters obtained from the shear tests parallel to the grain together with their related average values and coefficients of variation (COV). Finally, it is important to note that according to the European Standard EN408 [74], this type of test is not used to determine the shear modulus. The shear modulus is determined from the results obtained from the four-point bending test presented in the following section.

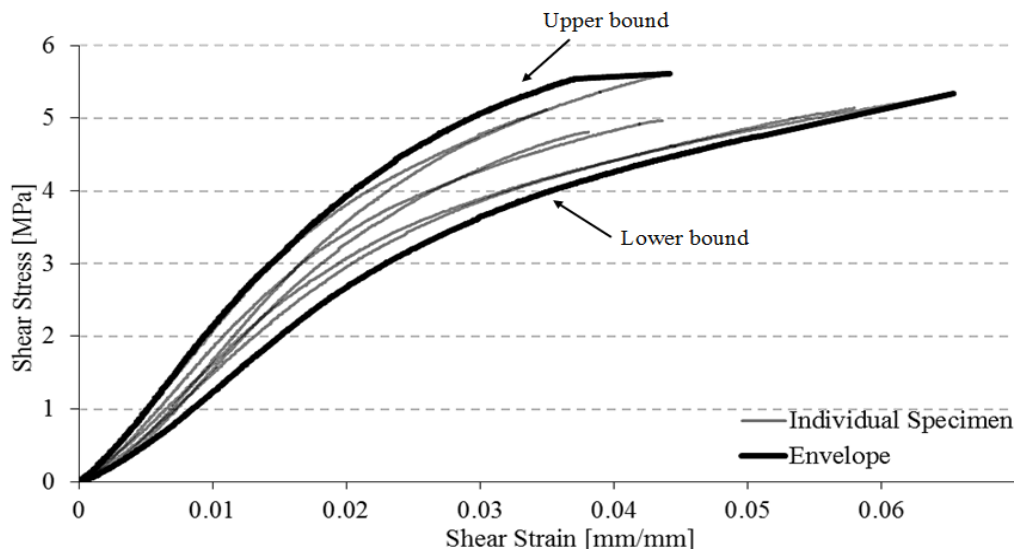


Figure 3.27 – Shear stress-strain relationships for Scandinavian Spruce Glulam.

Specimen	Compressive Area A [mm ²]	Width b [mm]	Shear strength σ_{12} [MPa]
TiSPa_01	9651	54.8	5.0
TiSPa_02	9664	54.2	4.8
TiSPa_03	9653	54.8	4.7
TiSPa_04	9647	55.0	5.0
TiSPa_05	9714	54.9	4.8
TiSPa_06	9646	54.9	5.5
TiSPa_07	9641	54.9	5.1
TiSPa_08	9599	54.8	5.5
Average	9652	54.8	5.0
COV [%]	0.3	0.4	5.9

Table 3.5 – Dimensions and characteristic parameters for Scandinavian Spruce glulam under shear parallel to grain.

The shear strength of each specimen was calculated as:

$$\sigma_{12,i} = \frac{F_{12,max,i}(\cos 14^\circ)}{A_i} \quad (3.8)$$

where, $F_{12,max,i}$ is the maximum compressive load reached and A_i is the area under stress ($h_i \cdot l_i$).

Figure 3.28 below, shows the state of the specimen subjected shear after testing. As it can be observed from this figure, the specimen split almost in half in a brittle manner. The nonlinear stress-strain relationship indicates that the specimen was subjected to a combination of compressive and tensile loads.

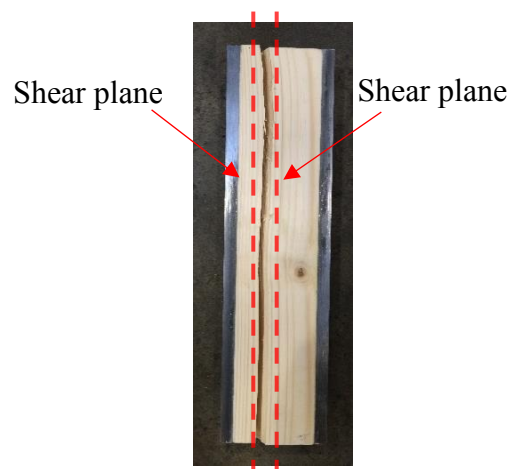


Figure 3.28 – Failure state of timber specimen subjected to shear parallel-to-grain.

3.3.4 Bending resistance

Figure 3.29, shows the force-displacement relationships obtained from the bending tests while, Table 3.6 presents the mean response parameters together with their related average values and coefficients of variation (COV).

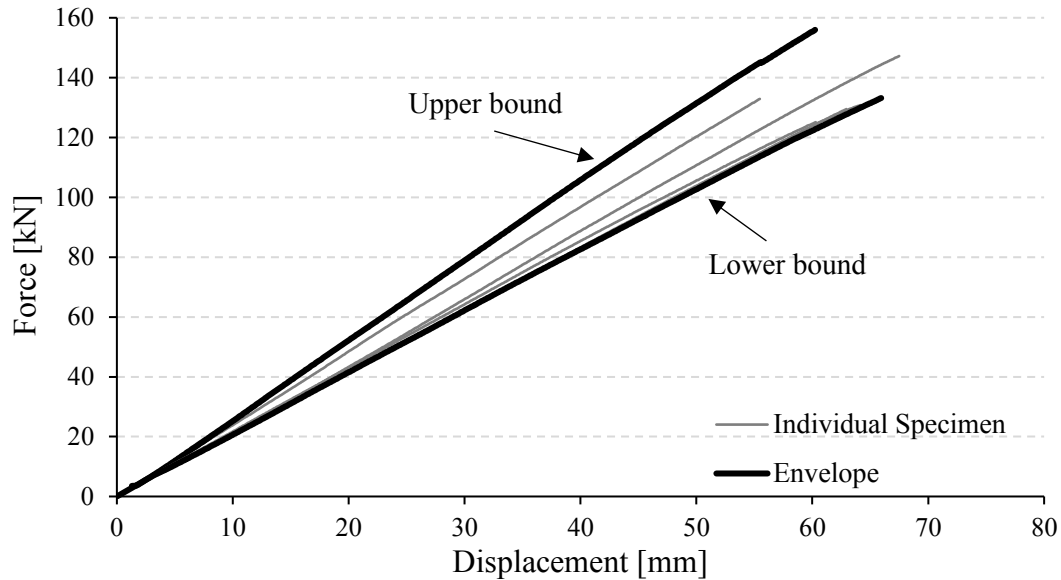


Figure 3.29 – Bending force-displacement relationships for Scandinavian Spruce Glulam.

Specimen	Bending Stress σ_b [MPa]	Young's modulus E_1 [MPa]		Shear Modulus G_{12} [MPa]	
		Method A	Method B	Method A	Method B
Beam 01	33.2	12996.1	-	1935.3	-
Beam 02	37.3	11379.9	11803.7	2322.8	764.1
Beam 03	31.2	10516.7	11030.6	1863.5	565.1
Beam 04	33.8	13481.7	15128.5	1737.7	364.1
Beam 05	28.8	12234.7	12494.8	2219.8	1101.2
Beam 06	28.9	10302.8	10329.6	2266.2	1964.6
Average	32.2	11818.6	12157.4	2057.5	951.8
COV [%]	9.2	10.1	13.6	10.8	59.0

Table 3.6 – Characteristic parameters for Scandinavian Spruce glulam under bending.

The Young's modulus parallel to the grain, E_1 , and the bending strength, σ_b , were estimated from the equations given in EN 408 [74] that is

$$E_1 = \frac{3a \cdot l^2 - 4a^3}{2b \cdot h^3 \left(2 \frac{u_2 - u_1}{F_2 - F_1} - \frac{6a}{5G_{12} \cdot b \cdot h} \right)} \quad (3.9)$$

and

$$\sigma_b = \frac{3a \cdot F_{max}}{b \cdot h^2} \quad (3.10)$$

Where, a is the distance between the loading position and the support, l is the span in bending, b is the width of the beam, h is the height of the beam, $(F_2 - F_1)$ is an increment of load, $(u_2 - u_1)$ is the increment of deformation corresponding to $(F_2 - F_1)$ and G_{12} in the shear modulus. It can be seen from equation (3.24) that the estimation of the Young's modulus depends on the shear modulus, and since two methods were used to determine the shear modulus, the two corresponding values are reported in Table 3.6.

In order to compare the mechanical properties of the glulam beam with the characteristic values tabulated in European EN 1194 [81], the data obtained from the bending tests were multiplied by

$$k_{size} = \left(\frac{b}{150} \right)^{0.05} \cdot \left(\frac{h}{600} \right)^{0.1}, \quad (3.11)$$

since the cross-sections of the beam were smaller than the 150 mm × 600 mm.

As it is illustrated in Figure 3.30, when a simple supported glulam beam is subjected into bending loads, the top laminas are subjected to compressive loads while the bottom laminas are subjected to tensile loads. Besides, the beams are more susceptible to fail in the bottom part since, timber is weaker in tension compared to compression. In fact, all the beams failed when bottom laminas could not sustain tensile force.

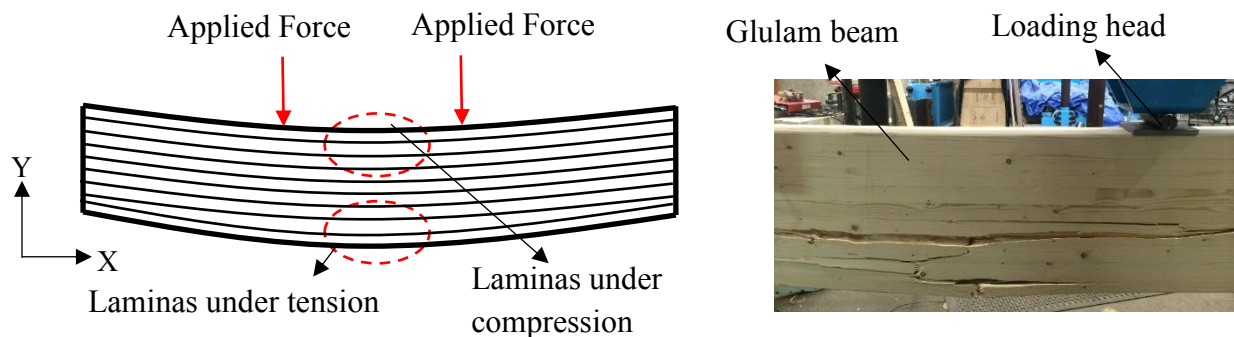


Figure 3.30 – Compression and tension regions and failure of glulam beam after a four-point bending test.

3.4 Concluding remarks

This chapter has described an experimental programme conducted in order to determine the mechanical properties of the Scandinavian Spruce glulam beams employed in this thesis. The test results provided essential information for the validation of the detailed analytical and numerical models proposed in the following chapters.

By comparing Table 3.1 with Table 3.2 and Table 3.3 with 3.4, it can be easily deduced that the Young's modulus and strength of the timber along the direction parallel to the grain is much higher than that in the radial directions, as expected. Similarly, from Figures 3.20 and 3.21, it can be observed that when compressed in the direction parallel to the grain wood exhibits ductile behaviour with softening, while when compressed perpendicular to the grain. The response is, ductile with hardening. These results, agree with the work conducted by Poulsen [21] and Franke [22], where clear *Picea abies* (spruce) specimens of different heights were compressed parallel to grain and perpendicular to grain.

Also, from the comparison of Figures 3.25 and 3.26, it is clear that Timber specimens are susceptible to brittle failure when subjected to tension, regardless of the direction of grain. It should be noted that the behaviour of these wood specimens under tension in the direction perpendicular to grain, exhibits a narrow region of plastic deformation. Nevertheless, there is no significant difference in the general shape of the curves produced by loads parallel and perpendicular to the grain. The major differences are in the load carrying capacity and stiffness values. To this end wood under tension parallel to the grain is able to carry far higher loads than when loaded perpendicular to the grain. Similarly, a brittle behaviour in the timber was observed under the effect of shear. The abrupt failure of the timber specimens at strains close to 5%, is clear from Figure 3.27. However, the failure behaviour is slightly smoother compared to the tensile behaviour. The reason for that, lies on the interaction between the two shear planes induced by the testing arrangement

Finally, the bending behaviour of the glulam beams was also brittle under the effect of monotonic loading. Peak mid-span displacements of up to 65 mm of mid-span were. This chapter has also presented the determination of the shear modulus accomplished by two instantiations of the shear-field method. The first implementation relies on a cross-shaped disposition of transducers European Standard EN 408 [74], whereas the second alternative employs direct digital image capture. Both methods were used to measure shear distortions at

particular locations in the beams from which the shear modulus can be obtained. By examining the values obtained for the shear modulus presented in Table 3.6 and comparing them with the corresponding characteristic value presented in European Standard EN 1194 [81], it can be observed that the average shear modulus estimated from digital image tracking is far more consistent than the experimental configuration originally proposed in the European Standard EN 1194 [81].

Table 3.7 summarises the material parameters obtained herein which will be used on subsequent chapters of this thesis for the development of the FE models. By comparing the results of table 3.7 with the corresponding values of EN 338 [82], good general agreement between the experimental and the characteristic values for C30 strength class timber is observed well within the variation expected. Nonetheless, the organic nature of timber makes it necessary a precise characterization such as the one offered in this chapter, especially if advanced FE models are to be developed on this basis and contrasted with experimental results.

Bending Test	
Bending Stress σ_b [MPa]	32.2
Young's modulus E_1 [MPa]	12157.4
Shear Modulus G_{12} [MPa]	951.8
Compression Tests	
Compressive strength $\sigma_{11,c}$ [MPa]	33.0
Young's modulus $E_{1,c}$ [MPa]	2050.8
Compressive strength $\sigma_{22,c}$ [MPa]	3.6
Young's modulus $E_{2,c}$ [MPa]	172.1
Tension Tests	
Tensile strength $\sigma_{11,t}$ [MPa]	36.2
Young's modulus $E_{1,t}$ [MPa]	10949.2
Tensile strength $\sigma_{22,90}$ [MPa]	1.6
Young's modulus $E_{2,t}$ [MPa]	181.1
Shear Tests	
Shear strength σ_{12} [MPa]	5.0

Table 3.7 – Summary of the average mechanical properties obtained for the Scandinavian Spruce.

Chapter 4

Embedment Tests

4.1 Introduction

The literature review presented in Chapter 2 has highlighted the difficulties associated with modelling the response of structural wood. These difficulties are accentuated when dealing with the behaviour of timber joints due to the intricate and highly non-linear interactions that take place between the different connection components. To this end, the previous chapter has presented the experimental characterisation of the material properties of the glulam under various loading configurations. Although the determination of those parameters is essential for the numerical study of beam-to-column connections, the issue of adequately modelling embedment in wood still constitutes an important challenge. Furthermore, an accurate modelling of embedment is crucial for a truthful representation of any joint connection behaviour since, as also highlighted in Chapter 2, it constitutes the basic building block of the full connection response. To this end, this chapter presents and discusses the experimental programme conducted on the characterization of the embedment behaviour of single-dowel connections in glulam. The experimental program described in this chapter examines the localized nonlinear behaviour of timber around the steel dowels and serves as the basis for the development of the advanced Finite Element (FE) models presented in subsequent chapters.

4.2 Single-dowel connection test arrangement and details

Six (6) test-series were conducted in order to characterise the embedment behaviour in the glulam material employed in this thesis making a total of fifty-two (52) single-dowel embedment individual tests. Thirty-seven (37) compression embedment tests were conducted at both parallel and perpendicular to grain directions, while fifteen (15) tension embedment tests were carried out in the direction parallel to the grain. Two dowel diameters were considered (i.e. 12 and 16 mm) of Grade 10.9. The tests were carried out in accordance with the European EN 383 Specification [83]. It is important to note that tensile tests perpendicular to

the direction of the grain, were not performed since the failure mode associated with these tests is not embedment and is beyond the scope of this project. A constant rate of 1 mm/min was applied to the dowels by means of a purpose-built metallic assembly consisting of two steel side plates 100 mm wide and 20 mm thick welded and bolted together via a spacer steel block, as shown in Figure 4.1. The load and displacement at the centreline of the dowel were constantly recorded with accuracies of ± 0.1 kN and ± 0.05 mm, respectively. All specimens were tested until failure. The dimensions of the one-dowel connection specimens which depend on the dowel diameter were determined with reference to EN 383 [83] and are summarised in Table 4.1.

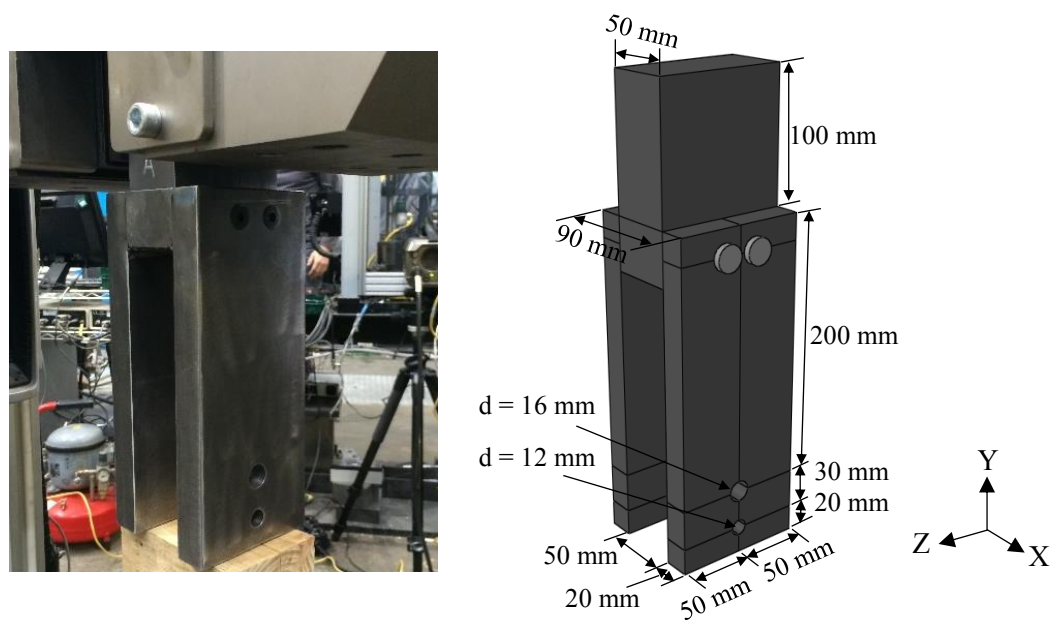


Figure 4.1 – Embedment test holder assembly.

d	Comp. Parallel to grain			Comp. Perpendicular to grain			Tension Parallel to grain		
	Width	Height	Thickness	Width	Height	Thickness	Width	Height	Thickness
12	120	168	40	480	120	40	120	528	40
16	160	224	40	640	160	40	160	704	40

Table 4.1 – Specimen dimensions for the embedment tests. (Units: mm).

4.2.1 Compression embedment parallel to grain

Twelve (12) specimens were fabricated to study the embedment behaviour of single-dowel connections under compression parallel to grain with a dowel of 12 mm diameter and eight (8) specimens with 16 mm dowel diameter. The experimental set-up of is shown in Figure 4.2. The specimen's nomenclature employed is depicted in Figure 4.3:



(a) Dowel diameter: 12 mm



(b) Dowel diameter: 16 mm

Figure 4.2 – Experimental set-up of single-dowel connection compression test parallel to the grain.

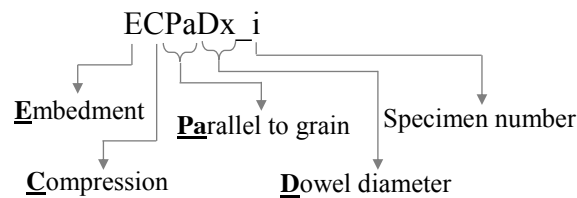
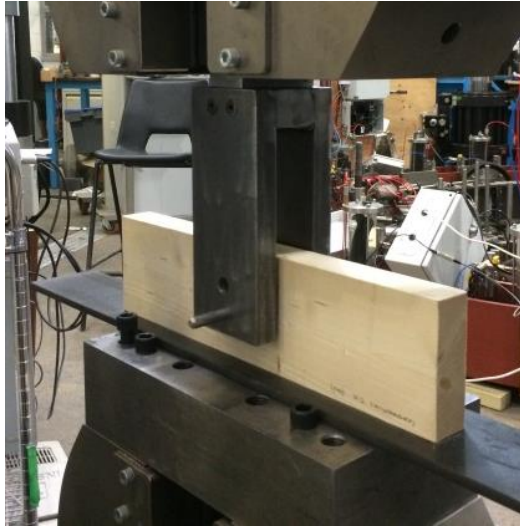


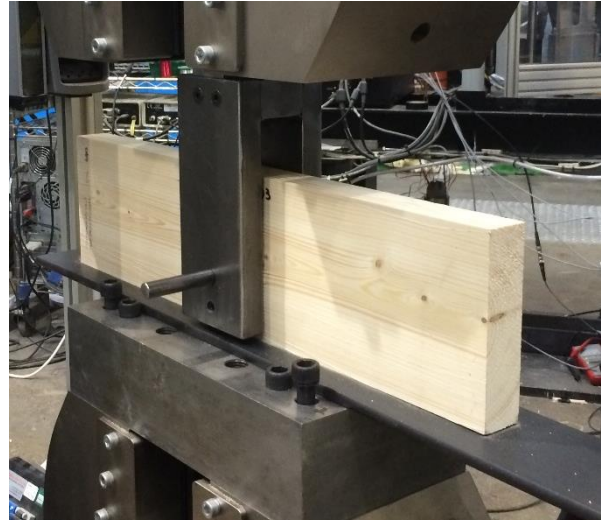
Figure 4.3 – Nomenclature of embedment specimens subjected to compression parallel to the grain.

4.2.2 Compressions embedment perpendicular to grain

Eight (8) specimens were manufactured to examine the embedment behaviour of single-dowel connections under compression perpendicular to grain with a 12 mm diameter dowel and nine (9) specimens with 16 mm diameter dowel. The corresponding experimental set-up is shown in Figure 4.4. The tested specimens were named as depicted in Figure 4.5.



(a) Dowel diameter: 12 mm



(b) Dowel diameter: 16 mm

Figure 4.4 – Experimental set-up of single-dowel connection compression test perpendicular to the grain.

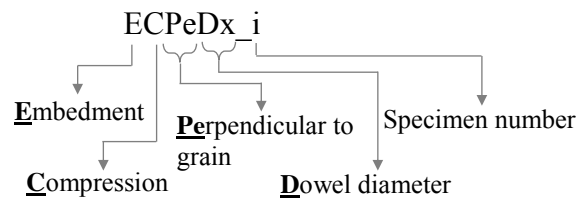
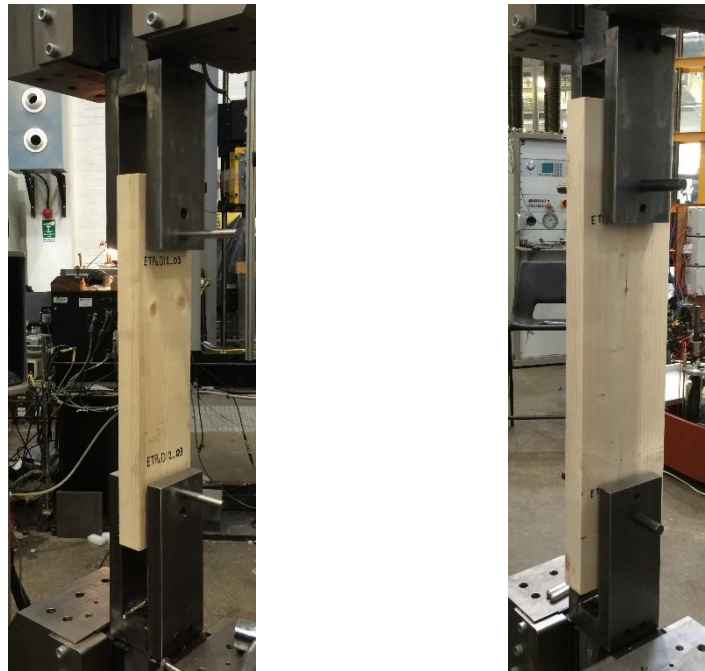


Figure 4.5 – Nomenclature of specimens for the embedment tests under compression perpendicular to the grain.

4.2.3 Tension Embedment test parallel to grain

For the embedment tests of single-dowel connection under tension, two metallic brackets one at each extreme were used as depicted in Figure 4.6. One bracket was used to connect the top part of the specimens to the moving clamp of the testing machine, while the second was attached to the bottom part of the specimen. Fifteen (15) specimens were tested under these conditions with a 12 mm and 16 mm dowels. The tested specimens were named as depicted in Figure 4.7.



(a) Dowel diameter: 12 mm

(b) Dowel diameter: 16 mm

Figure 4.6 – Experimental set-up of single-dowel connection tensile test parallel to the grain.

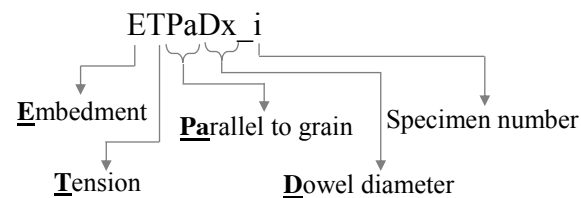


Figure 4.7 – Nomenclature of specimens for the embedment tests under tension parallel to the grain.

4.3 Main results and observations

The main response parameters obtained from the embedment tests discussed above are presented in Tables 4.2, 4.3 and 4.4. These tables include the number of specimens, specimen dimensions, embedment strength, embedment elastic modulus and their corresponding coefficient of variation (COV).

4. Embedment Tests

12 mm dowel						16 mm dowel					
Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{11,Emb}$ [MPa]	Embed. Elastic modulus $E_{1,Emb}$ [MPa]	Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{11,Emb}$ [MPa]	Embed. Elastic modulus $E_{1,Emb}$ [MPa]
ECPaD12_01	120.0	168.2	40.0	28.5	347.6	ECPaD16_01	160.1	224.1	40.5	32.5	618.4
ECPaD12_02	120.1	168.3	40.4	29.2	263.7	ECPaD16_02	160.0	224.0	40.5	30.2	852.2
ECPaD12_03	119.9	168.0	40.2	30.6	481.5	ECPaD16_03	159.8	224.2	40.4	30.1	819.5
ECPaD12_04	119.8	168.1	39.7	24.4	316.0	ECPaD16_04	160.1	224.6	40.6	28.2	856.5
ECPaD12_05	120.3	168.0	40.1	29.1	378.4	ECPaD16_05	160.0	224.1	40.4	25.8	716.4
ECPaD12_06	120.2	168.0	40.1	34.4	362.5	ECPaD16_06	159.9	223.9	40.7	33.9	1056.8
ECPaD12_07	120.1	168.1	40.2	28.6	334.4	ECPaD16_07	160.1	224.2	40.3	26.7	675.3
ECPaD12_08	120.1	168.1	40.0	27.4	460.2	ECPaD16_08	160.0	224.0	40.1	29.2	664.5
ECPaD12_09	120.1	167.9	39.9	32.0	424.0						
ECPaD12_10	119.8	167.9	40.2	30.4	368.0						
ECPaD12_11	120.0	167.8	39.7	29.0	400.0						
ECPaD12_12	120.1	167.8	39.7	29.4	376.0						
Average	120.0	168.0	40.0	29.4	376.0	Average	160.0	224.2	40.5	29.6	782.5
COV [%]	0.1	0.1	0.5	7.9	15.4	COV [%]	0.1	0.1	0.4	8.7	17.1

Table 4.2 – Results of embedment tests under compression loads parallel to the direction of the grain.

4. Embedment Tests

12 mm dowel						16 mm dowel					
Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{22,Emb}$ [MPa]	Embed. Elastic modulus $E_{2,Emb}$ [MPa]	Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{22,Emb}$ [MPa]	Embed. Elastic modulus $E_{2,Emb}$ [MPa]
ECPeD12_01	478.3	119.1	39.2	11.6	55.7	ECPeD16_01	640.3	159.3	40.2	13.3	240.3
ECPeD12_02	480.0	119.4	40.2	14.5	123.2	ECPeD16_02	640.6	159.6	40.2	18.4	240.9
ECPeD12_03	479.9	119.5	39.8	13.9	186.4	ECPeD16_03	640.4	159.7	40.2	15.0	251.7
ECPeD12_04	480.0	119.2	39.7	13.1	112.7	ECPeD16_04	640.3	159.6	40.7	10.0	258.0
ECPeD12_05	479.8	119.5	40.2	16.1	165.9	ECPeD16_05	640.0	159.4	40.5	12.7	205.4
ECPeD12_06	479.9	119.5	40.2	17.5	185.7	ECPeD16_06	640.1	159.3	40.2	15.0	225.2
ECPeD12_07	480.0	119.7	39.8	14.9	154.9	ECPeD16_07	640.2	159.7	40.2	13.5	231.2
ECPeD12_08	480.0	118.7	39.5	16.2	110.1	ECPeD16_08	640.6	159.3	40.3	18.4	250.9
						ECPeD16_09	640.4	159.4	40.0	15.0	263.3
Average	479.7	119.3	39.8	14.7	136.8	Average	640.3	159.4	40.3	14.6	240.8
COV [%]	0.1	0.2	0.9	11.9	30.7	COV [%]	0.1	0.1	0.4	17.2	7.1

Table 4.3 – Results of embedment tests under compression loads perpendicular to the direction of the grain.

12 mm dowel						16 mm dowel					
Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{11,Emb}$ [MPa]	Embed. Elastic modulus $E_{1,Emb}$ [MPa]	Specimen	Width [mm]	Height [mm]	Thickness [mm]	Embed. strength $\sigma_{11,Emb}$ [MPa]	Embed. Elastic modulus $E_{1,Emb}$ [MPa]
ETPaD12_01	119.5	527.8	39.8	26.2	260.8	ETPaD16_01	159.7	752.5	40.3	27.2	386.4
ETPaD12_02	119.3	527.7	39.9	26.1	274.7	ETPaD16_02	159.7	752.7	40.3	30.1	506.6
ETPaD12_03	119.7	527.8	39.9	24.1	269.4	ETPaD16_03	159.8	752.6	40.1	28.0	437.8
ETPaD12_04	119.6	527.7	40.2	30.6	308.2	ETPaD16_04	159.7	752.2	40.0	29.5	491.3
ETPaD12_05	119.8	527.7	40.0	29.2	284.4	ETPaD16_05	159.2	752.3	40.0	32.3	600.0
ETPaD12_06	119.5	527.7	40.1	29.2	291.8	ETPaD16_06	159.6	752.5	40.2	33.0	559.5
ETPaD12_07	119.3	527.7	40.0	27.5	272.5	ETPaD16_07	159.6	752.3	40.1	28.2	478.9
ETPaD12_08	119.4	527.8	39.9	28.3	313.0						
Average	119.5	527.7	40.0	27.6	284.4	Average	159.6	752.4	40.2	29.7	494.4
COV [%]	0.1	0.01	0.3	7.2	6.2	COV [%]	0.1	0.02	0.3	6.9	13.4

Table 4.4 – Results of embedment tests under tensile loads parallel to the direction of the grain.

A bi-linear approximation was employed to calculate the values of strength and Elastic modulus reported in Tables 4.2, 4.3 and 4.4. To this end, elastic-plastic relationships were obtained by employing two lines tangent to the elastic and post-elastic branches of the experimental force-displacement curves as depicted in Figure 4.8. Accordingly, the embedment stress, $\sigma_{a,Emb}$, and Elastic modulus, $E_{a,Emb}$, for each specimen can be determined as:

$$\sigma_{a,Emb} = \frac{F_{y,a,emb}}{d \times t} \quad (4.1)$$

$$E_{a,Emb} = \frac{k_{a,emb}}{t} \quad (4.2)$$

where $F_{y,a,emb}$ is the embedment force at the yield point, u_{emb} is the embedment in the timber in [mm], $k_{a,emb}$ is the slope of the force-embedment curve in the linear-elastic region in [kN/mm] and d and t are the dowel diameter and specimen width in [mm], respectively.

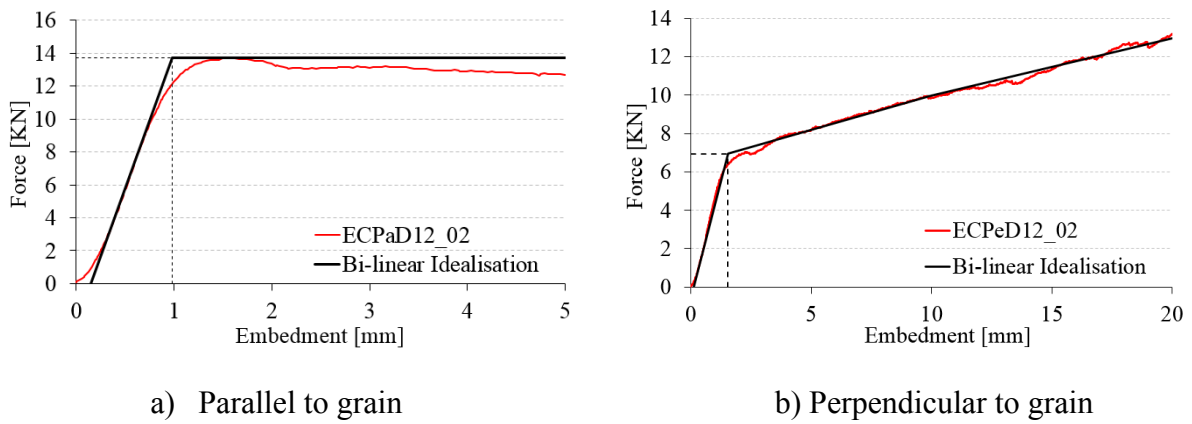


Figure 4.8 – Typical experimental force-embedment curves and their corresponding bi-linear approximations (dowel diameter $d = 12$ mm).

4.3.1 Embedment response parallel to grain under tension

Figure 4.9 shows the results obtained from the embedment tests on glulam with 12 mm and 16 mm dowels. An initial relatively flexible stage is evident from this figure for loads lower than 2 kN. This behaviour is attributed to the non-uniform contact and initial clearance existing between the dowel and the timber material, which leads to the rapid accumulation of localized deformations. Once a uniform contact is attained along the full embedment width, a linear stage develops with forces increasing in direct proportion to the deformation until they reach the embedment strength of the specimen, after which a third behavioural stage develops. This final response stage corresponds to deformations greater than 0.8 mm and 0.5 mm embedment

for 12 and 16 mm dowels, respectively and is characterized by a largely plastic response as can be observed in Figure 4.9.

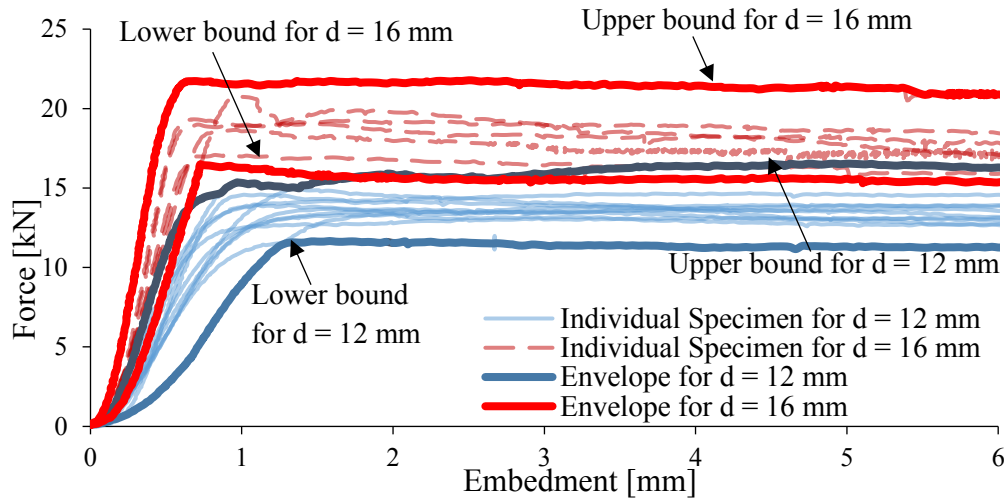


Figure 4.9 – Force-embedment relationship in the direction parallel to the grain under compression.

Mean embedment strength values of 29.4 MPa and 29.6 MPa are observed for the 12 and 16 mm doweled connections, respectively. These values can be compared to the embedment strengths assumed by Eurocode 5 [29] calculated as:

$$\sigma_{11,Emb,DX,k} = 0.082 \times (1 - 0.01d)\rho \quad (4.3)$$

where, $\sigma_{11,Emb,DX,k}$ is the characteristic embedment strength for X dowel diameter, d is the diameter of the dowel and ρ is the density of the timber specimen. By introducing the corresponding values into Equation (4.3), mean characteristic embedment strengths of 31.1 MPa and 30.2 MPa are obtained for 12 mm and 16 mm dowels, respectively. These values are in close agreement with the experimentally obtained mean embedment resistances. Similarly, mean initial stiffness values of values of 376.0 MPa and 1033.9 MPa were observed for the connections with 12 mm and 16 mm dowels, respectively. It is evident from the results reported in Figure 4.9 and Table 4.2 that the larger the dowel diameter the stiffer the response.

4.3.2 Embedment response perpendicular to grain under compression

The experimental procedure followed was identical to the one previously described for embedment tests parallel to the grain direction. Figure 4.10 summarizes the results in terms of force-embedment curves.

The embedment behaviour in the direction perpendicular to the grain follows the general trends already identified before for parallel-to-grain specimens where three behavioural stages are evident from Figure 4.10. An initial stage where the contact area between dowel and timber increases progressively up to a displacement of 0.1 mm is followed by an elastic stage up until the embedment strength of wood is realised at around 7.3 kN for 12 mm dowels (1.8 mm of embedment) and 9.7 kN for 16 mm dowels (1.2 mm of embedment). On the other hand, a marked post-elastic hardening behaviour is observed for embedment in the direction perpendicular to grain (Figure 4.10), which is not evident when the load is applied in the parallel-to-grain direction (Figure 4.9). The crushing of the fibres underneath the dowel, as shown in Figure 4.11, and the associated densification of wood in the contact area cause this hardening response. This will be discussed further with reference to the DIC measurements presented in the following section.

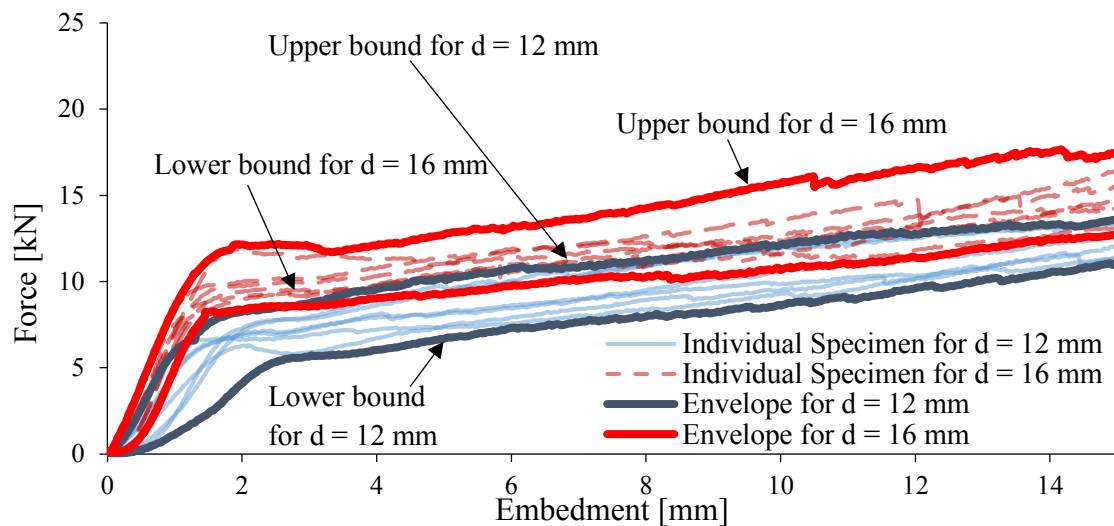


Figure 4.10 – Force-embedment relationship in the direction perpendicular to the grain under compression.

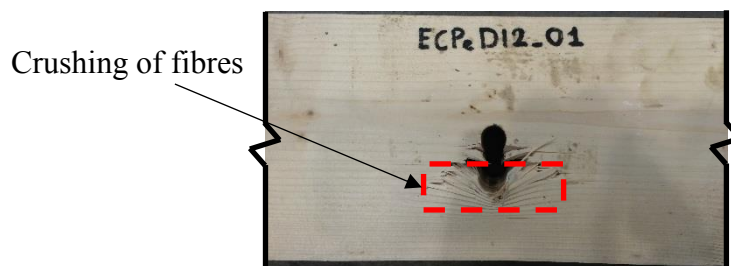


Figure 4.11 – Failure state of embedment tests perpendicular to the direction of the grain.

4.3.3 Embedment response parallel to grain under tension

Figure 4.12 shows the results obtained from the embedment tests on glulam under tension with 12 mm and 16 mm dowels. It can be appreciated from Figure 4.12, that the timber under consideration exhibits similar force-embedment response shapes under both tension and compression parallel to the direction of the grain (see Figure 4.9). Also, Figure 4.13, shows the failure state of the specimens under the effects of compression and tension after testing. In both tests, the specimens split in half at failure.

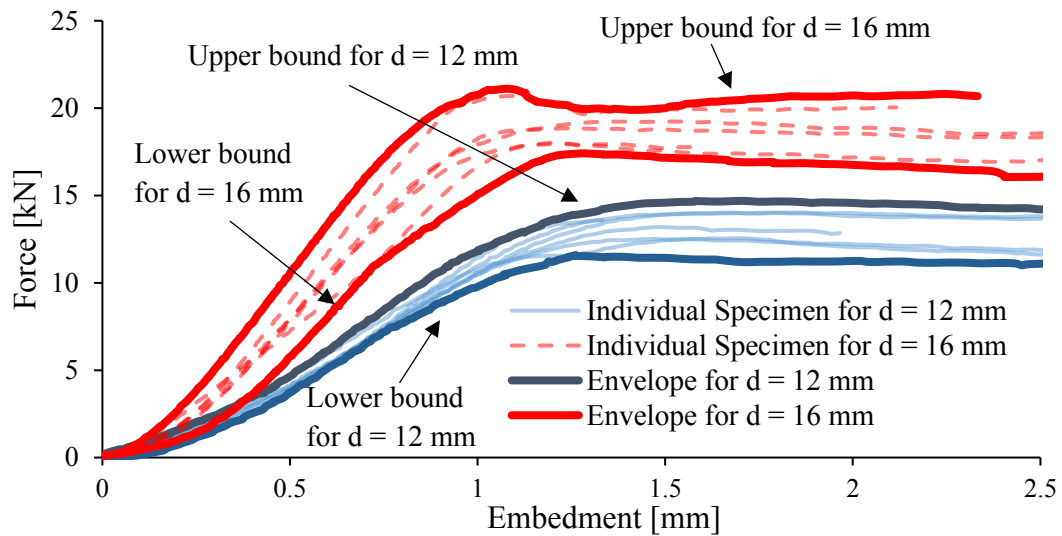
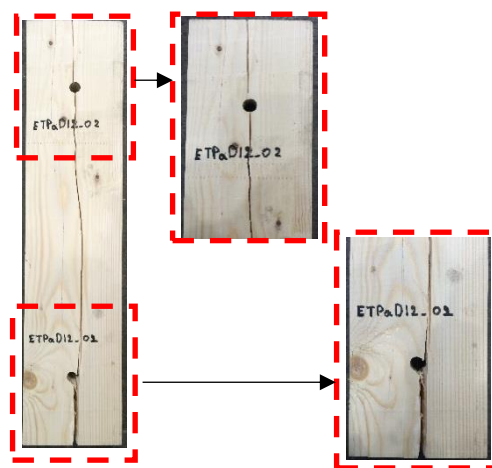


Figure 4.12 – Force-embedment relationship in the direction parallel to the grain under tension.



(a) Embedment compression



(b) Embedment tension

Figure 4.13 – Failure state of embedment tests parallel to the direction of the grain.

From Figure 4.12, it is evident that the specimens experienced the same three behavioural stages as in the compression embedment tests parallel to the grain. This is because the load is acting parallel to the direction of the grain and hence the timber fibres undergo the same deformation stages as the compression tests. However, the final response stage for the tests under tension occurred at higher deformation levels (1.2 mm and 1.0 mm embedment for 12 and 16 mm dowels, respectively) and the plastic response was less ductile compared to the compression tests parallel to the grain. Additionally, by comparing the average values of strength and Young's modulus presented in Table 4.2 with the values shown in Table 4.4, it is evident that the embedment behaviour of timber parallel to the direction of the grain under the effects of tension, produced slightly lower levels of embedment strength and stiffness compared to the respective compression test. The reason for those small differences lies in difference size between two test series. The specimens tested under tensile loads were larger in size and hence they were more prompt to contain defects, such as knots, which affected the material response.

4.3.4 Non-contact field strain measurements

To investigate the localisation of strains around the embedment area, a LaVision [84] Digital Image Correlation (DIC) system was employed. Two cameras were employed in order to reduce the potential in-plane two-dimensional measurement errors associated with small out-of-plane motion. A full description of the binocular stereovision principles on which the DIC technique is based can be found in [85]. It is worth mentioning that only the embedment tests under compression were investigated herein due to the small differences between the embedment compression and tension tests discussed in from the previously. Additionally, care was taken embedment distance did not cause any strut action. Figure 4.14 presents a view of the experimental set-up together with the DIC instrumentation utilized. Two cameras spaced at 500 mm from each other were placed at a distance of 1200 mm from the specimen. The cameras were rotated 15° as presented in Figure 4.14. High quality images were captured during the testing of the specimen at a rate of 31 frames per minute. Calibration of the focus areas was conducted by means of a 200×200 mm² calibration plate while the specimens were prepared by spraying a randomized pattern of dots into the face where strains were to be measured. The DIC algorithm [86] tracks the motion of surface patterns during the experiment and calculates the corresponding 2D strain fields.

Figures 4.15 and 4.16 show typical relative deformation contours obtained by means of the DIC system previously described together with their corresponding force-embedment curves for parallel-to-grain and perpendicular-to-grain specimens, respectively. Only the contours corresponding to the left bottom quarter of the specimen, below the dowel hole, are presented due to symmetry. It is evident from Figure 4.15, which presents the results for loading parallel to the grain, that as the embedment progresses, the area with largest relative displacement (shown in purple) also increases. Importantly, beyond 1.5 mm of embedment deformation, noticeable accumulation of localized damage around the dowel was evident during the experiment.

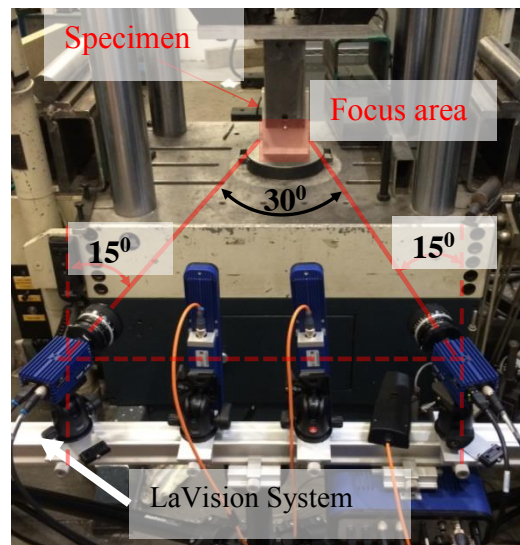


Figure 4.14 – Experimental set-up with the DIC system. Embedment tests on a 12 mm dowel specimen.

Similarly, Figure 4.16 shows the force-embedment curve together with the relative displacement contours at four key behavioural stages for a perpendicular-to-grain test that employs a 12 mm dowel. In this figure, the area of largest concentration of deformations (purple) decreases in size as the embedment increases while a more pronounced horizontal orientation of contours relative to Figure 4.15 is evident.

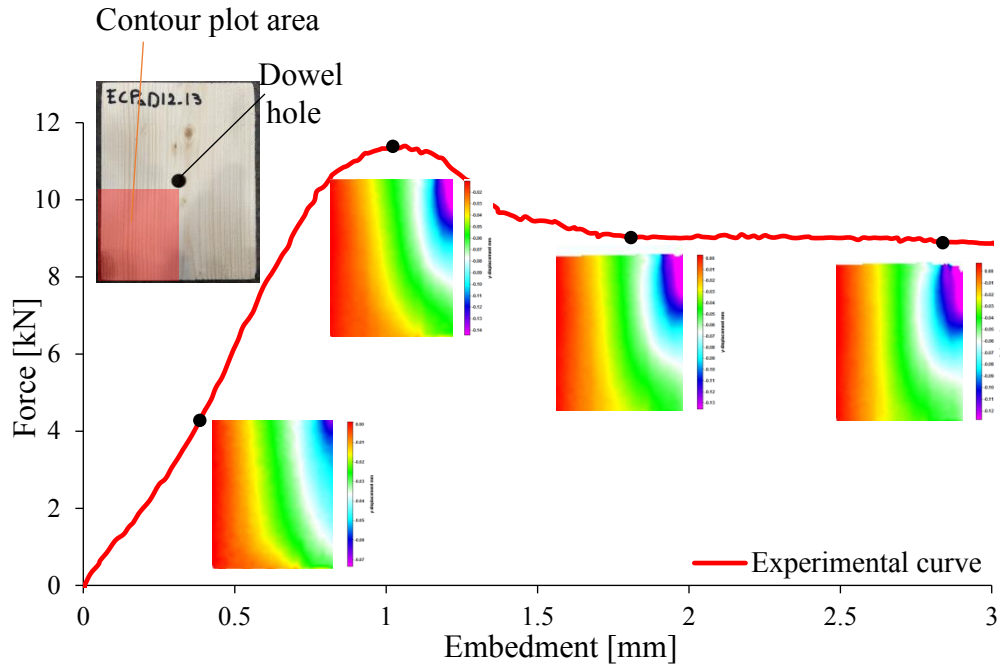


Figure 4.15 – Force-embedment curve from the embedment test parallel to the grain ($d = 12$ mm).

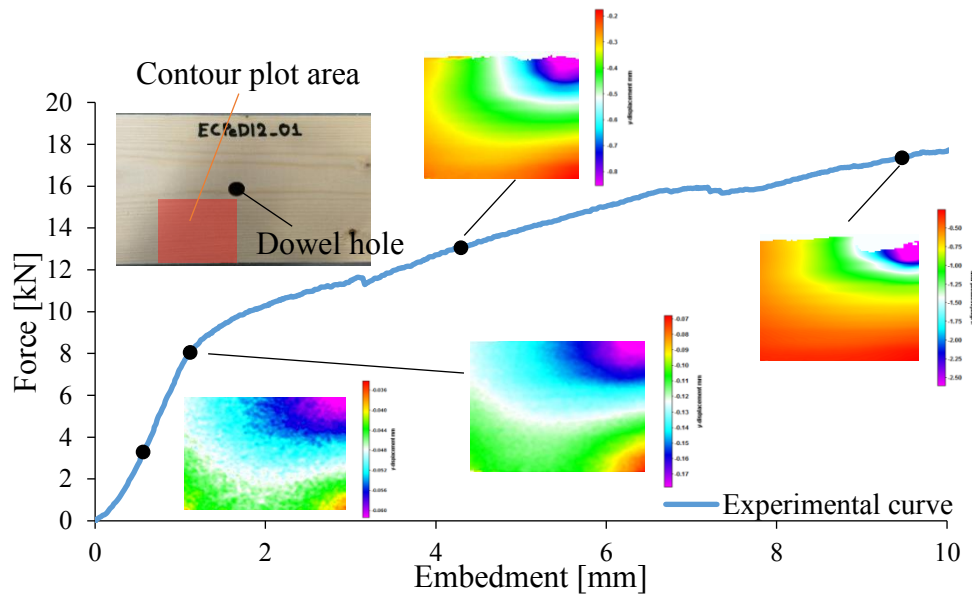


Figure 4.16 – Force-embedment curve from the embedment test perpendicular to the grain ($d = 12$ mm).

These differences in the evolution of areas of high deformation between parallel-to-grain (Figure 4.15) and perpendicular-to-grain (Figure 4.16) specimens can be attributed to the different mechanisms of load resistance involved as schematically shown in Figure 4.17. In parallel-to-grain loading, local buckling of some of the vertical fibres in direct contact with the dowel occurs at large deformation levels. With increasing demands, the confining action of the surrounding vertical fibres will favour a further concentration of deformation in the already

buckled fibres directly below the dowel hence engaging more material underneath the dowel in the same direction of loading. On the other hand, when the action is applied perpendicular to the direction of the grain, the displacement of wood material tends to localize on the upper fibres with the lower fibres acting as an elastic foundation. This behaviour will encourage the concentration of non-linear response on the upper fibres once they have yielded while spreading the deformation field horizontally as observed in Figures 4.11 and 4.16.

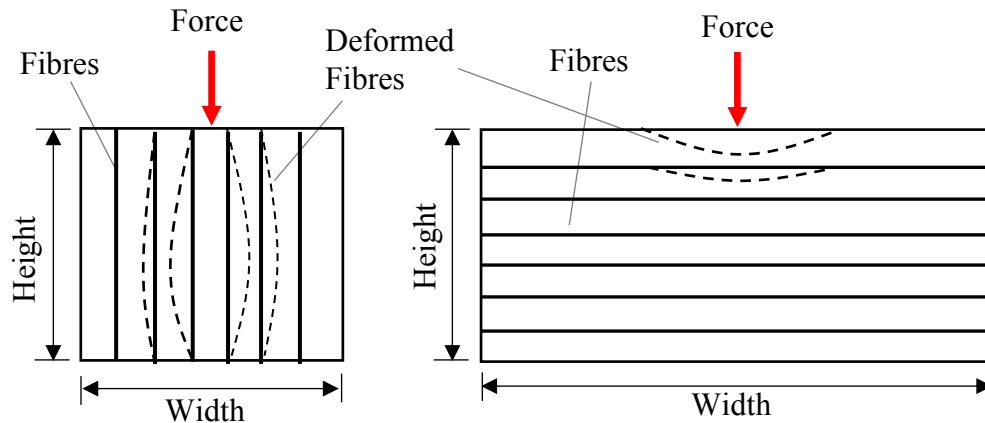


Figure 4.17 – Interaction between the embedment force and the fibre orientation.

These differences in strain localization brought about by the changes in fibre orientation with respect to the direction of load application can be further studied with reference to Figures 4.18 and 4.19 which present the y-displacement contours at maximum embedment force for parallel-to-grain and perpendicular-to-grain specimens, respectively. It can be seen from these figures that the area under analysis is located 3 mm below the hole in the left-side of the YZ-symmetry plane. Similarly, it can be seen from the contour plots of Figures 4.18 and 4.19, that the vertical distance along which the greater concentration of displacements is located is approximately 20 mm when the maximum load is applied parallel to the grain direction (Figure 4.18) while it is half of this (10 mm) when the maximum load is applied perpendicular to the grain direction. The effects of grain orientation on the main directionality of the strain field are also evident from Figures 4.18 and 4.19 with the contours tending to follow the grain orientation over the whole strain field computed. Finally, it is worth noting that the maximum absolute values of y-displacement in Figures 4.18 and 4.19 are 0.15 and 0.18 mm, respectively. This indicates that more than 80 % of the total embedment displacement (~ 0.85 mm for Figure 4.18 and ~ 0.84 mm for Figure 4.19) is concentrated within the area in close proximity to the hole (e.g. within 3 mm from the dowel surface in the case of parallel to grain loading as presented in Figure 4.18).

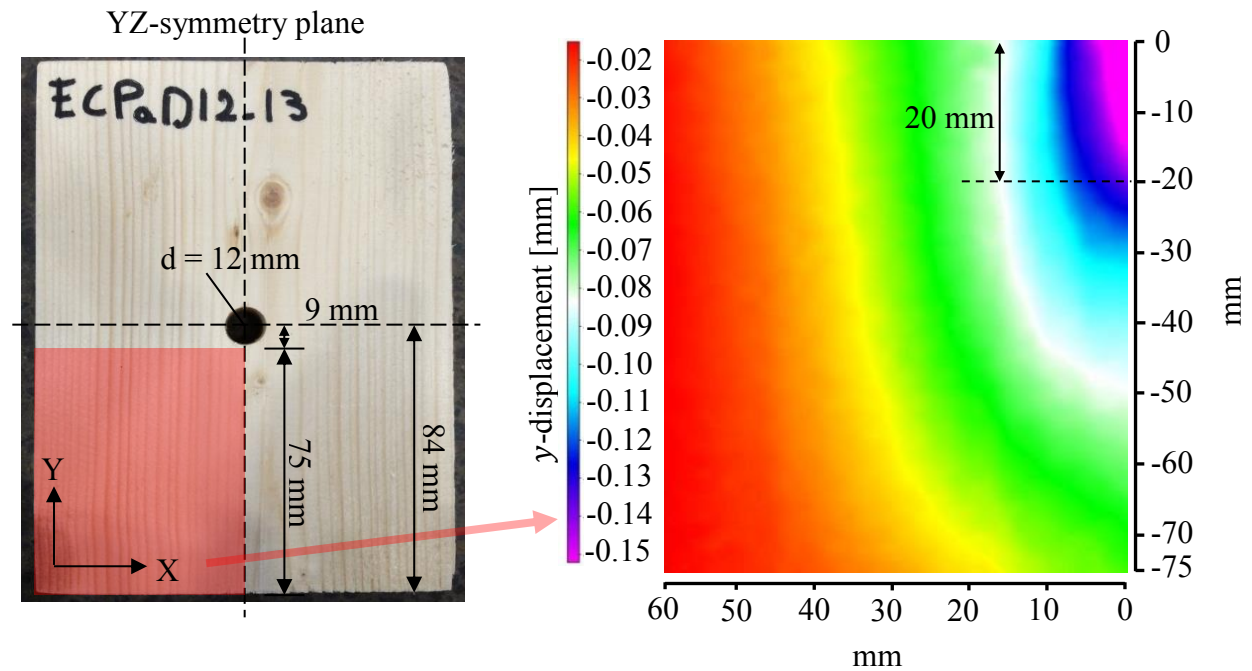


Figure 4.18 – Displacement contours of one-dowel connection test, parallel to grain with a 12 mm dowel at maximum embedment force (~1.0 mm of vertical displacement).

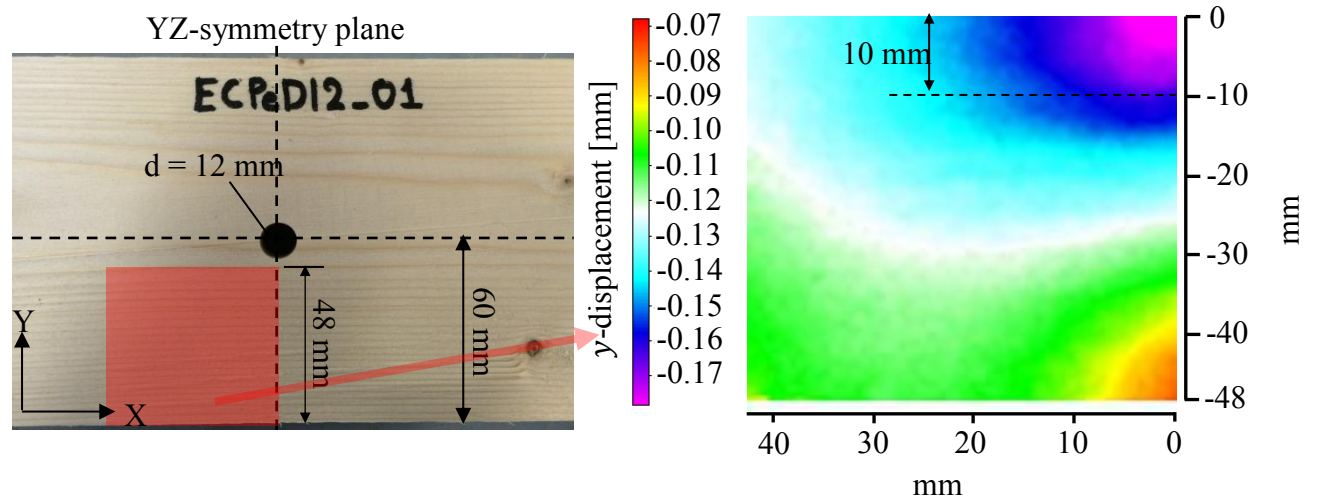


Figure 4.19 – Displacement contours of one-dowel connection test, perpendicular to grain with a 12 mm dowel at the yield point (~1.02 mm of vertical displacement).

4.4 Concluding remarks

This chapter has studied the embedment behaviour of single-dowel connections in Scandinavian Spruce with loads acting parallel and perpendicular to the grain direction. An experimental programme comprising fifty-two (52) connection details with 12 mm and 16 mm dowels was described. Force-embedment curves were obtained to study the embedment properties for this type of timber. The tests enabled a direct assessment of the influence of dowel diameter and load application with respect to the direction of grain on the behaviour of the timber material. The results also provide necessary information for the validation and calibration of the finite element (FE) models described in Chapter 5.

Three behavioural stages exhibited by the one-dowel connections were identified in the tests described above. An initial stage where the contact area between dowel and timber increases progressively followed by the elastic stage up to the embedment strength of wood is reached followed by the post-elastic stage. The tests showed that a post-elastic hardening behaviour is observed for embedment perpendicular to the grain, due to the densification of the wood fibres underneath the dowel. This behaviour is not evident where the load is applied parallel to the direction of the grain. Additionally, when the loads were applied in the direction parallel to the grain the specimen reached higher levels of embedment strength (100% increase for specimens with dowel diameter 12 mm and 102% for specimens with dowel diameter 16mm) and stiffness strength (175% increase for specimens with dowel diameter 12 mm and 225% for specimens with dowel diameter 16mm) at the expense of some reduction in ductility. The dowel diameter did not have any effect on the general shape and distribution of the behavioural stages.

From the results presented in Tables 4.2, 4.3 and 4.4 it can be concluded that the dowel diameter affects primarily the stiffness of the dowel-timber connection. A stiffer response for larger dowels can be observed from the test results. An observation that appears contrary to previously reported data [75, 87] but which is consistent with more recent experimental outcomes [88]. Additionally, the tests showed that there are small differences between the embedment tests parallel to the grain under the effects of compression and tension. Those changes can be attributed to the different size specimens as well as the presence of defects. From the results presented in Tables 4.2, 4.3 and 4.4 it can be concluded that a stiffer response for larger dowel specimens can be observed from the test results. The differences between the stiffness of the two tests series, relates to differences of the specimen size and not to the size of the dowel.

Similar, tests from the literature on single-dowel embedment tests [70,71,89] indicate that timber specimens having the same dimensions but unequal dowel diameter exhibits similar initial stiffness, an observation that is reported in [75,87]. Size effects play an important role on both strength and stiffness for timber specimens [90,91]. Additionally, the tests showed that there are small differences between the embedment tests parallel to the grain under the effects of compression and tension. Those changes can be attributed to the different size specimens as well as the presence of defects.

In terms of failure modes, it was shown that the primary nature of failure parallel to the direction of the grain was crushing of the timber underneath the dowel followed by a symmetric or sometimes non-symmetric splitting of the specimen throughout its whole height. This phenomenon is caused by the generation of local stresses which overcome the strength of the fibres laying as tangents to the dowel hole [71]. Importantly, the non-symmetric splitting was influenced by the presence of knots. On the other hand, for the embedment tests perpendicular to the grain, the main failure mechanism was crushing of the timber fibres beneath the dowel. To further examine those effects, a detailed monitoring of the evolution of local strain concentrations around the fastener at increasing levels of bearing deformation by means of Digital Image Correlation (DIC) techniques was presented. This served to examine the tendency of the strain contours to align with the direction of the grain. Also, the differences in load bearing mechanisms brought about by the different grain orientations were found to influence the degree to which strain concentrations are able to penetrate in the wood material beneath the dowel area. To this end, significant increments in the highly strained area for parallel-to-grain specimens were observed in comparison with perpendicular-to-grain joints where a tendency towards an increasingly more localized concentration of the area of highest strains was documented. Overall, the experimental findings of this chapter offer vital information on the localisation effects around the dowel diameter, which is useful for the formulation of the numerical models presented in the following chapters.

Chapter 5

Modified foundation FE modelling of embedment in timber

5.1 Introduction

As summarized in the literature review presented in Chapter 2, a number of sophisticated numerical modelling methods have been proposed to model the embedment behaviour of steel dowels. Such models have generally required the implementation of advanced failure criteria which numerical implementation is highly complex. Moreover, the difficulties associated with the definition of interactive failure modes and localized damage material models further complicate their adoptions. An alternative approach, also identified in Chapter 2, relies on the definition of an equivalent cylindrical area around the dowel with equivalent material properties initially proposed by Fochi [28] and known as “foundation model”. This chapter revisits and extends the foundation model for the simulation of embedment in Scandinavian Spruce laminated wood. A new set of relationships are proposed for the estimation of equivalent material characteristics such as strength and stiffness as a function of the crushing volume. The proposed improved foundation model is then validated against the results presented in the previous chapter and is proved to produce reasonable response estimates.

5.2 Finite element model

To be as close as possible to the physical problem, the models presented in this chapter were created in a three-dimensional (3D) space. Typical one-dowel connection models, parallel and perpendicular to the grain directions, are illustrated in Figure 5.1. Linear and quadratic solid brick elements were initially considered but it was found that both gave almost identical results despite the significantly longer time to complete observed in quadratic elements. Therefore, eight-node solid brick elements of Type C3D8 [7] were finally employed. Similarly, symmetry displacement conditions/restraints were applied whenever possible to increase the modelling efficiency. The nodes located at the symmetry planes were restricted to move in the x-direction ($u_x = 0$) whereas the nodes at the bottom of the models were constrained in the y-direction ($u_y = 0$). Additionally, the nodes at the sides of the dowel models were displaced in

the negative y-direction in the simulation the embedment tests. The material model for the FE timber model was determined from the experimental tests presented in Chapter 3 and their values of each mechanical property is presented in Table 5.1 below.

Timber properties	
E_1	2050.8 MPa
$E_2 = E_3$	172.1 MPa
ν_{12}	0.372
ν_{13}	0.467
ν_{23}	0.435
$G_{12} = G_{13}$	951.8 MPa
G_{23}	95.2 MPa
σ_{11}	33.0 MPa
σ_{22}	3.6 MPa
σ_{33}	3.60 MPa
$\sigma_{12} = \sigma_{13}$	5.1 MPa
σ_{23}	0.5 MPa

Table 5.1 – Mechanical properties for the timber model.

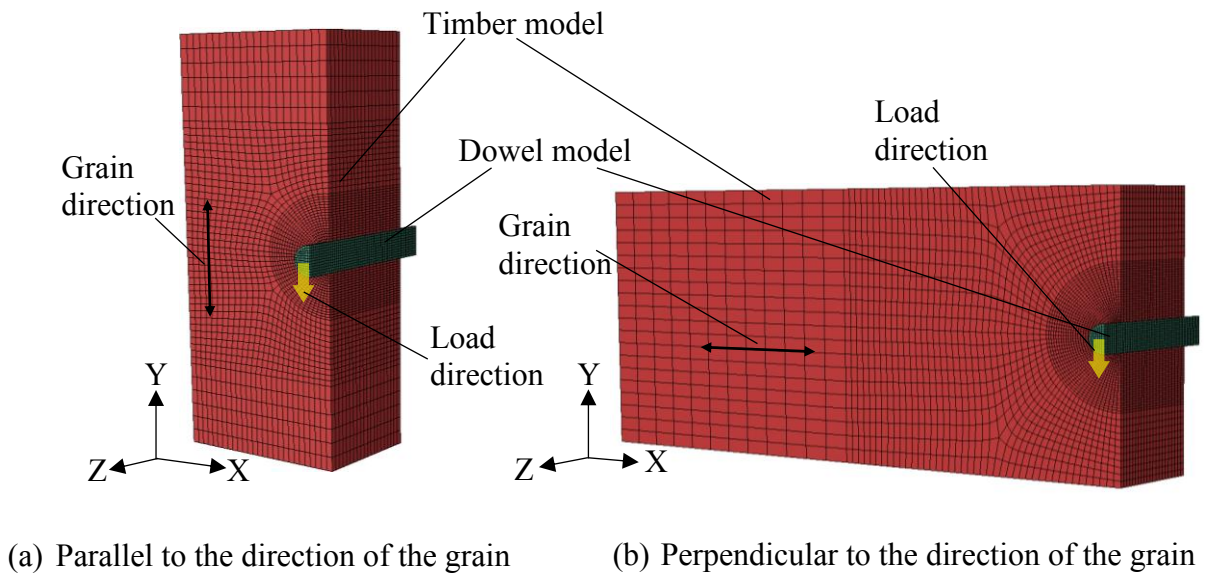


Figure 5.1 – FE symmetry Models of one-dowel timber connection.

A mesh optimisation sensitivity study was carried out in order to choose the optimum element number and size while minimising the errors due to crude meshing. Figure 5.2, Shows the results in terms of average reaction forces at the base of the FE model (represented in Figure 5.1) against total number of elements. The adopted mesh arrangement involved a coarser mesh within the main body of the wood sample, whereas a finer mesh was employed around the dowel hole. The dimensions of the finally adopted mesh ranged between 1 mm and 5 mm, which resulted in approximately 32,000 elements.

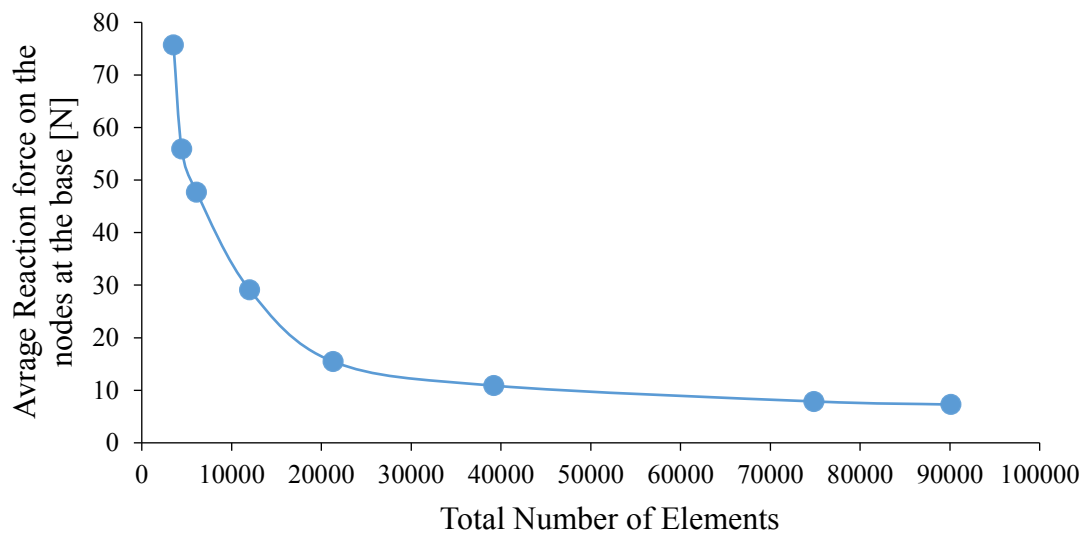


Figure 5.2 – Mesh optimization curve.

The contact interaction between steel and wood was modelled by utilising the contact pair option of a “master” and “slave” surfaces in ABAQUS [7]. To this end, the master surface was chosen to be the stiffer steel dowel surface whereas the relatively more flexible wood surface was defined as “slave”. In this FE package, contacts remain normal to the “master” surface at the smallest distances from the nodes. The contact interaction between the timber and the steel dowel proved to be prone to convergence difficulties. For situations like that, convergence performance can be enhanced by allowing limited penetration between the slave and master surfaces through the HCRIT parameter in the CONTACT PAIR ABAQUS command. This strategy was adopted in the present study with an absolute interpenetration of 0.1 mm.

Another important option which helps on the convergence of the solution, is the hard contact over-closure pressure relationship of ABAQUS [7]. This option was modified by adopting a linear relationship with significantly stiff response. Friction between the contact surfaces was introduced via the relevant command with a finite sliding formulation. As proposed by Smith [92] the friction coefficient between steel and wood surfaces adopted was 0.7.

The material model employed for the dowels is steel with a tri-linear kinematic hardening representation an elastic modulus of 210 GPa and a Poisson's ratio of 0.3. The stress-strain relationship for this type of steel, incorporates three stages. Initially, there is an elastic stage, which is followed by a constant stress plateau up to a strain level of approximately 0.01, and finally a 1 % strain-hardening stage. It is worth mentioning, that during the experimental testing, all steel dowels remained elastic and this was also reflected in the numerical simulations. On the other hand, timber was modelled as an orthotropic elasto-plastic material, implementing anisotropic yield criterion using Hill's plasticity model [7]. Some of the values used to define the material properties are presented in Table 3.7.

Figure 5.3 shows the force-embedment deformation curve obtained by means of a FE model and its corresponding experimental results for a 12 mm diameter dowel, whereas Figure 5.4 shows the stress contours at 2 mm of embedment. The experimental results are represented in Figure 5.3 by their corresponding upper and lower bound curves (or envelope) as obtained in Chapter 4. It is evident that there are significant differences between the predicted and the observed response, especially at yielding and in the post-elastic range. These differences are due to the localised nonlinear behaviour of timber. From Figures 5.3 and 5.4, it can be appreciated that the FE model with a homogeneous timber material idealisation overestimates the response of the tests up to 0.8 mm of embedment. At this point the wood elements that come into close contact with the steel dowel get significantly distorted leading to a marked softening behaviour. This suggests that the modelling of dowel type connections is not feasible through conventional Finite Element (FE) approaches by assuming homogenous material properties along the wooden volume, and points to the inability of such models to simulate the intricate bearing phenomena of the wood matrix. In this context, these intricacies in the mechanical behaviour of wood including damage accumulation can be approximated in the FE simulation by incorporating a region around the hole with empirically calibrated material properties, which is referred herein as the 'foundation zone' approximation [28].

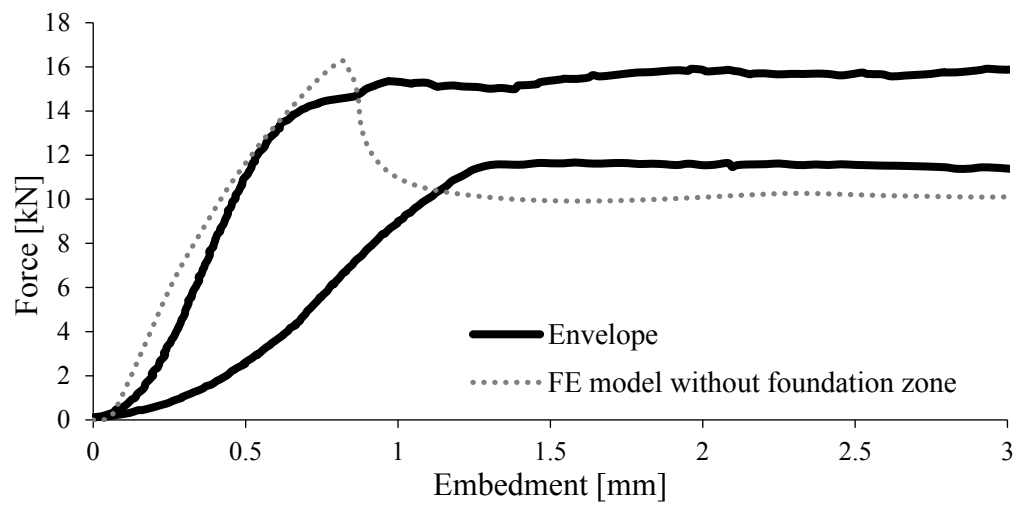


Figure 5.3 – Comparison between experimental embedment force-displacement response and FE simulation for parallel to grain direction.

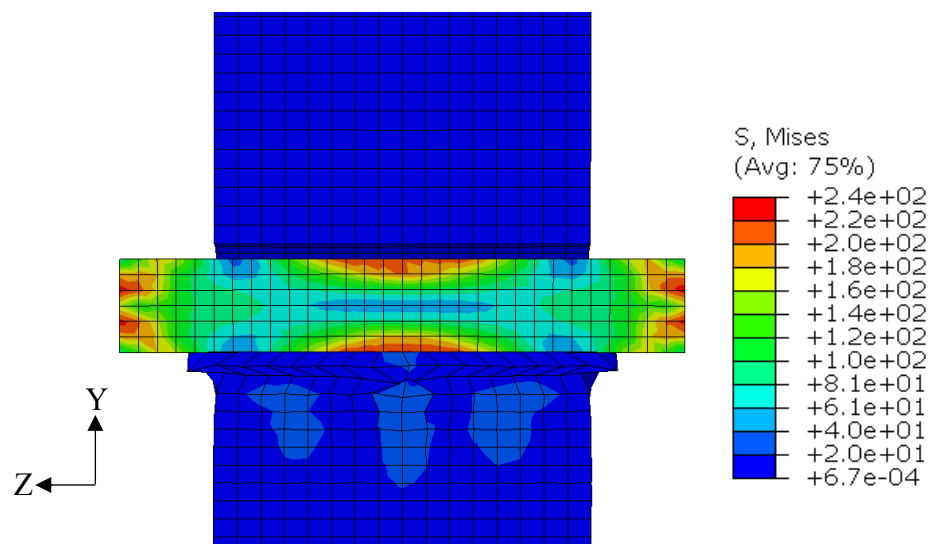


Figure 5.4 – Numerical response of one-dowel connection parallel to the grain without foundation zone at 2 mm of embedment.

5.3 Modified foundation zone

A simple yet effective way, to incorporate the effects of localized wood crushing into a FE simulation is via a foundation zone approach. This method was initially proposed by Foschi [28] for Douglas-fir (*Pseudotsuga menziesii*). Figure 5.5 below, shows the updated version of the timber FE model originally described in the previous section but after implementing the foundation model approach. The method hinges around the definition of a modified material model within a certain volume around the dowel hole. This volume is referred to as the foundation zone (presented in white in Figure 5.5) for which the material properties are determined via empirical calibration procedures. In its original application for 3D models [70], the foundation model assumed a cylindrical volume with a constant radius of 2.8 times the dowel diameter. This value was selected on the basis of experimental constraints related to the maximum size of the specimen tested [28] and no rational justification was offered. Furthermore, no provision was made for the use of the method when alternative geometric constraints may govern the definition of the foundation zone dimensions. In this chapter, the foundation material model is modified and extended for the case of Scandinavian Spruce Glulam. Also, a non-arbitrary procedure for the definition of material parameters within the foundation zone as a function of its volume are suggested and implemented.

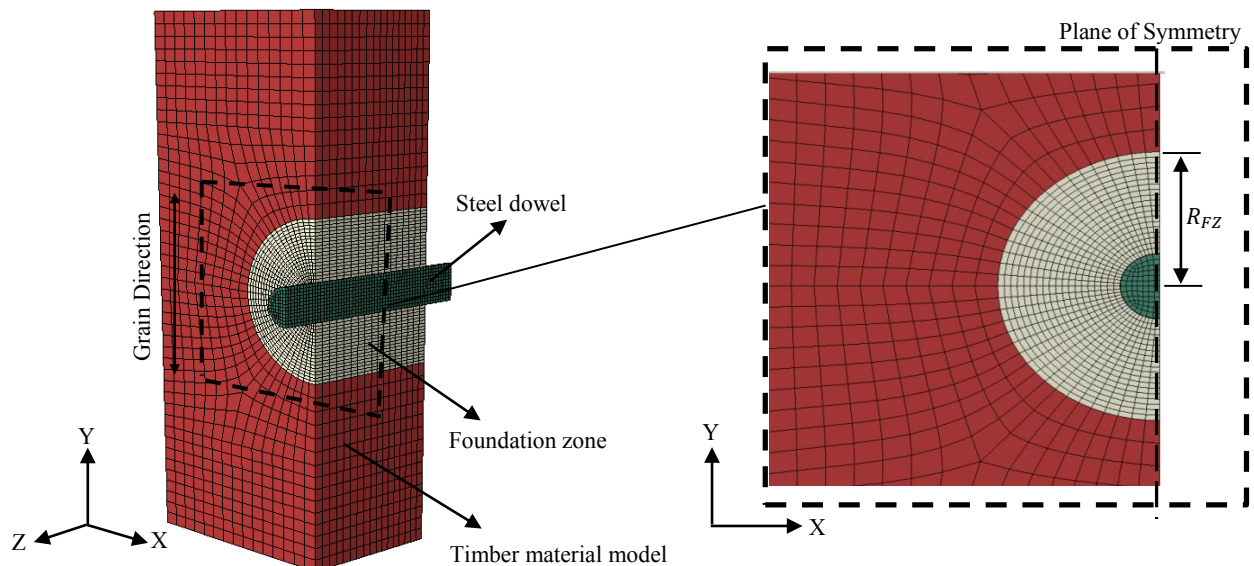


Figure 5.5 – Implementation of a single-dowel foundation FE model.

To successfully simulate the embedment behaviour of the timber, the material parameters adopted for the foundation zone were softer and weaker compared to the rest of the timber model in order to approximate the experimentally observed damage accumulation process. Table 5.2 presents the ratios between the material parameters as obtained from embedment tests (identified by the subscript *Emb*) and those measured from direct compression tests (identified by the subscript *m*) the indices 1 to 3 refer to the three principal stress directions. These ratios were employed to define a basic set of material parameters for the foundation region.

	d = 12 mm	d = 16 mm
$E_{1,Emb}/E_{1,m}$	0.183	0.504
$E_{2,Emb}/E_{2,m} = E_{3,Emb}/E_{3,m}$	0.795	1.398
$G_{12,Emb}/G_{12,m} = G_{13,Emb}/G_{13,m}$	0.122	0.122
$G_{23,Emb}/G_{23,m}$	0.0122	0.0122
$\sigma_{11,Emb}/\sigma_{11,m}$	0.743	0.746
$\sigma_{22,Emb}/\sigma_{22,m} = \sigma_{33,Emb}/\sigma_{33,m}$	5.795	5.748

Table 5.2 – Material properties for the foundation modelling approach.

In the case of Poisson ratios and shear strengths, nominal values were assumed regardless of the foundation zone distinction. Similarly, the values for the shear modulus G introduced into the foundation zone were estimated from the empirical formula given in the annex of EN 338:2009 [82] as:

$$G_{12} = \frac{E_1}{16} \quad (5.1)$$

As discussed before, the original foundation zone approach for 3D models proposed by Hong [70] assumes a constant radius $R_{FZ} = 1.4d$ for the foundation zone thus disregarding any variation in the definition of a weakened volume and its effects on the corresponding weakened material characteristics. This can limit the applicability of the method when the fastener to be modelled is part of a group or a different geometrical arrangement of the connection is to be simulated. In order to overcome those limitations, the radius of the foundation zone (R_{FZ}) is expressed herein in terms of the dowel diameter (d) by means of a ratio P such that:

$$R_{FZ} = P \cdot d \quad (5.2)$$

Twelve models of one-dowel connection parallel and perpendicular to the grain were developed, each having a different value of P and keeping the dowel diameter constant. The results of these simulations are depicted in Figures 5.6 and 5.7. It can be evaluated from these figures that the volume of the foundation zone, affects the response of the tests, both in the elastic and post-elastic regions. Hence, there is a proportionality between the material properties and size of the foundation zone. This difficulty was overcome by introducing parameters that limited that effect as explained below.

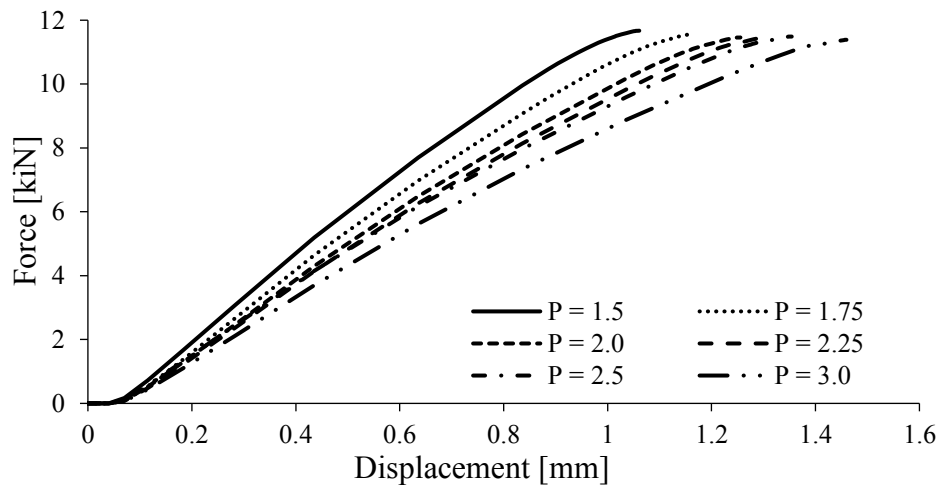


Figure 5.6 – Effect of foundation zone size in one-dowel tests parallel to the grain.

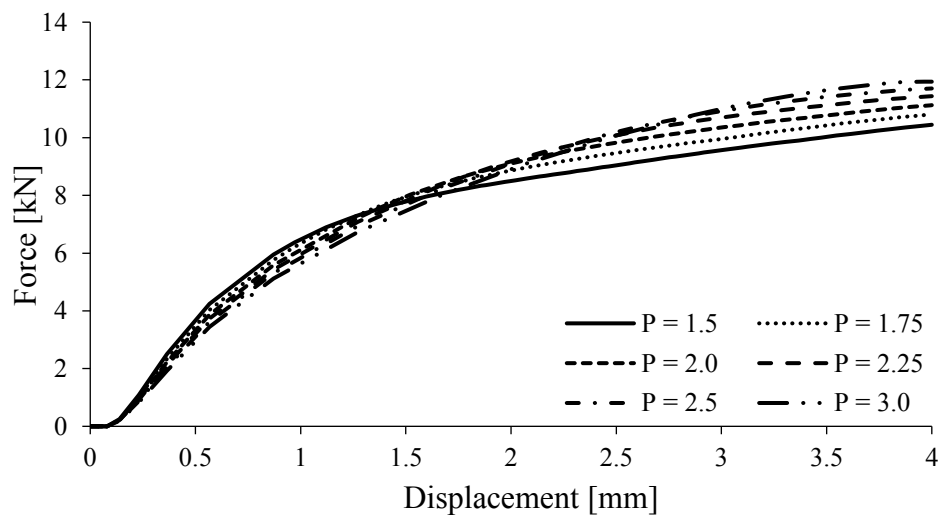


Figure 5.7 – Effect of foundation zone size in one-dowel tests perpendicular to the grain.

By conducting several analyses with different values of P under both parallel and perpendicular to the grain directions, the following expressions can be proposed for the determination of

effective Elastic Modulus within the foundation zone in the directions parallel ($E_{1,FZ}$) and perpendicular ($E_{2,FZ}$) to the grain direction, respectively:

$$E_{1,FZ} = \left(1 + \beta_1 \frac{P}{t}\right) E_{1,Emb} \quad (5.3)$$

$$E_{2,FZ} = \left(1 + \beta_2 \frac{P}{t}\right) E_{2,Emb} \quad (5.4)$$

where, t is the thickness of the specimen in [mm], E_{Emb} is the Young's modulus determined from the embedment tests in [MPa], E_{FZ} is Young's modulus of the foundation zone in [MPa] and β_1 and β_2 are empirical parameters for which optimal values are reported in Table 5.3.

Parameter	d = 12 mm	d = 16 mm
β_1	6.27	2.28
β_2	5.51	3.13

Table 5.3 – Values for parameters β_1 and β_2 as a function of the dowel diameter.

Similarly, the following expressions are proposed for the estimation of yield stresses in the directions parallel ($\sigma_{11,FZ}$) and perpendicular ($\sigma_{22,FZ}$) to the grain direction:

$$\sigma_{11,FZ} = \left(1 - \gamma_1 \frac{P}{d \cdot t}\right) \sigma_{11,Emb} \quad (5.5)$$

$$\sigma_{22,FZ} = \left(1 - \gamma_2 \frac{P}{d \cdot t}\right) \sigma_{22,Emb} \quad (5.6)$$

where, t is the thickness of the specimen in [mm], d is the diameter of the dowel in [mm], σ_{Emb} is the stress determined from the embedment tests in [MPa], σ_{FZ} is the stress of the foundation zone in [MPa] and γ_1 and γ_2 are parameters for which values can be found in Table 5.4 for 12 mm and 16 mm dowels.

Parameter	d = 12 mm	d = 16 mm
γ_1	14.67	14.60
γ_2	21.57	21.74

Table 5.4 – Values for parameters γ_1 and γ_2 as a function of the dowel diameter.

Finally, Figure 5.8, shows the stress contours at 2 mm of embedment for a 12 mm diameter single-dowel connection test with foundation zone. The elements that come in contact with the dowel are less distorted in comparison with the model without foundation zone shown previously in Figure 5.4 above. This is another indication that the implementation of the foundation zone is of paramount importance when simulating timber connection tests.

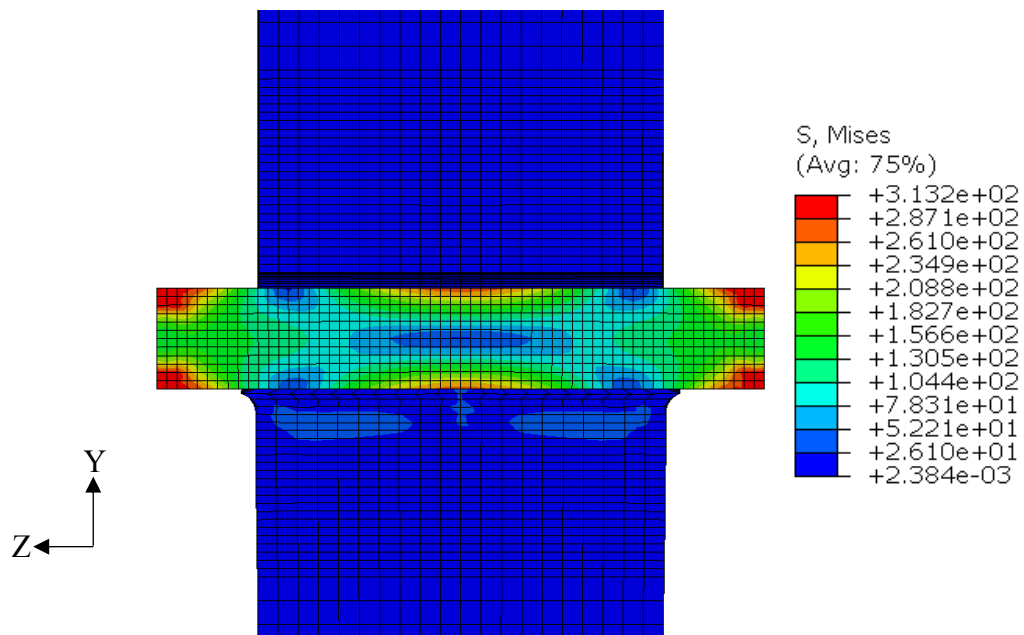


Figure 5.8 – Numerical response of one-dowel connection parallel to the grain with foundation zone at 2 mm of embedment.

5.4 Comparison with test results

The modified foundation model proposed in the previous section was validated against the results of the laboratory tests described in Chapter 4. Figure 5.9 presents the comparisons between the force-displacement envelopes obtained from the experimental results and those predicted by means of the modified foundation model when a 12 mm dowel is employed, while Figure 5.10 depicts the same comparisons for the case of 16 mm dowels. The experimental envelope was obtained by plotting the observed maximum and minimum forces along the full embedment range while the numerical results were obtained from the modified foundation model described above assuming P ratios in the range of 1.5 to 3. Figures 5.9 and 5.10 demonstrate that the estimations of the modified foundation model are in close agreement with the experimental results. The model is able to reproduce key response parameters such as

stiffness and embedment strength well within the bounds of the experimentally observed values in all cases. In particular, the experimental force-displacement relationships of parallel to grain specimens (Figures 5.9a and 5.10a) is well simulated up to displacements of 4 mm (i.e. strain levels of around 8 %). However, in the case of perpendicular to grain response, the foundation model produces noticeable differences in the post-elastic range with appreciable strain hardening in the numerical estimations in lieu of the almost perfectly-plastic response observed in the experiments. This is attributed to the inability of the continuum FE model to reproduce localized splitting between fibres at very large local deformations as well as the associated softening effects. Nevertheless, the foundation model provides estimates within the experimentally observed bounds for embedment displacements of up to 2.5 mm in both cases (Figures 5.9b and 5.10b). A very important feature of the numerical responses presented in Figure 5.9 and 5.10 is their relatively low dependence on the radius of the foundation volume adopted (characterized here by the parameter P as introduced in Equation (5.2)). This provides confidence in the effectiveness of Equations (5.3) to (5.6) in liberating the numerical response from a unique value of the foundation zone radius making the model consistently applicable to a wider range of varying geometrical configurations.

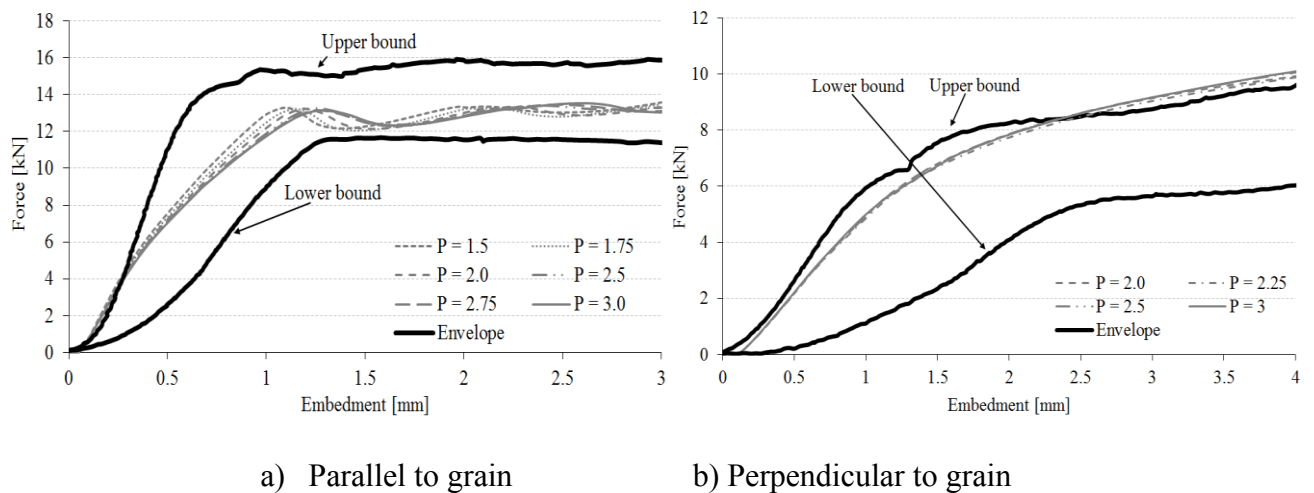


Figure 5.9 – Comparison of experimental and numerical force-embedment envelope relationships ($d = 12$ mm).

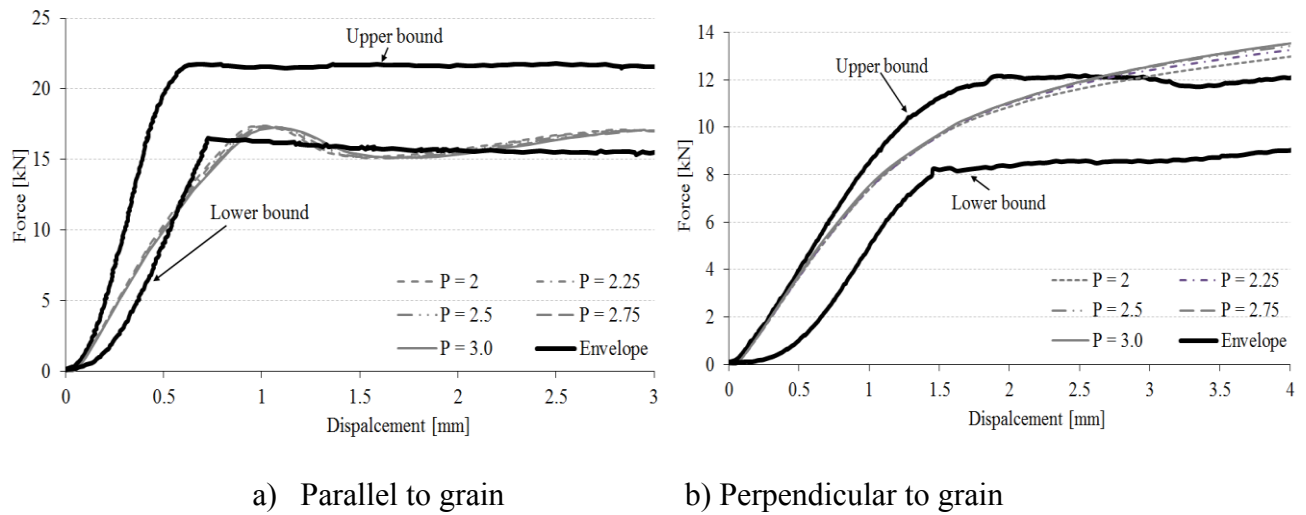


Figure 5.10 – Comparison of experimental and numerical force-embedment envelope relationships ($d = 16$ mm).

Finally, Figures 5.11 and 5.12 compare the experimentally observed displacement contours obtained by means of the DIC against the numerical predictions of the modified foundation model at yield for two representative parallel-to-grain and perpendicular-to-grain specimens, respectively. It is evident from these figures that the proposed foundation model is able to replicate the general tendencies in magnitude and distribution of the deformations within the specimens. A concentration of relatively higher localized strains immediately below the dowel are predicted by the FE model which can be attributed to the numerical definition of hard contact between timber and steel and the averaged nature of the material properties employed for the foundation model. Nevertheless, a good correlation of mean displacement orientations is apparent from Figures 5.11 and 5.12 including the effects of grain direction on the predominant contour alignment as explained in Chapter 4 with the use of Figure 4.17.

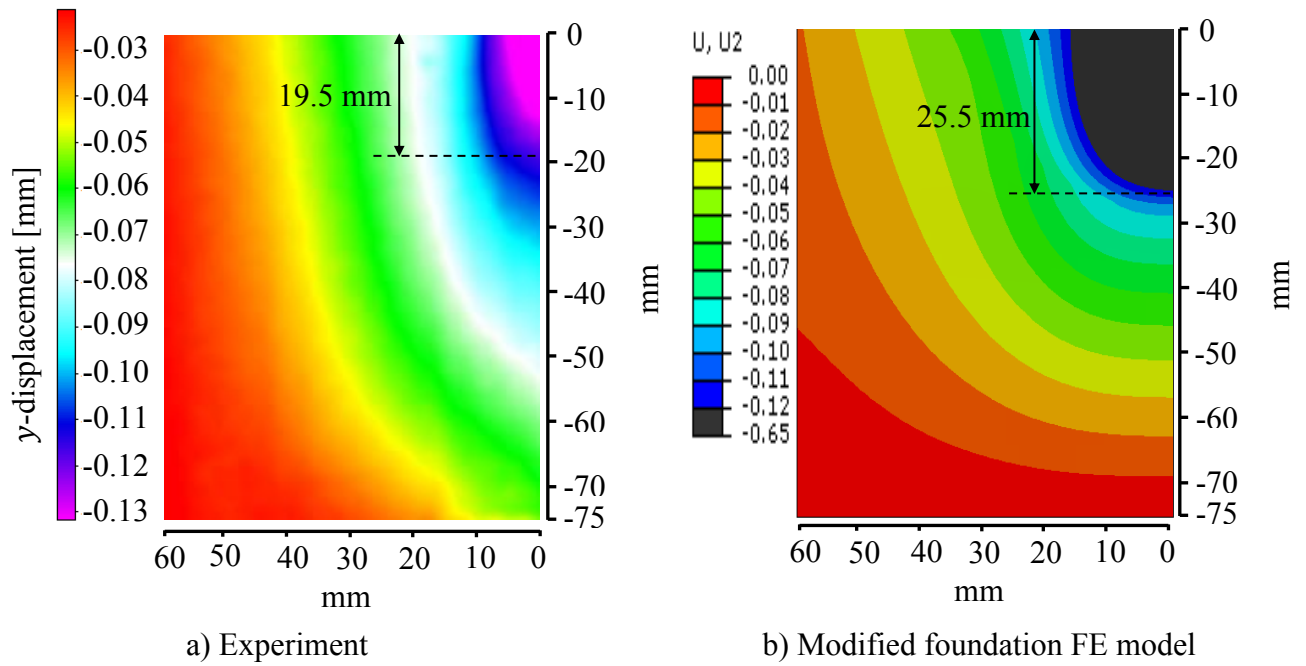


Figure 5.11 – Comparison of experimental and numerical displacement contours at yield.
Embedment parallel to grain ($d = 12$ mm).

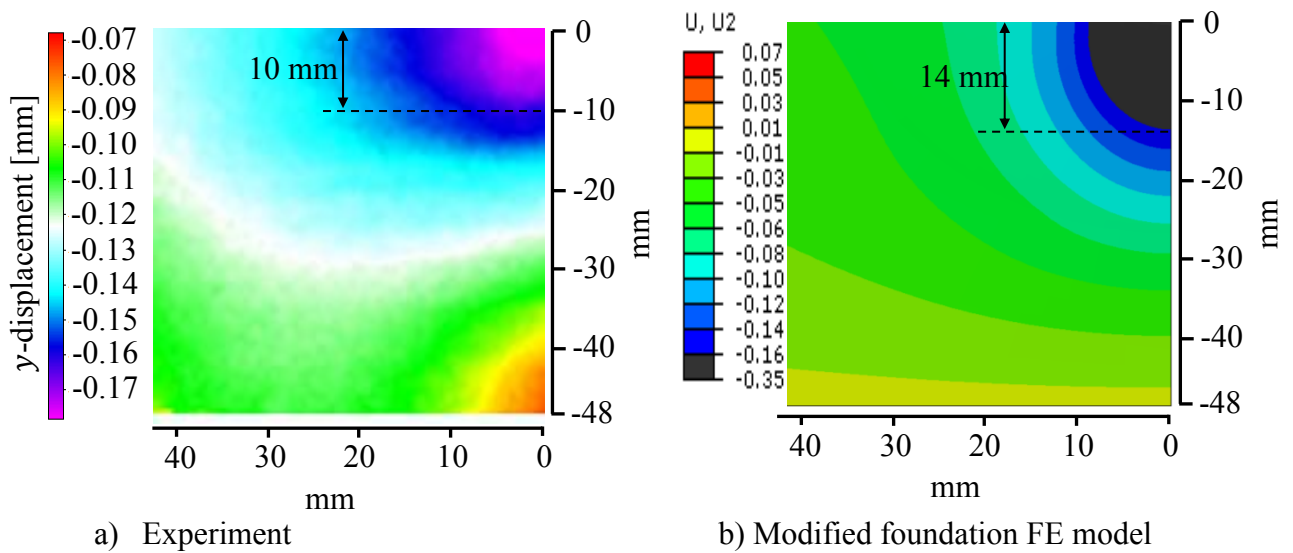


Figure 5.12 – Comparison of experimental and numerical displacement contours at yield.
Embedment perpendicular to grain ($d = 12$ mm).

5.5 Concluding remarks

This chapter described an alternative FE modelling approach to simulate one-dowel connection tests known as foundation model. The model which was initially conceived for Douglas-fir (*Pseudotsuga menziesii*) timber has been improved and adopted in this thesis to model the embedment response of Scandinavian Spruce Glulam (*Picea abies*). Single-dowel embedment tests for *Picea abies* timber specimens of C30 strength class has not been previously conducted. Previous foundation zone methods proposed by Foschi [28] and Hong [70] limited the foundation zone to have a radius of 2.8 times the dowel diameter. This fixed value of foundation radius restricted the size of specimens and, in the case of multi-dowel timber-steel connection, this restriction hindered the applicability of the model to connections with large spacing between the fasteners. From the curves presented in Figures 5.6 and 5.7 it is evident that the volume of the foundation zone affects the response of the single-dowel connections. This thesis has addressed these issues and extended the applicability of the foundation modelling approach by liberating it from the constraint on the size of the foundation zone radius.

The proposed new model is based on the definition of equivalent material parameters for the crushing region around the dowel hole. In order to improve the versatility of the FE simulations, new relationships for the estimation of material characteristics as a function of the crushing volume were suggested. These relations were based on the foundation zone idea but were obtained using a different approach to the one proposed by Foschi [28] and Hong [70]. The elimination of the foundation zone radius restriction allows the researcher to use this models in modeling of any dimension and also to select any fasteners spacing. The validity and accuracy of the proposed modified foundation models were examined against the experimental results, and good agreement was found between the experimental response and the numerical simulations. Overall, the results presented in this chapter suggest that the proposed modelling procedures can be employed in the assessment and design of single-dowel joints or as the basis for the formulation of alternative simplified approaches.

The work presented in this chapter constitutes a fundamental step towards a reliable and effective modelling approach of timber connections. It demonstrates the predictability, suitability and simplicity of the foundation zone approach for modelling timber-fastener interaction. The modified foundation modelling procedure, is employed for the development and validation of FE models for the timber beam-to-steel column hybrid connection discussed in Chapters 7 and 8.

Chapter 6

Experimental response of timber beam-to-tubular column connections

6.1 Introduction

The literature review presented in Chapter 2 has highlighted the lack of information and design guidance on practical semi-rigid connection details for joining glulam timber beams and tubular steel columns. More generally, hybrid solutions involving tubular steel columns and timber flooring systems has not been previously studied despite the numerous architectural and structural advantages these can present. This chapter aims to study the behaviour of practical and cost-effective semi-rigid hybrid glulam timber beam-to-steel tubular column connection alternatives. To this end, two details with the potential for a wider application in practice are suggested and examined: (i) top and seat bolted angle connections with long bolts, and (ii) slotted-in T-stub connections with bolts. In addition, two reinforcing techniques aimed at enhancing the response of the slotted-in T-stub configuration are investigated experimentally. These include the provision of a bottom wedge angle between the beam and the column as well as the use of perpendicular-to-grain screws to delay wood splitting. Subsequent sections of this chapter present the experimental set-up, connection configurations and material. This is followed by a detailed account of the test results and observations. The main behavioural patterns are identified from and the discussion focuses on issues related to the stiffness, capacity and failure mechanisms, and their implications on the performance and design of these connections.

6.2 Experimental arrangement and specimen details

6.2.1 Test set-up

Figure 6.1 shows the experimental set-up used for testing the glulam beam-to-steel column connections under bending. Previous research on semi-rigid connection to tubular columns [43] have shown that when the column length exceeds 1 m, it does not have a significant influence

on the connection response. Therefore, a 2-metre long steel SHS (Square Hollow Section) column was employed for all tests. The column was fixed to the strong floor at both ends by means of stressing bolts and plates as depicted in Figure 6.1. A double-hinged hydraulic actuator operating in displacement control was attached to the tip of the glulam beam through a specially designed clamping device. The displacement at the top of the beam was increased gradually at a rate of around 2 mm/min until the stroke of the actuator was reached (at 200 mm of top displacement) or until clear connection failure was observed. In order to eliminate the possibility of out-of-plane displacement of the beam, a frictionless roller-based lateral restraint was adopted as indicated in Figure 6.1.

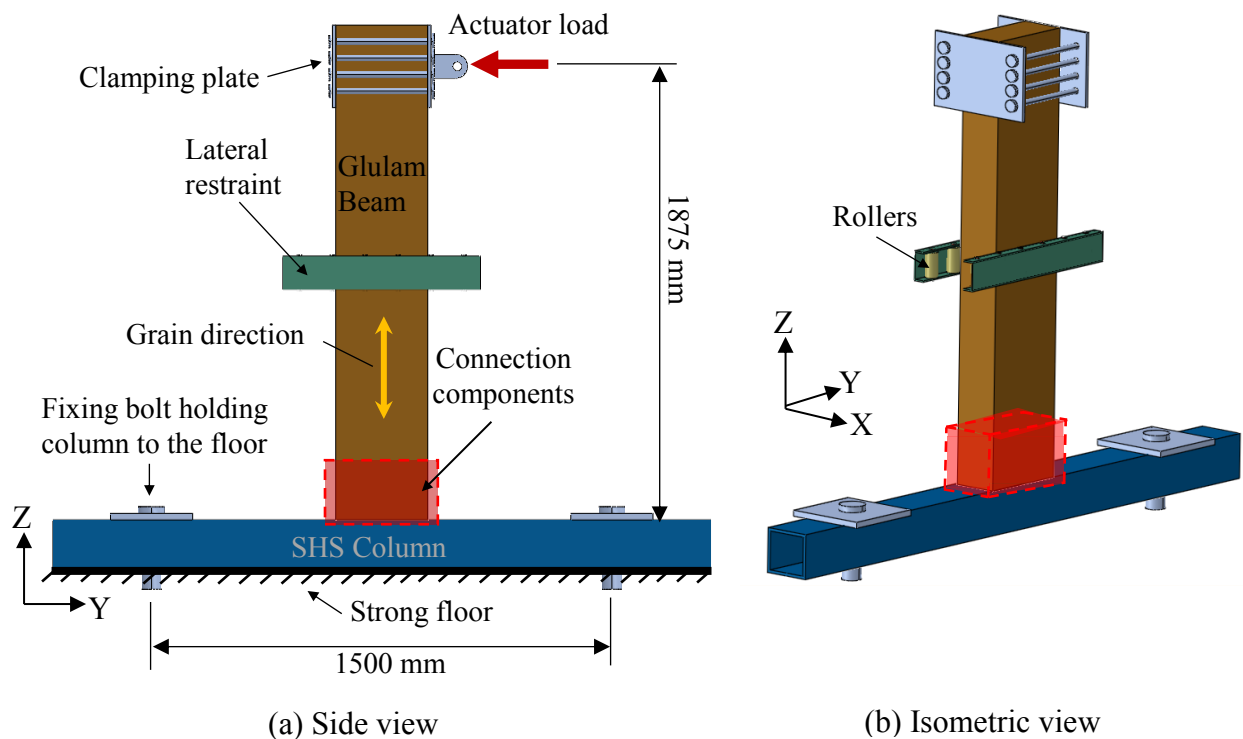


Figure 6.1 – Experimental set-up.

The displacement transducer and load cell incorporated within the actuator were employed to record the applied displacement and corresponding load at the tip of the beam, respectively. Besides, an inclinometer was attached to the beam to verify its angle of rotation. The fixed position of the column relative to the floor was continuously monitored by means of displacement transducers. A number of strain gauges were used in each specimen, and a digital image correlation (DIC) system was also employed to map the displacement field at the joint, as discussed in subsequent sections. A S375 Steel Square Hollow Section of 150x10 mm was used for the column in all cases while GL28h glulam beams with laminas of 45 mm were

utilised for the beams. Two timber beam sections (280x140 mm and 405x140) were employed depending on the connection type as described below.

6.2.2 Connection specimens

As noted before, two connection types are examined. The first (referred to as Type 1) incorporates top and seat angle brackets blind-bolted to the SHS column and tied to the glulam beam by means of long steel bolts running parallel to the beam depth. The second detail (referred to as Type 2) uses a steel T-stub, with its flanges welded to the SHS section, while its web is slotted into the timber beam. Twenty transversal bolts are used to attach the stub to the timber beam as depicted in Figure 6.2. Minimum distances and spacing codified criteria was respected in order to avoid premature brittle failure modes. Table 6.1 shows the minimum allowable distances for bolts in timber specimens adopted based on Table 8.4 of Eurocode 5 [29].

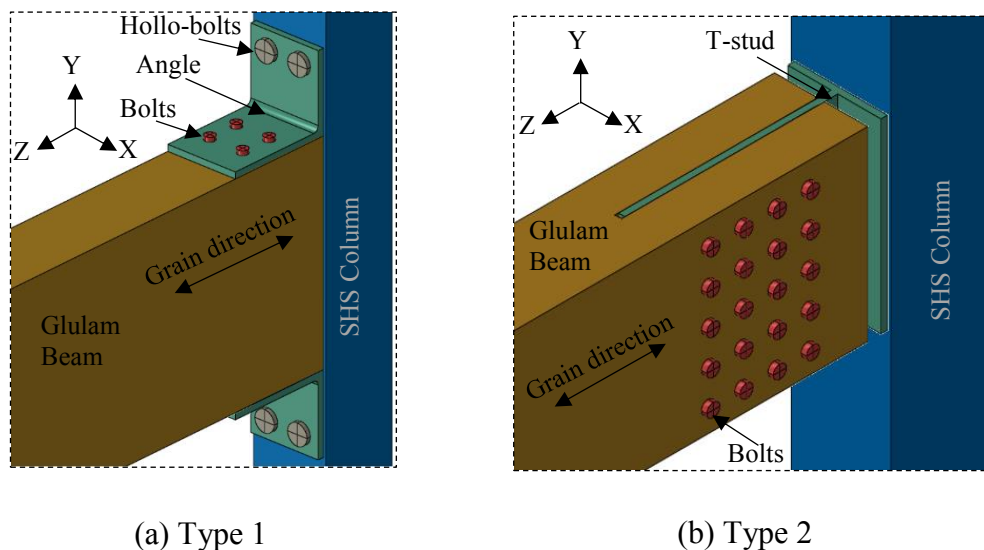


Figure 6.2 – Isometric view of connection details.

Spacing or distance	Angle	Minimum Spacing or distance
Spacing Parallel to the grain: a_2	$0^\circ \leq a < 360^\circ$	$(4 + \cos a)d_{0,SB}$
Spacing Perpendicular to the grain: a_1	$0^\circ \leq a < 360^\circ$	$4d_{0,SB}$
Loaded end distance: $b_{2,l}$	$-90^\circ \leq a < 90^\circ$	$\max(7d_{0,SB}; 80\text{mm})$
Unloaded end distance: $b_{2,u}$	$90^\circ \leq a < 150^\circ$	$\max((1 + 6\sin a)d_{0,SB}; 4d_{0,SB})$
	$150^\circ \leq a < 210^\circ$	$4d_{0,SB}$
	$210^\circ \leq a \leq 270^\circ$	$\max((1 + 6\sin a)d_{0,SB}; 4d_{0,SB})$
Loaded edge distance: $b_{1,l}$	$0^\circ \leq a \leq 180^\circ$	$\max((2 + 2\sin a)d_{0,SB}; 3d_{0,SB})$
Unloaded edge distance: $b_{1,u}$	$180^\circ \leq a \leq 360^\circ$	$3d_{0,SB}$

Table 6.1 – Minimum allowable distances and spacing for bolts for timber specimens.

In Table 6.1 $d_{0,SB}$ is the clearance of the hole, a_2 is the spacing between fasteners in a row in the direction parallel to grain, a_1 is the spacing between fasteners in one row perpendicular to grain, $b_{1,l}$ is the distance between fasteners and the loaded edge of the beam, $b_{1,u}$ is the distance between fasteners and the unloaded edge of the beam, $b_{2,l}$ is the distance between fasteners and the loaded end of the beam, $b_{2,u}$ is the distance between fasteners and the unloaded end of the beam and a is the angle between the applied force and the direction of the grain. Figure 6.3, shows a schematic representation of the fastener spacing as well as the end and edge distances nomenclature.

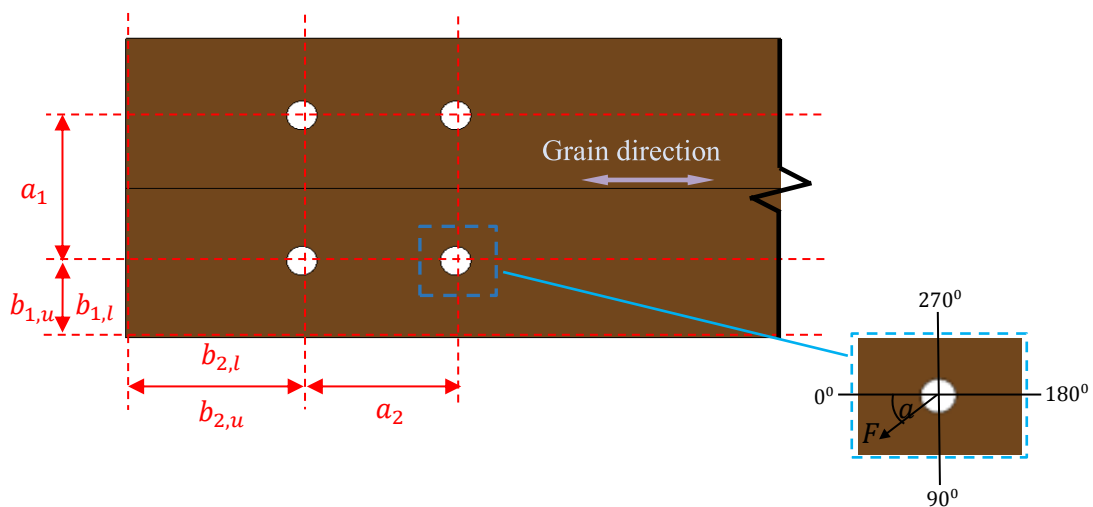


Figure 6.3 – Nomenclature of Fastener spacing and end and edge distances for timber specimens.

Given that the definition of these distances depends on the direction of loading and since both compression and tension regions are developed within the wooden beam in the connection, the most unfavourable loading configuration was assumed for their definition. Similarly, Table 6.2 shows the minimum and maximum allowable distances and spacing of bolts and Hollo-bolts for the metal plates, based on Eurocode 3 part 1.8 [34] while the nomenclature employed is shown in Figure 6.4.

Spacing or distance	Bolts and Dowels	Hollo-bolts	All
	Min	Min	Max
Spacing $p_{1,SB}$ & $p_{1,HB}$	$2.2d_{0,SB}$	$2.2d_{0,HB}$	$\min(14t; 200\text{mm})$
Spacing $p_{2,SB}$ & $p_{2,HB}$	$2.4d_{0,SB}$	$2.4d_{0,HB}$	$\min(14t; 200\text{mm})$
Edge distance $e_{1,SB}$ & $e_{1,HB}$	$1.2d_{0,SB}$	$1.2d_{0,HB}$	$40\text{mm} + 4t$
End distance $e_{2,SB}$ & $e_{2,HB}$	$1.2d_{0,SB}$	$1.2d_{0,HB}$	$40\text{mm} + 4t$

Table 6.2 – Min and Max allowable distances and spacing of bolts and Hollo-bolts for an angle flange [as shown in Figure 6.4].

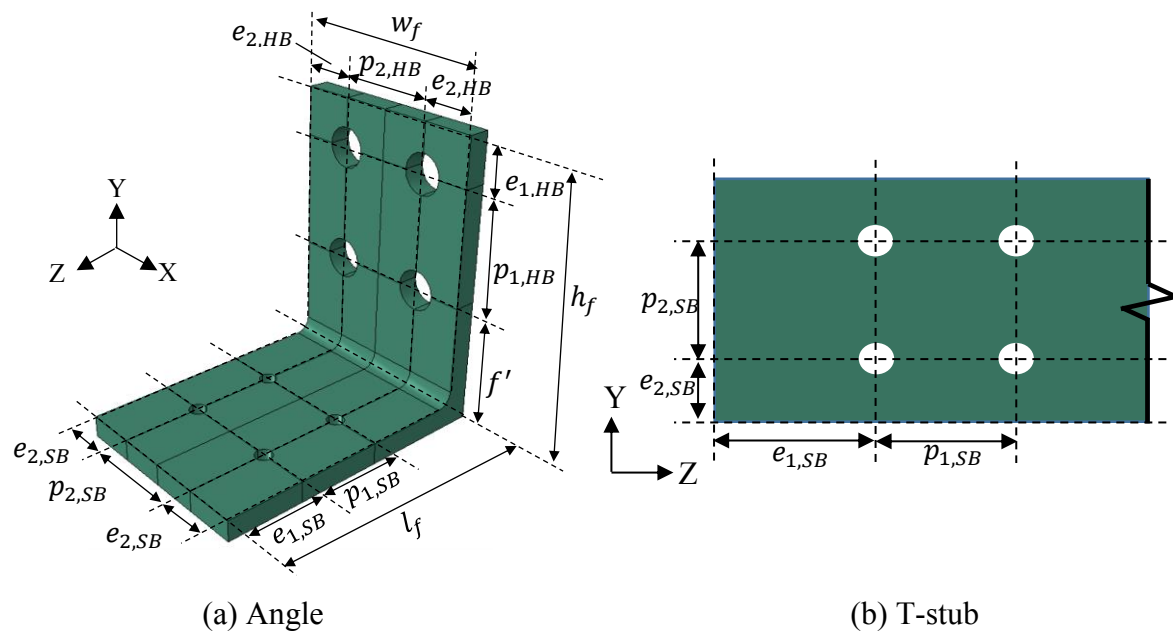


Figure 6.4 – Nomenclature of plain metal parts according to Eurocode 3 [34].

In Figure 6.4 and Table 6.2 w_f is the width of the angle flange, h_f is the height of the angle flange, l_f is the length of the angle flange, $d_{0,SB}$ is the clearance hole diameter for the bolts/dowels, $d_{0,HB}$ is the clearance hole diameter for the Hollo-bolts, $e_{1,SB}$ is the end distance of fasteners in the axis parallel to the length of the beam, $e_{2,SB}$ is the end distance of fasteners in a line perpendicular to the length of the beam, $p_{1,SB}$ is the spacing between fasteners in a

along the length of the beam, $p_{2,SB}$ is the spacing between fasteners in the axis perpendicular to the length of the beam, $e_{1,HB}$ is the end distance of Hollo-bolts in a line parallel to the height of the beam, $e_{2,HB}$ is the end distance of Hollo-bolt along the length of the beam, $p_{1,HB}$ is the spacing between fasteners in a line parallel to the height of the beam, $p_{2,HB}$ is the spacing between fasteners in a line perpendicular to the length of the beam and b_3 is the distance between the Hollo-bolts and the corner of the angle flange in the y -direction. Finally, it should be noted, that when designing an angle flange, it is common practice to have $e_2 \equiv e_{2,HB} = e_{2,SB}$ and $p_2 \equiv p_{2,HB} = p_{2,SB}$.

From the above discussion, it can be concluded that when designing hybrid timber-steel connections the distances and spacing of the bolts should satisfy the limits for both timber and steel materials. The process of determining appropriate spacing and distances for different angle, beam and fastener sizes, can be very tedious, therefore, two FORTRAN programs (one for each connection type) were formulated to speed up that process. The mathematical background and algorithms created can be found in Appendix C.

6.2.2.1 Top and seat bolted angle connections (Type 1)

In the case of Type 1 (top and seat angle) connections, L 200x150 mm angles made of S275 steel were blind-bolted to the SHS column by means of M16 Grade 10.9 Lindapter Hollo-bolts [44]. The horizontal (beam) legs of the top and seat angles were connected with each other and with the timber beam via four long bolts (12 mm diameter and Grade 10.9) passing through the beam depth as depicted in Figure 6.2a. Tightening torques of 190 Nm and 100 Nm were applied to the Hollo-bolts and to the long bolts, respectively. Given the availability of previous experimental data on the response of SHS faces in tension, compression and bending [6, 93], as well as previous experimental research on the behaviour of blind-bolt/angle assemblages [4, 43, 94], focus was mainly given to the interaction between timber and steel components in this study. This was achieved by employing relatively stiff angles with different thicknesses (15 mm and 12 mm) selected in order to avoid angle failure whilst allowing for different degrees of timber-steel interaction. Table 6.3, and Figures 6.2a and 6.5, provide the details of the two tests conducted on Type 1 connection specimens (Specimens S1 and S2). The dimensions reported in Table 6.3, were obtained by means of a FOTRAN mentioned above and described in Appendix C and are consistent with the minimum and maximum allowable distances for timber and steel given in Tables 6.1 and 6.2.

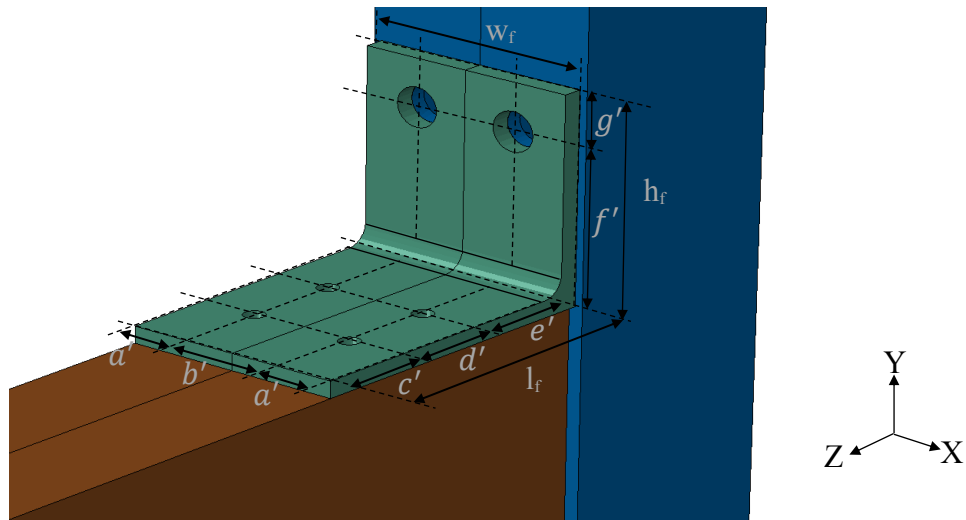


Figure 6.5 – Angle nomenclature for the Type 1 connection configuration.

Reference	Column	Beam	Angle	Dimensions (as shown in Fig. 6.5)						
				a'	b'	c'	d'	e'	f'	g'
S1	SHS 150 x 150 x 10	405 x 140x 2000	L 200 x 150 x 15	36	68	56	60	84	116	34
S2	SHS 150 x 150 x 10	405 x 140x 200	L 200 x 150 x 12	36	68	56	60	84	116	34

Table 6.3 – Summary of the two test specimens for Type 1 connections. Dimensions in mm.

6.2.2.2 Slotted-in T-stub connections with bolts (Type 2)

The configuration adopted for Type 2 connections is depicted in Figure 6.2(b). This connection employs a steel T-stub pre-welded to the SHS column and slotted vertically into the glulam beam as shown in Figure 6.6. In addition, twenty M10 bolts of Grade 10.9 are installed perpendicularly to the steel plate through the full width of the glulam beam and tightened with nuts on both sides. A total of four tests were performed on this type of connection. Details of the tested specimens are given in Table 6.4 and Figure 6.6. Specimen S3 represents the original configuration of Type 2 connection as described above. In addition, a number of reinforcing techniques were examined in Specimens S4, S5 and S6 aimed at enhancing the strength and

ductility of the connection. Specimen S5 incorporated a bottom angle wedged between the timber beam and the SHS column, which has the effect of providing a continuous bearing support and of moving the point of rotation of the joint from its theoretical point C towards point D as depicted in Figure 6.6.

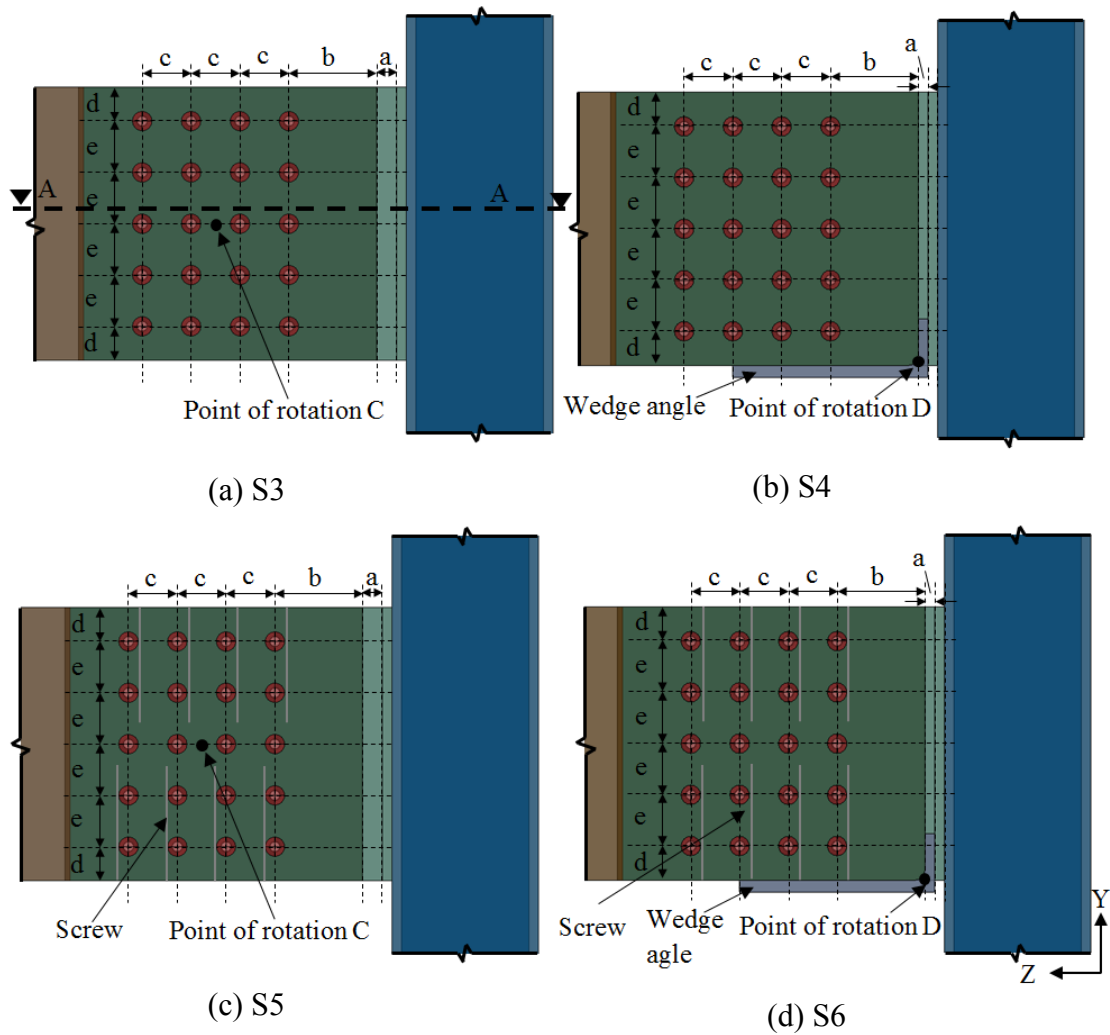


Figure 6.6 – Details of connection Type 2.

Reference	Column	Beam	Angle	Screws	Dim. (as shown in Fig. 6.6)				
					a	b	c	d	e
S3	150 x 150 x 10	280 x 140x 1875	No	No	20	80	50	35	52.5
S4	150 x 150 x 10	280 x 140x 1875	L 200 x 60 x 12	No	10	90	50	35	52.5
S5	150 x 150 x 10	280 x 140x 1875	No	Yes	20	80	50	35	52.5
S6	150 x 150 x 10	280 x 140x 1875	L 200 x 60 x 12	Yes	10	90	50	35	52.5

Table 6.4 – Summary of the four test specimens for Type 2 connections. Dimensions in mm.

Also, with the purpose of improving the overall joint ductility by preventing premature splitting failure of the wood, 32 carbon-steel self-tapping screws were inserted in Specimens S5 and S6 near the bolts, in the areas where high stresses were expected as shown in Figure 6.7. These screws were 120 mm long and had a diameter of 6 mm.

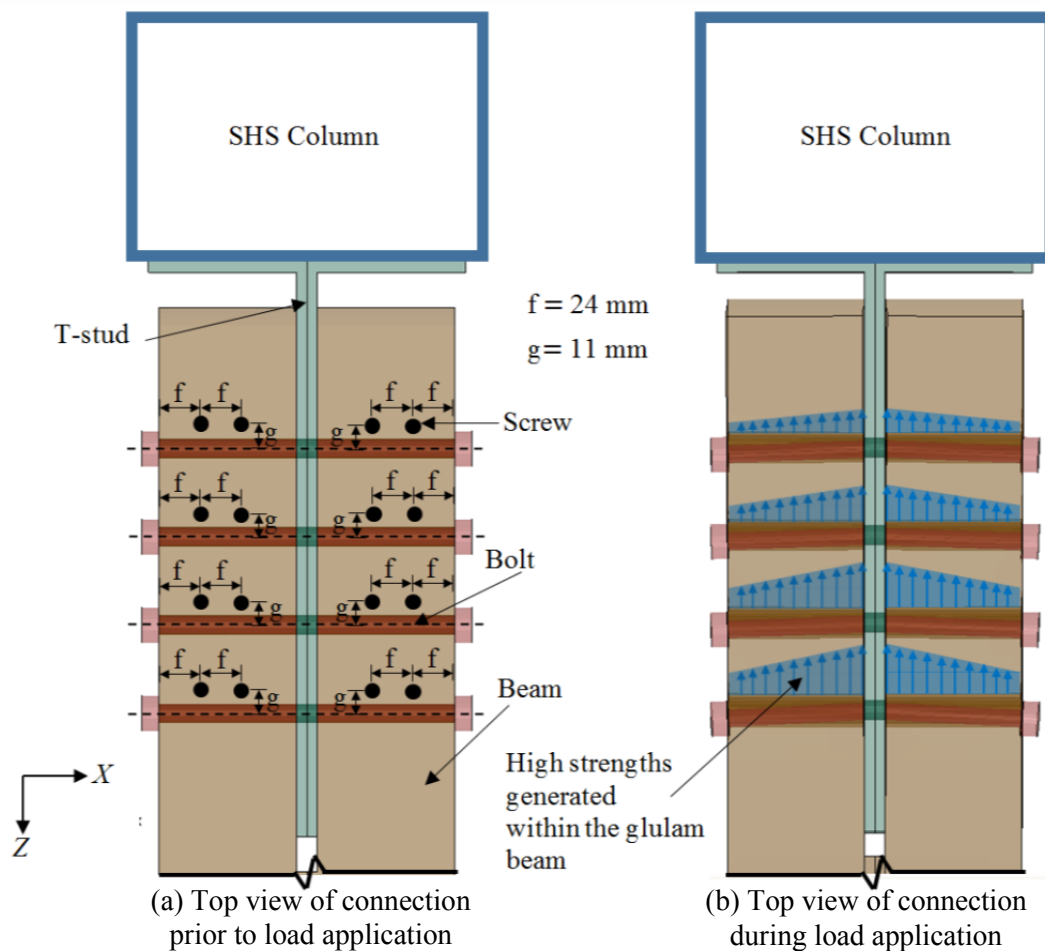


Figure 6.7 – Location of reinforcing screws in Specimens S5 and S6. (Plan view corresponding to Section A-A in Figure 6.6).

6.2.3 Material characterization

The detailed experimental assessment conducted in order to characterize the material properties of the structural timber components employed has been presented in Chapter 3. Table 6.5 below summarizes the key relevant results of interest for the hybrid connections tested herein.

Mech. Properties	Bending Strength	Compressive Strength		Young's modulus		Shear Modulus	Density
		Parallel	Perpend.	Parallel	Perpend.		
	σ_b [MPa]	$\sigma_{11,c}$ [MPa]	$\sigma_{22,c}$ [MPa]	E_1 [MPa]	E_2 [MPa]	G_{12} [MPa]	ρ [kgm ⁻³]
Average	32.2	33.0	3.6	12157.4	425.5	951.8	431
COV [%]	9.2	6.9	8.9	13.6	13.6	59.0	3.5

Table 6.5 – Summary of the mechanical properties for glulam GL28h.

Please note that the Young's modulus for the glulam beam perpendicular to the direction of the grain E_2 , was obtained from the expression given in European Standard EN 1194:1999 [81]:

$$E_2 = 0.035 E_1 \quad (6.1)$$

In addition, fourteen coupon tests were conducted to obtain the mean yield and mean ultimate strength values of the steel components as presented in Table 6.6. The same batch of Holo-bolts adopted by Elghazouli et al. [4] was used, for which the axial capacity had been determined experimentally as 78 kN.

Component	Mean Yield Stress [Mpa]	Mean Ultimate Stress [Mpa]
L 200 x 150 x 12	408	514
L 200 x 150 x 15	361	486
SHS 150 x 150 x 10	334	433

Table 6.6 – Material properties of steel angles and column.

6.3 Test results and observations

6.3.1 Top and seat bolted angle connection (Type 1)

Table 6.7 summarises the main response parameters obtained from the tests on steel-timber connections of Type 1 (Top and seat bolted angle connections). The initial stiffness and the

yield moment were calculated from a bilinear idealisation of the experimentally-obtained moment-rotation curves. Figures 6.8 and 6.9 show the deformation patterns and moment-rotation relationships for Type 1 connections, respectively, while Figure 6.10 shows the strain measurements of the angle parts in specimens S1 and S2.

Reference	Initial Stiffness	Yield moment	Rotation at yield moment	Peak moment reached	Rotation at peak moment
	[kNm/mrad]	[kNm]	[mrad]	[kNm]	[mrad]
S1	7.3	19.1	5.9	48.8	90.1
S2	7.2	14.4	2.8	42.8*	100.0*

*The actuator stroke was reached before evident signs of joint failure.

Table 6.7 – Summary of the test results for Type 1 connections.

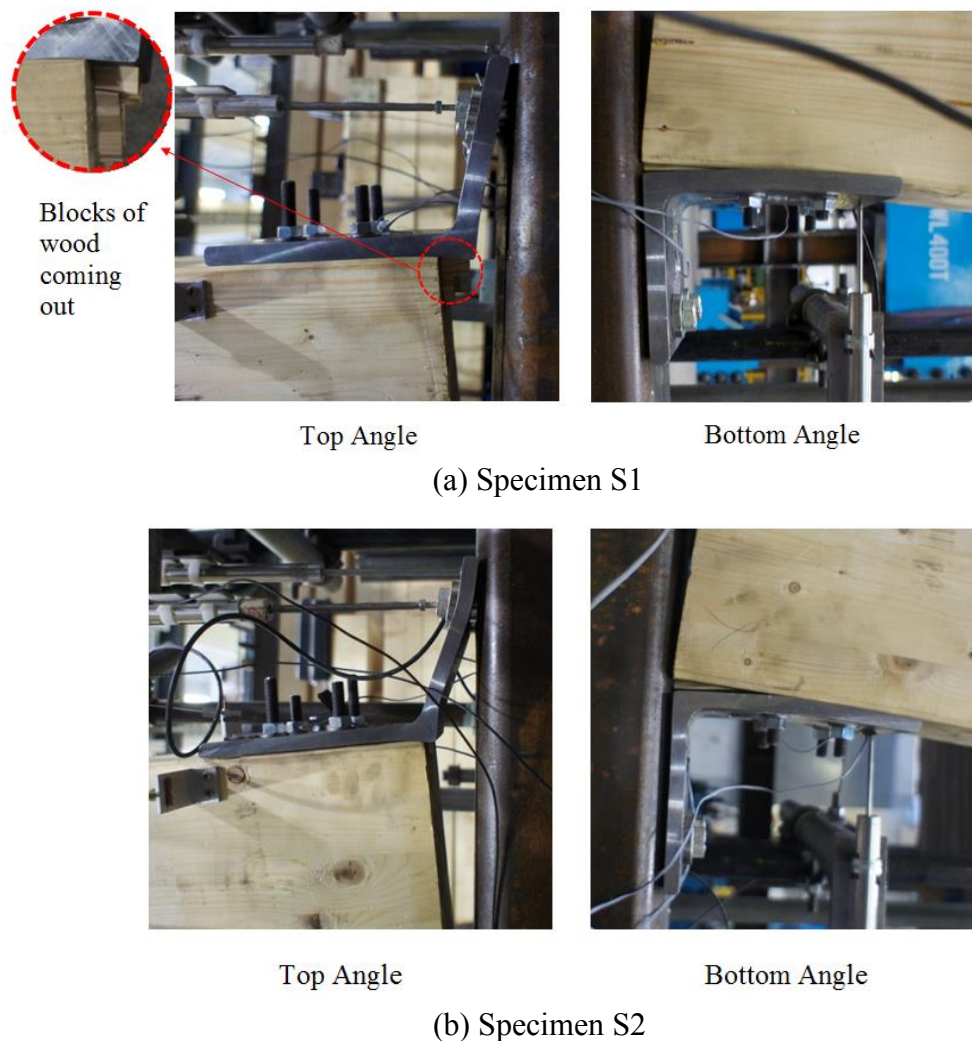


Figure 6.8 – Failure modes of Type 1 connections.

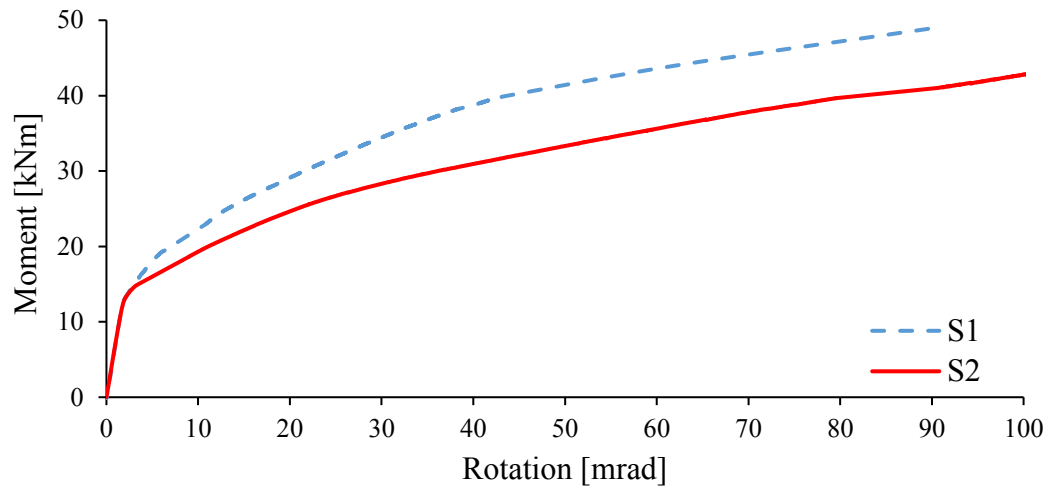
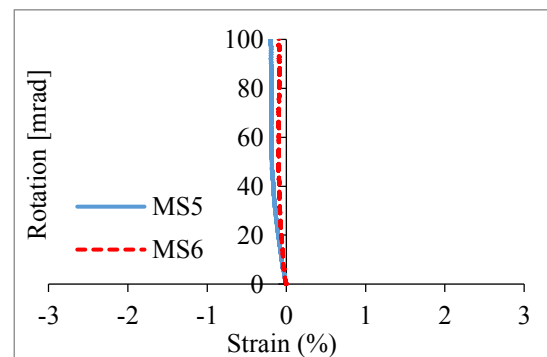
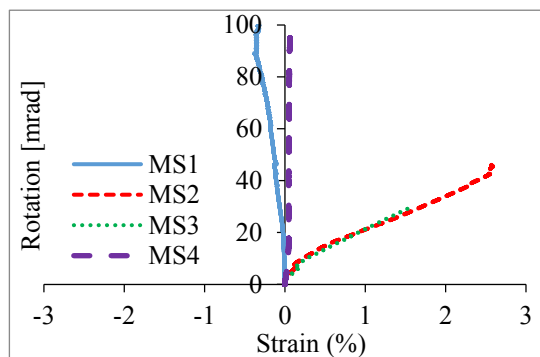


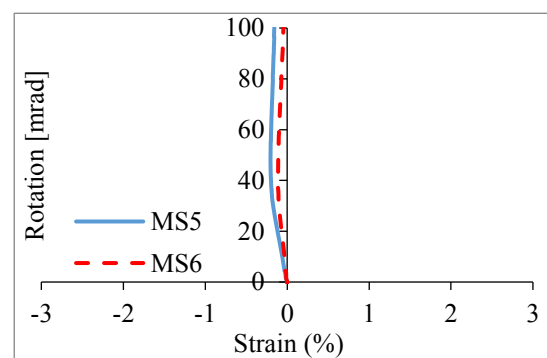
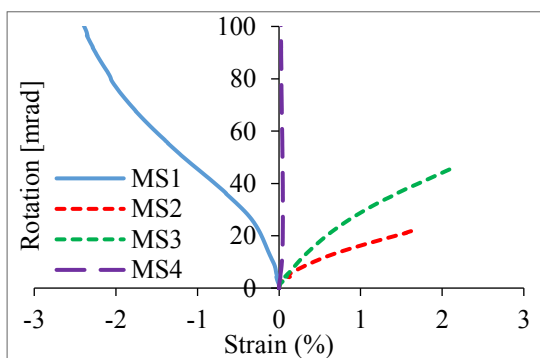
Figure 6.9 – Moment-rotation relationships for Type 1 connections.



(a) Location of strain gauges



(b) Specimen S1



(c) Specimen S2

Figure 6.10 – Measured strains in Type 1 connections.

As explained above, the experimental programme was designed in order to induce different levels of interaction between the steel components and the timber beam. To this end, the effects of decreasing the angle thickness from 15 mm in Specimen S1 to 12 mm in Specimen S2 can be observed by directly comparing the results of both specimens in Table 6.7 and Figures 6.8 to 6.10. It can be concluded that for the relatively stiff angles employed, a 3-mm reduction in angle thickness has negligible influence on the initial rotational stiffness of the connection. However, this angle thickness decrease is sufficient to cause notable differences in the connection yield strength and ultimate capacity. The flexural yield and ultimate strength of Specimen S1 (with 15 mm angles) were approximately 34% and 15% higher than that of S2 (with 12 mm angles), respectively. This difference in ultimate connection strength results from the dissimilar failure modes observed in Figure 6.8a (Specimen S1) and 6.8b (Specimen S2). From these figures, it can be seen that larger plastic deformations are accumulated in the thinner angle (Specimen S2) whereas the thicker angles of Specimen S1 are able to transfer larger forces into the long bolts, eventually leading to a bolt-row shear failure in the timber at a maximum rotation of 90 mrad. It should be noted that the test on Specimen S2 was halted at the maximum stroke of the actuator corresponding to a joint rotation of 100 mrad associated with a peak flexural strength of 42.5 kNm. These differences between the two deformation modes can be further explained with the aid of Figures 6.10b and 6.10c that present the corresponding strain measurements. In the case of Specimen S2, yielding in the top angle occurred first near the toe of the vertical (column) leg (MS2 location in Figure 6.10c) followed by the development of large plastic deformations. Plastic strain levels were also attained in the vertical (column) leg between the Hollo-bolts for rotations greater than 28 mrad (MS1 location in Figure 6.10a). Conversely, and unlike Specimen S2, limited plastic deformation was generated in the vertical leg of the top angle between the Hollo-bolts in Specimen S1 (MS1 location in Figure 6.10a). This resulted in lower plastic strains around the fillet area (locations MS2 and MS3 in Figure 6.10a) and a stiffer response of the angle leading to greater forces on the long bolts attached to the glulam beam. Finally, very limited plastic deformations were observed in the bottom angles of both connection specimens.

6.3.2 Slotted-in T-stub connection with bolts (Type 2)

Table 6.8 summarises the main response parameters for connection Type 2 while Figures 6.11 and 6.12 show the key experimental results in terms of deformation patterns and moment-

rotation relationships. The initial rotational stiffness and yield moments presented in Table 6.8 were obtained from a bilinear idealisation of the corresponding moment-rotation curves.

Reference	Initial stiffness	Yield moment	Rotation at yield moment	Peak moment reached	Rotation at peak moment
	[kNm/mrad]	[kNm]	[mrad]	[kNm]	[mrad]
S3	1.9	20.1	11.1	28.6	57.9
S4	2.0	37.5	19.4	43.9	66.4
S5	1.9	24.5	12.3	38.9*	130.0*
S6	2.1	51.4	22.3	54.4*	130.0*

* The actuator stroke was reached before evident signs of joint failure.

Table 6.8 – Summary of the test results for Type 2 connections.

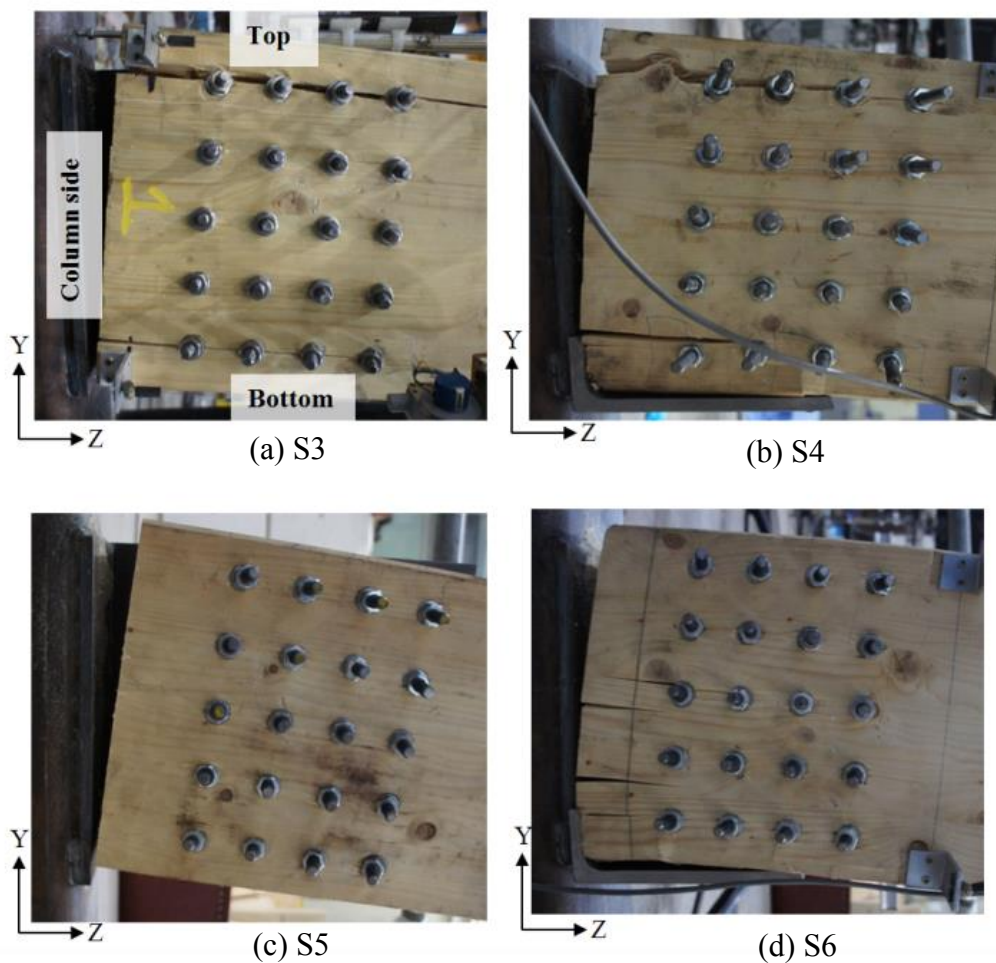


Figure 6.11 – Failure modes of Type 2 connections at the end of test.

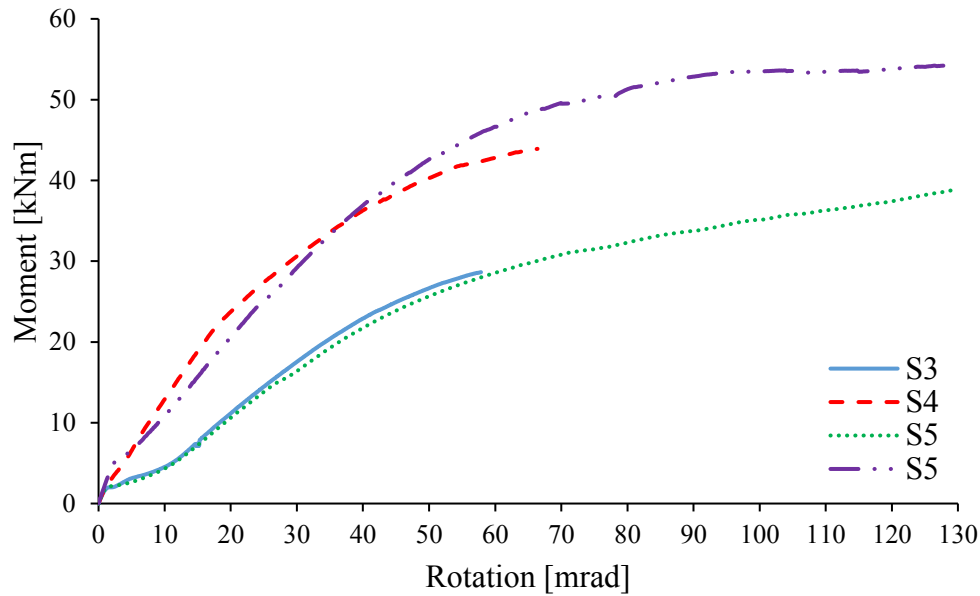


Figure 6.12 – Moment-rotation relationship for Type 2 connections.

6.3.2.1 Deformation patterns and failure modes

Figure 6.11 shows the failure modes of the four Type 2 specimens tested in the present study. It can be observed from Figure 6.11a that Specimen S3 developed shear failures at the transverse section planes containing the top and bottom outmost bolt-rows. The top shear crack, which propagates inside the timber beam, is wider than the crack at the bottom row due to the configuration of stresses in the connection with the tension stresses developed in the upper part of the beam contributing towards the crack opening. In contrast, when reinforcing screws were inserted in the critically stressed zones, as in Specimen S5 of Figure 6.11c, shear splitting was effectively prevented. Alternatively, when a bottom wedge angle was utilised in Specimen S4 without reinforcing screws (Figure 6.11b), the failure mechanism involved a combination of compressive failure and wood splitting at the bottom of the glulam beam. This failure mechanism is caused by the compression forces resulting from the continuous bearing of the beam onto the angle wedge. This compression crushing was observed even in the case of Specimen S6, which besides the wedge angle also incorporated screw reinforcement as shown in Figure 6.11d. It is also worth noting that Specimens S5 and S6 did not reach failure and achieved maximum moments of 38.9 and 54.4 kN/m, respectively, when the actuator attained its full stroke (at 130 mrad of rotation). The different failure modes developed in these connections have significant effects on the moment-rotation curves as discussed below with particular reference to the contribution of wedge angles and screw reinforcement.

6.3.2.2 *Influence of screw reinforcement*

The effects of the presence of screws on the response of the connection can be explored by comparing the experimental results of Specimen S3 with that of Specimen S5 (both without wedge angle) as well as the responses of Specimens S4 and S6 (both with wedge angle). As indicated in Figure 6.12 and Table 6.9, the ultimate capacity of Specimen S5 was around 34% higher than that of S3, whereas the capacity of Specimen S6 was around 24% higher than that of S4. This significant difference in ultimate capacity is a consequence of the improved ductility experienced by connections with screw reinforcement. These enhancements are brought about by the action of screws in delaying the splitting of wood along their axis while at the same time resisting transverse forces perpendicular to their longitudinal axis by bending. Importantly, the yield capacity of the connection appears to be insensitive to the presence of the screws since their favourable effects become significant only after some plastic deformation has already taken place at rotations of ~ 50 mrad or greater.

6.3.2.3 *Influence of wedge-angle*

The influence of the provision of a bottom wedge angle on the connection moment-rotation response can be examined by comparing the results of Specimens S3 (without wedge angle) and S4 (with wedge angle). As Figure 6.12 suggests, the use of a contact angle to enforce early and continuous bearing contact between the bottom part of the beam and the column significantly enhances the stiffness and capacity of the connection. As a general remark, all four Type 2 specimens have an almost identical initial stiffness. However, between ~ 1 and 2 mrad of joint rotation, specimens without wedge angles (i.e. S3 and S5) experience sudden bolt slippage. Importantly, this slippage phenomenon is prevented in S4 and S6 due to the continuous bearing support provided by the bottom wedge angle. Additionally, it can be noted from Figure 6.12 and Table 6.9, that the capacity of Specimen S4 was around 53% higher than that of S3 for which the only difference was the use of a bottom wedge angle in Specimen S4. This gain in capacity is caused by a reconfiguration of stresses instigated by the introduction of a compressive bearing zone at the bottom of the beam as well as the modification of the point of rotation in the joint that lengthened the lever arm as described in the following section.

6.3.2.4 *Non-contact field strain (DIC) measurements of point of rotation*

The exact location of the point of rotation was examined with the aid of a LaVision [84] Digital Image Correlation (DIC) system as shown in Figure 6.13 below. Two cameras spaced at 500

mm from each other and rotated 15° inwards, were placed at a distance of 1500 mm from the specimen.

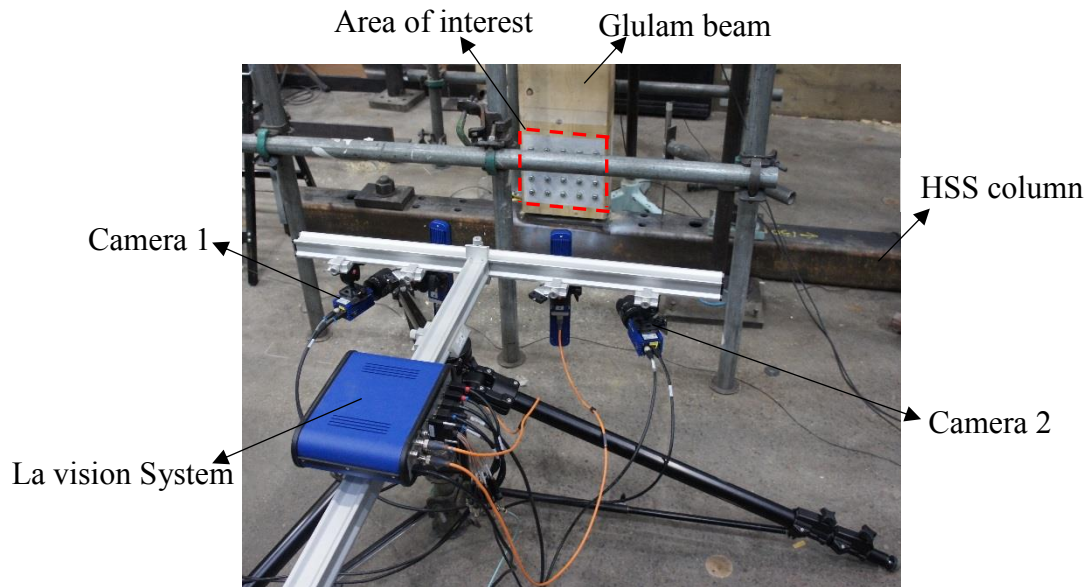


Figure 6.13 – DIC system set-up for Type 2 connections.

The area of interest was prepared by spraying a randomized pattern of black dots onto a white pre-painted beam surface as depicted in Figure 6.14 below. High quality pictures of 5 mega pixels were taken at a rate of 6 frames per minute during testing. The DIC algorithm employed [84] tracks the motion of surface patterns and calculates the corresponding 3D strain fields at each frame.



Figure 6.14 – Area of interest sprayed into a randomized pattern.

Figures 6.15a and 6.15b show the displacement contours in the y (along the beam axis) and z (perpendicular to the beam axis) directions for Specimens S5 and S6 respectively, at a joint

rotation of 34 mrad. From the information given in these figures, the *neutral* planes parallel to the y and z directions can be easily identified. The point of rotation is located at the intersection between those two planes of zero relative displacement. It is important to note that Specimens S3 and S4 presented an identical evolution in the location of their points of rotation as those observed in Specimens S5 and S6, respectively.

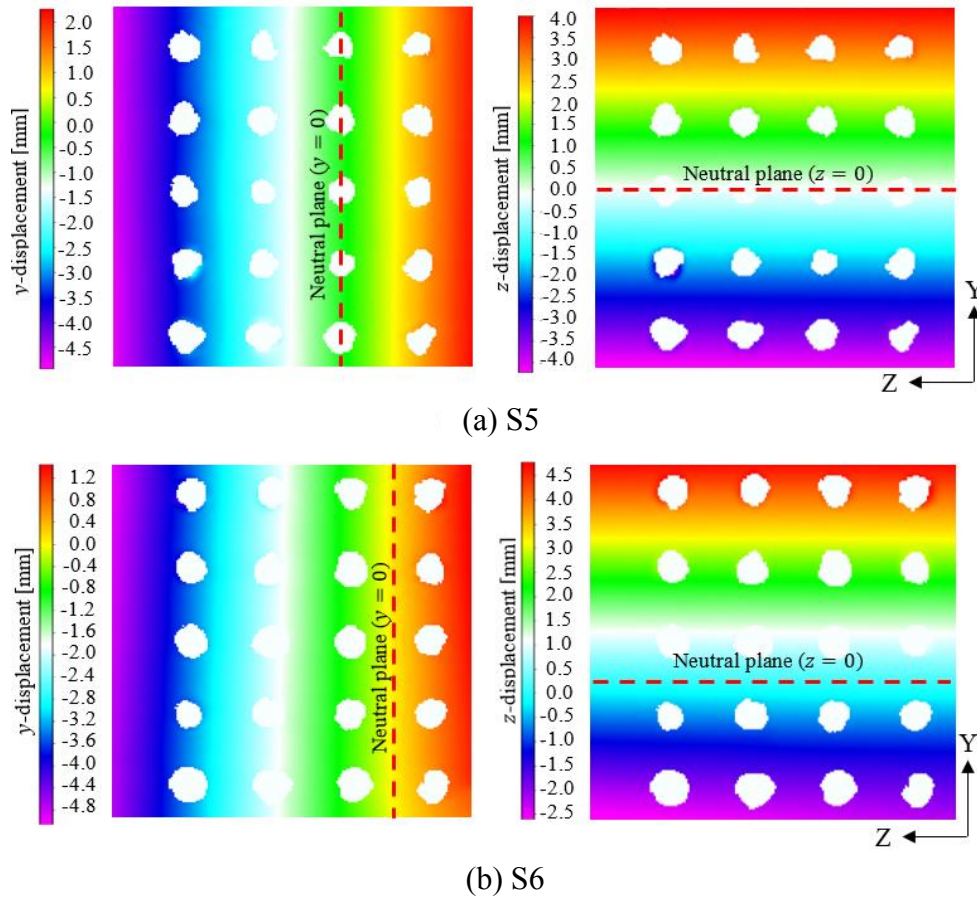


Figure 6.15 – Displacement contours of specimens S5 and S6, at 34 mrad of rotation.

By following the procedure described above, an accurate and continuous monitoring of the point of rotation within the bolt group of Type 2 connections can be made. Figure 6.16 depicts the evolution of the point of rotation (indicated as a red dot) at different levels of joint rotation for specimens S5 and S6. It is observed from Figure 6.16 that after a certain level of rotation is attained (i.e. 34 mrad in S5 and 25 mrad in S6) the location of the point of rotation remains unaltered until the end of the test. It is also interesting to note with reference to Figures 6.12 and 6.16 that for rotations larger than 8.82 mrad the point of rotation displaces vertically with negligible horizontal translation in both specimens, whereas prior to 8.82 mrad of joint rotation, the main translation component in the point of rotation's motion is horizontal. This level of

connection rotation (8.82 mrad) corresponds to the point at which Bolt-1 (Figure 6.16) starts yielding.

Finally, Figure 6.16 also depicts the ideal locations for the points of rotation (shown as a purple dot) at the centroids of the fastener groups obtained with a rigid motion assumption. In the case of Specimen S5 without the bottom wedge angle (Figure 6.16a) the centre of rotation lies at the geometric centre of the group. On the other hand, the ideal point of rotation in the presence of a wedge angle (as in Specimen S6 in Figure 6.16b), was obtained by idealizing the vertical leg of the angle as a continuous series of equivalent bolts of equal diameter ($d = 10$ mm). This will be further discussed in Chapter 7. It can be appreciated that although some minor discrepancies are present between the ideal estimations and the real location, the magnitude and effects of alternation of the bolt group configuration are well represented, and the predicted points of rotation can be employed for the calculation of the connection stiffness and strength as described in the following chapter.

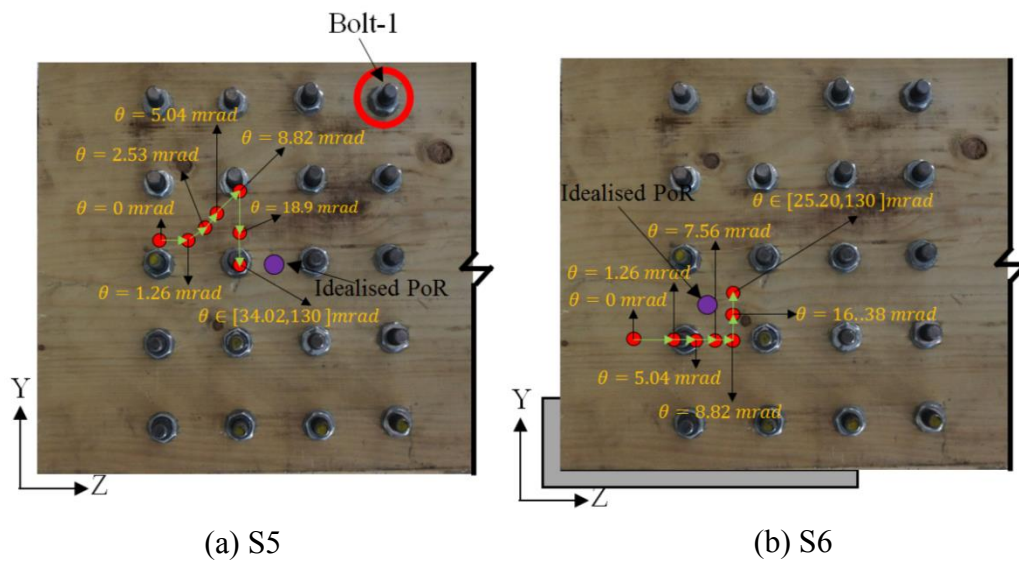


Figure 6.16 – Evolution of the location of the point of rotation for specimens S5 and S6 at various rotations.

6.4 Concluding remarks

In this chapter the behaviour of newly proposed hybrid bolted connections between glulam timber beams and tubular steel columns has been experimentally examined. Two different connection alternatives were suggested and studied involving top and seat angle details as well

as slotted-in T-stub bolted configurations. The experimental programme, comprising six monotonic tests, on the joints subjected to bending was described in detail. The main behavioural patterns and failure mechanisms were identified, and the key response characteristics such as stiffness, capacity and failure mechanism were discussed.

Given the growing body of research on the influence of the characteristics of the tubular column face, angle component and blind bolt arrangement, this study focused on the interaction between the steel components and the timber beam. In the case of top and seat blind-bolted connections, it was observed that when a stiffer angle is employed, higher forces are generated in the long bolt running parallel to the beam depth leading to localized shear failure in the wood.

In the case of slotted-in T-stub bolted connections, a detailed evaluation of the migration of the point of rotation was made based on continuous non-contact strain field measurements and Digital Image Correlation techniques. Two distinct stages were identified. Prior to 8.3 mrad of joint rotation, a markedly horizontal displacement of the point of rotation was observed while a predominantly vertical migration of the point of rotation was recorded for larger joint rotations.

Two alternatives of joint reinforcement were examined in the case of T-stub slotted-in bolt-group details, namely: the use of reinforcing screws against wood splitting, and the provision of a bottom wedge angle to enhance the joint strength. It was observed that the screws can significantly increase the connection failure moment, by around 36 %, through a successful prevention of wood splitting, whereas the incorporation of a wedge bottom angle reduces the effects of instantaneous bolt slippage and significantly enhances the ultimate connection capacity in excess of 50 %. The range of stiffness and capacity obtained from these specimens indicate that for typical Glulam beam configurations, the proposed detail could be designed as a semi-rigid connection with significant transfer of moment to the columns. Finally, the experimental results provided the necessary information for the validation and calibration of complementary finite element and simplified component-based models presented in the next chapter.

Chapter 7

Numerical simulations and simplified component-based models

7.1 Introduction

Having studied the experimental behaviour of hybrid connections between glulam timber beams and tubular steel columns in the previous chapter, this chapter deals with the prediction of their moment response by means of continuous FE models and simplified component-based formulations. Due to the complex behaviour of timber beam-to-tubular column connections which is influenced by the intricate interactions between fasteners and wood examined in Chapters 4 and 5, assessing their response by means of experimental investigations alone is cost-prohibitive and unrealistic. In this sense, the use of appropriately validated finite element models offers a rigorous alternative for the examination of the influence of various connection parameters on their moment-rotation response. To this end, this chapter presents three-dimensional Finite Element (FE) models constructed by means of the commercial FE software ABAQUS [7]. These models make use of the foundation modelling approach presented in Chapter 5 and are validated against the experimental data presented in the previous chapter. Also, the need for simplified design-oriented expression for these hybrid connections has been highlighted previously in this thesis. Therefore, this chapter also examines the applicability of component-based expressions for the estimation of stiffness and capacity of the timber-steel moment connections proposed and suggests specific modelling simplification to take into account the nail reinforcing and wedge support modifications studies in Chapter 6.

7.2 Finite element model

7.2.1 Connection modelling details

Detailed 3D models for the beam-column hybrid connections described in the previous chapter were constructed using the FE program ABAQUS V6.14 [7]. Figure 7.1 illustrates the models created for both connection configurations, whereas Figure 7.2 presents the FE model

representation of the Lindapter Holo-bolt [44] used for the Type 1 connection. Such models made use of 8-node brick solid elements of Type C3D8 for all steel and timber components except for the reinforcing screws in Specimens S5 and S6 which were simulated with 3D 2-node linear beam elements of Type B31. Additionally, as indicated in Figure 7.1 symmetry conditions were applied in YZ-plane for efficiency of composition.

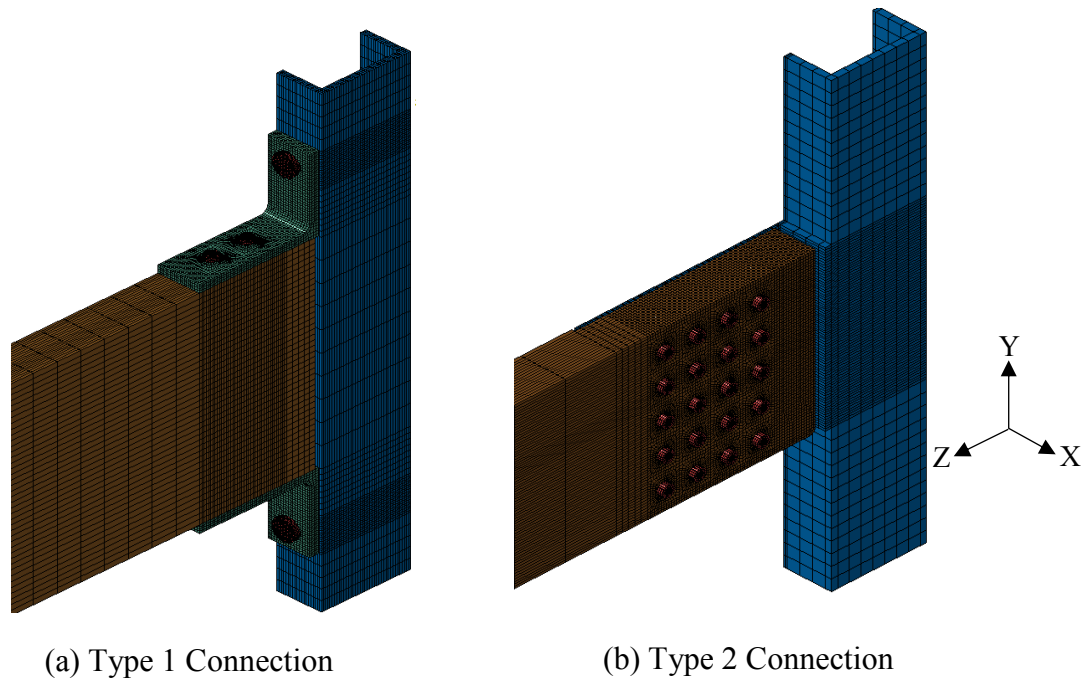


Figure 7.1 – Numerical models of connection Types 1 and 2.

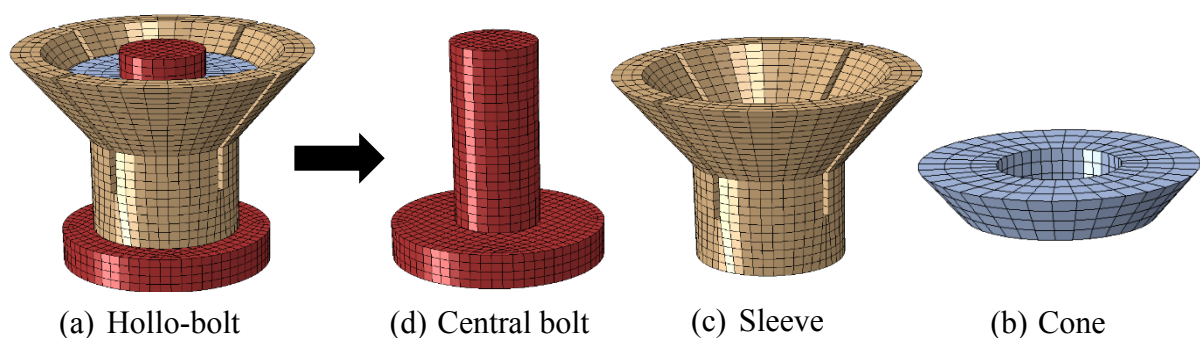


Figure 7.2 – FE modelling of the Lindapter Holo-bolt.

A tri-linear kinematic hardening representation with an elastic modulus of 210 GPa and a Poisson's ratio of 0.3 was employed for the steel components. This steel stress-strain relationship incorporates an elastic stage, a constant stress plateau up to a strain level of approximately 0.01, and a 1% strain-hardening stiffness thereafter. Similarly, the experimentally-obtained mean material properties of wood as presented in Table 6.5, were adopted for the beam outside the foundation zone while modified values of strength and stiffness were employed within the foundation zone as described in Chapter 5.

Contact phenomena between steel and wood were modelled by defining pairs of “master” and “slave” contact surfaces in ABAQUS. To this end, the relatively more flexible wood surface was defined as “slave” whereas the stiffer steel surface was chosen as “master”. The hard contact over-closure pressure relationship of ABAQUS was modified by adopting a linear relationship with significantly stiff response to minimise non-convergence. A friction coefficient of $\mu = 0.7$ was used as proposed by Smith [92].

A number of mesh size sensitivity studies were carried out in order to arrive at an optimum element size distribution. Figures B5 and B6 presented in Appendix B, show the results in terms of the average reaction force at the nodes and the end of the beam against the number of elements. The adopted mesh arrangement involved a coarser mesh within the main body of the wood sample, whereas a finer mesh was employed for the foundation zone around the bolt-hole. The dimensions of the finally adopted mesh ranged between 1 and 5 mm.

7.2.2 Modified foundation embedment modelling applied to groups of fastener

As it was demonstrated in Chapter 5 and in previous research [67,69], it is not possible to model the response of dowelled/bolted connections by means of conventional FE strategies that assume standard material constitutive properties throughout the continuum. Hence, the *foundation zone* approach [28,67,70], introduced and discussed in Chapter 5, is utilised herein in order to incorporate the effects of localised wood crushing. The foundation model as previously stated assumes a cylindrical volume with dimensions proportional to the diameter of the fastener, and this is the approach adopted when modelling Type 1 Connections. However, in the case of Type 2 Connections, involving a group of various fasteners, it is more convenient to merge their corresponding foundation zones into a larger enclosing volume defined by the radius R_{FZ} associated with each individual dowel, as shown in Figure 7.3.

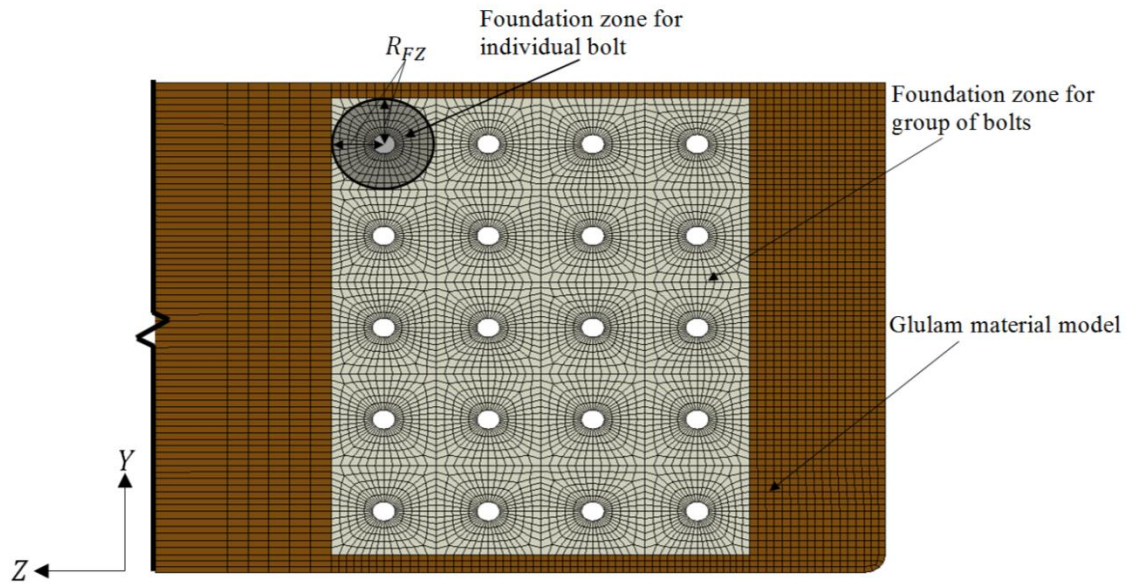


Figure 7.3 – Implementation of the modified foundation FE model for the bolt group of Specimen S3 (RFZ = 25 mm).

7.2.3 Equivalent material properties for the foundation zone

The mean material properties from the material characterisation reported in Chapter 3 were adopted herein for the main wood volume outside the foundation zone. On the other hand, for Type 1 Connections, the material properties for the foundation zone were directly based on embedment test results presented in Chapter 4, which utilized 12 mm fasteners. However, since the 10 mm bolts employed in Type 2 Connections were not directly considered in Chapter 3, a linear extrapolation process was carried out to determine the required material modelling parameters. Table 7.1 presents the material parameters as obtained from the embedment tests (identified by the subscript *Emb*) with fasteners of diameters 12 and 16 mm together with those estimated for 10 mm bolts. The indices 1 to 3 in Table 7.1 refer to the three principal stress directions.

Properties	Experimental data		Estimated data
	d = 12 mm	d = 16 mm	d = 10 mm
$E_{1,Emb}$	376.0	1033.9	376.0
$E_{2,Emb} = E_{3,Emb}$	136.8	240.8	136.8
$\sigma_{11,Emb}$	29.4	29.6	29.3
$\sigma_{22,Emb} = \sigma_{33,Emb}$	14.7	14.6	14.8

Table 7.1 – Estimated material properties for fastener diameter of 10 mm.

The foundation moduli, $E_{i,Emb}$, (for $i = 1,2,3$), were estimated in accordance with the findings of Hong[70] who showed that the influence of the fastener diameter on the embedment stiffness is greater for large diameters ($d > 12$ mm), and becomes negligible for smaller diameters ($d \leq 12$ mm). Furthermore, these size effects are more important where the direction of the grain is parallel to the direction of the load. Hence, as shown in Table 7.1, the estimated values for the foundation moduli, $E_{i,Emb}$, with $d = 10$ mm were assumed to be identical to those obtained for $d = 12$ mm. In the case of Poisson ratios and shear strengths, nominal values were assumed regardless of the foundation zone distinction [67]. Equations (5.3) to (5.6), were used for the determination of effective Elastic Modulus and yield stresses within the foundation zone in the directions parallel ($E_{1,FZ}$ & $\sigma_{11,FZ}$), and perpendicular ($E_{2,FZ}$ & $\sigma_{22,FZ}$), to the grain orientation. The values for the empirical parameters β_1 , β_2 , γ_1 and γ_2 for the connection Type 1 ($d = 12$ mm) can be obtained from Tables 5.3 and 5.4. These tables are reproduced below as Tables 7.2 and 7.3 and are extended to include the corresponding values for $d = 10$ mm cases employed in connections Type 2.

Parameter	d = 12 mm	d = 16 mm	d = 10 mm
β_1	6.27	2.28	8.27
β_2	5.51	3.13	6.70

Table 7.2 – Values for parameters β_1 and β_2 as a function of the fastener diameter

Parameter	d = 12 mm	d = 16 mm	d = 10 mm
γ_1	14.67	14.60	11.22
γ_2	21.57	21.74	11.03

Table 7.3 – Values for parameters γ_1 and γ_2 as a function of the fastener diameter

7.2.4 FE Model validation

The finite element (FE) models described above were validated against the results from the experimental programme described in Chapter 6. Figure 7.4 presents the comparisons between the numerical simulations and the experimental curves for Specimens S1 and S2. This figure demonstrates that reasonable estimates of stiffness and capacity can be obtained by means of

the detailed FE model described above. Some differences are evident between the FE predictions and the observed moment-rotation relationships at approximately 1.5 mrad. This phenomenon can be attributed to bolt slippage that occurs through a series of instantaneous movements in the numerical model whereas a more gradual slip displacement is observed in the tests. Nevertheless, a good agreement can be observed between the numerical estimation and the experimental moment capacity with a maximum difference of less than $\pm 10\%$ for both specimens. The numerical FE analysis of Specimen S1 stopped at ~ 40 mrad due to the large concentration of element deformations around the bolt area which correlates well with the experimentally observed shear failure in this specimen. Figure 7.5 illustrates the deformation patterns of the top angle of Specimens S1 and S2 at 40 mrad and 80 mrad rotation, respectively, together with the corresponding numerical predictions. From these figures it is evident that the deformation and plastic mechanisms are well captured by the proposed FE model. Similarly, the higher concentration of non-linear strains at the blind-bolt row-line observed in Specimen S2, in comparison with Specimen S1, is also captured by the numerical simulations.

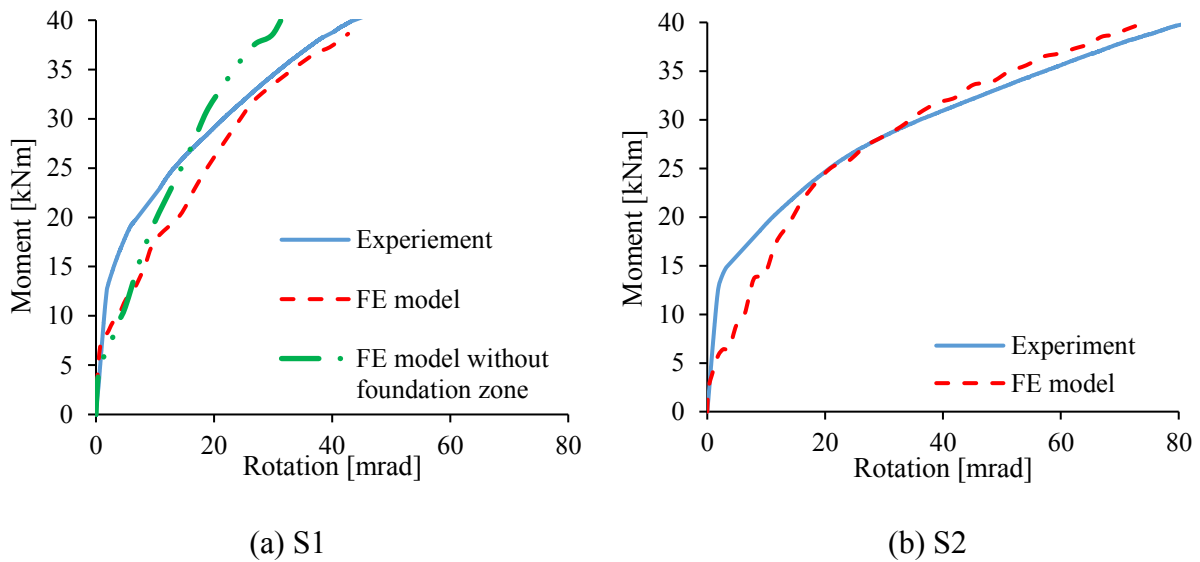


Figure 7.4 – Comparison of moment-rotation response between experimental tests and FE model for connection Configuration 1.

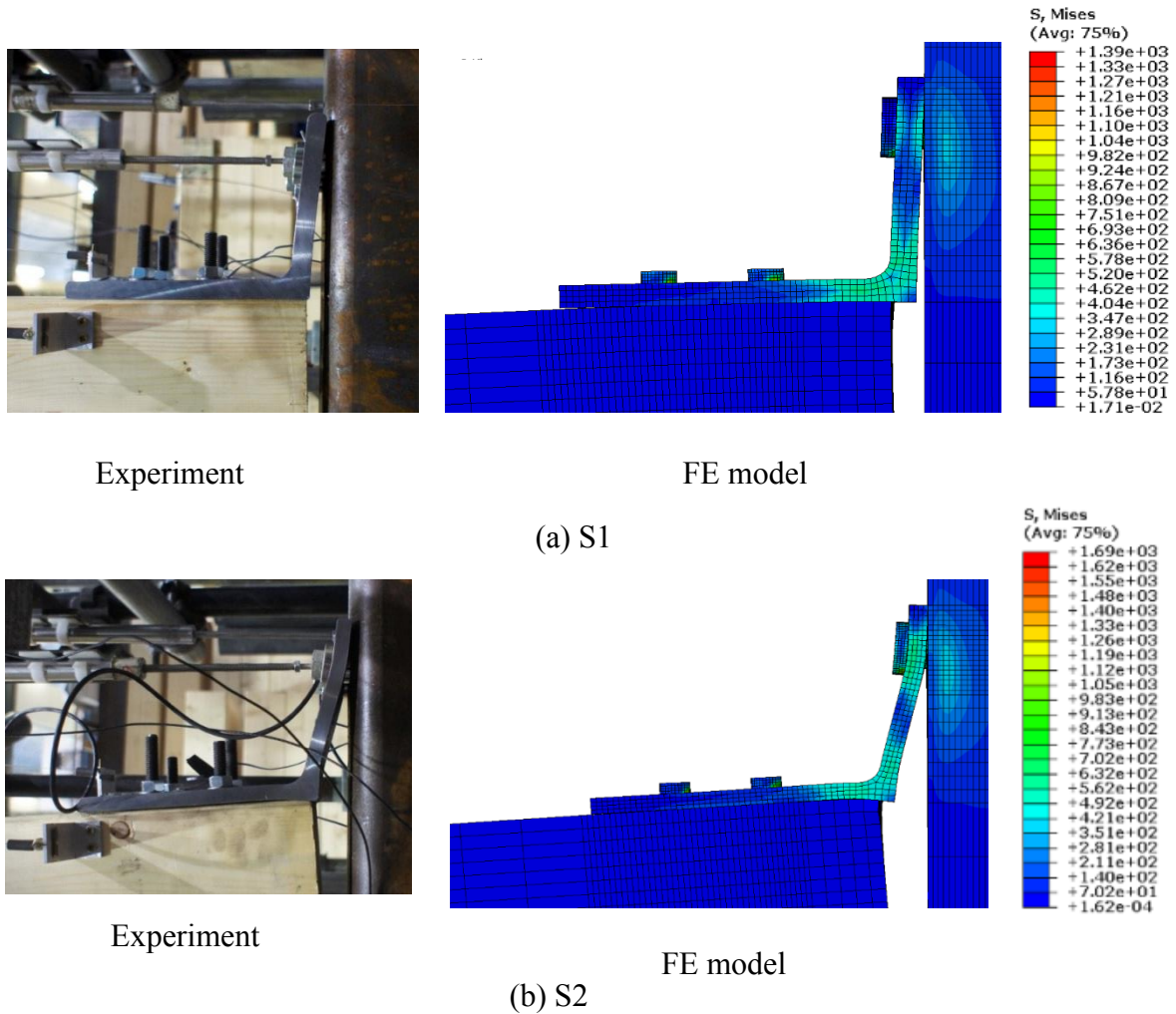


Figure 7.5 – Comparison of top angle deformation patterns [Stress in MPa]

Figure 7.6 presents a comparison between experimental and numerical moment-rotation responses of Type 2 Specimens S3, S4, S5 and S6. The plots confirm that the numerical predictions are in good agreement with the experimental results. Importantly, the numerical models were able to capture the full non-linear response of Specimens S5 and S6 very accurately, including their ultimate capacity. In the case of Specimens S3 and S4, the response of the connection was very well predicted up to around 60 mrad of joint rotation. Larger rotations are associated with the initiation of shear failure in the glulam beam (see Figures 9a and 9b) and the corresponding onset of significant damage interactions, which are not taken into account within the constitutive material models employed. Also, all models experienced an instantaneous bolt slippage at around 1.5 kNm of moment due to the clearance between the bolt shank and the glulam beam.

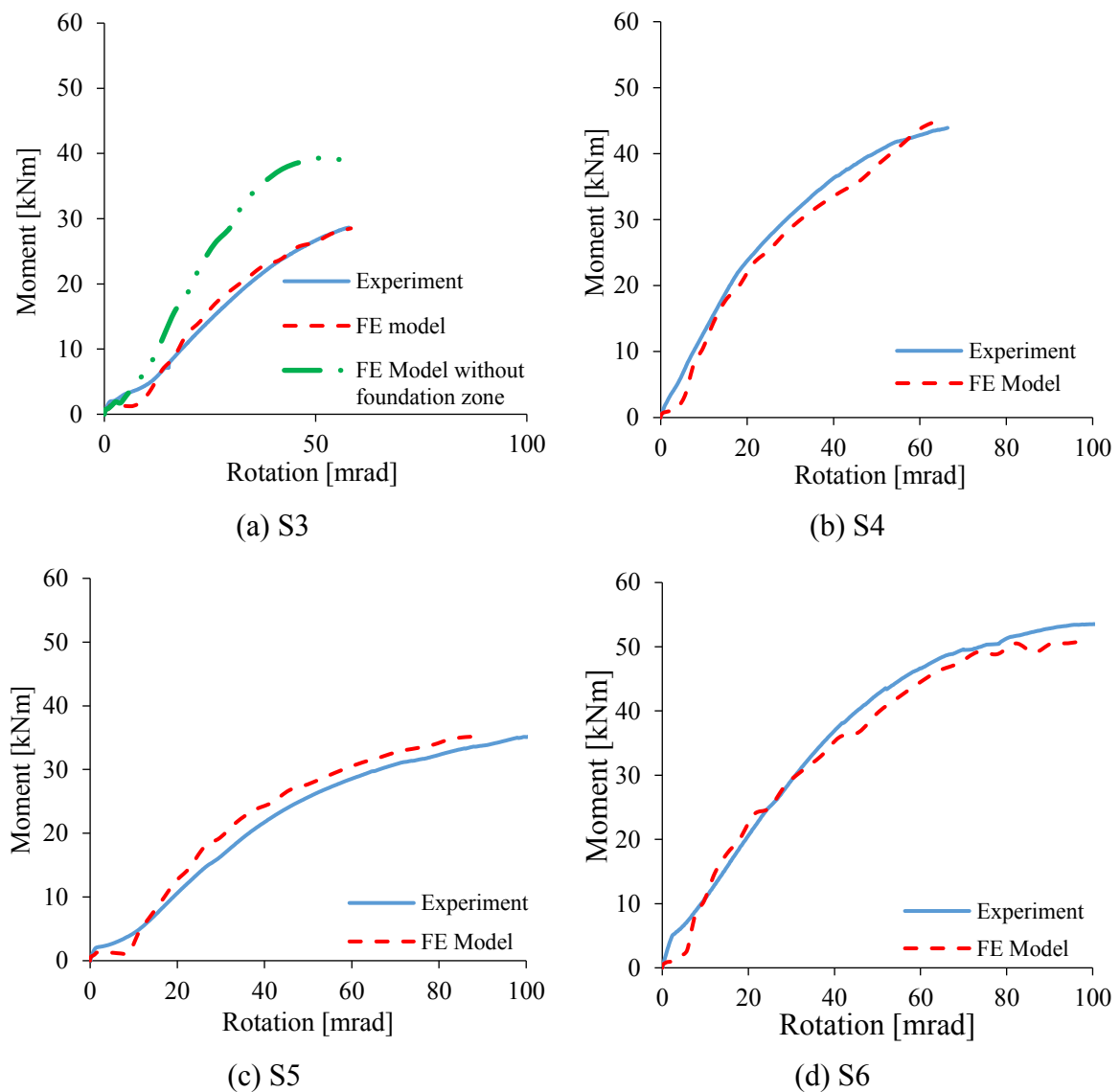


Figure 7.6 – Comparison of moment-rotation response between experimental tests and FE model for Type 2 connections.

In addition to the experimental and numerical comparisons already discussed, Figure 7.4a and 7.6a present the moment-rotation relationship obtained from a conventional FE modelling approach without the definition of a foundation zone for illustration purposes. It is evident from these curves that the implementation of the foundation zone is essential for obtaining accurate predictions of the response of timber connections since the absence of damage related to localised wood crushing during embedment can lead to significant over-predictions of the joint response.

Figure 7.7 compares the residual plastic deformation of the outermost bolt in the tensile region of Specimen S3 (see Figure 6.11a of chapter 6) with the corresponding numerical simulation. It can be observed from Figure 7.7 that the model is able to capture the deformation pattern of the bolt including the full development of a plastic hinge. A further comparison can be established in the case of the screws used to reinforce Type 2 connections (Specimens S5 and S6 presented in Figures 6.11c and 6.11d). This figure illustrates the close comparison of residual deformed states, including the location and number of plastic hinges in the most stressed screw of Specimen S6. All these results and comparisons provide confidence in the modelling strategy and validate the accuracy of the proposed models for further studies presented in Chapter 8 aimed at complementing the experimental findings described herein.

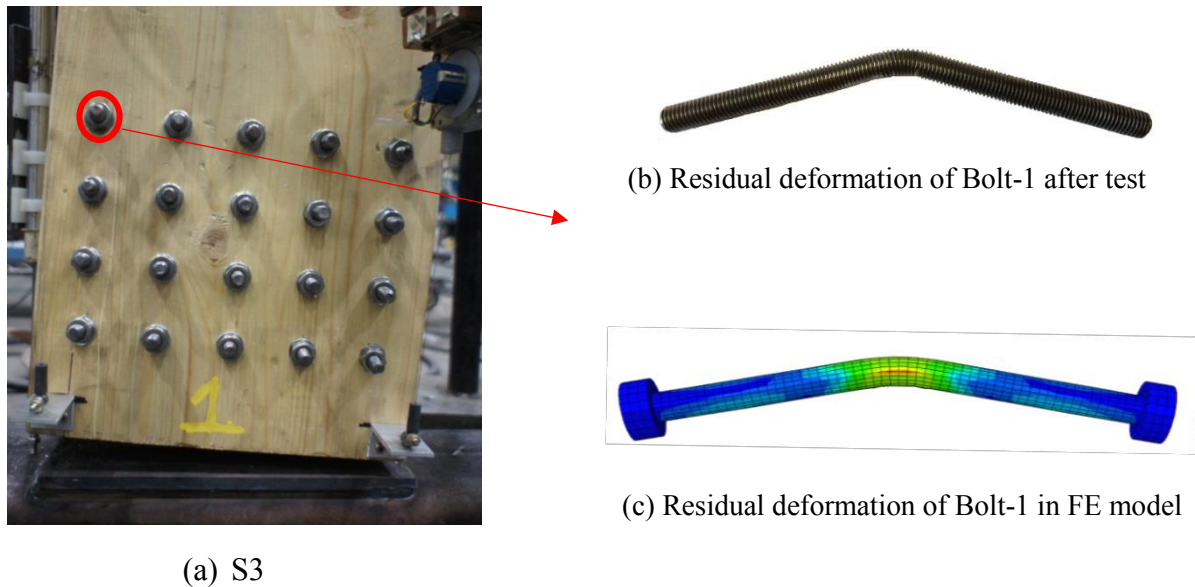


Figure 7.7 – Comparison of bolt deformation pattern between experimental and FE model for Specimen S3.

7.3 Component-based models

In this section, the widely used simplified component approach [29,34,95], which is suitable for practical design and assessment applications, is utilised for establishing the stiffness and capacity of the new hybrid timber beam-to-steel column connections proposed and compared with the more time-consuming and computationally expensive FE simulations presented above. The evaluation of the flexural capacity of hybrid connections is discussed first with reference to Type 1 and Type 2 configurations, and considering shear failure modes where relevant. Subsequently, the prediction of the connection stiffness is considered. Finally, the

results of the expressions presented are validated against the experimental results presented in earlier sections.

7.3.1 Capacity estimation of Type 1 connections

The yield strength of this type of connection (i.e. top and seat angle with bolts) is governed by two main failure modes. The first mode is associated with the bending resistance of the blind-bolted angles while the second mode is related to the interaction between the glulam beam and the fastener-angle assemblage. In addition, prediction of the row shear fracture capacity can be important for the estimation of the ultimate resistance of the connection as observed from the results of Specimen S1 presented in Figure 6.8a.

7.3.1.1 Blind-bolted Angle Capacity

According to EC3 Part 1-8 Section 6.2.6.6 [34], the tension design resistance and the failure mode of a bolted angle in bending can be associated with those of an equivalent T-stub flange. In general T-stub flanges can be subjected to three distinct failure modes under tensile forces: a) complete yielding of the flange, b) bolt failure with yielding of the flange and c) bolt failure. These three failure modes are illustrated in Figure 7.8 below.

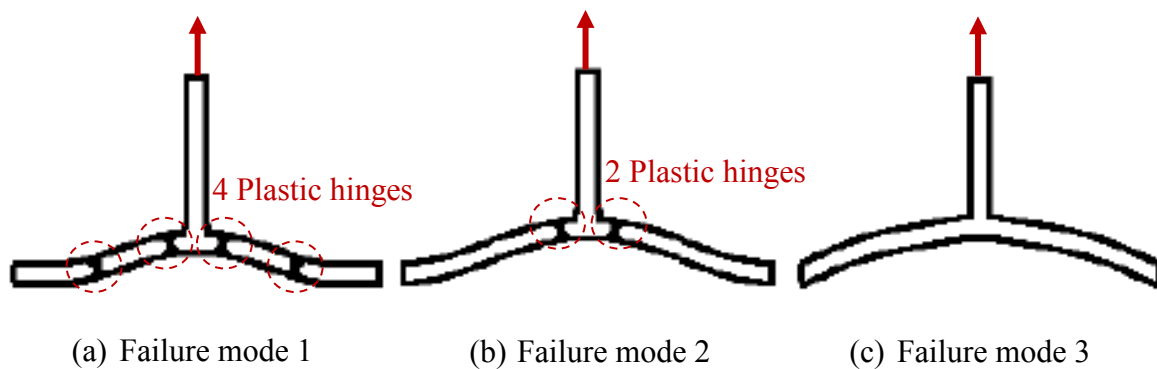


Figure 7.8 – Failure modes of equivalent T-stub under tension

Consequently, the design resistance of the blind-bolted angle ($F_{T,Rd}$) can be estimated as the smallest of the three failure modes represented in Equations (7.1) to (7.3) [34]:

$$F_{T,Rd} = \min \left\{ \begin{array}{l} \frac{4 \cdot M_{pl,1,Rd}}{f' - t_f - r_{fill}} \\ \frac{2 \cdot M_{pl,2,Rd} + g' \cdot \sum F_{t,Rd}}{(f' - t_f - r_{fill}) + g'} \\ \sum F_{t,Rd} \end{array} \right. \quad \begin{array}{l} (7.1) \\ (7.2) \\ (7.3) \end{array}$$

with:

$$M_{pl,1,Rd} = M_{pl,2,Rd} = \frac{w_f t_f^2 \cdot \sigma_{y,f}}{2 \cdot 4} \quad (7.4)$$

where, w_f is the angle width, t_f is the angle thickness, r_{fill} is the corner radius of the angle, f' and g' are the gauge distances defined in Figure 6.5 which can be modified to account for the inherent rotational flexibility of the Hollo-bolt as suggested in previous studies [43, 94], and $\sum F_{t,Rd}$ is the total value of the tension resistance $F_{t,Rd}$ for all the fasteners in the angle.

7.3.1.2 Vertical bolts capacity

The capacity associated with the long bolt-timber interaction can be determined from the expressions given in Chapter 8.2.3 of Eurocode 5 [29] for timber-to-steel dowel-type joints. The failure modes experienced by these connections depend on the thickness of the steel angle (t_f), the diameter of the standard bolts (d_{SB}), the characteristic embedment strength parallel to the grain ($\sigma_{11,emb,k}$), the distance parallel to the width of the beam between the side of the beam and the inserted metal plate (t_1), the characteristic sectional yield moment of the bolt ($M_{y,Rk}$) as well as the characteristic withdrawal capacity of the bolt in kN ($F_{ax,Rk}$). The failure modes that can occur for Specimens S1 and S2, which satisfy the relationship $t_f \geq d_{SB}$ (thick steel plate), are illustrated in Figure 7.9 below.

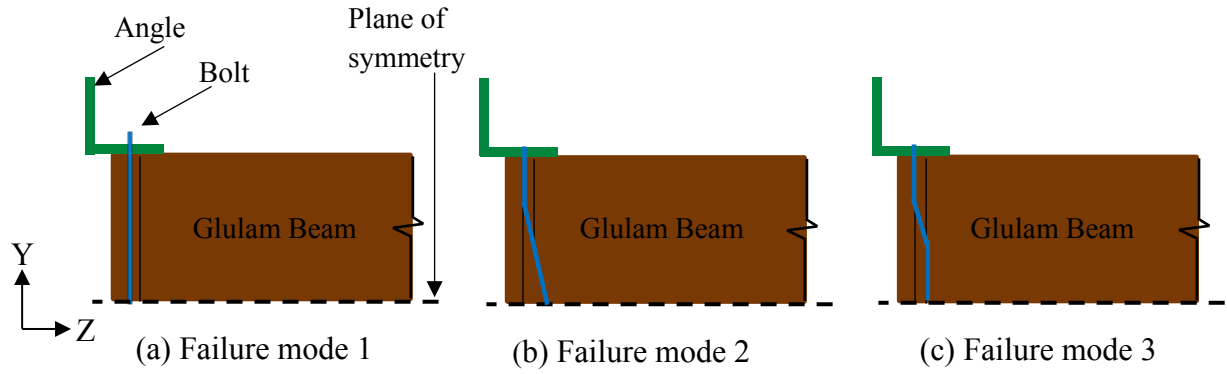


Figure 7.9 – Failure modes for steel-to-timber connection with thick angles

Based on the failure modes, shown above, their characteristic load-carrying capacity ($F_{v,Rk,conf1}$) can be calculated as:

$$F_{v,Rk,conf1} = \min \begin{cases} \sigma_{11,emb,k} \cdot h_{gl} \cdot d_{SB} & (7.5) \\ \sigma_{11,emb,k} \cdot h_{gl} \cdot d_{SB} \left[\sqrt{2 + \frac{4M_{y,Rk}}{\sigma_{11,emb,k} \cdot h_{gl}^2 \cdot d_{SB}}} - 1 \right] + \frac{F_{ax,Rk}}{4} & (7.6) \\ 2.3 \sqrt{M_{y,Rk} \cdot \sigma_{11,emb,k} \cdot d_{SB}} + \frac{F_{ax,Rk}}{4} & (7.7) \end{cases}$$

where the term $(F_{ax,Rk}/4)$ in equations (7.6) and (7.7) is the contribution from the rope effect. Moreover, the withdrawal capacity of the bolt can be estimated from the following relationship [29]:

$$F_{ax,Rk} = \min \begin{cases} \sigma_{ub} \cdot A_t \\ 3 \cdot \sigma_{22,c} \cdot \frac{\pi}{4} [d_w - (d_{SB} + 1)] \end{cases} \quad (7.8)$$

$$(7.9)$$

where σ_{ub} is the tensile strength the appropriate bolt grade in MPa, A_t is the tensile stress area in mm², $\sigma_{22,c}$ is the compression strength of the glulam beam perpendicular to the direction of the grain in MPa, and d_w is the bearing capacity of the steel plate inserted into the glulam beam in mm that can be taken as:

$$d_w = \min \begin{cases} 12 \cdot t_f \\ 4 \cdot d_{SB} \end{cases} \quad (7.10)$$

$$(7.11)$$

Additionally, Eurocode 5 [29] suggests that the contribution to the load carrying capacity due to the rope effect should be limited to 25% of the Johansen yield equation when bolts are used as fasteners.

By including all fasteners parallel to the direction of the grain, the effective design load-carrying capacity $F_{v,conf1}$ can be determined as:

$$F_{v,conf1} = \frac{1}{\gamma_M} n_{b,ef} \cdot m_b \cdot F_{v,Rk,conf1} \quad (7.12)$$

with,

$$n_{b,ef} = \min \left\{ n_b, n_b^{0.9} \sqrt{\frac{d'}{13d_{SB}}} \right\} \quad (7.13)$$

$$n_b^{0.9} \sqrt{\frac{d'}{13d_{SB}}} \quad (7.14)$$

where $n_{b,ef}$ is the effective number of fasteners connecting the angle to the glulam beam in the direction parallel to the grain (Z axis in Figure 7.9), n_b is the number of bolts in the direction parallel to the width of the beam (X axis in Figure 7.9), and γ_M is the material safety factor .

Finally, the overall joint yield resistance will be governed by the attainment of yielding in the Hollow-bolt at $F_{T,Rd}$, or the formation of a mechanism in the long bolt-timber assemblage (at $F_{v,conf1}$). Therefore, in order to estimate the overall connection yield capacity, the following equation can be used:

$$M_{y,angle} = \min \begin{cases} F_{T,Rd} \cdot h_{gl} \\ F_{v,conf1} \cdot h_{gl} \end{cases} \quad (7.15)$$

$$F_{v,conf1} \cdot h_{gl} \quad (7.16)$$

where h_{gl} is is the high of the glulam beam.

7.3.1.3 Bolt Shear Failure

The expressions presented above, give a fairly accurate estimation for the determination of the ductile failure modes with significant post-yield stiffness for such type of hybrid connections. However, these predictions constitute a lower-bound estimate of the joint capacity. In some circumstances, it necessary to calculate the overall ultimate strength of the connection associated with the brittle shear bolt-row failure as observed during the tests reported in Chapter 6 for specimen S1and shown in Figure 7.10.

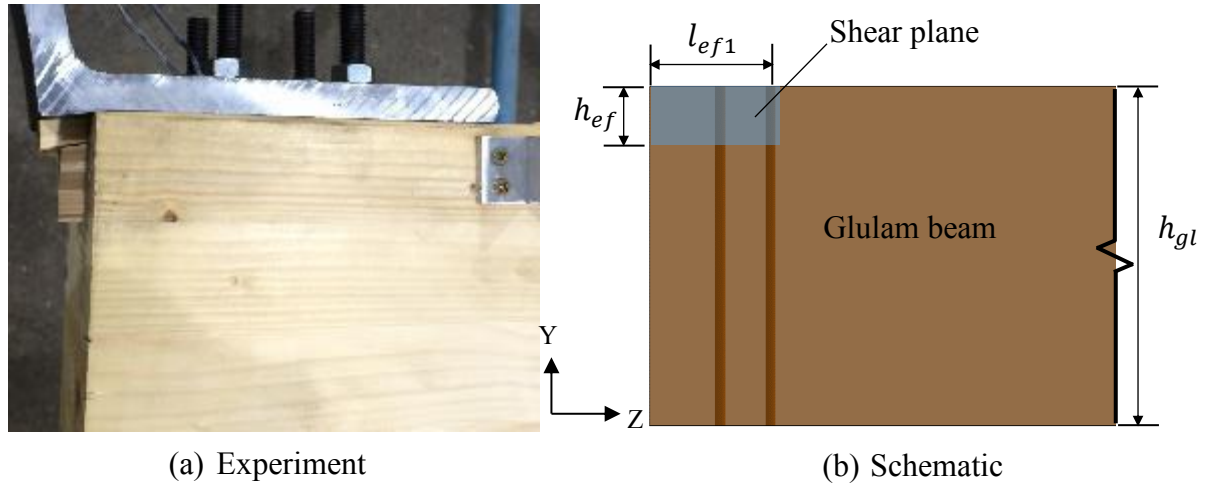


Figure 7.10 – Shear plane generation for connection of configuration 1.

The shear capacity (F_{shear}) of the tensile region of the glulam beam can be determined from the shearing force developed in the failure vertical plane shown in grey in Figure 7.10b such that:

$$F_{shear} = 2 \cdot n_{s,pl} \cdot \frac{\sigma_{12,k}}{\gamma_M} \cdot l_{ef1} \cdot h_{ef} \quad (7.17)$$

with an effective height h_{ef} dependent on the failure mode governing the plastic deformation of the joint as discussed above. Such that:

$$h_{ef} = \begin{cases} 2 \sqrt{\frac{M_{y,Rk}}{\sigma_{11,emb,k} \cdot d}} & , \text{ if Eq(7.6) or Eq(7.7) governs} \\ h_{gl} & , \text{ if Eq(7.5) governs} \end{cases} \quad (7.18)$$

$$(7.19)$$

where, l_{ef1} is the effective length, $\sigma_{12,k}$ is the characteristic shear strength of glulam and $n_{s,pl}$ is the number of bolt-rows. Hence the moment capacity at ultimate associated with brittle shear failure can be defined as:

$$M_{shear} = F_{shear} \cdot h_{gl} \quad (7.20)$$

7.3.2 Capacity estimation of Type 2 connections

Two fundamental failure modes can occur for this type of connection. That is a) bolt yielding and/or bolt embedment into the timber beam, and b) bolt-row plug shear. The capacity evaluation for both failure modes will be presented separately below followed by considering

on the influence of the reinforcing screws and the effects of adding a wedge angle. The section starts with a fundamental on the classification of forces among fasteners with a bolt-group.

7.3.2.1 Force distribution in bolt-groups

Figure 7.11 depicts a Type 2 connection under a generalized loading case expressed in terms of a moment M_d and a set of vertical F_V and horizontal F_H forces acting at C denotes the geometric centre of the bolt-group. It is worth noting that for the connections examined herein, no significant external axial actions are applied on the joint and hence the horizontal force F_H is negligible.

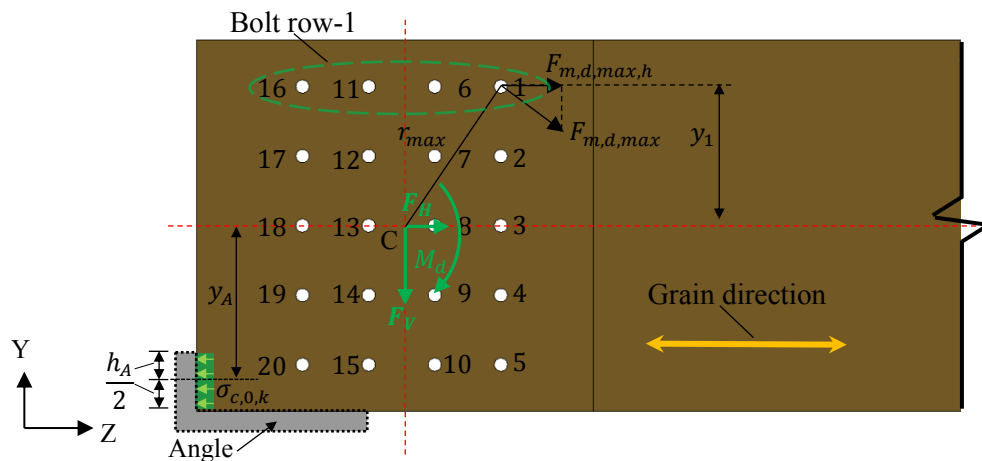


Figure 7.11 – Schematic view of a Type 2 connection under generalized loading.

It can be appreciated from that from Figure 7.11 that in the case where the point of rotation is at the centre (point C) of the bolt group, the bolts that will experience the largest demand will be the corner bolts (i.e. 1, 5, 16 and 20) since they are the bolts furthest from the point of rotation. However, when a wedge angle is inserted at the bottom of the beam, additional compressive stresses are developed in that region as illustrated also in Figure 7.11. These will result in a re-configuration of forces within the joint that moves the point of rotation towards the bottom corner of the timber beam, as discussed previously in Chapter 6. This will increase further the distance between the point of rotation and Bolt-1 which continues to be the most critical fastener of the group. Therefore, for all connection Bolt-1 will be the determinant for the joint overall capacity estimations for all connections. This conclusion can be further proven

from the results of the FE models. Figure 7.12 illustrates the deformation of the bolts for specimens S3 and S4 at the end of the final step of the simulations, whereas Figures 7.13 presents the displacement in Z-direction of the nodes located at the middle YZ-plane of the bolts. From the figures, it is evident, that when the point of rotation is at point C (no angle) the corner bolts 1, 5, 16 and 20 deform by the same amount. On the other hand, when the point of rotation shifts from the introduction of an angle, Bolt-1 exhibits the largest deformation and stress concentration.

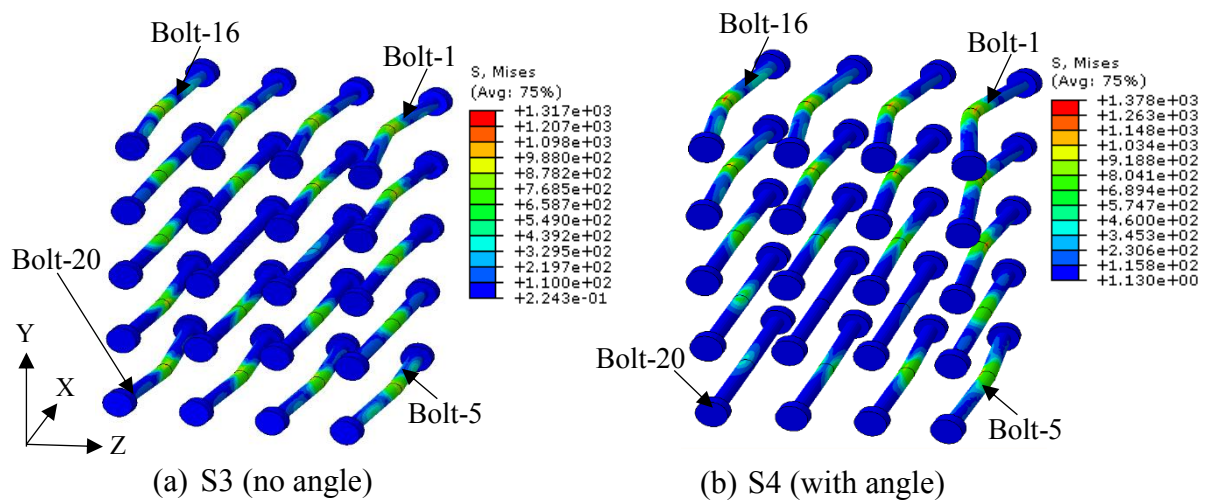


Figure 7.12 – Stress contours of bolt set for connection Type 2 at 60 mrad

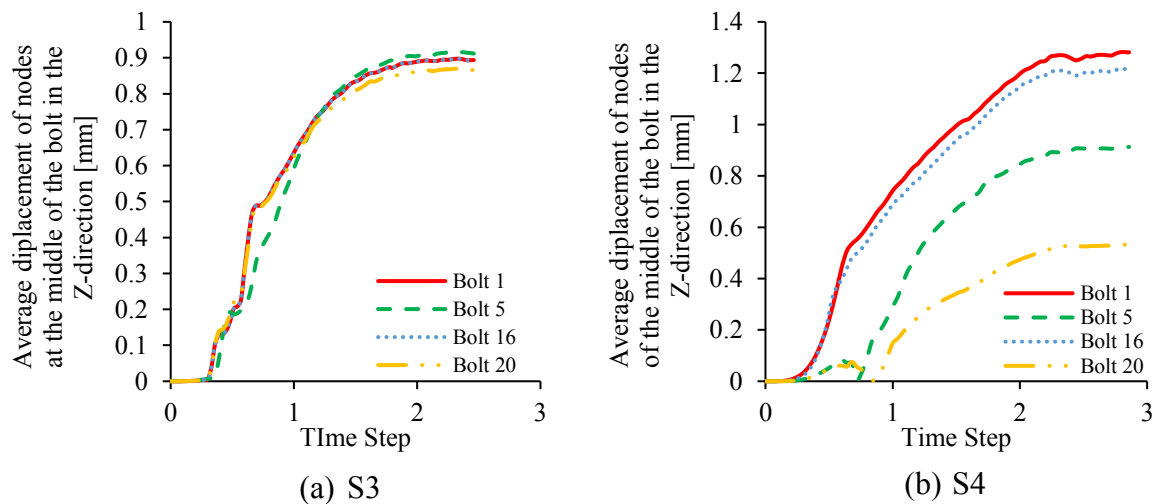


Figure 7.13 – Comparison of deformation between the corner bolts for connection type 2

The force taken by Bolt-1 ($F_{m,d,max}$) acts in a direction perpendicular to the vector r_{max} that defines the position of Bolt-1 with respect to the point of rotation C (e.g. the centroid of the bolt-group in the absence of a bottom wedge angle) as illustrated in Figure 7.11. Since Bolt-1 resists the largest force of all bolts in the group, it can be assumed that the corresponding horizontal bolt-row will be more prone to bolt row split failure than the others [38]. In this case, the total effective horizontal force $F_{d,H,row1}$ acting on Bolt Row-1 can be expressed as:

$$F_{d,H,row1} = F_{m,d,max,h} \cdot n_{row1,ef} \quad (7.21)$$

with,

$$n_{row1,ef} = \min \begin{cases} n_{row1} \\ n_{row1}^{0.9} \sqrt[4]{\frac{c}{13d}} \end{cases} \quad (7.22)$$

$$(7.23)$$

where $F_{m,d,max,h}$ is the horizontal component of the force acting on Bolt-1, n_{row1} is the number of bolts in the direction parallel to the grain, and c is the spacing between bolts as shown in Figure 6.6.

Form the above discussion, it follows that the expression relating the total effective horizontal force $F_{d,H,row1}$ with the force $F_{m,d,max}$ acting on Bolt-1 can be defined as:

$$F_{m,d,max} = F_{d,H,row1} \cdot \frac{r_{max}}{y_1 \cdot n_{row1,ef}} \quad (7.24)$$

And the moment of the connection (M_d) can be obtained as:

$$M_d = \frac{F_{m,d,max}}{r_{max}} \cdot D \quad (7.25)$$

where y_1 is the vertical distance between Bolt-1 and the centre of rotation, and D is the summation of all fasteners radii r_i , $i \in [1, n]$ squared.

7.3.2.2 Bolt group capacity

The load-carrying yield capacity of the slotted-in T-stub connection can be estimated on the basis of the critical force acting on Bolt-1 ($F_{m,d,max,h,k}$) as discussed above. To this end, the expressions suggested in Chapter 8.2.3 of Eurocode 5 [29] can be employed such that:

$$F_{m,d,max,h,k} = \min \begin{cases} \sigma_{11,emb,k} \cdot t_1 \cdot d_{SB} & (7.26) \\ \sigma_{11,emb,k} \cdot t_1 \cdot d_{SB} \left[\sqrt{2 + \frac{4 \cdot M_{y,Rk}}{\sigma_{11,emb,k} \cdot t_1^2 \cdot d_{SB}}} - 1 \right] + \frac{F_{ax,Rk}}{4} & (7.27) \\ 2.3 \sqrt{2 \cdot M_{y,Rk} \cdot \sigma_{11,emb,k} \cdot d_{SB}} + \frac{F_{ax,Rk}}{4} & (7.28) \end{cases}$$

where $\sigma_{11,emb,k}$ is the characteristic embedment strength parallel to the grain in MPa, t_1 is the distance parallel to the width of the beam between the side of the beam to the inserted metal plate in mm, and $M_{y,Rk}$ and $F_{ax,Rk}$ are the characteristic yield moment of the bolt in kNmm and its withdrawal capacity in kN, respectively.

The moment yield capacity $M_{d,y}$ of the slotted-in T-stub connection can be determined from the following expression

$$M_{d,y} = F_{m,d,max,h,k} \cdot \frac{D}{y_1} \quad (7.29)$$

where y_1 is the vertical distance between Bolt-1 and the centre of rotation C, and D is the summation of all fasteners radii r_i , $i \in [1,20]$ squared. A material safety factor, γ_M , can be employed for conservative capacity estimations.

7.3.2.3 Bolt-row plug shear failure

The capacity of the connection associated with plug shear failure (F_{shear_limit}) on the plane containing the top bolt-row (Bolt row-1 in Figure 7.11) can be estimated on the basis of the splitting resistance, as suggested by Porteous and Kermani [38]. Therefore:

$$F_{shear_limit} = \frac{2}{3} \cdot \frac{1}{\gamma_M} (w_b - t_{Tstub}) \cdot l_{ef} \cdot \sigma_{12,k} \quad (7.30)$$

with

$$l_{ef} = b + (n_{row1} - 1)(c - d) - \frac{d}{2} \quad (7.31)$$

where w_b is the width of the glulam beam, t_{Tstub} is the thickness of the metallic plate inserted into the beam, l_{ef} is the effective depth and $\sigma_{12,k}$ is the characteristic shear strength of glulam. Finally, the moment capacity for plug shear failure ($M_{d,PS}$) takes the form:

$$M_{d,PS} = F_{shear_limit} \cdot \frac{D}{y_1 \cdot n_{row1,ef}} \quad (7.32)$$

7.3.2.4 Contribution of reinforcing screws

The load carrying capacity for the two connection specimens of Type 2 where reinforcing screws were introduced, will have an additional load-carrying capacity contributed by the screws. Figure 7.14 shows the forces developed on one side of Bolt-1.

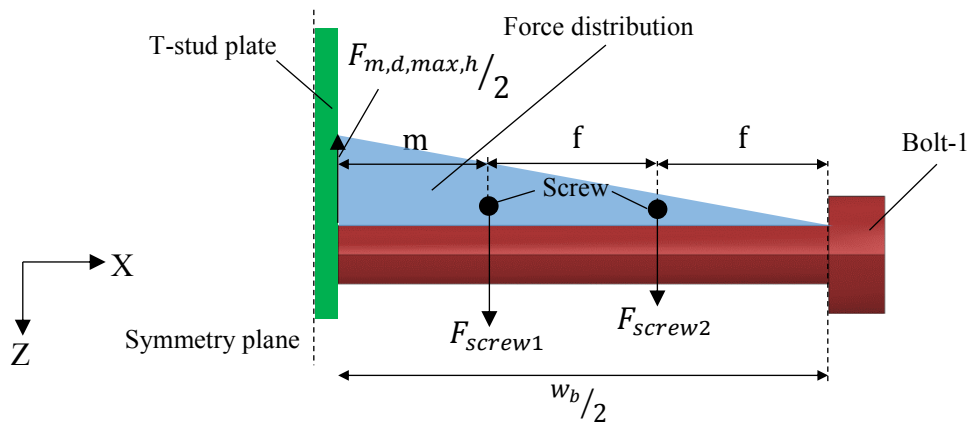


Figure 7.14 – Forces applied to Bolt-1 when screws are inserted to the beam.

By establishing equilibrium conditions on the system of forces represented in Figure 7.14, the reaction forces F_{screw1} and F_{screw2} can be found as:

$$F_{screw1} = \frac{F_{m,d,max,h}}{2} \frac{(f + m - w_b/6)}{f} \quad (7.33)$$

and

$$F_{screw2} = \frac{F_{m,d,max,h}}{2} \frac{(w_b/6 - m)}{f} \quad (7.34)$$

In the case where two adjacent bolts are reinforced by one screw, six possible failure modes should be considered, and the strength associated with screw failure, R_{VE} , will be [72,73]:

$$R_{VE} = \min[R_1, R_2, R_3, R_4, R_5, R_6] \quad (7.35)$$

with

$$R_1 = 0.5 \cdot \sigma_{emb} \cdot d_s \cdot l_s \quad (7.36)$$

$$R_2 = 0.5 \cdot \sigma_{emb} \cdot d_s \cdot l_s \cdot \left[\sqrt{\frac{16 \cdot M_{y,s}}{\sigma_{emb} \cdot d_s \cdot l_s^2} + 2 \left(\frac{e}{l_s} - 1 \right)^2} + \frac{2e}{l_s} - 1 \right] \quad (7.37)$$

$$R_3 = 0.5 \cdot \sigma_{emb} \cdot d_s \cdot e + 4 \sqrt{M_{y,s} \cdot \sigma_{emb} \cdot d_s} \quad (7.38)$$

$$R_4 = 4 \sqrt{M_{y,s} \cdot \sigma_{emb} \cdot d_s} \quad (7.39)$$

$$R_5 = 0.5 \cdot \sigma_{emb} \cdot d_s \cdot l_s \cdot \left[\sqrt{\frac{16 \cdot M_{y,s}}{\sigma_{emb} \cdot d_s \cdot l_s^2} + 2 \left(\frac{e}{l_s} - 1 \right)^2} + 4 \sqrt{\frac{M_{y,s}}{\sigma_{emb} \cdot d_s \cdot l_s^2} + \frac{e}{l_s}} - 1 \right] \quad (7.40)$$

$$R_6 = 0.5 \cdot \sigma_{emb} \cdot d_s \cdot l_s \cdot \left[\sqrt{\frac{8 \cdot M_{y,s}}{\sigma_{emb} \cdot d_s \cdot l_s^2} + \left(\frac{e}{l_s} - 1 \right)^2} - \frac{e}{l_s} + 1 \right] \quad (7.41)$$

where σ_{emb} is the embedment stress in MPa, d_s is the diameter of the screw in mm, l_s is the length of the screw in mm, $M_{y,s}$ is the yield moment of the screw section in Nmm, and e is the spacing between the bolts in mm, as shown in Figure 6.6.

In addition, two different scenarios have to be examined. The first scenario considers bolts yielding before the capacity of screws is reached, $F_{screw1} < R_{VE}$. In this case, the horizontal force acting in Bolt row-1 will be:

$$F_{d,H,row1} = F_{m,d,max,h} \cdot n_{row1,ef} + n_{row1,ef}(2F_{screw1} + 2F_{screw2}) \quad (7.42)$$

And combining Equations (7.24) and (7.42) results in:

$$F_{m,d,max} = (F_{m,d,max,h} + 2F_{screw1} + 2F_{screw2}) \frac{r_{max}}{y_1} \quad (7.43)$$

Alternatively, the second scenario involves the screws yielding before the capacity of the bolts is reached, $F_{screw1} \geq R_{VE}$. Hence, the force $F_{m,d,max,h}$ transmitted to the bolts can be determined by rearranging Equation (7.33) and assuming $F_{screw1} = R_{VE}$ such that:

$$F_{m,d,max,h} = \frac{2 \cdot f \cdot R_{VE}}{(f + m - \frac{w_b}{6})} \quad (7.44)$$

Consequently, by combining Equations (7.24) and (7.42), the force resisted by Bolt-1 can be estimated as:

$$F_{m,d,max} = (F_{m,d,max,h} + 2R_{VE} + 2F_{screw2}) \frac{r_{max}}{y_1} \quad (7.45)$$

Finally, it is worth noting that the connection resistance to bolt-row plug shear failure is increased by the presence of screws and these effects can be incorporated by employing standard calculation procedures described in previous studies [38, 72, 73].

7.3.2.5 Consideration of wedge angle effect

Connections that incorporate a bottom wedge angle between the timber beam and the SHS column have an increased moment capacity with respect to connections that do not have that detail. In order to consider these effects in a simplified manner while ensuring conservative estimates, the total moment capacity M_T of such connections can be idealised as the summation of independent contributions of the original detail, $M_{connection}$, plus the moment increment associated with the wedge angle contribution, $M_{d,angle}$:

$$M_T = M_{connection} + M_{d,angle} \quad (7.46)$$

where $M_{connection}$ can be obtained by means of the corresponding expressions presented above, with due consideration of the modified location of the point of rotation, as discussed section 7.3.2.1. On the other hand, the moment capacity contribution of the wedge angle can be calculated from the additional bearing resistance generated at the bottom of the timber beam:

$$M_{d,angle} = \sigma_{11,c,k} \cdot h_A \cdot (w_b - t_{Stub}) \cdot y_A \quad (7.47)$$

where $\sigma_{11,c,k}$ is the characteristic wood compression strength parallel to the direction of the grain, h_A is the length of the smaller leg of the angle excluding its thickness, and y_A is the distance between the point of rotation and the centroid of the bearing zone (see Figure 7.11).

7.3.3 Stiffness estimation

7.3.3.1 Type 1 connection

The initial stiffness of the top (K_{Top}) and seat (K_{Bottom}) angle components of Type 1 connections can be evaluated from the added contributions of the bearing stiffness of the column face (K_{CF}), the Holo-bolt axial stiffness (K_{HB}) and the tension stiffness of the angle horizontal leg (K_t) acting in series as:

$$K_{Top} = \frac{1}{\frac{1}{K_{ser}} + \frac{1}{K_{CF}} + \frac{1}{K_{HB}} + \frac{1}{K_t}} \quad (7.48)$$

$$K_{Bottom} = \frac{1}{\frac{1}{K_{CF}} + \frac{1}{K_t}} \quad (7.49)$$

where K_{CF} , K_t and K_{HB} can be taken from the corresponding expressions available from previous studies [4, 94]. For simplicity, a pre-defined point of rotation can be assumed at the horizontal leg of the bottom angle for top and seat angle connections. The rotational stiffness $S_{config1,init}$ can be estimated by applying the following expression given by Pucinotti [96]:

$$S_{config1,init} = \frac{h_{gl}^2}{\frac{1}{K_{Top}} + \frac{1}{K_{Bottom}}} \quad (7.50)$$

7.3.3.2 Type 2 connection

For Connection Type 2, the rotational stiffness, $S_{config2}$, of the connection can be estimated following the suggestion of Porteous and Kermani [38] as:

$$S_{config2} = 2 \cdot K_{ser} \cdot \sum_{i=1}^n r_i^2 \quad (7.51)$$

where, K_{ser} is the slip modulus of each individual bolt in N/mm]:

$$K_{ser} = \rho^{1.5} \cdot \frac{d}{23} \quad (7.52)$$

7.3.4 Validation of mechanical model

The estimations of stiffness and flexural strength obtained using the expressions presented in the sections above for Type 1 and Type 2 connections were validated against the corresponding experimental moment-rotation curves. These comparisons are presented in Figures 7.14 and 7.15. It is worth mentioning that the material values used in the component-based expressions were the ones obtained in Chapter 6. Besides, in order to provide a direct validation a nominal shear strength value of 3.2 MPa was employed. Similarly, all material safety factors were assumed as unity for the purpose of these comparisons. Finally, the ideal locations of the point of rotation calculated as described in section 7.3.2.1.

For the Type 1 connections shown in Figure 7.15, it is worth noting that the initial yield is governed by formation of a plastic mechanism in the angle components for Specimen S2, while for specimen S1 the initial yield is governed by yielding of the blind-bolts. Also, Specimen S1 experienced an ultimate failure mode governed by bolt-row shear failure in the top tension region of the joint that can be conservatively estimated using Equation (7.20). In both cases, an accurate estimation of the connection stiffness is obtained as well as a conservative prediction of the connection capacity as evident from Figure 7.15.

For the second connection type, presented in Figure 7.16, it is clear that the suggested component model offers a realistic and conservative estimate of the joint initial stiffness and yield moment capacity. In particular, in all cases the initial stiffness matches the experimental results with an accuracy of $\pm 10\%$. However, Figures 7.16a and 7.16b demonstrate that the moment capacity estimation for plug-shear failure $M_{d,PS}$ as described by Equation (7.32) overestimates the fracture capacity of the connections by 15% and 28% respectively. Likewise, the assumptions of rigid motion and equivalent bolt-row areas for the angle wedge, as explained above, seem to provide an adequate basis for estimating the point of rotation of the connection. It is clear from Figures 7.15 and 7.16 that the suggested component based approach provides reasonable representations of the stiffness and capacity in all cases and can be used for parametric studies and further design-oriented explorations.

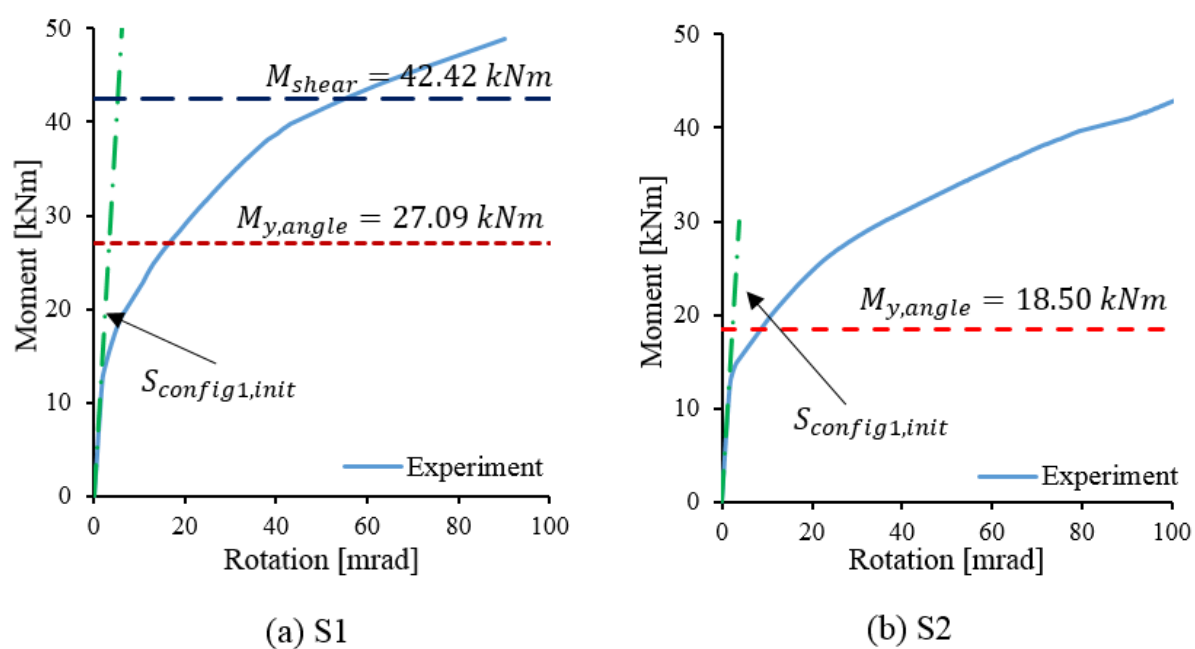


Figure 7.15 – Comparison of moment-rotation response between experimental tests and component-based estimation for connection configuration 1.

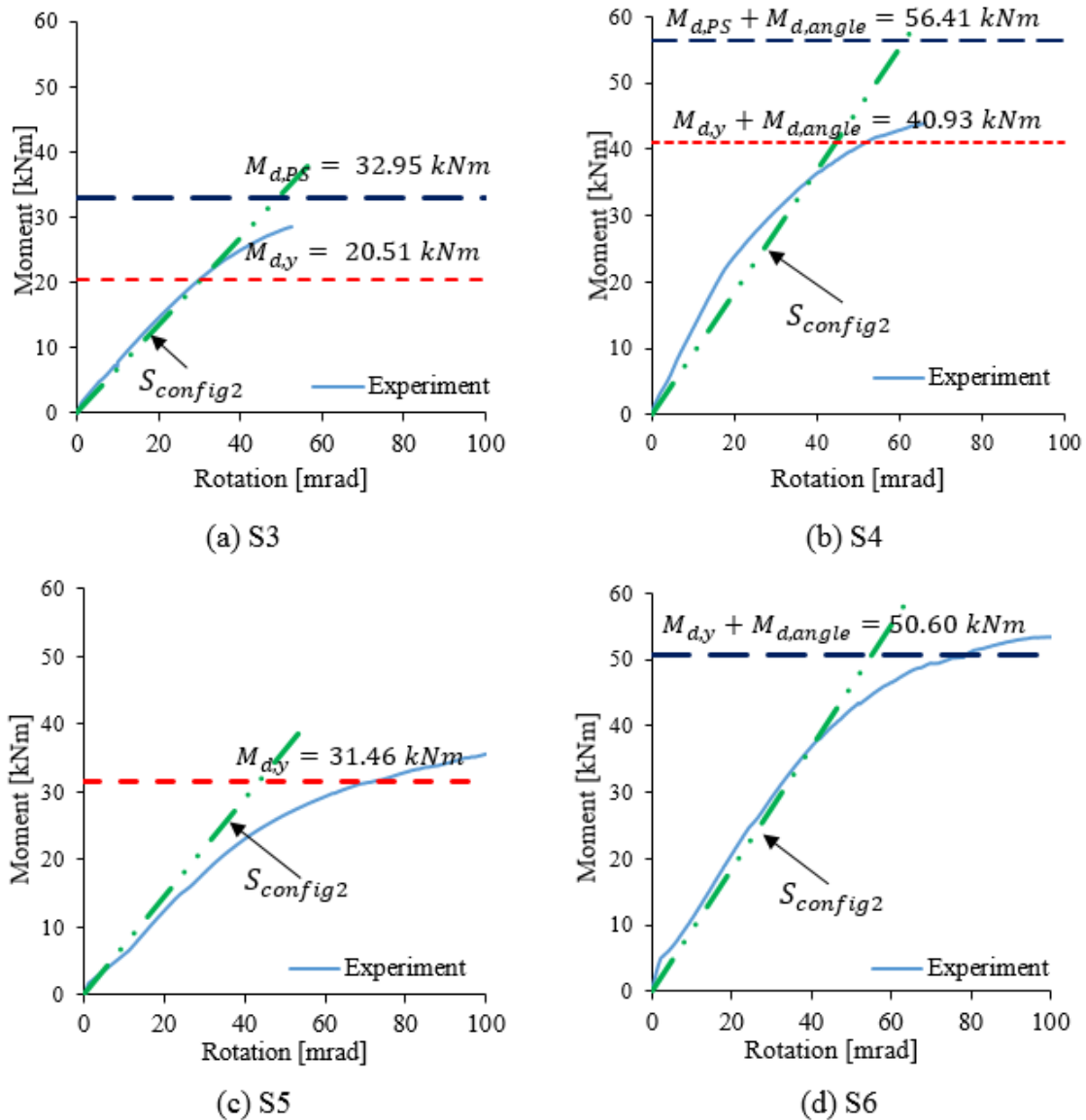


Figure 7.16 – Comparison of moment-rotation response between experimental tests and mechanical model for connection configuration 2.

7.3 Concluding remarks

This chapter proposed and validated continuum-based numerical modelling and component-based approaches for the estimation of the hybrid timber beam-to-steel column connections tested in Chapter 6. To this end, a detailed explanation of the modelling assumptions and necessary parameters was presented and the modified foundation model, discussed in Chapter 5, was employed to incorporate the effects of localized wood crushing due to fastener embedment. The material models implemented into the parts of the numerical models were

obtained from the experimental programme described in Chapters 3 and 4. Furthermore, this chapter has proved that the implementation of the foundation zone as a volume encompassing the group of bolts as presented in Figure 7.3 provides an accurate and efficient way of including damage into FE simulations. Besides, the results obtained by means of this FE representation compared well with the experimental response as it can be observed from Figures 7.4 and 7.6. Importantly, the deformation patterns presented in Figures 7.5, 7.7, 7.12 and 7.13, were also well predicted by the FE models proposed herein. The agreement between the FE simulations and the test results showed that the proposed FE models can capture the actual response of both connection types.

Regarding the applicability of component-based approximations for Type 1 connections, Equations (7.1) – (7.3) describe the three distinct failure modes for the steel bolt-angle components whereas equations (7.5) – (7.7) describe the failure modes for the bolt-timber interaction. Accordingly, equations (7.1) – (7.3) were taken from EC 3 [34] and expressions (7.5) – (7.7) are found in EC 5 [29]. Although the mentioned expressions are already available in codified guidance, their application is non-standard for the type of connection with long bolts under study. To this end, this thesis has proposed the range of failure modes to which the connection is susceptible (Figure 7.9) and defined the way in which prior knowledge from component modelling approximations can be used by means of Equations (7.8) to (7.16). Furthermore, although expressions for block shear strength estimations are readily available from codes (Equation 7.17) their application to the non-standard connection Type 1 necessitates the definition of an effective height for which Equations 7.18 and 7.19 are proposed herein. All expressions were converted to moment by assuming that the point of rotation was located at the bottom corner of the timber beam.

With regards to connection Type 2, without the introduction of angle and screws. This thesis built upon equations (7.26), (7.27) and (7.28) of Eurocode 5 [29] and elsewhere [38]. On the other hand, the work conducted as part of this thesis has allowed the formulation of component based models for the cases where screws were introduced (Specimens S3 and S4). To this end, the forces acting on the screws can be estimated by assuming that the force distribution acting on the bolts have the shape of a right-angled triangle (see figure 7.14 and equations 7.33 and 7.34). Accordingly, new expressions have been proposed for the determination of forces acting on 1 bolt-row based on two scenarios (Equations 7.43 and 7.44). Besides, new component based formulations are offered for cases where a wedge angle was placed in the bottom corner

of the connection (specimens S2 and S4, Equation 7.47 and its references). In cases where the bearing connection of the connection needs to be increased, a wedge angle was proposed and the corresponding component-based formulations have been derived in this thesis.’

These newly formulated expressions were found to produce reliable and mostly conservative estimates of the connection stiffness and yield capacity. Finally, the information gathered from the experimental and numerical results and observations presented so far, provide essential data for the parametric studies conducted in the next chapter that aim to evaluate the proposed connections for a range of dimensions and configurations examined in this chapter.

Chapter 8

Practical application and design considerations

8.1 Introduction

Chapters 6 and 7 have studied experimentally and numerically the response of two new connections for joining glulam timber beams and tubular steel columns, namely: i) blind-bolted top and seat angle connections with long bolts (Type 1), and ii) slotted in T-stub connections (Type 2). These developments have been underpinned by an extensive investigation into the mechanical properties of wood and efficient modelling approaches for dowel embedment described in earlier chapters of this thesis. The results presented have demonstrated the structural suitability of these new connections as well as the positive effects of simple reinforcement techniques such as nailing perpendicular to grain or the provision of bottom wedges in T-stub joints. Simplified component-based expressions for the estimation of the flexural capacity and stiffness of these connections have also been introduced and validated.

The present chapter builds upon these findings and aims to illustrate the use of hybrid timber-steel blind-bolted top and seat angle connections as well as slotted in T-stub connections over a wider range of geometric parameters and beam or column dimensions beyond those already examined in previous chapters. The list of parameters for Type 1 connection are: column size, beam/angle width, beam depth, long-bolt diameter and number of long bolts per row while the list of parameters for Type 2 connection are: beam width, beam depth, distance between bolts (c), T-stub thickness, bolt diameter and bolt strength grade. Starting with a realistic building case study, the range of variations were selected to cover the practical limit cases for a given parameter. For example, beam width was varied from 90 to 185 for the Type 2 connection which are thought to be the smallest and largest section sizes for a typical 5-storey building. Similarly, beam depths between 280 and 360 for Type 2 connection were analysed to cover the reasonable extremes of shallow and deep beams encountered in practice. The same for other parameters. This approach aligns with the aim of this chapter which is to shade light on the practical use of the connections previously studied within realistic geometric configurations in bending.

The chapter starts by discussing the moment-rotation responses obtained from the experiments described in Chapter 6 in light of the Eurocode 3 connection classification framework [34], this encompasses the presentation of two reference connection configurations that are defined on the basis of typical designs for mid-rise buildings. Subsequently, a numerical investigation is carried out into the effects on the connection's overall flexural response of possible variations in the beam, column, and connection component dimensions and strength characteristics. These results are presented separately for Type 1 (top and seat angle) and Type 2 (T-stub) connections with particular attention to the joint rotational stiffness and strength. The chapter finishes with a discussion on the implications of the results gathered and on the applicability of the component-based formulations for the range of connections studied.

8.2 Connection classification and reference cases

Figure 8.1 below, shows the response of top and seat angle connections (Type 1) as obtained experimentally in Chapter 6 together with the Eurocode connection classification limits [34] for an unbraced frame situation. Evidently, the limits of rigid and pinned behaviour presented in Figure 8.1 are dependent on the section of the beam attached to the connection. For example, Eurocode 3 [34] classifies connections based on their normalized stiffness (in relation to the beam stiffness) and normalized capacity (in relation to the beam capacity). Also presented in Figure 8.1, is the numerically estimated moment-rotation relationship of the 'reference' joint. This reference case corresponds to a joint at the bottom storey of a hypothetical 5-storey braced frame with tubular steel columns and timber flooring systems. Table 8.1 presents the dimensions of the reference beam-column connection together with the timber and steel elements being connected. The frame consists of six 6-metre wide bays and 5 storeys of 3 metre high. It was designed according to Eurocodes 3 and 5 [29,34] considering a dead load of 6 kN/m² and superimposed of 2 kN/m². This reference case and its corresponding connections will serve as the basis for the parametric studies that follow in the following sections of this chapter. By considering possible practical ranges of GL28h glulam beams e.g. [6000, 9000 mm] the stiffness of Type 1 connections ranges from 7.2 to 7.3 kNm/mrad which classifies them as nominally pinned.

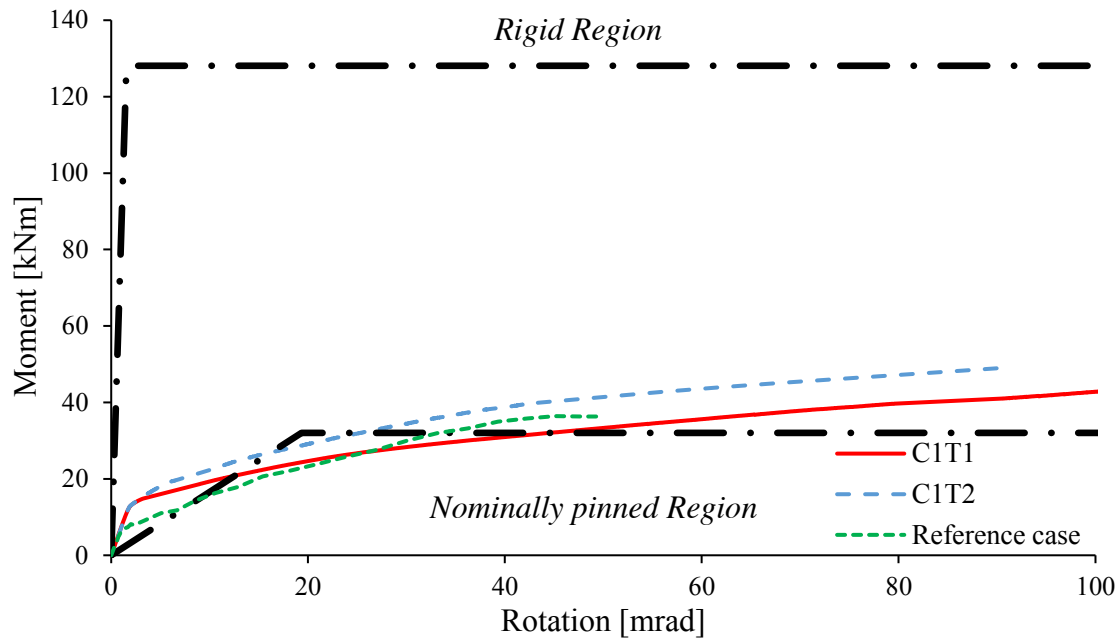


Figure 8.1 – Eurocode 3 [34] connection classification for Type 1 connection.

Column	Beam	Bolt diameter	Angle	Angle dimensions (as shown in Fig. 6.5)						
				a'	b'	c'	d'	e'	f'	g'
400 x 400 x 16	405 x 140	12	L 200 x 150 x 15	36	68	56	60	84	116	34

Table 8.1 – Summary of the reference Type 1 connection. Dimensions in

Figure 8.2 depicts the moment-rotation relationships of all Type 2 connections tested in Chapter 6 together with the corresponding relationship for the Type 2 ‘reference case’ connection obtained by means of a FE simulation. As with Type 1, the reference case for Type 2 connections corresponds to the ground-level beam to column joint of a 5-storey six-bay steel frame. The frame has 6-m long beams and 3-m high SHS columns and is formed by six bays. The design was carried out according to Eurocode [29, 34] with a permanent load of 6 kN/m² and a superimposed load of 2 kN/m² while the numerical simulation followed the same procedure outlined previously in Chapter 7. Importantly, in this case, the semi-rigid behaviour of the connections was approximately taken into account in the design. Table 8.2 presents the dimensions of the reference beam-column connection and elements. It can be observed from Figure 8.2 that the T-stub hybrid connections offer reasonable levels of stiffness and capacity within the range of typical semi-rigid connections. If 6000-mm long glulam beams of strength

class GL28h are considered, Type 2 connections would have stiffness in the order of 2 kNm/mrad which lies firmly within the semi-rigid range.

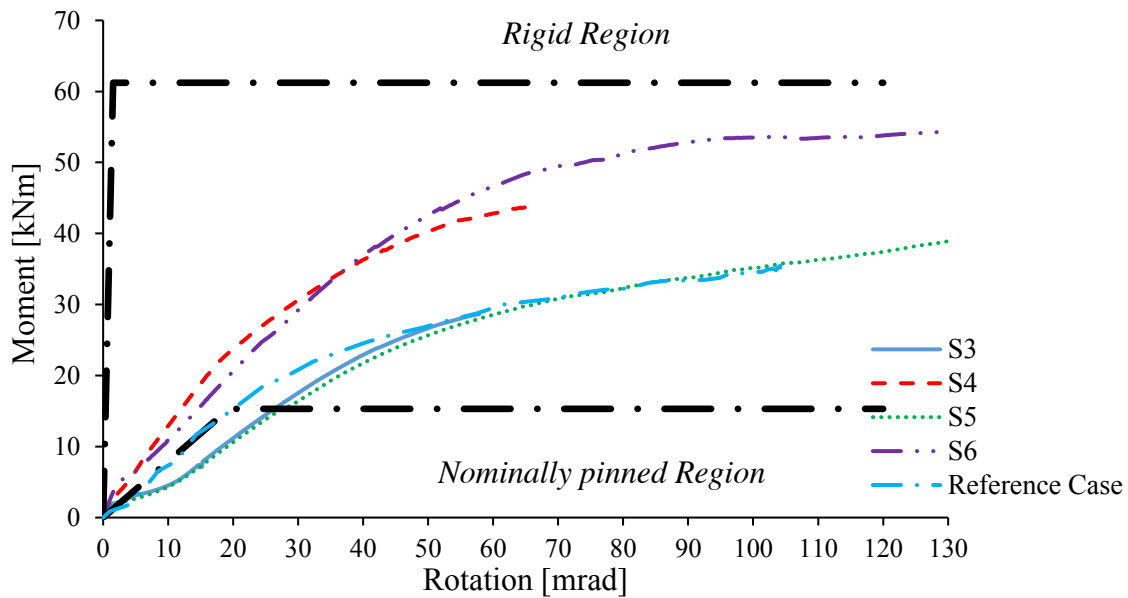


Figure 8.2 – Eurocode 3 [34] connection classification for Type 2 connection.

Column	Beam	Bolt diameter	Bolt grade	T-stub thickness	Dimensions (as shown in Fig. 6.6)				
					a	b	c	d	e
400 x 400 x 16	280 x 140	10	10.9	10	20	80	50	35	52.5

Table 8.2 – Summary of the reference Type 2 connection. Dimensions in mm.

It should also be noted that in addition to the estimations of connection stiffness and capacity, the joint ductility is a key design consideration. In that regards, the connections examined in Chapter 6 exhibited failure rotations exceeding 57 mrad for unreinforced configurations. These ultimate rotation levels are well above normal requirements for monotonic or cyclic loading conditions. Also, these ductility levels observed in Chapter 6 were significantly influenced by the presence of reinforcing screws in Type 2 connections which prevented brittle bolt-row plug shear failure and extended the rotation capacity of the connection beyond the reach of the actuator stroke at 130 mrad.

8.3 Parametric studies

Having gained confidence in the ability of the detailed Finite Element (FE) models developed in Chapter 7 to represent the flexural response of the hybrid timber-to-steel moment connections under study this section presents the results of numerical simulations performed in order to illustrate the effects of various connection geometries and element sizes on the joint response. A comparative assessment is also conducted between the FE simulations and the corresponding simplified component-based model estimations.

In order to facilitate the parametric analyses a FOTRAN code was implemented. This code operates in the *journal file* (JNL file) of ABAQUS recreating the FE models with the specified values of the parameters under consideration. The foundation zone approach presented in Chapter 5 was used in all models here presented. Similarly, the “*explicit solver*” of ABAQUS was utilised in order to overcome the complexities of the multiple contact conditions and care was taken to ensure that the load application was always quasi-static. The following sections deal with the results of the analyses performed in Type 1 (top and seat angle) and Type 2 (T-stub) connections separately.

8.3.1 Top and seat angle connections (Type 1)

In the case of Type 1 connections, the parameters studied include the column section size, beam width, beam height, long bolt diameter and the number and disposition of long bolts. As discussed before, the response of Type 1 connections is typically associated with a low rotational stiffness characteristic of pinned joints. Therefore, the analyses performed herein were aimed at identifying if any reasonable change in the above-mentioned parameters, starting from the ‘reference case’ introduced in the last section, was likely to induce a significant change in its flexural behaviour while preventing all forms of brittle failure governing the capacity estimations. Attention was placed on two key structural response parameters, namely: (a) stiffness, and (b) connection yield moment. A bi-linear idealization was employed to obtain the stiffness and yield capacity of the connections from the FE moment-rotation simulations. On the other hand, equations (7.16) and (7.50) (Chapter 7) were employed to determine the corresponding component-based yield moment and stiffness estimations, respectively. In the sub-sections that follow, all connection characteristics are kept constant and only one parameter (the one under consideration) is varied at a time.

8.3.1.1 Column size

In addition to the reference case defined in Table 8.1 (Section 8.2) which incorporated a 400x400x16 SHS (with a width over thickness ratio of $b/t = 25$), a Type 1 connection with a 150x150x10 SHS ($b/t = 15$) column was also analysed. All other parameters remained the same. Figure 8.3, presents the response of the connection obtained from the two FE models. It can be observed from this figure that the connection with a smaller column face slenderness ($b/t=15$) achieved a slightly higher initial rotational stiffness but almost identical yield moment. Some very small differences are present in the post-elastic stages but no clear effects can be identified. The same observations can be made with reference to Figures 8.4 and 8.5, that show the influence of the column face slenderness on the yield moment and stiffness obtained from the FE and component models. The ratio between the value obtained from the FE model and the corresponding component model is also presented in these figures.

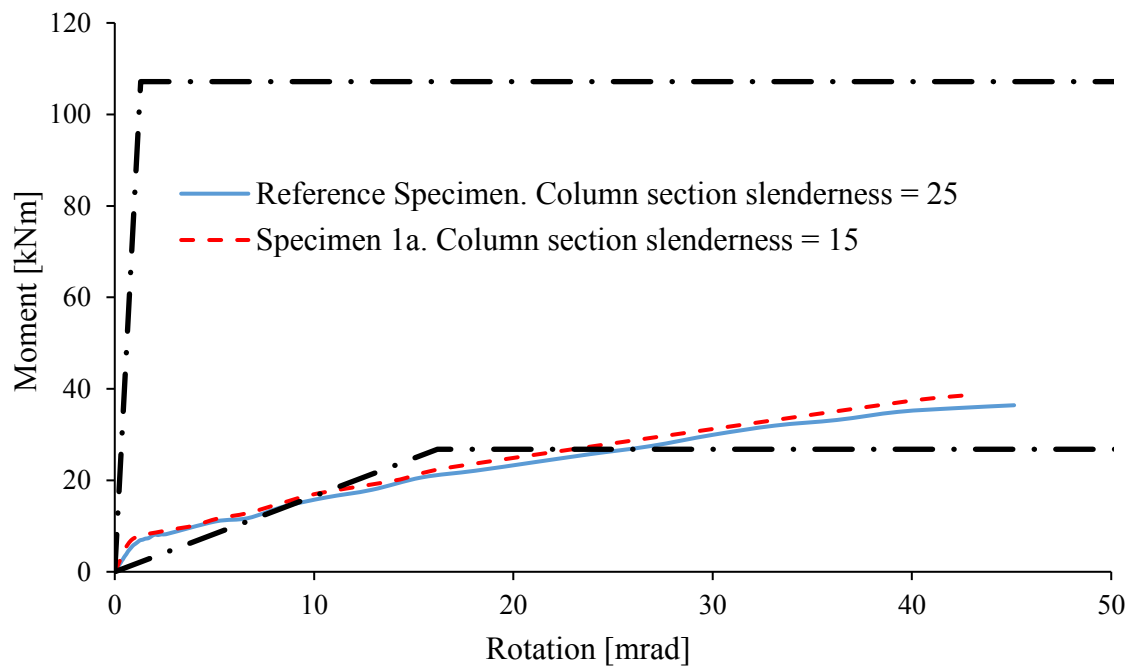


Figure 8.3 – Influence of column section slenderness on the moment-rotation response for connection Type 1, including classification limits..

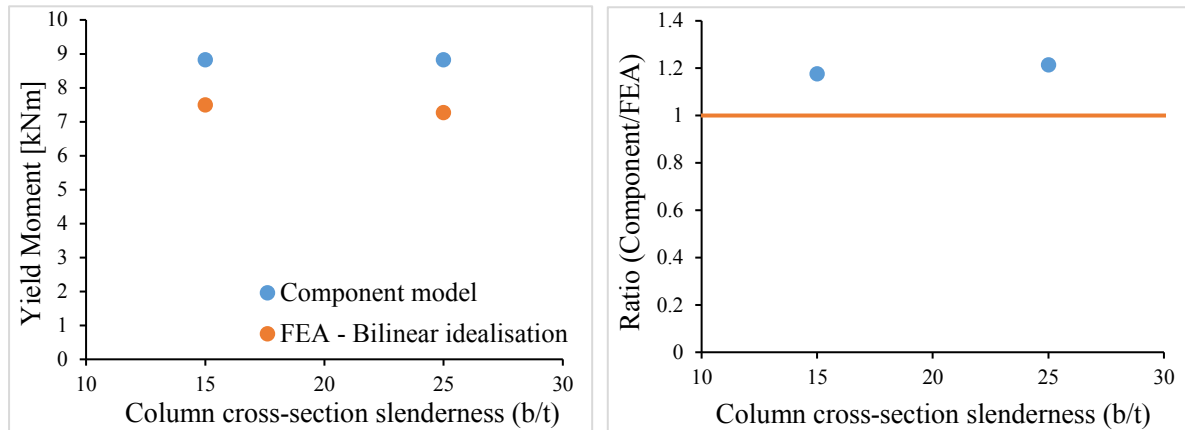


Figure 8.4 – FE and component model estimation of the yield moment by varying column cross-section slenderness for Type 1 connection.

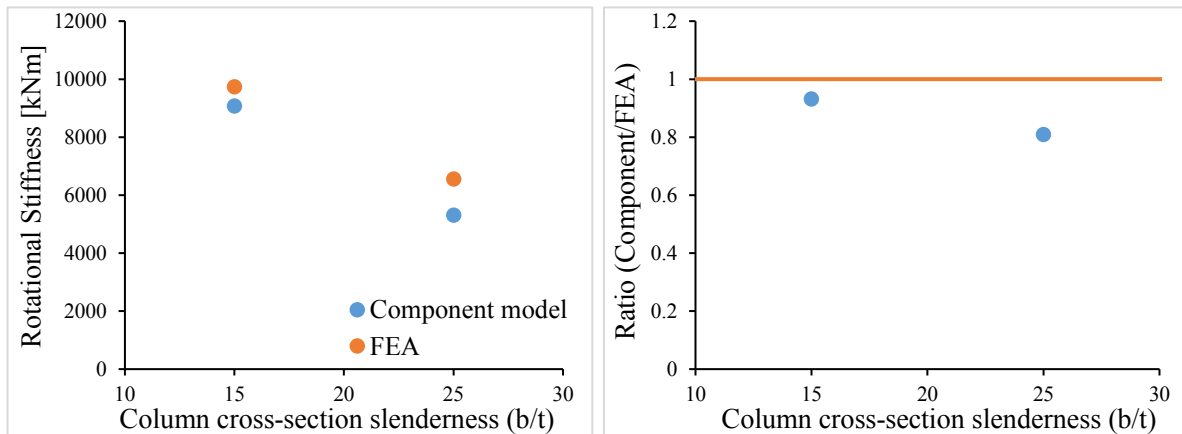


Figure 8.5 – FE and component model estimation of the initial rotational stiffness by varying column cross-section slenderness for Type 1 connection.

From the results presented in Figures 8.3 to 8.5, it can be observed that both the FE and component models present a consistent evolution of stiffness and capacity for different values of column face slenderness. A 48.5% increase in stiffness is associated with a change in the column face slenderness from 25 to 15. This effect is adequately accounted for in the component-based formulation and a discrepancy of less than 20% is evident between the numerical and mechanical calculations. Similarly, the yield moment capacity increases by 3.2% with a change in the column face slenderness from 25 to 15. and a discrepancy of less than 4% is evident between the numerical and mechanical calculations.

8.3.1.2 Beam/angle width

The effect of the beam/angle width on the behaviour of the connection was examined by analysing the response of connections with two different beam and angle widths, i.e. 140 mm and 185 mm. In line with common practice, it is assumed that both the beam and angle width are varied simultaneously. Figure 8.6, presents the response of the connection obtained from the FE models. It can be observed from this figure that increasing the width of the angle from 140 to 185 mm produces an increases of 24.85% in the yield capacity and negligible change in stiffness of the connection.

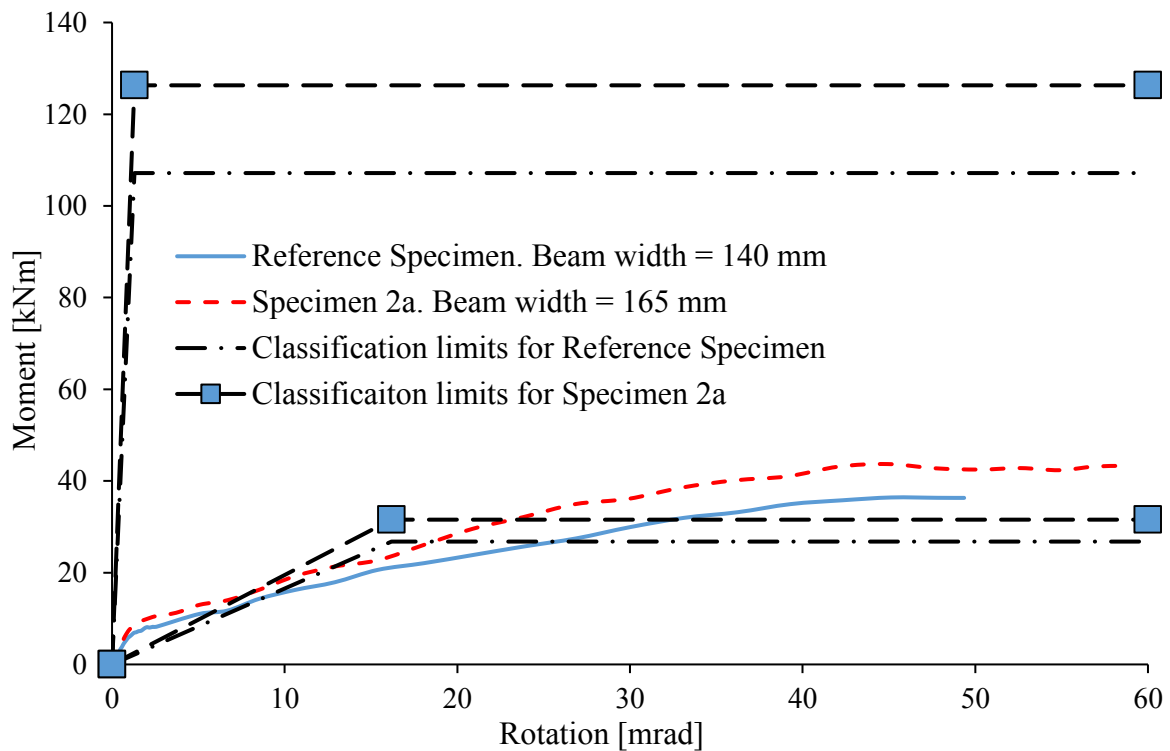


Figure 8.6 – Influence of beam's width on the moment-rotation response for connection Type 1, including classification limits..

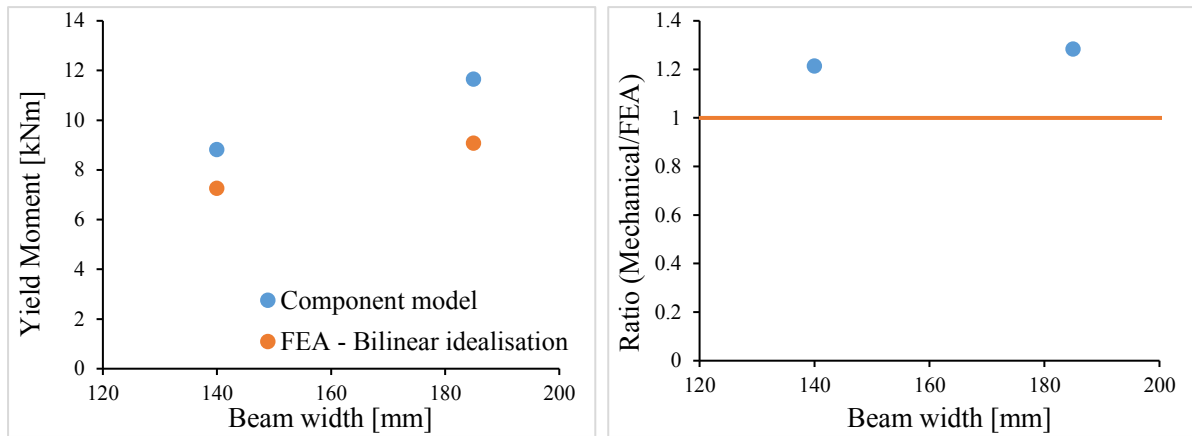


Figure 8.7 – FE and component model estimation of the yield moment by varying the beam's width for Type 1 connection.

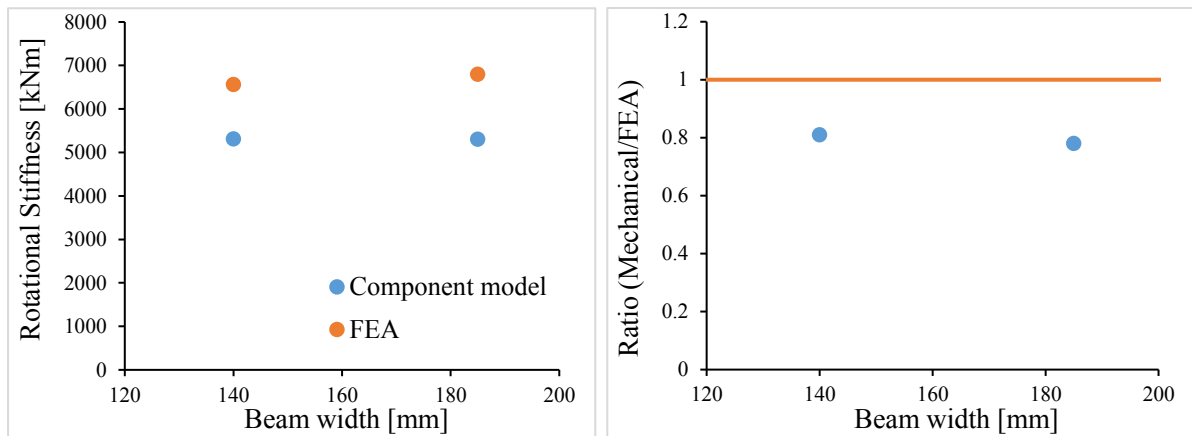


Figure 8.8 – FE and component model estimation of the initial rotational stiffness by varying the beam's width for Type 1 connection.

Figures 8.7 and 8.8 show the influence of the beam's width on the yield moment and rotational stiffness obtained from the FE and component models, respectively. From the results of the components models presented in Figures 8.6 to 8.8, it is estimated that the width of the glulam beam affects only the yield moment of the connection. These effects were adequately predicted from the component-based formulation and a discrepancy of less than 10% and 4% is evident between the numerical and mechanical calculations of the yield moment and rotational stiffness, respectively.

8.3.1.3 Beam depth

In order to study the effects of increasing the beam depth, beams 405 mm and 495 mm deep were examined. Figure 8.9, presents the response of the connection obtained from the FE models. It is evident from this figure that the glulam beam depth affects the yield capacity and the stiffness of the connection through the provision of a longer rotational lever arm between the bottom angle (centre of rotation) and the top angle when a longer depth is employed. Further to this, Figures 8.10 and 8.11 show the influence of the beam height on the yield moment and stiffness obtained from the FE and component models, respectively. From the results presented in Figures 8.10 to 8.11, it can be observed that both the numerical and component models capture the effects of this parameter on the joint response. The FE models predicted a 20% increase in the yield-moment and a 38% increase in the rotational stiffness of the connection for the 90 mm increase in beam depth. The effects of the beam's depth on the yield moment and rotational stiffness are also predicted from the component-based formulation and a discrepancy of less than 2% and 10% is evident between the numerical and mechanical calculations of the yield moment and rotational stiffness, respectively.

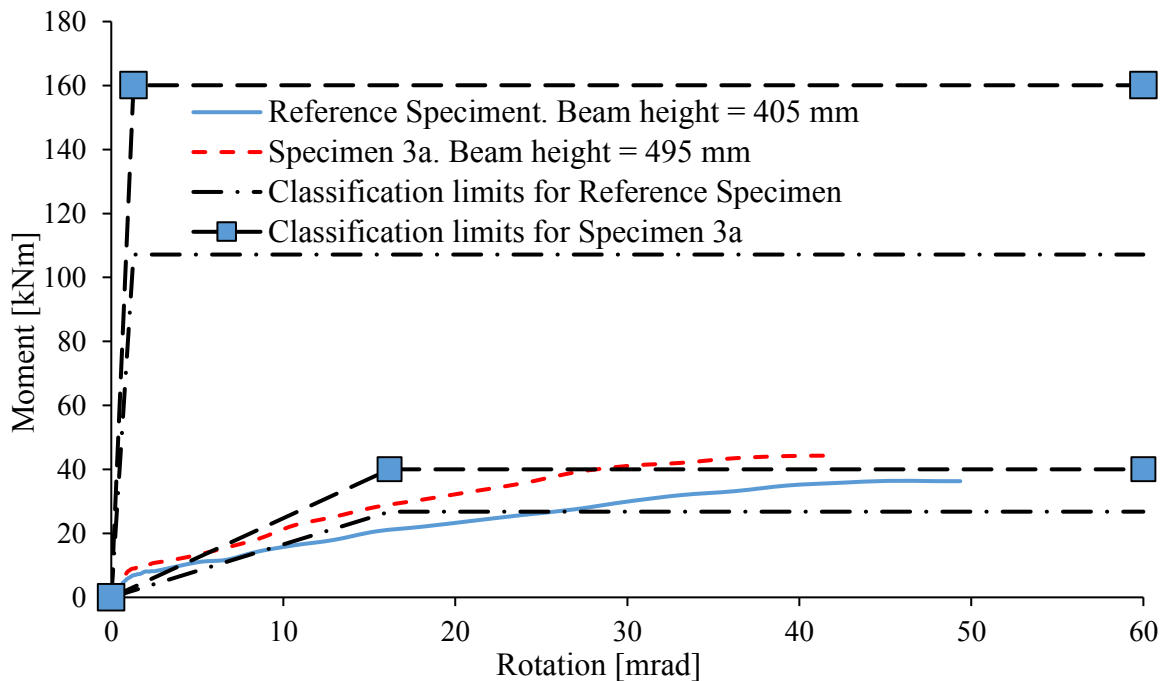


Figure 8.9 – Influence of beam's height on the moment-rotation response for connection Type 1, including classification limits..

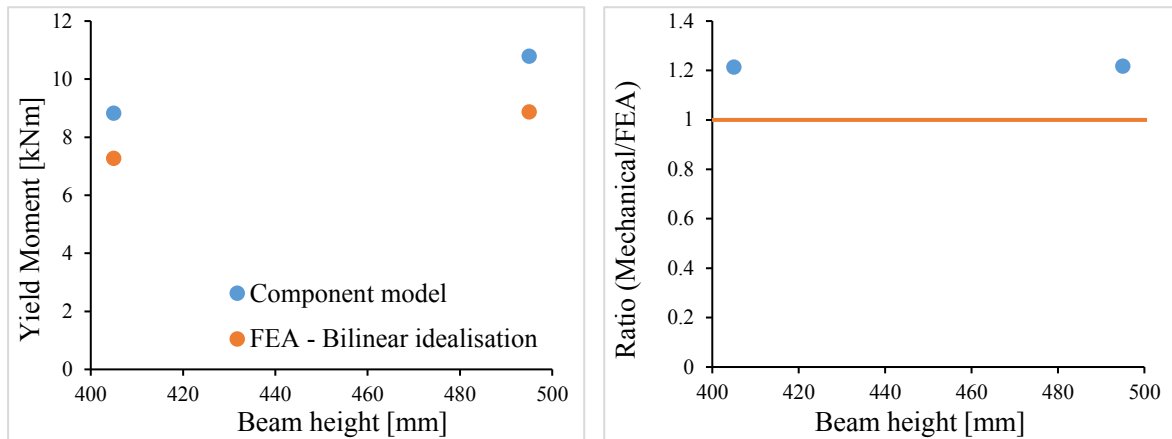


Figure 8.10 – FE and component model estimation of the yield moment by varying the beam's height for Type 1 connection.

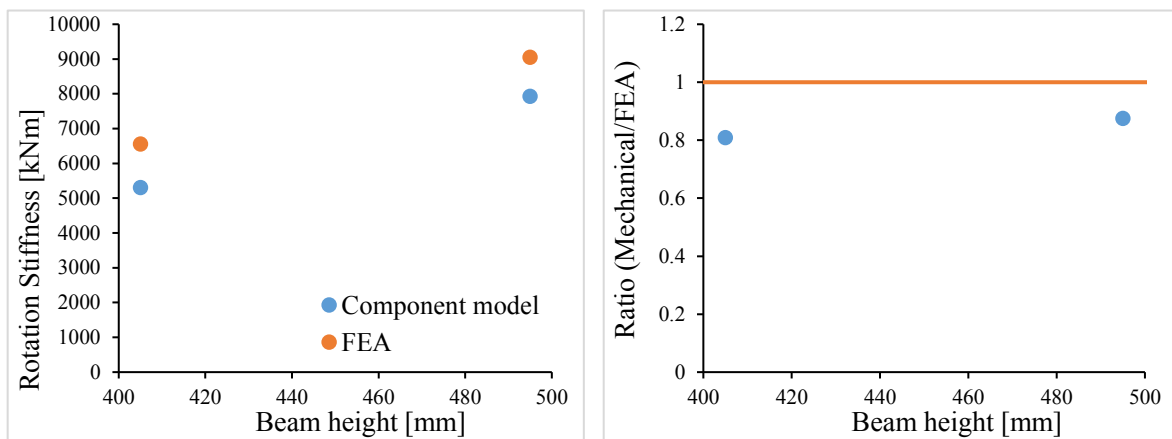


Figure 8.11 – FE and component model estimation of the initial rotational stiffness by varying the beam's height for Type 1 connection.

8.3.1.4 Long bolt diameter

A beam-column connection with a long bolt diameter of 10 mm was simulated to compare the moment-rotation response with the curve obtained from the reference case which incorporates a 12 mm bolt. Figure 8.12, presents the response of the connection obtained from the FE models representing both connections. By comparing the two moment-rotation curves, it can be observed that this level of change in the bolt diameter has negligible affects in the yield moment and rotational stiffness of the connection. Figures 8.12 to 8.14, shows the influence of the bolt diameter on the yield moment and stiffness obtained from the FE and component models. The only difference, between the two responses is observed after 30 mrad of rotation, where the capacity of the connection with the thinner bolts decreases due to the accumulation of significant bending in the smaller 10 mm bolts.

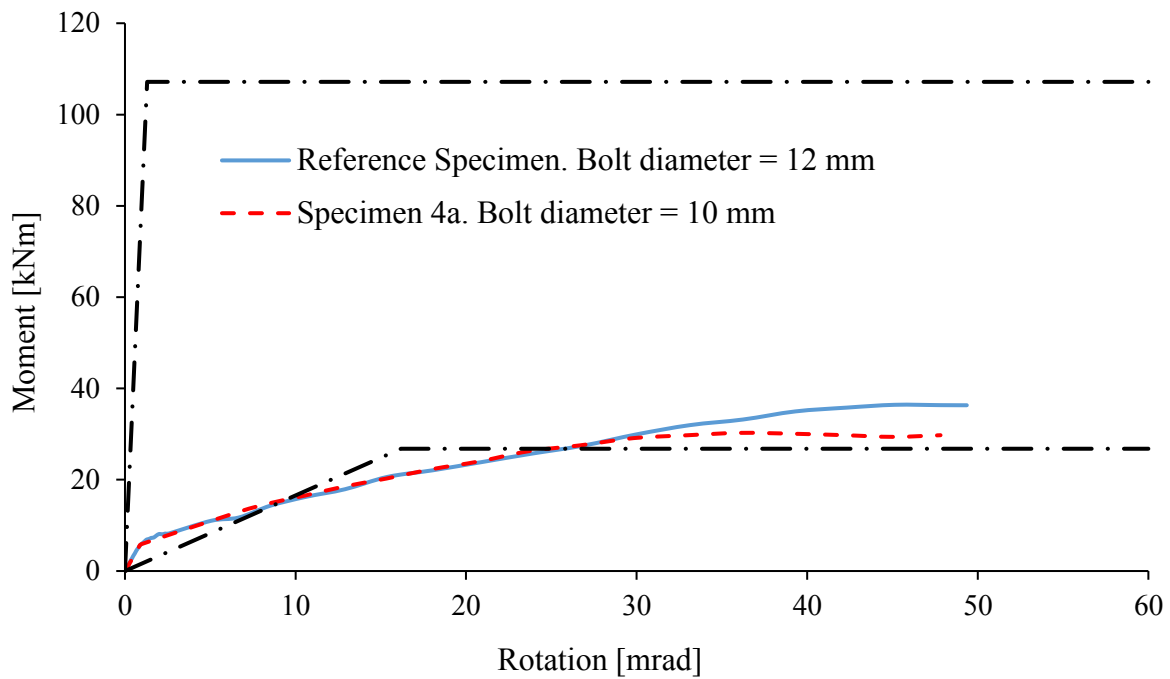


Figure 8.12 – Influence of bolt diameter on the moment-rotation response for connection Type 1, including classification limits..

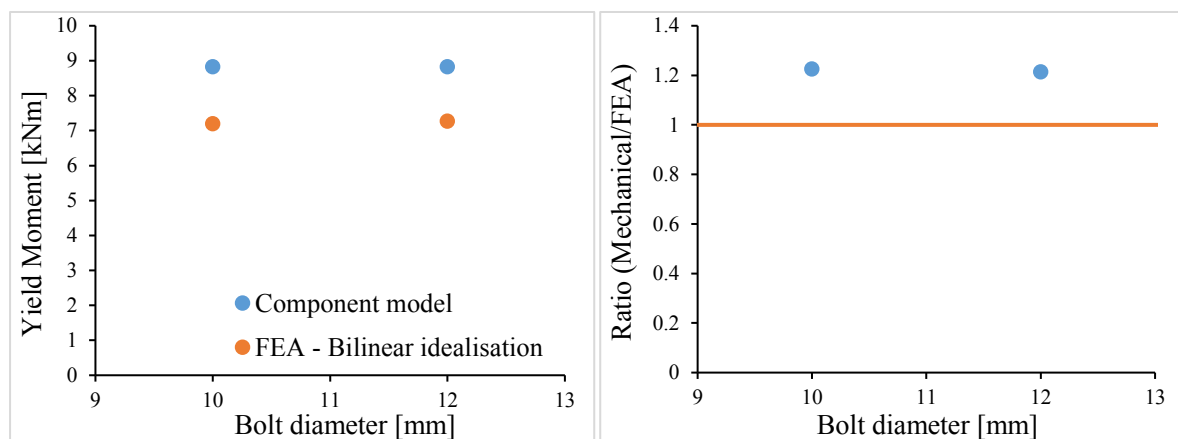


Figure 8.13 – FE and component model estimation of the yield moment by varying the diameter of the bolts for Type 1 connection.

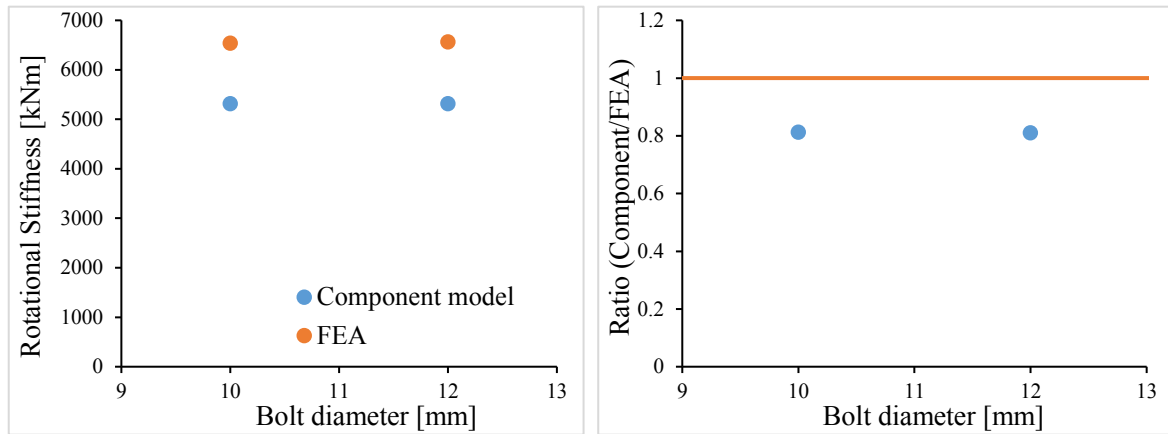


Figure 8.14 – FE and component model estimation of the initial rotational stiffness by varying the diameter of the bolts for Type 1 connection.

8.3.1.5 Number of long bolts per row

The number of long bolts connecting the two angles to the glulam were increased from 2 in a row to 3 in a row in order to examine any possible difference in the moment-rotation response of the connection brought about by group effects on the timber beam. Figure 8.15, presents the response of the connection obtained from the FE models. By comparing the two moment-rotation curves presented in Figure 8.18, it can be observed that number of bolts per row along the direction of the glulam beam has a very limited effect on the yield capacity of the connection and none on its stiffness.

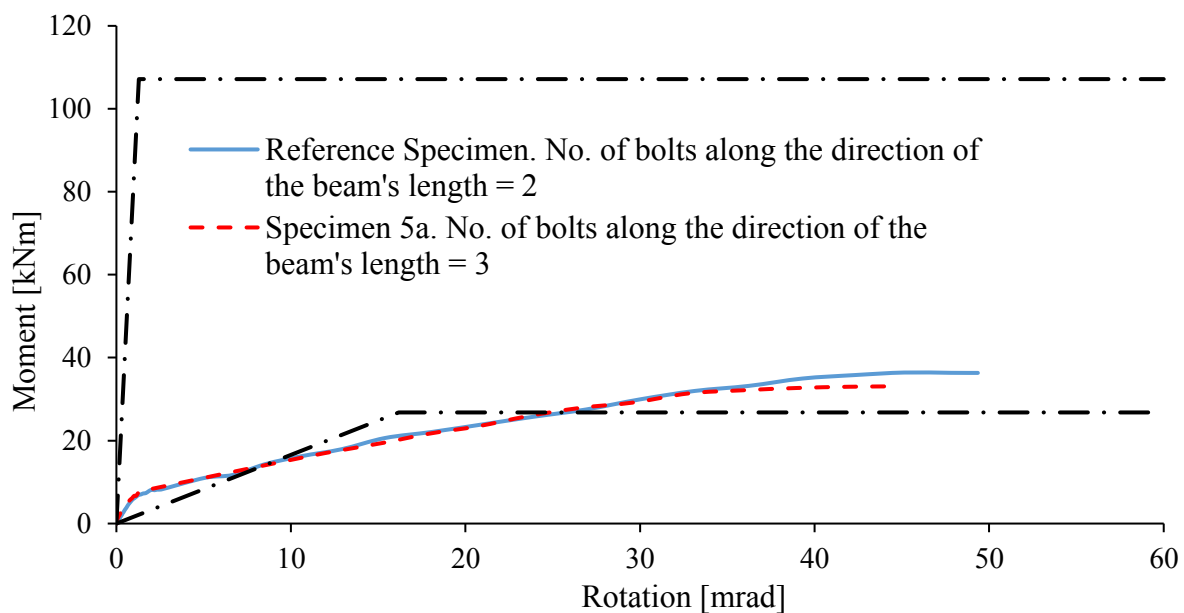


Figure 8.15– Influence of the number of bolts along the direction of the beam's length on the moment-rotation response for connection Type 1, including classification limits..

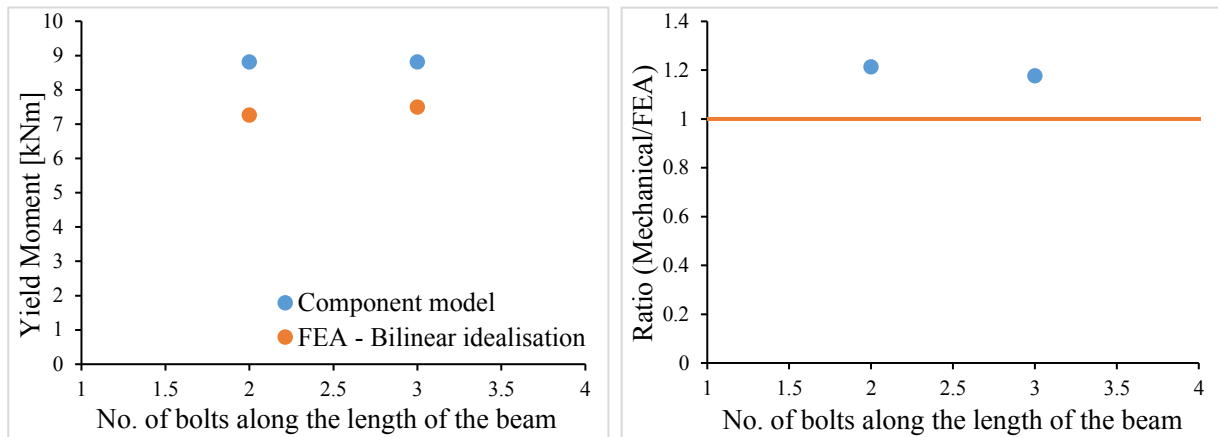


Figure 8.16– FE and component model estimation of the yield moment by varying the number of the bolts along the length of the beam for Type 1 connection.

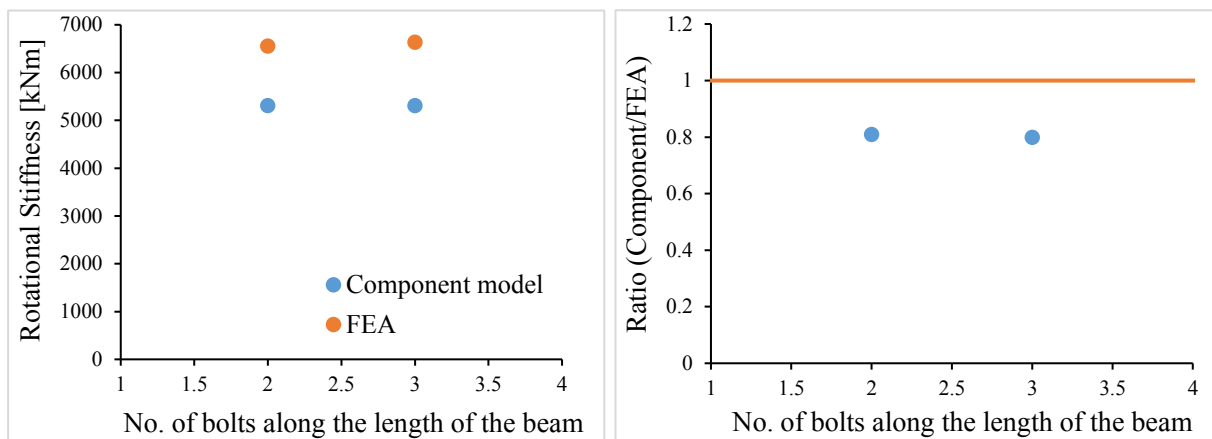


Figure 8.17 – FE and component model estimation of the initial stiffness by varying the number of the bolts along the length of the beam for Type 1 connection

Figures 8.16 and 8.17 show the influence of the number of bolts on the yield moment and rotational stiffness obtained from the FE and component models, respectively. From the results presented in these two figures, it can be observed that the estimation of yield capacity and rotational stiffness made by means of the component-based approach and the FE model are in good agreement and generally within a 20% difference. Both models, agreed that the number of bolts in the direction along the length of the beam has negligible effects on the yield moment and rotational stiffness estimations due to the nonlinear distribution of stresses along the bolt-row that reduces the effectiveness of individual fasteners when more bolts are added. The only difference, between the two connections, is observed at approximately 35 mrad of rotation, where the failure mode expressed by Equation (7.6) (corresponding to steel-timber connection failure mode) as discussed in Chapter 7 occurs for Specimen 5a.

8.3.2 Top and seat angle connections (Type 2)

In contrast with Type 1 details, Type 2 connections (T-stub) normally present important levels of stiffness and strength associated with semi-rigid joints. Semi-rigid joints play an important role in the behaviour of framed structures and an adequate structural design of those frames require a careful consideration of the joint moment-rotation response. For this reason, the range of parameters and number of cases studied for Type 2 connections is relatively larger than that presented before for Type 1 joints. Starting from the reference case introduced in Section 8.2, the parameters analysed include: beam width, beam height, distance between bolts (c in Figure 6.6), T-stub thickness, bolt diameter and bolt grade. As before, attention was placed on the connection yield moment and rotational stiffness. A bi-linear idealization was employed to obtain the stiffness and yield capacity of the connections from the FE moment-rotation simulations. On the other hand, equations (7.29) and (7.51) were employed to determine the corresponding component-based yield moment and stiffness estimations, respectively. The following sections presents comparisons between different connection responses when all its characteristics are kept constant and a single parameter, under study, is varied at a time.

8.3.2.1 Beam width

The effect of the beam's width on the behaviour of the connection was examined by testing three different widths, i.e. 90 mm, 140 mm and 185 mm. Figure 8.18, presents the response of the connection obtained from the FE models. From these curves, it can be observed that the width of the glulam beam affects the yield capacity whereas there are negligible changes in the stiffness of the joint. Additionally, Figures 8.19 and 8.20, shows the influence of the beam's width on the yield moment and rotational stiffness obtained from a bilinear idealization of the FE prediction and the corresponding component model estimations. From the results presented in these two figures, it can be observed that both the numerical and component models are able to replicate the effects of an increased beam width with relative consistency and the differences between the numerical and analytical predictions are within 20%. Finally, it is worth mentioning that for a beam width of 185 mm the connection is classified as pinned connection, whereas for lower beam widths (90 and 140 mm) the connection is classified a semi-rigid.

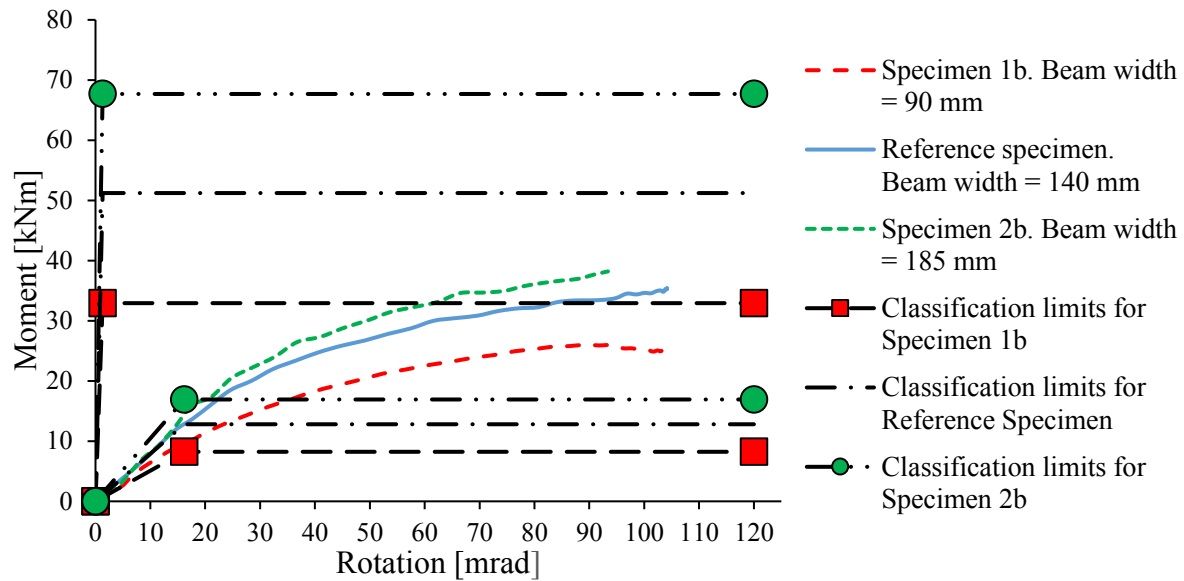


Figure 8.18 – Influence of beam's width on the moment-rotation response for connection Type 2, including classification limits..

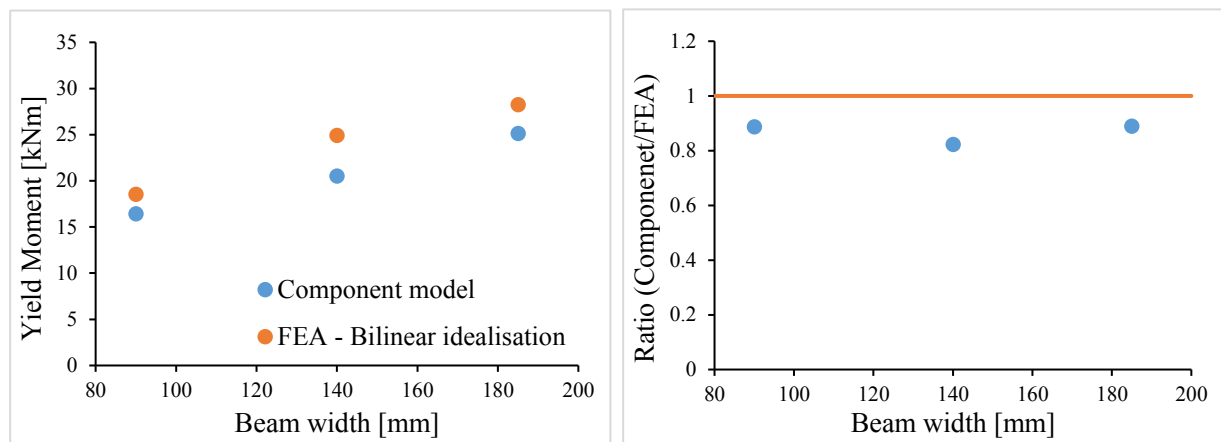


Figure 8.19 – FE and component model estimation of the yield moment by varying the beam's width for Type 2 connection.

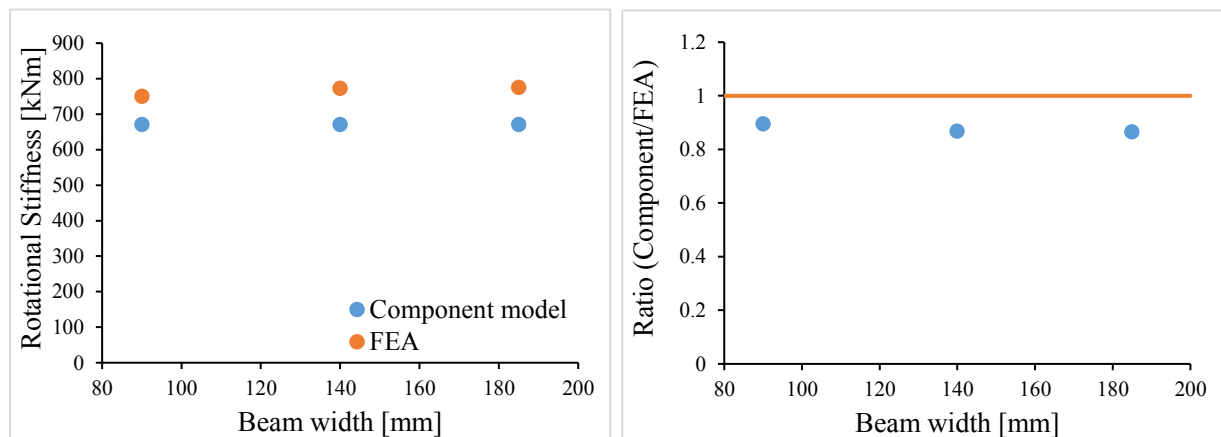


Figure 8.20 – FE and component model estimation of the initial rotational stiffness by varying the beam's width for Type 2 connection.

8.3.2.2 Beam depth

Three values for the depth of the glulam beam were chosen to test its effect on the moment-rotation response of the connection (i.e. 280, 315 and 360 mm). Figure 8.21, presents the response of the connection obtained from the FE models. By comparing these moment-rotation curves, it is evident that the height of the glulam beam has relatively minor effects on the yield capacity of the connection and even less on its rotational stiffness. Since the behaviour of the connection will be governed by the distance between its centroid of rotation and the outermost bolt-row, it follows that in order to operate significant changes in that distance, relatively large increments in the depth of the beam are required which may place the connection detail outside practical ranges of implementation. Finally, from the results presented in Figures 8.22 and 8.23, it can be observed that both the mechanical and component models present consistent estimations with differences of less than 20% in all cases. Finally, from Figure 8.21 it can be observed that for beam heights of 315 and 360 mm the connection is classified as pinned connection, whereas for lower beam height (280 mm) the connection is classified a semi-rigid.

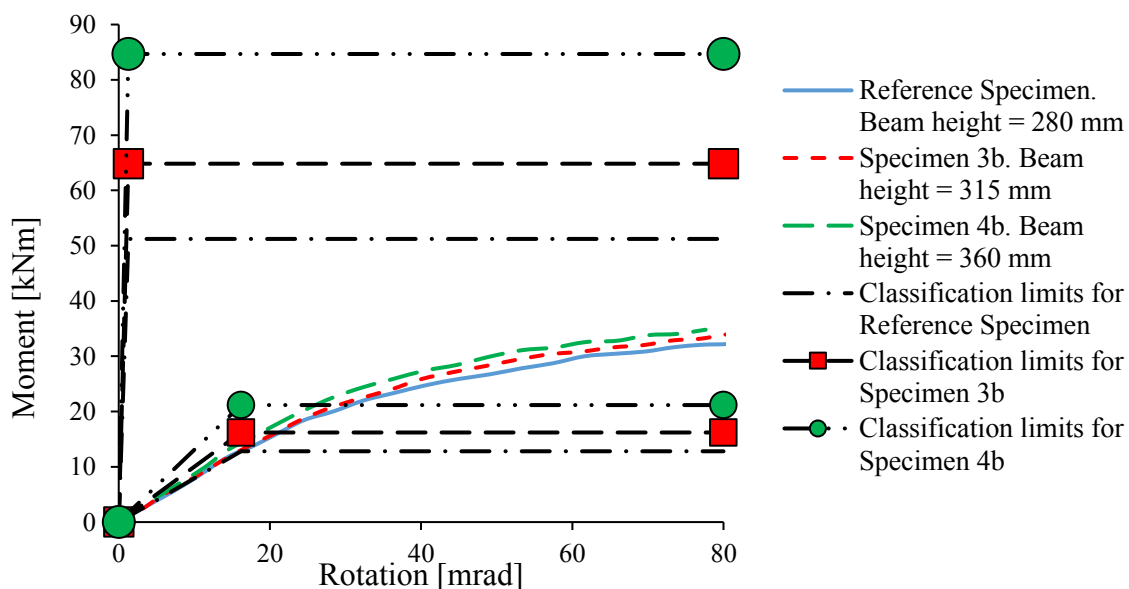


Figure 8.21 – Influence of beam's height on the moment-rotation response for connection Type 2, including classification limits..

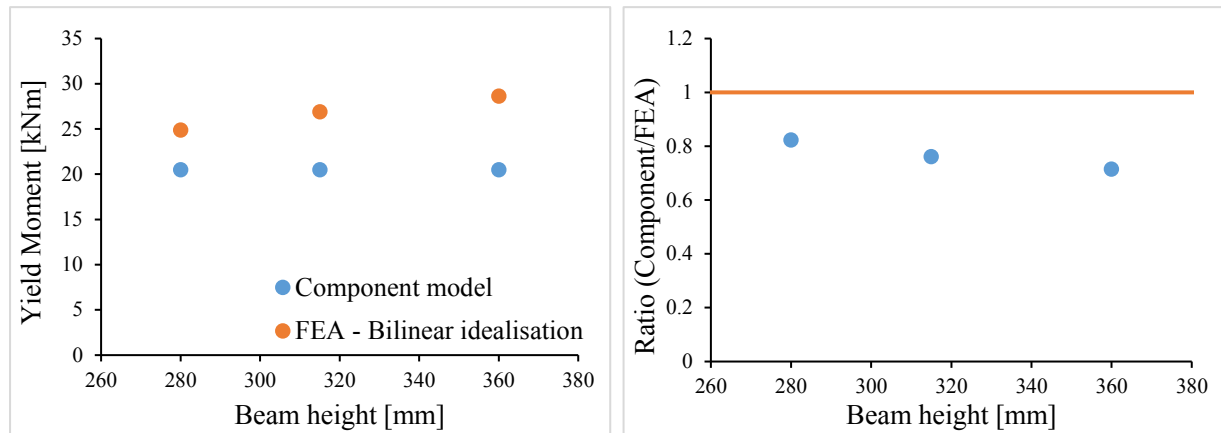


Figure 8.22 – FE and component model estimation of the yield moment by varying the beam's height for Type 2 connection.

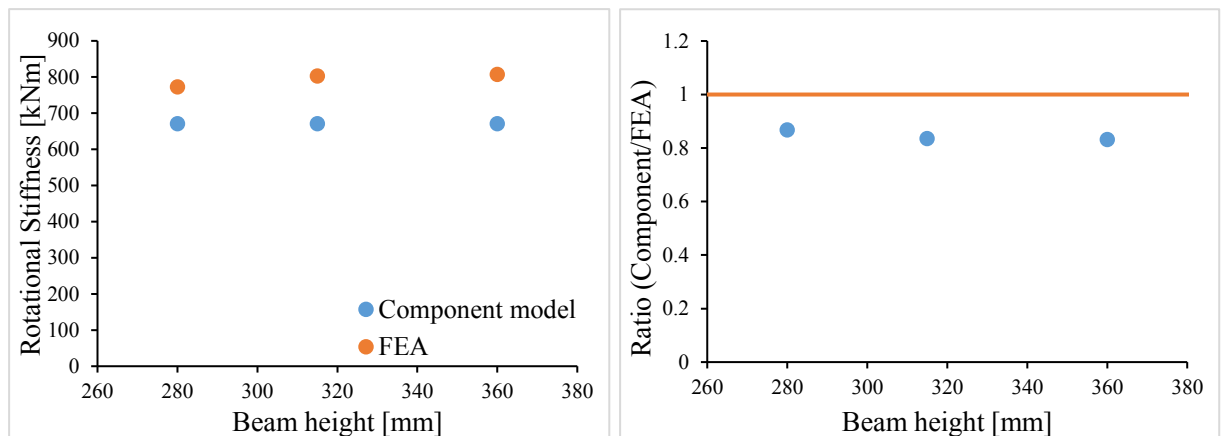


Figure 8.23 – FE and component model estimation of the initial rotational stiffness by varying the beam's height for Type 2 connection.

8.3.2.3 Distance between bolts (c)

The effects of bolt spacing were examined by varying the distance c (see Figure 6.6) between bolts in a given horizontal row. The reference case defined in Section 8.2 considers a $c = 50$ mm. Two additional cases were studied herein by assuming c values equal to 55 and 60 mm. Figure 8.24, presents the response of the connection obtained from the FE models. It can be observed from this Figure that, for the range of bolt distances employed herein, very little change in the overall moment-rotation response of the connection is expected. Figures 8.25 and 8.26 show the influence of the bolt spacing on the yield moment and stiffness obtained from

the FE and component models, respectively. From the results presented in Figures 8.25 and 8.26, it can be observed that there is a good agreement between FE and component models of the yield moment and stiffness estimations. The differences between the two methods are less than 20% in all cases. Importantly, even the small increments in stiffness and capacity brought about by the changes in bolt spacing seem to be well captured by the component-based formulation. Finally, the distance between bolts (c) has no effect on the classification of the connection. The connection is classified as semi-rigid regardless the value of c .

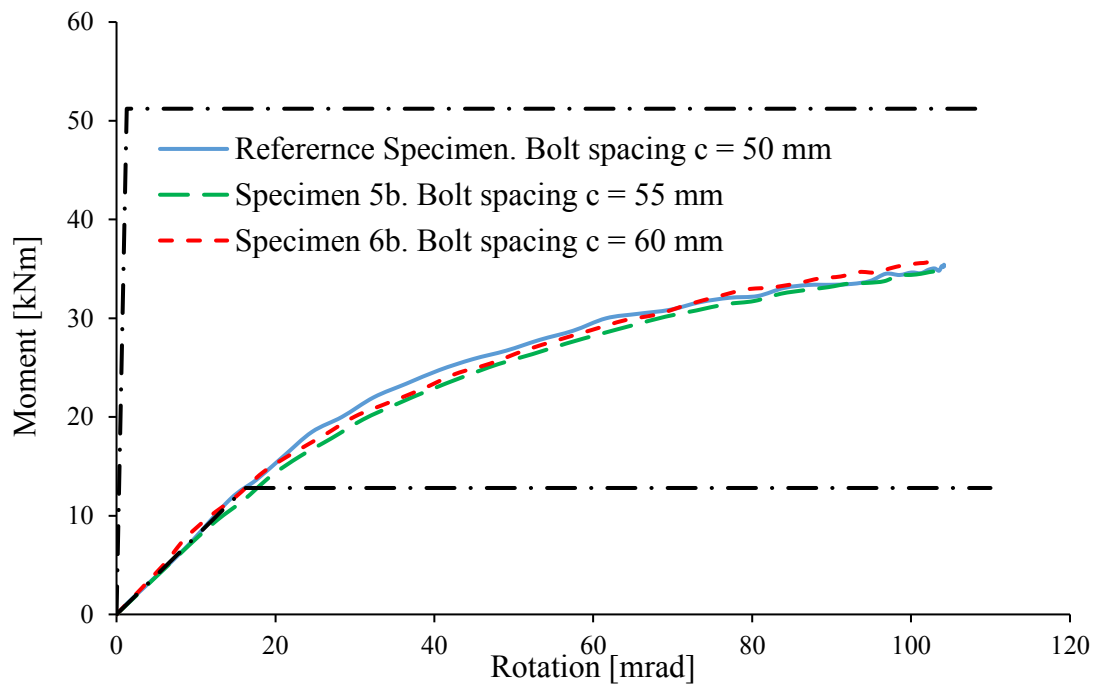


Figure 8.24 – Influence of bolt spacing c on the moment-rotation response for connection Type 2, including classification limits..

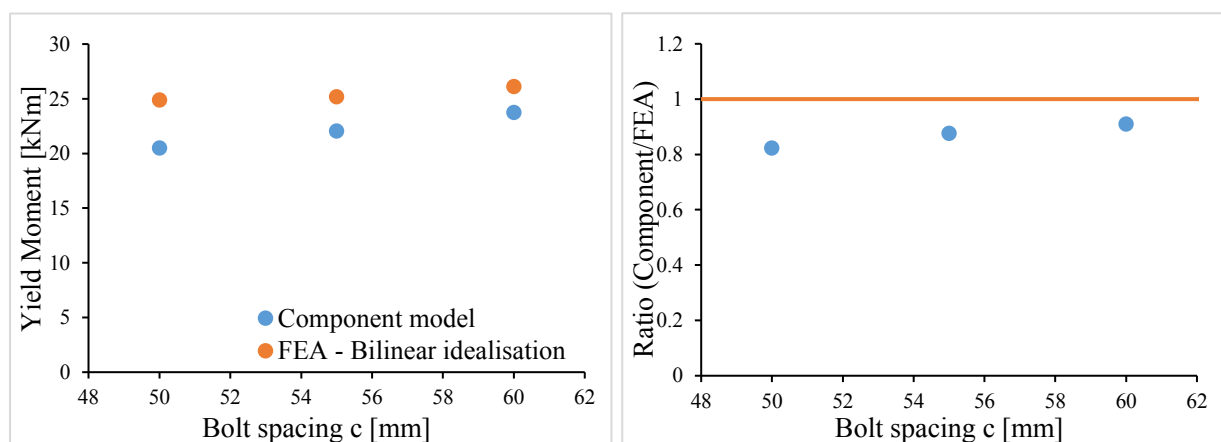


Figure 8.25 – FE and component model estimation of the yield moment by varying the bolt distance c for Type 2 connection.

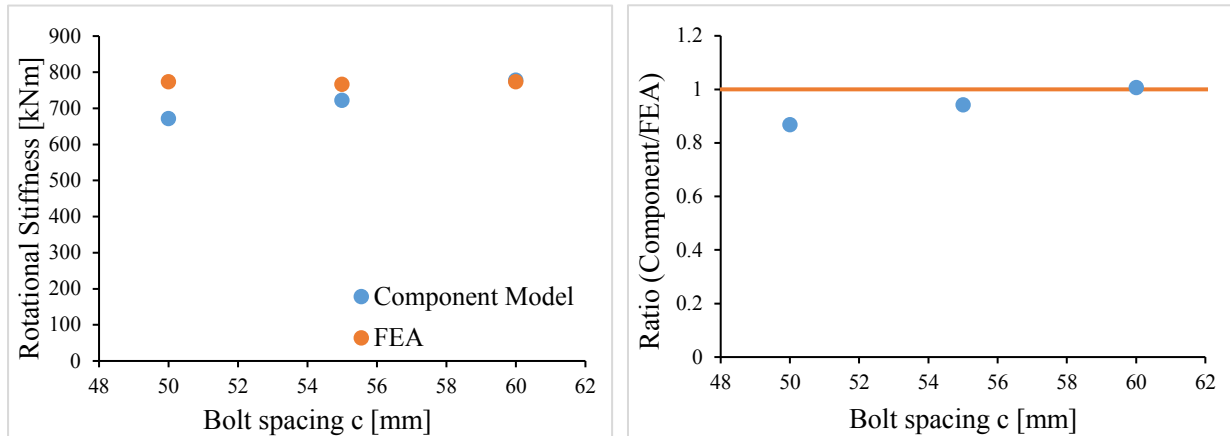


Figure 8.26 – FE and component model estimation of the initial rotational stiffness by varying the bolt distance c for Type 2 connection.

8.3.2.4 T-stub thickness

Starting with the reference case (Type 2), three practical values of T-stub thicknesses were analysed, namely: 10, 12 and 14 mm. Figure 8.27, presents the response of the different connections obtained from the FE models in terms of moment-rotation relationships. It is evident from this figure that the thickness of the T-stub does not affect the response of the connection. Figures 8.28 and 8.29 shows the influence of the T-stub thickness on the yield moment and stiffness obtained from the FE and component models, respectively. From both those figures it can be observed that both the numerical and component models are able to replicate lack of T-stub thickness effect on the connection response. Finally, the T-stub thickness has no effect on the classification of the connection. The connection is classified as semi-rigid regardless the T-stub thickness.

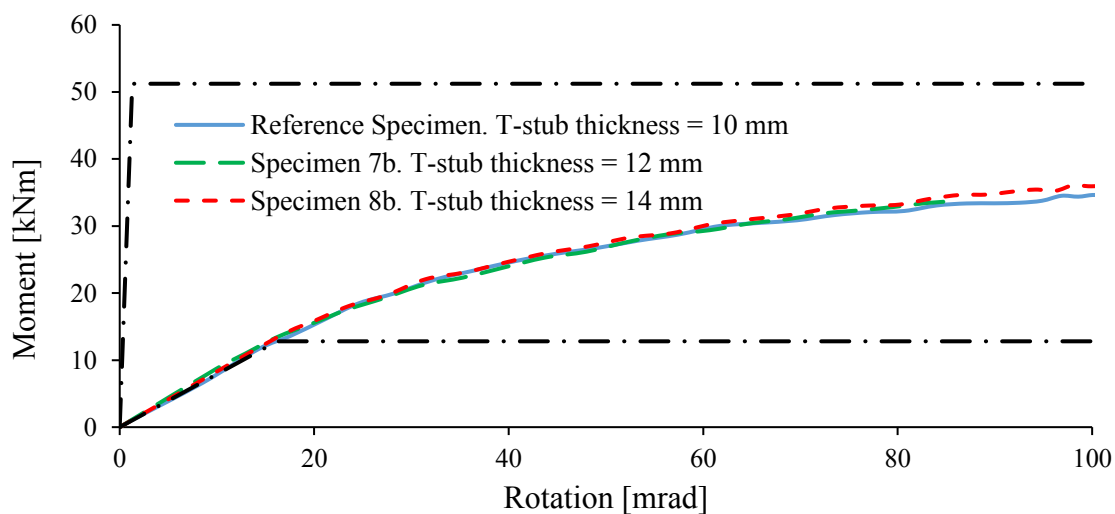


Figure 8.27 – Influence of the T-stub thickness on the moment-rotation response for connection Type 2, including classification limits..

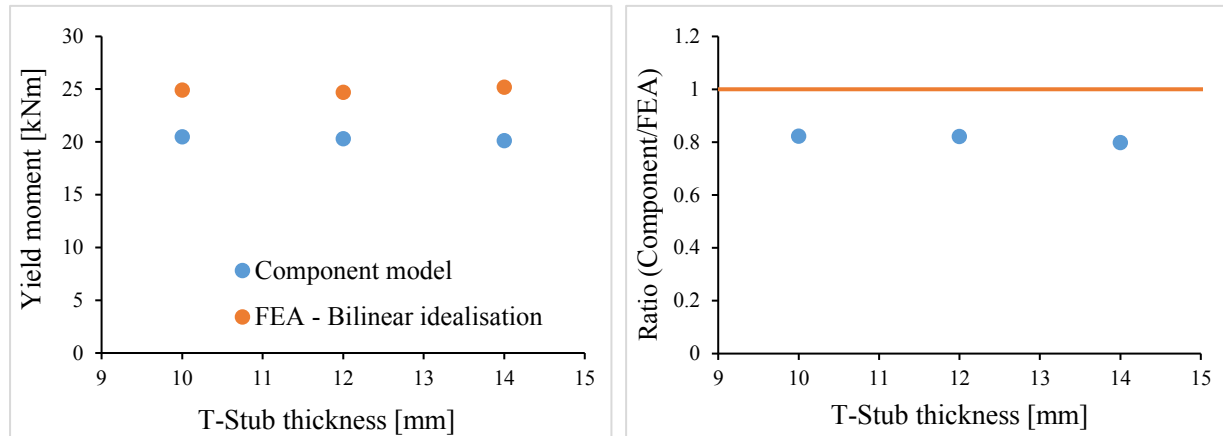


Figure 8.28 – FE and component model estimation of the yield moment by varying the T-stub thickness for Type 2 connection.

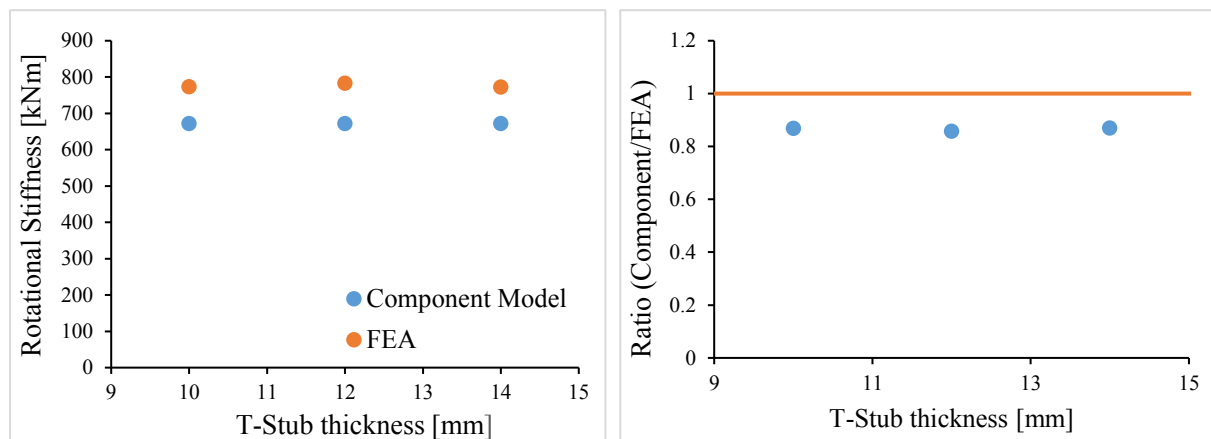


Figure 8.29 – FE and component model estimation of the initial rotational stiffness by varying the T-stub thickness for Type 2 connection.

8.3.2.5 Bolt diameter

Two bolt-diameter was varied from 8 mm to 10 mm in order to evaluate its influence on the T-stub joint response. The corresponding moment-rotation relationships are presented in Figure 8.30. From this figure it can be seen that, as expected, the diameter of the bolts greatly affects the response of the connection in both stiffness and capacity. These effects are well captured by the component-based formulations as depicted in Figures 8.31 and 8.32 which present the comparison between FE and mechanical estimations for yield moment and stiffness. It can be observed from the results presented in Figures 8.31 and 8.32 that the mechanical estimations are generally conservative and within a 20%. Finally, it is worth mentioning that the design proposed for the reference Type 2 connection in section 8.2 of this chapter was able to satisfy

the limitations in spacing and distances only for the 8 and 10 mm diameter bolts analysed. Any other bolt size would require a change in either the T-stub thickness, length and/or bolt distance c hence preventing direct comparisons to be made. Finally, from Figure 8.30 it can be observed that connection with bolt diameter of 8 mm classifies the connection as pinned.

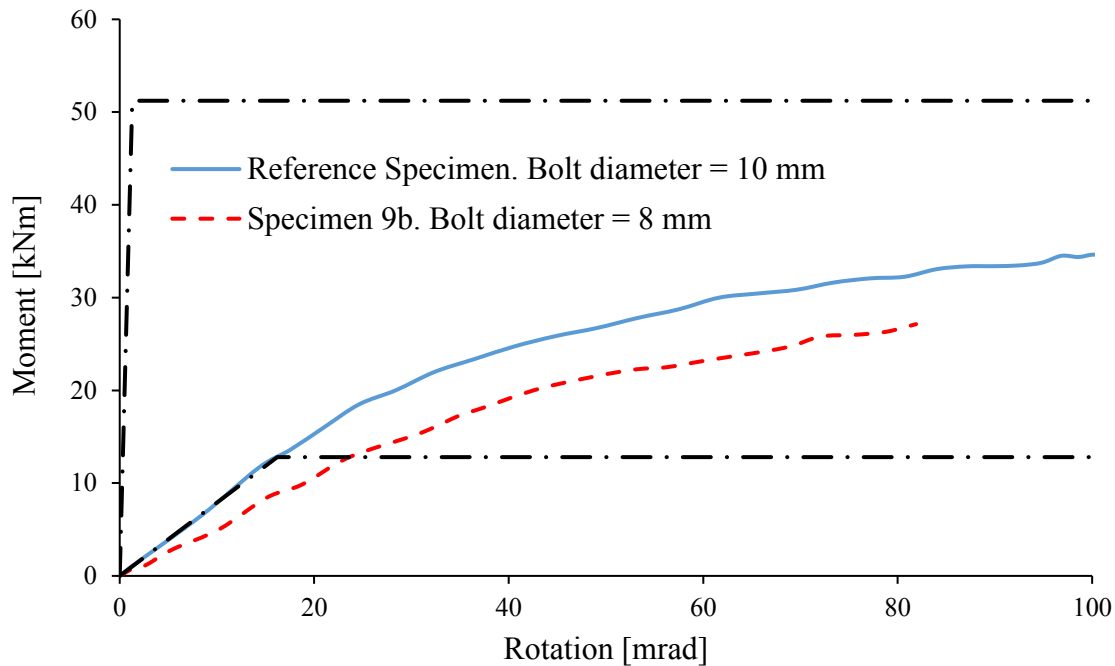


Figure 8.30 – Influence of the bolt-diameter on the moment-rotation response for connection Type 2, including classification limits..

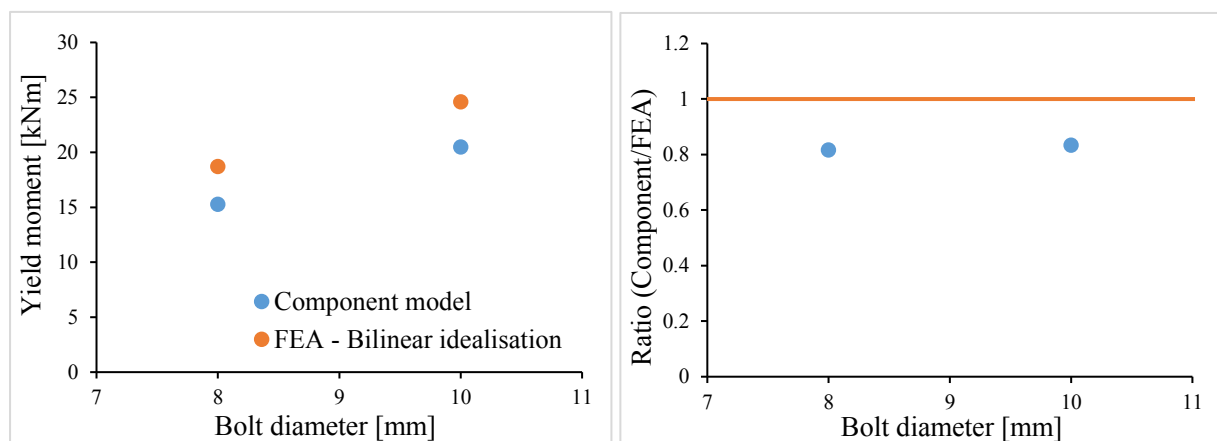


Figure 8.31 – FE and component model estimation of the yield moment by varying the bolt-diameter for Type 2 connection.

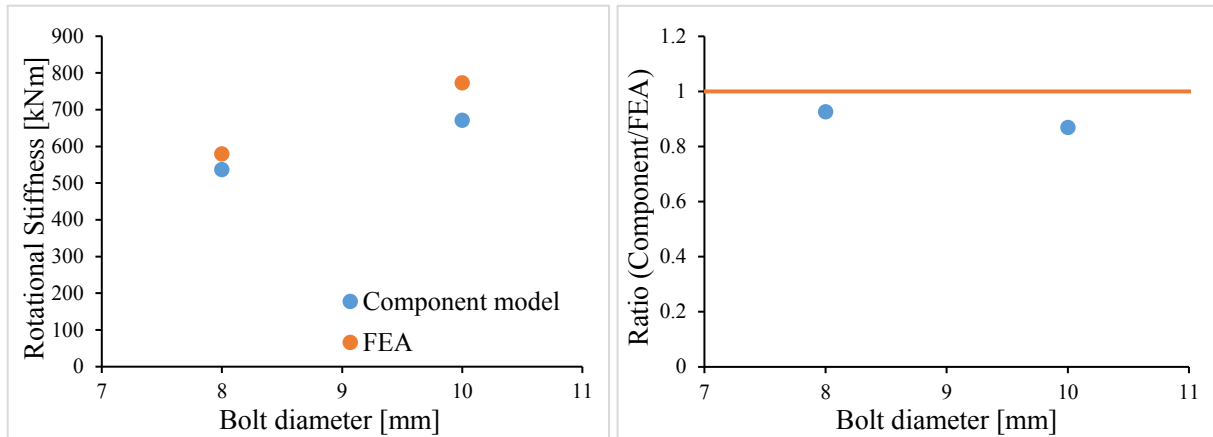


Figure 8.32 – FE and component model estimation of the initial rotational stiffness by varying the bolt-diameter for Type 2 connection.

8.3.2.6 Bolt strength grade

Three strength grades were employed for the bolts to study their effect on the connection behaviour, namely 6.8, 8.8 and 10.9. These grades correspond to yield and ultimate strength values of $F_y = 480 \text{ kN/m}^2$ and $F_u = 600 \text{ kN/m}^2$, $F_y = 640 \text{ kN/m}^2$ and $F_u = 800 \text{ kN/m}^2$ and $F_y = 900 \text{ kN/m}^2$ and $F_u = 1000 \text{ kN/m}^2$, respectively. Figure 8.33 shows the moment-rotation responses of the connection obtained from the FE models with different bolt strengths as described before. It can be seen from this figure that the bolt strength (grade) affects the joint response to some degree. It affects the connections yield moment slightly whereas it has a negligible effect on its stiffness. Figures 8.34 and 8.35 present the comparisons in terms of yield moment and stiffness estimations obtained from the FE and component-based approaches. In general, a good agreement is observed between numerical and mechanical estimations always within the 20%. Finally, the bolt strength grade has no effect on the classification of the connection. The connection is classified as semi-rigid regardless bolt strength grade.

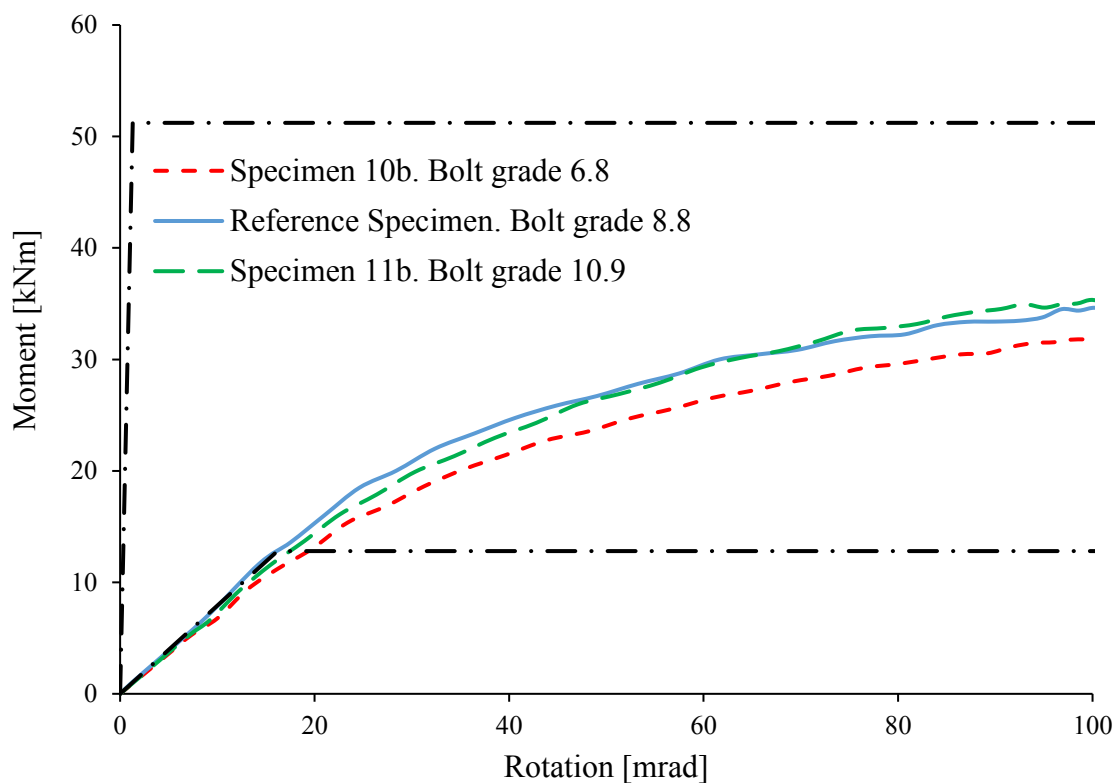


Figure 8.33 – Influence of the bolt strength grade on the moment-rotation response for connection Type 2, including classification limits..

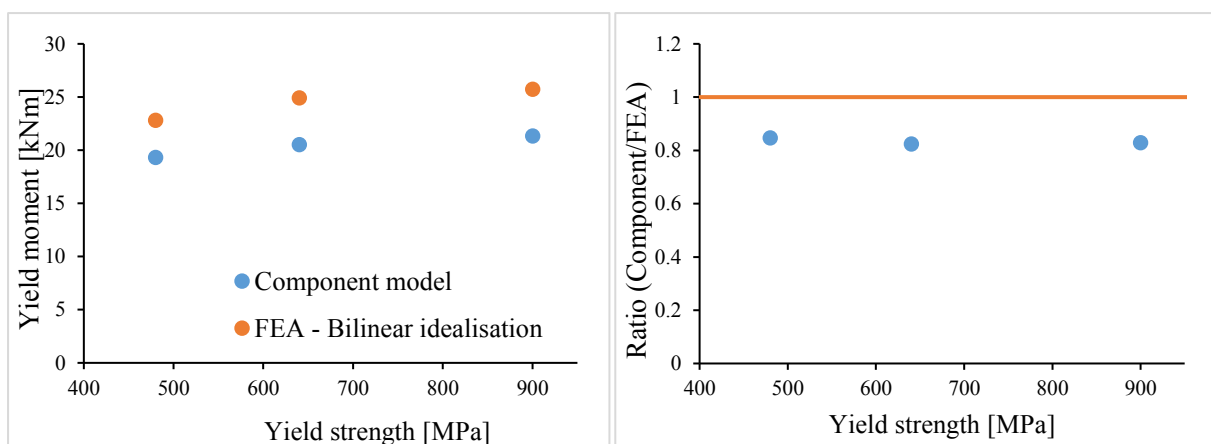


Figure 8.34 – FE and component model estimation of the yield moment by varying the bolt strength grade for Type 2 connection.

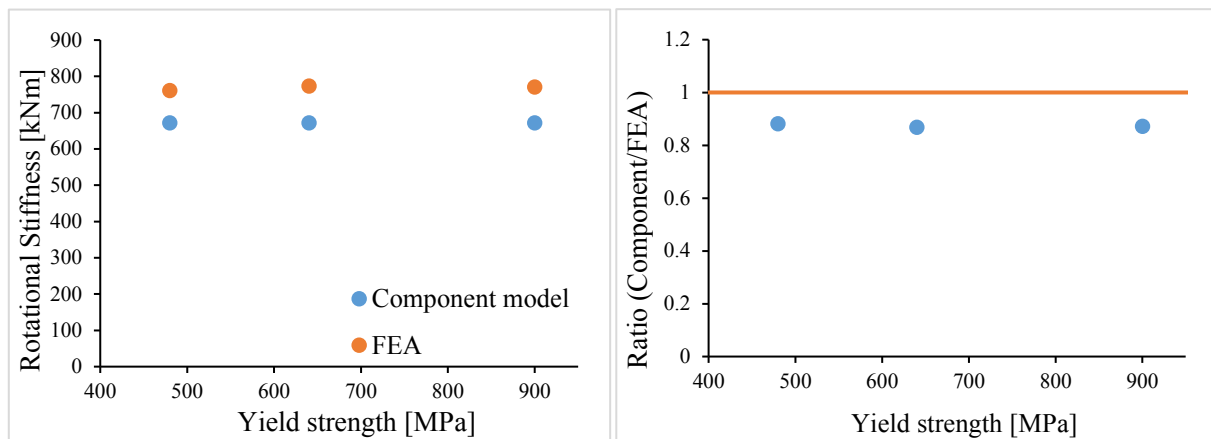


Figure 8.35 – FE and component model estimation of the initial rotational stiffness by varying the bolt strength grade for Type 2 connection.

8.4 Concluding remarks

The influence of various parameters on the moment-rotation behaviour of top and seat as well as slotted-in T-stub connections between timber beams and steel tubular columns have been examined via numerical and component-based models in this chapter. For Type 1 connections, a good correlation between FE and component-based estimations was appreciated in all cases. A 48.5% increase in rotational stiffness was identified for a column face slenderness reduction from 25 to 15 while this change produced negligible effects on the joints moment capacity. Similarly, in the case where the beam/angle width increased from 140 to 165mm negligible change in the rotational stiffness and a relatively minor increase in the yield moment of the connection was operated. Also, when depth was increased by 90 mm from 405 to 495 mm appreciable enhancements of stiffness and capacity were observed. Additionally, it was found that the change in the diameter and the number of the long bolts along the length of the beam do not affect the initial rotation stiffness and yield moment of Type 1 connections. In general, Type 1 connection configuration is classified as nominally pinned for the full range of parameter variations investigated

Similarly, a good agreement was appreciated from the numerical and component model estimations for Type 2 connections. Both models predicted that a yield moment increase for wider beams. Additionally, both models agreed that alterations in the beam width do not affect the rotational stiffness of the connection and the difference between the FE and the

corresponding component models predictions were within the 20%. According to the theory proposed by Kermani [38], the height of the beam should not affect the yield moment and rotational stiffness of the connection if the configuration of the bolted area remains constant. This was partially corroborated by the results described in this chapter where alternations in the beam's height do not affect the rotational stiffness. Nevertheless, a 10% increase in the yield moment was observed as the height of the beam increased from 280 to 360 mm. In addition, when the distance between bolts, c , is altered only minor effects on the connection stiffness and capacity are evidenced. Moreover, both methods agreed on the influence of the bolt diameter that had on the joint. Both the FE and component models predicted an increase in the yield moment of 32% as the bolt diameter increased from 8 to 10 mm. A significant increase in the joint rotational stiffness was also predicted by both approaches. Additionally, it was verified that for Type 2 connection, the bolt grade has important effects on the yield moment of the connection, whereas no change in the rotational stiffness was observed as the bolt grades increased. Both the FE and component models predicted that response and the differences between the two methods were always within 20%. It is worth mentioning that for a beam width of 185 mm the connection is classified as pinned connection, whereas for lower beam widths (90 and 140 mm) the connection is classified as a semi-rigid. Similar effect was observed for beam heights of 315 and 360 mm where connection is classified as pinned connection, but for lower beam height (280 mm) it is classified a semi-rigid. It can be concluded that the developed FE and mechanical models offer a practical approach for representing the connection response under various typical design configurations.

Finally, the results presented in the previous Chapter indicate that the component models are often an efficient and reliable tools for design. In addition, the figures presented in this chapter have verified the applicability of such models for the limiting practical cases investigated. They could therefore be used in practice for design and assessment of the connections studied in this thesis subjected to bending actions. The shear resistance of these connections and their response under combined loading are outside the scope of the present thesis and further studies are recommended on this topic.

Chapter 9

Conclusions and recommendations for further research

9.1 Summary and main findings

The main aim of this thesis was to provide practical solutions for connecting glulam timber beams and tubular steel columns and to provide essential information on the interaction between their constituent components as well as tools for their modelling and design. To this end, experimental assessment as well as detailed numerical and mechanical models were employed to investigate two new hybrid connection details proposed in this thesis. The main findings can be summarised as follows:

- **Material characterisation test:** A total of forty-three tests were performed to determine the mechanical properties of the glulam timber used. These tests were carried out in order to obtain the material properties for the *Picea abies* timber species employed in this research. Due to variations in the type and quality of timber available, the variety of end uses and the size of production output of the local timber industry, many different combinations of species and strength grade exist with distinct strength properties. Since, the strength classes presented in EN 338 do not disguise between wood species nor correspond to the material batch employed, it is thus beneficial to perform experimental tests on the timber type of interest. However, some properties which require special equipment and which are associated with smaller variations, such as the Poisson's ratio were acquired from the specific handbooks on timber. The experimental programme included shear, compression and tension (parallel and perpendicular to grain) tests, as well as four-point bending tests on large scale glulam beams. All tests were performed at Imperial College's Structures Laboratory and in accordance with European standards EN 408 [74]. Two implementations of the shear-field method described in [74] were used for the determination of the shear modulus from shear distortions measured on beams subjected to four-point bending. These included the use of a cross-shaped disposition of transducers as well as digital image analysis of a dotted

pattern painted on the surface of the beam. It was observed that the average shear modulus estimated from digital image tracking is far more consistent than the experimental configuration of transducers originally proposed in the European Standard EN 1194 [81]. Finally, the trends of the force-displacement curves obtained from each test-series were judged reasonable in comparison with available codified expressions.

- Experimental response of single-dowel connections. Fifty-two (52) single-dowel connection tests were experimentally tested to study the embedment interaction between the timber specimen and the steel dowel. The embedment tests were conducted in accordance to the European standard EN 383 [86] to examine the embedment behaviour for *Picea abies* timber specimens. The methodology of the experimental process as well as the dimensions of the specimens are given in this European standard. Two dowel diameters were employed and the load was applied both parallel and perpendicular to the direction of the grain. From the force-embedment plots, three behavioural stages were identified for which the general shape and distribution was not affected by the size of the dowels. Nevertheless, the dowel diameter was found to affect the embedment stiffness of the dowel-timber connection. The failure mode when load was applied parallel to the direction of the dowel, was crushing of the timber underneath the dowel followed by splitting of the specimen along its whole height. It was observed that the embedment behaviour of timber parallel to the direction of the grain under the effects of tension, produced slightly lower levels of embedment strength and stiffness compared to the respective compression test. The reason for these small differences is attributed to size effects brought about by the different geometries of the two test series. On the other hand, when the load was applied perpendicular to the grain direction the main failure mechanism was crushing of the timber fibres beneath the dowel followed by densification of the timber. In addition, Digital Image Correlation (DIC) techniques were used to monitor the evolution of local strain concentrations around the fastener. By examining the strain contours, it was observed that perpendicular-to-grain specimens exhibited a far more localized concentration of the area of highest strains in comparison with the parallel-to-grain joints. The experimental data from single-dowel connections, constitute the fundamental building-block for designing timber joints. The cellulose microstructure of timber generates difficulties when dealing with the behaviour of timber joints due to the intricate and highly non-linear interactions arising at the interface between fasteners and wood. Additionally, the properties determined

from these tests were used to create the material model for the foundation zone presented in Chapter 5.

- Numerical modelling of dowel embedment. FE models were created to simulate single-dowel connections under embedment. After demonstrating that conventional FE approaches cannot be used to model dowel type wood connections faithfully due to their inability to account for material damage, the foundation zone approach was elaborated. This model was shown to be able to correctly simulate the crushing response of wood in the embedding region. The material properties of the foundation zone are determined such that a softer and weaker cylindrical volume is defined around the fastener compared to the rest of the timber model to account indirectly for damage effects. The implementation of this region into the model allows an accurate simulation of the damage accumulation process. To this end, new relationships were formulated as part of this thesis, that can be used to estimate the material characterisation as a function of the crushing volume. The proposed model allows for a relatively low dependence on the radius of the foundation volume adopted making it applicable to a wider range of varying geometrical configurations. The validity and accuracy of the proposed modified foundation models were examined against the experimental force-displacement curves, and good agreement was found between the experimental response and the numerical simulations. Finally, a comparison was conducted between the experimentally attained displacement contours obtained by means of the DIC against the numerical predictions. The proposed foundation model is able to replicate the general tendencies such as magnitude of the deformations within the specimens and distribution of displacement contours. Furthermore, the localized strains immediately below the dowel predicted by the FE model are relatively higher (~30%) than the actual stains. This is attributed to the definition of hard contact between timber and steel and the averaged nature of the material properties employed for the foundation model. For a localised phenomenon such as the embedment behaviour of timber specimens, 30% difference in the vertical displacement contours is deemed to be acceptable.
- Top and seat angle hybrid connection. Two solutions were proposed in this thesis for connecting glulam timber beams and tubular steel columns: (a) top and seat blind-bolted angle connections with long bolts, and (b) slotted-in T-stub connections. Two monotonic tests were performed on top and seat angle blind-bolted connections with

different angle thickness. From the tests results the key response characteristics as well as the behavioural patterns were identified. The moment-rotation curves of both top and seat angle connections exhibited two distinct behavioural stages. A linear-elastic phase up to the yielding point and a linear-hardening stage up to the end of test or point of failure. Additionally, both connection had equivalent initial stiffnesses whereas the connection with the thicker angles reached higher levels of forces leading to a localised bolt-row shear failure in the wood. From the strain gauges installed in key locations on the angles, it was verified that the thicker angle experienced lower plastic strains around the fillet area which lead to greater forces on the bolts. Finally, according to the Eurocode connection classification limits [34], the connection tested can be classified as nominally-pinned.

- Slotted-in T-stub connection with bolts. Four connections of this type were experimentally tested under monotonic bending loads. The moment-rotation response of the original T-stub slotted-in connection can be divided into three main stages. Initially, a linear moment-rotation response is attained up to the point where bolt-slippage occurs and the connection response remains almost constant as the rotation increases. Beyond that phase, the moment-rotation response of the connection increases up to the point of failure. Two reinforcement alternatives were examined, namely: (i) reinforced screws perpendicular to the fibre, and (ii) bottom wedge angle. It was found that the presence of the screws significantly increases the ductility of the connection by preventing wood splitting, whereas the presence of a bottom wedge angle ensures a continuous contact of the compression zone and reduces the bolt-slippage hence enhancing the ultimate capacity of the connection. Non-contact strain field measurements and Digital Image Correlation (DIC) techniques were used to obtain the point of rotation at different rotation levels. Finally, according to the Eurocode connection classification limits [34], this connection can be classified as semi-rigid.
- Numerical simulations and simplified mechanical models. Detailed finite element models were created to simulate the experimental timber beam-to-steel column connection tests. The models included advance features such as contact phenomena, bolt pretension and orthotropic material definitions. The foundation zone approach developed in earlier chapters of this thesis was implemented. The results of the models were validated against the experimental results and good agreement was found. In

particular, the stiffness, yield point, ultimate capacity as well as the deformation characteristic of the various components were all closely predicted. Additionally, component-based models were also formulated for the prediction of the flexural response of the connection types tested. The purpose of the component based models is to an initial simplified evaluation of the effects of various parameters on various desired properties. This is very beneficial when designing a structure or an assembly of components because it can effectively limit the number of numerical simulations and experiments. These expressions were developed to estimate the stiffness, capacity and ultimate loads of the connections and the results were validated against the experimental and numerical findings. The component models proposed were able to predict the response of the connection reasonably well including those cases where screws and bottom-wedge angles were involved.

- Practical application and design implications. Since, there is a dearth of information on timber-steel hybrid connection of this type, design methodologies must be developed to offer a preliminary assessment on the prediction and evaluation of the key characteristics. To this end, the influence of various parameters on the moment-rotation behaviour of top and seat as well as slotted-in T-stub connections between timber beams and steel tubular columns were examined via numerical and component-based models for a range of typical design values. Starting with a realistic building case study, the range of variations were selected to cover the practical limit cases for a given parameter. For example, beam width was varied from 90 to 185 for the Type 2 connection which are thought to be the smallest and largest section sizes for a typical 5-storey building. Similarly, beam depths between 280 and 360 for Type 2 connection were analysed to cover the reasonable extremes of shallow and deep beams encountered in practice. The same applies for other parameters. This approach aligns with the aim of this chapter which is to shed light on the practical use of the connections previously studied within realistic geometric configurations in bending. A good correlation between FE and component-based estimations was obtained in all cases. It was concluded that the developed FE and mechanical models offer a practical approach for representing the connection response under various typical design configurations. The range of parameters for which individual connections can be classified as pinned or semi-rigid have been also highlighted in Chapter 8.

9.3 Limitations and future work

The main findings and contributions presented in the previous chapters address the objectives of this thesis. However, the work undertaken can be extended in a number of ways. Some of the most interesting and significant further research directions are listed in the paragraphs that follow.

For this project an experimental program was conducted to determine the material properties for Scandinavian Spruce. Although the experimental programme was extensive, some key material properties were either taken from the literature or derived from other properties by utilising design formulae from the British standards. These material properties were the Poisson's ratio ($\nu_{12}, \nu_{13}, \nu_{23}$), the Young's modulus for the glulam beam perpendicular to the direction of the grain E_2 (equation 6.1) and the shear strength in the YZ plane σ_{23} . Resource limitations as well as availability of experimental set-ups did not allow the completion of these tests. Even though the values for these properties were not obtained experimentally the end results were very satisfactory. Nevertheless, an experimental program could be conducted to determine the experimental values for these properties. Additionally, the Digital Image Correlation (DIC) system employed in this thesis provided essential information on determining some key characteristics which helped in understating the behaviour of timber as a material and in the validation of the numerical models. Additionally, the DIC system can be used as a third method to determine the shear modulus in 4-point bending tests.

The embedment behaviour for this type of timber was examined under two dowel diameters of 12 mm and 16 mm. Additional tests should be performed to further investigate the effects of dowel diameter and specimen sizing on the embedment strength. The results obtained from the embedment tests under the two diameters were used to estimate the parameters $\beta_1, \beta_2, \gamma_1$ and γ_2 for cases with $d = 10$ mm. Although this assumption was validated against experimental results on full joints (presented in Chapter 7), it would very beneficial to compare these values with the corresponding values obtained experimentally.

One of the main objectives of this project was the implementation and improvement of the foundation zone approach into the FE models. Although the foundation zone method provided accurate results for static loads, its application for more complex loading conditions such as

dynamic actions may be inappropriate. Since timber behaves differently under compression and tension, it is nearly impossible to simulate that behaviour without developing a dedicated material subroutine. Furthermore, this method is also weak for simulating damage in the timber at large deformation. To this end, material subroutines must be created that incorporate techniques for failure analysis such as Weibull's weakest link theory which can predict brittle failure. Finally, another technique that can provide a useful solution to the issue of simulating the damage mechanics in wooden members is by modelling the timber components as a foam (multi-cell structure).

Considerable time had been allocated to the development and study of timber beam-to-steel columns connections under bending and thus two types were investigated. It was concluded that Type 1 connection is characterised as pinned-connection even with thicker angles and thus it is inappropriate for building applications. Hence, further investigations can be conducted to strengthen the connection without overcomplicating the design. Additionally, the point of rotation for the moment estimations in the component models of Type 1 connection was assumed to remain at the bottom part of the beam through the whole force application. This assumption has been indirectly verified in Chapter 7 when dealing with a larger set of connection configurations. Nevertheless, further studies are recommended on the exact location of the point of rotation in timber beam to steel column connections and their potential influence (or lack of it) on the overall response of the joint.

In cases where angles and screws were introduced for Type 2 connections, it was assumed that the force/stress distributions acting on the timber were rectangle (see Figure 7.11) and triangular (see Figure 7.14), respectively. This might not be applicable when significant shear action is carried onto the connection or when a combination of loads is to be expected. Moreover, the failure modes presented in Figure 7.9 are based on tensile loads generated by bending action. This assumption produced satisfactory component models as verified in Chapters 7 and 8. However further studies on the shear response of these connections are recommended as well as experimental and numerical evaluations of the effects of multiple loading combinations.

The study of timber beam-to-steel columns joints in this project were treated as single units and hence there is the need to investigate their response as an assembly within a framed structure. The conducted experiments were designed to apply monotonically increasing bending loads in all connection tests. Therefore, the investigation of the proposed connections

under different loading configurations such as axial and shear load should be undertaken. Furthermore, the connection response to dynamic loads such as blast, impact and progressive response as well their response under fire conditions will provide an improved understating and confidence on the use of this type of hybrid connections. Finally, a more detailed parametric analysis with significantly larger number of cases and parameter variations may be able to provide valuable information for a more comprehensive design guidance.

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Appendix A

Determination of the shear modulus using shear-field testing method

From triangle $\widehat{DEO}$, shown in Figure 3.15, it can be evaluated that,

$$L_{SF} = \frac{D_{SF}}{\sqrt{2}} \quad (A1)$$

Applying the sine rule in triangles $\widehat{AD'D}$ and $\widehat{BC'C}$ appearing in Figure 3.16, by assuming small rotation and utilising equation (A1, 3.1) the following two expressions are formed

$$\gamma_{SF,A} = \frac{dD_{SF,A}}{D_{SF}} \quad (A2)$$

$$\gamma_{SF,B} = \frac{dD_{SF,B}}{D_{SF}} \quad (A3)$$

The shear strain γ_{SF} of the shear field is the average of the two shear strains $\gamma_{SF,A}$ and $\gamma_{SF,B}$ estimated above. That is,

$$\gamma_{SF} = \frac{1}{2}(\gamma_{SF,A} + \gamma_{SF,B}) \quad (A4)$$

Hence, by introducing equations (A2) and (A3) to the above equation, yields

$$\gamma_{SF} = \frac{1}{2} \frac{dD_{SF,A} + dD_{SF,B}}{D_c} \quad (A5)$$

By assuming that the shear stress is uniformly distributed in the area where the shear fields are placed, the mean shear stress τ_{mean} can be expressed as the shear force applied to the cross section of the beam over the area of the cross-section. That is,

$$\tau_{mean} = \frac{F}{2} \times \frac{1}{h \times b} \quad (A6)$$

Where, h is the height of the beam, b is the width of the beam and F is the applied load in kN. The shear modulus G_{12} of the glulam beams can be obtained from the ratio of shear stress τ to the shear strain γ_{SF} , that is

$$G_{12} = a_G \times \frac{\tau_{mean}}{\gamma_{SF}}, \quad (A7)$$

where, a_G is the shear correlation factor. This correlation factor is required due to the underestimation of shear distortion captured by the shear fields over the depth of the glulam beam, see Figure 3.15(c). Brander et al. [76] calculates the correlation factor from the following relationship.

$$a_G = \frac{\tau_{SF}}{\tau_{mean}} + \frac{2}{3} \left(\frac{\tau_{max}}{\tau_{mean}} - \frac{\tau_{SF}}{\tau_{mean}} \right) \quad (A8)$$

Where, τ_{max} is the maximum shear stress obtained at the neutral surface of the cross-section (see Figure 3.15(b)) and τ_{SF} is the shear stress obtained at the area covered by the shear field, and it can be obtained from the following equation

$$\tau_{SF} = \frac{1}{16} \frac{F}{I} \times (h^2 - L_{SF}^2) \quad (A9)$$

Where, I is the moment of inertia. Additionally, the maximum shear stress τ_{max} can be formulated as shown below

$$\tau_{max} = \frac{3F}{4bh} \quad (A10)$$

Therefore, from equations (A6) and (A10) it can be deduced that

$$\frac{\tau_{max}}{\tau_{mean}} = \frac{1}{2} \quad (A11)$$

Additionally, from equations (A6) and (A9) it can be easily obtained that

$$\frac{\tau_{SF}}{\tau_{main}} = \frac{bh(h^2 - L_{SF}^2)}{8I} \quad (A12)$$

Introducing equations (A11) and (A12) into equation (A8), we get

$$a_G = 1 + \frac{1}{3} \frac{bh(h^2 - L_{SF}^2)}{8I} \quad (\text{A13})$$

Finally, by introducing equations (A5), (A6) and (A,13) into equation (A7), yields

$$G_{12} = 1 + \frac{1}{3} \frac{bh(h^2 - L_{SF}^2)}{8I} \times \frac{D_{SF}}{h \times b} \times \frac{F}{dD_{SF,A} + dD_{SF,B}} \quad (\text{A14})$$

Appendix B

Figures

Figures B1 to B4 correspond to Figures 3.20, 3.21, 4.9 and 4.10 respectively, and present the rest of the data. As stated in the main body of the Thesis, the full presentation of data draw attention away from the regions of the curves that are subject of immediate study for practical applications and since the behaviour at such large levels of deformation is less consistent with the conceptualization of the load characterization.

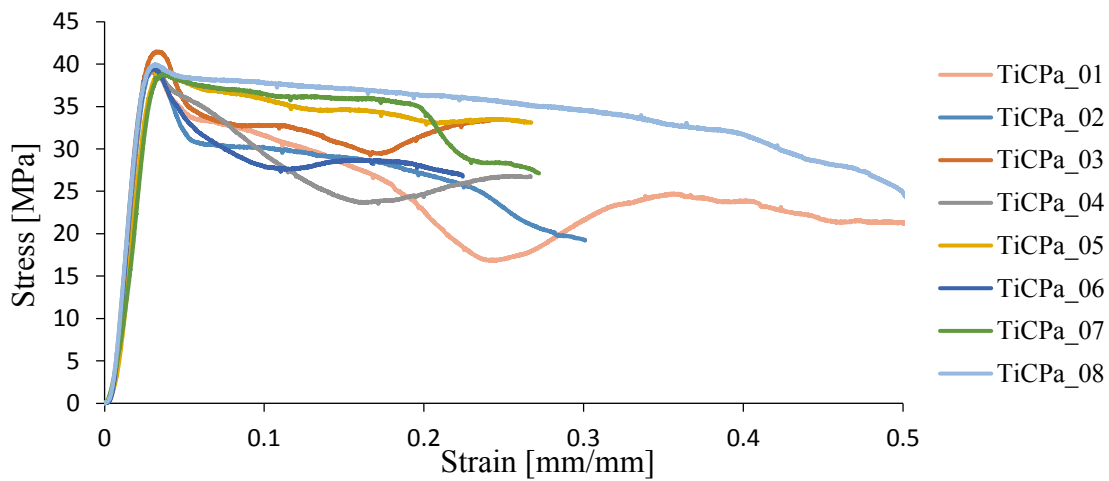


Figure B1 – Compression stress-strain relationships for Scandinavian Spruce
Glulam parallel to the grain direction

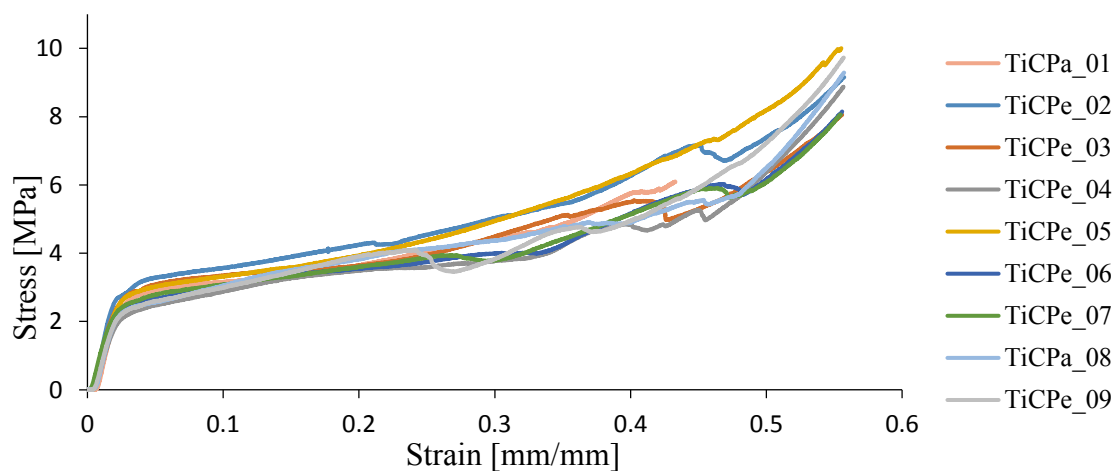


Figure B2 – Compression stress-strain relationships for Scandinavian Spruce
Glulam perpendicular to the grain direction

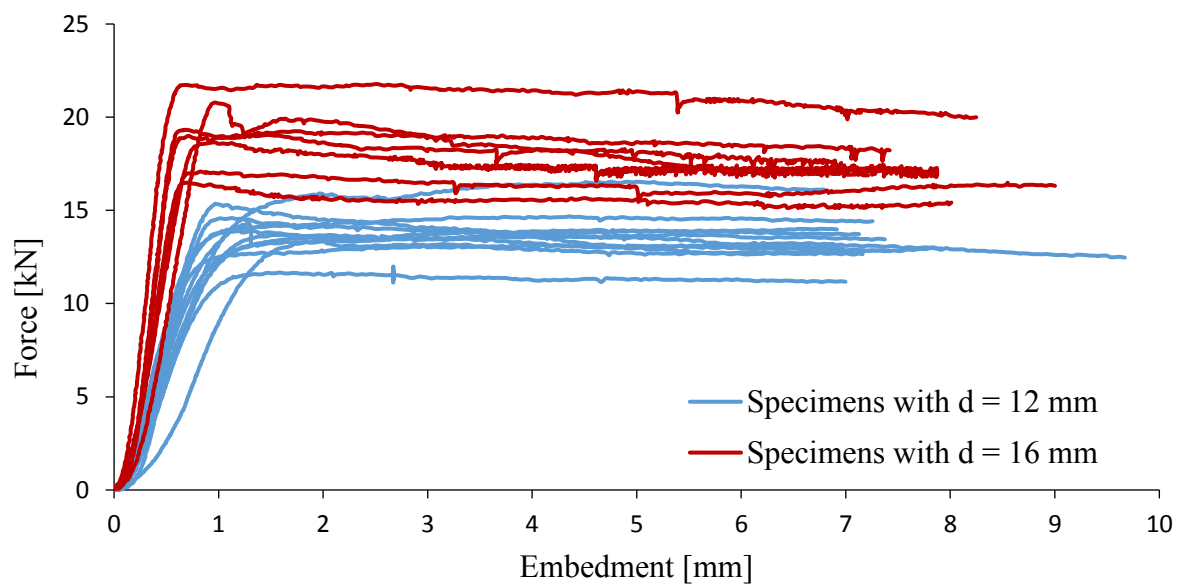


Figure B3 – Force-embedment relationship in the direction parallel to the grain under compression.

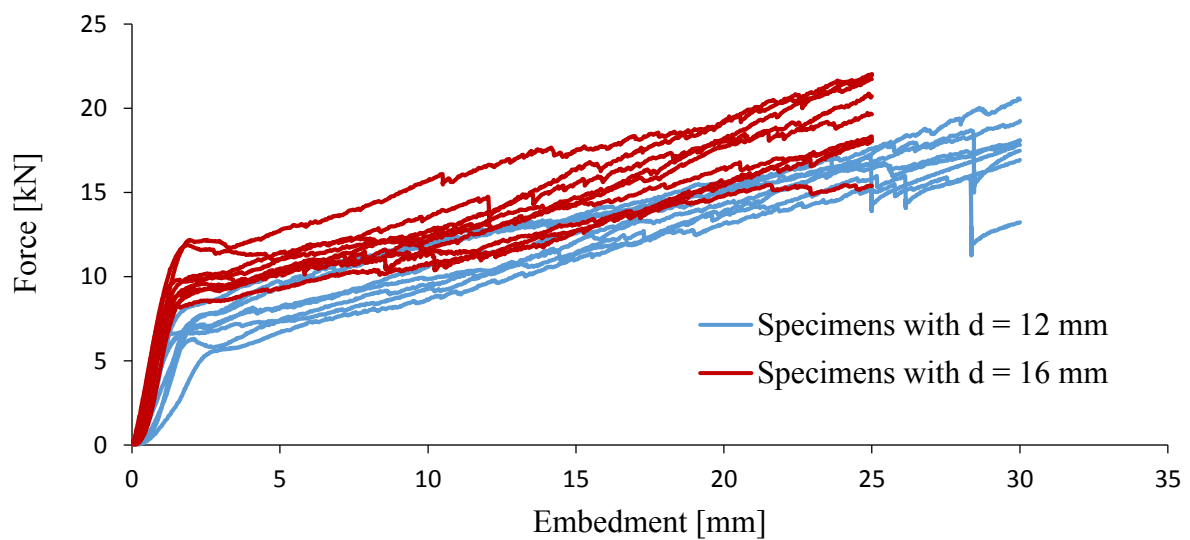


Figure B4 – Force-embedment relationship in the direction perpendicular to the grain under compression.

Figures B5 and B6 presented the results from the mesh sensitivity study conducted for the connection types 1 and 2 respectively, discussed in Chapter 6-7. The analysis showed that elements of size 1 mm to 5mm in the areas where large stresses/deformations are generated produce accurate results

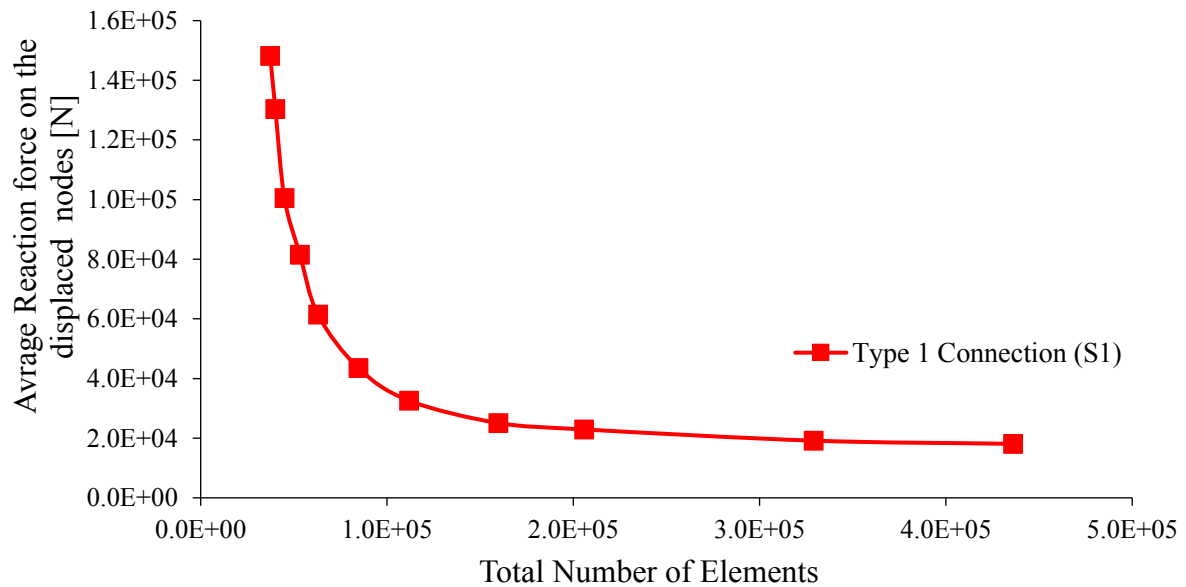


Figure B5 – Mesh optimisation curve for Type 1 connection

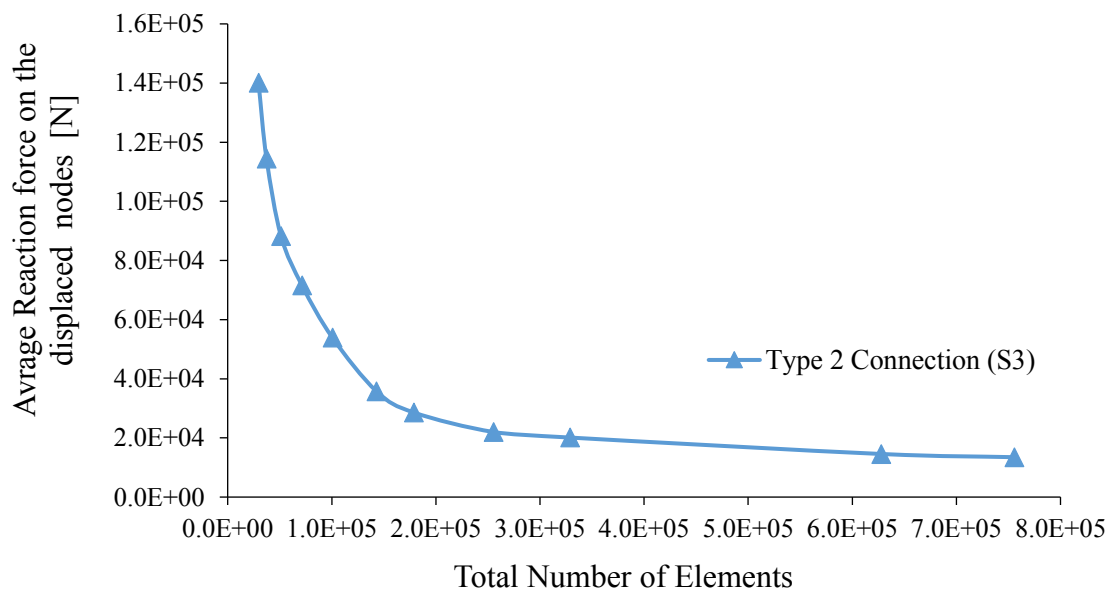


Figure B6 – Mesh optimisation curve for Type 2 connection

Appendix C

Determination of distances and spacings for the steel-timber connection.

Figure C1 below shows the combined nomenclatures for the spacings and distances of the glulam beam displayed in Figure 6.3 and the angle flange displayed in Figure 6.4a. To properly connect the glulam beam to the angle flange, the following expressions must held true; that is, $e_2 = b_1$, $p_2 = a_1$ and $d' \equiv p_{1,SB} = a_2$.

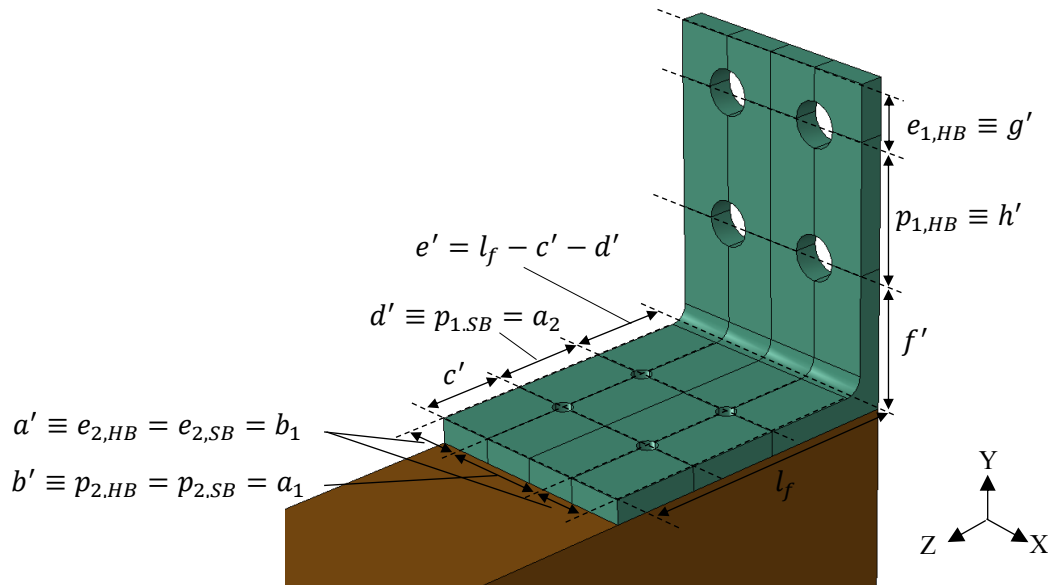


Figure C1 - Nomenclature of spacings and distances for a glulam beam-angle connection.

Along the width of the beam (x -axis).

From the geometry shown in Figure C1 we can have the following expression

$$w_f = 2a' + (m_b - 1)b' \quad \text{Eq(C1)}$$

where, m_b is the number of fasteners in the direction perpendicular to the grain (x -axis).

Rearranging the above equation, yields

$$b' = \begin{cases} \frac{w_f - 2a'}{m_b - 1}, & \text{for } m_b > 1, m_b \in \mathbb{N}^+ \\ 0, & \text{for } m_b = 1, m_b \in \mathbb{N}^+ \end{cases} \quad \text{Eq(C2)}$$

With the above expression the program determines a value for b' by selecting an appropriate value for a' for any given width and number of fasteners.

The minimum allowable values for a' should be the larger of $e_{2,HB}|_{min}$, $e_{2,SB}|_{min}$ and $b_1|_{min}$ and the minimum allowable values for b' should be the larger of $p_{2,HB}|_{min}$, $p_{2,SB}|_{min}$ and $a_1|_{min}$. Hence, expressing those into mathematical expressions, we get

$$a'|_{min} = \max [e_{2,HB}|_{min}; e_{2,SB}|_{min}; b_1|_{min}] \quad \text{Eq(C3)}$$

$$b'|_{min} = \max [p_{2,HB}|_{min}; p_{2,SB}|_{min}; a_1|_{min}]. \quad \text{Eq(C4)}$$

By substituting the relevant expressions from Tables (6.2) and (6.3) into the above relationships we get the following

$$a'|_{min} = \max [1.2d_{0,HB}; 1.2d_{0,SB}; 3d_{0,SB}] \quad \text{and} \quad \text{Eq(C5)}$$

$$b'|_{min} = \max [1.2d_{0,HB}; 1.2d_{0,SB}; 4d_{0,SB}]. \quad \text{Eq(C6)}$$

The above two equations can be reduced to:

$$a'|_{min} = \max [1.2d_{0,HB}; 3d_{0,SB}] \quad \text{Eq(C7)}$$

$$b'|_{min} = \max [1.2d_{0,HB}; 4d_{0,SB}]. \quad \text{Eq(C8)}$$

Moreover, the maximum allowable values for a' should be the minimum between the two expressions shown in equation (C9), below. The first limiting case is based on the geometric constrains for the connection configuration, while the second limiting case is given by EC3[34].

$$a'|_{max} = \min \left[\frac{w_f - (m_b - 1)b'|_{min}}{2}; 40mm + 4t \right]. \quad \text{Eq(C9)}$$

As in the above case, the maximum value for b' should be the minimum between the three expressions shown in equation (C10) below. The first expressions is based on the geometric constrains and the selected value of a' by the user, while the second limiting case is given by EC3.

$$b'|_{max} = \min \left[\frac{w_f - (m_b - 1)a'}{2}; 14t; 200mm \right]. \quad \text{Eq(C10)}$$

Figure C2 below shows the Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction of the angle's width are within the specified limits.

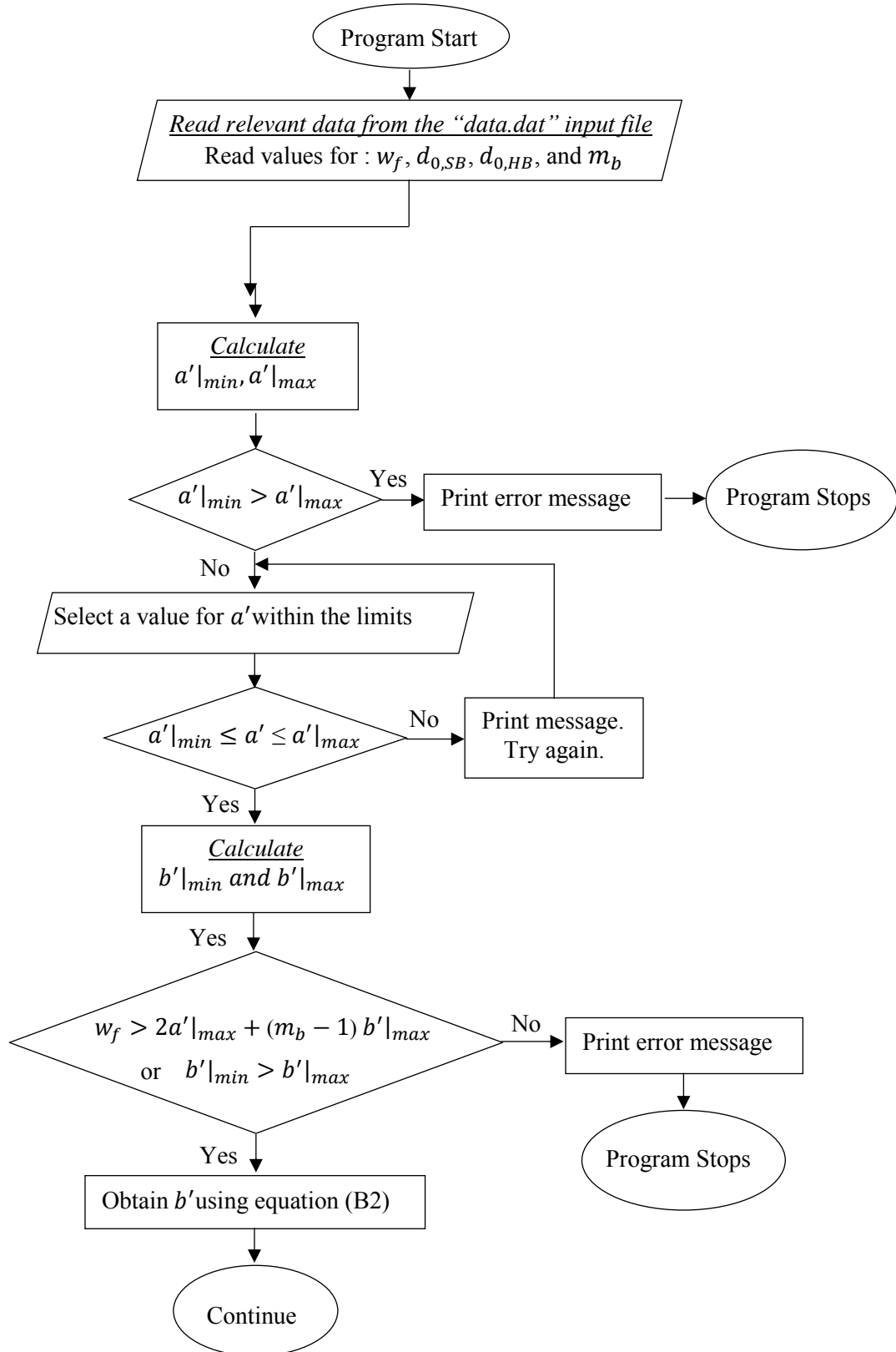


Figure C2 - Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction along the width of the angle are within the specified limits.

Along the length of the beam (z-axis).

From Figure C1 we have the following expression

$$l_f = c' + (n_b - 1)d' + e', \quad \text{Eq(C11)}$$

where, n_b is the number of fasteners connecting the angle to the glulam beam in the direction parallel to the grain (z –axis). Additionally, in Figure 6.4a, it is shown that four fasteners are used to connect the angle to the glulam beam, and hence, $n = 2$. If there were six fasteners connecting the angle to the glulam beam, the number of fasteners in the z –direction would have been equal to $n = 3$. Therefore, the following equation stands true

$$n_b = \frac{(N_{o_of_fastener})_b - m_b}{m_b} + 1, \text{ with } m_b \in \mathbb{N}^+, \quad \text{Eq(C12)}$$

where, $(N_{o_of_fastener})_b$ is the total number of fasteners connecting the glulam beam to the angle and m_b is the number of fasteners in the direction perpendicular to the grain (x –axis). Please note, that $N_{o_of_fastener}$ should be a positive even number.

The FOTRAN program is designed to calculate the value of e' while the user inputs the appropriate values for l_f , c' and d' , in that specific order. Hence, equation (C11) was rearranged as follows

$$e' = l_f - c' - (n_b - 1)d'. \quad \text{Eq(C13)}$$

The minimum allowable values for e' and c' are depicted in Tables (6.2) and (6.3) respectively. The minimum allowable distance spacing between the fasteners d' should be taken as the larger of $p_{1,SB}|_{min}$ and $a_2|_{min}$. Hence, expressing those into mathematical expressions, we get

$$e'|_{min} = \max[7d_{0,SB}; 80mm], \quad \text{Eq(C14)}$$

$$c'|_{min} = 1.2d_{0,SB} \quad \text{and} \quad \text{Eq(C15)}$$

$$d'|_{min} = \max[p_{1,SB}|_{min}; a_2|_{min}]. \quad \text{Eq(C16)}$$

By substituting the relevant expressions from Tables (6.2) and (6.3) into equation (C16) we get the following

$$d' = \max[2.2d_{0,SB}; 5d_{0,SB}], \quad \text{Eq(C17)}$$

which can be reduced to

$$d'|_{min} = 5d_{0,SB}. \quad \text{Eq(C18)}$$

Moreover, the maximum allowable values for c' should be the minimum between the two expressions shown in equation (C19), below. The first limiting case is based on the geometric constrains for the connection configuration, while the second limiting case is given by EC3.

$$c'|_{max} = \min[l_f - (n_b - 1)d'|_{min} - e'|_{min}; 40mm + 4t]. \quad \text{Eq(C19)}$$

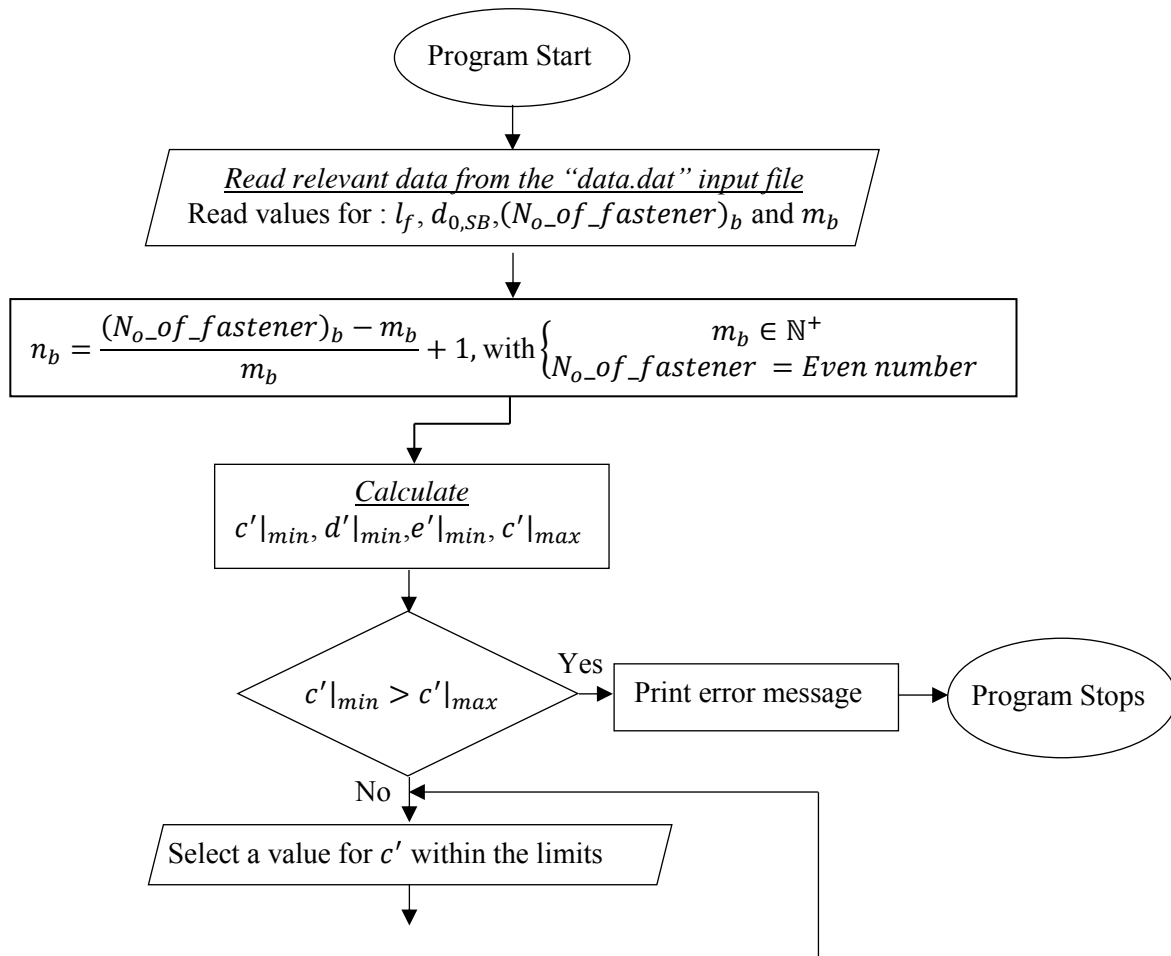
As in the above case, the maximum value for d' should be the minimum between the three expressions shown in equation (C20) below. The first expressions is based on the geometric constrains and the selected value of c' by the user, while the second limiting case is given by EC3.

$$d'|_{max} = \min \left[\frac{l_f - c' - e'|_{min}}{(n_b - 1)}; 14t; 200mm \right]. \quad \text{Eq(C20)}$$

Finally, the maximum value for e' can be determined from the following equation

$$e'|_{max} = \min [l_f - c'|_{min} - (n_b - 1)d'|_{min}]. \quad \text{Eq(C21)}$$

Figure C3 below shows the Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction of the angle's length are within the specified limits.



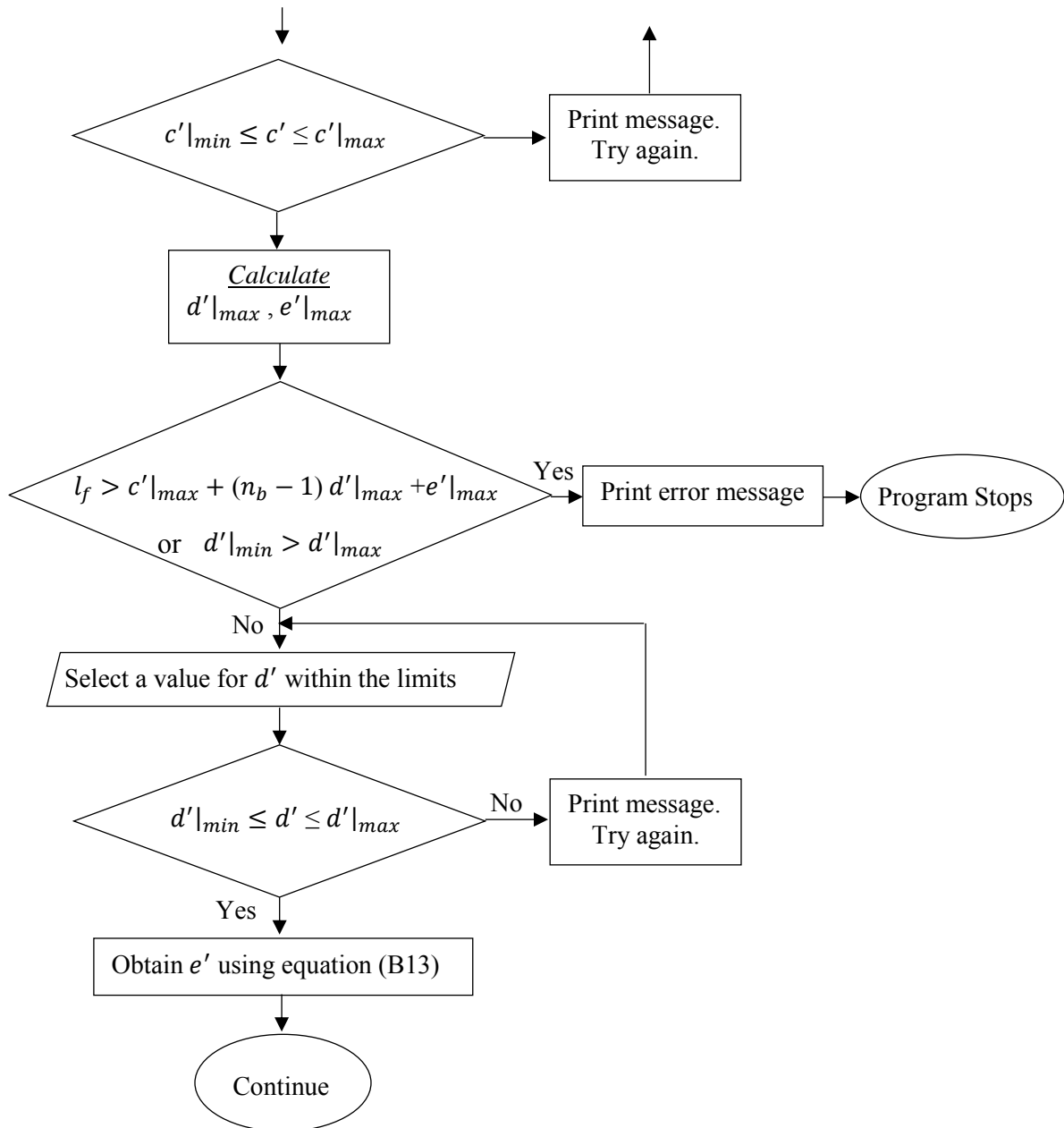


Figure C3 - Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction parallel to the grain are within the specified limits.

Along the height of the beam (x-axis).

This is the simplest case, as only one material type is involved. From Figure C1 above, we have the following geometrical constraints in the direction of the angle's height.

$$h_f = g' + (n_c - 1)h' + f', \quad \text{Eq(C22)}$$

where, n_c is the number of fasteners connecting the angle to the column in the direction parallel to the height of the column (y –axis). As in the previous case, the following equation stands true

$$n_c = \frac{(N_{o_of_fastener})_c - m_c}{m_c} + 1, \text{ with } m_c \in \mathbb{N}^+, \quad \text{Eq(C23)}$$

where, $(N_{o_of_fastener})_c$ is the total number of fasteners connecting the angle to the column and m_c is the number of fasteners in the direction parallel to the height of the column (y –axis).

The FOTRAN program is designed to calculate the value of f' while the user inputs the appropriate values for h_f , g' and h' , in that order. Hence, equation (C22) was rearranged as follows

$$f' = h_f - g' - (n_c - 1)h'. \quad \text{Eq(C24)}$$

The minimum allowable distance between the centre of the hole and the bottom edge of the angle (f') for a M16 Holo-bolt, was obtained by Liu, Malaga and Elghazouli [5] and it was found to be around 40mm. The minimum value for g' and h' are depicted in Table (6.3). Hence, expressing those into mathematical expressions, we get

$$f'|_{min} = 40mm, \quad \text{Eq(C25)}$$

$$g'|_{min} = 1.2d_{0,HB} \quad \text{and} \quad \text{Eq(C26)}$$

$$h'|_{min} = 2.2d_{0,HB}. \quad \text{Eq(C27)}$$

Moreover, the maximum allowable values for g' should be the minimum between the two expressions shown in equation (C28), below. The first limiting case is based on the geometric constraints for the connection configuration, while the second limiting case is given by Eurocode 3.

$$g'|_{max} = \min [h_f - (n_c - 1)h'|_{min} - f'|_{min}; 40mm + 4t]. \quad \text{Eq(C28)}$$

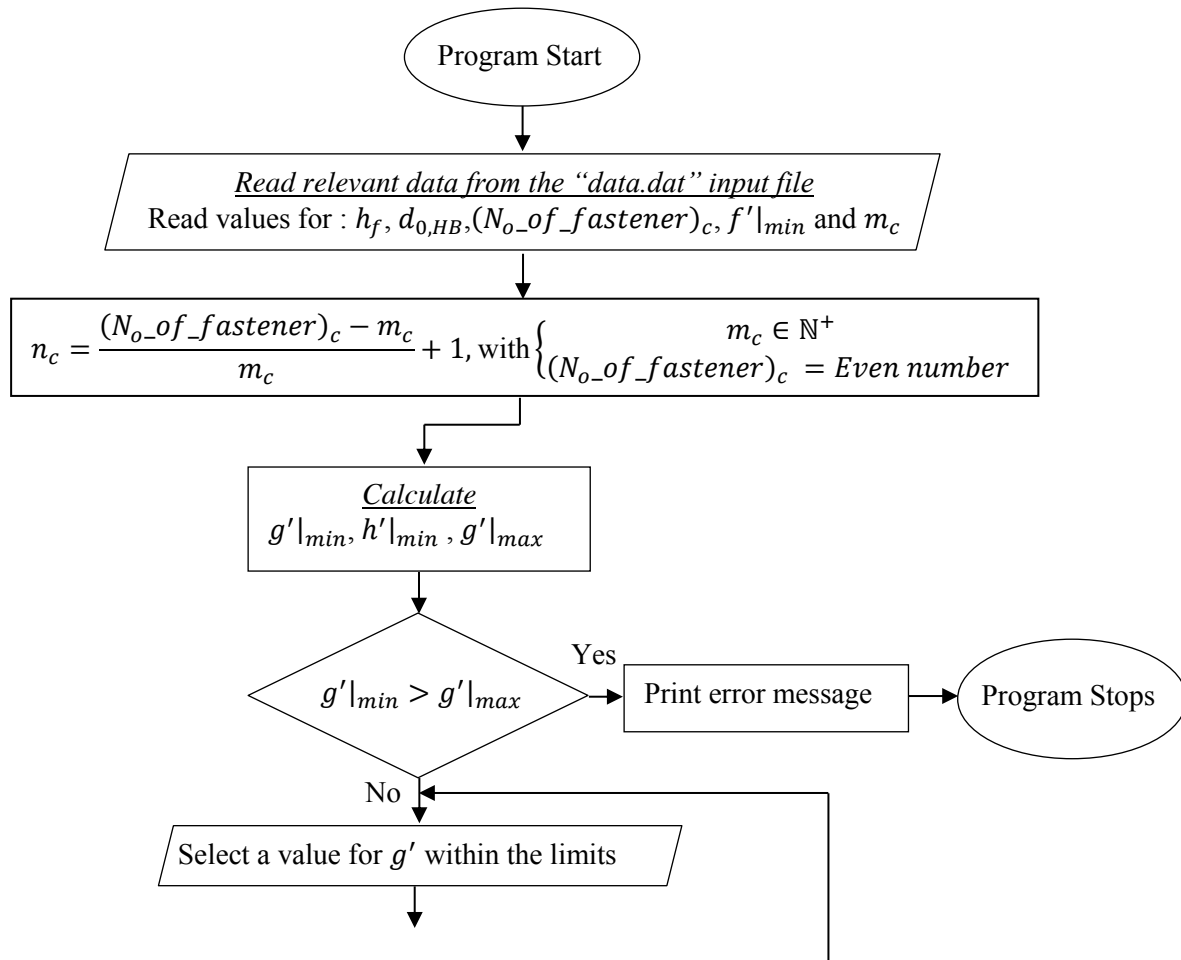
As in the above case, the maximum value for h' should be the minimum between the three expressions shown in equation (C29) below. The first expressions is based on the geometric constrains and the selected value of g' by the user, while the second limiting case is given by Eurocode 3.

$$h'|_{max} = \min \left[\frac{h_f - g' - f'|_{min}}{(n_c - 1)}; 14t; 200mm \right]. \quad \text{Eq(C29)}$$

Finally, the maximum value for f' can be determined from the following equation

$$f'|_{max} = \min [h_f - g'|_{min} - (n_c - 1)h'|_{min}]. \quad \text{Eq(C30)}$$

Figure C4 below shows the Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction of the angle's height are within the specified limits.



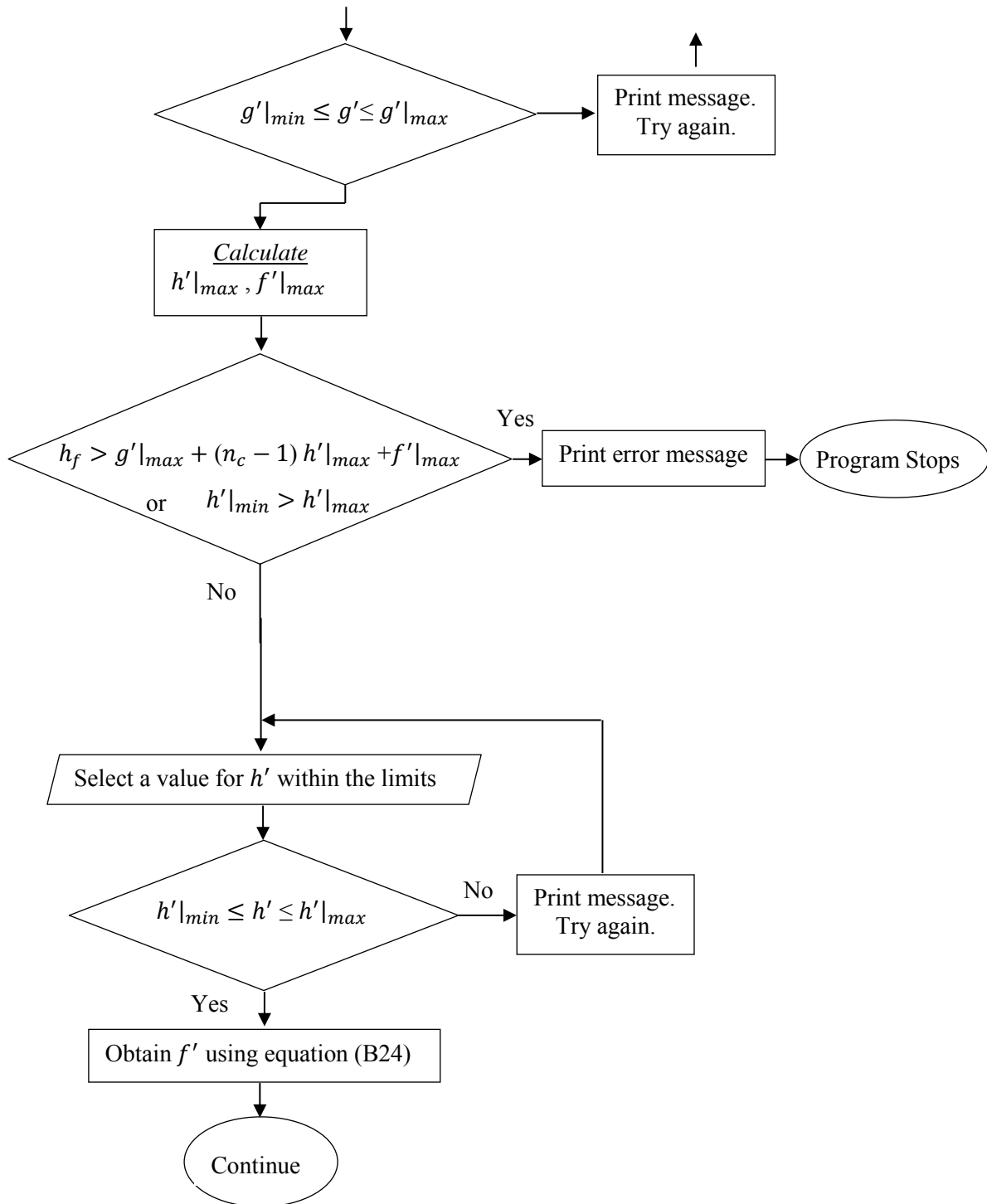


Figure C4 - Flowchart of the pseudocode used to determine whether the minimum distances chosen in the direction of the angle's height are within the specified limits.