ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master’s Thesis

José Humberto Matias de Paula Filho

Numerical modeling of traditional timber connections
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Portugal | 2014
DECLARATION

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ACKNOWLEDGEMENTS

I would like to express my gratitude to my supervisor, Prof. Jorge Manuel Gonçalves Branco, for his guidance and support. Thank you for your teaching, experience and friendship. There are many important contributors that made possible to achieve this work. I am grateful to Nuno Mendes for his support during the numerical modeling work performed at University of Minho. I am also grateful to all the staff of the University of Minho and of the Czech Technical University in Prague. To all my SAHC colleagues and friends, without them this work will not be possible. Special thanks to my friends Wilson Ricardo and Vitor Finnoto who helped me tremendously. Thanks to my family for their love and support. Last but not least, thanks for the Consortium that granted me with an Erasmus Mundus Category A scholarship.
Numerical modeling of traditional timber connections
ABSTRACT

Connections are the critical point in the behavior of traditional timber trusses. The overall behavior and stresses distribution are function of connections strength and stiffness, in particular under non-symmetric loads and extreme events. In these situations, the connections are loaded well beyond their pseudo-elastic limit.

Normally, traditional timber connections are assumed as perfect hinges or rigid links. However, even without any strengthening or improvements, they have a significant rotational stiffness and may be better classified as semi-rigid and friction-based. It is the lack of information in the behavior of connections that normally leads to the full replacement of old timber structures, when other conservative approaches to satisfy serviceability and safety, such as strengthenings, are left behind.

In order to better predict and understand the behavior of existing timber structures, numerical models of their connections are crucial. Numerical models, using nonlinear laws and the Finite Element Method, intend to represent the behavior of traditional timber connections with a comparable level of detail based on experimental results.

In this thesis, structural analysis of full-scale traditional timber connections are presented. Firstly, an extensive experimental program, using full-scale notched connections, investigating the monotonic behavior of traditional timber connections, identifying, and evaluating suitable strengthening techniques with metal devices, performed at the University of Minho during the last years, was analyzed. Then, based on those test results, a numerical model of unstrengthened traditional timber connection was proposed, validated and confirmed. The same numerical model was later then extended to strengthened traditional timber connections with metal devices. Finally, the numerical models were used to better understand the influence of some parameters, such as friction and compression level of the rafter, on the behavior of the studied connections.
RESUMO

As ligações são o ponto crítico no comportamento de treliças de madeira tradicionais. O comportamento geral bem como a distribuição tensões dependem da resistência e rigidez da ligação, em particular sob cargas não-simétricos e casos acidentais. Nestas situações, as conexões são carregados bem além de seu limite de pseudo-elástico.

Normalmente, as ligações de madeira tradicionais são consideradas como rótulas perfeitas ou como totalmente rígidas. No entanto, mesmo sem qualquer reforço ou melhoria, essas ligações possuem uma rigidez à rotação significativa, podendo assim ser melhores classificadas como semi-rígidas e dependente de atrito. É a falta de informação no comportamento das ligações que normalmente leva à substituição completa das estruturas de madeira antigas, quando outras abordagens, mais conservadoras, tais como reforços, são deixados de lado.

A fim de melhor prever e entender o comportamento de estruturas de madeira existentes, os modelos numéricos de suas ligações são de extrema importância. Os modelos numéricos, utilizando leis não-lineares e o Método dos Elementos Finitos, tem a intenção de representar o comportamento das ligações tradicionais da madeira com um nível de detalhe aceitável e baseando-se em resultados experimentais.

Nesta tese, a análise estrutural das ligações tradicionais de madeira em escala real são apresentadas. Primeiro, um extenso programa experimental realizado na Universidade do Minho nos últimos anos, usando conexões com entalhe em escala real, foi elaborado para investigar o comportamento monotônico de ligações tradicionais da madeira, identificar e avaliar técnicas de reforço adequadas com dispositivos metálicos. Então, com base nos resultados dessa campanha experimental, foi proposto um modelo numérico dessas ligações. O mesmo modelo numérico foi, mais tarde, estendido para as conexões tradicionais de madeira reforçadas com dispositivos metálicos. Por fim, os modelos numéricos foram utilizados para melhor compreender a influência de alguns parâmetros, tais como atrito e nível de compressão da viga.
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CHAPTER 1

Introduction

ABSTRACT

This chapter presents a brief introduction about traditional timber trusses and connections. The objectives of this study are presented and it also points out the organization of the thesis and its chapters descriptions.
1.1 GENERAL

Traditional building construction in Portugal (from the 18th to early 20th century) adopted timber roof and floor structures and, in some cases, also timber reinforced masonry walls. The majority of these buildings are still in use, and some of them suffered significant modifications. Even with the popularization of reinforced concrete structures, timber structures continued to play an important role in roof construction.

The common Portuguese timber roof structures are composed of trusses, with different geometries, depending on the architecture of the building, its use and the span of the roof. Traditionally, Portuguese timber roofs are formed by trusses with an average free span of 6 m, mostly following a king-post configuration. Figure 1.1 shows a traditional Portuguese king-post timber truss.

![Figure 1.1: Traditional Portuguese king-post timber truss (JPEE 2006)](image)

Notched joints, in some cases presenting mortise and tenon, single (one notch) or double (two notches), make the connections between truss members. Forces are directly transferred by compression and/or friction. To improve the contact between the connected members, joints are usually strengthened with metal elements.

These metal elements significantly increase the resistance of the joints to out-of-plane loads as well to cyclic loads such as a seismic. Usually, stirrups, binding strip and bolts are used to strength traditional timber connections.

With the spread of these metal devices, the carpentry work, considered as an art, become to be undervalued. Because of that, a significant part of the know-how about the design and construction of timber trusses disappeared with the arrival of the concrete and steel structures. As consequence, wrong design assumptions (model, connections, details, etc.) became common in the case of timber structures, in particular, when dealing with existing timber structures.
Nowadays, traditional timber trusses are analyzed through simple models assuming the connection as rigid links or as perfect hinges. However, their behavior are semi-rigid presenting a significant moment-resisting capacity and they are friction based (Branco, J., 2008). In particular, under non-symmetric loading like wind, snow or earthquake, the connections are crucial, because in these situations the connections are loaded well beyond their pseudo-elastic limit (Branco J., 2008).

The lack of a realistic models for the connections normally leads to the replacement of old timber structures, instead of their repairing and strengthening to satisfy safety. Moreover, the misunderstanding of the overall behavior of the truss and its connections can result in an unrealistic stress distribution in the members, because of inappropriate joints strengthening regarding stiffness and force capacity.

In order to better predict and understand the behavior of existing timber structures, experimental tests and finite element models of traditional timber connections are crucial. The restoration or the design of new timber structures is optimal when the real behavior of the structure is known through tests and/or numerical models.

A research program aimed on investigating the monotonic and cyclic behaviors of traditional timber connections, identifying, and evaluating suitable strengthening techniques with metal devices, was developed by the University of Minho (Portugal) in collaboration with the University of Trento (Italy) and the Timber Structures Division of National Laboratory for Civil Engineering (Portugal). This thesis uses the experimental results obtained in this research to develop a numeric modeling using Finite Element Method for unstrengthened and strengthened traditional timber connections.

1.2 OBJECTIVES

The main goals of the thesis work can be summarized in:

- Evaluation of traditional timber connections behavior and the influence of different strengthening techniques with metal devices;
- Formulation and validation of a numerical model of notched timber connection;
- Formulation and validation of a numerical model of notched timber connections strengthened with metal devices.
1.3 OUTLINE OF THE THESIS

The thesis is organized over the following chapters:

- **Chapter 1 “Introduction”**: This chapter presents a brief introduction of traditional timber trusses and connections. The objectives of this study are presented and the organization of the thesis and its chapters descriptions is presented and discusses;
- **Chapter 2 “State of the art”**: This chapter presents an overview of traditional timber connections history, followed by the characterization depending on their type and mechanical behavior. In addition, strengthening methods are pointed out, and relevant works that has been done in the field of traditional timber connections are presented;
- **Chapter 3 “Experimental analysis of traditional timber connections”**: This chapter presents an overview of the mechanical characterization of the selected wood, followed by the description and analysis of the experimental campaign carried out with unstrengthened and strengthened connections under monotonic loading. Finally, obtained results are analyzed and compared;
- **Chapter 4 “Numerical modelling of unstrengthened traditional timber connections”**: This chapter addresses the development of the numerical model of unstrengthened timber connections (notched). Then, a comparison with experimental results is made to validate and confirm the model. After that, a sensitivity analysis was carried out to analyze the influence of some connection parameters on the behavior of the joint.
- **Chapter 5 “Numerical analysis of strengthened traditional timber connections”**: This chapter presents and overview on how the numerical model of the strengthened connections with binding strip was built. Then, a comparisons with experimental results are made to validate and confirm the model. After that, the model is used to analyze the behavior of the 60° skew angle connection. Finally, results are evaluated and important conclusions are pointed out.
- **Chapter 6 “Conclusions and recommendations”**: This chapter summarizes all the conclusions previously presented. Finally, based on the findings of this work, recommendations are made for future developments.
CHAPTER 2
State of the art for traditional timber joints

ABSTRACT
This chapter presents an overview on traditional timber joints history and then the characterization depending on their type and mechanical behavior. Moreover, strengthening methods are pointed out and relevant past researches in the field of traditional timber connections are presented.
2.1 HISTORICAL OVERVIEW OF TRADITIONAL TIMBER CONNECTIONS

Up to the beginning of 19th century, joints of timber structures were built intuitively based on long century knowledge, i.e. no specific calculations and resistance design were done. Most of the joints were woodworking joints, mainly made by notches, with the main purpose of connecting at least two structural elements. Stresses were directly transferred by compression and friction between the contact surfaces.

Through the industrial revolution and the consequent need for large-scale production, metallic devices such as nails, bolts, stirrups, screws, straps and metal plates started to play an important role in timber connections. These metallic devices were expensive when they first started to be used; nonetheless, they became cheaper as a result of industrial development. Consequently, their use increased considerably.

In this new scenario, carpentry, considered a work of art until then, lost importance being gradually substituted by metallic elements for the correction, improvement and assembling of new timber connections.

Nowadays, because of the lack of carpenters, metallic devices are well spread in modern timber structures. They prevent out of plane instability and promote adequate contact between the elements.

2.2 TYPES OF TRADITIONAL TIMBER CONNECTIONS

Traditional timber connections, due to their arrangement, geometry and purpose, can be categorized in five different types:

- **Lap joint**: they are divided in two types i) full lap joints and ii) half lap joints. The first is characterized by timber elements overlaid by a notch and held in place by a peg; therefore, the thickness of full lap joint is equal to the thickness of the two connected members. Whereas the second is characterized by beams that are hollowed out laterally. In particular, two elements are connected. Because of the material removal, the thickness of the joint is equal to the thickness of the thickest member;
- *Mortise and tenon joint*: timber elements are connected through a mortise and a tenon. A mortise with the exact shape of the tenon tongue is carved in the timber element. The tenon is then inserted into the mortise; generally, square or rectangular connections are used;

- *Scarf joints*: they are mainly used to connect long timber elements; in particular, they are used when either the required length is too long or a plain timber piece is not available. The connection consists of two flat planes meeting on an angle relative to the axis of the elements being joined;

- *Dovetail joints*: also known as swallow-tail joint or fantail joint, this connection is composed of two parts: i) tails and ii) pins. The tails have similar shape to the tail doves. The pins are on the opposite timber element and fit in between the tails;

- *Birdsmouth joints*: it consists of a notched joint, where a notch is cut out of an angled piece. The timber element to be connected is shaped to fit the notch, and then it is put to rest on the cut.

Figure 2.1 shows the above discussed connections.

![Figure 2.1: Most common types of traditional timber joints: (a) Lap joint, (b) Mortise and tenon joint, (c) Scarf joint, (d) Dovetail joint, (e) Birdsmouth joint.](image-url)
2.3 MECHANICAL BEHAVIOR OF TRADITIONAL TIMBER CONNECTIONS

Traditional timber connections work mainly under compression, which can be parallel or at an angle to the fibers, and friction (shear) between contact surfaces. Therefore, timber response to these actions has to be discussed.

When compressed parallel to its fibers, timber shows greater resistance than when compressed perpendicularly to its fibers. When loaded along the grain axis the material’s fibers have a reasonable compression strength until the fibers start to buckle. However, when the fibers are loaded perpendicularly, they tend to crush when compressed. This peculiar behavior of timber characterizes it as an orthotropic material. In summary, timber is more efficient when loaded in grain direction. It is important to refer that tension normally is not present in carpentry joints. Moreover, tension perpendicular to the grain must be avoid, as is the weaker resistance of wood, nearly null. The resisting mechanisms of wood under compression and tension are presented in Figure 2.2.

![Figure 2.2: Resisting mechanism of wood: (a) Strong when loaded parallel to the grain, (b) Weak when loaded perpendicular to the grain.](image)

According to Branco et al. (2008), the design of traditional timber connections depends essentially on the verification of compression transmitted between contact surfaces of connected elements. Bearing this idea in mind, a single step birdsmouth joint between a tie beam and a rafter will be used, as example, to present the main resisting mechanism in a traditional timber connection (see Figure 2.3).
From Figure 2.3, it can be seen that actions on the rafter have the following effects on the tie beam:
- Compression at an angle to the fibers in B;
- Compression perpendicular to the fibers in C;
- Compression perpendicular to the fibers in the region of the support;
- Shear in A due to the horizontal thrust coming from the rafter;
- Shear in D due to the vertical component that comes from the arch;

According to Branco (2008), attention must be drawn to:
- Eccentricity in force transmission;
- Cracks are likely to appear in the step edges;
- Concentration of stresses due to sharpened geometry of the notches;
- Presence of non-negligible tension perpendicular to the grain often followed by significant shear stresses.

2.3.1 STRENGTH

Keeping in mind the connection of Figure 2.3, the slope of the notch has to induce an angle between internal forces and direction of the fibers as minimal as possible in order to increase the resistance of the joint. The most efficient joint configuration (see Figure 2.4) can be geometrically...
deduced; specifically, the slope of the notch is \( \frac{\pi - \alpha}{2} \). The depth of the notch \( (t_v) \) should not exceed \( h/4 \) for sew angles, i.e. \( \alpha \leq 50^\circ \), and \( h/6 \) for skew angles \( \alpha \geq 60^\circ \). These values of notch depth are in accordance with Götz et al. (1993), DIN 1052 (2004) and C.T.E. (2006).

The friction forces and geometric imperfections were neglected in order to simplify calculations. This is a rough assumption that does not represent the exact behavior of the connection. Notice that relying on these assumptions can be sometimes risky, since it is known that friction increases the shear stresses acting on connecting surfaces. Based on this simplification, \( F_1 \) and \( F_2 \) can be formally declared as:

\[
F_1 = N \cdot \cos \left( \frac{\alpha}{2} \right) - N \cdot \sin \left( \frac{\alpha}{2} \right) \cdot \tan \left( \frac{\alpha}{2} - \beta \right) < N \cdot \cos \left( \frac{\alpha}{2} \right),
\]

\[
F_2 = \frac{N \cdot \sin \left( \frac{\alpha}{2} \right)}{\cos \left( \frac{\alpha}{2} - \beta \right)}.
\]

The frontal part of the step should be verified for shear of the notch and for compression at an angle to the grain. In addition, the rear part of the notch should be checked for compression at an angle to the grains. The following equations provide means for the discussed verifications

\[
\tau = \frac{N \cdot \cos \alpha}{b \cdot v} \leq f_v,
\]

Figure 2.4: Resisting mechanism of a birdssmouth joint neglecting friction and geometry imperfections. (Branco, 2008)
Numerical modeling of traditional timber connections

\[ \sigma_c = \frac{N \cdot \cos^2 \left( \frac{\alpha}{2} \right)}{b \cdot t} \leq f_{c,\alpha} \]  

(2.4)

\[ \sigma_c = \frac{p_2}{b \cdot d} \leq f_{c,90-\beta} \]  

(2.5)

where \( d \) and \( b \) are the rear compressed length and the width of the timber element, respectively. According to Parisi and Piazza (2000), the variable \( d \) in Eq.(2.5) is computed by

\[ d \equiv \left( \frac{1}{5} - \frac{1}{3} \right) \cdot \frac{t}{\sin \beta} \]  

(2.6)

where \( t \) is the vertical projection of the step. Attention must be drawn to this check because stresses in the joint may increase considerably depending on \( d \).

2.3.2 STIFFNESS

Traditional timber connections can resist a significant amount of moment even without any strengthening (Branco, 2008). Therefore, they should be considered neither perfect hinges nor rigid links. They must be considered semi-rigid links and friction based. It is safer and more economical to consider traditional timber connections as semi-rigid and friction based. When the connections are modeled as perfect hinges, bending moments are neglected. This approach is rather risky, since the neglected bending moments may induce compression perpendicularly to the grain. Conversely, modeling connections as rigid links will lead to a safe, but not necessarily economic approach. Differences between stresses up to 20% were obtained when assuming semi-rigid behavior of the joints while when assuming the connections pinned or rigid the differences were up to 40% (Uzielli, 2004).

Work carried out on full-scale timber connections by Candelpergher (1999), Palma and Cruz (2007) and Branco (2008) shows that the moment resistance capacity of the connection depends on:

- Compression level of the rafter;
- Width of the rafter;
- The friction angle;
- The skew angle;
- The notch depth.
The moment resistance capacity is related positively with the first three conditions.

2.3.3 DUCTILITY

Ductility corresponds to the ability of a structure to yield, through repetitive inelastic deformations, without great loss of resistance. This behavior is preferred when dealing with seismic actions since it increases, for example, energy absorption capacity.

Timber elements have limited ductility, which can make timber structures less forgiving than steel and reinforced concrete. However, careful designing and detailing of timber structures allows for achieving ductile structural systems. In particular, the introduction and proper use of mechanical connections lead to structures with excellent ductility. Therefore, in timber structures, structural ductility is highly dependent on the structural connections.

According to Branco (2008), the dissipation of energy in timber connections is a result of cyclic yielding of metallic (usually steel) dowel type fasteners of connections (nails, staples, screws, dowels or bolts) and crushing of the wood fibers in contact with the fasteners.

Regarding ductility, the Eurocode 8, 2004 classified timber structures as:

- Ductility Class Medium (DCM).
- Ductility Class High (DCH).
- Ductility Class Low (DCL).

The ductility classes DCM and DCH can be considered as dissipative structures; whereas DCL can be considered as non-dissipative structures.

As discussed in this section, structural ductility classification is strongly dependent on the capacity of the structural connections to dissipate energy. The evaluation of such capacity is more accurate when quantified by tests following the technical standard EN 12512:2001. Since Eurocode 8, does not propose a behavior factor for carpentry joints, testing connections according with EN 12512:2001 is the best way to evaluate the behavior factor and the ductility class classification of timber structures.
2.4 STRENGTHENING OF TRADITIONAL TIMBER CONNECTIONS

As discussed in Section 2.1, the use of metallic devices in timber connections increased during the 19th century when developments in industry allowed for producing bolts, screws and other metallic elements.

At that time, metal devices had mainly one purpose: counteract out of plane actions that the assembly could not resist by itself. Nowadays, adding to this previous concern, metal devices also aim to improve the in plane behavior of the friction-based connection and to avoid detachment of the connected members. This detachment is critical when dealing with high seismicity areas due to a possible decrease of compression force on the friction surfaces.

Under cyclic loading, strengthening methods can make the behavior of the structure more stable (Parisi and Piazza, 2000 and Branco, 2008). Figure 2.5 introduces some traditional strengthening methods.

![Traditional strengthening methods](image)

Figure 2.5: Traditional strengthening methods: (a) Stirrups, (b) Internal bolt, (c) Binding strip, (d) Tension ties. (Branco, 2008)

2.5 EXISTING STUDIES ON TRADITIONAL TIMBER CONNECTIONS

Improving the knowledge on the behavior of traditional timber connections is not an easy task since timber connections comprise material and geometrical uncertainties, and there are no regulated methods or codes that take traditional timber joints in consideration.
In general, these types of connections can be evaluated through:

- Observation of the pathology of existing timber structures in an extensive and representative site survey;
- Numerical modeling using computer software such as TNO DIANA, ABAQUS, ANSYS, to mention a few. The studied connection is simulated in form of finite elements method (FEM);
- Full-scale models, where the timber connections is tested and information such as forces and displacements are acquired and interpreted.

Traditional timber joints have been extensively tested and modelled and important work in the field has been developed by Dradácky, Wald and Mares (1994), Parisi and Piazza (1998), Parisi and Piazza (2000), Feio (2005), Chang, Hsu and Komatsu (2006), Palma and Cruz (2007) and Guan, Kitamori and Komatsu (2008) (see Table 2.1).

Since it is the basis of this work, attention must be drawn to the experimental campaign, followed by numerical modelling, carried out at the University of Minho (Portugal) and at the University of Trento (Italy) by Branco (2008). The objective of this work was to determine the influence of the joints stiffness on the monotonic and cyclic behavior of traditional timber trusses. Additionally, the assessment of the efficacy of different strengthening techniques was evaluated. Figures 2.6 and 2.7 show some of the experimental configuration of the tested connections.

<table>
<thead>
<tr>
<th>AUTHORS</th>
<th>CONNECTION TYPE</th>
<th>TYPE OF LOADING</th>
<th>TEST OBJECTIVE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feio, A., 2006</td>
<td>Mortise and tenon</td>
<td>Cyclic</td>
<td>Compression capacity</td>
</tr>
<tr>
<td>Palma, P. and Cruz, H., 2007</td>
<td>Single step birdsmouth joint with and without mortise and tenon, with and without metal devices.</td>
<td>Monotonic and Cyclic</td>
<td>Mechanical behavior</td>
</tr>
<tr>
<td>Guan, Z., Kitamori, A., and Komatsu, K., 2008</td>
<td>Nuki joint</td>
<td>Monotonic</td>
<td>Load carrying capacity</td>
</tr>
</tbody>
</table>
Figures 2.6 and 2.7 show some of the experimental configuration of the tested connections.

Figure 2.6: Single step birdsmouth joints: (a) Branco, J., 2008, (b) Palma, P. and Cruz, H., 2007, (c) Feio, A., 2006.

Figure 2.7: Nuki joints: (a) Chang, W., Hsu, M., and Komatsu, K., 2006, (b) Guan, Z., Kitamori, A., and Komatsu, K., 2008.
CHAPTER 3
Experimental analysis of traditional timber connections

ABSTRACT
This chapter presents an overview of the experimental campaign carried out by Branco (2008) which served to develop the numeric models of the traditional timber connections presented in the following chapters. Here, a brief summary of mechanical characterization of the selected timber, followed by the description and analysis of the experimental campaign carried out with unstrengthened and strengthened connections under monotonic loading is presented. Finally, the obtained results are analyzed and compared.
Numerical modeling of traditional timber connections
3.1 INTRODUCTION

The experimental campaign described here was carried out at the Laboratory of Structures of University of Minho, in Portugal. It includes monotonic loading tests of full-scale traditional timber connections, notched joints, and material characterization of the wood used. Further information on this subject is found in Branco (2008).

Firstly, mechanical properties of the timber elements were determined. Then, monotonic loading tests were performed on unstrengthened specimens in order to understand the behavior of the connection and its characteristics as well as the sensitivity to a few parameters, compression level of the rafter and skew angle. Finally, strengthened specimens with basic metal devices were tested, at the same conditions, in order to determine investigate differences in the behavior of the connection or in the metal device itself determining the need for different strengthening types.

Surveys done in historical constructions in Portugal pointed out a higher frequency of skew angles of 30° and 60°. These two angles were adopted on the full-scale specimens. In addition, the compression level of the rafter of 1.4 MPa and 2.5 MPa, were adopted. These values are in accordance with the Serviceability Limit State defined for common Portuguese timber structures (Branco et al., 2005b).

For all types of unstrengthened connections, 6 monotonic loading tests, 3 in each load direction (positive and negative), were considered. Four types of strengthening techniques were tested and later one was discarded. Strengthened connections were only tested with the rafter under a compression level of 1.4 MPa.

Table 3.1 summarizes the most important tests performed on traditional timber connection in this experimental campaign. Only the monotonic tests are presented, despite cyclic tests were performed in that experimental campaign, because the numerical modelling will address only this type of loading.

3.2 TIMBER CHARACTERIZATION

The selected wood specie for the experimental campaign and fabrication of all full-scale models was the Maritime pine timber (Pinus pinaster Ait.). Its mechanical properties were determined and characterized.
The timber elements were visually strength graded and classified as quality class E in accordance with the Portuguese National Standard NP 4305 (1995). Samples collected during the fabrication of the connections were tested at the laboratory following the EN 408 (2003) prescriptions. These tests were divided in two phases:

- 1\textsuperscript{st}: When connections with the skew angle of 30° were fabricated, tests were done to determine the local and global modulus of elasticity in bending and the bending strength.
- 2\textsuperscript{nd}: When connections with the skew angle of 60° were fabricated, tests were done to determine the compression strength perpendicular and parallel to the grain and modulus of elasticity.

The failure time and moisture content of the specimens were measured with a wood electronic thermo hygrometer.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading method</th>
<th>Type of connection</th>
<th>Compression level [MPa]</th>
<th>Skew angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>U3-1,4 - 1,2,3</td>
<td>Monot. +</td>
<td>Unstrengthened</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>U3-1,4 - 4,5,6</td>
<td>Monot. -</td>
<td>Unstrengthened</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>U3-2,5 - 1,2,3</td>
<td>Monot. +</td>
<td>Stirrup</td>
<td>30°</td>
<td></td>
</tr>
<tr>
<td>U3-2,5 - 4,5,6</td>
<td>Monot. -</td>
<td>Bolt</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>S3 - 1,2,3</td>
<td>Monot. +</td>
<td>Rigid binding strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3 - 4,5,6</td>
<td>Monot. -</td>
<td>Binding strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B3 - 1,2,3</td>
<td>Monot. +</td>
<td>Unstrengthened</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>B3 - 4,5,6</td>
<td>Monot. -</td>
<td>Unstrengthened</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>S6 - 1,2,3</td>
<td>Monot. +</td>
<td>Stirrup</td>
<td>60°</td>
<td></td>
</tr>
<tr>
<td>S6 - 4,5,6</td>
<td>Monot. -</td>
<td>Bolt</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>B6 - 1,2,3</td>
<td>Monot. +</td>
<td>Rigid binding strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B6 - 4,5,6</td>
<td>Monot. -</td>
<td>Binding strip</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.2.1 PROPERTIES IN BENDING

The timber properties in bending were determined based on four-point bending tests on beams with a cross section of 80x100 mm² and 1.90 m length. The tests were performed in accordance with the technical standard EN 408 (2003). A 100 kN load cell and a displacement control procedure of 0.14 mm/s were used in all cases.

The data from the four-bending tests allowed for computing the local modulus of elasticity \(E_{m,l}\) and the global modulus of elasticity \(E_{m,g}\). The bending strength \(f_m\) of the specimen was also measured by loading the specimens up to failure. The bending tests results obtained are presented in Table 3.2.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Skew angle</th>
<th>W [%]</th>
<th>(F_{\text{max}}) [kN]</th>
<th>Time [s]</th>
<th>(E_{m,l}) [MPa]</th>
<th>(E_{m,g}) [MPa]</th>
<th>(f_m) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>30°</td>
<td>24.3</td>
<td>22.32</td>
<td>415</td>
<td>10249</td>
<td>9902</td>
<td>54.71</td>
</tr>
<tr>
<td>B2</td>
<td></td>
<td>20.8</td>
<td>19.76</td>
<td>380</td>
<td>9935</td>
<td>9658</td>
<td>44.45</td>
</tr>
<tr>
<td>B3</td>
<td>23.8</td>
<td>19.60</td>
<td>374</td>
<td>207</td>
<td>8894</td>
<td>8666</td>
<td>26.98</td>
</tr>
<tr>
<td>B4</td>
<td>22.3</td>
<td>11.99</td>
<td>226</td>
<td>11560</td>
<td>13384</td>
<td>78.42</td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td></td>
<td>10427</td>
<td>10169</td>
<td>69.77</td>
<td></td>
</tr>
<tr>
<td>CoV</td>
<td></td>
<td></td>
<td></td>
<td>0.16</td>
<td>0.18</td>
<td>0.27</td>
<td></td>
</tr>
<tr>
<td>B5</td>
<td>60°</td>
<td>19.6</td>
<td>31.01</td>
<td>420</td>
<td>12021</td>
<td>11221</td>
<td>69.77</td>
</tr>
<tr>
<td>B6</td>
<td></td>
<td>16.6</td>
<td>12.16</td>
<td>152</td>
<td>8552</td>
<td>8144</td>
<td>27.36</td>
</tr>
<tr>
<td>B7</td>
<td>18.7</td>
<td>27.46</td>
<td>420</td>
<td>8808</td>
<td>9184</td>
<td>61.79</td>
<td></td>
</tr>
<tr>
<td>B8</td>
<td>21.4</td>
<td>34.85</td>
<td>351</td>
<td>13468</td>
<td>13834</td>
<td>78.42</td>
<td></td>
</tr>
<tr>
<td>B9</td>
<td>18.9</td>
<td>20.61</td>
<td>235</td>
<td>10390</td>
<td>9754</td>
<td>46.38</td>
<td></td>
</tr>
<tr>
<td>B10</td>
<td>20.3</td>
<td>21.57</td>
<td>226</td>
<td>11560</td>
<td>10768</td>
<td>48.52</td>
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<tr>
<td>Mean</td>
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<td></td>
<td></td>
<td>10800</td>
<td>10409</td>
<td>55.37</td>
<td></td>
</tr>
<tr>
<td>CoV</td>
<td></td>
<td></td>
<td></td>
<td>0.18</td>
<td>0.18</td>
<td>0.33</td>
<td></td>
</tr>
</tbody>
</table>

As expected, the local modulus of elasticity \(E_{m,l}\) is greater than the global modulus of elasticity \(E_{m,g}\) in all cases, since no shear deformation is taken into account for the first and it is taken into account for the second.

The results obtained in both sets of bending tests present low variability in terms of the local modulus of elasticity \(E_{m,l}\) and global modulus of elasticity \(E_{m,g}\). On the other hand, greater
variability was observed on the bending strength results ($f_m$). In particular, beams B4 and B6 failed at considerable lower stress levels than all other tested beams.

Values obtained for global and local modulus of elasticity in bending are quite consistent, since lower CoV were observed. Bending strength ($f_m$) values show more dependence of eventual defects of the wood, they are quite heterogeneous, where the minimum value obtained represents 64% of the mean value.

Figure 3.1 shows the load-deformation curves obtained in the bending strength tests at the mid-span of the specimens.

![Figure 3.1: Load mid-span deformation curves obtained in bending tests (Branco, 2008).](image)

### 3.2.2 COMPRESSION PERPENDICULAR TO THE GRAIN

To determine the compression perpendicular to the grain, specimens of 45 x 70 x 90 mm³ were used and all recommendations prescribed by EN 408 (2003) were followed. A load cell of 200 kN and a displacement control procedure with a displacement rate of 0.005 mm/s were used with the objective to reach the maximum load within 300 ± 120 s.

Deformations were measured with LVDT’s (Linear Voltage Differential Transducer). The force deformation curve was plotted using the average deformation values obtained by the LVDT’s placed on the larger sides (i.e. 70 x 90 mm² faces) as shown in Figure 3.2.

For the determination of the maximum load ($F_{c,90,max}$) all EN 408 (2003) prescriptions were followed. Table 3.3 presents the results of the compression tests perpendicular to the grain.
Only one test result (C90-2) failed to fulfill the EN 408 (2003) requirements since the time of failure was out of the allowable range. The results from this test were discarded.

Values for the compression strength perpendicular to the grain ($f_{c,90}$) present low variability (in particular, $\text{CoV}=0.06$), but lower than expected. Even though presenting larger variability, the modulus of elasticity perpendicular to the grain ($E_{c,90}$) exhibit low variability.

![Figure 3.2: Force-deformation curves of the compression perpendicular to the grain (Branco, 2008).](image)

Table 3.3: Compression perpendicular to the grain tests results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$W$ [%]</th>
<th>Time [s]</th>
<th>$F_{c,90,\text{max}}$ [kN]</th>
<th>$F_{c,90}$ [kN]</th>
<th>$E_{c,90}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C90-1</td>
<td>16.3</td>
<td>278</td>
<td>14.23</td>
<td>4.52</td>
<td>469</td>
</tr>
<tr>
<td>C90-2*</td>
<td>18.2</td>
<td>117</td>
<td>11.96</td>
<td>3.80</td>
<td>339</td>
</tr>
<tr>
<td>C90-3</td>
<td>14.9</td>
<td>332</td>
<td>15.83</td>
<td>5.03</td>
<td>356</td>
</tr>
<tr>
<td>C90-4</td>
<td>16.5</td>
<td>259</td>
<td>14.24</td>
<td>4.52</td>
<td>312</td>
</tr>
<tr>
<td>C90-5</td>
<td>14.8</td>
<td>218</td>
<td>15.75</td>
<td>5.00</td>
<td>482</td>
</tr>
<tr>
<td>C90-6</td>
<td>16.8</td>
<td>200</td>
<td>13.45</td>
<td>4.27</td>
<td>307</td>
</tr>
<tr>
<td>C90-7</td>
<td>16.6</td>
<td>222</td>
<td>13.64</td>
<td>4.33</td>
<td>383</td>
</tr>
<tr>
<td>C90-8</td>
<td>15.4</td>
<td>280</td>
<td>14.10</td>
<td>4.48</td>
<td>381</td>
</tr>
<tr>
<td>C90-9</td>
<td>15.2</td>
<td>234</td>
<td>14.12</td>
<td>4.48</td>
<td>386</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>4.58</td>
<td>385</td>
<td></td>
</tr>
<tr>
<td>CoV</td>
<td></td>
<td></td>
<td>0.06</td>
<td>0.17</td>
<td></td>
</tr>
</tbody>
</table>

*test results discarded based on EN 408 (2003).
3.2.3 COMPRESSION PARALLEL TO THE GRAIN

For the determination of compression parallel to the grain, specimens of 80 x 100 x 480 mm³ were used and all recommendations prescribed by EN 408 (2003) were followed. A load cell of 500 kN and a displacement control procedure with a movement rate of 0.01 mm/s were used with the objective to reach the maximum load within 300 ± 120 s.

Deformations were measured with LVDT’s. The force-deformation curve was plotted using the average deformation value obtained by the LVDT’s placed on each side of the specimen, as shown in Figure 3.3

Based on the deformation measured within the elastic range, it was possible to quantify the modulus of elasticity ($E_{c,0}$), and with the maximum force value ($F_{c,0,max}$) to quantify the compression strength parallel to the grain ($F_{c,0}$).

Table 3.4 presents the results of the compression tests parallel to the grain.

![Figure 3.3: Force-deformation curves of the tests of compression parallel to the grain (Branco, 2008).](image)

3.2.2 DISCUSSION OF THE TESTS RESULTS

The reference values from LNEC (1997) used for the comparison with the data obtained in the present experimental investigation were collected from two sets of specimens with non-structural dimensions; specifically, $4 \times 10 \times 200$ cm³ and $5 \times 15 \times 300$ cm³. Each set was composed of pieces from different regions, trying to represent the national population of this wood specie.

The results obtained in the two samples were corrected by adjustment factors suggested by EN 384 (2004) based on sample size, timber size and tested length. The same values are adopted by the Portuguese Nationally Determined Parameters of Eurocode 5 (NP ENV 1995-1-1, 1998).
Table 3.4: Compression parallel to the grain tests results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>W [%]</th>
<th>Time [s]</th>
<th>$F_{c,90,\text{max}}$ [kN]</th>
<th>$F_{c,90}$ [kN]</th>
<th>$E_{c,90}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C0-1</td>
<td>16.3</td>
<td>420</td>
<td>253.34</td>
<td>31.67</td>
<td>9125</td>
</tr>
<tr>
<td>C0-2</td>
<td>28.2</td>
<td>408</td>
<td>208.95</td>
<td>26.12</td>
<td>9589</td>
</tr>
<tr>
<td>C0-3</td>
<td>29.1</td>
<td>399</td>
<td>181.06</td>
<td>22.63</td>
<td>8170</td>
</tr>
<tr>
<td>C0-4</td>
<td>25.1</td>
<td>381</td>
<td>155.85</td>
<td>19.48</td>
<td>10410</td>
</tr>
<tr>
<td>C0-5</td>
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<td>382</td>
<td>168.29</td>
<td>21.04</td>
<td>10993</td>
</tr>
<tr>
<td>C0-6</td>
<td>18.7</td>
<td>288</td>
<td>259.47</td>
<td>32.43</td>
<td>9380</td>
</tr>
<tr>
<td>C0-7</td>
<td>27.3</td>
<td>368</td>
<td>212.76</td>
<td>26.59</td>
<td>6552</td>
</tr>
<tr>
<td>C0-8</td>
<td>18.6</td>
<td>420</td>
<td>238.86</td>
<td>29.86</td>
<td>13270</td>
</tr>
<tr>
<td>C0-9</td>
<td>18.3</td>
<td>290</td>
<td>271.35</td>
<td>33.52</td>
<td>10053</td>
</tr>
</tbody>
</table>

Mean 27.08 9727

CoV 0.19 0.19

To compare the test results obtained in this work with the ones reported in LNEC (1997), it is also necessary to apply the adjustment factors suggested by EN 384 (2004) to timber size and moisture content. Notice that adjustment factors for the number and size of samples were not used because of the number of tests performed.

The comparison of tests results obtained in this experimental campaign with the values reported by LNEC (1997), for *Pinus pinaster* Ait. of quality class E, yields the following conclusions:

- all tests results obtained for bending and compression strength parallel to the grain ($f_m$ and $f_{c,0}$) were greater than the values suggested by LNEC (1997);
- all tests results obtained for the compression strength perpendicular to the grain ($f_{c,90}$) were lower than the value suggested by LNEC (1997);
- test results obtained for the modulus of elasticity in bending, mean and characteristic values, are similar to the reported in LNEC (1997).

The comparison between mechanical properties for *Pinus Pinaster*, Ait. Quality class E are summarized in Table 3.5.

The sample size is rather small (considering that the objective is to characterize the mechanical properties of the Maritime pine timber) and the moisture content stems significant variability. Therefore, the analysis of the tests results must be done carefully.
Table 3.5: Comparison of mechanical properties for Pinus Pinaster, Ait. Quality class E obtained in the test results and mechanical properties reported by LNEC (1997).

<table>
<thead>
<tr>
<th>Source</th>
<th>$E_m$ [MPa]</th>
<th>$E_{0.05}$ [MPa]</th>
<th>$f_m$ [MPa]</th>
<th>$f_{c,0}$ [MPa]</th>
<th>$f_{c,90}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LNEC (1997)</td>
<td>12000</td>
<td>8000</td>
<td>18</td>
<td>18</td>
<td>6.9</td>
</tr>
<tr>
<td>Test results</td>
<td>11156</td>
<td>7992</td>
<td>25</td>
<td>20</td>
<td>4.9</td>
</tr>
</tbody>
</table>

As a natural material, wood presents significant variability in physic and mechanical properties, which are highly dependent on faults and moisture content. In addition, significant variability on the mechanical properties exists as function of the tree class quality and location of the specimens on the stem (Machado and Cruz, 2005).

### 3.3 TEST SETUP AND INSTRUMENTATION

The test arrangement allows independent control of two hydraulic jacks as shown in Figure 3.4. One hydraulic jack, aligned with the rafter, induced constant compression throughout the test. Whereas the other, a double-acting hydraulic jack positioned above the center of the joint, applied a transversal force, with a programmed load cycle, and generated a moment at the joint.

Load-displacement curves were determined from measurements. The two hydraulic jacks have a maximum loading capacity of 50 kN and 100 kN and a maximum stroke of 160 mm and 50 mm, respectively. The type and location of instrumental channels, including load cells and (LVDTs), are shown in Figure 3.4.

Tests were performed under displacement control for the typical notched joint with skew angle of 30° and 60°. In all cases, the cross sections of the elements were $80 \times 220$ mm², the step depth was of 45 mm and the notch length was of 422 mm as displayed in Figure 3.5.

The first load applied in monotonic test was an axial compressive force on the rafter. This axial force, simulating the dead load of the structure, was held constant during the test. Subsequently, a transversal force ($F$), acts perpendicular to the rafter axis. The load direction is defined as “positive” when the skew angle increases, and as “negative” when the skew angle decreases.
Figure 3.4: Testing apparatus and instrumentation layout (Branco et al., 2011).

Figure 3.5: Full-scale models geometry: (a) 30° skew angle, and (b) 60° skew angles (Branco, 2008).

The monotonic tests were performed to determine the elastic behavior of the connection; in particular, the apparent elastic limit displacement $d_{+e}$ and $d_{-e}$. Under displacement control at channel 00, located in Figure 3.4, a maximum displacement value of 50 mm was imposed under a movement rate of 0.028 mm/s and 0.18 mm/s for connections with skew angle of 30° and 60°, respectively.

### 3.4 STRENGTHENING TECHNIQUES STUDIED

The four basic types of strengthening considered in this study are modern implementations of traditional strengthening techniques; specifically, the stirrups, internal bolt, binding strip and tension ties shown in Figure 3.5.
Metal stirrups placed in pairs at the opposite sides of the joint were commonly used in the past and are still adopted frequently. In this experimental investigation, each stirrup was composed of two steel plates welded in a V-shape. Each leg was 50 mm wide and 5 mm thick. The legs were placed parallel to the rafter and tie beam, and bolted to the timber with seven bolts that were 10 mm in diameter.

A steel rod, with 12 mm in diameter, was fixed by a nut in both ends and secured by a rectangular-shaped washer (70 × 30 mm² and 5 mm thick). The rod was located in the middle of the joint and perpendicularly to the tie beam axis.

Metal binding strips were built in two configurations:

- The joint was bound with a hollow steel ribbon, 50 mm wide and 5 mm thick, located in the middle of the joint and perpendicularly to the tie beam (BSi).
- The joint was bound with two steel plates located at the bottom surface of the tie beam and at the upper surface of the rafter, with the dimensions of 40 × 159 mm² and 10 mm thick, tightened through two rods of 12 mm diameter (TTi). The rods, having a nut in both ends, are located in middle of the joint and perpendicularly to the tie beam. They enable full control in the tightening force during the lifetime of the connection.

The first version of the metal binding strips was used only for 30° skew angle case.

3.5 ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS

3.5.1 UNSTRENGTHENED CONNECTIONS UNDER MONOTONIC LOADING

The first set of full-scale models tested was composed of three unstrengthened joints with a skew angle of 30°, namely U3-1.4-1, 2 and 3. A permanent compression stress of 1.4 MPa was applied on the rafter by a vertical hydraulic jack, and a monotonic transversal force was gradually applied perpendicularly to rafter axis by a second hydraulic jack.

An elasto-plastic behavior was observed in all three tests as shown in Figure 3.7. The behavior can be assumed perfectly elastic until a displacement of 8 mm was reached, then the behavior started to be non-linear, but only within a small range. Subsequently, a quasi-perfect plastic
behavior appeared. This pseudo-plastic phase, starting at a 10 mm displacement, remained practically constant until a maximum displacement of 50 mm was reached, presenting a slight resistance decrease after 25 mm displacement.

The subsequent sets of tests aimed to evaluate the influence of the compressive stress applied to the rafter. The first set was analyzed under 1.4 MPa in compression. Next, a compressive stress of 2.5 MPa was used.

When the connection was loaded in the positive direction comparing the force-displacement curves of the two different compression levels, it was observed an increase in the maximum force and stiffness when the compression level is increased as shown in Figure 3.7. With a higher compression stress of the rafter, the stiffness of the connection more than duplicates. The maximum force reached with a compression level of 2.5 MPa is 61.3% greater than that of 1.4 MPa, but the elastic displacement decreases 34.2%. However, the development of the force-displacement curves remains similar.

A more brittle behavior and extremely dependent on the frictional characteristic of the connection was observed when the connection was loaded in the negative direction. The curves presented in Figure 3.7 show an elastic behavior until the maximum force is reached. After that, a slip of the touching members, tie beam and rafter occurs concurrently with a loss of friction, inducing a sharp decrease in resistance. When the joint stabilizes itself again, the brittle behavior is replaced with a pseudo-plastic phase. The observed ductile behavior is likely a result of a local compression of the wood. Finally, a total loss of friction occurs and the connection fails.
Comparing the force-displacement curves obtained in the experimental investigation of the two rafter compressive stresses on the negative direction, only an increase in the maximum force and corresponding elastic limit displacement can be pointed out, see Figure 3.9. A similar brittle behavior is observed after the elastic displacement limit is reached. The development of the force-displacement curves, in particular their stiffness, remains constants.

Comparing the force-displacement curves obtained for both rafter compression stress levels, under monotonic loading in positive and negative directions, shown in Figure 3.9, important conclusions can be pointed out. The behavior of the curve is asymmetric. The elastic-plastic behavior presented in the positive direction turns into a brittle and more friction-dependent in the negative direction. Such difference stems from the distinct mechanism developed in the joint. The ductile behavior of wood under compression, contrary to the fragile behavior that is characteristic
of the friction performance, which dictates the behavior in the negative loading direction. No failure is reached in the positive loading direction, while an early failure happened at about 15 to 20 mm of displacement in the negative direction.

For a rafter compressive stress of 1.4 MPa, the stiffness and the maximum force obtained for the positive loading direction represents 50% of the ones measured in the negative direction. The exact values measured in the experimental investigation are listed in Table 3.6. Conversely, the behavior in the positive loading direction is more ductile than the behavior in the negative direction, allowing greater displacements and elastic limits. For a rafter compressive stress of 2.5 MPa, the stiffness in both directions is considered similar, but the maximum force is greater in the negative loading direction than in the negative loading direction.

The stiffness values shown in the Table 3.6 was calculated using the following methodologies:

- force-displacement linear regression, where the coefficient of determination should be greater than 0.99;
- division of force \((F_e)\) by displacement \((d_e)\) measured in the elastic limit;
- division of force \((F_e^{50\%})\) by the corresponding displacement \((d_e^{50\%})\);

![Figure 3.9: Force-displacement curves of unstrengthened connections with 30° skew angle for monotonic tests under different rafter compression stress (Branco, 2008).](image)
Table 3.6: Effect of the rafter compression levels on unstrengthened connection with 30° skew angle under monotonic loading.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression Compressive stress [MPa]</th>
<th>$d_e$ [mm]</th>
<th>$F_{max}$ [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>$F_e/d_e$</th>
<th>$F_{e50%}/d_{e50%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>1.4</td>
<td>8.31</td>
<td>6.72</td>
<td>674</td>
<td>674</td>
<td>634</td>
<td>647</td>
</tr>
<tr>
<td></td>
<td>-5.76</td>
<td>-10.75</td>
<td></td>
<td></td>
<td>1771</td>
<td>1785</td>
<td>1958</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>5.47</td>
<td>10.84</td>
<td>1569</td>
<td>1569</td>
<td>1389</td>
<td>1408</td>
</tr>
<tr>
<td></td>
<td>-8.13</td>
<td>-15.32</td>
<td></td>
<td></td>
<td>1705</td>
<td>1661</td>
<td>1758</td>
</tr>
</tbody>
</table>

For a skew angle of 60°, some differences in the monotonic tests results under the two different rafter’s compression stress (see Figure 3.10) have to be pointed out. The asymmetric behavior between negative and positive loading directions is still present, but the difference is less visible. With a 60° skew angle, the same elasto-plastic behavior is observed in the positive loading direction. Also, the brittle nature of the frictional resistant mechanism in the negative loading direction is reduced. This reduction is explained by a change in the step angle. The increase in the compression component in the grain direction, which is caused by a reduction in the step angle, makes the resistant mechanism in the negative loading direction less friction-dependent.

The mean values of the maximum force ($F_{max}$) and the elastic displacement ($d_e$) in both directions are different, but the stiffness values are similar, see Table 3.7. The elastic stiffness of the connection seems not to depend on the loading direction and does not change with the rafter compressive stress level. The maximum force and the elastic displacements values increase with the compressive stress in the rafter.

Figure 3.10: Force-displacement curves of unstrengthened connections with 60° skew angle for monotonic tests under different rafter compression stress (Branco, 2008).
Table 3.7: Effect of the rafter compression levels on unstrengthened connection with 60° skew angle under monotonic loading (Branco, 2008)

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>$d_e$ [mm]</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>Regression</th>
<th>$F_e/d_e$</th>
<th>$F_{e/50%}/d_{e/50%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>60°</td>
<td>1.4</td>
<td>2.98</td>
<td>3.31</td>
<td>621</td>
<td>668</td>
<td>882</td>
</tr>
<tr>
<td></td>
<td>-5.32</td>
<td>-5.12</td>
<td>-9.02</td>
<td>692</td>
<td>715</td>
<td>779</td>
</tr>
</tbody>
</table>

The effect of rafter compression stress and skew angle on the behavior of unstrengthened connection under monotonic loading are compared in Figure 3.11. While the effect of the first variable has already been discussed, the change of the skew angle value results in:

- when the skew angle is increased, the connection is less resistant and the elastic stiffness decreases;
- when the skew angle is decreased, the connection is more friction-dependent when loaded in the negative direction;
- when the skew angle is increased, the difference between the elastic stiffness on both loading directions is decreased;

Figure 3.11: Force-displacement curves of unstrengthened connections with 30° and 60° skew angle for monotonic tests under different rafter compression stress (Branco, 2008).
3.5.2 STRENGTHENED CONNECTIONS UNDER MONOTONIC LOADING

Four strengthening methods, representing modern implementations of traditional techniques using metal elements, were studied. Three of them, i.e. stirrup, bolt and binding strip, were analyzed for 30° and 60° skew angle, while the forth, i.e. a rigid binding strip, was only evaluate for a 30° skew angle. This last technique was discarded from the experimental campaign, due to implementation difficulties.

All strengthening techniques were evaluated under monotonic and cyclic loading for a constant rafter compressive stress of 1.4 MPa. Notice, however, that only monotonic loading will be subject of this work. Further information on cyclic loading results and analyses can be found in Branco (2008). Tests under monotonic loading were performed to determine the cyclic test procedure for each strengthening technique. In addition, it yields relevant information on how to improve the connection in terms of ductility and resistance.

3.5.2.1 STIRRUP

Force-displacement curves obtained from the connections strengthened with stirrups under monotonic loading for 30° and 60° skew angles are shown in Figure 3.12.

Figure 3.12: Force-displacement curves of connections strengthened with stirrups, 30° and 60° skew angles under monotonic loading on both directions (Branco, 2008).
Small differences were observed in the strengthened connections’ behavior depending on the loading direction. The main difference detected was the variation of stiffness regarding the direction of loading in the case of 30° skew angle. The effect of the skew angle is related to:

- resistance and stiffness decrease with the increase in the skew angle;
- increase in the apparent elastic displacement limit \( (d_e) \), in the positive direction, with the increase in the skew angle.

The maximum force increases over the entire test, thus indicating that the maximum force value could be higher if the tests were extended.

Table 3.8 summarizes the monotonic tests results obtained for strengthened connections with stirrups under monotonic loading.

At the end of the tests, different damages were observed (see Figure 3.13). In the negative direction, local compression in the posterior part of the notch can be seen, while in the positive direction, the failure of the stirrup happens, proving that they were not designed properly.

Table 3.8: Monotonic tests results of strengthened connections with stirrups.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>( d_e ) [mm]</th>
<th>( F_{\text{max}} ) [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>( F_e/d_e )</th>
<th>( F_e^{50%}/d_e^{50%} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>1,4</td>
<td>5,62</td>
<td>16,48</td>
<td>1517</td>
<td>1312</td>
<td>1239</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5,79</td>
<td>-15,34</td>
<td>1428</td>
<td>1257</td>
<td>1310</td>
<td></td>
</tr>
<tr>
<td>60°</td>
<td>2,5</td>
<td>18,71</td>
<td>9,23</td>
<td>377</td>
<td>360</td>
<td>356</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-8,51</td>
<td>-9,38</td>
<td>526</td>
<td>546</td>
<td>580</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.13: Final position of the strengthened connection under monotonic loading and 30° skew angle: (a) Negative loading direction, (b) Positive loading direction (Branco, 2008).
3.5.2.2 BOLT

Force-displacement curves obtained for the connections strengthened with bolts, under monotonic loading for 30° and 60° skew angles, show a full non-linear development with high ductility. The results are displayed in Figure 3.14.

The experimental results were very consistent, only diverging in the maximum force \((F_{\text{max}})\) for a 30° skew angle connections under monotonic positive loading.

Higher is the skew angle, the smaller is the resistance and the smaller is the stiffness achieved by the strengthened connection. For skew angle of 60°, the loading direction does not seem to influence the connection resistance; nonetheless, it affects the connections’ stiffness and the apparent elastic displacement limit \((d_e)\).

Despite the asymmetry observed on the values of the apparent displacement limit \((d_e)\) and on the maximum force reached \((F_{\text{max}})\) for a 30° skew angle, the stiffness does not depend on the loading direction. Table 3.9 summarizes the monotonic tests results obtained for strengthened connections with bolts.

The ductility that results from the strengthening is greater when the bolt is aligned with the force caused by the external load applied. The best configuration is achieved for a 30° skew angle under monotonic loading in the negative direction. It corresponds to the case when the tension is higher on the bolt. The higher the tension on the bolt, the higher the connection resistance and more ductile behavior is obtained.
Table 3.9: Monotonic tests results of strengthened connections with bolts.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>$d_e$ [mm]</th>
<th>$F_{max}$ [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>$F_e/d_e$</th>
<th>$F_{e50%/d_e50%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>1.4</td>
<td>4.94</td>
<td>13.83</td>
<td>1468</td>
<td>1381</td>
<td>1459</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-7.83</td>
<td>-22.81</td>
<td>1488</td>
<td>1387</td>
<td>1409</td>
<td></td>
</tr>
<tr>
<td>60°</td>
<td>2.5</td>
<td>10.92</td>
<td>7.85</td>
<td>442</td>
<td>446</td>
<td>465</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-5.22</td>
<td>-7.08</td>
<td>623</td>
<td>657</td>
<td>838</td>
<td></td>
</tr>
</tbody>
</table>

When loading in the positive direction, the bolt is tensioned but it is also subjected to bending; particularly, for a 30° skew angle shown in Figure 3.15. In the negative direction, another resistant mechanism is noticed; specifically, the local compression perpendicular to the grain in the tie beam that can be seen in Figure 3.16.

As observed in the unstrengthened connections, but now amplified due to the strengthening effects, local compression of wood is observed in the back of the connection. This local failure is responsible for the plastic phase of the force-displacement curves.

Figure 3.15: Damages observed in the 30° skew angle strengthened connections with bolt under monotonic loading in the positive direction: a) Yielding of the bolt, b) Gap between the two elements c) bending of the washer (Branco, 2008).

Figure 3.16: Damages observed in the 30° skew angle strengthened connections with bolt under monotonic loading in the negative direction: a) Bending of the bolt b) Local compression perpendicular to the grain (Branco, 2008).
3.5.2.3 BINDING STRIP

Connections strengthened with the binding strip were analyzed only in the case of 30° skew angle.

The experimental results show that this strengthening technique improves the connection behavior in terms of resistance, ductility. However, the assembling of the binding strip on the connection, especially in situ, is rather difficult when compared to the already presented techniques. The rigid hollow steel ribbon suits new connections better. Even though, the experimental results obtained can be of practical interest.

Figure 3.17 shows the force-displacement curves obtained for connections strengthened with rigid binding strip, under monotonic loading for 30° skew angle.

The obtained curves indicate a completely nonlinear behavior. However, specimens BS3-1, BS3-2 and BS3-4 revealed limited ductility. The elastic phase of the force-displacement curves are small.

When loaded in the negative direction, firstly the binding strip is tightly fit to the connection and all materials exhibit linear elastic behavior. Next, the elastic part is ceased and the formation of a plastic hinge at the contact rafter/tie beam occurs, see Figure 3.17. In this phase, a slight decrease in the applied force is observed, corresponding to the beginning of the plastification of the wood. Finally, a constant hardening process takes place. A high ductility is characteristic of this phase in consequence of the plastification of wood and yielding of the binding strip.

The resistance of the strengthened connections is considerably high and seems to have no influence of loading direction. When changing the loading direction, the stiffness presents an insignificant variation.

Tables 3.10 and 3.11 present the main results obtained from the monotonic tests on strengthened connections with binding strip in positive and negative load directions.
Figure 3.17: Force-displacement curves of connections strengthened with biding strip, 30° and 60° skew angles under monotonic loading on both directions (Branco, 2008).

Table 3.10: Monotonic tests results of strengthened connections with biding strip in positive load direction.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>(d_e) [mm]</th>
<th>(F_{max}) [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>(F_e/d_e)</th>
<th>(F_e^{50%}/d_e^{50%})</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS3-1</td>
<td>1.4</td>
<td>4.00</td>
<td>17.01</td>
<td>1629</td>
<td>1489</td>
<td>1608</td>
<td></td>
</tr>
<tr>
<td>BS3-2</td>
<td>4.80</td>
<td>15.82</td>
<td>1430</td>
<td>1306</td>
<td>1367</td>
<td>1349</td>
<td></td>
</tr>
<tr>
<td>BS3-3</td>
<td>4.30</td>
<td>18.81</td>
<td>1357</td>
<td>1252</td>
<td>1384</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>4.37</td>
<td>17.81</td>
<td>1472</td>
<td>1349</td>
<td>1453</td>
<td></td>
</tr>
<tr>
<td>CoV</td>
<td></td>
<td>9.25</td>
<td>8.75</td>
<td>9.56</td>
<td>9.21</td>
<td>9.26</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.11: Monotonic tests results of strengthened connections with biding strip in negative load direction.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>(d_e) [mm]</th>
<th>(F_{max}) [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>(F_e/d_e)</th>
<th>(F_e^{50%}/d_e^{50%})</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS3-4</td>
<td>1.4</td>
<td>-7.00</td>
<td>-19.10</td>
<td>1583</td>
<td>1452</td>
<td>1634</td>
<td></td>
</tr>
<tr>
<td>BS3-5</td>
<td>1.4</td>
<td>-5.00</td>
<td>-20.00</td>
<td>1809</td>
<td>1796</td>
<td>1778</td>
<td></td>
</tr>
<tr>
<td>BS3-6</td>
<td>1.4</td>
<td>-7.00</td>
<td>-18.81</td>
<td>1428</td>
<td>1239</td>
<td>1194</td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>-6.33</td>
<td>-19.30</td>
<td>1607</td>
<td>1496</td>
<td>1535</td>
<td></td>
</tr>
<tr>
<td>CoV</td>
<td></td>
<td>18.23</td>
<td>3.21</td>
<td>11.93</td>
<td>18.79</td>
<td>19.82</td>
<td></td>
</tr>
</tbody>
</table>

The strengthened connections with binding strip are strong in terms of resistance, but the damages induced on the timber elements during the tests are important and shown in Figure 3.18.
Under monotonic loading in the positive direction, the tie effect provided by the binding strip is so high that the bottom part of the rafter does not move, and all imposed displacement is absorbed by the top part. Because of that, major bending stresses are induced on the rafter. In the negative loading direction, on the other hand, the damages are essentially located on the tie beam member, at the back part of the connection, where there is a local compression of wood perpendicular to the grain.

### 3.5.2.4 TENSION TIES

Force-displacement curves obtained from the connections strengthened with tension ties, under monotonic loading for 30° and 60° skew angles, show a full non-linear development with high ductility, see Figure 3.19.

All results are consistent and the response is not influenced by either the skew angle value and or the monotonic loading direction. Resistance, stiffness and the development of the force-displacement curves are quite constant.
Variations were observed only when the skew angle is 30° and the strengthened connections are subjected to monotonic loading in the negative direction. In this loading direction, and a 60° skew angle, a significant plastic phase is observed in the force-displacement experimental curves (see Figure 3.19).

Table 3.12 summarizes the main results of the monotonic tests of strengthened connections with tension ties.

Only the 60° skew angle strengthened connections with tension ties showed visible damages. The damages are different depending on the loading direction, see Figure 3.20. When loading in the positive direction, local compression of the front of the knot is observed. In the negative direction, conversely, local compression perpendicular to the grain occurs in the back of the connection.

Table 3.12: Monotonic tests results of strengthened connections with tension ties.

<table>
<thead>
<tr>
<th>Skew angle</th>
<th>Compression stress [MPa]</th>
<th>$d_e$ [mm]</th>
<th>$F_{max}$ [kN]</th>
<th>Stiffness [MN/mm]</th>
<th>Regression</th>
<th>$F_e/d_e$</th>
<th>$F_e^{50%}/d_e^{50%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>1,4</td>
<td>4.91</td>
<td>7.65</td>
<td>556</td>
<td>555</td>
<td>1459</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-4.91</td>
<td>-11.26</td>
<td>610</td>
<td>651</td>
<td>1409</td>
<td></td>
</tr>
<tr>
<td>60°</td>
<td>2,5</td>
<td>9.72</td>
<td>7.91</td>
<td>461</td>
<td>471</td>
<td>465</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>-6.71</td>
<td>-6.08</td>
<td>639</td>
<td>664</td>
<td>838</td>
<td></td>
</tr>
</tbody>
</table>
3.5.2.5 COMPARISON OF STRENGTHENING TECHNIQUES

The comparison of experimental force-displacement curves obtained from the unstrengthened and strengthened connections indicates that strengthening techniques allows for improving the behavior of the originals connections. Figures 3.21 and 3.22 show the comparison of strengthened and unstrengthened connection for skew angles of 30° and 60°, respectively.

a) 30° skew angle (see Figure 3.21):

The strengthened connections yields to an increase in stiffness in the positive loading direction and in the maximum resistance in both loading directions.

The elasto-plastic behavior with limited ductility, which was observed in unstrengthened connections under negative loading direction, is substituted by a full nonlinear behavior exhibiting high ductility in strengthened connections.

Based on the compared strengthening techniques, the tension tie was the less efficient in terms of maximum force and stiffness. The stirrups and binding strip strengthened connections exhibited the same range of maximum force; however, the later presented lower ductility.

The maximum resistance for the strengthened connections with stirrups and internal bolt is achieved at the end of the test. The same is not observed for connections strengthened with binding strip; in particular, the load exhibited a decreasing trend at the end of the test. Therefore, the internal bolt is more efficient than the binding strip in terms of ductility.
The effects of the strengthening on the negative loading direction are the increase of the maximum force and ductility. The gain in terms of stiffness is not significant and for the tension tie method, the stiffness is even reduced. The brittle behavior exhibited by unstrengthened connections disappears in all strengthened specimens. Therefore, the main advantage of strengthening the connections with one of the studied methods is the ductility improvement. Only the binding strip showed limitations in terms of maximum displacement.

b) 60° skew angle (see Figure 3.22):

In connections with 60° skew angle, the main conclusions are kept. The most evident advantage is the improvement of the connection ductility. Particularly, under loading in the negative direction, the behavior of the unstrengthened connections is characterized by a limited ductility. When the connection is strengthened, this behavior is substituted by a full non-linear behavior with considerable ductility.

The strengthening techniques evaluated did not cause changes on stiffness. However, they increased the maximum force in both directions; in particular, under monotonic loading in positive direction.

Considering the strengthening techniques that were studied, the differences in force-displacement curves are greater under loading in the negative direction. In this case, stirrup is the most efficient strengthening method because greater loads are achieved, while the response of the tension ties and the bolt are quite similar.
3.6 CONCLUSIONS

Traditional timber connections, even without any strengthening device, usually have a significant moment-resisting capacity (Branco, 2008). Therefore, they cannot be modeled as perfect hinges or rigid links. They should be considered semi-rigid and friction based joints.

Experimental results show that the moment-resisting capacity is a function of the compressive stress applied in the rafter and the skew angle. Moreover, the height of the rafter, friction angle (Parisi and Piazza, 2000), existence of mortise and tenon and moisture content (Palma and Cruz, 2007) are also relevant factors.

The experimental analysis helped elucidating the mechanical behavior of the connections. The obtained results pointed out important aspects such as force transmission mechanisms, failure modes and practical guidance for efficient strengthening solutions.

Strengthening, usually performed by insertion of metal devices, is indispensable for ensuring adequate joint response (ductility), especially for seismic loading and/or other unpredictable loading conditions.

All analyzed strengthening techniques lead to a significant increase in resistance and ductility improvements. Among the strengthening techniques evaluated, the insertion of one bolt across the joint axis and metal stirrups positioned and bolted to the timber on the two sides of the joint, presented the best results.
CHAPTER 4
Numerical modelling of unstrengthened traditional timber connections

ABSTRACT

This chapter presents an overview on how the numerical model of the connections in study was built. Then, comparisons with experimental results are made to validate and confirm the model. After that a sensitivity study is carried out to analyze the influence of some connection parameters on the behavior of the joint. Finally, results are analyzed.
4.1 INTRODUCTION

The Finite Element Method (FEM) has become one of the most used methods for analyzing physical phenomena in the structural field, solid and fluid mechanics.

Regarding wood, the Finite Element Method (FEM) proved to be a suitable tool to study the material, their mechanical properties, and, for this work, to evaluate the mechanical performance of its connections. Here, FEM is adopted to simulate the structural behavior of unstrengthened traditional timber connections. However, it is important to say that numerical analysis is not intended to substitute the full-scale tests. Their results are complementary.

4.2 NUMERICAL MODELLING

The tests results on full-scale specimens became the fundamental reference for the subsequent step of the research: finite element modelling of unstrengthened connections. Bi-dimensional models were set up using the commercial package TNO Diana v9.4.4.

The geometry and loading of the connections permits the assumption of a state of plane stress for the model, because:

- Geometry is plane, the coordinates of the element nodes are in one flat plane, the \( xy \) plane of the element. In addition it is thin, the thickness \( t \) is small in relation to the dimensions \( b \) in the plane of the element as shown in Figure 4.1;
- Loading \( F \) acts in the plane of the element as shown in Figure 4.1.

![Figure 4.1: Plane stress element (TNO DIANA).](image-url)
It is also important to say that plane stress elements are characterized by the fact that the stress components perpendicular to the elements face are zero ($\sigma_{zz} = 0$).

Experimental results were first used do calibrate the numerical model and then to validate it. Details of the finite element model concerning the material model used, the geometric constraints adopted, the applied loading, and the assumed assumptions are reported in the following sections of this chapter.

### 4.2.1 MATERIAL MODEL

Timber is an anisotropic material, but can be satisfactorily represented as orthotropic (Bodig and Jayne, 1982) in an anatomic cylindrical coordinates system corresponding to the longitudinal, $L$, radial, $R$, and transversal, $T$, directions of the tree trunk.

When the material is extracted from the outer region, curvature may be neglected and the coordinate system may be approximately considered orthogonal. To completely describe the behavior of an orthotropic material, values of three Young’s moduli, $E$, three shear moduli, $G$, and six Poisson’s ratios, $\nu$ are needed. If symmetry is considered, only nine independent parameters need be defined.

For softwood, the following approximate values are acceptable:

\[
E_L : E_R : E_T \approx 20 : 1.6 : 1 \quad (4.1)
\]

\[
G_{LR} : G_{LT} : G_{RT} \approx 10 : 9.4 : 1 \quad (4.2)
\]

\[
E_L : G_{LT} \approx 14 : 1 \quad (4.3)
\]

For the present numerical model the following axisymmetric simplifications were adopted:

\[
E_R = E_T \quad (4.4)
\]

\[
G_{LR} = G_{LT} \quad (4.5)
\]

Therefore, it was used in the numerical model one elastic modulus for the direction parallel to the grain, $E_0$, and a different elastic modulus, $E_{90}$, for the direction perpendicular to the grain. Finally, a shear modulus, $G$, and a Poisson’s ratio $\nu$. 

Compression in the direction normal to the fibers easily brings the wood beyond its elastic limit (Kollmann and Cote, 1984). A relatively simple yield condition that can capture orthotropic strength properties has been proposed by Hill (1947) as an extension of the Von Mises yield condition and it was adopted in this numerical model. DIANA supports the Hill yield condition for ideal plasticity only. The Figure 4.2 shows the considered limit strengths in the direction of the fibers, $\sigma_{xx} = f_{c,0}$, and in the direction perpendicular to the fibers $\sigma_{yy} = f_{c,90}$.

![Figure 4.2: Orthotropic yield strengths (TNO DIANA).](image)

The plastic behavior of wood is directly given by the experimental curve $\sigma \times \epsilon$ of compression perpendicular to the grain (Edlund, 1995). The law adopted in the numerical modelling, ideal plasticity, depends on the compression strength and modulus of elasticity in the direction normal to the grain, $f_{c,90}$ and $E_{90}$, respectively.

Table 4.1 shows the material properties adopted in the numerical model. The values adopted for the shear properties were obtained using the relations proposed by EN 338 (2003) while the others properties were obtained from the material characterization previously reported. Commonly, 0.3 is assumed as the Poisson coefficient for wood.

<table>
<thead>
<tr>
<th>$E_0$ [MPa]</th>
<th>$E_{90}$ [MPa]</th>
<th>$\nu_{12}$</th>
<th>$G_{12}$ [MPa]</th>
<th>$\rho$ [kg/m³]</th>
<th>$f_{c,0}$ [MPa]</th>
<th>$f_{c,90}$ [MPa]</th>
<th>$f_t$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7992</td>
<td>266</td>
<td>0.3</td>
<td>500</td>
<td>460</td>
<td>20</td>
<td>4</td>
<td>2.63</td>
</tr>
</tbody>
</table>
4.2.2 GEOMETRIC CONSTRAINTS, MESH AND LOADING

Figure 4.3 shows how the test set-up was simulated in the numerical model, including loading, boundary conditions and material axes, where $x$ is the fiber direction.

The tie beam of the joint assembly is constrained not allowing any displacement at the base and at left-side border. The rafter is in contact with the tie beam at the notch and has not been otherwise restrained.

The loading reproduce exactly the experimental campaign. The first loading step, consists of an axial compression at top part of the rafter, which is engaged to the contact surfaces at the joint, and the self-weight of all wood elements. Subsequently, a punctual force, perpendicular to the rafter, was applied with a constant increment rate, at the appropriate height of the rafter and is shown in Figure 4.3.

![Figure 4.3: Boundary conditions, loading and material axis adopted in the numerical model.](image)

The element type adopted in the numerical model, named by DIANA as CQ16M, is an eight-node quadrilateral isoparametric plane stress element with a strain thickness of 80 m as shown in Figure 4.4. It is based on quadratic interpolation and Gauss integration. The polynomial for the displacements $u_x$ and $u_y$ can be expressed as following:
The key element of the finite element model (FEM) is on the interaction between the rafter and the tie beam. The overall response of the joint is highly dependent on the behavior of this interaction. This interaction was modelled with an interface element type of 3 nodes between 2 lines in a 2-dimensional configuration. The element is named by DIANA as CL12I and is shown in Figure 4.5.

To simulate the interaction behavior it was attributed to the interface element 2 stiffness’s along the length [Pa/m], one normal to the interface and the other tangential to the interface. The interface material model is a simplification of the Coulomb friction model and can only predict nonlinear elastic behavior. Figure 4.6 shows the material friction model adopted, where $c$ is the cohesion, $t_t$ is the tangential traction and $t_n$ is the normal traction.

$$u_i(\xi, \eta) = a_0 + a_4 \xi + a_2 \eta + a_3 \xi \eta + a_4 \xi^2 + a_5 \eta^2 + a_6 \xi^2 \eta + a_7 \xi \eta^2$$  \hspace{1cm} (4.6)
4.3 NUMERICAL VERSUS EXPERIMENTAL RESULTS

A first set of numerical analysis were performed reproducing the specimens used in the experimental work. Tests results were used to calibrate the numerical models for 30° and 60° skew angles. Results are expressed in terms of force-displacement curves, evaluated at the same location and under the same load history as the tests performed.
Figures 4.8 and 4.9 compares the curves obtained with the numerical model calibrated for the case of 60° and 30° skew angles and for both compression level of the rafter, $\sigma_c=1.4$ MPa and $\sigma_c=2.5$ MPa with the experimental tests results, respectively.

In general, the numerical analysis seems to reproduce satisfactory the joint behavior. Even with some differences, the numerical models show to be adequately fitted to the experimental results.

The numerical models are more accurate under negative rotation. In this direction, the reaction mechanism induces high compressive stresses at the back of the joint (see Figure 4.10) and the final configuration of the resisting mechanism is shown in Figure 4.11. In negative direction, the model is not capable to reach the same yield point. However, the post-elastic behavior showed by numerical results is more stable. The elastic stiffness of the negative rotation show a good agreement considering the heterogeneity obtained in the experimental tests, in particular, under 2.5MPa compression level of the rafter.

![Figure 4.8: Comparison between experimental and numerical force-displacement curves (60° skew angle): a) Positive rotation direction, 1.4 MPa, b) Negative rotation direction, 1.4 MPa, c) Positive rotation direction, 2.5 MPa, d) Negative rotation direction, 2.5 MPa.](image-url)
Figure 4.9: Comparison between experimental and numerical force-displacement curves (30° skew angle): a) Positive rotation direction, 1.4 [MPa], b) Negative rotation direction, 1.4 [MPa], c) Positive rotation direction, 2.5 [MPa], d) Negative rotation direction, 2.5 [MPa]

Figure 4.10: Stresses distribution on negative rotation resisting mechanism.
The numeric model show to be less effective comparing to the tests results obtained for positive rotation, in particular, for 60° skew angle. Nevertheless, it is important to point out that the tests results for both skew angles presented significant variations when submitted to positive rotation. The reaction mechanism is located in the front notch surface (see Figure 4.12). The rotation progressively separates the surfaces at the back of the connection and it rapidly moves toward its ultimate configuration. Finally, only a small contact area remains as Figure 4.13 shows.

The joint behavior under positive rotation is highly dependent of the notch geometry (Branco, 2008). During the assembling of the connection a perfect fabrication of the notch is needed. The carpentry which supplied the specimens did not have CAD-CAM systems and all cutting process, even though performed by machines, it was controlled by humans. This notch geometry dependency explain the variation and the lower adequacy of the numerical results when compared under positive rotation direction.
Figure 4.12: Stresses distribution on positive rotation resisting mechanism.

Figure 4.13: Final configuration of the connection under positive rotation.
Branco (2008) on his work, analyzed the influence of the geometry and concluded that small geometry imperfections, which can easily occur, represents a significant decrease in the rotational capacity of the connections. The resistance capacity is affected due to the change on the resisting mechanism configuration, which under positive rotation, the center point of the rotation moves from the upper edge of the front notch surface to the lower one and can be seen in Figure 4.14.

Figure 4.14: Resisting mechanism change due to geometric imperfection for 60° skew angle under positive rotation (Branco, 2008).

4.4 SENSITIVITY STUDY

Experimental tests cannot always be done, they are expensive and time consuming. Therefore, numerical simulations are normally used to extend the study to see how not studied parameters, such as material properties, geometry, loading, influence the results.

The results of the sensitivity study carried out are reported in the following sections.

4.4.1 FRICTION

Many factors influence the friction coefficient, \( \mu \), between timber surfaces: the degree of roughness, fiber directions, age, temperature and moisture content (McKenzie and Karpovic, 1968). Therefore, its value may change during the life time of the structure. Despite its importance in some particular applications, friction has not been adequately studied.
The friction coefficient was simulated on the numerical models as tangential stiffness attributed to the surface that joins the rafter and the tie beam. The value of 7992 MPa/m was used initially on the models and on this sensitivity study this stiffness was increased until 200 times and decreased until 200 times.

The force-displacement curves are shown in Figures 4.15 and 4.16. During the result analysis, it was seen that the analyzed connections are not sensitive to the change of the tangential stiffness under positive rotation so their curves are not presented. However, and, as indicative of the frictional behavior of the connection in negative direction, tangential stiffness has significant influence on the response of the connections. Numerical results shows that stiffness has slight dependency on the tangential stiffness, but the resistant capacity is considerably affected by this value. It is also seen that the initial stiffness adopted on the model, when it was being calibrated, was considerably high, since no considerable changes on the force-displacement curves are seen beyond this limit.

![Figure 4.15](image1.png)

Figure 4.15: Tangential stiffness influence on the numerical force-displacement curves under negative rotation and a skew angle of 30°: a) 1.4 MPa, b) 2.5 MPa.

![Figure 4.16](image2.png)

Figure 4.16: Tangential stiffness influence on the numerical force-displacement curves under negative rotation and a skew angle of 60°: a) 1.4 MPa, b) 2.5 MPa.
### 4.4.2 COMPRESSION LEVEL OF THE RAFTER

The response of traditional timber joints is directly dependent of the rafter compression level (Branco, 2008). One characteristic of the studied connections is that the yield point, the post elastic behavior and the ultimate resistance are directly related to the level of compression in the rafter.

This influence is more relevant when variations of axial forces of truss elements occurs, due to asymmetric loads, as snow, wind, and seismic forces. Therefore, it is important to study the effect of the axial compression level of the rafter on the behavior of the connections. Figure 4.17 and Figure 4.18 shows numerical results obtained for the case of 30° and 60° skew angle of unstrengthened connections for both rotation directions under the different compression level of the rafter.

**Figure 4.17:** Compression level of the rafter influence on the numerical force-displacement curves for a skew angle of 30°: a) Positive rotation, b) Negative rotation.

**Figure 4.18:** Compression level of the rafter influence on the numerical force-displacement curves for a skew angle of 60°: a) Positive rotation, b) Negative rotation.
4.5 CONCLUSIONS

Numerical analysis have confirmed the tests results: traditional timber connections have a non-symmetric semi-rigid friction based behavior. The rotational response and capacity depend on the joint geometry.

Parameters such as friction, simulated by a tangential stiffness, and axial compression level of the rafter are of primary importance and draw the following conclusions:

- The tangential stiffness has influence in the connection response only under negative rotation. The stiffness of the connection has slight dependency while the resistance can be significantly affected. This influence decreases over a tangential stiffness value of 7992 MPa/m;
- The connection behavior is highly dependent on the compression level of the rafter. The connections stiffness’s are not affected, but a higher axial compression stress of the rafters corresponds to a higher resistance. The influence seems to be linear and is not affected by direction of rotation.

Branco, (2008) extended a similar sensitivity study to more parameters such as, skew angle, notch depth, height of the rafter and wood species and concluded:

- Connection behavior is dependent on the skew angle. The rotational capacity and stiffness of the connection decreases together with the skew angle. This dependency is not linear and, in the particular case of connections response under negative rotation, the skew angle increase results in a more fragile behavior (see Figure 4.19);
- In a precise joint geometry, the notch depth has influence only on the response under negative rotation. If the connection has geometry imperfections, for example, a deviation of 3.00 mm due to the cutting process of the notch, the notch depth also influence the connection behavior in the positive rotation direction. In both cases, the notch depth has influence only on the resistant capacity (see Figure 20);
- The height of the rafter cross-section has significant influence on the connection behavior. Higher cross-sections increases the resistance and stiffness of the joint. This increase is less seen under positive rotation (see Figure 4.21);
- Wood species have minor influence on the connection behavior (see Figure 4.22).
Figure 4.19: Effect of the skew angle on the numerical force-displacement curves: a) 1.4 MPa under negative rotation b) 2.5 MPa under negative rotation c) 1.4 MPa under positive rotation d) 2.5 MPa under positive rotation (Branco, 2008).

Figure 4.20: Effect of the skew angle on the numerical force-displacement curves: a) 1.4 MPa under negative rotation, 30° b) 2.5 MPa under negative rotation, 30° c) 1.4 MPa under positive rotation, 60° d) 2.5 MPa under positive rotation, 60° (Branco, 2008).
Figure 4.21: Effect of the rafter height’s on the numerical force-displacement curves (30° skew angle): a) 1.4 MPa under negative rotation b) 2.5 MPa under negative rotation c) 1.4 MPa under positive rotation d) 2.5 MPa under positive rotation (Branco, 2008).

Figure 4.22: Material properties effects on the numerical force-displacement curves (60° skew angle): a) 1.4 MPa under negative rotation b) 2.5 MPa under negative rotation c) 1.4 MPa under positive rotation d) 2.5 MPa under positive rotation (Branco, 2008).
Numerical modeling of traditional timber connections
CHAPTER 5
Numerical modelling of strengthened traditional timber connections

ABSTRACT
This chapter presents and discuss the numerical model of traditional timber connections strengthened with binding strip. The model has been calibrated based on the experimental results obtained for connections with 30° skew angle and after, numerically, the behavior of connections with 60° skew angle is simulated and studied.
5.1 INTRODUCTION

Despite of being very useful, the numerical modelling of traditional timber connections is not common. On the following sections, it will be presented an extension of the numerical model shown on the previous chapter.

Since the binding strip was the only strengthening technique that was not analyzed for a skew angle of 60°, it was decided to build a numerical model of this type of joint. Based on the experimental results obtained for the connections strengthened with metal binding strips using specimens with a skew angle of 30°, a numeric model was calibrated. Then, the same strengthened connection but now with a skew angle of 60° was simulated and studied. However, it is important to say that the objective of this numerical model is not to substitute previous tests, but is used to complement the research with more information.

5.2 NUMERICAL MODELLING

The tests results of the strengthened connections with binding strip were of great importance for the numerical modelling as they were used for the calibration of the model. Bi-dimensional models were, again, set up using the commercial package TNO Diana v9.4.4.

Some characteristics of the previous models were maintained:

- Plane stress assumption;
- Material model for the interface;
- Loading routine;
- Element type for the surface and for the beam elements;
- Meshing.
5.2.1 STRENGTHENING AND GEOMETRIC CONSTRAINTS

For the new model, it was needed to change the wood material model. The previous one, proposed by Hill, does not describe different yield conditions in tension and compression. And through the experimental results with the binding strip strengthening it was seen that the bottom part of the after is in tension (Branco, 2008). The solution was to adopt a material model proposed by Hoffmann (1967) that describes the orthotropic behavior of wood under tension and compression. The values for the yield stress adopted in the model for the tension in the grain direction and perpendicular to it was 15 MPa and 0.6 MPa, respectively.

The strengthening, materialized by a metal binding strip, was implemented on the model as a nonlinear elastic support defined by an axial stiffness. This nonlinearity aimed to predict the yielding of steel, because otherwise, after the plastification of wood, the model would represent correctly the behavior of the connection if the steel never yields. In addition, the sliding of the strip against the rafter was neglected. Figure 5.1 shows the geometric constraints assumed for the model.

![Figure 5.1: Boundary conditions, loading and material axis adopted in the numerical model.](image)

5.2.2 NUMERICAL VERSUS EXPERIMENTAL RESULTS

Differently from the previous models, the strengthened one was only calibrated and validated with the results from the test performed with a 30° skew angle and under a compression level of
the rafter of 1.4 MPa. Results are expressed in terms of force-displacement curves, evaluated at the same location and under same load routine as the tests were analyzed.

Figure 5.2 compares the curves obtained from the numerical model calibrated for the case of 30° skew angle for compression level of the rafter, $\sigma_c=1.4$ MPa with the experimental tests results.

The numerical model seems to reproduce satisfactory the strengthened joint behavior. The model shows higher convergence with experimental curves when comparing it with the unstrengthened models when loaded on the positive direction. Results show high accuracy on both loading directions.

Figure 5.2: Force-displacement curves for connection strengthened with binding strip, 1.4 MPa compression level of the rafter and 60° skew angle: a) Negative loading direction, b) Positive loading direction

The model is able to reproduce the resisting mechanism in the negative direction where the back part of the connection is locally compressed perpendicularly to the grain. No relevant compression is seen in the contact of the binding strip with the rafter. Figure 5.3 show the resisting mechanism of the connection when loaded in the negative direction.

Regarding the loading capacity and stiffness, the model presents the same symmetry showed by the experimental results.

In the positive direction, the resisting mechanism is also accurately reproduced by the numerical model. The tie effect provided by the binding strip is so high, that the bottom part of the rafter does not move and all displacement is absorbed by the top part of the rafter. In consequence of this bending, tension stresses arises on the on the bottom left part of the rafter. Compression stresses arises where the rafter is in contact with the binding strip. Figure 5.4 shows the resisting mechanism when the connection is loaded in the positive direction.
The 2.5 MPa compression level of the rafter was also simulated and its force-displacement curves are shown in Figure 5.5. As expected the same behavior, regarding the compression level of the rafter, observed on the unstrengthened connection happens with the strengthened connection with binding strip.

![Figure 5.3: Resisting mechanism of the connection when loaded in the negative direction](image)

![Figure 5.4: Resisting mechanism of the connection when loaded in the positive direction](image)
5.2.3 NUMERICAL MODELLING OF 60° SKEW ANGLE CONNECTION

The main reason for formulating a numerical model of the strengthened connection with binding strip was to extend the study of this strengthening technique to 60° skew angle connections, since the experimental campaign performed did not include this configuration.

The strengthening, metal binding strip, was simulated on the 60° skew angle connection adopting the same position and geometry tested for the 30° skew angle joints.

Figure 5.6 shows the monotonic curves of force-displacement on both directions of a 60° skew angle notched connection strengthened with a metal binding strip under compression level of the rafter of 1.4 and 2.5 MPa.

When the connection is loaded in the negative direction, the same resisting mechanism described before occurs. The back part of the connection is compressed perpendicularly to the grain followed by plastification of the wood in this area.

This plastification of the material is seen on Figure 5.6 where the stiffness of the curve changes. The binding strip also induces bending stresses on the rafter so its left part is under tension stresses in the direction of the fibers. Figure 5.7 shows the resisting mechanism of strengthened connection with binding strip under a compression level of 1.4 MPa and skew angle of 60° loaded on the negative direction.°
In the positive direction the same described resisting mechanism happens, but with one important difference: the full plastification of the wood. The tight effect provided by the metal binding strip is so intense that high tension stress at the bottom of the rafter arises until the full formation of a plastic hinge. Differently from the 30° strengthened connection it can be seen compression perpendicular to the grain in the contact area between the rafter and the binding strip. Figure 5.8 shows the resisting mechanism of the strengthened connection with binding strip under a compression level of 1.4 MPa and skew angle of 60° under negative rotation.

Figure 5.6: Force-displacement curves for connection strengthened with binding strip, 1.4 and 2.5 MPa compression level of the rafter and 60° skew angle: a) Negative loading direction, b) Positive loading direction.

Figure 5.7: Resisting mechanism of the strengthened connection with 60° skew angle and 1.4 MPa compression level of the rafter when loaded in the negative direction.
When comparing with the unstrengthened connection the same elastic stiffness was obtained, but with different yield point. Regarding the strength capacity, the strengthened 60° degree joint showed a higher value. The strengthening also improved the connection in terms of ductility.

When comparing with the 30° skew angle strengthened connection it can be seen that the strengthening method is less effective. This can be seen particularly on the positive loading direction because the lower force capacity achieved by the connection. Figure 5.9 shows the comparison of the force-displacement curves for 30° and 60° skew angles under 1.4 MPa compression level of the rafter.

### 5.3 CONCLUSION

The numerical analysis confirmed the tests results for 30° strengthened traditional timber connections with binding strip under both loading directions:

- Force-displacement curves show a full nonlinear behavior;
The elastic phase of the force displacement curve is small, particularly on the positive loading direction;
- High ductility is seen in the curves, due to plastification of the wood, for both loading directions;
- Stiffness is not affected by the loading direction;
- The connection present high resistance capacity and the binding strip induces damages on the connection (tie effect).

The numerical model was also predicted the same resisting mechanisms for the connection with 60° skew angle, 1.4 MPa compression level of the rafter and strengthened with a binding strip. The finite element model revealed an important damage: the complete formation of a plastic hinge due to the tie effect under positive loading direction.

Full characterization of the steel used is also of great use and can be used to better adjust and calibrate the numerical model.
CHAPTER 6
Conclusions and recommendations

ABSTRACT

This chapter summarizes all the conclusions previously presented. Finally, recommendations are made for future work.
6.1 CONCLUSIONS

The thesis aimed to first formulate a numerical model for reproducing the behavior of traditional timber connections. Later, on the same model, adaptations were made to predict the behavior of the same type of connection, but this time, strengthened with a metal device, the binding strip.

The work is based on two important experimental campaigns performed in the University of Minho by Branco (2008). First, a full characterization of the wood used on the full-scale connections specimens was done. Then, an extensive experimental evaluation of the monotonic behavior of traditional timber connections, through full-scale specimens was performed. Strengthened specimens with traditional metal strengthening techniques have been also evaluated.

The results of the first tests, material characterization, were used on the numerical model to describe material orthotropic properties. Then the full-scale specimens tests were used to calibrate and validate the numerical models.

In Chapter 3 a depth analysis is presented about the series of tests on monotonic unstrengthened specimens performed in order to characterize the original behavior of the studied traditional timber connections. Subsequently, connections strengthened with metal devices were tested under monotonic loading were also studied. The tests results obtained allow to draw the following conclusions:

- Traditional timber connections, even without any strengthening device, usually have a significant moment-resisting capacity;
- The moment-resisting capacity is function of the compression stress applied in the rafter and skew angle of the connection. Additionally, the height of the rafter, the friction coefficient (Branco, 2008), the existence of mortise and tenon and the moisture content (Palma and Cruz, 2007) are also important;
- Strengthening, usually performed by insertion of metal devices, is indispensable for ensuring adequate joint response, in particular, for seismic loading, or in other adverse and unpredictable loading conditions;
- All tested strengthening techniques resulted in a significant increase of resistance and significant improvement of the ductility;
- Between the strengthening techniques evaluated, the insertion of one bolt across the joint axis and metal stirrups positioned at the two sides of the joint, bolted to the timber elements, have given better results. The proper design of all the strengthening metal devices are very important;
Full characterization of the steel used on all strengthening must be done to aid the test results and help improving the numerical models.

Since tests are time consuming and, economically, they are expensive, a parametric study was done using the numerical model calibrated based on the tests results reported and discussed in Chapter 3. With this study, reported in Chapter 4, the influence of some parameters not assessed in the experimental campaign was numerically simulated. Based on those simulations, it was possible to conclude that:

- The tangential stiffness, used to simulate friction, has influence in the connection response only under negative rotation. The stiffness of the connection has slight dependency while the resistance can be significantly affected. This influence decreases over a tangential stiffness value of 7992 MPa/m;
- The connection behavior is highly dependent on the compression level of the rafter. The connections stiffness’s are not affected, but a higher axial compression stress of the rafters corresponds to a higher resistance. The influence seems to be linear and is not affected by direction of rotation.

Branco (2008) extended a similar sensitivity study to more parameters such as, skew angle, notch depth, height of the rafter and wood species and concluded:

- The connection behavior is dependent on the skew angle. The rotational capacity and stiffness of the connection decreases together with the skew angle. This dependency is not linear and, in the particular case of connections response under negative rotation, the skew angle increase results in a more fragile behavior;
- In a precise joint geometry, the notch depth has influence only on the response under negative rotation. If the connection has geometry imperfections, for example, a deviation of 3.00 mm due to the cutting process of the notch, the notch depth also influence the connection behavior in the positive rotation direction. In both cases, the notch depth has influence only on the resistant capacity;
- The height of the rafter cross-section has significant influence on the connection behavior. Higher cross-sections increases the resistance and stiffness of the joint. This increase is less seen under positive rotation;
- Wood species have minor influence on the connection behavior.

After validation of the unstrengthened numerical model, the model was modified to predict the behavior of a traditional timber connection strengthened with a binding strip. The new model was calibrated and validated with tests results performed only for connections with 30° skew angle and compression level of the rafter of 1.4 MPa. The model was used to extend the study of this strengthening technique to connections with a 60° skew angle. However, it is important to say that
the model does not substitute the need for further experimental tests on this matter. Based on the numeric simulations, it was concluded that:

- Force-displacement curves show a full nonlinear behavior;
- The elastic phase of the force displacement curve is small, particularly on the positive loading direction;
- High ductility is seen in the curves, due to plastification of the wood, for both loading directions;
- Stiffness is not affected by the loading direction;
- The connection present high resistance capacity and the binding strip induces damages on the connection (tie effect);
- The position of the binding strip has high influence on the resisting mechanism on the positive rotation direction. Therefore, it influences on the resisting force of the connection on this loading direction.

### 6.2 RECOMENDATIONS

This thesis is a result of an international research program involving University of Minho (Portugal) and Czech Technical University in Prague (CTU). The work was carried out in both Universities, all the required courses were done in Czech while the thesis was written in Portugal.

The work present in this thesis is a small contribution for the structural analysis of traditional timber connections and trusses. Further research is necessary to improve the effectiveness of the models behavior proposed for traditional timber connections.

The recommendations are listed as follows:

- Others geometries, in particular the double notched joints, and the effect of the presence of mortise and tenon must be analyzed;
- A parametric study, similar to the one presented in Chapter 4, should be extended to the strengthened numerical model in order to understand the influence of some parameters on the behavior of such timber connections;
- The numerical models should be improved to be able to simulate cyclic loading;
- Interface elements should be replaced for contact elements and comparisons on the numerical model should be done;
- Material yield criterion with hardening should be used since the ones used assumed ideal plasticity only;
• The experimental campaign of the binding strip strengthening technique should be extended to 2.5 MPa compression level of the rafter and skew angle of 60°;
• Since the strengthening plays an important role on the connection behavior, a full characterization of the steel used is needed.
BIBLIOGRAPHY


Palma, P. and Cruz, H. (2007). Mechanical behavior of traditional timber carpentry joints in service conditions - results of monotonic tests. In From material to Structure - Mechanical behavior and failures of the timber structures XVI International Symposium, Venice, Italy. ICOMOS IWC.

APPENDIX A

DIANA OUTPUT FILE: “UNSTRENGTHEND CONNECTION, 30° SKEW ANGLE, NEGATIVE ROTATION DIRECTION UNDER 1.4 MPa COMPRESSION LEVEL OF THE RAFTER”.

FEMGEN MODEL : 30UN_V00_INT
ANALYSIS TYPE : Structural 2D

'UNITS'

LENGTH M
TIME SEC
TEMPER CELSIU
FORCE N

'COORDINATES' DI=2

1 8.000000E-02 2.486000E-01
2 6.000000E-02 2.139500E-01
3 4.000000E-02 1.793000E-01
4 2.000000E-02 1.446500E-01
5 0.000000E+00 1.100000E-01

(…)

3080 2.275000E-01 4.590500E-01
3081  2.225000E-01  4.603944E-01
3082  2.175000E-01  4.617389E-01
3083  2.125000E-01  4.630833E-01
3084  2.075000E-01  4.644278E-01

'ELEMENTS'

CONNECTIVITY

1 CQ16M  1 51 2 56 7 60 6 55
2 CQ16M  2 52 3 57 8 61 7 56
3 CQ16M  3 53 4 58 9 62 8 57
4 CQ16M  4 54 5 59 10 63 9 58
5 CQ16M  6 60 7 65 12 69 11 64

(…)

952 CQ16M  2708 3061 2709 3071 2719 3080 2718 3070
953 CQ16M  2709 3062 2710 3072 2720 3081 2719 3071
954 CQ16M  2710 3063 2711 3073 2721 3082 2720 3072
955 CQ16M  2711 3064 2712 3074 2722 3083 2721 3073
956 CQ16M  2712 3065 2713 3075 2723 3084 2722 3074
957 CL12I  681 1366 1176 2714 3066 2704
958 CL12I  1176 1365 1175 2704 3047 2694
959 CL12I  1175 1364 1174 2694 3028 2684
960 CL12I  1174 1363 1173 2684 3009 2674
961 CL12I  1173 1362 1172 2674 2990 2664

(…)

980 CL12I  636 771 645 2719 3080 2718
981 CL12I  645 789 654 2718 3079 2717
982 CL12I  654 807 663 2717 3078 2716
983 CL12I 663 825 672 2716 3077 2715
984 CL12I 672 843 681 2715 3076 2714

MATERIALS
/ 669-956 / 2
/ 1-668 / 3
/ 957-975 / 4
/ 976-984 / 5

GEOMETRY
/ 957-984 / 2
/ 1-668 / 3
/ 669-956 / 4

'MATERIALS'

2 YOUNG     7.992000E+09    2.660000E+08
POISON      3.000000E-01
SHRMOD      5.000000E+08
DENSIT      4.600000E+02
YIELD       HILL
YLDOPT MATAXI
YLDSIG      2.000000E+07    4.000000E+06    0.000000E+00
            2.630000E+06

3 YOUNG     7.992000E+09    2.660000E+08
POISON      3.000000E-01
SHRMOD      5.000000E+08
DENSIT      4.600000E+02
YIELD       HILL
YLDOPT MATAXI
Numerical modeling of traditional timber connections

YLDSIG  2.000000E+07  4.000000E+06  0.000000E+00  2.630000E+06
4 DSTIF  4.002000E+09  7.992000E+09
FRIC
FRCVAL  2.320000E+05  5.700000E-01  0.000000E+00
5 DSTIF  4.002000E+09  7.992000E+09
FRIC
FRCVAL  2.320000E+05  5.700000E-01  0.000000E+00
'GEOMETRY'
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2 THICK  8.000000E-02
CONFIG MEMBRA
3 THICK  8.000000E-02
XAXIS  0.500008E+00  0.866021E+00  0.000000E+00
4 THICK  8.000000E-02
XAXIS  0.000000E+00  0.100000E+01  0.000000E+00
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3 D3 / 176-180 258-262 266-272 276 281 285-289 /
4 D4 / 1 6 11 16 21 26 31 36 41 46 55 64 73 82 91 100 109 118
   127 290-295 300 305 310 315 320 325 330 335-345 355 365
Numerical modeling of traditional timber connections

375 385 395 405 415 425 430-434 /
5 D5 / 46 136 141 146 151 156 161 166 171 176 181 190 199 208
217 226 235 244 253 335-339 430-435 440 445 450 455 460
465 470 475-480 490 500 510 520 530 540 550 560 565-569 /
6 D6 / 176 262 267 272 281 475-479 565-570 575-580 590 595-599 /
7 D7 / 290 295 300 305 310 315 320 325 330 335 345 355 365 375
385 395 405 415 425 600-609 618 627 636 645 654 663 672
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Numerical modeling of traditional timber connections

627 636 645 654 663 672 681-699 717 735 753 771 789 807
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25 TBSURF / 1-668 /

NODES
26 TBSURF_N / 1-2137 /

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257 266-271 276 285-289 575-579 595-599 1113-1121 1149-1157 1728-1746
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257 266-271 276 285-289 575-579 595-599 1113-1121 1149-1157 1728-1746
1804-1822 2003-2008 2027-2032 2123-2125 2135-2137 / TR 5

'LOADS'

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FORCE -0.113636E+06
DIRECT 2
/ 670 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2
/ 671 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 672 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 673 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 674 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 675 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 676 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT 2

/ 677 /
EDGE ETA1
FORCE -0.113636E+06
DIRECT  2

CASE 2

NODAL

2158 FORCE 1  0.100000E+05

'DIRECTIONS'

 1  1.000000E+00  0.000000E+00  0.000000E+00
 2  0.000000E+00  1.000000E+00  0.000000E+00
 3  0.000000E+00  0.000000E+00  1.000000E+00
 4  5.000077E-01  8.660210E-01  0.000000E+00
 5 -8.660210E-01  5.000077E-01  0.000000E+00

'END'