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Douaa Adil Osman Elbashir

Assesment and Reinforcement of existing dowel-type joints



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OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

I hereby declare that the MSc Consortium responsible for the Advanced Masters in Structural Analysis of Monuments and Historical Constructions is allowed to store and make available electronically the present MSc Dissertation.

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This work is dedicated to Adil and Suaad, my dear parents, the reason behind every step forward towards my dreams...

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ABSTRACT

Joints in timber structures have an evident effect on the behaviour of the structure in general. The failure mechanism of timber joints with mechanical fasteners among other things depends on the geometry of the joint and the type of fastener. Substantially, under compressive stresses, timber as a material may be loaded far over its elastic limit. For the case of dowel-type joints with mechanical fasteners, the steel used is able to deform in a distinctly plastic manner. Even though, not in all cases timber connections deform plastically before failure resulting in brittle failure modes. Avoiding the causes for brittle failure modes, specifically high tensile perpendicular to the grain and shear stresses, leads to distinctly plastic failure modes of connections with mechanical fasteners. Joints with dowel-type fasteners are considered potentially ductile due to the interaction between the highly nonlinear behaviour of the wood under embedding stresses and the bending behaviour of the steel fasteners. The challenge is to avoid the brittle failure mechanisms by either avoiding perpendicular to the grain tensile stresses or by using reinforcement techniques. These stresses are caused by many reasons such as the wedge effect of the mechanical fasteners or as an amplification of restrained shrinkage. Both effects can be counteracted by reinforcing the connection area. Thus, the main aim of reinforcement is to increase the embedment strength of the timber elements so that it can resist these effects.

Regarding dowel-type connections in general, the mechanical fasteners deform under load resulting in their embedment into the surrounding timber fibres. This deformation is assumed a rigid-plastic deformation according to Johansen K.W, 1949, which is developed based on the European Yield Model (YEM), both approaches are discussed thoroughly within the scope of this work. However, dowel-type joints reinforced with self-tapping screws are more protected against brittle failure modes, as they tend to increase the embedment strength, consequently increasing the load-carrying capacity with the possibility of reducing the end-distance. Since embedment is a nonlinear process, the stiffness of the connections then depends on the nature of the applied load. Thus, for this research project, the specimens will be tested under monotonic and cyclic loading in order to assess the behaviour of the joints in both cases.

This research addresses the assessment of timber-to-timber dowel-type joints in double shear, with reinforcement using FRP, specifically fiberglass (GFRP), and self-tapping screws (STS). For the reinforcement with GFRP, a previous study, (Matos, 2011), had addressed the failure of this type of joints, with and without reinforcement, under both monotonic and cyclic loading. Subsequently, the results obtained in this study, are analysed within the scope of this work for conducting a comparison of reinforcement techniques and discussed accordingly.

Regarding reinforcement with self-tapping screws, an experimental campaign was carried out for the same wood species used in the former study mentioned above, following the same testing procedures, monotonic (EN 26891) and cyclic (EN 12512), for the purpose of comparison and validation.

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RESUMO

As ligações estruturais de madeira têm um efeito óbvio no comportamento global da estrutura. O modo de rotura destas ligações que usam ligadores metálicos depende, entre outros parâmetros, da geometria da ligação e do tipo de ligador. Sob compressão, a madeira apresenta comportamento pós elástico. Também o ligador metálico é capaz de apresentar deformação plástica. No entanto, nem todas as ligações apresentam deformações plásticas ocorrendo roturas de natureza frágil, sem plasticidade. Deste modo, prevenir modos de rotura frágeis, que envolvam tensões de tração perpendicular às fibras e de corte, promove um comportamento plástico das ligações.

As ligações do tipo cavilha são consideradas potencialmente dúcteis dada a interação entre o comportamento não linear da madeira sob tensões de esmagamento localizado e a flexão do ligador metálico. É assim crucial evitar os mecanismos de rotura frágeis reduzindo as tensões de tração perpendiculares às fibras ou reforçando as ligações. As tensões de tração perpendicular às fibras resultam de vários fatores, como são exemplos o efeito grupo dos ligadores e as tensões internas induzidas por variações do teor de água da madeira. Ambos estes efeitos podem ser reduzidos recorrendo a técnicas de reforço das ligações. Normalmente, o principal objetivo do reforço é o aumento da capacidade da ligação ao esmagamento localizado de forma a aumentar a sua resistência a estes dois efeitos.

No que se refere às ligações tipo cavilha em geral, o ligador metálico deforma-se sob carregamento provocando o esmagamento das fibras da madeira. Esta deformação é assumida como rígido-plástica de acordo com Johansen 1949, teoria esta que está na base do European Yield Model (EYM) e que será analisada ao longo deste trabalho. Contudo, ligações tipo cavilha reforçadas com parafusos autoperfurantes apresentam uma maior resistência a modos de rotura frágeis, uma vez que tendem a aumentar a capacidade ao esmagamento localizado da madeira e assim aumentam a capacidade resistente da ligação, permitindo distâncias mínimas, entre ligadores e entre estes e os topos e lados dos elementos ligados, inferiores. Uma vez que o esmagamento localizado é um processo não-linear, a rigidez da ligação depende do tipo de carga aplicada. Deste modo, neste trabalho, as ligações estudadas foram submetidas a ensaios monotónicos e cíclicos.

Este trabalho visa análise de ligações de madeira do tipo cavilha em corte duplo, reforçadas com FRP, nomeadamente, fibras de vidro (GFRP), e parafusos autoperfurantes (STS). No caso do reforço com GFRP, uma dissertação anterior, Matos (2011), analisou os modos de rotura destas ligações com e sem reforço, sob carregamento monotónico e cíclico. Os resultados obtidos são alvo de uma nova análise servindo ainda como comparação entre as duas técnicas de reforço consideradas.

O reforço com parafusos autoperfurantes, foi estudado tendo por base uma vasta campanha experimental que foi conduzida adotando as mesmas condições, incluindo as duas espécies de madeira, geometria das ligações, setup, procedimentos de ensaio, monotónicos (EN 26891) e cíclicos (EN 12512) etc., da dissertação que analisou o reforço com GFRP, para efeitos de comparação.

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تقييم وتعزيز المفاصل الخشبية المعززة باللاوتاد الفولاذية

نبذة مختصرة

المفاصل في الهياكل الخشبية لها تأثير واضح على سلوك الهيكل بشكل عام. تعتمد آلية فشل مفاصل الأخشاب مع السحابات الميكانيكية من بين أشياء أخرى على هندسة المفصل ونوع السحابة. بشكل كبير ، تحت الضغوطات الضاغطة ، يمكن تحميل الأخشاب كمواد بعيداً عن الحد المرن. في حالة الوصلات من نوع وتد مع السحابات الميكانيكية ، فإن الفولاذ المستخدم قادر على التشوه بطريقة بلاستيكية متميزة. على الرغم من ذلك ، ليس في جميع الحالات تشوه اتصالات الأخشاب قبل الفشل مما أدى إلى أوضاع الفشل الهشة. تجنب أسباب أوضاع الفشل الهشة ، وتحديد الشد العالي بشكل عمودي على ضغوط الحبوب والقص ، يؤدي إلى أوضاع عيوب بلاستيكية مميزة للتوصيلات مع السحابات الميكانيكية. تعتبر الوصلات المزودة بسحابات من نوع المسامير قابلة للامتداد نظراً للتفاعل بين السلوك غير الخطي للخشب تحت ضغوط التضمين وسلوك الانحناء للمثبتات الفولاذية. ويتمثل التحدي في تجنب آليات الفشل الهشة إما عن طريق تجنب عمودي على الإجهاد الشد الحبوب أو باستخدام تقنيات التعزيز. هذه الضغوط ناتجة عن العديد من الأسباب مثل تأثير إسفين السحابات الميكانيكية أو تضخيم الانكماش ضبط النفس. يمكن مواجهة كلا التأثيرين عن طريق تعزيز منطقة الاتصال. وبالتالي ، فإن الهدف الرئيسي من التعزيز هو زيادة قوة التضمين لعناصر الأخشاب بحيث يمكنها مقاومة هذه التأثيرات.

فيما يتعلق بالتوصيلات من نوع وتد بشكل عام ، فإن المشابك الميكانيكية تشوه تحت الحمل مما يؤدي إلى تضمينها في ألياف الأخشاب المحيطة. يفترض هذا التشوه تشوهاً صلباً وفقاً لـ Johansen K.W ، 1949 ، والذي تم تطويره استناداً إلى نموذج العائد الأوروبي (YEM) ، حيث تتم مناقشة كلا النهجين بدقة في نطاق هذا العمل. ومع ذلك ، فإن الوصلات من نوع وتد قد عززت مسامير التنصت الذاتي أكثر حماية ضد أوضاع الفشل الهشة ، لأنها تميل إلى زيادة قوة التضمين ، وبالتالي زيادة قدرة الحمل مع إمكانية تقليل مسافة النهاية. نظراً لأن التضمين عملية غير خطية ، فإن صلابة الاتصالات تعتمد بعد ذلك على طبيعة التحميل المطبق. وبالتالي ، بالنسبة لهذا المشروع البحثي ، سيتم اختبار العينات تحت التحميل الأحادي والدوري من أجل تقييم سلوك المفاصل في كلتا الحالتين.

يتناول هذا البحث تقييم المفاصل من نوع الأخشاب إلى الأخشاب في القص المزدوج ، مع التعزيز باستخدام FRP ، وتحديد الألياف الزجاجية (GFRP) ، ومسامير التنصت الذاتي (STS) من أجل التعزيز باستخدام GFRP ، تناولت دراسة سابقة (Matos 2011) ، فشل هذا النوع من المفاصل ، مع وبدون تعزيز ، تحت التحميل الرتيب والدوري. بعد ذلك ، يتم تحليل النتائج التي تم الحصول عليها في هذه الدراسة ، في نطاق هذا العمل لإجراء مقارنة بين تقنيات التعزيز ومناقشتها وفقاً لذلك.

فيما يتعلق بالتعزيز ببراعي التنصت الذاتية ، تم إجراء حملة تجريبية لنفس الأنواع الخشبية المستخدمة في الدراسة السابقة المذكورة أعلاه ، باتباع نفس إجراءات الاختبار ، رتبة (EN 26891) ودورة دورية (EN 12512) ، لغرض المقارنة والتحقق من الصحة.

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CHAPTER 1

1. INTRODUCTION

1.1 WHY TIMBER?

Wood as a building material, had been used for centuries, mainly incorporated in roofs and later on, flooring systems and columns, yet it reflects a promising future with many advantages that can be adopted for the needs of generations to come with no threat to the environment. Nowadays its use has been extended to build houses and shelters.

As an anisotropic hygroscopic material, its behaviour is highly dependent on its exposure conditions, especially if exposed to water, either direct or indirect, and humidity, where the factors that influence its performance on a long term are governing in most cases. Nevertheless, despite of its complex properties and chemical composition, which are highly dependent on the rates/direction of growth, defects and natural durability, which all circle back to the considerably wide range of species, mankind have successfully harnessed these unique characteristics to build seemingly unlimited variety of structures. The timber supply chain has responded to nature's variability by providing controlled supplies from managed forests, conforming to country-specific standards and requirements, with the possibility to fabricate and produce special engineered products, defying the need to use only solid timber that could take years for trees to provide the quantity needed for structural purposes. This could lead to a number of elements being introduced to the structure while relatively green without completely drying, which can cause later on problems of tilting that can be seen for example in some historical constructions.

Being a versatile material that is a product of a natural resource, prospered in many advantages over other building materials, such as availability, that made it economically feasible. One of the biggest advantages that is in fact a perfect example of an environmentally sustainable product with the possibility of complete recycling due to the fact that it carries the lowest carbon footprint compared to any other building material, which makes it biodegradable and renewable. Over and above, in terms of production, no high-energy fossil fuels are required and it's highly machine-able so that it can be fabricated into limitless shapes and sizes to fit various architectural and structural purposes.

1.2 OBJECTIVES

The areas of focus for this dissertation is the reinforcement of timber-to-timber dowel-type connections in double shear. One of the biggest challenges in designing of timber structures is the design of connections. Various reinforcing techniques are available when dealing with timber-to-timber dowel-type connections. The main goal of providing reinforcement to dowel-type joints is to increase the capacity and the ductility of the connection for a better behaviour under seismic loading, in addition to preventing brittle failures. This leads to an increase in the connection resistance to shear and tensile forces perpendicular to the grain, as the capacity of the connection in this case is governed by the high tensile and shear capacity of the reinforcement instead of the low resistance

of wood to the tensile and compression perpendicular to the grain in general. Ductility is a crucial parameter if earthquakes are a potential threat, as the seismic response of the structure depends on the ductility of the joints. Nevertheless, by providing reinforcement, the load-carrying capacity of the connection increases, which leads to increasing the embedding strength of the timber element, subsequently preventing the dowel fasteners from causing splitting.

Reinforcing techniques include but not limited to: Glued joints, which can be glued-on wood-based panels on both sides of the shear plane in timber-to-timber connections, or Fibre Reinforced Polymers (FRP). Glued joints in general are not frequently used due to many reasons such as, the need for highly controlled environmental conditions for production, which makes it quite feasible as a solution, as well as the lack of rules that governs the design, which added more to the challenges associated with the use of these materials specially when considered for retrofitting existing structures. There is also the use of punched metal plate fasteners, nail plates and Self-Tapping Screws (STS) as common reinforcement techniques for this type of connections.

Nevertheless, the main areas of focus on this work are reinforcement of dowel-type joints with Self-Tapping Screws (STS), and evaluate their usage in terms of reinforcement as well as repairing. Then compare the results opposed to reinforcing with Fibre Reinforced Polymers (FRP), specifically reinforcement with fibreglass (GFRP). The behaviour of the joints is compared under both monotonic and cyclic loading. The testing procedure will be carried out according to the following standards: EN 26891 and EN 12512, for monotonic and cyclic tests, respectively.

Since both techniques are yet to be included in the European standards, Eurocode 5 ^[3], the lack of specific rules that governs the design has presented more difficulties to their proper adaptation. However, as of recently, efforts have been made to propose design models in order for structural designers to follow upon reinforcing dowel-type joints with self-tapping screws and FRP. Thus, another objective of this dissertation is to discuss the approaches under discussion in the new version of EC5 ^[3] for reinforcement of dowel-type joints with the above mentioned techniques, and validating the proposed design models with the results obtained from the experimental campaign.

1.3 SCOPE AND METHODOLOGY

As mentioned earlier, this research addresses the assessment and reinforcement of timber-to-timber dowel-type joints in double shear with two different approaches.

- I. Regarding the reinforcement with GFRP, the work on this thesis is set to be a continuation of a former master thesis ^[1], which has been focused on this reinforcement technique. The study presented an experimental campaign addressing the reinforcement of dowel-type joints with fibreglass (GFRP), performing monotonic and cyclic tests on a number of specimen. Therefore, a full take on the results obtained in this previous work, analysis and discussion of the outcomes are carried out within the scope of this dissertation, covering three aspects:

- Analysis and validation of tests results in terms of reinforcement efficiency and its contribution to the overall capacity of the connections under focus, and of the connections' behaviour in general when subjected to different loading procedures;
 - Comparison of the results obtained from the former study ^[1], with the results obtained from the experimental campaign performed within the scope of this work from reinforcing similar joints with the same characterization with self-tapping screws;
 - In the former study ^[1], cyclic tests were performed according to the standards: EN 12512:2001, following the governing equations and values which are provided within. These rules are proposed to be updated on the working document for the proposal of the latest version of the standards: EN 12512. Therefore, the tests' results obtained from ^[1] have been processed and the parameters have been obtained according to the new proposed method, and the results were compared and validated accordingly and discussed in chapter 5.
- II. Which leads to the main focus of this dissertation which is conducting a full-scale experimental campaign for a number of dowel-type timber-to-timber joints in double shear, reinforced and repaired with self-tapping screws. Multiple number of reinforcing and repairing schemes were developed and tested.
- Hence a comparison with a previous study is in the scope of this work, then in order to obtain realistic reliable results, the wood species and specimen characterization are the same as the previous study mentioned, ^[1]. Two species are used in this work which are Maritime Pine (*Pinus pinaster*), and Scots Pine (*Pinus sylvestris*), and the screws are VGZ fully threaded self-tapping screws, and HBS shear screws. More details on the geometry of the timber elements, dowels and screws are detailed and presented later in this report.
- The experimental campaign consists of two main tests: Monotonic (EN 26891:1991) and cyclic (EN 12512:2001), performed on both, *Pinus pinaster* and *Pinus sylvestris* wood species. As mentioned before, the tests were carried out for a number of different reinforcing and repairing scenarios, which are detailed and presented in chapter 3.
- Finally, the results were weighed in and presented in terms of load-carrying capacity, embedment strength, energy dissipation and ductility of the joints under study, for each of the schemes presented for reinforcement and repairing, for both specimens. In light of the results obtained from testing, the efficiency of the reinforcement and repairing schemes in increase and restoring the connection's capacity was calculated and discussed.
- III. Leading to the third and last part of this dissertation, which is the discussion and validation of the design models that are presented to be added in the new version of EC5 ^[3] based on the results of the two previous steps. Based on which, conclusions and recommendations for research continuation are reasoned and driven accordingly.

CHAPTER 2

2. LITERATURE REVIEW

2.1 JOINTS IN TIMBER STRUCTURES

Joints are the main source of ductility in timber structures. Moreover, in most cases, the design of the connections governs the design of the elements. In many circles, the competitiveness of a timber structure, compared to other building materials, is dependent on the efficiency of the connections. Therefore, to ensure a robust structure, the design of timber joints must be bounded by many factors differing accordingly with design necessities for each specific case. However, the mechanical behavior of dowel-type joint is the main focus when it comes to design, especially for seismic design. In order to explore the behavior of dowel-type joints under seismic loading, the dynamic performance and response to the joint under cyclic loading is rather a necessity. The embedment strength of the joints contributes to the dynamic behavior in general. Thus, ductility and energy dissipation capabilities are substantial when it comes to ensuring a safe design of the joints.

Wide variety of joints and reinforcing techniques are available, and their behaviour contribution to the structure's ductility in general is dependent on many factors such as, the type of the joint itself, its position on the structure, usage and if it is mechanically fastened. Timber connections can be steel-to-timber joints, timber-to-timber joints, or hybrid joints, which include glued joints either with a wood-based material or FRP.

Wood species in general are identified by two main categories, Softwood and Hardwood, which are botanical terms that do not always reflect on the density. There are many factors based on which the wood is categorized. Balsa wood is an example of a hardwood that is in fact used in lightweight structures, whereas, softwoods like Douglas Fir has a good natural durability and relatively high strength properties. However, for structural use, softwoods are commonly used due to their availability, high workability and low cost. Their fast rate of growth provides a continuous supply. As for hardwoods, in addition to being very expensive, it's quite difficult to obtain large quantities at a time especially for big scale projects, due to the fact that hardwood trees grow in a slow rate, opposed to coniferous, which makes softwoods preferable in construction industry. Nonetheless, hardwoods are a better option for external exposed areas and claddings where durability and sometimes special aesthetic features are a requirement. Regarding timber connections, they are made from softwood in most cases, therefore in this research project the specimens tested are pines, as mentioned earlier.

Defects on timber elements in general, and in connections, specifically, tend to reduce the load-carrying capacity, therefore, the presence of which should be limited in the joint area.

2.2 DOWEL-TYPE CONNECTIONS

Connections has a great influence on the overall stiffness of the structure, as mentioned before, as well as ductility. Therefore, they should have high rigidity but at the same time good ductility. High rigidity without enough ductility can result in brittle failures, especially when the connection is

subjected to horizontal actions. In this case, the damage becomes irreversible. In contrast, if the connection is ductile, the damage could still be reversible to some extent. Mirdad & Hei Chui (2018) studied the behaviour of different types of timber-concrete joints, including dowel-type joints and the load-slip behaviour of these joints, Figure 2:1, reflects on how stiffness and ductility in a connection are complementary parameters. As glued and notched joints showed almost linear behaviour up to failure, which projects high stiffness, the lack of ductility in such types results usually in brittle failures, opposed to dowel-type fasteners that are ductile with non-linear behaviour and low stiffness. This shows that not only in timber-to-timber connections that the slenderness and plasticity of the dowels provide more ductility to the connection.

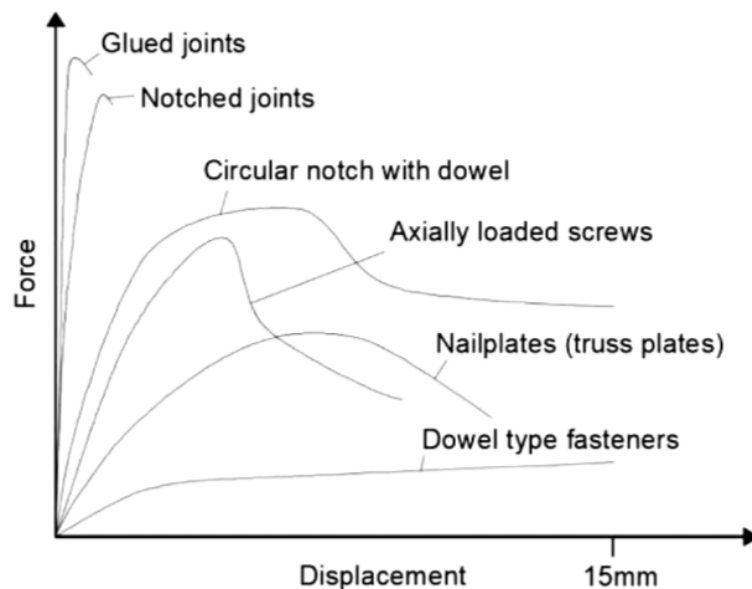


Figure 2:1 - Load-displacement relationship for different types of connections

However, dowel-type connections can also be subjected to brittle failures that are a result of high tensile forces perpendicular to the grain, as well as shear stresses. When this is the case, this can be avoided either by preventing these forces from emerging, which cannot always be guaranteed, or by introducing reinforcement to the joint.

The lack of information about their in-service dynamic vibrations is still a challenge for engineers in some ways. Lately, the use of dowel-type connections in timber structures has increased significantly due to their huge contribution in relieving the timber elements from carrying tensile and compression forces perpendicular to the grain, in addition to shear stresses.

Mechanical fasteners are the main source of ductility in joints. Their function is originally to transmit the forces between the two pieces of wood, as well as reinforcing notch joints and generally, keep the timber elements in position. Consequently, they deform in a plastic manner and bend under load resulting in their embedment into the surrounding timber fibres. Therefore, when estimating the shear resistance capacity of the joint, the plastic nature of the dowels should be taken into account.

The failure mechanism of timber joints with mechanical fasteners among other things depends on the geometry of the joint and the type of fastener.

A fastener in timber connections could be a metal fastener, wooden, or an FRP rod. Normally, a fastener is either a dowel, which is a smooth or rarely rugged cylinder without a head and its diameter varies between 6mm or 30mm, it can be a nail, which is a smooth or rugged cylinder with a head and a diameter that is $\leq 8\text{mm}$, in addition to bolts, screws, coach screws and staples.

In order to determine the load-carrying capacity for each type of fastener in dowel-type joints, the possible failure mechanisms are driven and the load corresponding to each failure mode is determined directly from stress equilibrium conditions. These failure modes were developed based on the European Yield Model (EYM) that was later developed by (Johansen K.W, 1949). The failure associated with the modes developed in this theory is dependant directly on the slenderness of the dowel. In order to obtain the final failure mode, which corresponds to failure mode (k) in Figure 2:2 below, the maximum yield capacity of the fastener has to be reached, and the fastener is assumed to have a rigid-plastic behaviour up to failure. Eventually, to reach the full yield capacity of the dowel, the entire cross-section has to be plasticised and two plastic hinges per shear plane occur as a result.

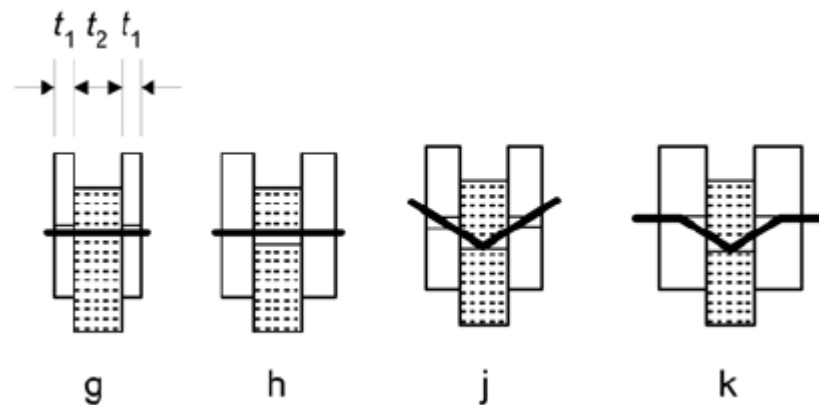


Figure 2:2 - Failure modes for timber-to-timber dowel-type joint in double shear [3].

For the case under study in this project, EC5 ^[3] has provided the failure modes that correspond to dowel-type joints in double shear, shown in Figure 2:2 above.

Four equations were developed to estimate the load-carrying capacity of dowel-type joints, both in single and double shear. Each equation corresponds to a specific failure mode and is a function of the embedding strength of the wood, the yield moment of the fastener and its diameter. Thus, the slenderness of the dowel plays an important role in predicting the maximum capacity and the failure mode for this type of joints. Another important parameter that influences the load-carrying capacity of the joints is the contribution of the withdrawal capacity of the fastener itself, which is known as the rope effect, detailed in section 2.2.2. The withdrawal capacity of the fastener is a measure of its ability to resist tensile forces perpendicular to the grain, based on its tensile strength. This capacity

can add to the overall capacity of the joint, as these forces in this case can be taken by the dowel instead of the timber which is known to be weak in resisting these type of forces. The equations of the load-carrying capacity are included and elaborated in chapter 3.

2.2.1 GROUP EFFECT

Dowel-type joints can comprise a group of fasteners instead of one in some cases, and that is due to the low load-carrying capacity provided by a single dowel. Hence, the quantity, diameter and spacing of fasteners depend on the load transferred to the dowels and the possibility to mitigate these emerging forces. However, for a connection to be considered a joint, a minimum of two fasteners should be present.

Specific spacing criteria should be adopted in order to limit the possibility of splitting due to wedge effect, as well as restrained shrinkage, which assists in increasing the tensile stresses perpendicular to the grain, subsequently causing splitting. Figure 2:3 shows the possible failure mechanisms in a connection with a group of dowels. Splitting of dowel-type joints can be prevented if the axial load-carrying capacity of each screw is greater than 30% of the lateral load-carrying capacity by shear plane, of each dowel [3].

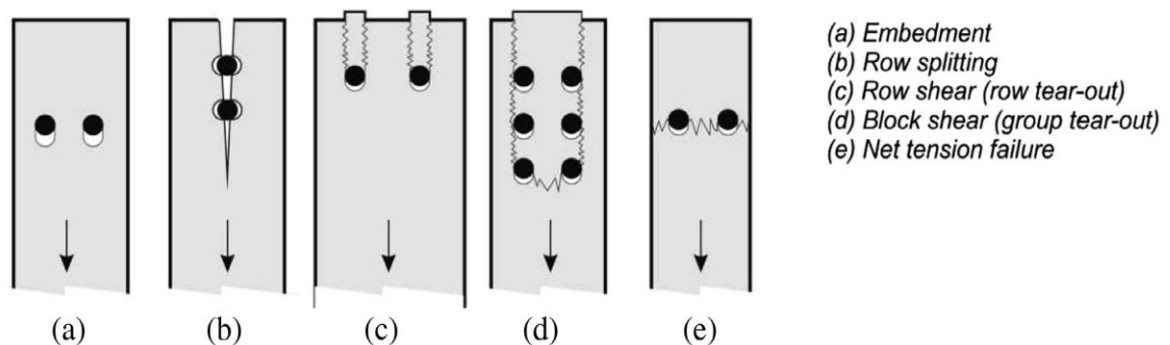


Figure 2:3 - Different failures modes of unreinforced dowel-type joints [13].

When a group of fasteners is loaded in tension, the fasteners usually do not reach their maximum load-carrying capacity at the same time due to several factors that include but not limited to:

- Local variations within the timber element;
- Bore-holes sizes and misalignments;
- Uneven load distribution between the connecting elements.

If the fasteners are placed in a row, and the joint is loaded in tension, then it's most likely for the first and last fasteners to reach their ultimate capacity first and fail before the rest of the fasteners. (Cramer, 1968, Lantos, 1969).

However, the group effect is maximum in the joint with stout dowels, for example, due to their linear-elastic behaviour under loading. Adding to this, the uneven load distribution between the dowels, which causes the total capacity of such a joint to become further less than the sum of the capacities

of the dowels when measured individually. In contrast, the group effect becomes much less apparent in the joints with slender dowels, such as nails, due to their high plasticity. Plasticity in dowels helps in redistributing the load so that the total capacity of the joint is only slightly less or equal to the sum of capacities of the individual dowels. The minimum spacing between the fasteners governs the group effect, if the spacing exceeded a value of 13 times the fastener diameter, group effect should not be considered [2].

2.2.2 ROPE EFFECT

EC5 [3], provides an approach to estimate the characteristic load-carrying capacity of dowel-type connections following Johansen's theory, taking into account the contribution of the fasteners rope effect. The fastener's rope effect is the contribution of the tension component of the fastener to the connection's strength. In other words, it's the tensile capacity of the single dowels in shear connections. In shear failures, part of the loads is taken in tension as well, and this part depends on the end anchorage of the dowel and the surface roughness. Subsequently, the effect can be either large or small depending on many factors such as, the type of the fastener, its surface and end anchorage.

Regarding the surface of the dowel, the direction of the elements annual rings, if the dowel is threaded or not and if it's twisted, are all factors that contribute to the magnitude of the rope effect. As for the anchorage, it can provide higher resistance if it is enhanced by washers and nuts, either on the head side or the point side of the fastener. For shear screws in general, washers can be quite important in the sense that the axial load-carrying capacity of the screws is limited by the pull-through strength of the screw's head. Threading in the case of screws and bolts, increases the resistance of a joint by 2.6 times its shear capacity [2]. Therefore, the part of the equation to calculate the load-carrying capacity of a fastener per shear plane, that accounts for the rope effect, which is represented by the characteristic withdrawal capacity divided by 4, can be omitted if the dowels are threaded. The introduction of a smooth and threaded shank enables the screws to be used for loading in shear, bending, or a combination of both. Eurocode 5 [3] listed the contribution of the rope effect to the load-bearing capacity of a dowel type joint according to the fastener type. Table 2-1. However, the tensile action effect of a dowel is better derived from experiments.

Table 2-1 - Rope effect of different fasteners [3].

FASTENER	EFFECT
Round nails	15%
Square Nails	25%
Other Nails	50%
Screws	100%
Bolts	25%
Dowels	0%

2.3 REINFORCEMENT TECHNIQUES

2.3.1 GENERAL OVERVIEW

Wood in general presents low resistance to tension and compression perpendicular to the grain as well as shear. High tensile stresses perpendicular to the grain occur mostly at connections, notches, holes or curved beams and beams with a variation in depth. Thus, the design should be made in order to avoid these forces, as well as providing higher stiffness, strength and aiming at more ductile failure modes for the structure. However, since it's not possible to completely avoid/prevent these forces from merging, the design should be carried out in order to compensate the wood's low resistance to these forces with providing adequate reinforcement. In order to increase the load-carrying capacity of a dowel-type connection, numerous reinforcement techniques can be adopted include, but not limited to:

- Punched metal plates;
- Nail plates;
- Glued in rods and glued in wood-based panels;
- Threaded rods;
- Self-tapping screws (STS);
- Fibre Reinforced Polymers (FRP).

Each of the above mentioned reinforcing techniques have its own advantages as well as limitations. The choice of reinforcement is governed by many factors including the type of connection itself, either timber-to-timber or timber-to-steel, wood material, service class, load duration, and the different types of fasteners. Therefore, several factors intervene in the design. All aim eventually to increase the load-carrying capacity, as mentioned earlier, and to prevent failure of the timber element either by splitting or embedment.

The load-carrying capacity in dowel-type connections is influenced by three main parameters, which are the dowel yield moment, withdrawal capacity and embedding strength. As for splitting mechanisms in dowel-type connections, they can vary according to the number of the dowel fasteners and their position with respect to the grain direction. However, for each reinforcement technique, the adaptation is different.

Taking reinforcing with threaded rods for instance, if the connection comprises a group of dowels, an optimal number of screws is required in this case with appropriate positioning and spacing to prevent splitting, for example, block shear failures. On one hand, if the reinforcement is less than the minimum required, the contribution of the reinforcement will be insufficient in order to provide the connection with enough capacity to sustain generated forces, especially, shear and tensile perpendicular to the grain. On the other hand, excessive reinforcement can weaken the elements and reduce the effective section by reducing the mechanical behaviour of the connection,

particularly if the reinforcement were placed in the areas where the highest stresses are concentrated.

2.3.2 AREA OF FOCUS

Self-tapping screws (STS) were chosen as the focus of this research project, in addition to reinforcing with FRP, specifically, Glass fibres (GFRP), for many reasons which are briefly mentioned below, and later on discussed with more depth throughout experiments and analysis within this report.

Regarding FRPs, these materials have proven to be very powerful, their development is considered a revolution in the construction industry. However, when it comes to timber, many limitations are yet to be overcome in order to utilize their full capabilities to reinforce existing timber structures, and to replace many other intrusive, less effective solutions. In order for these materials to be developed to better suit the application on timber, a comprehensive research is ought to be carried out in this area especially that many standards lack specifications and design rules for the use of these materials with timber structures. Therefore, extensive experimental campaigns for application on different members, various geometries and reinforcing scenarios using different forms of FRP materials, should be adopted in general and more studies should be conducted in order to clear any doubts around their use, and to conclude on the exact behaviour of timber elements when reinforced with these materials.

In addition to developing practical solutions to avoid failures resulting from the difference in strength between the two materials, the effect of fluctuant environmental conditions and the different formulations of adhesives and their impact on restricting the growth of a natural material such as wood. All in all, this is an open area for research with a handful of possibilities for further development.

As for self-tapping screws (STS), previous studies have already stated that this reinforcing technique has much more potential than normal shear screws and other metal fasteners in so many aspects. Reinforcing with STS increases the connection's ductility and moment capacity under monotonic and cyclic loadings. (Lam, Gehloff and Closen, 2010) mentioned that STS reinforcement has increased the moment resisting capacity of dowel-type connections up to 170% under reverse cyclic loading. Over and above, STS can effectively reduce the connection's tendency to split, due to their plasticity and high withdrawal capacity. Being less intrusive, economic and much easier to install added to their value as an effective reliable reinforcing technique. However, similar to FRPs, standards lack specific design models and rules to govern the design and use of STS, which makes this technique as well an open area of research and development.

2.3.2.1 SELF-TAPPING SCREWS

This technique is found to be very efficient in compensating the low strength properties of timber against tensile and compression forces perpendicular to the grain, as well as shear stresses. With

this technique it is possible to reduce the end-distance without reducing the load-carrying capacity of the dowel-type connection, in addition to increasing the bending capacity of a moment resisting connection. Consequently, the effect of reinforcement is maximal for small end-distances.

Many types of STS are available nowadays varying in length, diameters and if the shank is fully threaded or not, and the choice is dependent on the future intended use. Figure 2:4. However, self-tapping screws are mostly threaded throughout the entire length. The diameter of the threaded part, opposed to the normal shear screws, is not turned down from the original diameter of the shank, instead it is usually fabricated by rolling a wire around the shank resulting in a final diameter that is bigger than the original shank diameter. Threading in general increases the shear capacity and resistance of a joint. However, in self-tapping screws, the continuous thread is originally optimized to enhance the load-carrying capacity of the traditional screws by helping in uniformly distributing the load that is transferred from the timber element to the screw. During manufacture, the thread is hardened leading to higher torsion and bending capacity but also to more brittle failure mechanisms.

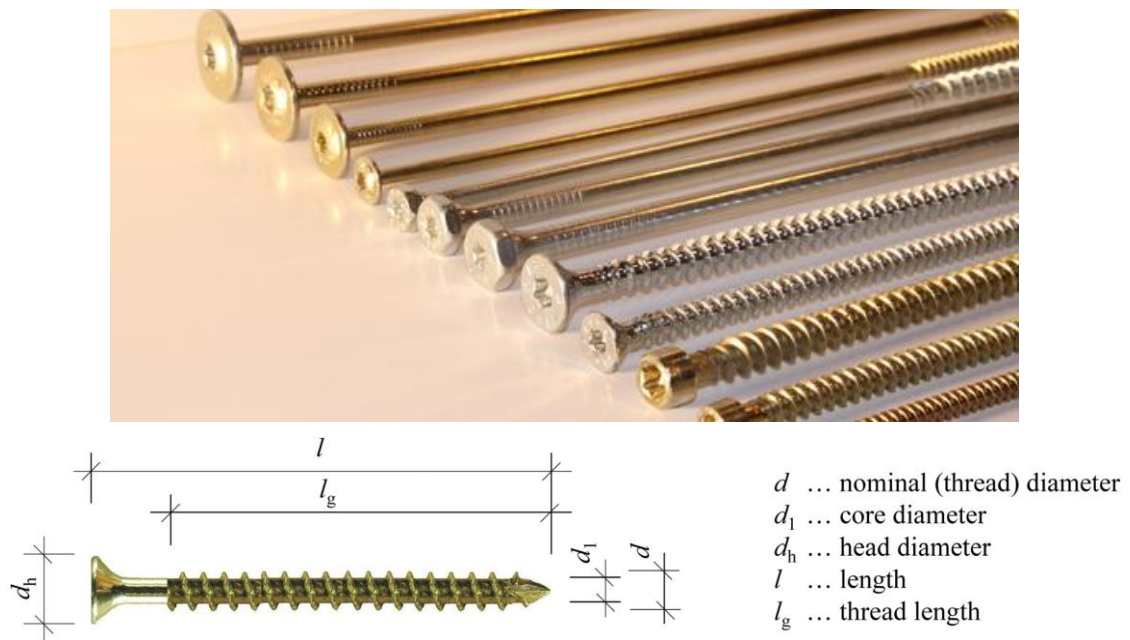


Figure 2:4 - Different configurations of self-tapping screws (STS), and geometry parameters [16].

Self-tapping screws tend to increase the local strength in shear and tensile perpendicular to the grain, thus, counteract the element's tendency to split. In other words, increasing the embedding strength, consequently increasing the load-carrying capacity with the possibility of reducing the end-distance.

Screws in general have a high tensile strength and their rope effect, or tensile resistance contribution, to the shear capacity of a dowel-type joint with a single fastener can reach 100% [3]. Therefore, they are usually placed perpendicular to the grain and perpendicular to the fasteners' axis. In contrast, self-tapping screws have higher withdrawal capacity, even higher than their shear resistance. Thus, in order to utilize this capacity in dowel-type connections in particular, they are

better placed inclined, at an angle to the grain, so that the capacity of the joint to resist tensile forces perpendicular to the grain is provided by the high tensile strength of the screws instead of the low resistance of timber elements to these forces. In some cases, screws can be placed crossed like a truss, referred to as a cross screw connection, so that the tensile and compressive forces on the joint are fully supported by the screws. Figure 2:5.

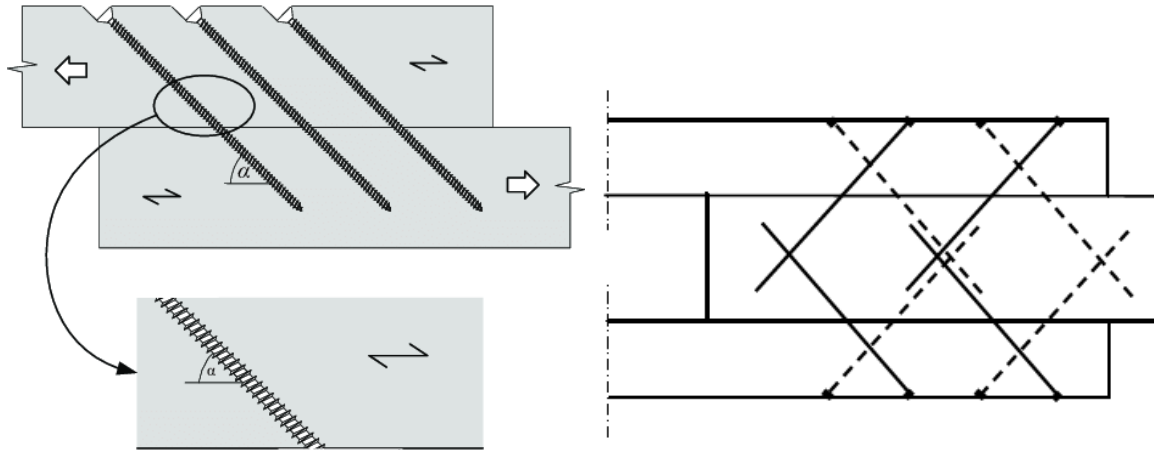


Figure 2:5 - STS placed inclined with respect to the grain direction [15].

Since embedment is a nonlinear process, the stiffness of the connections then depends on, amongst other factors, the nature of the applied load. Effective reinforcement is therefore achieved by ensuring that the resulting shear and tensile forces perpendicular to the grain are taken entirely by the screws and not the timber element. This technique in general protects the element against brittle failure, thus, only ductile failure modes can be achieved in this case and that is according to the Johansen's theory (Johansen, 1941&1949).

The basic assumption in Johansen's theory is that the materials have an ideal rigid-plastic behaviour in embedment and bending for timber and steel, respectively, Figure 2:6. Based on this assumption, Johansen developed a model based on the theory of plasticity that is referred to as the European Yield Model, which is the base for the failure mechanisms assumptions adopted in the current version of EC5 [3].

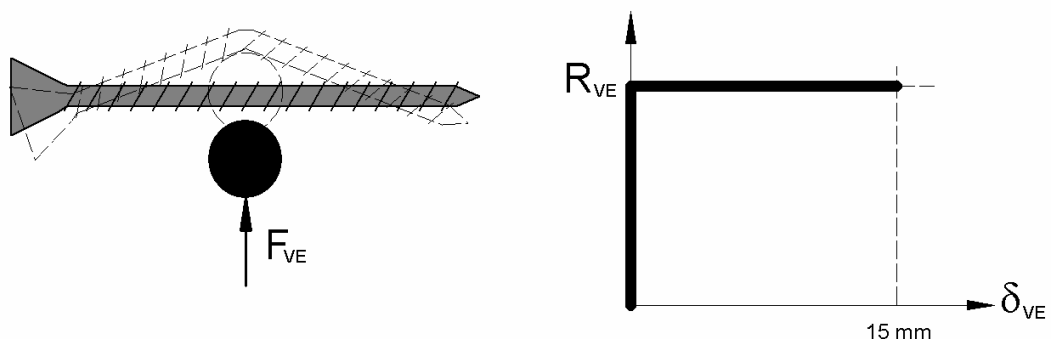


Figure 2:6 - STS load-carrying behavior based on Johansen's yield theory [4].

Many scenarios concerning the position of self-tapping screws with respect to the dowel-type fasteners can be adopted. For example, the screws can be placed at a certain distance between the fasteners, or in direct contact with the fasteners, which is the case in this work. Many studies were conducted over the years addressing the influence of the STS position to the dowels in a dowel-type connection and the failure modes resulting from each. Figure 2:7.

It was concluded in several case studies that the joints that were reinforced where the self-tapping screws are in direct contact with the dowels, have higher ductility, load-carrying capacity and stiffness than the connections that are reinforced without contact between the dowels and the STS, as well as unreinforced connections [4]. Therefore, in this research project, the experimental campaign was conducted with the STS placed in direct contact with the dowels for both cases where the STS are perpendicular to the grain direction and when inclined at an angle to the grain.

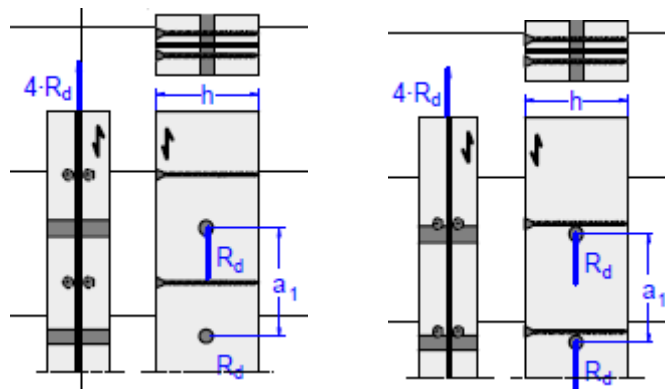


Figure 2:7 - STS position with respect to dowel fasteners, Left: Screws placed at a distance from the dowels, Right: Screws are placed in direct contact with the dowels [4].

Self-tapping screws do not require predrilling in most cases if the density of the timber element is less than 500 kg/m^3 , which is the case for most of softwoods [2]. Nevertheless, if they are introduced to timber elements in temperatures below $[0^\circ\text{C}]$, it is recommended then to pre-drill in order to avoid splitting of the timber element giving the fact that its most likely to happen in these conditions. If the screw's diameter is less than 6 mm, pre-drilling is preferred in this case to avoid buckling of the screw. However, pre-drilling is allowed in all cases as it helps in providing additional precision when positioning the screws, which is an important aspect especially if the screws are being placed inclined with respect to the grain direction and it can be a challenge if placed free-handed. For this kind of application, CNC can be used for further precision. In any case, if pre-drilling is to be carried out, the borehole diameter should not be more than 70% of the shank diameter in order to restrict the free movement of the screws. Some regulations, such as the European Technical Approvals (ETAs), allow for a reduced minimum spacing if pre-drilling was adopted. Subsequently, pre-drilling diameter should be determined by tests for densities more than 500 kg/m^3 .

When reinforcing with self-tapping screws, a reduction in the cross-section must be considered if the screw's nominal diameter is greater than 6mm, nonetheless, for screws with nominal diameter

that is $\leq 6\text{mm}$, without pre-drilling and if placed in areas with compressive stresses, no reduction is needed ^[3]. Although, recent researches have shown that even for members that are under compression, the cross-section reduction should be taken into account, and that is due to the stress peaks found in the vicinity of the fasteners, even if the holes were filled with a material that has higher stiffness than the wood. The only exception for this rule is in the case of glued-in rods, due to the contribution of the glue line in reducing the stress peaks.

2.3.2.2 FIBRE REINFORCED POLYMERS (FRP)

When it comes to repairing existing timber joints or elements in general, a search for more advanced and efficient solutions is always a challenge. FRPs have proven to be an excellent strengthening solution for many building materials especially concrete, but for timber structures it's still a questionable approach with many uncertainties. These doubts are even stronger if cultural heritage was involved and this technique becomes less favourable to be applied due to many reasons. Taking connections as an example in existing structures, specifically historical buildings, it is still quite difficult to guarantee a long service life because the implementation of FRP reinforcement for timber joints is relatively recent with the oldest joint being around 60 years old ^[2]. Over and above, this reinforcement technique in general requires accessibility to a large surface area to facilitate its installation, which can be a challenge in certain historical constructions.

Fibre Reinforced Polymers (FRP) are composite materials that consist of fibres that provide the load-carrying capacity and the strength, and of resin, which is a polymer matrix that primarily provides protection in addition to transferring the load between the fibres. The resin can be epoxy, vinyl-ester or polyester thermosetting plastic.

Many configurations and manufacturing processes are available depending on the type of material itself, the orientation of the fibres and the volume fraction. Carbon, Glass-fibre, Aramid and Basalt are all fibre materials with varying mechanical properties that are used in strengthening applications. In general, two forms of carbon fibres (CFRP) are available, one with high modulus of elasticity and the other is with high strength property, but less density. Both forms have very high mechanical properties compared to other forms of FRPs, which makes them far more powerful, therefore, more expensive. Hence, in most cases of application with timber structures, glass fibres (GFRP) are a preference. GFRP have the maximum density amongst other forms that are used for structural strengthening ($2.5 - 2.6 \text{ g/cm}^3$), low modulus of elasticity compared to both types of carbon fibres (CFRP), yet their tensile strength is very high ($2000 - 4800 \text{ MPa}$) ^[2]. FRP reinforcement in timber connections can be found in a form of pultruded plates, which is the case in this research, and pultruded rods.

In many circles, reinforcement techniques with FRPs are based on the use of adhesives on site. Selecting the adhesive is a sensitive process. The adhesive has to provide an adequate bond between the two materials. This was a challenge for a long time due to the dimensional changes of the timber surface following environmental changes and growth. Furthermore, epoxy-based

adhesives, being the most used on site applications, was found to be too rigid for applications with timber elements. Therefore, many attempts have been made lately to develop the adhesives' formula to be more suitable for use with timber structures.

A key to a successful design of a composite system of timber and FRP is the adequacy of the bond. Many factors contribute to the judgement of either the bond is effective, including the bond-slip relationship and the stress distribution on the substrate, which differ depending on the type of reinforcement either externally bonded reinforcement (EBR) or near surface mounted (NSM).

FRP applications in timber structures are found in many forms to serve different purposes and strengthening needs, such as:

- Beam end reinforcements;
- Tension reinforcement perpendicular to the grain;
- Flexural strengthening;
- Shear strengthening.

In this work, the use of FRP for tension reinforcement perpendicular to the grain is covered, as mentioned earlier, particularly GFRP plates are used for the reinforcement of timber-to-timber dowel-type connections. Usually for this type of application, the GFRP plates are pressured over the timber element that has the adhesive previously applied over a clean, dry surface. Figure 2:8

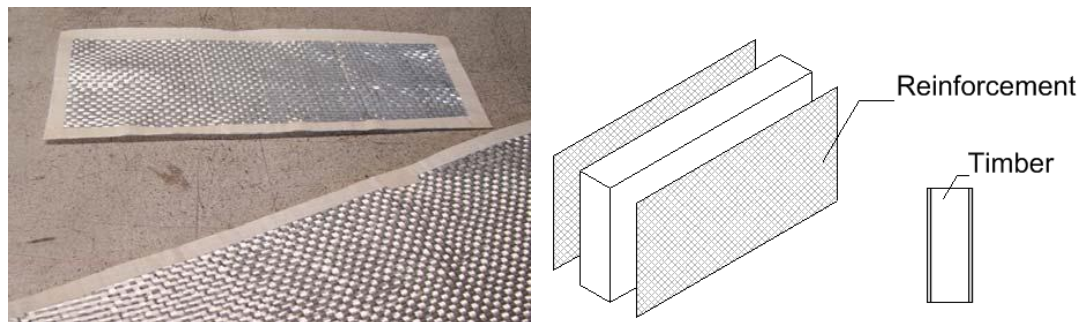


Figure 2:8 - Left: Fiber-glass Sheets (GFRP), Right: GFRP reinforcement schematic in a timber joint. [16].

FRP reinforcement in general is known through previous studies and experimental campaigns that it tends to increase the strength and stiffness but it does not provide much ductility to the structure. Nonetheless, most studies that are available for FRP reinforcement of areas with high tensile stresses perpendicular to the grain are for notches and there is no extended data base for timber-to-timber dowel-type connections reinforced with FRP. Thus, there are still many uncertainties about the contribution of FRP reinforcement to the over-all ductility of the connection. One of the reasons that this technique is not very common for reinforcing regions with tensile forces perpendicular to the grain is that there are many other known techniques that are less expensive and demanding such as the self-tapping screws. Therefore, this work provides a comparison of both techniques in

order to conclude based on experimental work which is the most suitable reinforcement technique to maintain these forces.

CHAPTER 3

3. MATERIAL CHARACTERIZATION AND MONOTONIC TESTS

In order to develop a better judgement on the effectiveness of reinforcing dowel-type joints with STS, original joints without any reinforcement were ought to be tested as a first step to determine the capacity of the joint under loading in comparison with the estimated capacity calculated according to EC5^[3] for dowel-type connections in double shear. Connections are therefore subjected to monotonic and cyclic loading to simulate the conditions that the structure can be subjected to in practice, whether it's seismic loading or an increase in loading due to a change of the structure function for example. As mentioned earlier, in this project, two wood species were chosen, Maritime Pine (*Pinus pinaster*) and Scots Pine (*Pinus sylvestris*) to compare the effect of the reinforcement when adopted for different densities and typologies. The geometry of the specimen as well as the procedures adopted for each test are elaborated below.

3.1 TIMBER ELEMENTS

The connections that were chosen for this work are dowel-type connections in double shear. Therefore, each joint consists of three timber pieces, connected with a mechanical fastener that is a 12 mm steel dowel. As specified before, the joints were formed for both pine species, Maritime and Scots pine. The sizing of dowel-type joints in general should be determined taking into account the characteristic shear strength and the dowels plastic yield moment. However, in this project the geometry of the connections was taken as the same of (Matos, 2011), for the purpose of comparison, as mentioned earlier in this report, with a cross-section of 80 x 60 mm and a height of 220 mm. Nevertheless, in order to perform cyclic testing, the dimensions of the central element had to be adjusted in order to allow for the fixation of the connection in the test apparatus. Figure 3:1 shows the elements' geometry for both, monotonic and cyclic testing procedures. Details of both tests apparatus, are elaborated within the tests' descriptions in 3.4 and chapter 4, for monotonic and cyclic tests, respectively.

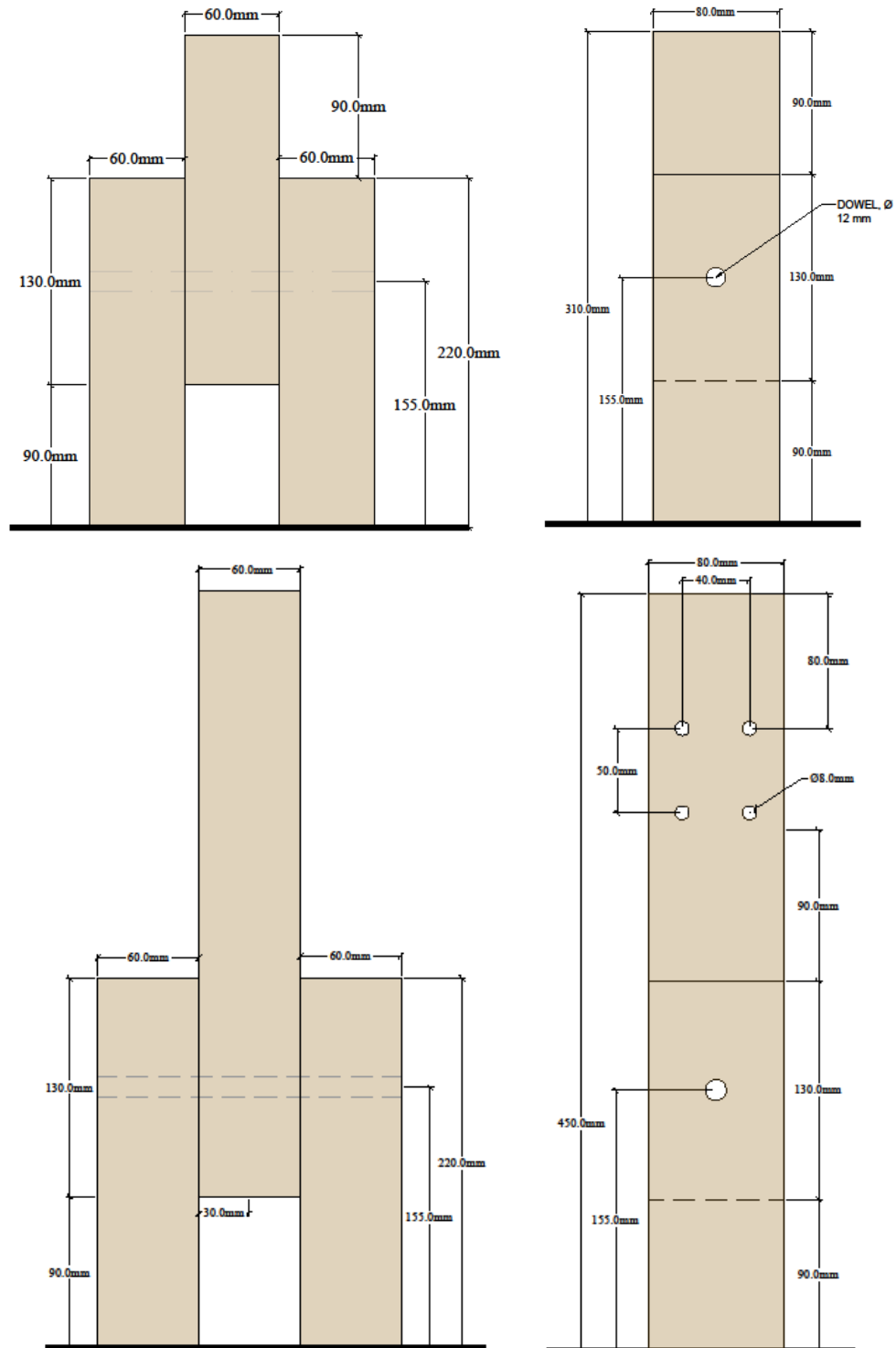


Figure 3:1 - Geometry of the specimen for monotonic and cyclic testing.

3.2 WOOD STABILIZATION PROCESS

Upon the arrival of the specimens they were placed in a climatic chamber in the Laboratory of Structures of the University of Minho (LEST) in order to stabilize the specimen before testing. The conditions of the climatic chamber were set following the conditions specified in EN 26891 [8], which are a Relative Humidity (RH) of $[65\pm5]$ % and a Temperature (T) of $[20\pm2]$ °C. The samples were kept in the climatic chamber until a value of variation between two consecutive readings was less than 0.5%. For this purpose, 6 samples from each of the species, 3 monotonic and 3 cyclic, were chosen to measure the variation. A total of 4 measurement were taken, after which the samples were stabilized. Table 3-1 shows the stabilization process and the associated densities of the specimen before testing. After the samples were stabilized, they were removed from the chamber and drilled to allow for the 12mm dowel to be installed.

Table 3-1 - Stabilization calculations

	Sample Tag	VOLUME (mm ³)	WEIGHT (g)	ρ (kg/m ³)	Δ (%)
Scots Pine	A-SC	1029565	521.3	506.28	0.15%
	B-SC	1040655	636.0	611.18	0.12%
	C-SC	1042440	502.9	482.41	0.06%
	1-C-SC	1702890	818.9	480.89	0.06%
	2-C-SC	1655349	786.9	475.35	0.04%
	3-C-SC	1673299	914.3	546.41	0.04%
Maritime Pine	A-MR	1016379	616.7	606.77	0.74%
	B-MR	1003664	653.0	650.62	0.47%
	C-MR	1058145	654.6	618.67	0.37%
	1-C-MR	1701660	1233.7	725.00	0.36%
	2-C-MR	1673299	959.5	573.41	0.40%
	3-C-MR	1701660	966.6	568.05	0.18%

3.3 DOWELS CHARACTERIZATION

Prior to forming the connection in order to apply monotonic and cyclic loading, a characterization of the steel dowel is an essential step to properly estimate the capacity of the dowel-type connection, as the load-carrying capacity of this type of joints is influenced directly by the properties of the mechanical fastener. Thus, a four-bending test for the dowels was carried out, and a value of the yield moment, M_y , was obtained accordingly. The value was implemented later on in the load-carrying capacity equations provided in EC5 [3] in order to estimate the overall capacity of the connections, which is crucial in order to set the protocol for monotonic testing.

The steel dowels used in this experimental campaign are Rothoblaas 12 mm dowel, of class S235 steel. The test were performed following the standard EN 409: 1993, which is specified for determination of yield moment for dowel-type fasteners. The test was performed using an electromechanical machine, with an actuator with a load cell of 100 kN and the results were recorded

and acquired later on with Dynatester software. The loading procedure was set according to EN 409:1993, which specifies that the load shall be applied in a rate that the maximum force is reached at $[10 \pm 5]$ seconds. Thus, the protocol followed for testing the dowels was calculated accordingly, applying a rate of 1 kN/sec. Referring to the apparatus described on the same standard, the force should be applied in two points with a distance (b) that is between $\emptyset$ and $3\emptyset$ and should not deviate by more than 5%. Regarding the distance from the force to the support on each side, (a) and (c), it should be larger than $2\emptyset$. The test apparatus is shown in Figure 3:2 along with the yield moment diagram.

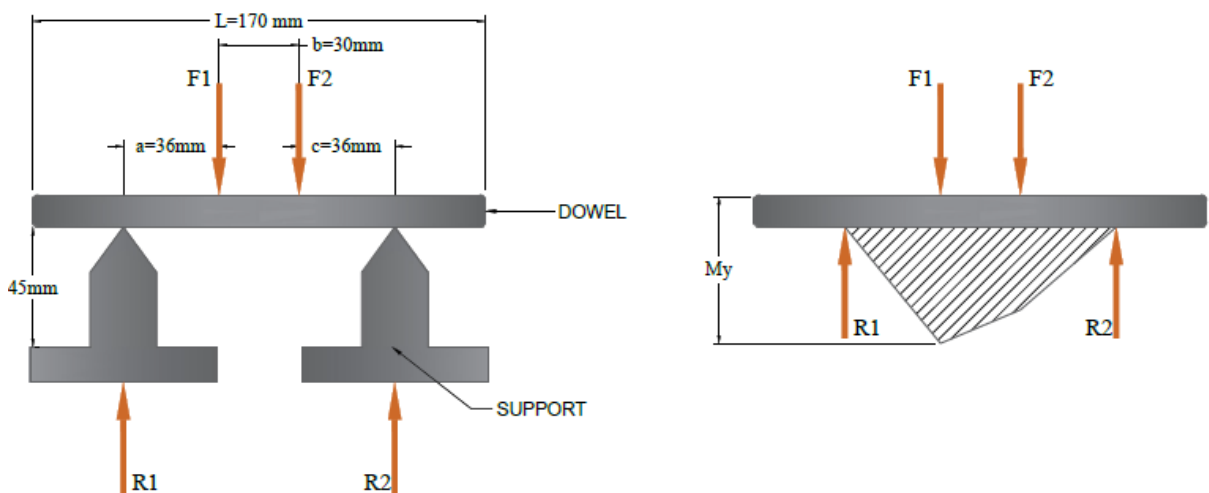


Figure 3:2 - Left: Test apparatus according to EN 409:1993, Right: Yield moment diagram (M_y)

The value of the bending moment is the force multiplied by the distance to the support, as shown in the following expression, (3-1), calculated to an accuracy of 1%, and the final value was taken as the maximum that is obtained from each support.

$$M_y = \begin{cases} F_1 R_1 \\ F_2 R_2 \end{cases} \quad (3-1)$$

The test was performed on a total of 6 dowels, and the average maximum force obtained was 12 kN with a coefficient of variation (CoV) of 5.92%. Figure 3:3 shows the force-displacement graphs obtained for the tested dowels. The yield force, however, was then determined statistically by finding the point at the intersection of the two tangent lines that represent the elastic and plastic slips.

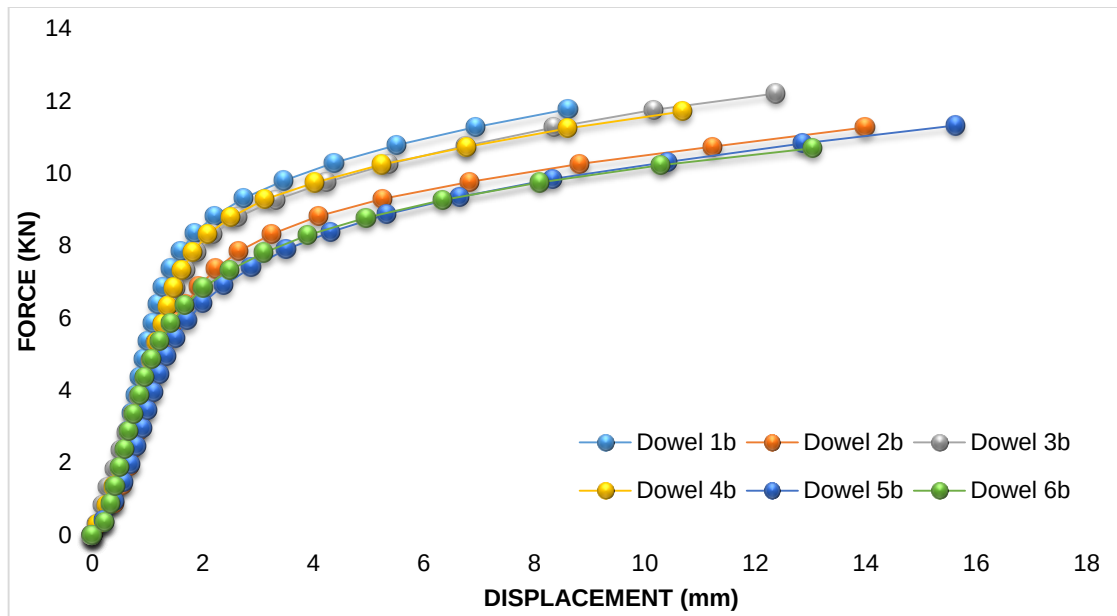


Figure 3:3 - Force-displacement curves of tested dowels.

Nonetheless, the characteristic value for the yield moment, $M_{y,RK}$, was calculated first before conducting the tests according to the equation provided by EC5^[3], that depends directly on the value of the characteristic tensile strength $f_{u,k}$. This value for a galvanized structural steel dowel of grade S235 with a diameter between 3 to 16 mm, according to EN 10025, lies in the range of 350 to 510 MPa. Hence, the corresponding characteristic yield moment is 69 kN.m, which is much less than the value obtained based on the yield moment from testing that is equal to 128.4 kN.mm. The characteristic value was calculated assuming a lognormal distribution based on the value of the yield moment from testing, 147.5 kN.mm.

Figure 3:4 shows the dowel in the tests apparatus before and after testing, as well as the tested dowels' deformation, and the calculations and the final value for the dowels' yield moment are presented in Table 3-2.

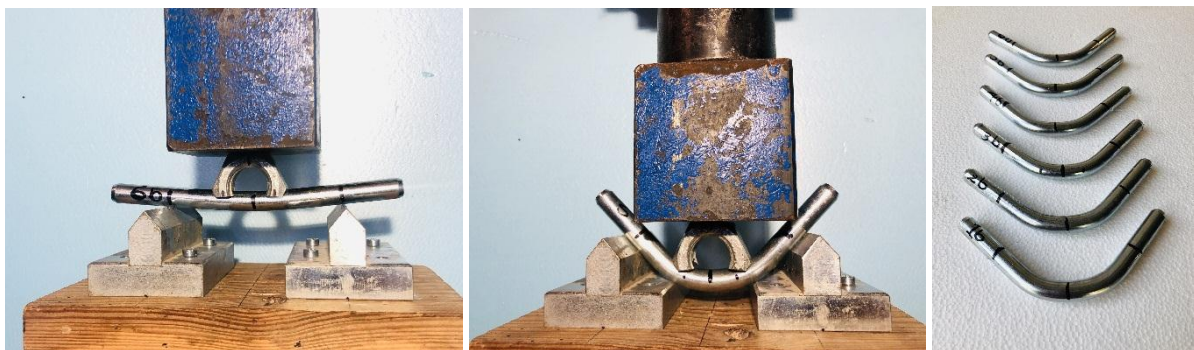


Figure 3:4 - Left: Dowel before testing, Middle: Dowel after testing, Right: Dowels after deformation.

Table 3-2 - Yield moment calculations.

Sample Tag	F_y (KN)	R1 (KN)	R2 (KN)	M_{ya} (N.mm)	M_{yc} (N.mm)	$M_{y, \max}$ (N.mm)	M_y (KN.m)
1b	8.3	4158	4158	149691	149691	149691	147.5
2b	8.7	4342	4342	156305	156305	156305	
3b	8.7	4341	4341	156259	156259	156259	
4b	7.4	3713	3713	133653	133653	133653	
5b	7.3	3637	3637	130920	130920	130920	
6b	8.8	4387	4387	157942	157942	157942	

3.4 MONOTONIC TESTS

Monotonic tests are performed in general to characterize the mechanical properties of a material, a component made of this material or a composite of different materials. In this project, monotonic tests were performed on the dowel-type joints in order to estimate the capacity of the joint and to obtain the elastic displacement in order to judge on the contribution of mechanical fasteners and reinforcement in increasing the capacity and ductility of a timber connection. The loading was applied in compression, parallel to the grain direction and in an increasing pattern until the ultimate strength of the connection is reached, following the specifications stated in EN 26891:1991 for determination of strength and deformation characteristics.

Dowel-type joints usually experience initial displacement before subjected to any load, thus, to exclude this effect, the loading is performed in two phases. The procedure provided in EN 26891:1991 defines these phases by means of 10% and 40% of the maximum force, which is estimated prior to testing, using the expressions given by EC5 [3] for timber-to-timber joints in double shear. Once the maximum load-carrying capacity of the joint is estimated, this allows to define the serviceability limit state which can be determined within this range of the maximum capacity, which is expressed by the slip modulus, K_{ser} . Detailed calculations of F_{est} are provided in 3.4.1.1.

The first phase comprises two steps, the first one is to start by applying 40% of the estimated maximum force, F_{est} , then to maintain the force for 30 seconds. The second step is to decrease the applied load to reach 10% of the estimated maximum force, F_{est} , then maintain the applied load once again for 30 seconds. The second phase is to increase the load until the failure or a displacement of 15 mm is reached, whichever occurs first. Figure 3:5.

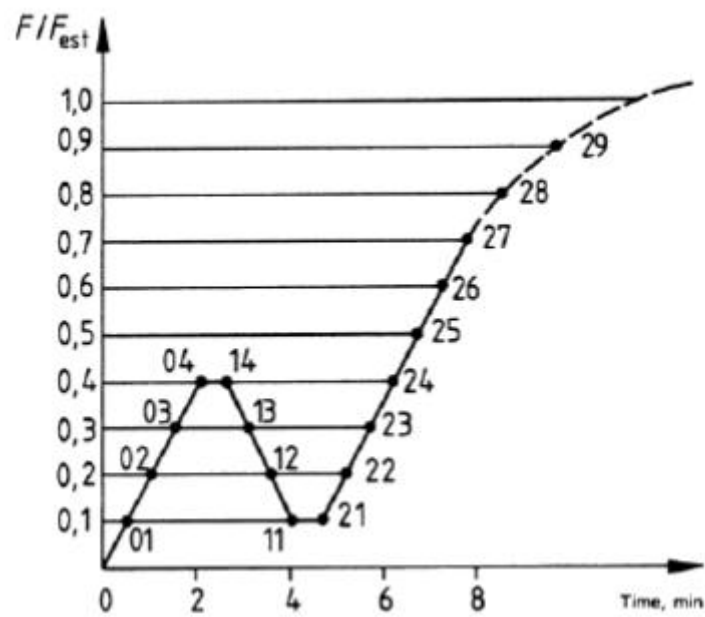


Figure 3:5 - Monotonic test procedure according to EN 26891:1991.

In order to control the procedure as described above, the test was performed under both force and displacement control. The first phase was carried out under force control with a rate of 0.058 kN/s based on an estimated maximum force F_{est} , equal to 20 kN. Then for the second phase, the test was performed under displacement control with a rate of 0.025 mm/s. The samples were placed on an electro-mechanic test machine, and the load was applied via an actuator of a 150 kN load cell, in addition to two linear variable differential transformers (LVDTs) fixed on both of the side elements of to measure the joint's slip. The readings were recorded using Dynatester software, and the load-slip curves were obtained accordingly. Figure 3:6 shows the test apparatus and the setup for the joints.

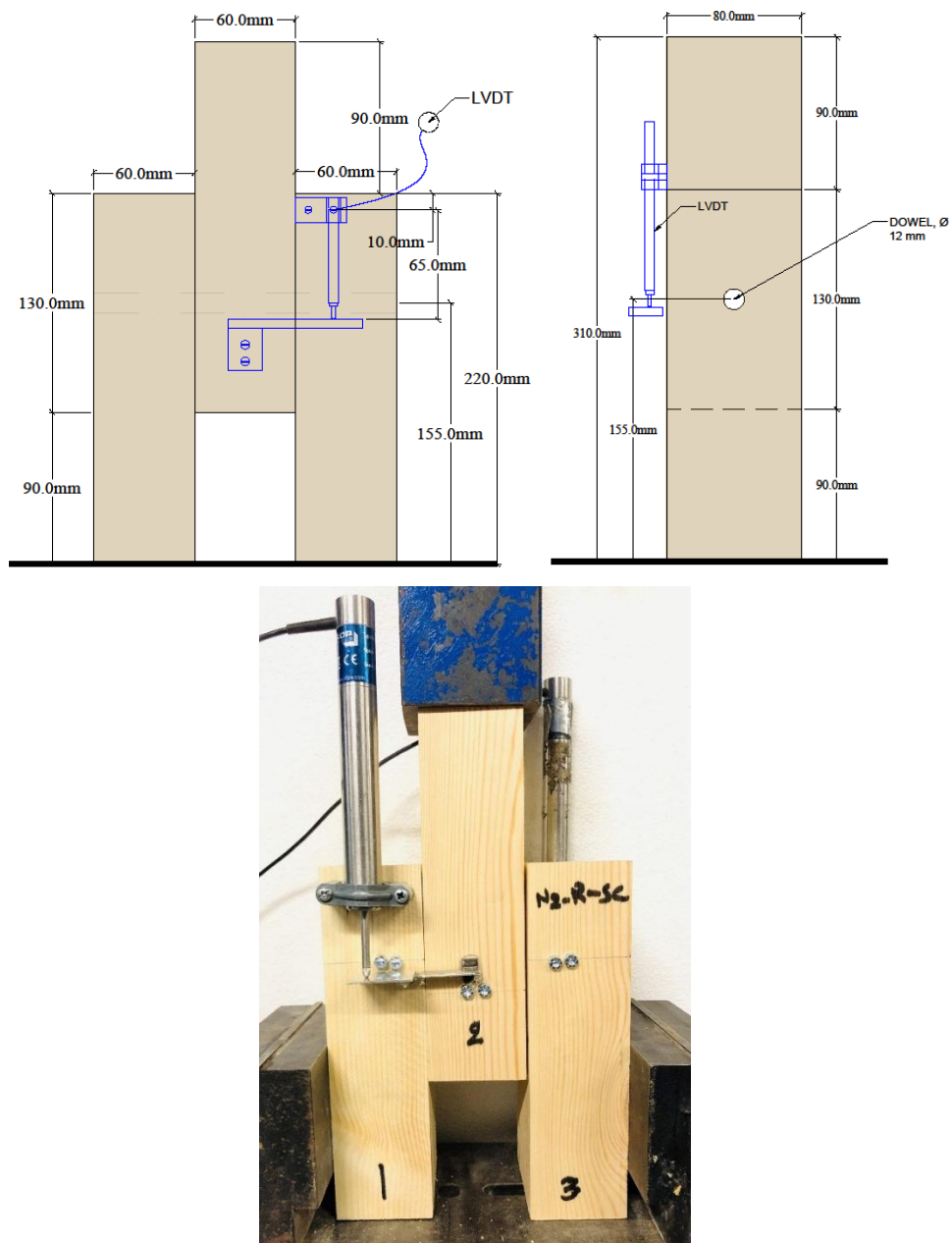


Figure 3:6 - Monotonic test apparatus.

3.4.1 SCOTS PINE

3.4.1.1 ORIGINAL JOINTS

Prior to testing and in order to set the test protocol, the maximum load-carrying capacity of the joint has to be determined. EC5 ^[3] has specified four equations that corresponds to each of the four failure modes proposed by Johansen, with regards to the European Yield Model (EYM). The load-carrying capacity of the joint is then taken as the minimum of these values, and the failure mode that correspond to that equation is the mode that is expected to occur in the joint after testing. Keeping in mind that the actual

value of maximum force reached by the joint after testing is ought to be higher than the value obtained by the specific equation that described the expected failure mode. The expressions to calculate the load-carrying capacity of the non-reinforced dowel-type joint are as follows:

$$F_{v,RK} = f_{h,1,k} t_1 \quad (3-2)$$

$$F_{v,RK} = 0.5 f_{h,2,k} t_2 d \quad (3-3)$$

$$F_{v,RK} = 1.05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,RK}}{f_{h,1,k} t_1^2 d}} - \beta \right] + \frac{F_{ax,RK}}{4} \quad (3-4)$$

$$F_{v,RK} = 1.15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,RK} f_{h,1,k} d} + \frac{F_{ax,RK}}{4} \quad (3-5)$$

Since the load-carrying capacity depends on the yield moment of the dowel and the embedding strength of the wood, the yield moment value was obtained from testing as illustrated above, and the embedding strength value is obtained with the following expression for predrilled holes ^[3]:

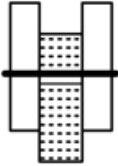
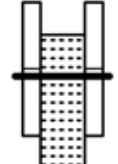
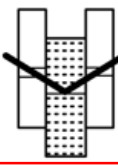
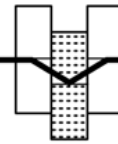
$$f_{h,k} = 0.082 (1 - 0.01d) \rho_k = 34.06 \text{ N/mm}^2 \quad (3-6)$$

The density of the specimen, ρ_k , was obtained through conducting a density test that was carried out for the specimens after monotonic and cyclic testing. The procedure is elaborated in section 5.1, and the final value obtained and used for calculations is 471.96 Kg/m³. The three elements that forms every connection are of the same species with the same density, thus the same embedding strength. Hence, the ratio between the embedment strength of the elements, β , is equal to 1. The thickness is 60 mm for all three elements, thus, $t_1 = t_2 = 60\text{mm}$.

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \quad (3-7)$$

The equations above allow for the consideration of the rope effect of the mechanical fasteners by adding to the equation the characteristic axial withdrawal capacity of the fastener, $F_{ax,RK}$, but since the rope effect for the dowels is 0% as specified in EC5^[3] and shown in Figure 2:2, then this part of the equation was omitted from the calculations of the load-carrying capacity. The values calculated for each failure mode for both shear planes are shown in Table 3-3.

Table 3-3 - Failure modes according to EC5 and associated load-carrying capacity values for the original joints - 12 mm dowels.

Failure Mode ^[3]	$F_{v,RK}$ (KN)
g 	49.04
h 	24.52
j 	21.39
k 	23.56

From the values obtained above, according to Johansen's theory, the load-carrying capacity of the original joints is the minimum of the values above, 21.39 KN therefore, the failure mode that is expected to occur in the joints is the third mode, mode (j) ^[3].

A total of 6 joints of Scots pine were formed following the geometry described above to be tested first as original connections before the introduction of any reinforcement. The tests were performed following the procedure described above. The average maximum force that was reached in the joints was 18.2 kN. The joints reached the maximum force around approximately 15 mm displacement. Nevertheless, the tests were continued until a slip of 30 mm is reached. Continuing the tests has allowed to clearly identify the failure mode for the joints based on their geometry, yield properties of the dowel, and the embedding strength of the wood itself. The test was continued to reach a slip of 30 mm based on the procedure described in the new proposal for EN 12512. The standard is originally for cyclic testing, but the monotonic procedure that is based on which the cyclic protocol is determined, specifies that the

monotonic ultimate displacement is set to either failure of the joint, 30 mm displacement or 80% of the maximum estimated force, whichever occurs first.

The failure mode for the tested connections found is failure mode (j) according to the classification of EC5 [3]. Figure 3:7. This failure mode coincides with the mode that is obtained from calculations of the estimated load-carrying capacity of the connections that is detailed above, mode (j), yet the value of the maximum capacity is less than the value obtained from calculations.

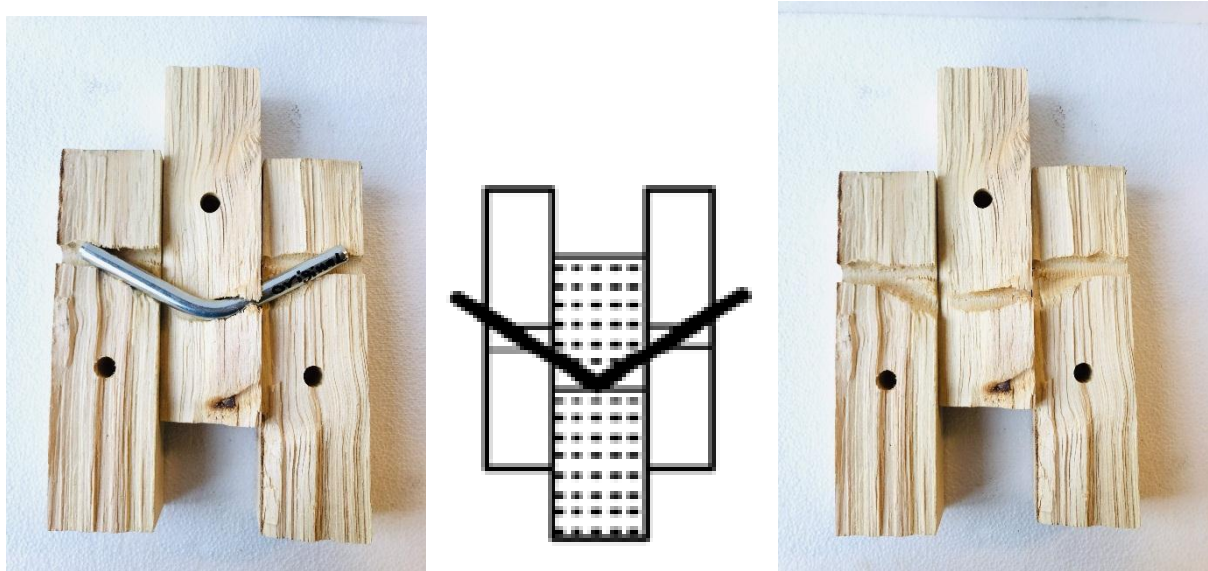


Figure 3:7 - Left: Original joint after testing, Middle: Failure mode (j) of EC5 [3], Right: Dowel embedment

Two of the tested joints experienced a brittle failure, splitting occurring after testing and the joints failed in embedment Figure 3:8. This brittle failure indicated less ductility for the joint as it can be seen from the force-slip curve of the first joint, J1_O_SC, in comparison to the other joints as shown in Figure 3:8 below. As for the rest of the joints, and as it is shown in Figure 3:7 above, the connections failed by the embedment of the dowel in the surrounding wood. Figure 3:9 shows the full force-slip curve of the joint in Figure 3:7, which is joint (4), for the purpose of portraying the application of the full monotonic procedure described above, and Figure 3:10 shows the force-slip curves of the 6 tested joints after processing, which were used later to obtain the monotonic equivalent energy elastic-plastic (EEEP) curves. Based on the force-slip curves obtained from monotonic tests it can be assumed as a preliminary step of the analysis that the failure is rather ductile than brittle.

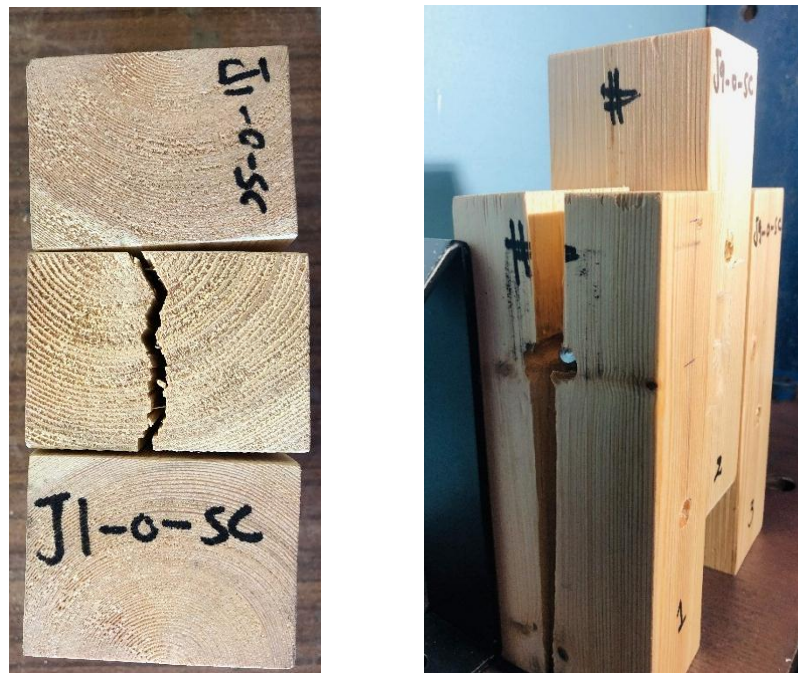


Figure 3:8 - : Left: Failure of joint (1) by splitting in the middle element, Right: Failure in joint (9) by splitting in the side element.

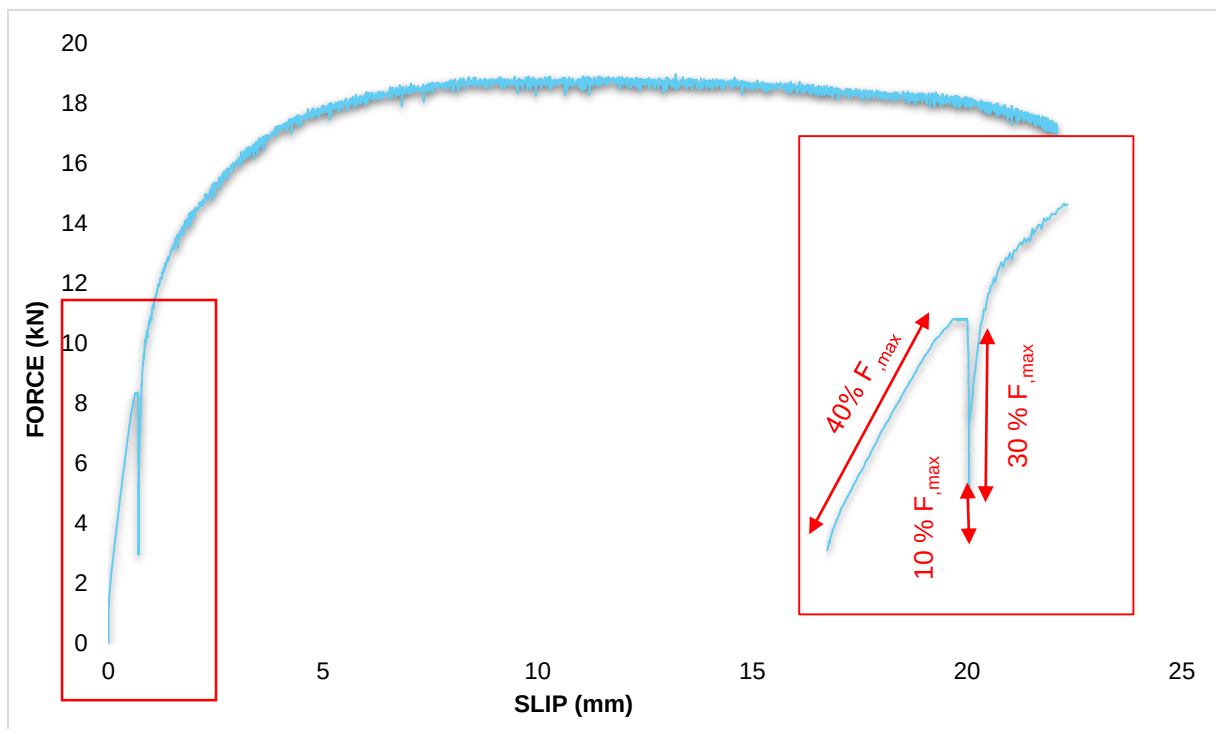


Figure 3:9 - Left: Full force-slip curve of joint (4).

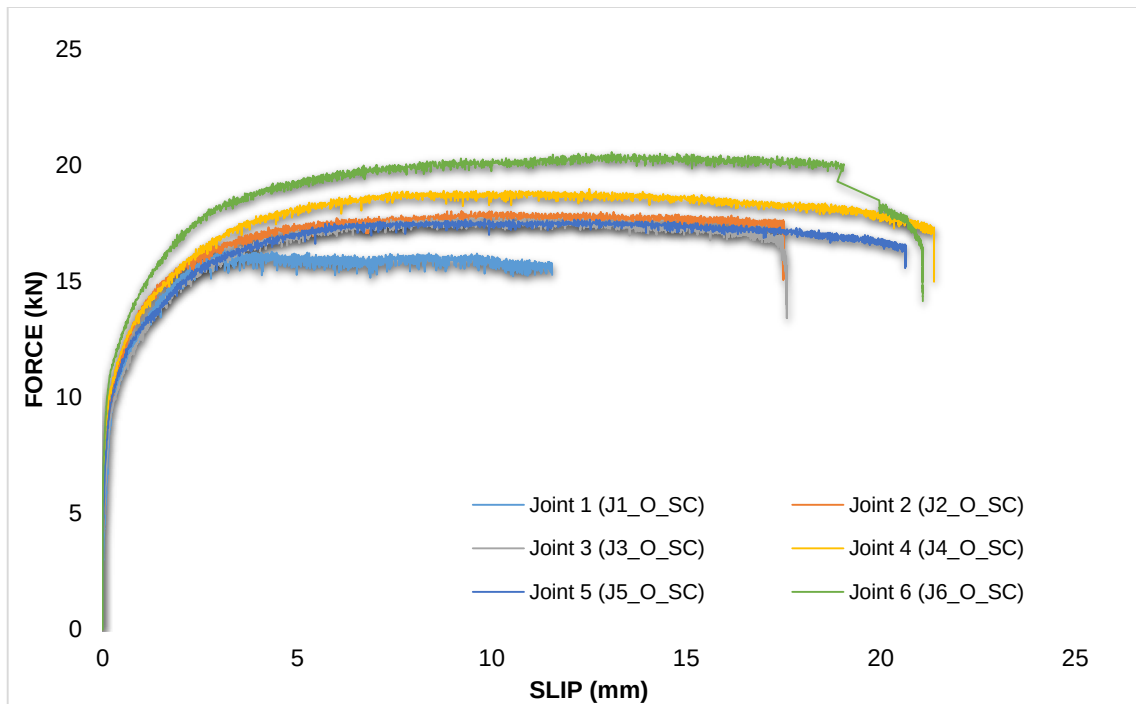


Figure 3:10 - Force-slip curves of the 6 original joints after excluding the initial displacement.

As mentioned above, the expressions to calculate the load-carrying capacity for the dowel-type joints that are provided in EC5^[3], are based on the European Yield Model (EYM) and the work proposed by Johansen, which was developed assuming a certain level of slenderness of the dowels. In other words, the equations were developed based on the assumption of rigid-plastic behaviour of the dowels under bending moment, and the wood under embedding stresses. Figure 3:11.

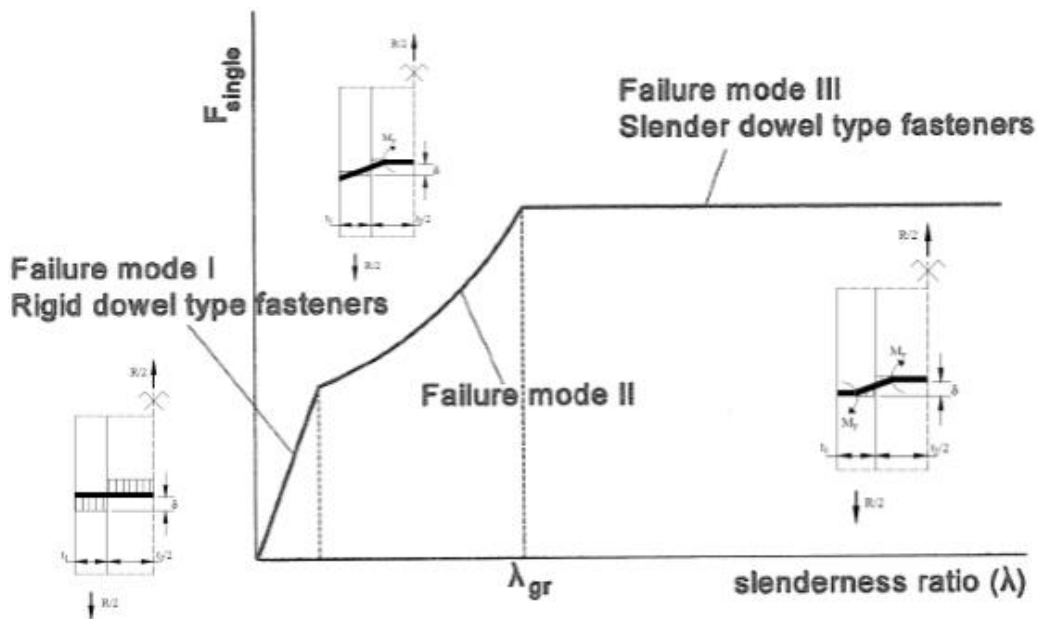


Figure 3:11 - Failure modes according to European Yield Model (EYM).

Giving that the dowels used in this experimental campaign are 12 mm in diameter, and 170 mm length, it was not possible to detect this ideal rigid-plastic behaviour that corresponds to mode (K) ^[3], which requires the dowels to behave plastically and deform by developing three plastic hinges. The strength of the joint was governed by the stiffness of the dowel, therefore, the connections failed before the additional two plastic hinges are formed. Consequently, the maximum force reached under monotonic loading in most of the tested joints was less than the force estimated by equations (3-2) to (3-4) that predicts the capacity of the joint based on the dowel's yield moment and the wood's embedding strength, despite the fact that the failure mode was the same as the mode acquired from the Johansen's theory which is mode (j). Table 2-1, shows the maximum forces of the tested joints.

Table 3-4 - Maximum force values obtained by testing.

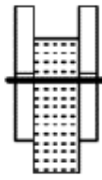
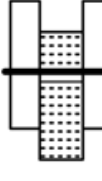
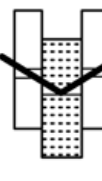
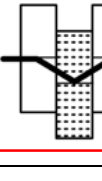
ORIGINAL JOINTS	
Joint Tag	F _{max} (KN)
J1_O_SC	16.33
J2_O_SC	18.03
J3_O_SC	17.71
J4_O_SC	18.98
J5_O_SC	17.68
J6_O_SC	20.57

Following these results, in order to conclude on the influence of the dowels' slenderness in the overall capacity of the joints, it was necessary to test the connections with dowels that have smaller diameter hence, higher tendency to behave more plastically. Thus, another 6 joints of scots pine were formed with dowels 8mm x180 mm diameter and length, respectively. The dowels are of class S235. The maximum load-carrying capacity of these joints was again estimated before testing using the same expressions from EC5 ^[3], detailed above. The yield moment of the new dowels, M_y , was calculated using (3-7), assuming a tensile strength, $f_{u,k}$, of 360 N/mm².

$$M_y = 0.3f_{u,k}d^{2.6} \quad (3-8)$$

The density of the samples, ρ_k , after testing was acquired as explained in section 5.1, and the value is 433.31 Kg/m³. The embedding strength, $f_{h,k}$, was acquired accordingly using (3-6), and the value obtained is 32.69 N/mm². Thus, the estimated load carrying capacity of the joints with 8mm dowels for the four failure modes are shown in Table 3-5 below.

Table 3-5 - Failure modes according to EC5 and associated load-carrying capacity values for the original joints – 8 mm dowels.

Failure Mode ^[3]	$F_{v,RK}$ (KN)
g 	31.38
h 	15.69
j 	11.81
k 	8.16

Based on the results of the calculations, the estimated load-carrying capacity corresponds to the fourth failure mode (k), which is a further proof that the slenderness of the dowels is a key factor in determining the failure mode, load-carrying capacity and ductility of the connection. The test protocol for the new joints was set based on this value of estimated maximum capacity, 8.16 KN. The tests were performed following the same procedure described above. However, based on the value of the estimated capacity, a different rate for the first phase was defined. The first phase, which was force controlled, the application rate was 0.029 KN/s, followed by a rate of 0.025 mm/s for the displacement controlled second phase. The maximum force reached by each joint is shown in Table 2-1 and Figure 3:12 shows one of the joints after testing as well as the deformation of the dowel in comparison to the 12 mm dowel of the previous connections.

Table 3-6 - Failure modes according to EC5 and associated load-carrying capacity values for the original joints – 8 mm dowels.

NEW ORIGINAL JOINTS	
Joint Tag	F_{max} (KN)
N1_O_SC	11.96
N2_O_SC	11.96
N3_O_SC	11.67
N1_ORP_SC	12.45
N2_ORP_SC	11.36
N3_ORP_SC	12.37



Figure 3.12: Top left: New joint after testing with 8 mm dowel, Top right: dowel embedment into surrounding fibres, Bottom: 12mm and 8mm dowels after testing

It can be seen from the comparison of the two cases that the equations can only provide accurate prediction of the failure mode if the dowels were slender. Based on the Force-slenderness ratio curve in Figure 3.12, both the 3rd and the 4th modes, which corresponds to mode (j) and (k) in EC5 [3], lie on the ideal plastic phase of the curve that can only be obtained if the slenderness of the dowel is higher

than a certain point. This point is defined by the 3rd and 4th equations (3-4 & 3-5), and although the failure mode obtained for the joints with 12 mm dowels was in fact the third mode as it was estimated by equation (3-4), yet the maximum capacity of the joints haven't reached the calculated value that should correspond to that mode.

On the other hand, for the joints that were tested with the 8 mm dowels, the value of maximum load-carrying capacity estimated for the joints before testing was in fact less than the actual maximum capacity that the joints sustained with monotonic loading which indicates that the design is safe and the geometry of the connection with the dowel yield capacity are sufficient to sustain the applied load. However, the failure mode (k) that was obtained according to calculations, wasn't accurate as the connection has failed prior to fully developing the three plastic hinges on the dowel, therefore, prior to reaching the full plastic bending moment of the fastener. Figure 3:13 shows the force-slip curves for the joints tested with 8 mm dowels.

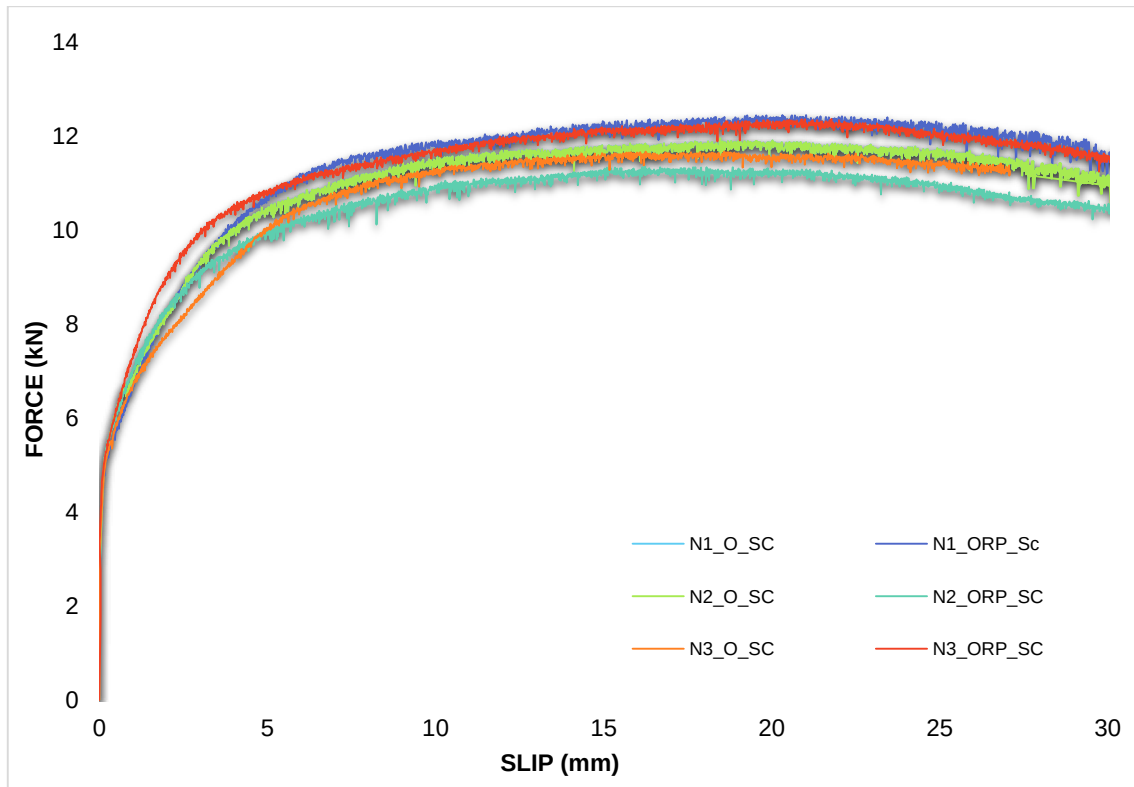


Figure 3:13: Force-slip curves of the 6 original joints with 8 mm dowels

Based on the governing failure mode of the tested original joints, the bending angle of the dowel, α , was below 45° , which is the bending angle at which the entire cross section of the fastener is assumed to be under plastic strain [18]. Since the angle acquired was less than this value, this means that the full plastic moment capacity of the dowel was not reached, thus, the plastic hinges on the dowel were not fully developed. Figure 3:14 shows the bending angle of both the 12mm and the 8mm dowels after testing, and as it can be noticed that the angles are much less than 45° . Thus, the dowels were only partially plasticized, which resulted in less load-carrying capacity.

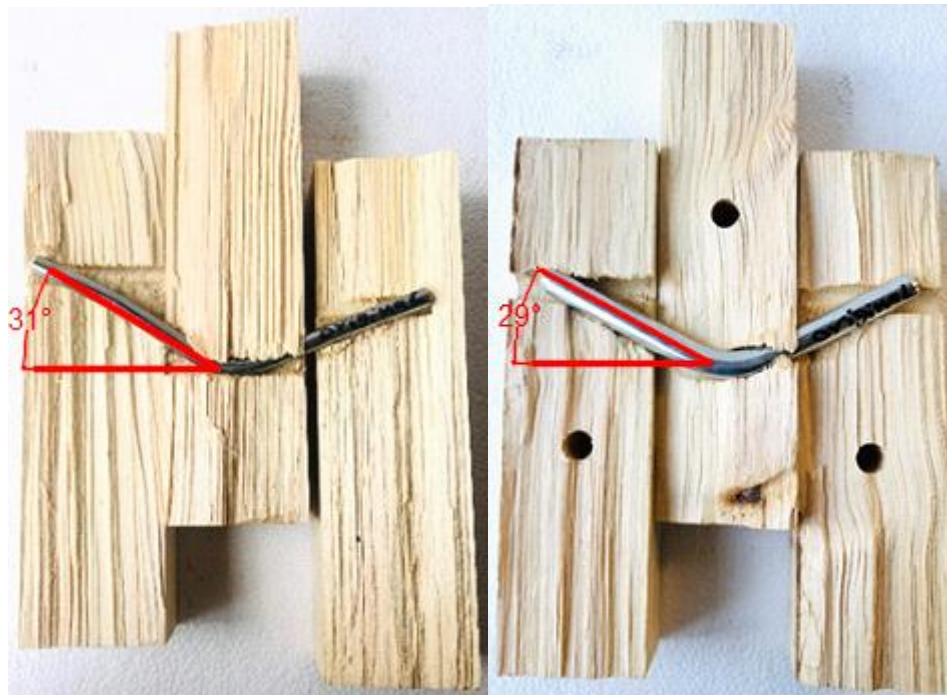


Figure 3:14: Left: Bending angle of the 8mm dowel, Right: Bending angle of 12mm dowel

3.4.1.2 REINFORCED JOINTS

Reinforcement is introduced to the dowel-type joints to increase the capacity and the ductility of the connections under loading. This can be achieved by preventing the splitting of the elements under tension forces perpendicular to the grain as well as shear forces generated in each shear plane. The adequacy of the reinforcement is detected by the ability to deviate these forces so that they are taken by the reinforcement instead of the timber elements, in addition to increasing the embedding strength of the wooden fibres, subsequently, increasing the overall ductility of the joint. In order to develop proper reinforcement techniques that can serve these purposes, it is essential to detect the failure mode of the original joint and based on which it can be judged on how to improve the failure of the joint to improve the capacity and increase the ductility.

As a first step in the process of reinforcement, 6 joints with the 12 mm dowel were assembled following the same geometry described above and 6 different reinforcement techniques were introduced to each of the joints. Two additional techniques were tested on 2 joints with 8 mm dowels in order to conclude on a reinforcement approach that provides the highest increase in the joint's capacity. Based on the results of these 8 reinforcement schemes, one techniques was chosen later on and then tested on 3 joints to have a better judgement. These techniques were developed based on the governing failure mode that was previously detected on the non-reinforced original joints, and the behaviour and bending capacity of the dowels.

Two types of self-tapping screws were used for reinforcement, as well as for repairing of the joints that is detailed in the next section, 3.4.1.4. The two types of STS used are Rothoblass HBS 8mm x 80mm shear screws, and Rothoblass VGZ 5mm x 80mm fully threaded screws, manufactured in accordance with the provisions of ETA, using an automated manufacturing process ^[17]. The effective diameter of the screws is calculated according to EC5 ^[3], clause 8.7.1 for screws with outer thread diameter not equal to the shank diameter, in this case the effective diameter is:

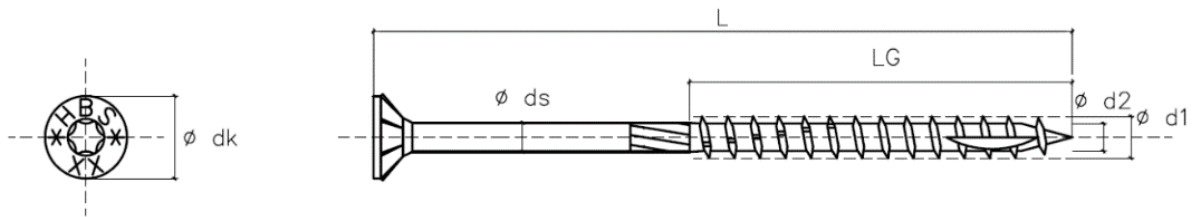
$$d_{ef} = 1.1d_2 \quad (3-9)$$

Regarding the threaded part diameter, for HBS screws, it coincides with the specifications of EC5 ^[3] that requires the threaded part to be $> 6d = 48$ mm, which in this case is fulfilled. The embedding strength of the screws, $f_{h,s}$, is calculated using equation (3-6). The load-carrying capacity of the screws, R_{VE} , is calculated based on the expressions developed by (Bejtka, 2005) ^[4], which are derived based on Johansen's yield theory:

$$R_{VE} = \min. \begin{cases} R_{A1} = f_{h,s} \cdot d \cdot l \\ R_{A2} = f_{h,s} \cdot d \cdot l \left[\sqrt{\frac{16M_{y,s}}{f_{h,s} \cdot d \cdot l^2} + 2} - 1 \right] \\ R_{A3} = 4 \sqrt{M_{y,s} \cdot f_{h,s} \cdot d} \end{cases} \quad (3-10)$$

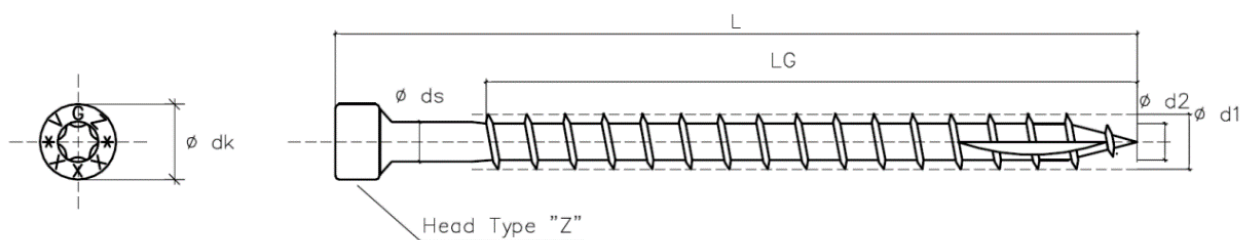
The yield moment of the screws, $M_{y,s}$, and it was calculated using equation (3-8). Tables 3.7 and 3.8 show the properties as well as the load-carrying capacity of the HBS and VGZ screws, respectively.

Table 3-7: Properties and load-carrying capacity of Rothoblass HBS screws



d1 (mm)	d2 (mm)	d _{ef} (mm)	dk (mm)	ds (mm)	L (mm)	LG (mm)	M _{y,s} (N.mm)	f _{h,s} (N/mm ²)	R _{A1} (N)	R _{A2} (N)	R _{A3} (N)	R _{VE} (N)
8	5.4	5.94	14.5	5.8	80	52	26743	35.60	22787	11277	11040	11040

Table 3-8: Properties and load-carrying capacity of Rothoblass VGZ screws



d1 (mm)	d2 (mm)	d _{ef} (mm)	dk (mm)	ds (mm)	L (mm)	LG (mm)	M _{y,s} (N.mm)	f _{h,s} (N/mm ²)	R _{A1} (N)	R _{A2} (N)	R _{A3} (N)	R _{VE} (N)
5	2.4	2.64	7.5	3	80	70	7930	36.77	14706	6645	4829	4829

Referring to the results of the original joints, with both 12 mm and 8 mm dowels, the reinforcement techniques were developed with the aim of increasing the dowels bending angle in order to change the deformation behaviour of the dowels so that three plastic hinges can fully develop within the connection. The 8 reinforcement schemes and their influence on the overall connection's capacity are described below. The main assumption for all schemes was that the STS are to be placed in direct contact with the dowels. However, in light of the difficulty of achieving such accuracy in practice, in addition to the risk of damaging the timber elements, a tolerance of 5 mm between the dowel and the screws was adopted.

3.4.1.2.1 SCHEME (1)

The first joint was reinforced by placing one HBS screw per element, a total of three screws in the joint. As discussed earlier in this report, the reinforcement shall be placed in direct contact with the dowels. According to Bejtka (2005) ^[4], the maximum increase in the load-carrying capacity can be obtained if the screws were placed in direct contact with the dowels, and in between the shear plane and the plastic hinge position. The plastic hinge position is then derived from the force and moment equilibrium in the shear plane, for timber-to-timber dowel-type joints with two plastic hinges per shear plane, which corresponds to failure mode (k). Figure 3:15. Thus in order to obtain failure mode (k) and increase the joint's capacity, the screw is ought to be placed accordingly. Since the failure mode obtained with the original joints was mode (j), and the fully developed plastic hinge has only occurred in the middle element, the position of the plastic hinge for the external elements was calculated following this theory with the expression below:

$$x_{1,3} = \frac{\sqrt{2 \cdot \beta}}{1 + \beta} \cdot \sqrt{(2 \cdot M_y) / (f_{h,1} \cdot d)} \quad (3-11)$$

$$x_{1,3} = \frac{\sqrt{2}}{\sqrt{\beta \cdot (1 + \beta)}} \cdot \sqrt{(2 \cdot M_y) / (f_{h,1} \cdot d)} \quad (3-12)$$

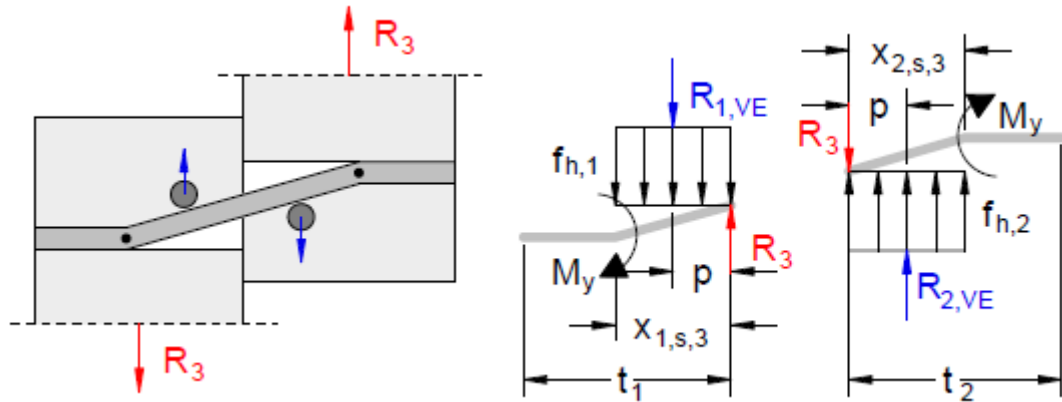


Figure 3:15: STS placed between the shear plane and the plastic hinge position, failure mode 'soft' ^[4]

Since the three elements of the joint are of the same wood and the embedding strength is equal in all of them, then, $X_{1,3} = X_{2,3} = 26.86$ mm. However, to test this assumption, for the first scheme, the STS were placed one per each of the three elements of the joint a distance, $P = 30$ mm, with P being the end spacing from the shear plane in a direction perpendicular to the grain. Figure 3:16

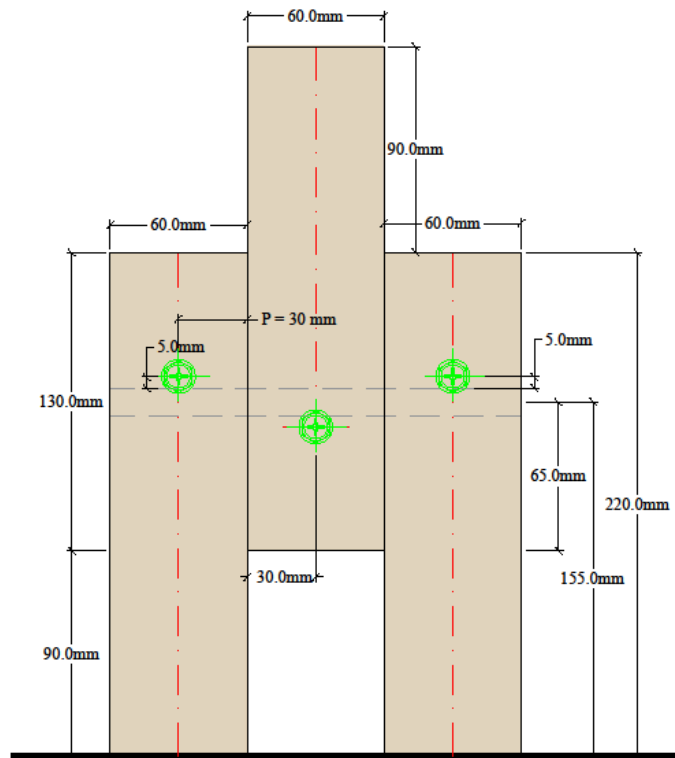


Figure 3:16: Reinforcement scheme (1)

The increase in the overall capacity of the joint is negligible, as the average maximum force of the original joints that is equal to 18.21 kN and the joint has only reached a maximum force, $F_{max} = 18.13$ kN with the reinforcement. As mentioned earlier in the original joints tests, the tests for the reinforced

joints as well was continued past 15 mm for the purpose of detecting the complete failure pattern of the joints and how this will be affected with and without reinforcement. Thus, for this joints the joint has sustained 21 mm of displacement before failure where the force has dropped down to 15 kN. The force-slip curve shows that the ductility has increased with reinforcement, although the capacity didn't increase as much. Figure 3:17 shows the joint after testing along with the resulting force-slip curve.

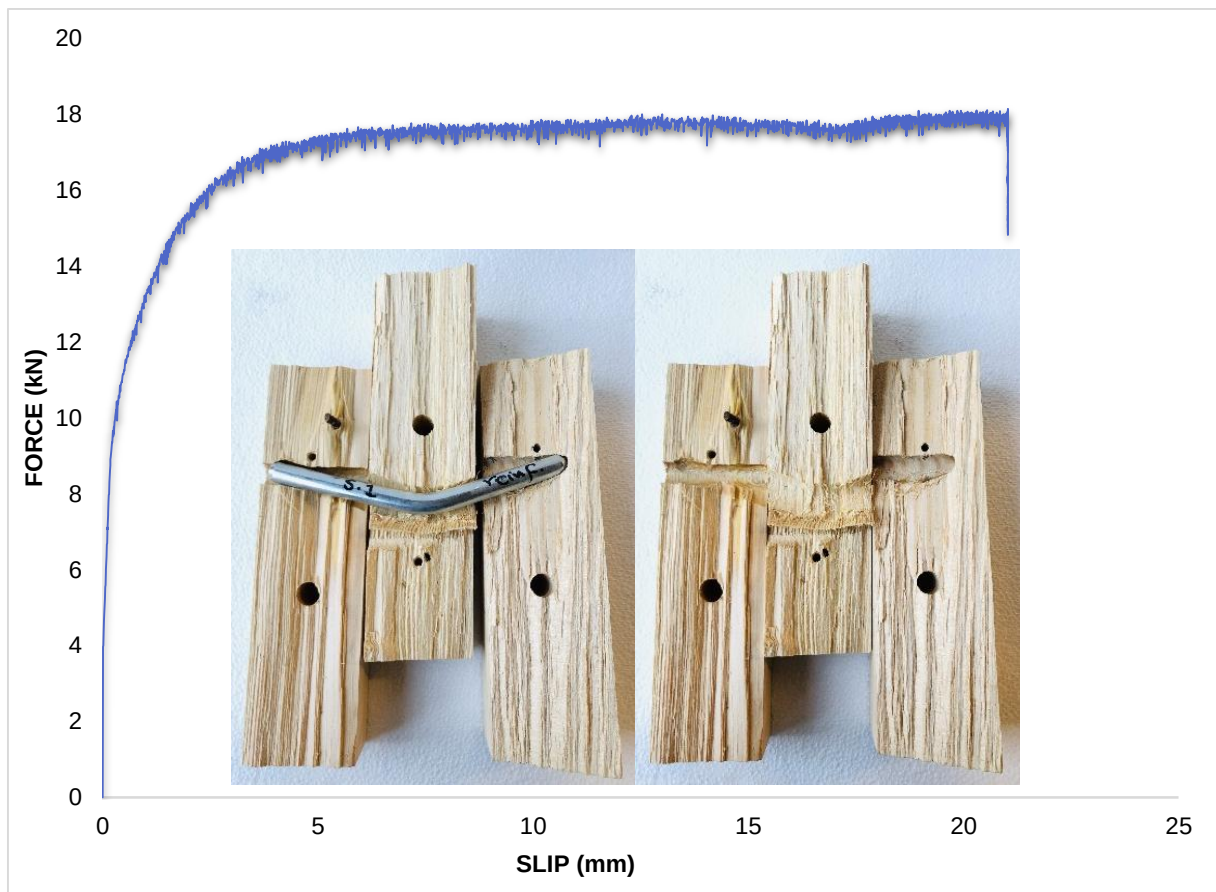


Figure 3:17: J1_R_SC tested with scheme (1), Failure and force-slip curve.

It can be noticed that the failure mode in this tested joint is again mode (j) with only one plastic hinge formed in the middle element. Hence, the full plastic moment capacity of the dowel was not reached, which explains the small increase in the overall load-carrying capacity of the joint.

3.4.1.2.2 SCHEME (2)

Based on the results obtained from scheme (1), it was concluded that the increase in the load-carrying capacity was not significant when STS were placed at a distance larger than the calculated plastic hinge position, $P > X$. Therefore, for the second reinforcement scheme, two HBS screws were positioned at both of the side elements of the joints, above and below the dowel in an attempt of forcing the dowel to rotate around the screws forming two plastic hinges. Figure 3:18 shows the draft for the scheme.

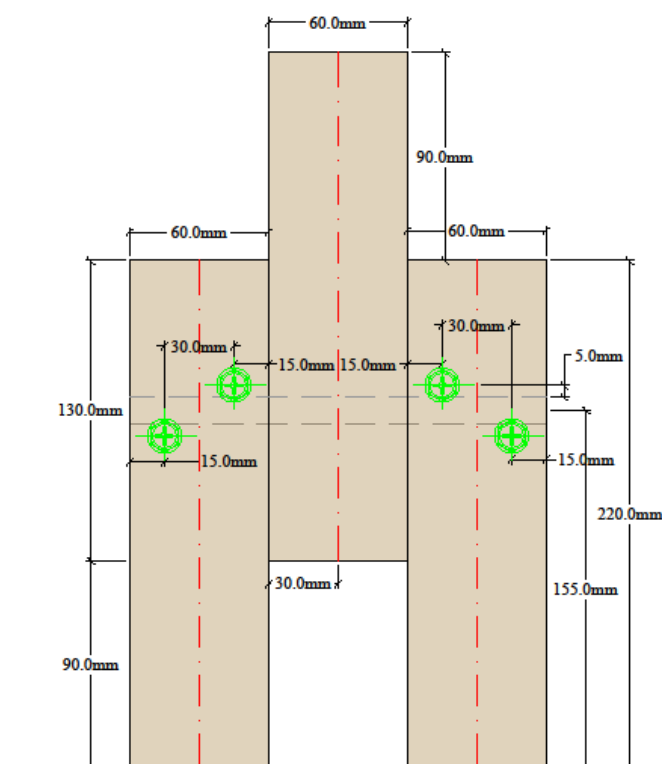


Figure 3:18: Reinforcement scheme (2)

The failure was recorded at a displacement of 12.58 mm, before reaching a maximum displacement of 15 mm. Resulting in a less ductile behavior, and the maximum force obtained, $F_{max} = 15.65$ kN is less than the average maximum capacity of the original joints. This failure mode obtained due to this reinforcement scheme is failure mode (h) ^[3], which corresponds to an embedding failure in the middle element. Figure 3:19 shows the force-slip curve along with the resulting deformation.

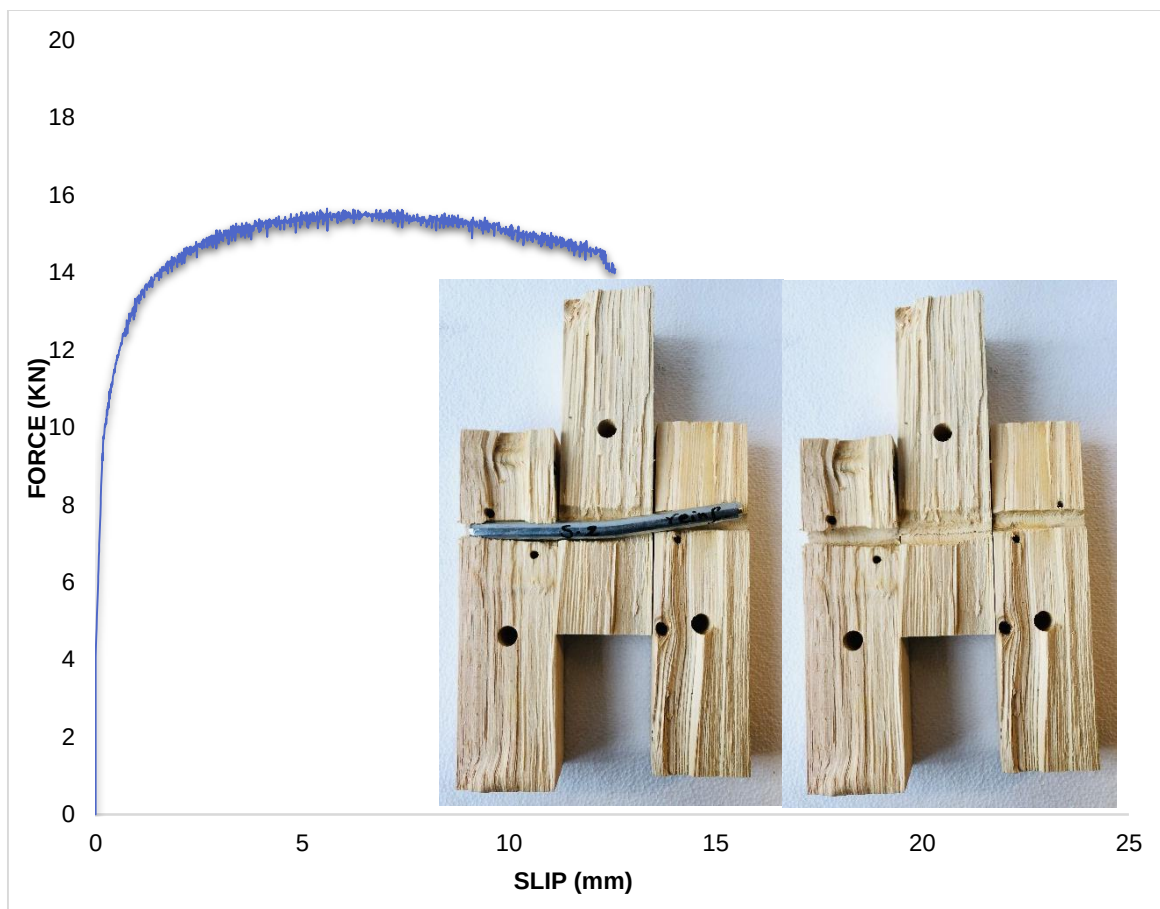


Figure 3:19: J2_R_SC tested with scheme (2), Failure and force-slip curve.

The screws in this scheme have generated higher stiffness for the joint, yet less ductility and overall capacity as it can be noticed in the force-slip curve above. The position of the two screws have restrained the deformation of the dowel allowing the timber elements to take a large amount of the generated forces, which lead to brittle failure and less capacity. Although the screw below the dowel at the side elements was placed between the shear plane and the plastic hinge position, at 15 mm, yet the lever arm created by the screw on top, with a distance of 30 mm between the two screws, has prevented the rotation of the dowel at the plastic hinge position resulting in a lower bending capacity of the dowel with the cross-section only partially plasticised.

3.4.1.2.3 SCHEME (3)

Since the positioning of STS below and above the dowel has prevented the development of the plastic hinges in all three elements, this scheme was developed with an aim of allowing for the dowel to deform freely by taking a larger portion of the forces, yet providing two rigid supports for the dowel at each of the side elements in order to develop two additional plastic hinges. Thus, the two HBS screws were placed 25 mm apart, above the dowel in each side element with the inner screw at 17.5 mm from the shear plane. Figure 3:20

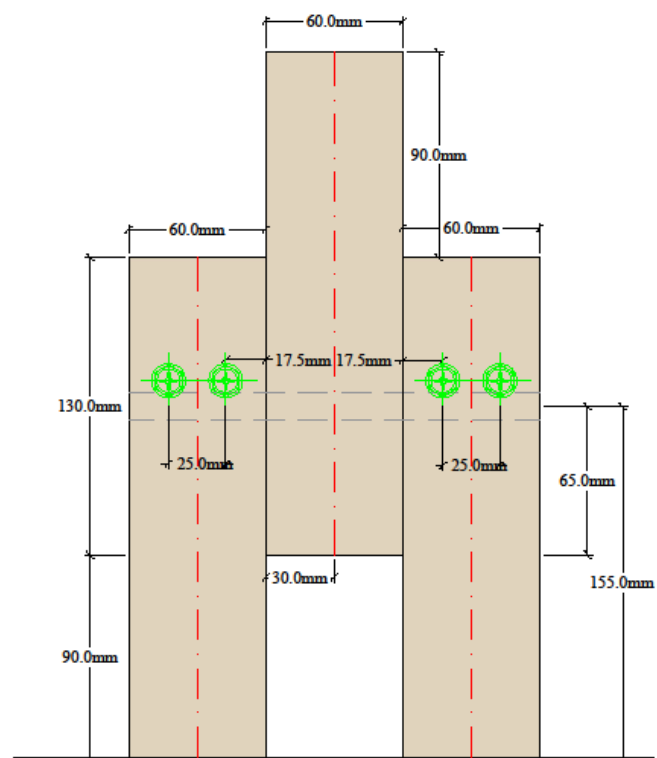


Figure 3:20: Reinforcement scheme (3)

The joint showed a slightly ductile behavior than the previous joint, and had sustained higher forces than the previous scheme yet the capacity of the joint has again increased by a small percentage from the average original, reaching a maximum force of 18.56 kN. Figure 3:21

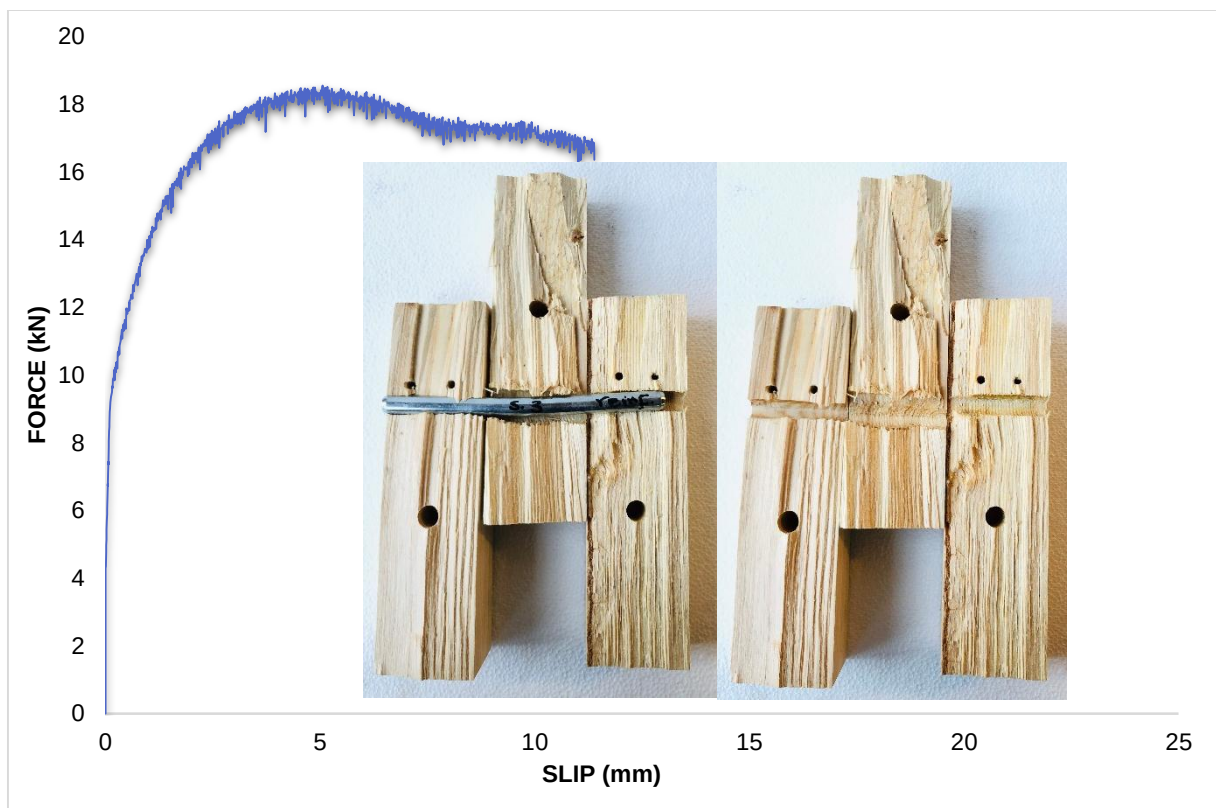


Figure 3:21: J3_R_SC tested with scheme (3), Failure and force-slip curve.

The joint failed due to embedment in the middle member, same as the previous joint, which is mode (h) [3]. It can be concluded with this result that the absence of reinforcement in the middle element has a direct influence on the formation of the plastic hinge in that element. Keeping in mind that the large diameter of the dowel has limited the possibility of reaching the full yield capacity of the entire dowel's cross-section before the failure of the joint. In other words, the low slenderness of the dowel has cancelled the effectiveness of the reinforcement, therefore, the capacity of the joints was governed by the stiffness of the dowel rather than the reinforcement.

3.4.1.2.4 SCHEME (4)

In light of the results obtained by scheme (2) and (3), reinforcing the middle element is rather important in terms of preventing brittle failure and increasing the dowel's plasticity. Nonetheless, regarding the diameter of the dowel and according to a parameter study performed by (Bejtka, 2005) [4], a significant increase in the load carrying capacity of dowel-type joints can be achieved with large dowel diameters in a case that the ratio between the diameter of the screw and the dowel's diameter, d_s/d , is between 0.35 – 0.40 %. The ratio between the dowels and the STS is shown in Table 3-9.

Table 3-9: Screw to dowel diameter ratio

	STS $\varnothing$ (mm)	Dowel's $\varnothing$ (mm)		Ratio %	
	ds	d1	d2	ds/d1	ds/d2
VGZ	5	12	8	0.42	0.63
HBS	8	12	8	0.67	1

The previous 3 joints were reinforced with HBS shear screws, thus according to this study ^[4], the ratio is 0.67 %, which is outside of the range specified. This could be a valid explanation to why the reinforcement has failed so far in increasing the joint's capacity. Therefore, to validate this theory, for this reinforcing scheme, VGZ screws were used instead in order to see if the ratio has in fact a significant influence on whether the reinforcement can be assumed effective or not, despite the fact that VGZ screws are fully threaded and are usually used for repairing purposes. Figure 3:22 shows the proposed scheme which is similar to the first technique but with VGZ screws.

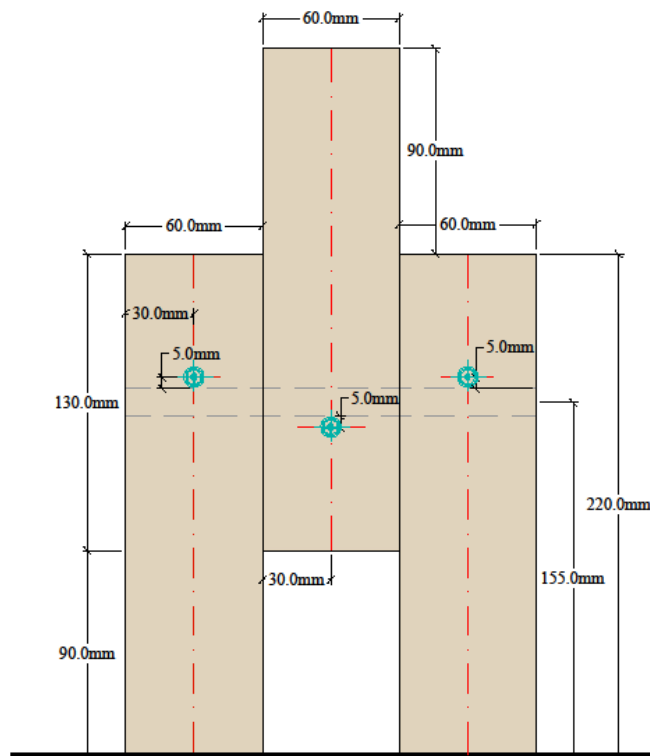


Figure 3:22: Reinforcement scheme (4)

The maximum force reached in this joint, $F_{max} = 17.73$ kN which is less than the average of the maximum force obtained by the original joints. Hence, this reinforcement technique is deemed ineffective in terms of improving the load-carrying capacity of the joint, yet the ductility of the joint has increased significantly compared to the previous schemes and to the original joints. The maximum force

was reached at approximately 11 mm of slip before the force started decreasing, followed by a plateau up to 30 mm without brittle failure. Figure 3:23

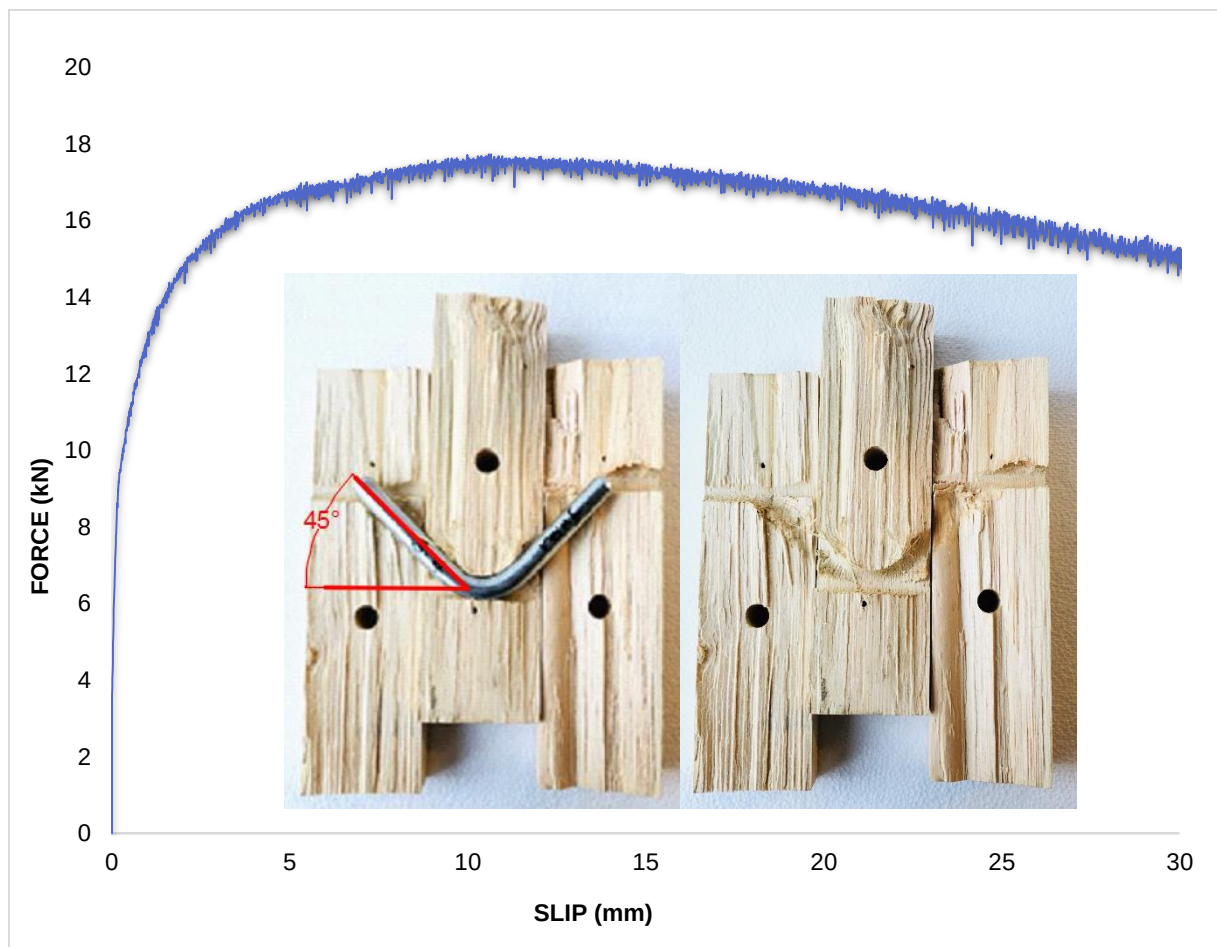


Figure 3:23: J4_R_SC tested with scheme (4), Failure and force-slip curve.

With this reinforcing scheme, failure mode (j) was obtained, which is similar to the original joints with the difference in ductility that is increased in this joint. As shown in Figure 3:23 above, the dowel's bending angle in this joint is 45° , which indicates that the full bending capacity of the dowel is reached. The fully threaded STS have significantly increased the embedding strength of the timber elements, consequently, the ductility, but failed to increase the overall capacity of the connection.

3.4.1.2.5 SCHEME (5)

Although the previous reinforcement scheme provided high ductility and embedment strength yet the main goal of reinforcement is to increase the load-carrying capacity of the joint. Thus, the reinforcement is continued with HBS shear screws. For this scheme, 3 screws, one per element were used to reinforce the joint. From the results of the 1st and 4th configurations, it was drawn that placing the screws at a larger distance from the shear plane than the plastic hinge, doesn't help improving the capacity. Therefore, in this configuration the screws in both side elements were placed at 15mm distance from the shear plane. Regarding the middle element, it was drawn from the 2nd and 3rd schemes that omitting

the reinforcement of the middle element causes embedment failure of the joint. Thus, one screw was placed at the center of the middle element. Figure 3:24

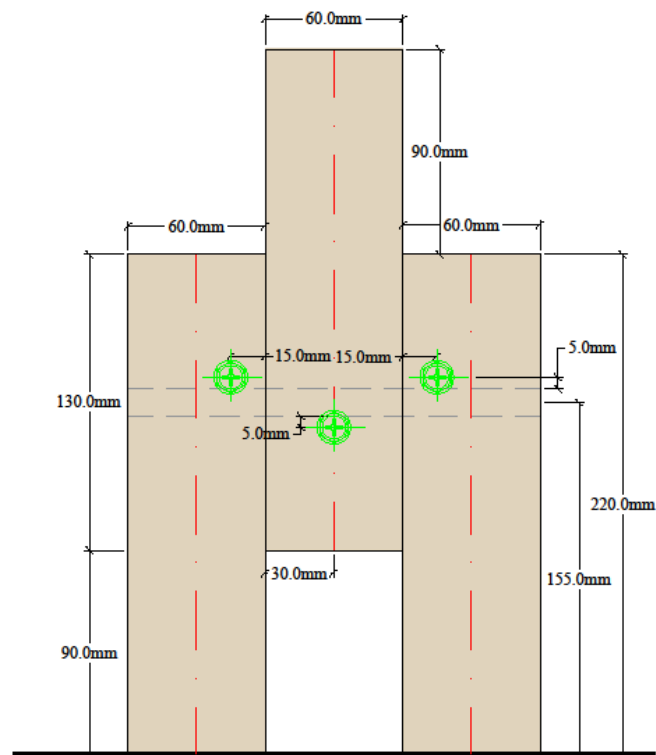


Figure 3:24: Reinforcement scheme (5)

The maximum force obtained with this scheme is 20.75 kN which is the highest that is obtained with reinforcement with an increase of 12.22 % of the average maximum force of the original joints. Figure 3:25 shows the failure and the force-slip curve.

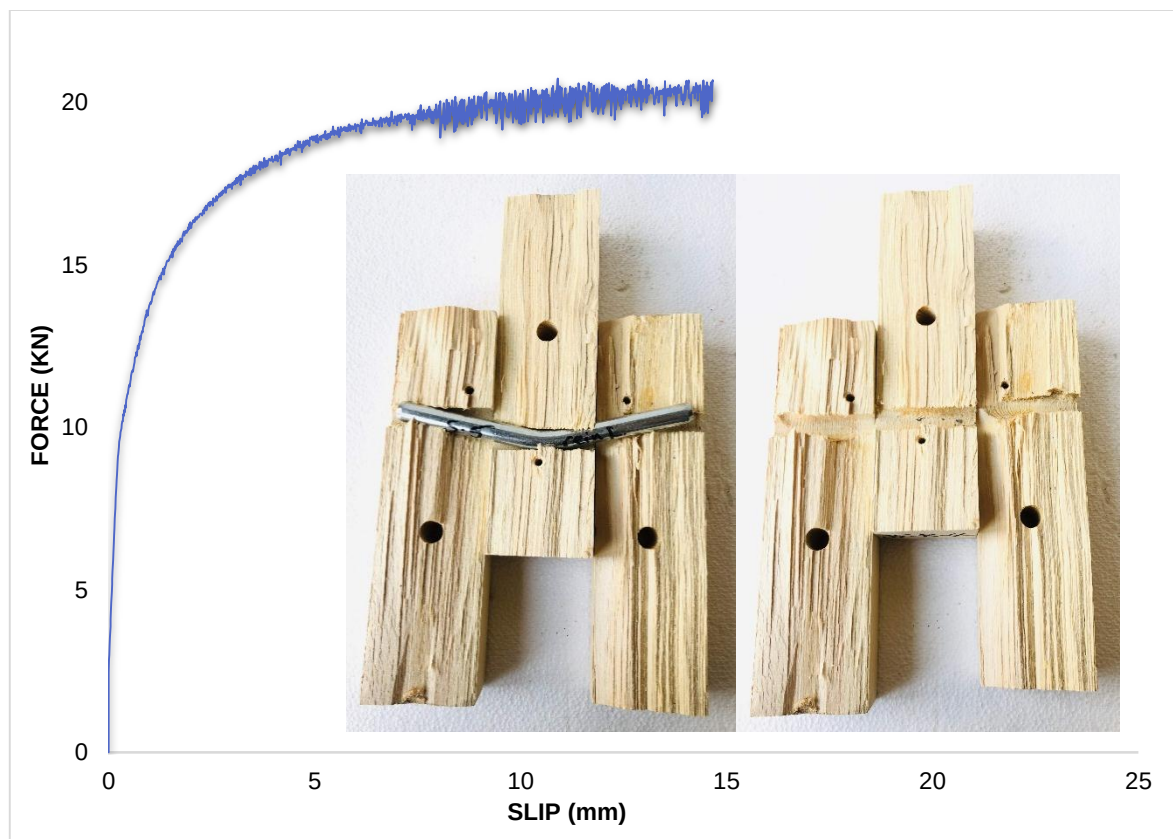


Figure 3:25: J5_R_SC tested with scheme (5), Failure and force-slip curve.

It can be noticed from the force-slip curve that the joint has higher stiffness and capacity than the previous joint, and less ductility and embedment strength. The joint failure mode corresponds to mode (g), which is an embedment failure on the side elements [3]. It is noticed from the reinforcement schemes applied so far that if the STS is placed closer to the shear plane, the capacity is increased but the embedment strength is much less, and the joint fails before the dowel has reached its full yield capacity.

3.4.1.2.6 SCHEME (6)

Since the capacity tends to increase with the reinforcement being closer to the shear plane as mentioned above, yet the improvement is minimum, two screws were added instead of one. So far 3 of the 4 failure modes proposed by EC5 [3] have occurred with the reinforcement schemes above. However, to achieve the 4th failure mode and to further increase the capacity, the number of screws in the connection is ought to be increased. However, the position of the reinforcement was based on the results obtained from the previous schemes.

Regarding the two side elements, with one placed close to the shear plane, and the second is at a distance further than the plastic hinge position and at more than $3\varnothing_s$ from the first screw, the forces were taken by the timber elements in a form of embedment failure, and the dowel's plastic moment capacity was not attained. The proposal is to place the screws as close as possible to one another to prevent the full restraining of the dowel rotation. Therefore, the spacing in the 6th configuration was set

to $1.5\varnothing$ s, with $\varnothing$ s indicating the screw diameter, in order to take into consideration the diameter of HBS screws which is 8mm. Nevertheless, due to the dimension of the screw head, the screws had to be placed on opposite sides on each element in order to facilitate the drilling process without destroying the wood fibers. Figure 3:26

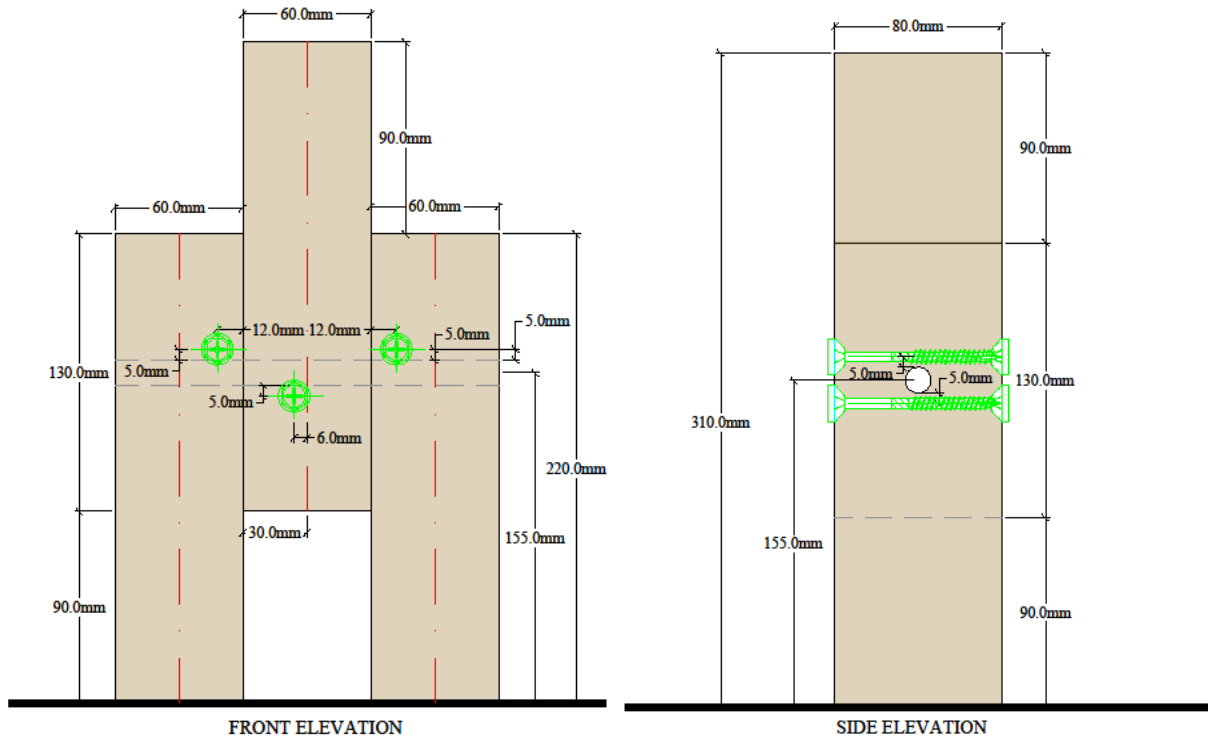


Figure 3:26: Reinforcement scheme (6), Left: Front elevation, Right: Side elevation

The maximum force obtained is 19.36 KN. The joint has less stiffness but more ductility, which is the opposite of the case of scheme (5). Although the capacity is slightly less, but the ductility has increased significantly, as well as the embedding strength. The force-slip curve, as well as the damage appeared on the joint are shown in Figure 3.27.

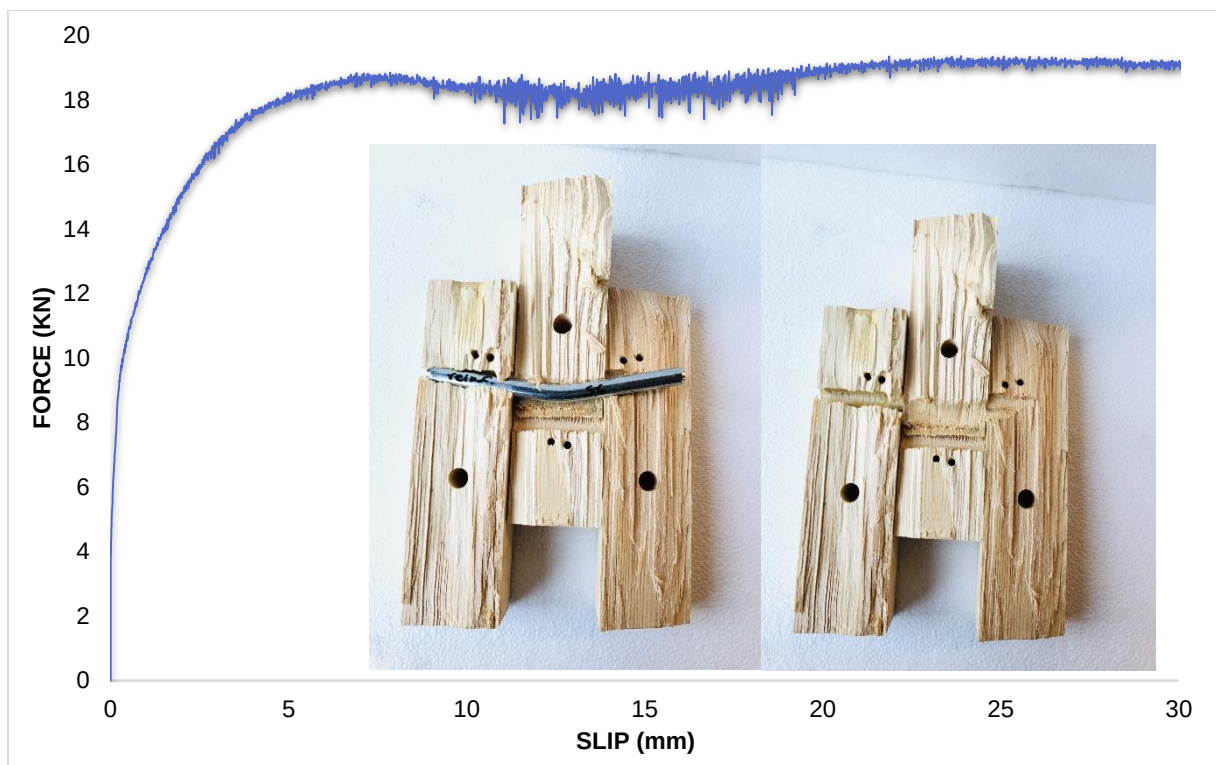


Figure 3:27: J6_R_SC tested with scheme (6), Failure and force-slip curve.

As shown in Figure 3:27 above, the failure mode obtained is mode (j), similar to scheme (5), but with higher embedment in the middle element which explains the higher ductility of the joint. The force has increased after it started decreasing. It reached a peak of 18.82 kN at only 7 mm slip, then it started decreasing before increasing once again to reach the maximum recorded force of 19.36 kN at a displacement of 23.89 mm. According to the standards, EN 26891:1991, the test should be terminated when a failure is reached or at 15 mm displacement, whichever occurs first. In this case the test was also continued, in light of this odd behaviour of the joint, according to the specifications for ultimate displacement for monotonic testing based on the new proposal for EN 12512, detailed above, in order to record when the ultimate failure is and what the failure mode with this scheme is. The increase in the force after decreasing, proceeded by a plateau up to 30 mm displacement before the total failure has occurred, could be a result of the contact between the dowel and the reinforcement that followed the dowel's deformation. With regards to the maximum force obtained with this scheme in comparison to scheme (5) despite that the number of screws is the double, can be a result of a natural defect in this joint. The presence of a knot at the top of the middle element could have an influence on the stiffness of the joint. Figure 3.28.

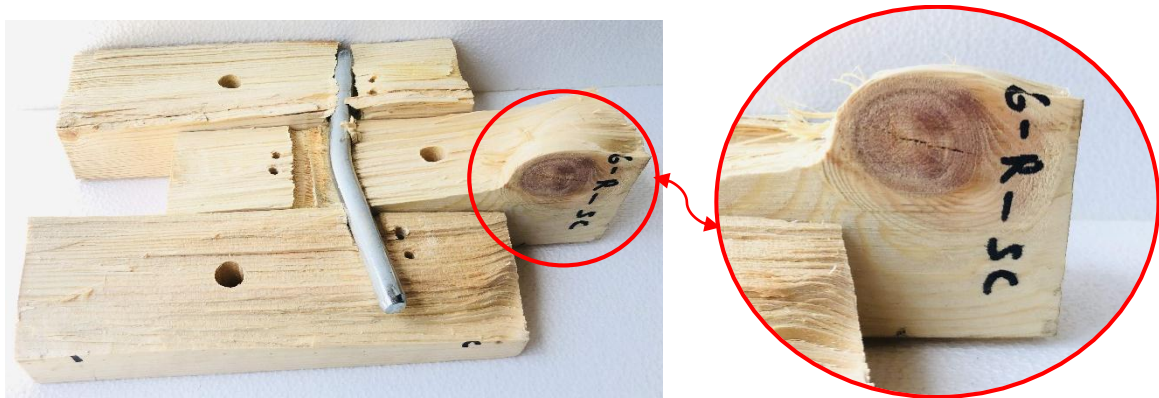


Figure 3:28: Presence of a knot in J6_R_SC tested with scheme (6).

3.4.1.2.7 SCHEME (7)

The previous schemes all showed different results and different failure modes, yet the aim is to obtain mode (k) ^[3] in order to increase the capacity of the joints significantly. So far with the previous schemes this was not possible and it is believed that the key issue is the slenderness of the dowel as previously stated. Therefore, to understand more in depth the relationship between the capacity of the joints and the slenderness of the dowel, this scheme was tested on the joints with 8mm dowel. The screws used for reinforcement are the fully threaded VGZ screws. These screws have failed to increase the capacity of the joints when used for reinforcement in the 4th scheme above, although the ratio of the screw diameter to the dowel diameter was slightly higher than the range specified in Bejtka (2005) ^[4] which is 0.3 – 0.4%. Nonetheless, the ductility was increased significantly. According to Table 3-9: Screw to dowel diameter ratio above, the ratio for this joint, with the 8mm dowel and the VGZ screws of a diameter equal to 5mm, is 0.67%, which is again higher than the specified range. Thus, this scheme was carried out to test this theory for the second time, however with a dowel of a different diameter and yield moment with increasing the number of screws per element in order to achieve better results. Figure 3:29 shows the proposal for scheme (7).

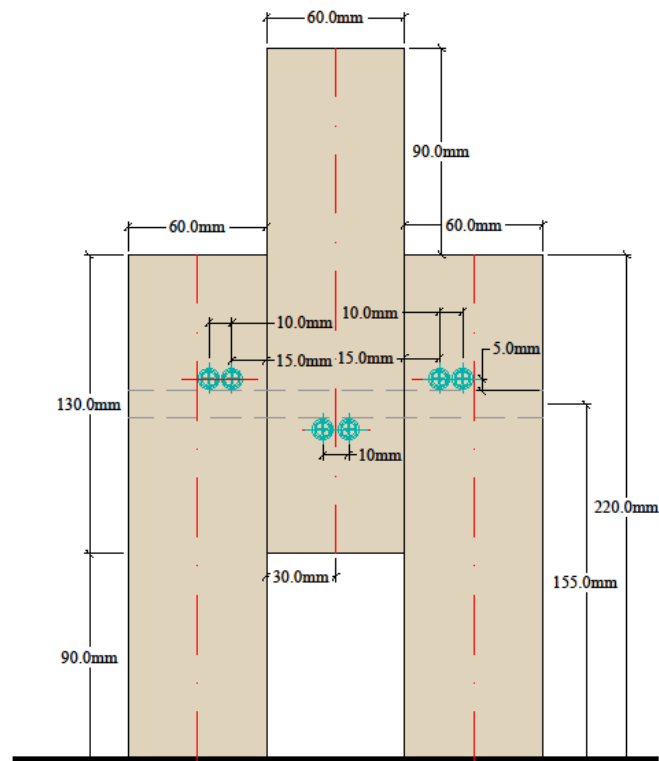


Figure 3:29: Reinforcement scheme (7)

The distance between the screws on a direction perpendicular to the grain is equal to $2d = 10$ mm, to allow for the screw head diameter. This scheme was tested on joint N2_R_SC, which has different density than the joints with 12mm dowel. The maximum force reached is 11.02 kN which is less than the average maximum force obtained by the original joints with 8 mm dowel, which corresponds to 11.96 kN. Figure 3:30 shows the failure of the joint and the force-slip curve obtained.

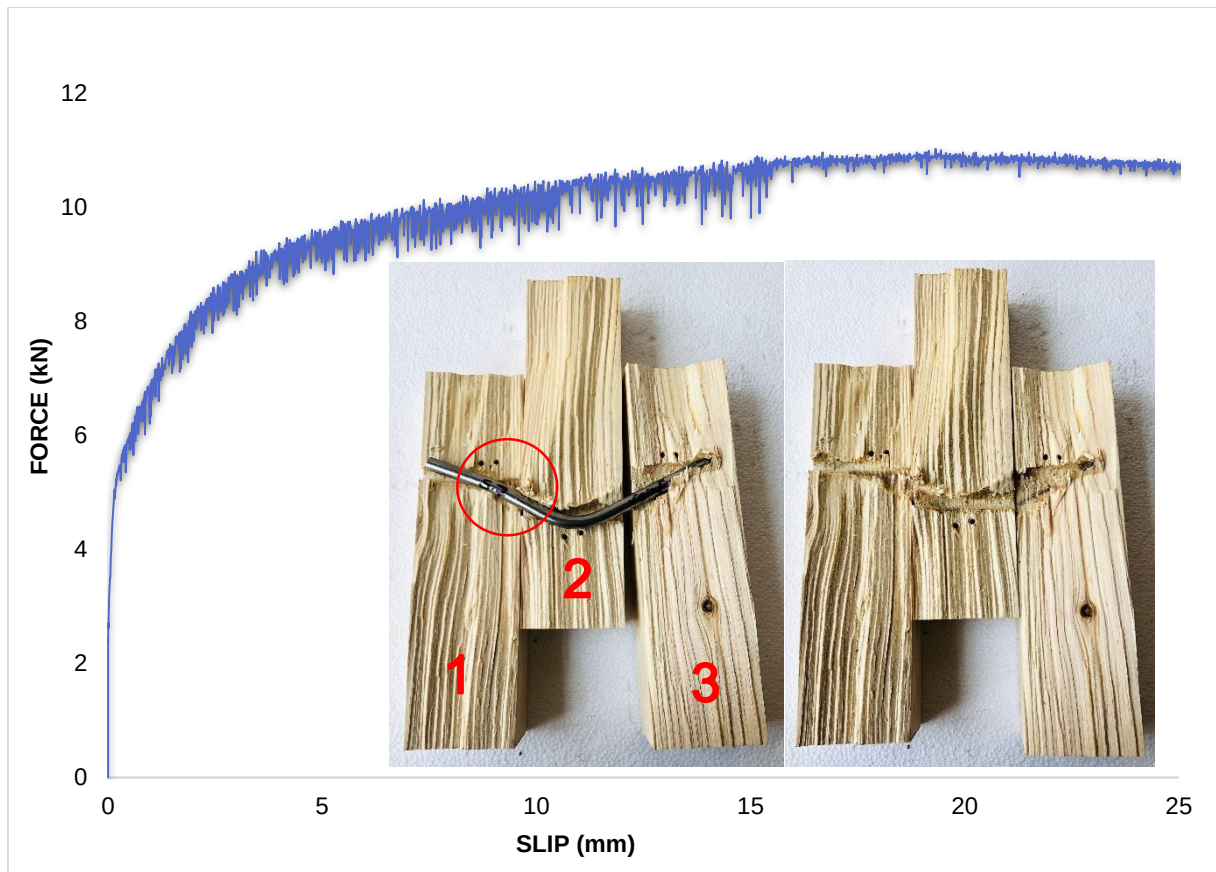


Figure 3:30: N2_R_SC tested with scheme (7), Failure and force-slip curve.

For this joint, and all the joints with 8 mm dowel, as it was mentioned earlier in this report, the embedding strength is different from the joints with 12 mm dowel, due to the difference in density, as well as the yield moment of the dowels. Therefore, the position of the plastic hinge in this case is different according to calculations with equation no (3-11 and 3-12). The plastic hinge position with respect to the shear plane, depends on the dowel's yield moment and its diameter. For the joints with 8 mm dowels, the position of the plastic hinge, $X_{1,3}$ and $X_{2,3}$, measured perpendicular to the grain from the shear plane, is equal to 13.57 mm. Thus the plastic hinge with this reinforcement scheme should form between the screw and the shear plane. As it can be noticed from the failure in Figure 3:30 above, that this condition was verified, however, only for one side element, element no. 1. Despite of the increased ductility and the additional plastic hinge, the capacity of the joints has not increased. Therefore, based on the results of this scheme and the results of scheme (4), VGZ fully threaded screws are not suitable for reinforcement purposes.

3.4.1.2.8 SCHEME (8)

The last reinforcement configuration within the experimental campaign in this project was developed based on the results obtained from the previous 7 configurations. Schemes (5) and (6) have provided the highest maximum force, with respect to the average obtained by the original joints, in comparison to

the other schemes. Looking at the results of the other reinforcement schemes, when omitting the reinforcement from the middle element, as in scheme (2) and (3), the stiffness of the joint increases but the failure is brittle and the timber fails on embedment, modes (g) and (h) [3]. On the other hand, when two screws were placed in the middle element, as in scheme (6), the capacity has increased, yet was slightly less than the capacity obtained by scheme (5) with one screw in the middle element. Another conclusion from scheme (1) and (4) that the screws are less effective if placed further than the plastic hinge position with respect to the shear plane in a direction perpendicular to the grain. Thus, this scheme was developed as a combination of the 5th and 6th schemes. Figure 3:31

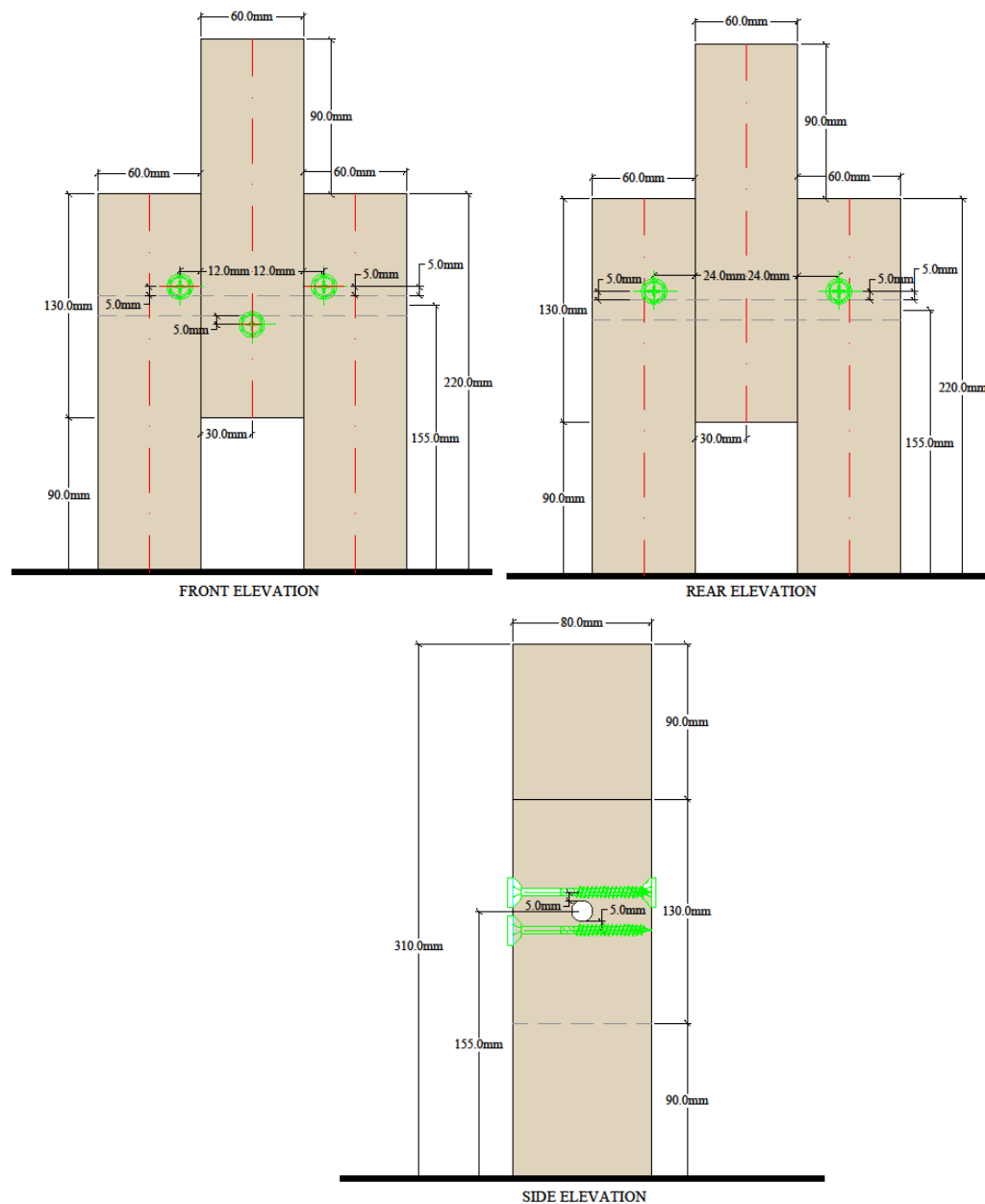


Figure 3:31: Reinforcement scheme (8), Left: Front elevation, Right: Rear elevation, Bottom: Side elevation

Two HBS screws in each of the side elements were installed at a distance of $1.5d = 12$ mm at opposite sides similar to scheme (6), and one screw in the middle element was placed, at the centre of the element, 30 mm, where the plastic hinge in previous test was developed. This scheme was tested on a joint with 8 mm dowel, N3_R_SC. The maximum force obtained is 11.88 N at 18.82 mm displacement. Figure 3.31.

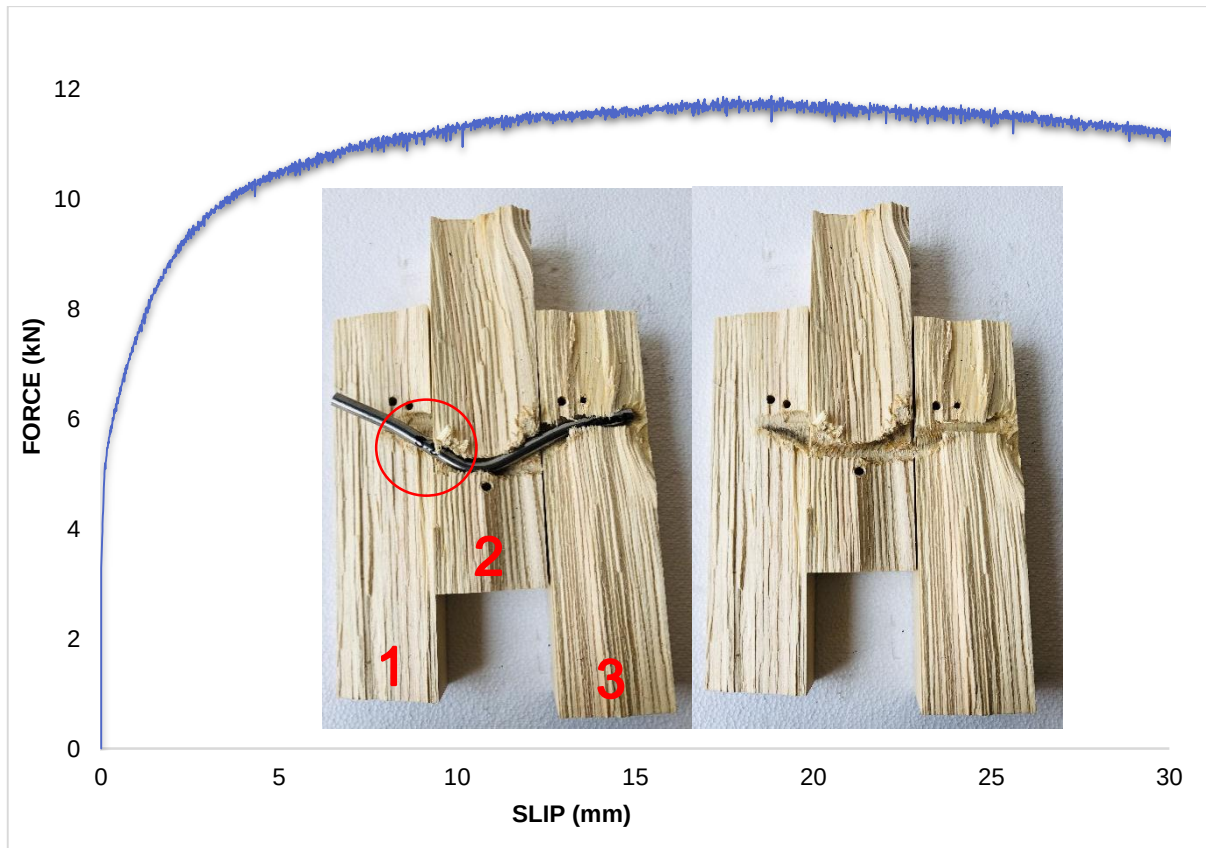


Figure 3.32: N3_R_SC tested with scheme (8), Failure and force-slip curve.

The failure in the joint with this scheme is mode (k) ^[3], which corresponds to 3 plastic hinges, one in each element, and it's the 3rd failure mode according to Johansen's and the EYM. Although the third hinge in element no. 1 in Figure 3.32 is not fully developed, yet this indicates that the key factor in achieving this failure pattern is the dowel's slenderness.

3.4.1.2.9 COMPARISON OF REINFORCEMENT SCHEMES

After testing the 3 different schemes on 3 different joints with 8 mm dowels, a comparison of the 3 joints, N1_R_SC, N2_R_SC and N3_R_SC, that were reinforced with schemes (5), (7) and (8), respectively, was carried out. Figure 3.33 shows the 3 joints with 8 mm dowels, and the dowels after monotonic loading and the failure modes associated with each scheme, and Figure 3.34 shows the dowels deformation and the plastic hinges formed in each joint.

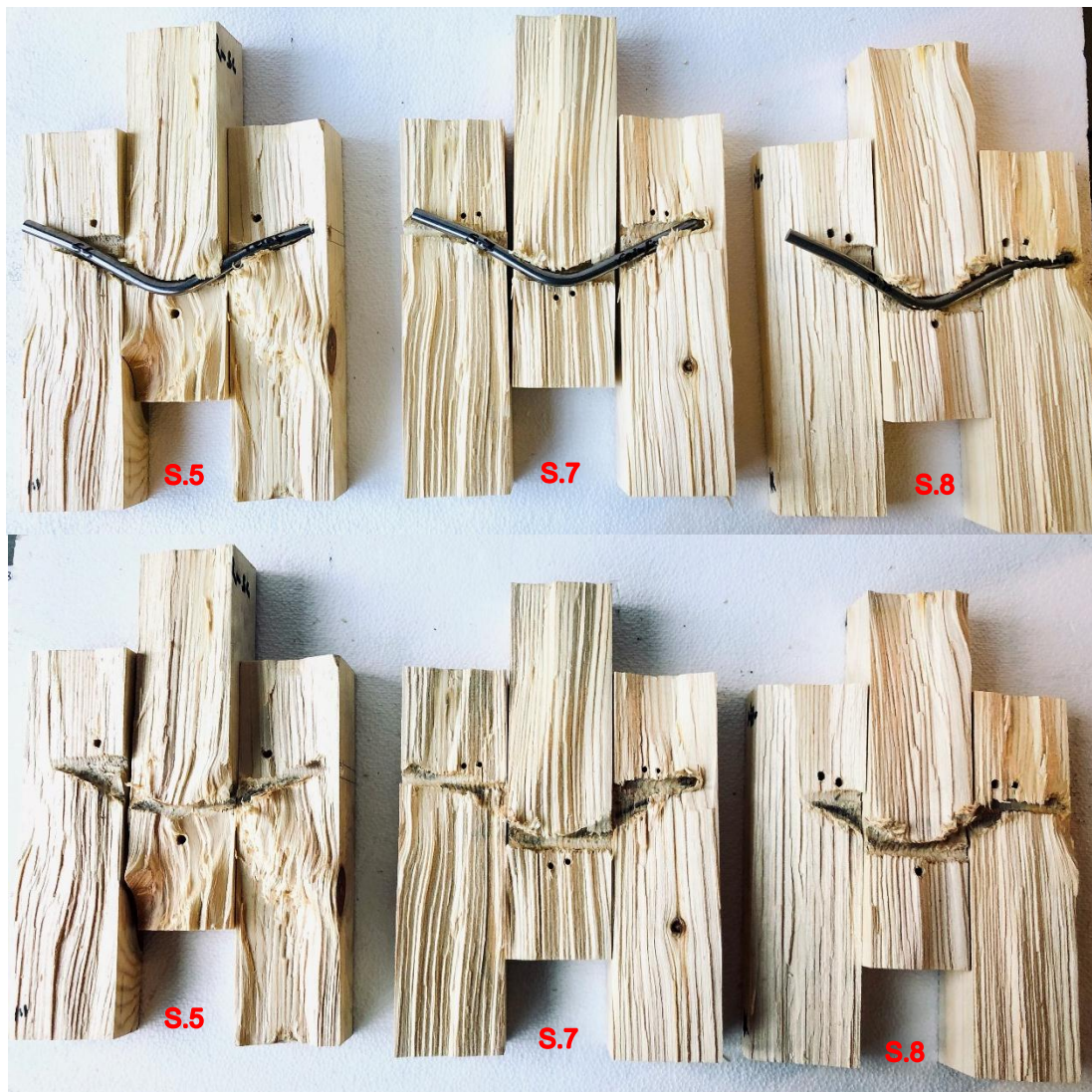


Figure 3:33: Failures associated with scheme (5), (7) and (8) for joints with 8 mm dowels



Figure 3:34: Left: failures in the dowels of the 3 schemes, Right: failures in comparison to the original joint

It can be noticed from Figure 3:33 and Figure 3:34 that the dowel used for scheme 8 had three hinges, with the hinge on element no. 1 not fully developed, as mentioned above. This indicates that the yield capacity of the dowel was not fully attained, yet compared to the other schemes, it is the only configuration that achieved mode (k) ^[3] despite of the fact that the maximum capacity obtained was slightly less than the capacity reached by the joint tested with scheme (5). This could be due to the presence of knots in the joint reinforced with scheme (5) compared to the other joint. The presence of knots affects the overall behaviour of the joint under loading. This can be seen in Figure 3:35, which shows the force-slip curves of the 3 joints opposed to the average of the 6 original joints. The stiffness has increased with scheme (8) in comparison to the original, yet the highest stiffness was obtained by scheme (5). Figure 3:36 shows the knots in joint N1_R_SC (Scheme 5).

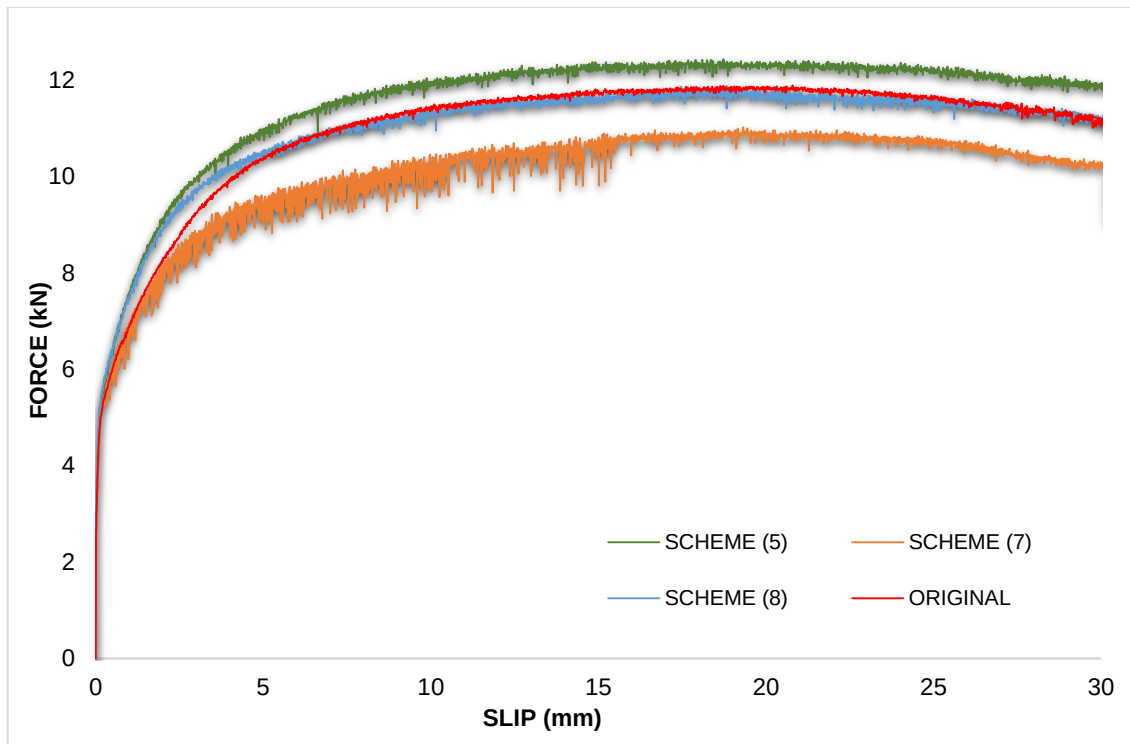


Figure 3:35: Force-slip curves of N1_R_SC, N2_R_SC and N3_R_SC, with 8 mm dowels vs. the original average

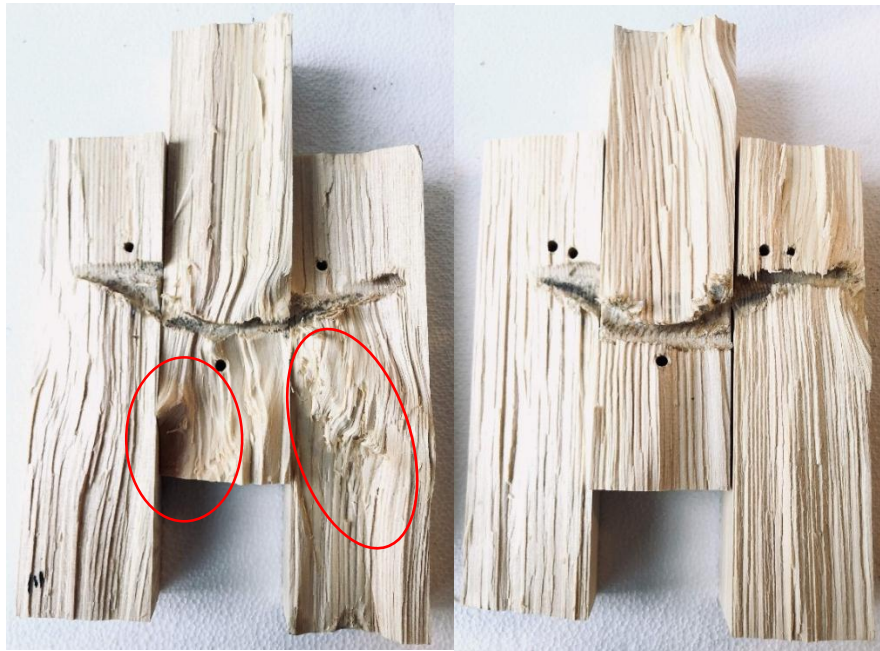


Figure 3:36: Left: Knots in joint N1_R_SC (Scheme 5), Right: Joint N3_R_SC (Scheme 8), sound wood

Consequently, the judgement on the effectiveness of the reinforcement cannot be concluded based solely on these results. Since the results of the joints with a dowel of a diameter of 8 mm showed that scheme (8) could be the most effective reinforcement technique in terms of ductility, improvement of embedment strength and dowel's deformation behaviour, scheme (8) was again tested on 3 joints with 12 mm dowels. Figure 3:37 shows the force-slip curves.

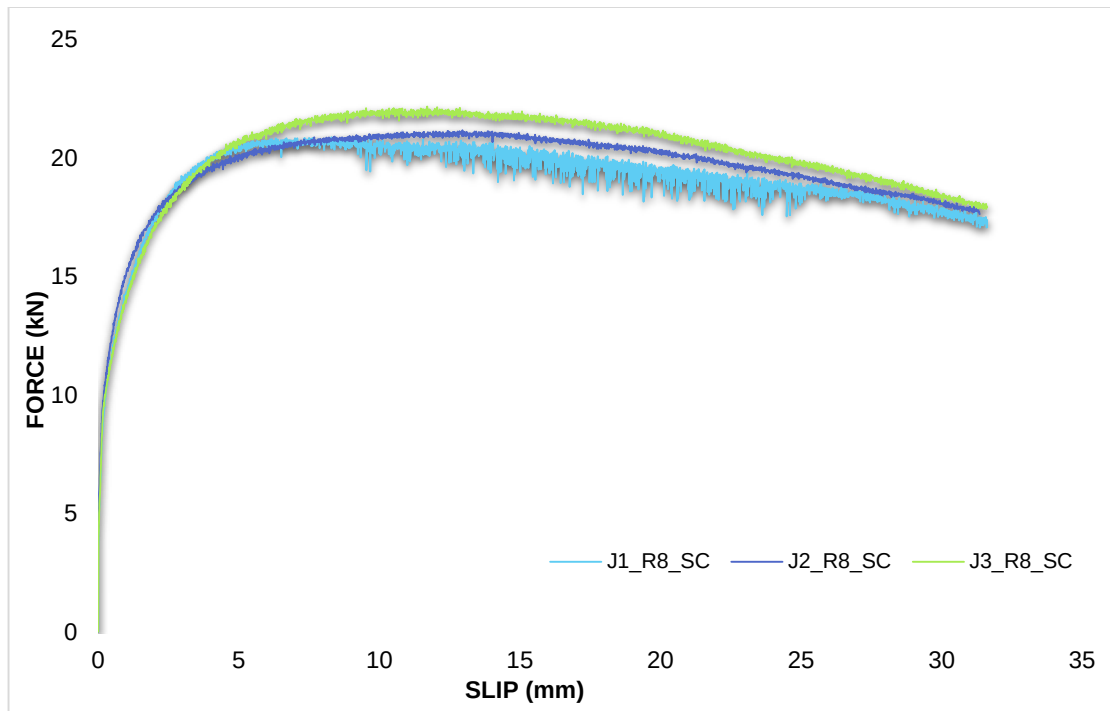


Figure 3:37: Force-slip curves of the 3 joints with 12 mm dowels, reinforced with scheme (8)

The maximum force reached for each one of these 3 joints with this reinforcement scheme, averaged around 21.4 kN Table 3-10, is higher than the maximum force obtained by each of the previous schemes.

Table 3-10: Maximum force values obtained by reinforcement scheme (8) vs. other reinforcement schemes

SCHEME (8)		SCHEMES (1) to (6)			
Joint Tag	F _{max}	Joint Tag	F _{max}	Joint Tag	F _{max}
J1_R8_SC	20.87	J1_R_SC	18.13	J4_R_SC	17.73
J2_R8_SC	21.17	J2_R_SC	15.65	J5_R_SC	20.75
J3_R8_SC	22.17	J3_R_SC	18.56	J6_R_SC	19.36

With regards to the previous results of 12 mm dowel joints with the previous 6 schemes, schemes (5) and (6) have provided the best results, and as mentioned earlier, and that scheme (8) was originally developed as a combination of these two schemes. This was carried out as a final step in choosing a final configuration to be tested for cyclic loading, and to have an in-depth evaluation of the effectiveness of the dowel's slenderness in increasing the load-carrying capacity, embedment strength and its relation to the failure modes proposed by Johansen. Figure 3:38 shows the resulting failure mode from scheme (8) on a joint with 12 mm dowel opposed to the original joint, and Figure 3:39 shows the results in terms of dowel deformation opposed to the other schemes and the original joint. Figure 3:40 shows the force-

slip curves of the 6 schemes opposed to scheme 8 with 12 mm dowels and the average of the original joints.

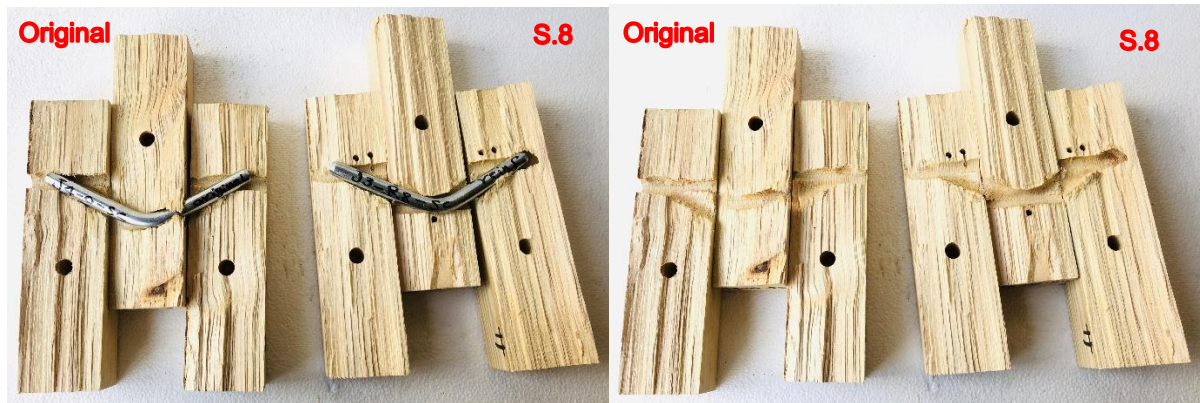


Figure 3:38: Joint with 12 mm dowel Reinforce with scheme (8) vs. an original joint

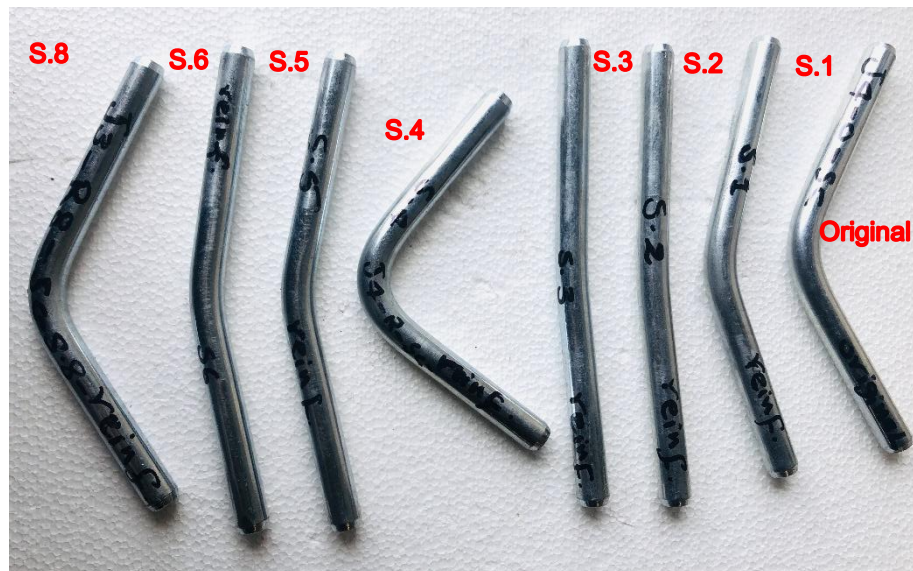


Figure 3:39: Dowels' deformation of the reinforcement schemes vs. the original

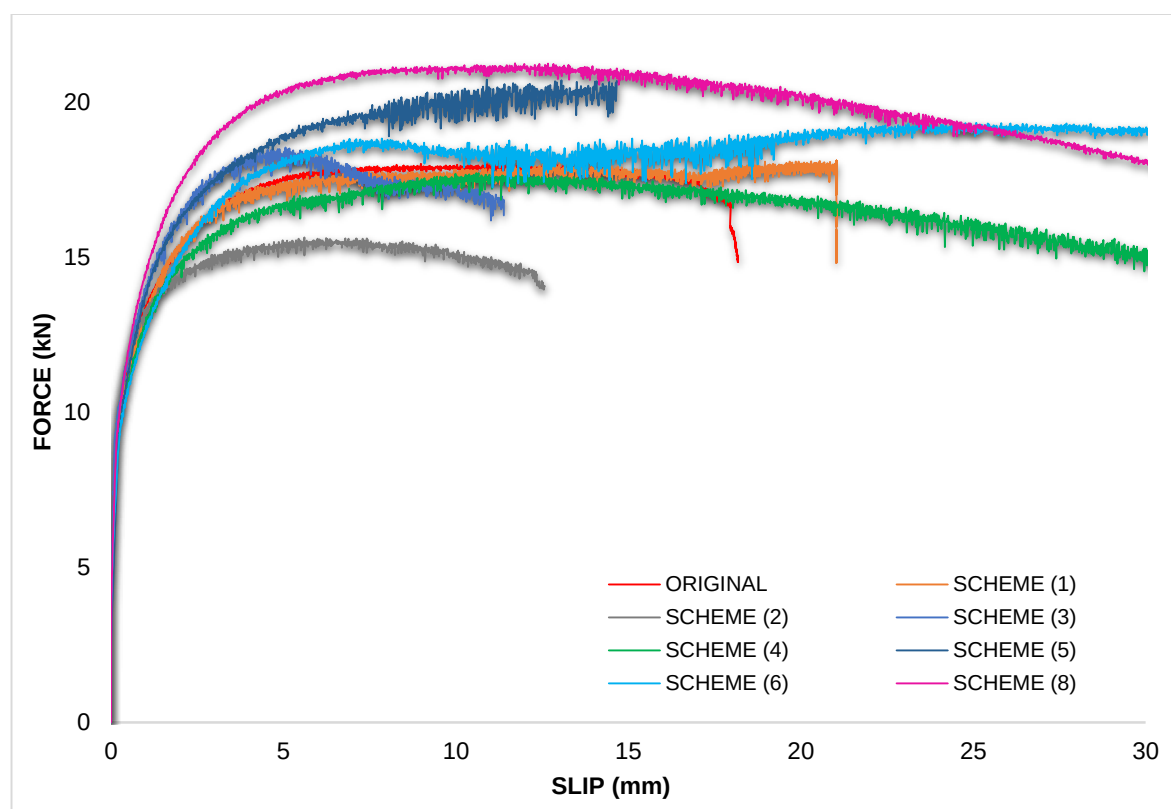


Figure 3:40: Force-slip curves of the 6 reinforced joints with 12 mm dowels vs. scheme (8) and the original

Although the failure mode obtained with this scheme shown in Figure 3:38 above, is the same as the original joints, mode (j), yet it can be noticed from the monotonic curves in Figure 3:40 that the capacity of the joint, the stiffness and the ductility all have improved. Thus, based on the results obtained in this section, scheme (8) was chosen to be adopted for reinforcement for cyclic testing, 3.2.3. Figure 3:41 shows the joint reinforced with scheme (8) in the test apparatus.

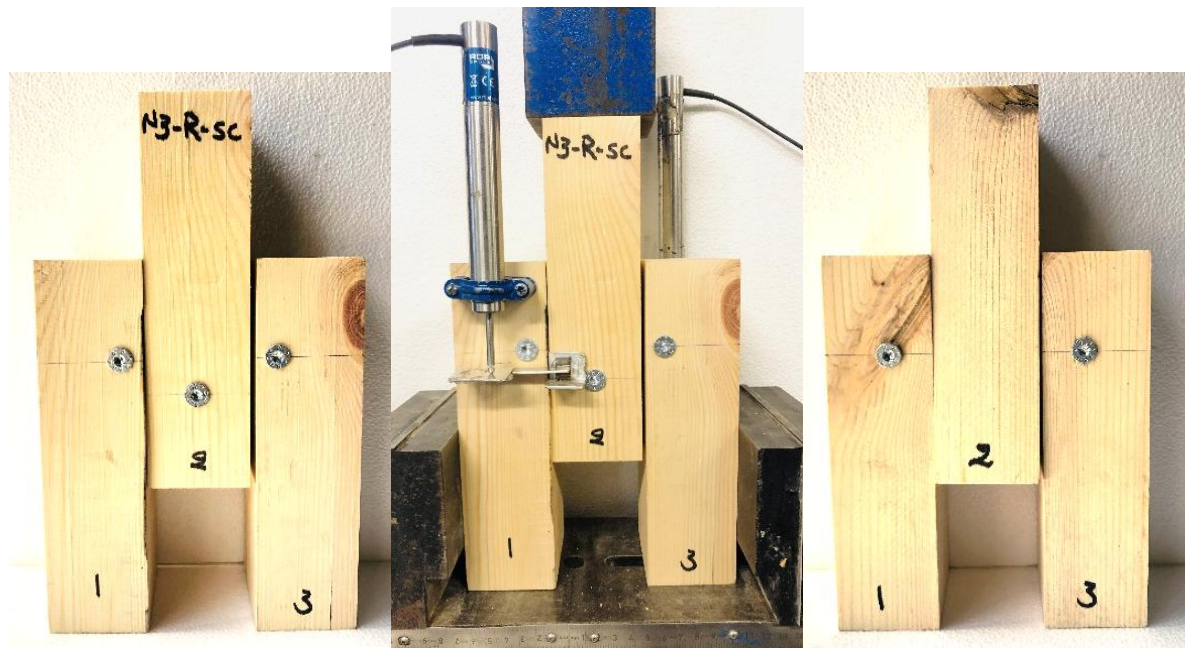


Figure 3:41: Left: Front Elevation, Middle: The joint while testing, Right: Rear Elevation

3.4.1.3 REPAIRED JOINTS

The same procedure was followed for the repaired connections, as 5 different repairing schemes were developed based on the failures that occurred on original connections after being subjected to monotonic loading, and what is the best approach to utilize STS to help limit the damage and restore the connection's capacity.

For all repairing schemes, fully threaded VGZ screws were used, and similar to reinforcement, joints with 12 mm and 8 mm dowels were tested. Only joints that are partially damaged were chosen to be repaired, joints that were totally split were excluded. The repairing techniques differed based on the type of damage that has occurred on each joint. Consequently, the STS were placed in order to cater to different damage scenarios.

3.4.1.3.1 SCHEME (1)

The first joint to be repaired is joint J1_O_SC, which had splitting in the middle element, in addition to cracking on both of the side elements. The joint was repaired with a total of 3 screws, one per each element, placed perpendicular to the grain direction. In the middle element, and since the splitting started at the bottom of the element below the dowel, one VGZ screw was placed at the center of the element, below the dowel in direct contact, in order to stop the splitting from prevailing to the top. The other two screws in the side elements were placed at the center of each element at a distance of 30 mm from the shear plane, perpendicular to the grain. This scheme is similar to reinforcement scheme (4). The only difference is that the position of the screws with respect to the dowel in the grain directions different as

it was necessary to cater for the dowel's deformation. The screws were placed based on the position of the dowel after deformation. The screws were inserted without pre-drilling. Figure 3:42

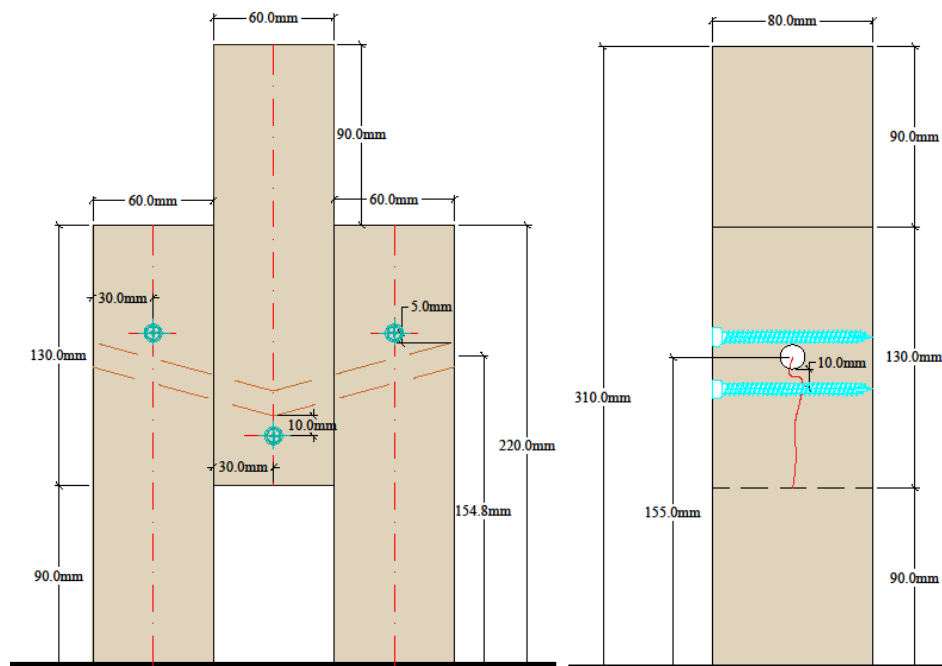


Figure 3:42: Repairing scheme (1), Left: Front elevation, Right: Side elevation

The original connection had a maximum force of 16.33 kN. After loading in compression for the second time with this repairing scheme, the joint reached a maximum force of 11.78 kN. This means that the recovered capacity with this scheme is 72.14 %. Figure 3:43 shows the force-slip curves for the repaired joint and the same joint after the first test and before repairing.

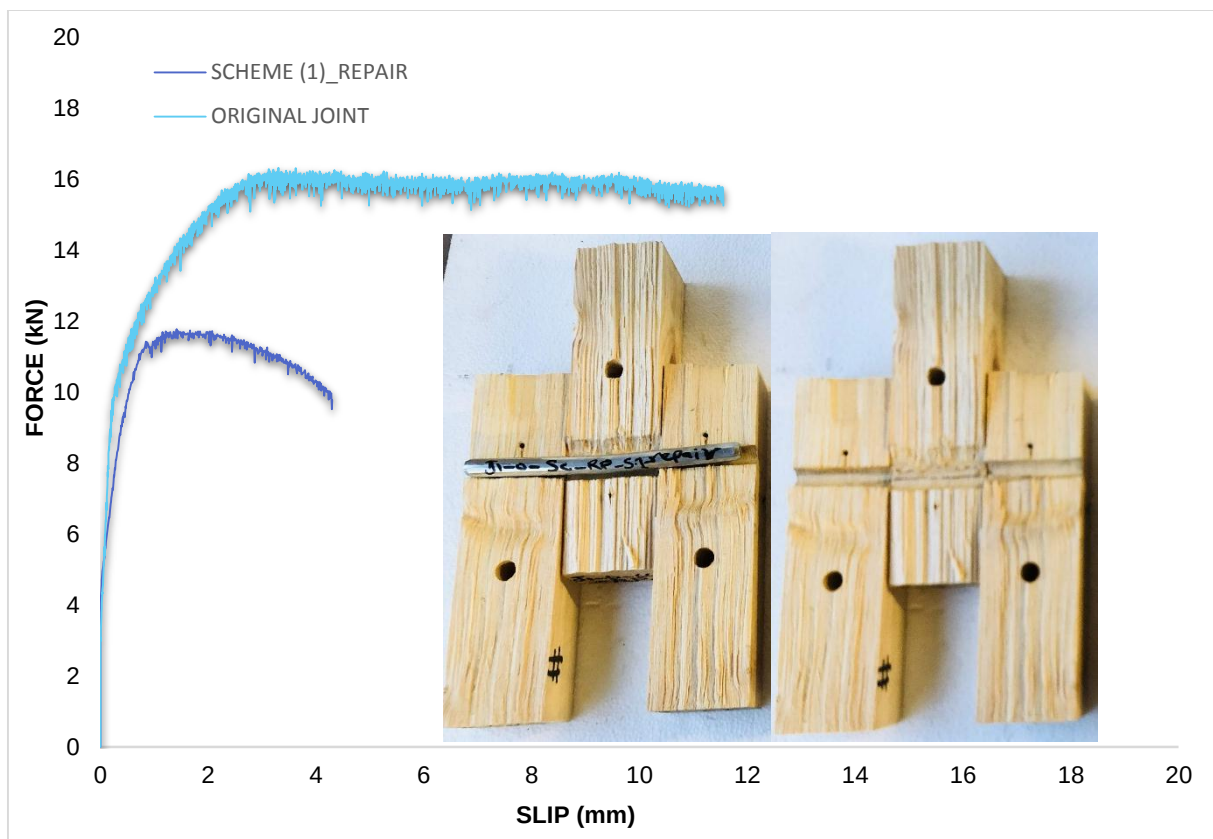


Figure 3:43: J1_O_SC_RP tested with scheme (1), Failure and force-slip curve opposed to the original joint.

As it can be seen from the force-slip curve that the repaired joint had failed with embedment in the middle element, mode (h). Although the joint has recovered 72.14 % of its original capacity, the failure is brittle and the joint has much less ductility than the joint that was reinforced with the same scheme with VGZ, reinforcement scheme (4). This scheme for reinforcement provided high ductility to the joint but the capacity was less than the original joint. In the case of repairing the effect is the opposite, which agrees with the assumption drawn based on the results obtained previously that with reinforcement, the fully threaded STS does not improve the capacity but increases the embedding strength. However, if adopted to repair splitting, it stops the crack from prevailing, but it does not increase the embedding strength despite assisting in recovering part of the joint's capacity, as it provides stiffness to the joint. Figure 3:44 shows the failure obtained with the same scheme but for reinforcement and repairing.

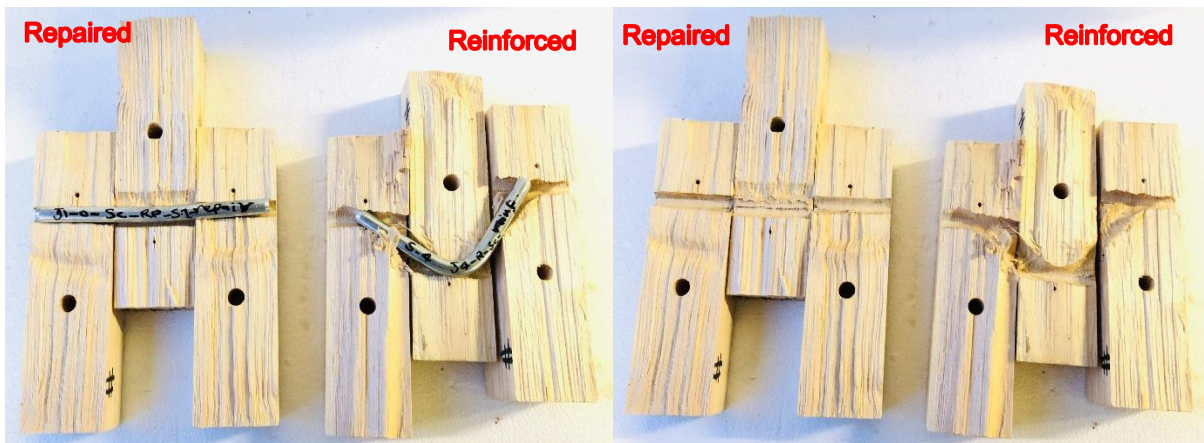


Figure 3:44: Different failure modes for the same scheme with reinforcement and repairing – VGZ screws

3.4.1.3.2 SCHEME (2)

The second repairing scheme was applied to joint J2_O_SC that failed with mode (j). Referring to the results of the previous repairing technique, and the results of the 1st and the 4th reinforcement schemes, it can be noticed that placing the screws at 30 mm, either HBS or VGZ screws and whether for the purpose was reinforcement or repairing, the screws do not form a rigid support in order for the dowel to reach its maximum yield capacity and form a plastic hinge. As mentioned earlier, the position of the plastic hinge for the joints with 12 mm dowels is 26.86 mm from the shear plane, measured perpendicular to the grain, therefore, placing the screws at 15 mm, between the hinge position and the shear plane, provided better results for reinforcement. Thus, for this scheme, the screws on the side elements were placed at 15 mm distance from the shear plane. Two screws in the middle element were placed 10 mm apart, perpendicular to the grain, each at 5 mm from the center of the element. The distance to the dowel parallel to the grain, a_1 , on the middle element, was increased to 10 mm to facilitate the positioning of the screws with the deformed dowel without damaging the fibers. Figure 3:45

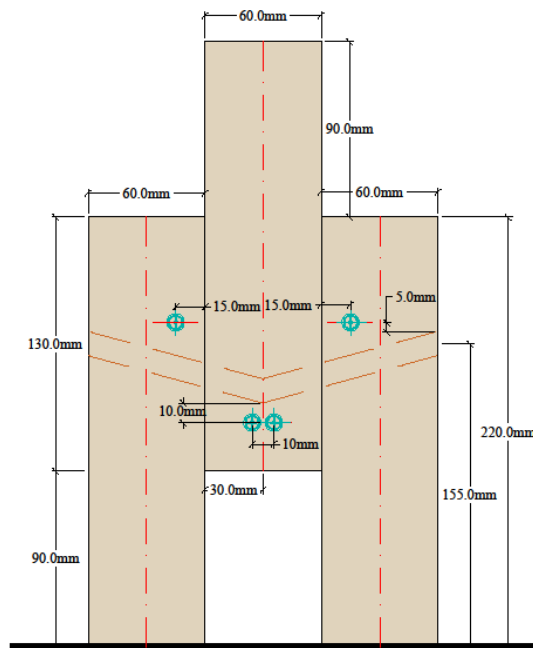


Figure 3:45: Repairing scheme (2)

The maximum force reached with this scheme for this joint is 12.9 kN which represents 71.55% of the original capacity of the joint. The two screws in the middle element have catalyzed the plastic hinge to form and part of the dowel's cross-section to plasticize. However, the two screws in the side elements have improved the embedment strength but still failed to help the formation of hinges. Figure 3:46 shows the failure on the joint and the force-slip curves of the joint before and after repairing.

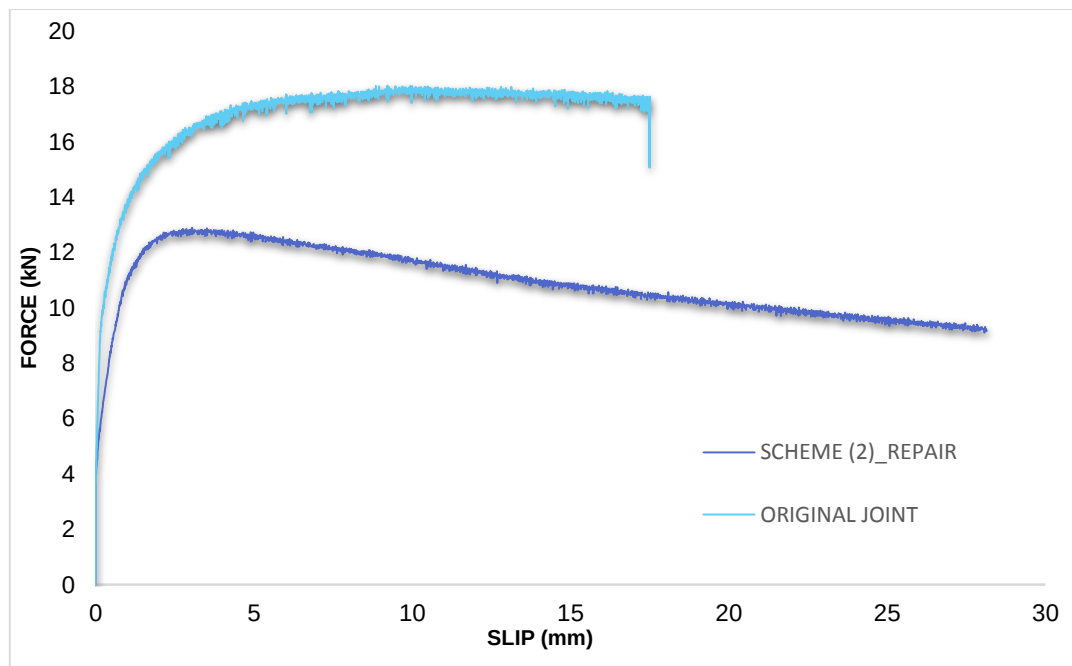


Figure 3:46: J2_O_SC_RP tested with scheme (2), force-slip curve opposed to the original joint.

It can be noticed from the capacity curve shown in the figure above that the stiffness has decreased but the ductility has increased significantly. No splitting has occurred on the joint after testing for the second time as well, but the elements had a tendency to separate, and the dowel to slip outwards. Figure 3:47

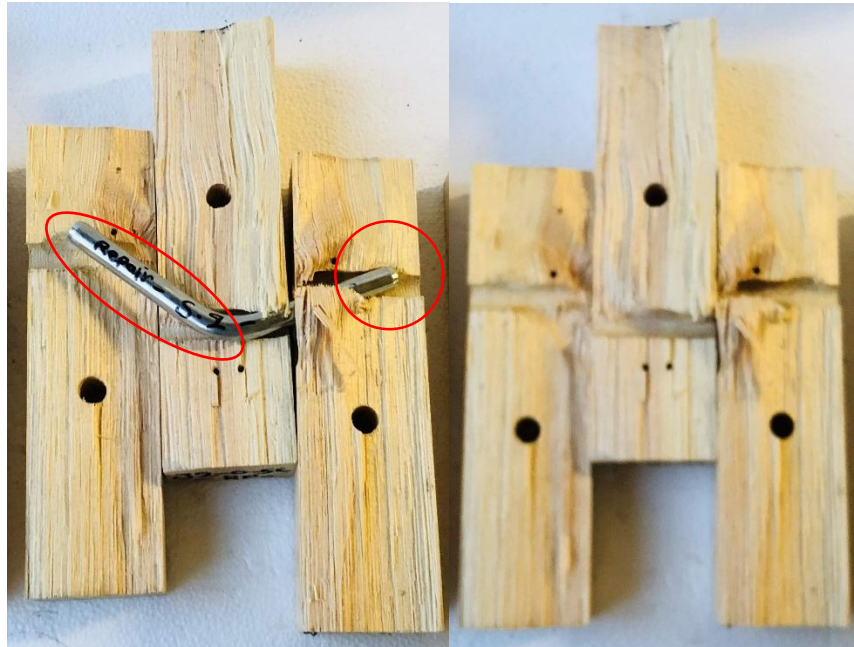


Figure 3:47: Left: Dowel slipping inside the joint, Right: Embedment in timber elements

3.4.1.3.3 SCHEME (3)

In order to avoid the brittle failure resulted from scheme (1), the number of screws was increased to 2 for the side elements to help improve the embedment strength of the timber. The distance between the screws in a direction perpendicular to the grain, a_2 , was set to 10 mm, and the distance to the shear plane, P , was kept as 15 mm. This scheme is similar to reinforcement scheme (7). As referred to earlier in section 3.2.2.2.2.7, VGZ screws did not increase the joint's capacity when used for reinforcement, but provided a significant improvement in terms of ductility, yet the exact opposite has been encountered when the same configuration was used for repairing.

Therefore, after comparing the use of VGZ fully threaded screws for reinforcement and repairing, applying the same geometrical configuration for reinforcement scheme(4) and repairing scheme (1), detailed above, the same configuration used for reinforcement scheme (7) was applied for repairing in this joint. This technique was ought to be tried once again, only this time for repairing purposes, in order to understand the reason behind the drastic difference in failure modes with the same configuration that was obtained previously and shown in Figure 3:43. The only difference, and as explained in repairing scheme (1) above, that the distance between the screws and the dowel in a direction parallel to the grain, a_1 , was increased to cater for the deformation of the dowel. Figure 3:48

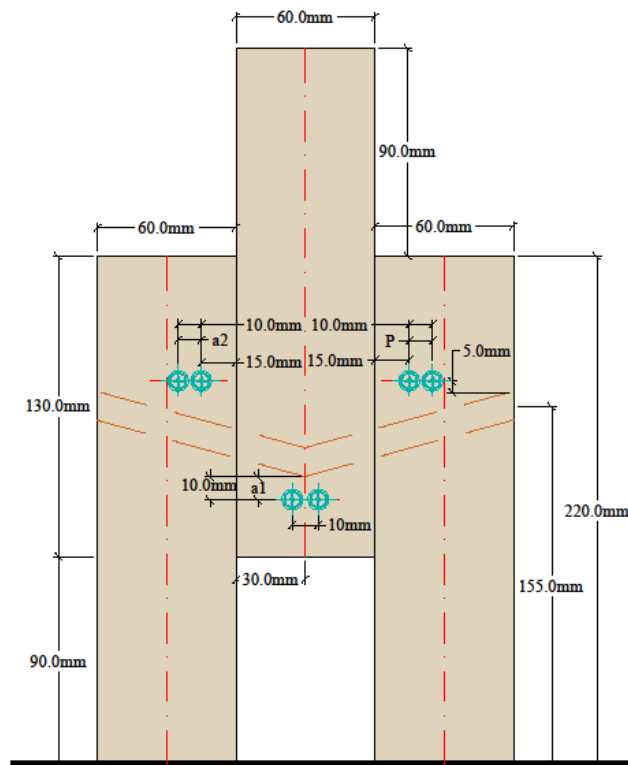


Figure 3:48: Repairing scheme (3)

This configuration was first applied to joint J3_O_SC_RP, but the joint could not sustain reloading for the second time, and the elements were also dismantled and totally separated only after applying a force of 7.5 kN thus before reaching 40% of the maximum load specified for the protocol. Thus, to judge if this behavior is a joint-specified behavior or that the scheme does not work for repairing, another joint was tested with the same configuration, J5_O_SC_RP. As a result, 73.74% of the original joint's capacity was recovered with the joint sustaining a maximum of 13.06 kN maximum force, which is the maximum obtained between the 3 schemes. Figure 3:49 shows the force-slip curve of the joint before and after repairing, and the associated failure.

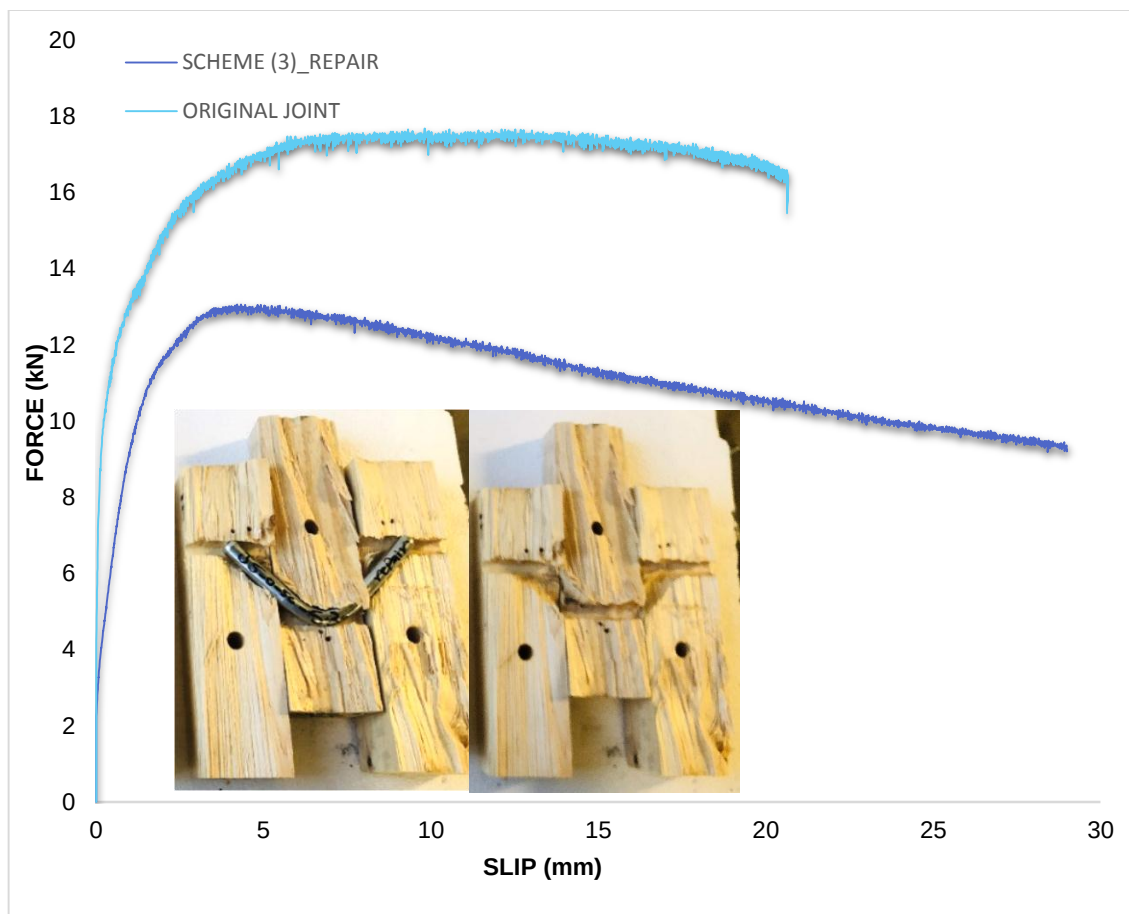


Figure 3:49: J5_O_SC_RP tested with scheme (3), Failure and force-slip curve vs. the original joint.

The additional screw on each of the side elements have improved the capacity opposed to the 1st repairing scheme, and helped to prevent the dowel from slipping and rotating outwards. Nonetheless, the members have also suffered a tendency to separate under loading, and the ability of the joint to resist this separation is highly dependent on the fixation of the joint on the test apparatus.

3.4.1.3.4 SCHEME (4)

To understand the behavior of the joints and their tendency to separate while testing instead of splitting, it was necessary to investigate if the position of the screws on the side elements and the density of the timber element are the main factor that is governing such failure. Therefore, this scheme was tested on a Maritime pine joint with the screws on the side element placed below the dowel. The repairing technique is detailed in section 3.2.2.3.3 of Maritime pine joints.

3.4.1.3.5 SCHEME (5)

In light of the results obtained from the previous repairing schemes, this configuration was proposed as a solution to prevent the shear and tensile forces that caused the elements forming the joints to separate at the shear planes causing the joints to dismantle and the dowel to slip. Consequently, restore the

joint's capacity. In this scheme the STS were placed inclined at an angle to the grain, and to the direction of the load, in order to mobilize their high withdrawal capacity and prevent separation of the elements and local splitting.

Fully threaded screws have a rope effect of 100 % ^[3], as explained earlier in this report. Therefore, to guarantee a proper utilization of the high withdrawal capacity of the self-tapping screws and to benefit from their high tensile strength, different VGZ screws with different characterization had to be used instead, giving the geometry of the connection and the dimensions of the screws that were used for the previous joints. Connecting the elements at the shear planes required longer screws, subsequently, bigger diameters. Therefore, for this case, 7 mm X 140 mm VGZ fully threaded screws were used.

The length of the screws was chosen based on the requirements by EC5 ^[3] that indicates for axially loaded screws, the minimum point side penetration depth is equal to $6.d = 42$ mm. Another parameter that is taken into account when deciding the length of the screws is the inclination angle, α . To achieve the maximum capacity of the screws, the angle of inclination, α , should be 45° . However, the limitations imposed by the geometry of these joints have excluded this option, as it was not possible to place the screws at an angle of 45° and keep the specified end distances. Thus, in this work it was adjusted to 30° , with reference to Rothoblass VGZ technical sheet ^[20], which specifies the minimum angle between the screw axis and the grain direction is 30° . Figure 3:50

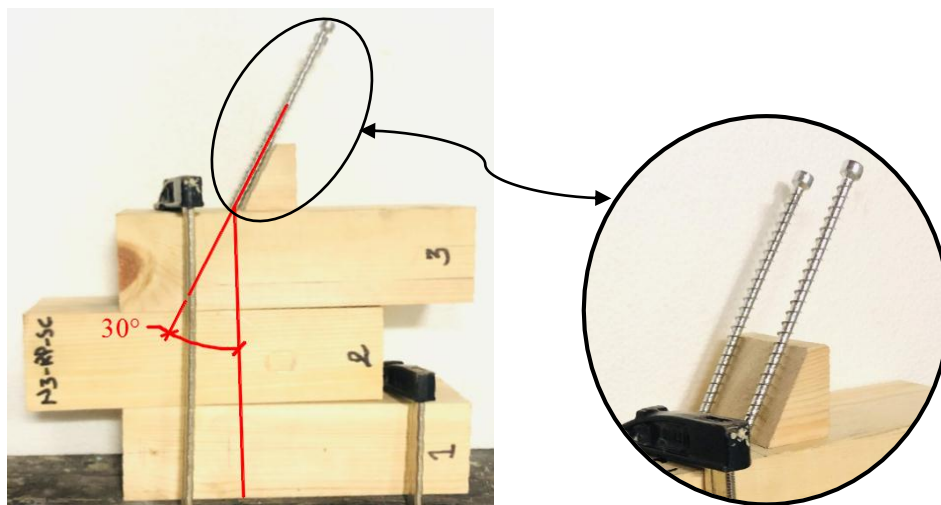


Figure 3:50: Fully threaded STS drilled inclined at 30°

According to EC5 ^[3], the end distances for the axially loaded screws should be $> 4.d$. Philipp Dietsch and Reinhard Brandner ^[19] have provided spacing and end distances requirements for this specific case of inclined screws in dowel-type joints in double shear, Figure 3:51

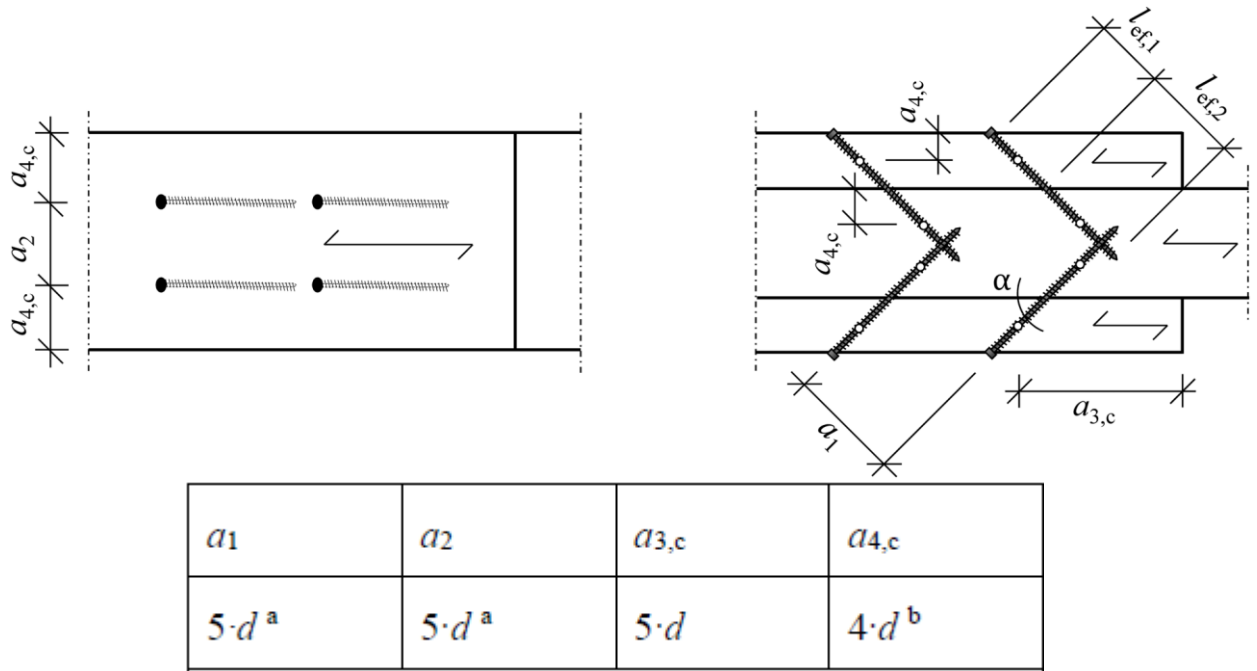


Figure 3:51: Typical minimum spacing and minimum distance requirements given in technical approvals for axially-loaded STS ^[19]

Following these specifications, the geometry of the joints has once again formed a barrier preventing the full adaptation of these rules. The end distance of the screws on the side elements parallel to the grain direction, $a_{3,c}$, is $5 \cdot d$ which is 35 mm. However, when 35 mm was adopted with both angles 30° and 45° , the screw was not fully embedded into the timber element which can damage the fibers. Therefore, for this case it was adjusted to $10 \cdot d = 70$ mm, based on Rothoblass, self-tapping screws technical document ^[17] that stated that for inclined STS, $a_{3,c} \geq 10 \cdot d$.

Regarding the end distance of the screws perpendicular to the grain direction, $a_{4,c}$, it is specified in [19] as $4 \cdot d$, which is 28 mm in this case. Once again, this dimension was adjusted to 22.5 mm in order to keep the distance between the screws, a_2 , as specified which is $5 \cdot d = 35$ mm.

The withdrawal capacity of the screws were calculated based on the equations provided by EC5 ^[3] for axially loaded screws. The characteristic yield moment value was obtained with equation no (3-13), and the embedment strength was calculated accordingly with the following expression:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad (3-13)$$

Where $f_{h,0,k}$, the characteristic embedment strength was calculated using equation (3-10) above, and K_{90} for hardwoods is given by the equation below:

$$K_{90} = 0.9 + 0.015 \cdot d \quad (3-14)$$

The characteristic withdrawal capacity is obtained by the following expression, for axially loaded screws:

$$F_{ax,\alpha,RK} = n_{ef} (\pi \cdot d \cdot l_{ef})^{0.8} f_{ax,\alpha,K} \quad (3-15)$$

Where:

$$f_{ax,\alpha,k} = \frac{f_{ax,k}}{\sin^2 \alpha + 1,5 \cos^2 \alpha} \quad (3-16)$$

And:

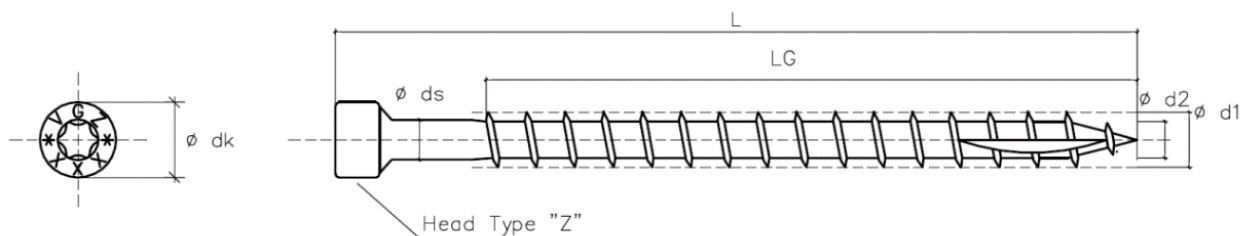
$$f_{ax,k} = 3,6 \times 10^{-3} \rho_k^{1,5} \quad (3-17)$$

The effective number of screws is equal to:

$$n_{ef} = n^{0,9} \quad (3-18)$$

A total of 4 screws per joint were used and were placed without pre-drilling. Table 3-11 shows the screws' characterization, and Table 3-12 shows the spacing, end distances and the withdrawal capacity calculations for the screws. The final configuration adopted for the joints was developed accordingly. Figure 3:52.

Table 3-11: Properties and load-carrying capacity of inclined Rothoblass VGZ screws



d1 (mm)	d2 (mm)	L (mm)	dk (mm)	ds (mm)	l _{ef} (mm)	LG (mm)	M _{y,s} (N.mm)	f _{h,s} (N/mm ²)	R _{A1} (N)	R _{A2} (N)	R _{A3} (N)	R _{VE} (N)
7	4.6	140	9.5	5	66	130	7930	35.99	35975	15221	5710	5710

Table 3-12: Withdrawal capacity calculations

a1 (mm)	a3,c (mm)	a2 (mm)	a4,c (mm)	α °	K	n	n _{ef}	f _{h,α,k} (N/mm ²)	F _{ax,k} (N/mm ²)	F _{ax,α,k} (N/mm ²)	F _{ax,α,Rk} (N/mm ²)
30	60	30	18	30	1.005	4	2.64	36.71	36.91	37.85	44598

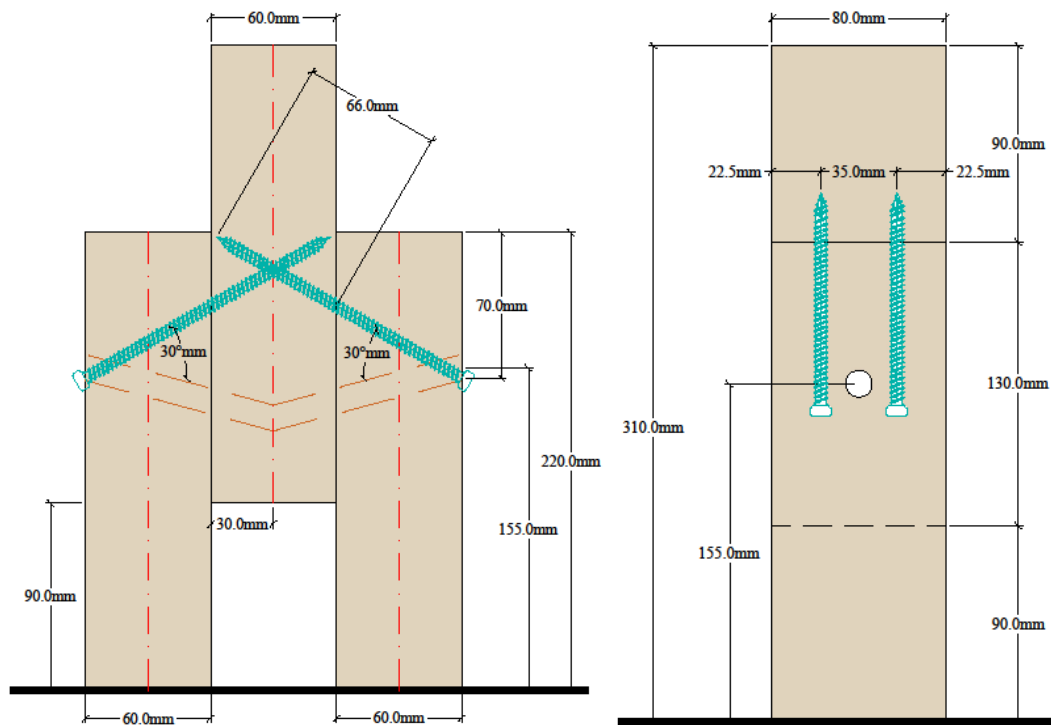


Figure 3.52: Repairing scheme (5), Left: Front elevation, Right: Side elevation

This scheme was tested on 3 joints, J7_O_SC_RP, J8_O_SC_RP and J10_O_SC_RP. Joint J9_O_SC_RP was a total failure by splitting and a total separation on one side element, thus repairing was inapplicable. However, regarding the 3 joints that were tested, the maximum forces obtained from applying this scheme were much higher than the forces obtained by the original joints. The failure on all 3 joints corresponded to mode (j) [3], with one plastic hinge in the middle element. Although the connections reached their maximum capacities and failed prior to the formation of 3 plastic hinges and the cross-section of the dowels was not yet fully plasticized, yet the stiffness and the ductility of the joints have significantly increased. Figure 3.53 shows the failure on one of the joints and Figure 3.54 shows the force slip curves for the 3 joints each in comparison with the original capacity curve.

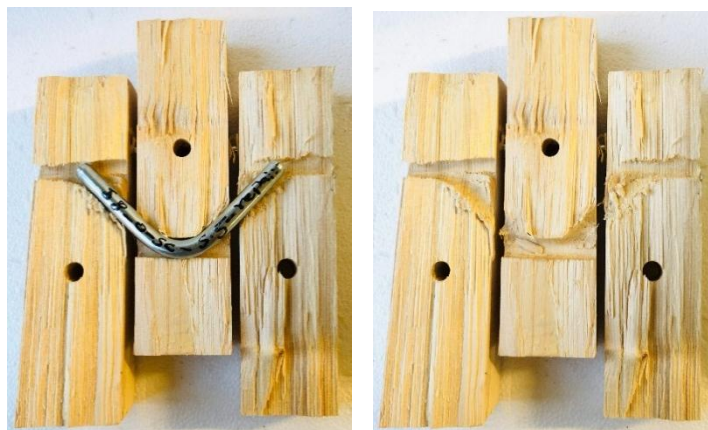




Figure 3:53 - Top: Dowel deformation, Bottom: Screws' deformation

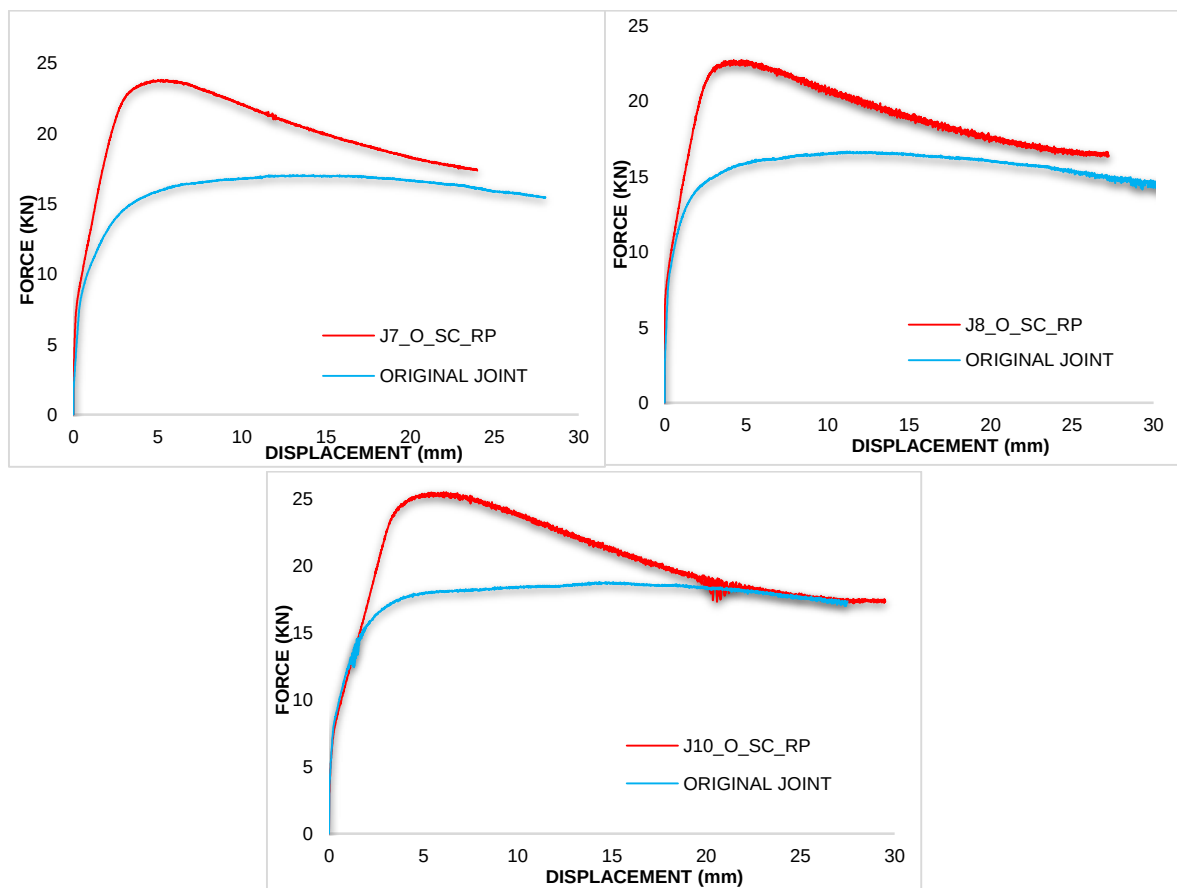


Figure 3:54 - Force-slip curve vs. the capacity curve of the original for the 3 tested joints.

It can be noticed that the capacity have increased significantly with this technique compared to the original capacity of the joint. This can be referred to the fact that the screws were placed in an undamaged part of the wood, therefore the connection sustained the applied load as it was a new connection. Not to mention that due to the position of the screws and their high withdrawal capacity, all the tensile forces perpendicular

to the grain were taken entirely by the screws resulting in a rigid-plastic deformation of the screws at the shear planes.

3.4.1.3.6 COMPARISON OF REPAIRING SCHEMES

All repairing schemes were tested with fully threaded VGZ screws as mentioned above. However, when comparing the first 3 schemes, it can be concluded that VGZ screws when used for both reinforcement and repairing, they tend to improve the ductility of the joint. Regarding the case of the joint that was repaired with the 1st scheme, the different behaviour and the failure was brittle when repaired with VGZ opposed to the results obtained in this other two joints as well as the reinforced joints, could be a result of the knots within both the side elements in addition to the different failure pattern that occurred in the original joint. J1_O_SC_RP had failed with splitting, which indicates that the original joint had already low embedment strength and less ductility. Figure 3:55.



Figure 3:55 - Failures associated with scheme (1), (2) and (3) for joints with 12 mm dowels.

Despite the different failure modes obtained with the 3 schemes, yet the recovered capacity of the joints is over 70 % in all 3 joints. It can be noticed from the capacity curves in Figure 3:56 below, that the screws in the first 3 schemes when placed perpendicular to the grain direction, in addition to restoring part of the joints' capacity, they have increased the ductility of the joints, which was a common result for VGZ screws with the repaired and reinforced joints within this campaign.

Nonetheless, the problem of the separation of the elements could not be avoided with the screws placed perpendicular to the grain. Shear forces that are generated on the shear planes, are usually accompanied by tensile forces perpendicular to the grain in this type of joints, and if these forces were avoided to be taken by the timber elements, then the joint can provide a better performance and sustain higher loads. Therefore, scheme (5) was developed aiming at providing a configuration that can guarantee these forces are to be taken entirely by the screws. This can be achieved by mobilizing the withdrawal capacity of the screws at the shear planes to take these tensile forces.

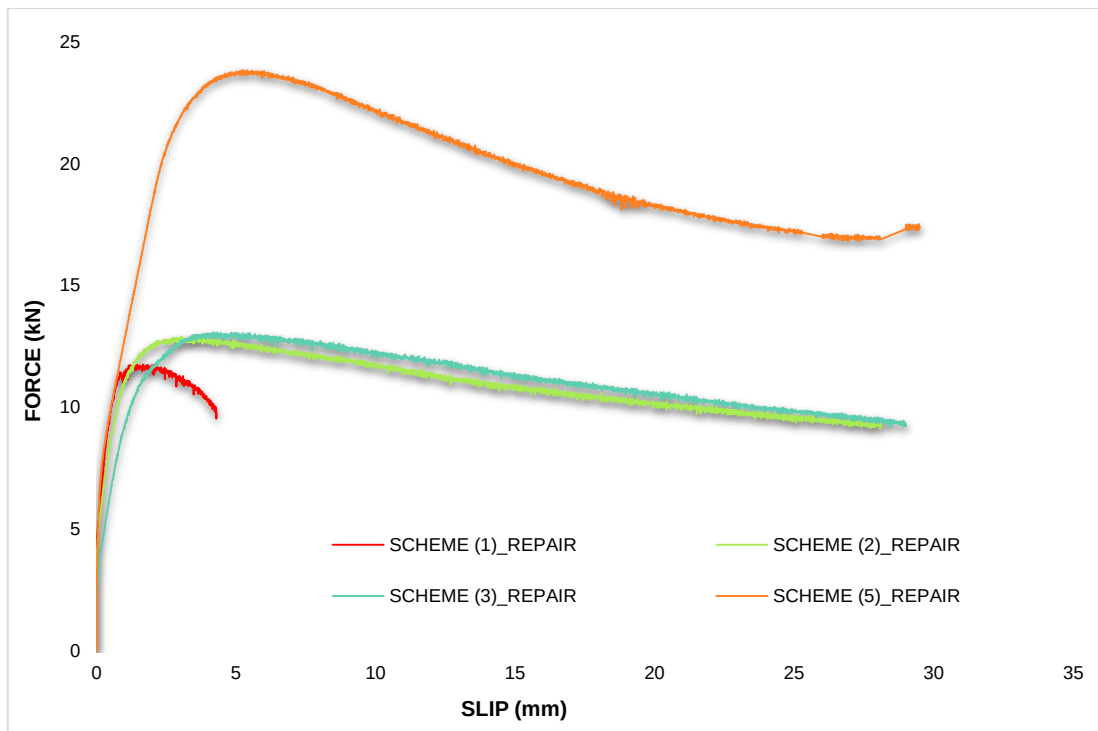


Figure 3:56 - Force-slip curves of repairing schemes (1), (2) and (3) vs. scheme (5) with inclined screws.

The effect of the screws positioning with respect to the grain direction and the effect of the tensile forces perpendicular to the grain in the overall capacity of the joints can be clearly seen judging by the force-slip curves in Figure 3:56. The capacity of the joint have not only significantly recovered, but increased to over 120 % of the original capacity of the joints. The ductility has also improved as well as the embedding strength on all 3 elements that form the joint. Thus, based on the results obtained by these 3 joints repaired with scheme (5), with 12 mm dowels, the scheme was applied for another 3 joints with 8 mm dowels in order to see the effect of the dowel slenderness on the recovered capacity and the failure mode when the tensile forces re taken by the screws instead. Figure 3:57 shows the failure

obtained by the joints with both 12 mm and 8 mm dowels, and Figure 3:58 shows the capacity curves of these joints in comparison.



Figure 3:57 - Left: Failure on 8 mm dowel joint, Right: Failure on 12 mm dowel joint.

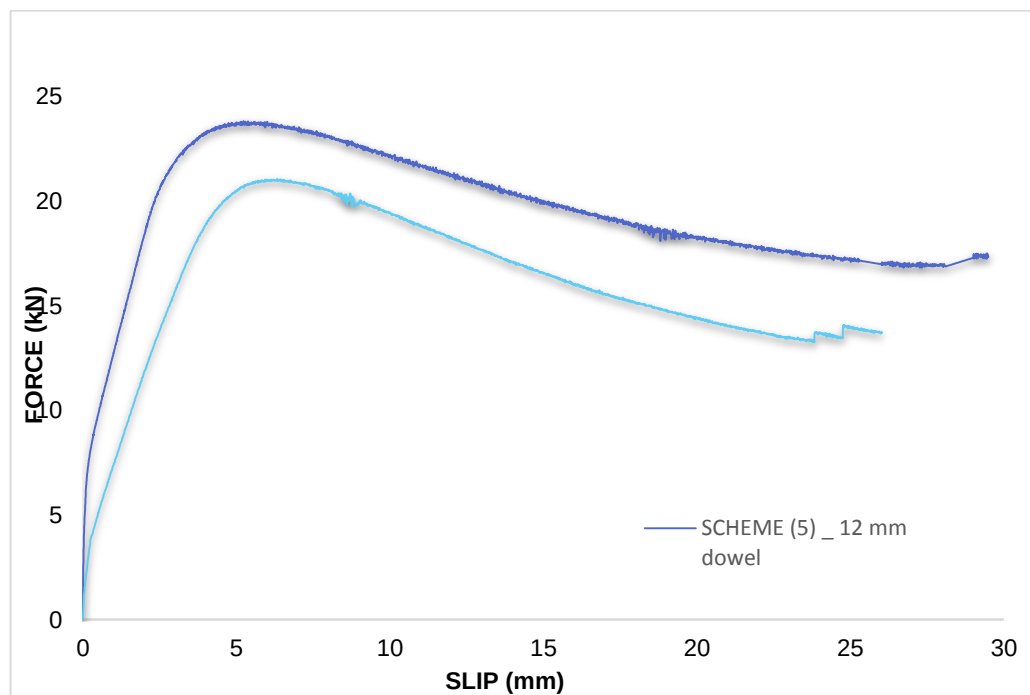


Figure 3:58 - Force-slip curves for both joints and the deformation of the dowels.

It can be noticed from the capacity curves that the capacity of the joint with 12 mm dowel is higher than the capacity of the joint with an 8 mm dowel, which is normal giving the influence of the yield moment of each of the dowels in addition to the different density of the timber elements in both of the cases. The scots pine that was used for the joints with 8 mm dowels has less density than the wood that was used to form the joints with 12 mm dowels. Analysing the failure modes obtained with both joints, it can be noticed in Figure 3:57, that the failure modes are different as the dowel with 8 mm has 2 plastic hinges in the middle element and one hinge on one side element, with the second side element has a plastic hinge that is only partially developed. As for the 12 mm dowels, the failure mode is mode (j) ^[3], with one plastic hinge in the middle element, similar to the failure mode obtained by the previous repairing schemes, (2), (3) and (4), yet the capacity has increased significantly. This further proves that the high withdrawal capacity of STS have allowed for the screws to take the majority of the shear and tensile forces that are generated at the shear planes, and the plastic hinges that are formed on the screws accordingly have increased the ductility of the connection and prevented local splitting and brittle failures. Figure 3:59. This explains the enormous increases in the joints' capacity even further than the original, and after loading twice until failure is reached, despite of having the same failure mode and dowel deformation as the original. In both cases with a dowel of 12 mm and 8 mm, original and repaired with scheme (5), the dowel has not reached its maximum capacity and the cross-section was not fully plasticized, yet the results have improved. Table 3.13 shows the maximum force reached with all repairing schemes for both joint sets.



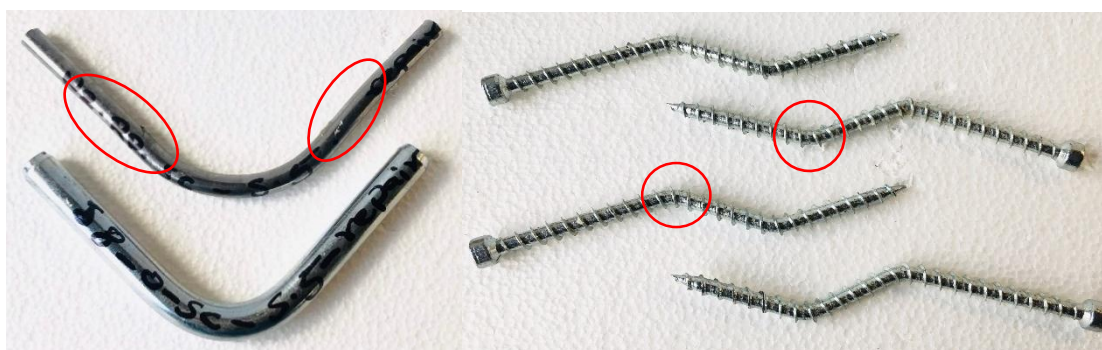


Figure 3:59 - Top: The evolution of failure modes with the repairing schemes with 12 mm dowels,
Bottom: Scheme (5) STS and dowels deformation with 12 mm and 8 mm dowel.

Table 3-13 - Maximum force obtained by repairing schemes, and the recovered capacity of the joints

Joint Tag	Repairing Scheme	Dowels Diameter (mm)	F_{max} (KN)	Recovered Capacity %
J1_O_SC_RP	1	12	11.78	72.14%
J2_O_SC_RP	2	12	12.90	71.55%
J5_O_SC_RP	3	12	13.06	73.74%
J7_O_SC_RP	5	12	23.82	125.49%
J8_O_SC_RP	5	12	22.73	128.58%
J10_O_SC_RP	5	12	25.51	124.00%
N1_RP_SC	5	8	16.74	134.45%
N2_RP_SC	5	8	23.08	203.19%
N3_RP_SC	5	8	23.55	190.35%

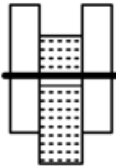
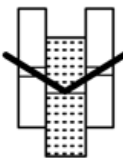
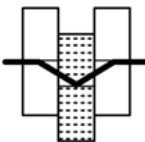
As it can be seen from the table above that the recovered capacity of the joints repaired with scheme (5) is above 100 %, which means that the capacity obtained is higher than the capacity of the original joints. Thus, this scheme was chosen as the repairing scheme for cyclic testing later on.

3.4.2 MARITIME PINE

3.4.2.1 ORIGINAL JOINTS

Similar to scots pine samples, the maximum capacity of maritime samples was estimated according to EC5 ^[3] following the same procedures and equations. The dowel used for these samples is 12 mm in diameter and 170 mm in length, similar to the scots pine samples tested with 12 mm dowels. Maritime pine has higher density, consequently, the capacity estimated for the joints is higher. The density obtained for the maritime samples is 556.06 Kg/m³, and the embedding strength is calculated accordingly and it's equal to 40.13 N/mm². Table 3-14 shows the maximum force obtained for non-reinforced maritime pine samples and the associated failure mode.

Table 3-14 - Failure modes according to EC5 and associated load-carrying capacity values for the original joints - 12 mm dowels

Failure Mode ^[3]	$F_{v,RK}$ (KN)
g 	57.78
h 	28.89
j 	24.49
k 	25.58

The same procedure and protocol for monotonic testing was adopted for maritime pine samples. 6 original joints were tested under monotonic loading prior to applying the reinforcement. These samples have higher density as mentioned above but the failure pattern obtained with testing was brittle. The test was stopped at failure, which occurred before reaching 30 mm of slip. One of the causes of brittle failure for these sample is the high concentration of knots and resin bags within the samples. Figure 2:1 shows the failure on the original samples, and the force-slip curves of the tested joints.

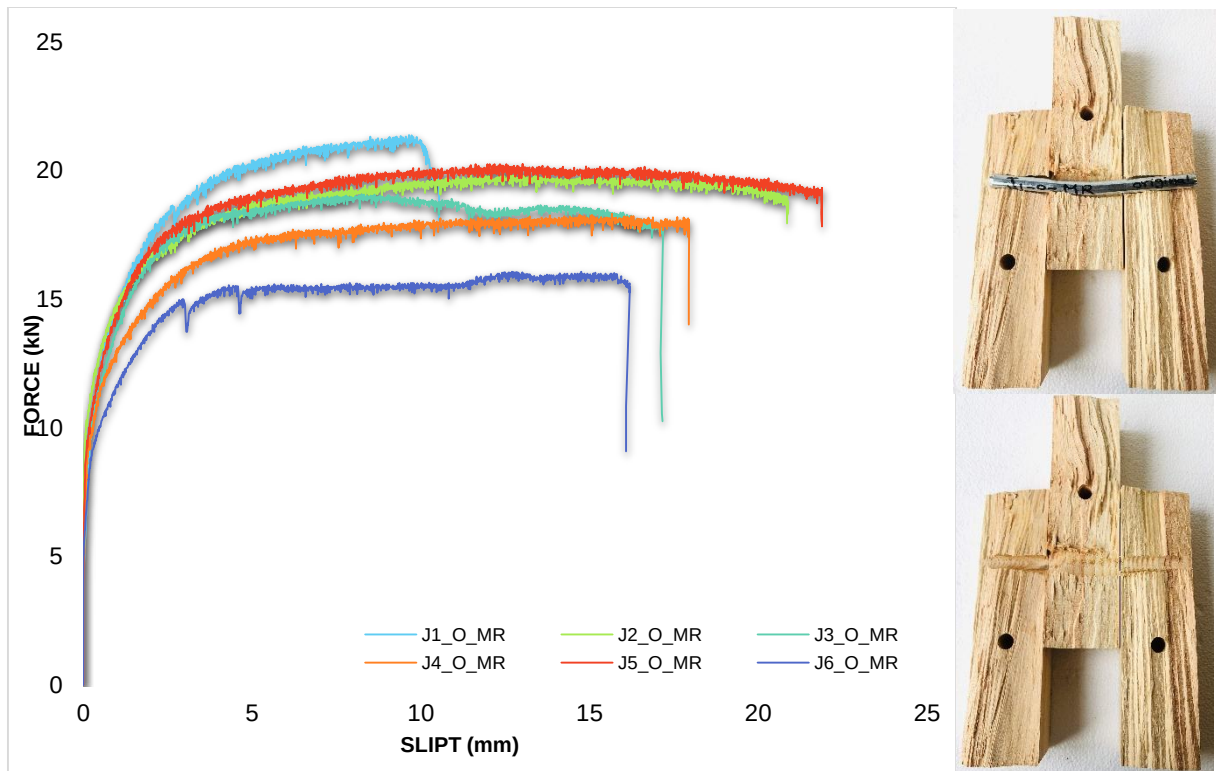


Figure 3:60 - Left: Force-slip curves for the 6 tested joints, Right: Typical failure obtained in the joints.

As it can be seen from the figure above, that the failure is brittle, and although according to estimations of the capacity, the failure mode that is expected is mode (j) ^[3], which is the third failure mode according to EC5 ^[3], yet the actual failure mode from the tests is the second failure mode, mode (h) ^[3] that corresponds to an embedment failure. This is another verification for the role of the dowel's slenderness in obtaining the failure modes specified by Johansen's that are based on the EYM. This can be further noticed referring to the maximum force values that are obtained after testing which are less than the value estimated by then equation. The equations are based on the assumption that the dowel reaches its maximum yield capacity, and that requires high percentage of slenderness which is not the case with these samples. Table 3-15 shows the maximum force values for the joints obtained by testing.

Table 3-15 - Maximum force values obtained by testing.

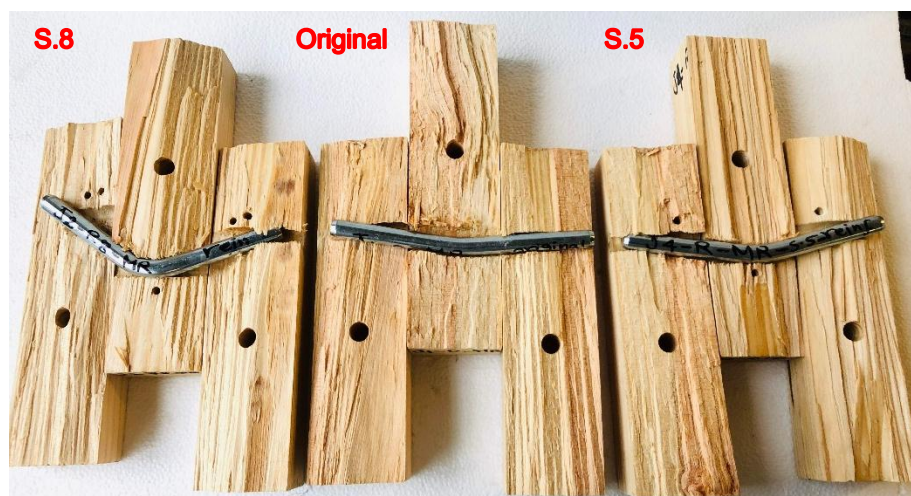
ORIGINAL JOINTS	
Joint Tag	F_{max} (KN)
J1_O_MR	21.39
J2_O_MR	19.89
J3_O_MR	19.09
J4_O_MR	18.33
J5_O_MR	20.27
J6_O_MR	16.07

3.4.2.2 REINFORCED JOINTS

For reinforcing maritime pine samples, scheme (8) that were chosen earlier for reinforcement was adopted for these samples as well due to the comparison that was conducted after carrying out several schemes. This was further verified by calculating the position of the plastic hinge referring to the equations provided in section 3.4.1.3.1, based on the force and moment equilibrium. From the results of the several reinforcement schemes that were tested with scots pine joints, it was concluded that the position of the plastic hinge plays an important role in the effectiveness of the reinforcement with STS. The reinforcement tends to increase the capacity of the joints only if the screws were placed in between the shear plane and the plastic hinge position. The position of the plastic hinge for these joints from both then shear planes, $X_{1,3} = X_{2,3} = 24.75 \text{ mm}$.

However, in order to further verify this theory, and conclude that scheme (8) gives the best results, reinforcement scheme (5) was tested on one joint prior to conducting scheme (8) on 3 joints of maritime pine. Scheme (5) was chosen based on the results that were obtained with scots pine joints. Aside from scheme (8) that provided the maximum capacity, scheme (5) have oddly given more capacity than scheme (6) that included a higher number of screws. According to the joints tested with scots pine, the reason was believed to be the presence of knots in the joint that was tested with scheme (5), and that it participated in increasing the joint's stiffness regardless of the reinforcement when compared to the joint that was tested with scheme (6). Thus, scheme (5) was tested once again with maritime pine to conclude on the results. Keeping in mind that the screws in scheme (5) are placed at a distance $P = 15 \text{ mm}$, and for scheme (8) are placed at a distance of 12 mm from the shear plane perpendicular to the grain, both reinforced with HBS screws.

Figure 3:61 shows the failure obtained by scheme (5), (8) both opposed to the original joint, and Table 3-16 includes the maximum forces obtained for the tested joints, along with the increase in the joints' capacity in percentage for both the schemes to the average of the original joints' capacity.



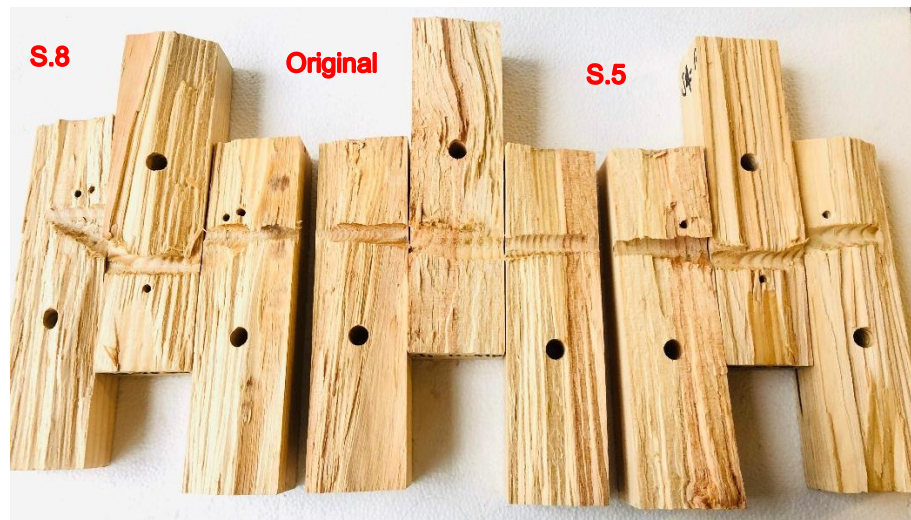


Figure 3:61 - Failure in reinforced joints opposed to the typical failure obtained with original joints.

Table 3-16 - Maximum force values obtained by reinforcement scheme (8) vs. scheme (5)

Joint Tag	Reinf. Scheme	F_{max} (KN)	P (mm)	Increase in capacity %
J1_R8_MR	8	22.65	12	15.35%
J2_R8_MR	8	22.38	12	14.33%
J3_R8_MR	8	22.83	12	16.02%
J4_R_MR	5	21.78	15	11.97%

In light of the results shown above and the evolution of the failure mode in the cases discussed above, scheme (8) once again shows the highest results. It concludes on the importance of positioning the screws closer to the shear plane. Thus, it is found to be suitable for reinforcing joints of maritime pine as well. A total of 3 joints were reinforced with this scheme as a final step, and the force-slip curve for the average of these 3 joints, along with the capacity curve of scheme (5) and the average of the original joints are shown in Figure 3:62 including the deformation of the dowel in each case.

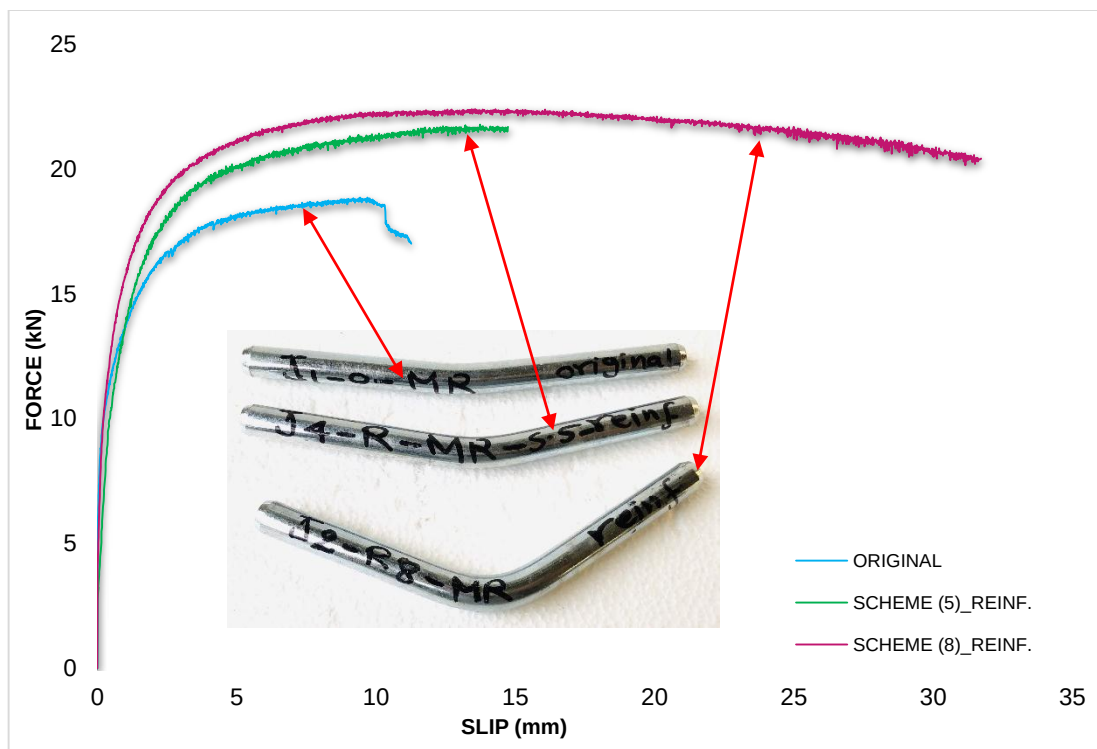


Figure 3:62 - Force-slip curves for the joints, reinforced with scheme (8), (5) and the original.

The final reinforcement scheme that has been chosen, has increased the ductility of the joint significantly compared to the original and the other scheme as well as the capacity. The yield moment capacity of the dowel in this case was higher with a fully developed plastic hinge in the middle element, mode (j) [3], therefore, the embedment strength of the timber element has improved. Over and above, the reinforcement have prevented the splitting that was encountered in the original joints.

3.4.2.3 REPAIRED JOINTS

Regarding the repairing of tested maritime joints, the same analogy that was followed for reinforcement above is adopted, and that is to test the scheme that was chosen for scots pine and has shown the best results, on maritime pine. In addition to one additional scheme with a different configuration and compare the results to conclude on the final configuration that can provide the best results for these wood species. After analysing the data obtained from monotonic testing of scots pine joints of the various repairing schemes, scheme (5) with inclined VGZ, fully threaded STS was chosen based on the significantly high capacities obtained following the accordingly.

However, the scheme was developed originally to repair the problem of separation of the elements on the joint due to high concentration of tensile forces perpendicular to the grain, in addition to shear forces. Nevertheless, this was not the case for maritime pine joints. The dominant failure pattern for these joints was local splitting, mainly in the middle element. Thus, placing the screws inclined at an angle to the grain does not provide a solution for the problem of splitting. As mentioned earlier in this report, one of the main benefits of STS is repairing the timber element's local splitting, but the screws has to be placed

perpendicular to the grain and to the dowel's axis. Thus, two schemes were developed for maritime joints in order to maintain this problem and help restore the joint's capacity with STS. So as a first step, scheme (4) was tested on the original joint, J2_O_MR. The two screws on the side elements were placed below the dowel for this technique as opposed to what was adopted previously, either for repairing or reinforcement, with a purpose of providing rigid supports for the dowel to encourage its yield capacity to increase, resulting in the formation of additional 2 plastic hinges. So far, the plastic hinge on the side element was only achieved with 8 mm dowels which supports the importance of the dowel's slenderness in increasing the capacity, yet, for dowels with less slenderness, the solution could lie in the position of the screw with respect to the dowel and the direction of the load application. Figure 3:63.

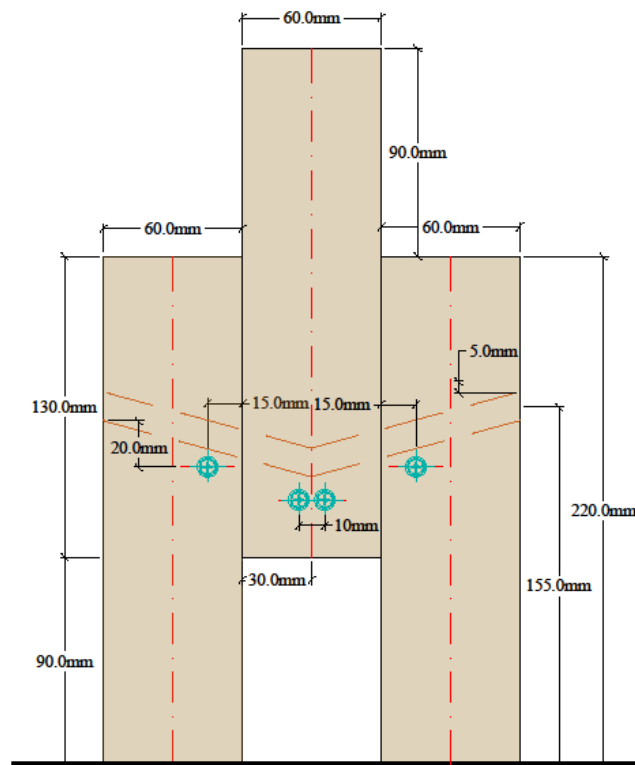


Figure 3:63 - Repairing scheme (4)

The maximum capacity obtained by this scheme on the tested joint was 15.2 kN which represents 76.42 % of the capacity reached by the joint originally. Similar to the results obtained by the previous repairing schemes for scots pine joints, the resulting joint has less stiffness but higher ductility than the original. This can be seen in the resulting failure mode, as no splitting has occurred. Figure 3.62.

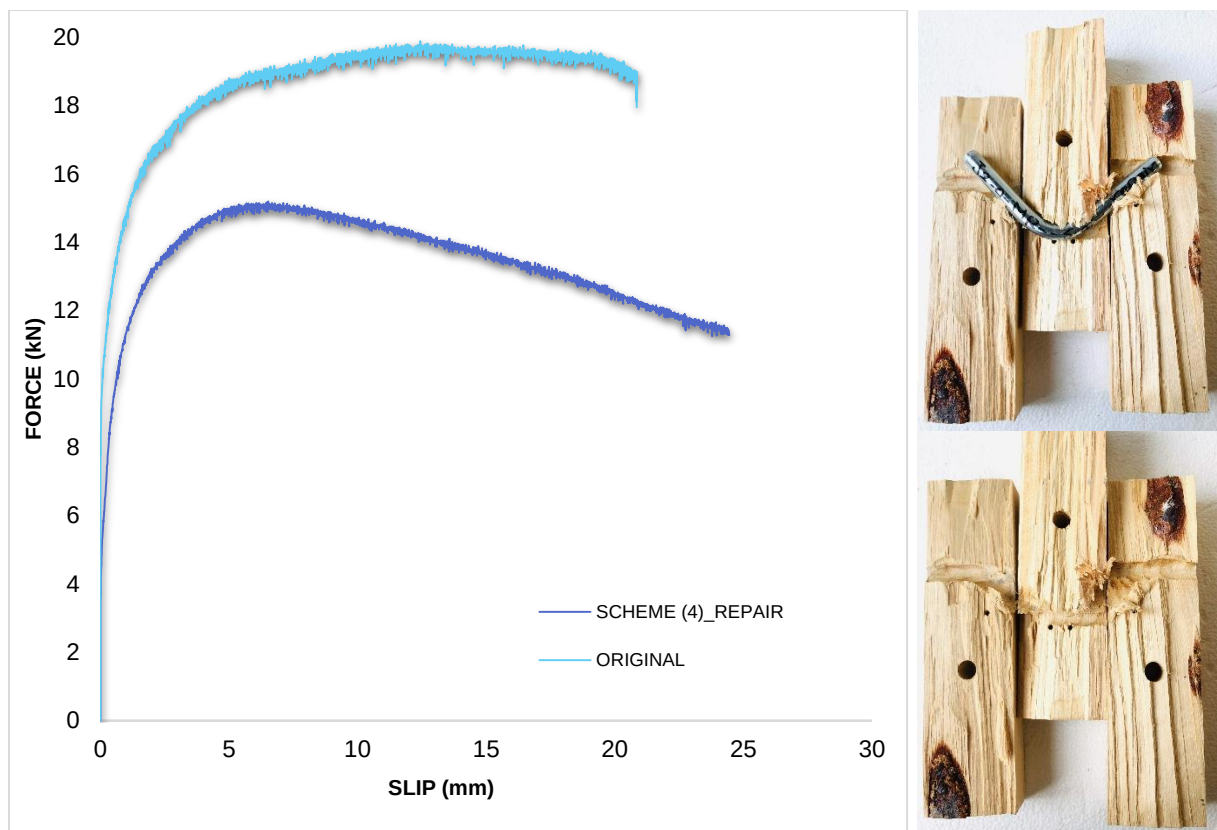


Figure 3:64 - Left: Force-slip curve for repairing scheme (4) vs. original, Right: Failure in the repaired joint.

Nonetheless, the joint's behaviour under loading was similar to the scots pine joints repaired with schemes (1), (2) and (3), in terms of separation of the elements and rotation of the dowel. Figure 3:65 shows the joint with the screws installed before and after testing. Based on the behaviour of the joint while testing and the failure obtained after testing, it is believed that the position of the screws on the side elements below the dowel might have caused the dowel to slip instead of deforming plastically.

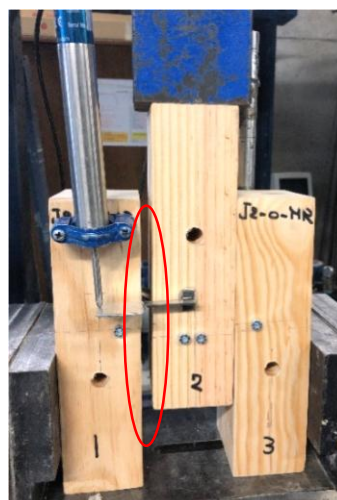


Figure 3:65 - The joint in the test apparatus, elements started to separate.

3.4.2.3.1 SCHEME (6)

Based on the results of the previous repairing schemes, an additional 6th scheme was developed to cater for the case of maritime pine. The main aim is to repair the splitting as mentioned before. Although the previous scheme has in fact achieved this goal and helped recover a considerable amount of capacity, yet it has created a different type of failure for the joint. Thus, the screws are once again placed over the dowel on the side elements, two screws per element, at $P = 15 \text{ mm}$. Regarding the middle element, which is the element that has encountered splitting the most amongst the original joints, two STS were added above and below the dowel, opposite to one another, in order to restrain the rotation of the dowel, stop the crack from prevailing to the top of the element and increase the embedding strength of the timber in the vicinity of the dowel. Figure 3:66.

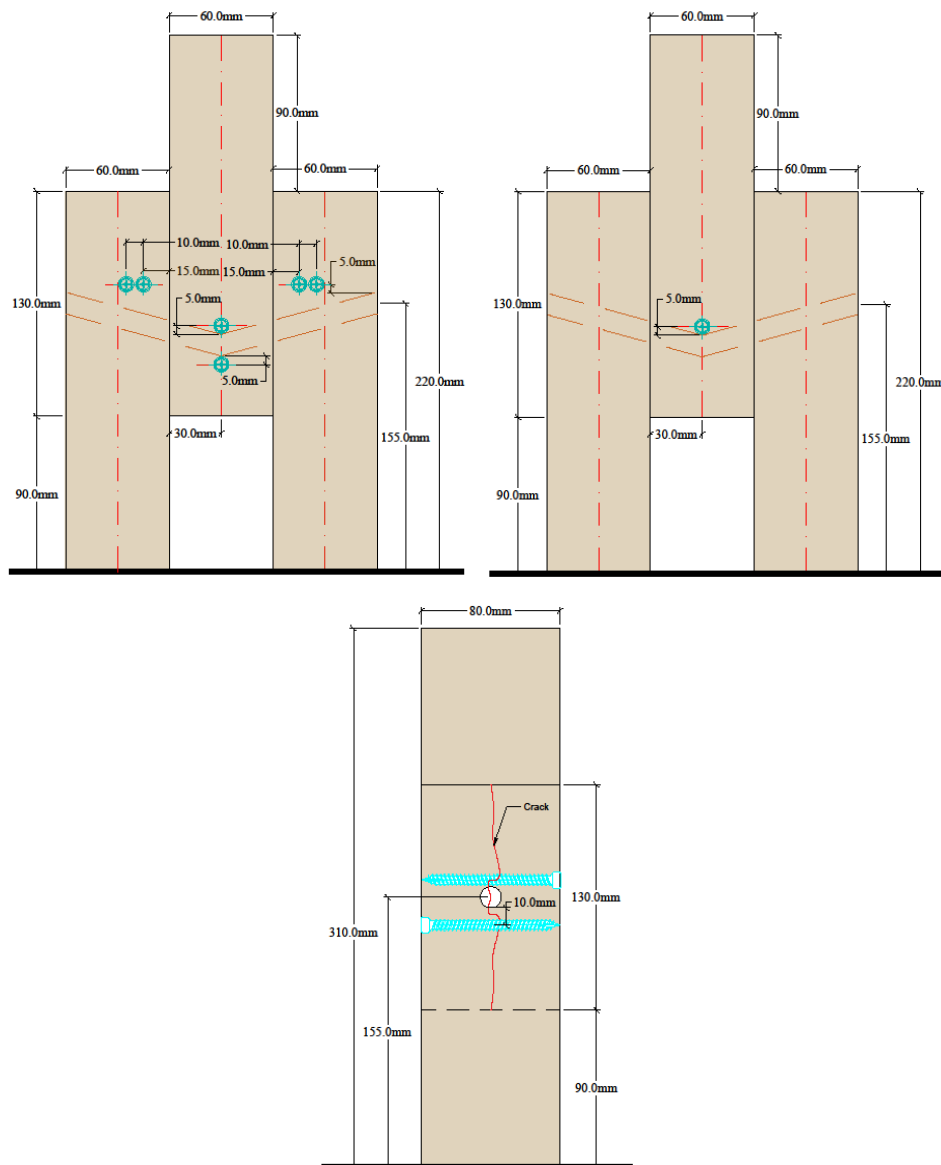


Figure 3:66 - Repairing scheme (6), Left: Front elevation, Bottom: Side elevation, Right: Rear elevation

This scheme was tested on 3 joints and more than 80% of the capacity of the original joints has been recovered in all 3 joints. The splitting of the element was stopped and the ductility of the joints has increased significantly. The maximum force obtained with the 3 joints with this scheme, in addition to the joint repaired with scheme (4) and the recovered capacities in percentage are shown in Table 3-17 below, and the force-slip curves of the 3 repaired joints opposed to the original are shown in Figure 3:67.

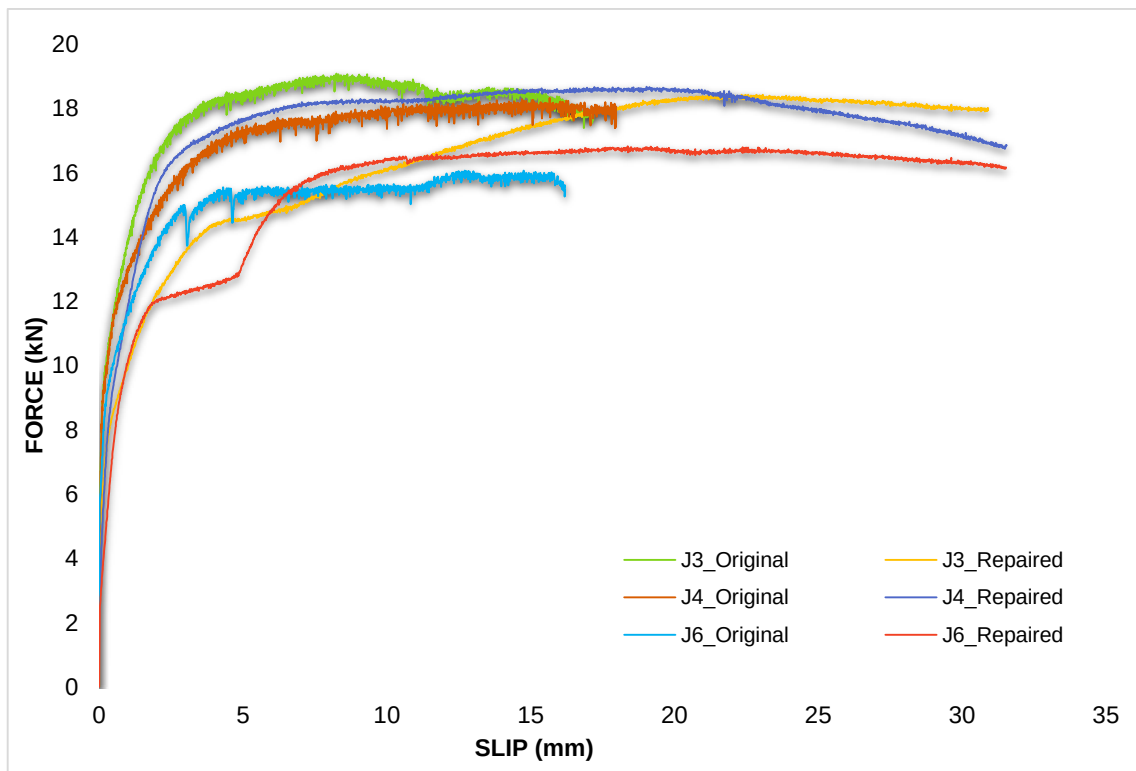


Figure 3:67 - Force-slip curves for the 3 tested joints, repaired with scheme (6).

Table 3-17 - Maximum force values obtained by repairing scheme (6) vs. scheme (4), and recovered capacity.

Joint Tag	Repairing Scheme	Dowels Diameter (mm)	F_{max} (kN)	Recovered Capacity %
J2_O_SC_RP	4	12	15.20	76.42%
J3_O_SC_RP	6	12	18.43	96.55%
J4_O_SC_RP	6	12	18.68	101.90%
J6_O_SC_RP	6	12	16.82	82.99%

Analyzing these results, it can be concluded that scheme (6) is the best solution for repairing maritime pine joints. The failure mode obtained from both schemes was mode (j) ^[3], but the deformation of the dowel is different in terms of the development of the plastic hinge in the middle element. With scheme (6), the hinge was fully developed and the deformation was rigid-plastic. This could be a result of placing

the screws above and below the dowel, forcing its deformation, opposed to the other scheme where the screws were both below the dowel. Figure 3:68 shows a plot of the capacity curve that corresponds to the average of these 3 joints after repairing, in addition to the joint repaired with scheme (4), both in comparison to the average of the original joints.

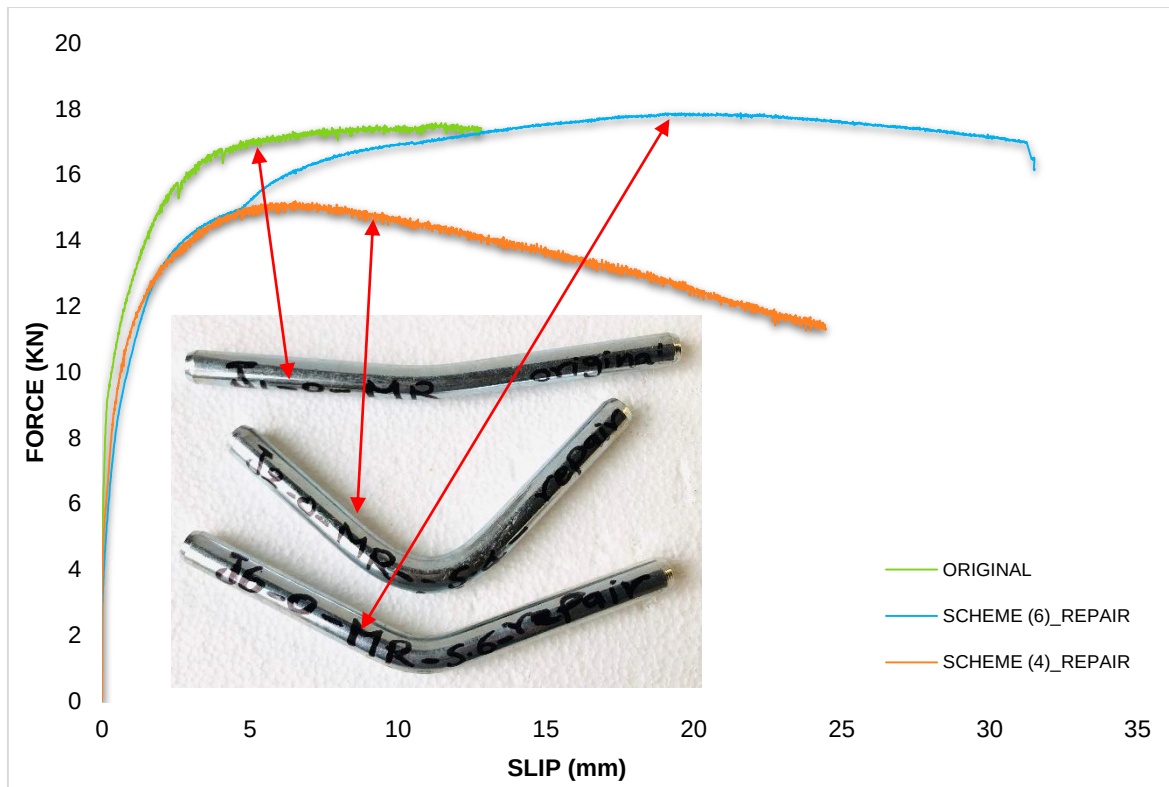


Figure 3:68 - Force-slip curves for the joints, repaired with scheme (6), (4) and the original.

Another factor that influences the results is the natural defects that are detected in maritime joints, such as knots and resin bags. In general, most of maritime pine specimens within this work had knots and resin bags, in different sizes and concentrations. Regarding these joints that were repaired with schemes (4) and (6), both have suffered degradation due to the defects within the timber elements that is believed have influenced the results. In joint J6_O_MR_RP, which was repaired with scheme (6), the knot was in one of the side elements in the vicinity of the dowel. This could restrain the deformation of the dowel, limiting the embedment strength. Although the performance of the repaired joints have improved considerably compared to the original joints regardless of these defects, yet for more certainty of the effectiveness of the repairing techniques on existing joints, more comprehensive experiments are encouraged. Figure 3:69.

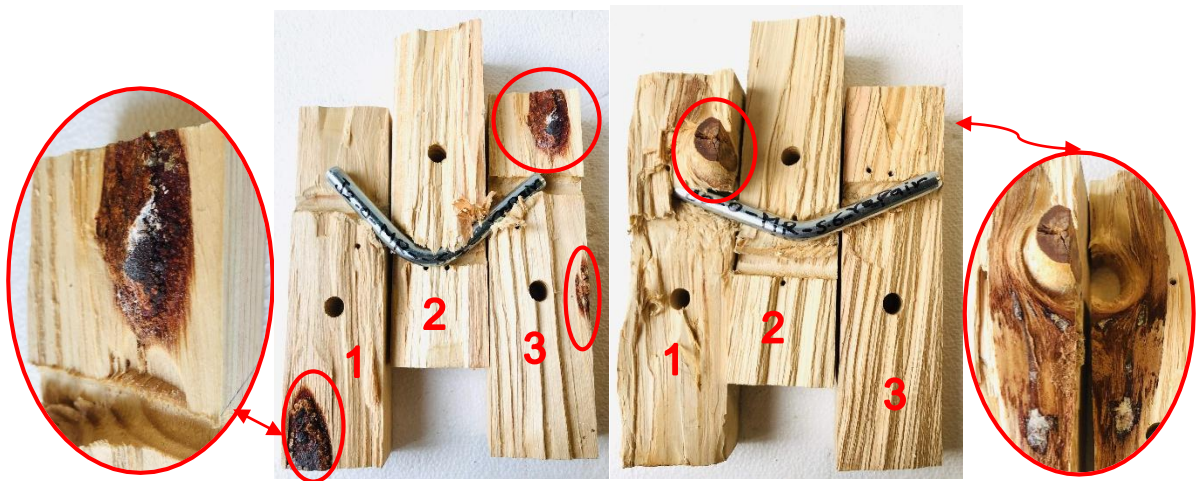


Figure 3:69 - Left: Resin bags in elements no. 1&2_Scheme (4), Right: Large knot in element no.1_scheme (6).

CHAPTER 4

4. CYCLIC TESTS

The second test on the dowel-type joints performed within the scope of this work as a part of the experimental campaign is cyclic testing. It was carried out following the European Standards EN 12512. A new proposal for this standard is currently under revision. The new draft differs from the previous version, EN 12512 – 2005, in terms of testing procedure, protocol, and by means of data processing, proposing different approaches to measure ductility, energy dissipation and strength impairment for the joints. This test is an essential step in the evaluation and design process of dowel-type joints with mechanical fasteners. It is a process of applying fluctuating tension and compression forces to the joints that ultimately leads to fatigue, referred to as degradation fatigue. The behavior of the joint under these fluctuations gives an indication of the joint's ductility and the ability to dissipate energy under seismic loading, which are key parameters in seismic design.

According to EN 12512 – new version, the test shall be carried out with two patterns. The first pattern comprises 5 steps, each step has a number of cycles specified. The first two steps have 2 single full cycles in compression and tension. As for the following 3 steps, each step comprises 3 full cycles in compression and tension. Then for the second pattern, a minimum of 4 steps is specified, each step comprises 3 full cycles. The two patterns shall be performed with two different rates, and the rate is increased for the second pattern. The amplitudes that based on which the rate is specified, are a percentage of the yield displacement obtained from monotonic tests. Thence, to perform cyclic tests, monotonic tests are an essential procedure. However, the value of the yield displacement defined from monotonic tests, $V_{EEEP,m}$, used for cyclic testing in this work is 3 mm. Details and comparison between both standard versions, in addition to calculations of this parameter is included in chapter 4. A total of 15 amplitudes for each joint was defined. Table 4-1 and Figure 4:1 shows the protocol and the procedure as it is defined in EN 12512 – new version.

Table 4-1 - Amplitudes for the reversed cycles [7].

<i>pattern</i>	<i>Step</i>	<i>Number of cycles</i>	<i>Amplitude</i>
1	1	1	$0.2 V_{y,EEEP,m}$
	2	1	$0.4 V_{y,EEEP,m}$
	3	3	$0.6 V_{y,EEEP,m}$
	4	3	$0.8 V_{y,EEEP,m}$
	5	3	$1.0 V_{y,EEEP,m}$
2	6	3	$1.5 V_{y,EEEP,m}$
	7	3	$2 V_{y,EEEP,m}$
	8	3	$3 V_{y,EEEP,m}$
	9	3	$4 V_{y,EEEP,m}$
	...		
	$j-1$	3	$(j-6) V_{y,EEEP,m}$
	j	3	$(j-5) V_{y,EEEP,m}$
	$j+1$	3	$(j-4) V_{y,EEEP,m}$

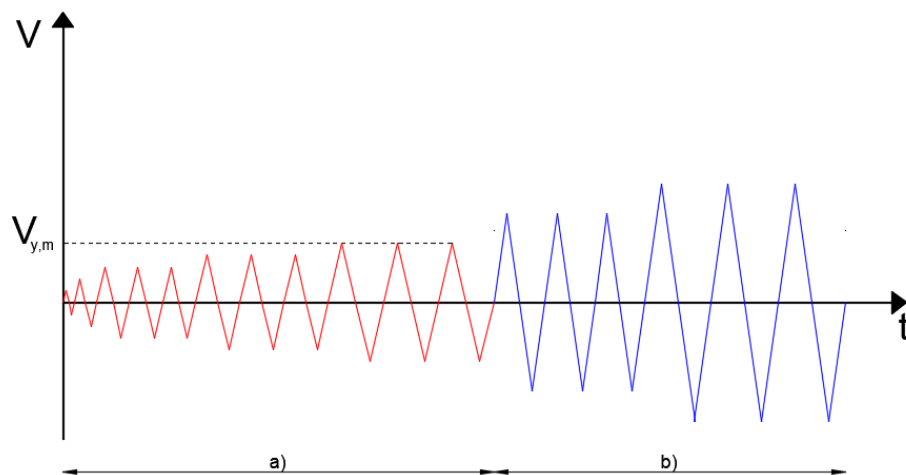


Figure 4:1 - Two patterns of cyclic loading in terms of Time-displacement [7]

The displacement rates for the amplitudes in the new proposal are yet to be specified, as this part is currently under discussion and no values are provided. Thus, the tests were performed referring to an experimental campaign that was carried out in University of Minho on angle-bracket joints. The rates adopted for that campaign were 0.275 mm/s for the first pattern, and 1.25 mm/s for the second pattern. Nonetheless, since the yield displacement, $V_{y,EEEP,m}$, in this work is different, the rates were then adjusted accordingly to a final value of 0.5 mm/s for the first pattern, and 1 mm/s for the second pattern. The detailed protocol adopted for the cyclic testing within this experimental campaign is provided in Annex 1.

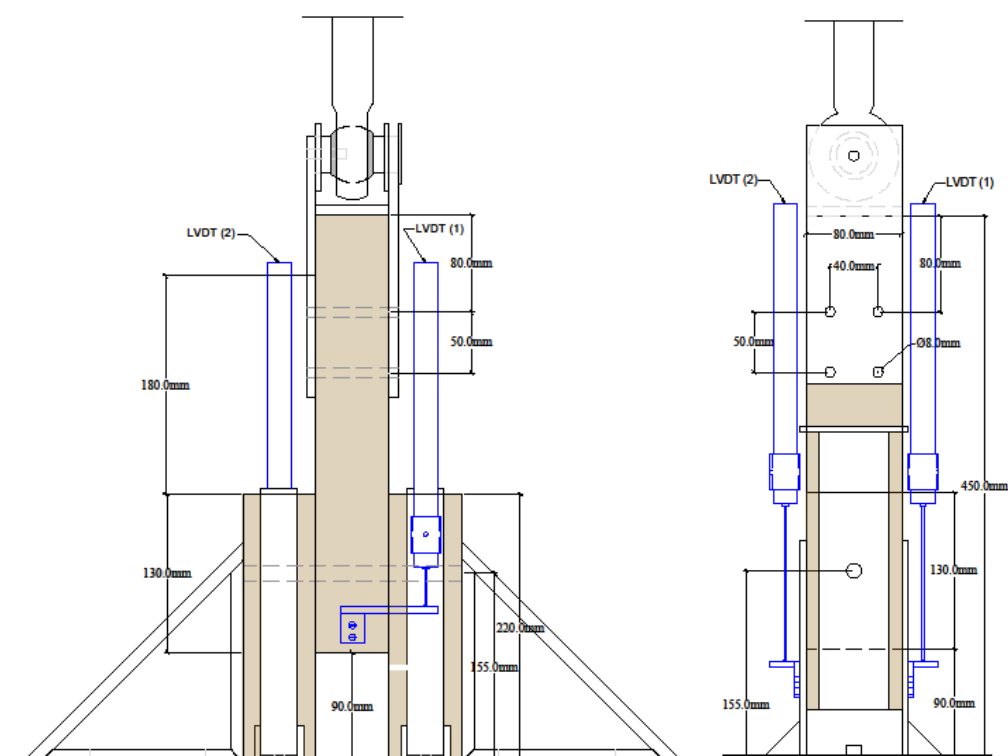


Figure 4:2 - Cyclic test apparatus

Regarding the test apparatus, the machine for testing was set to comply with the specifications in the standards. An electro-mechanic test machine equipped with an actuator of a 150 kN capacity, was used to apply the load with an accuracy of ± 1 , in addition to two linear variable differential transformers (LVDTs) with a range of ± 25 and ± 50 mm, fixed on both of the side elements. In order to accurately record the slip of the joint under cyclic loading, a proper fixation of the joint in the test apparatus was required. Thus, the joint was fixed on both sides to prevent horizontal movement during the test, therefore, ensuring that the dissipated energy measured after testing is resulting only from the applied compression and tension loads, and no dissipation due to horizontal movement is encountered. Over and above, the middle element of the joints for cyclic testing had a different geometry compared to monotonic samples. The geometry had to be adjusted to 80X60X360 mm, to allow for the connection with the actuator, which is connected to the middle element by fixing a metal piece, drilled with 4 $\varnothing$ 8 mm bolts on top of the element that is connected to the actuator with $\varnothing$ 30 bolt. Figure 4:3 shows a joint fixed in the test apparatus upon testing.

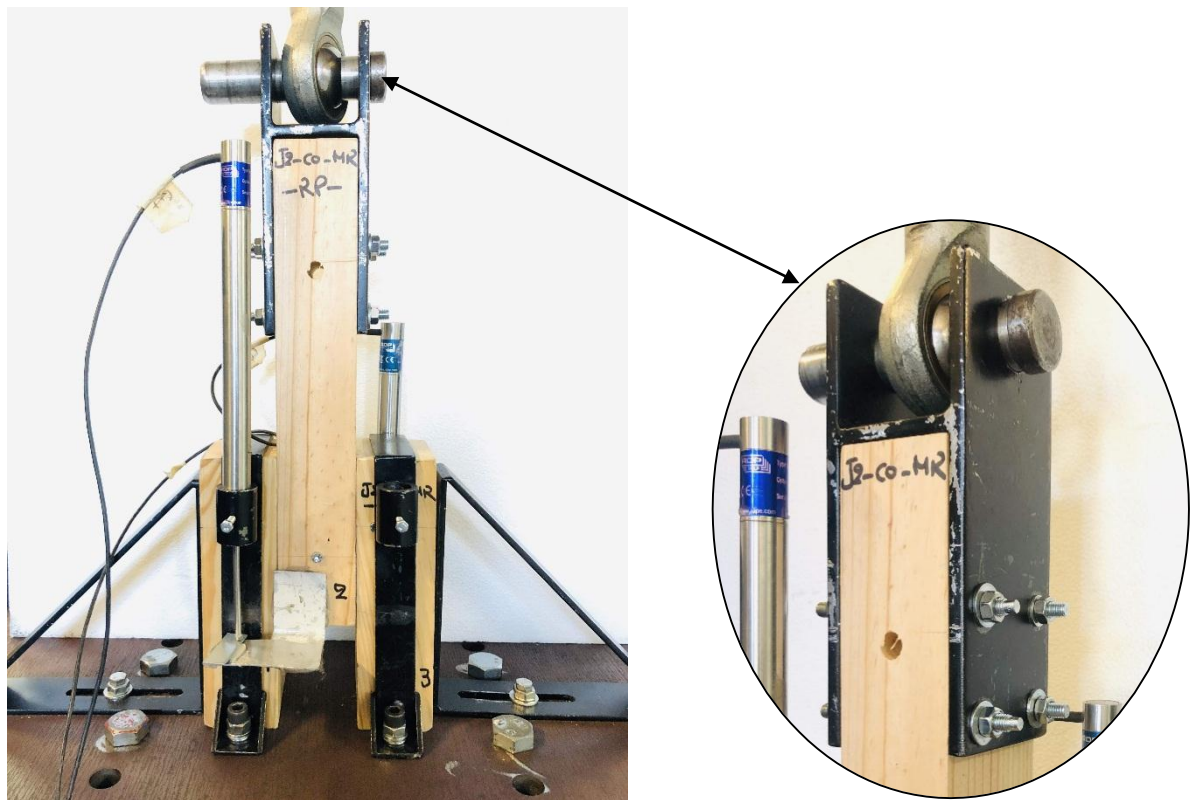


Figure 4:3 - An example of a joint fixed for testing

4.1 ORIGINAL JOINTS

4.1.1 SCOTS PINE

The test was carried out on 6 non-reinforced joints as a start, prior to introducing any type of reinforcement. All of the joints have reached a total failure by splitting, mainly in the middle element, and the test stopped before the 15 steps were completed. Although the failure was brittle in all 6 joints and the embedment on the joints was found to be on the side elements, yet the dowel has developed a plastic hinge in the middle element. The failure mode was considered accordingly as mode (j) ^[3] despite that the plastic hinge has not fully developed. Figure 4:4 shows the force-slip curves for all 6 joints. Figure 4:5 shows the typical failure mode on the joints, presented with joint J6_CO_SC, as well as the time-displacement curve of that joint. The curve shows that the joint have failed on the 12th cycle.

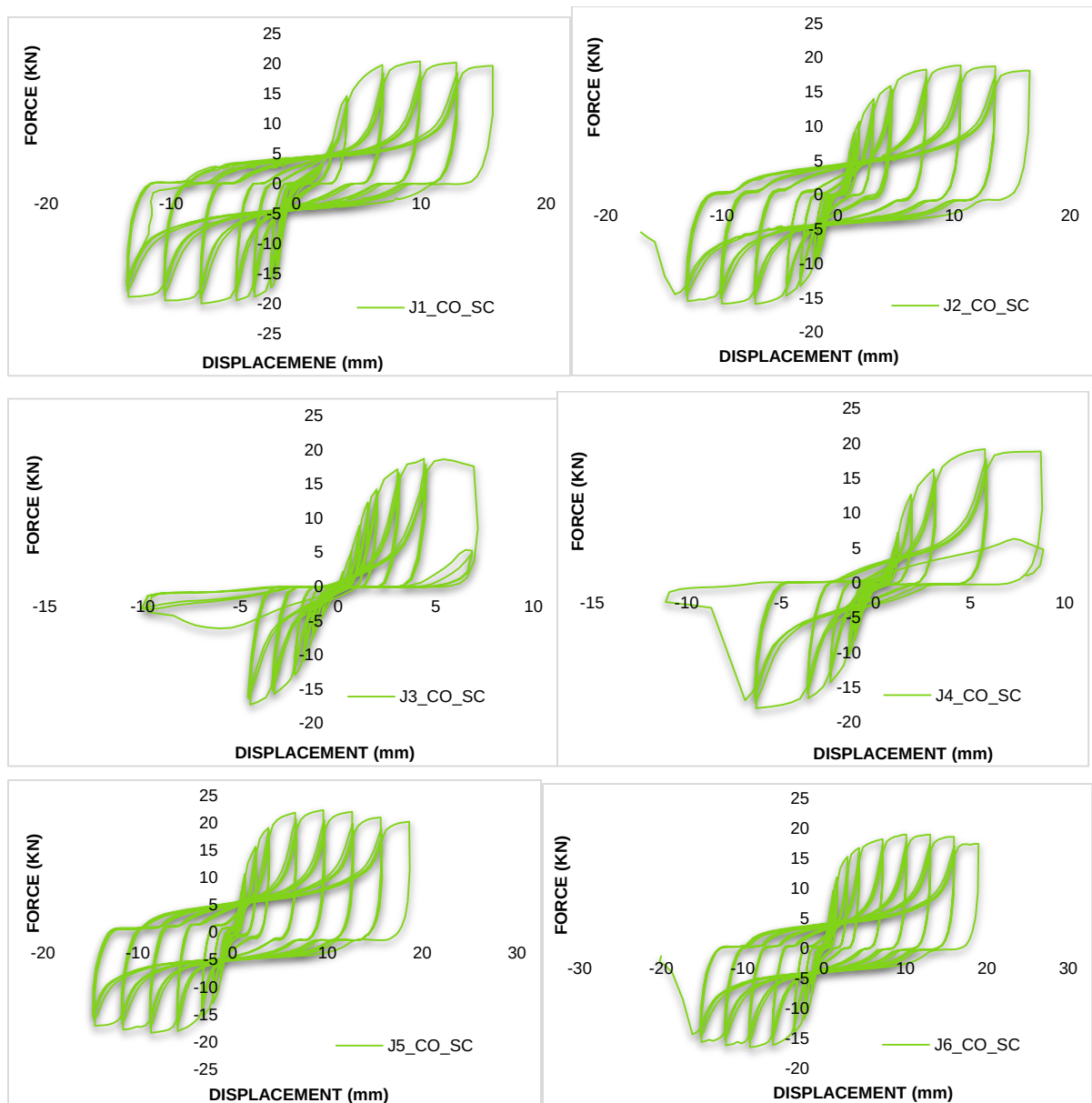


Figure 4:4 - Force-Displacement curves for 6 original joints-Scots pine

It can be noticed from the force-displacement curves above that the joints have reached their maximum capacity before completing the 15 cycles. The number of total cycles completed differs from one joint to the other. Referring to the curves of Joints 3 and 4, it can be seen that the behaviour of the two joints was more brittle. The joints had a total splitting of the middle element resulted in the termination of the test. The joints reached their ultimate tensile capacity before failing in compression, thus, the last cycle in tension shows an abrupt drop in force. The joints have also shown a pinching behaviour that is more visible in some of the joints. Figure 4:5.

The movement of the dowel in response to the applied load causes the dowel to embed into the surrounding fibres causing the diameter of the hole to increase and the hole to expand in the direction of the load. Thus, when the direction of the load changes from compression to tension or vice-versa, the

dowel moves in the empty space caused by the previous embedment prior being in contact with timber once again. The displacement resulting from this movement is referred to as the pinching effect. It is quite expected to encounter this kind of behaviour with cyclic loading, as the damage on the dowels vicinity and the expanding diameter of the hole can only be measured by LVDTs if the element is loaded in opposite directions. However, based on that, the pinching effect is expected to increase with the number of cycles, which can be seen in the capacity curves shown above.

The encounter of this effect differs also between the joints, as in some joints, such as 3 and 4, the effect is more evident in the last cycles. While in the 1st joint, this effect is recorded from the beginning of the test. Since the joint was tested for the first time, the only explanation to the pinching effect occurring within the first cycles of an original joint is that the diameter of the hole for the dowel was equal to or larger than the diameter of the dowel itself. Therefore, it is very important to control the fabrication process of the joints in order to avoid such behaviour. In general, the joints had higher capacity in compression than in tension, and compared to the capacity reached by monotonic tests for the same species. The maximum compression force, 22.25 KN which was reached by joint 5, is higher than the compression force obtained by monotonic tests that the maximum of which was 20.34 KN.

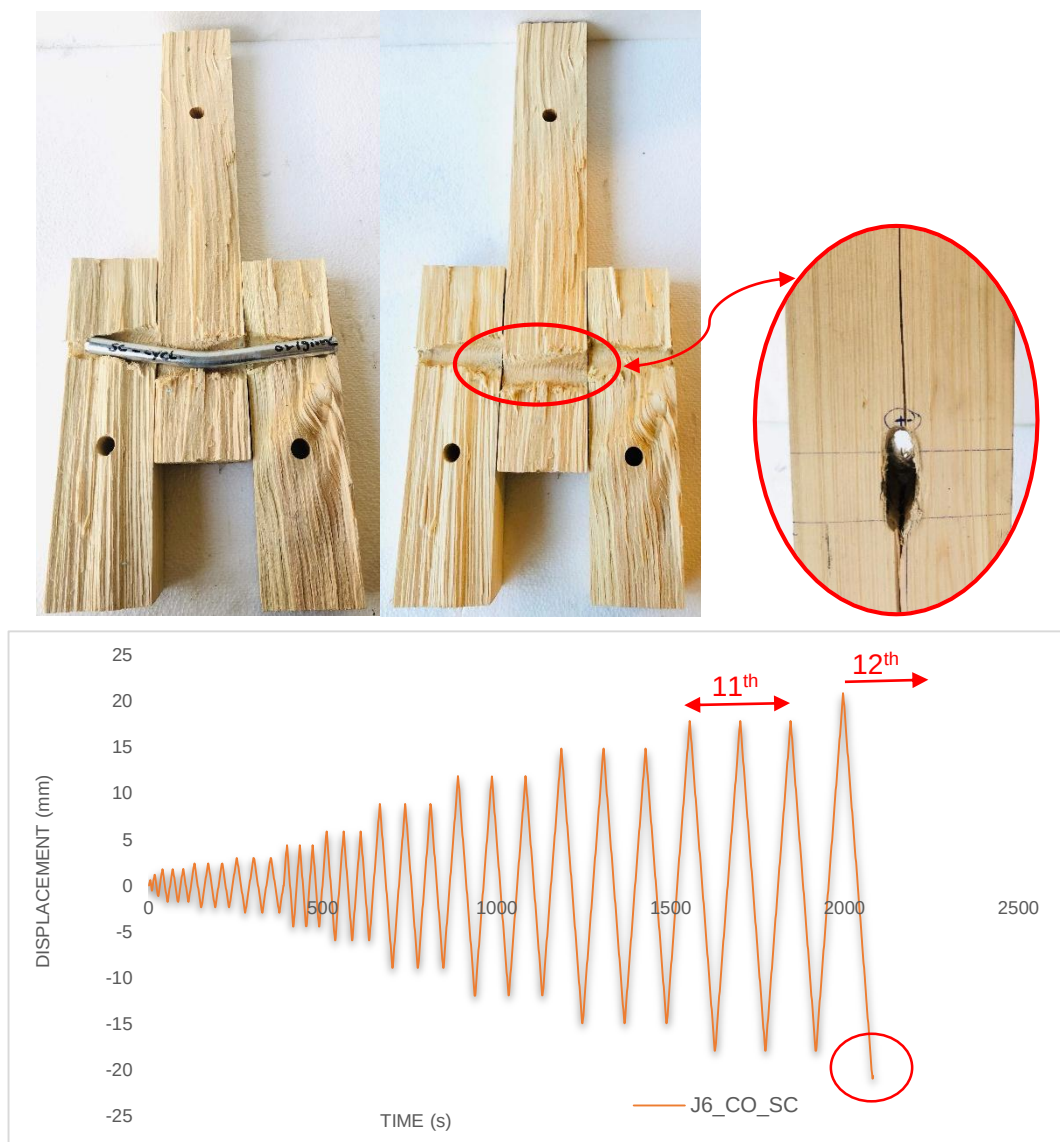


Figure 4:5 - Top: Example of typical failure mode of original joints shown by J6_CO_SC, Bottom: Time-displacement curve of the same joint.

4.1.2 MARITIME PINE

The same procedure was followed for maritime pine joints, as 6 joints were tested without reinforcement in order to determine the most suitable reinforcing and repairing techniques based on the joints behavior under cyclic loading and the failure encountered accordingly. Similar to scots pine joints, the dominant failure pattern was the splitting of the middle element, in addition to cracking of the side elements in some of the joints. However, these joints have sustained more cycles in general compared to scots pine joints, which is expected giving that the capacity of maritime pine wood in general and the density are higher. The first 3 joints have completed the full 15 cycles, but both failed in embedment despite the fact that the timber elements of the 2nd joint were completely sound without any cracks or splitting after testing. Figure 4:6.

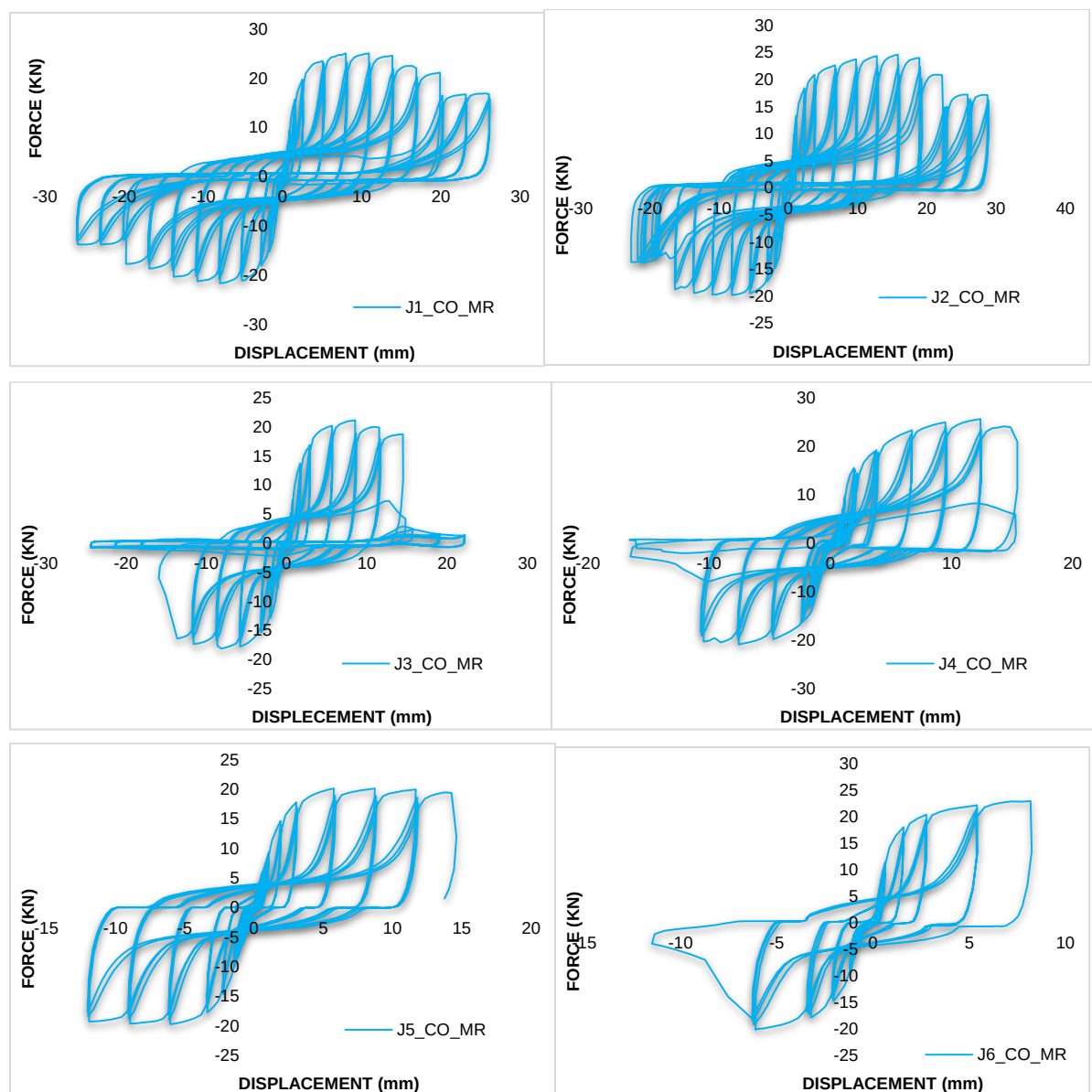


Figure 4:6 - Force-Displacement curves for 6 original joints-Maritime pine

Referring to the capacity curves displayed above, it can be noticed that the behaviour is similar to scots pine joints as the joints sustained more cycles in compression before breaking abruptly in tension. In addition to a large pinching effect that is more visible around the last cycles. The joints have failed in embedment with failure mode (g) ^[3], which is the first failure mode that corresponds to embedment in the side elements without deformation on the dowel. Figure 4:7.

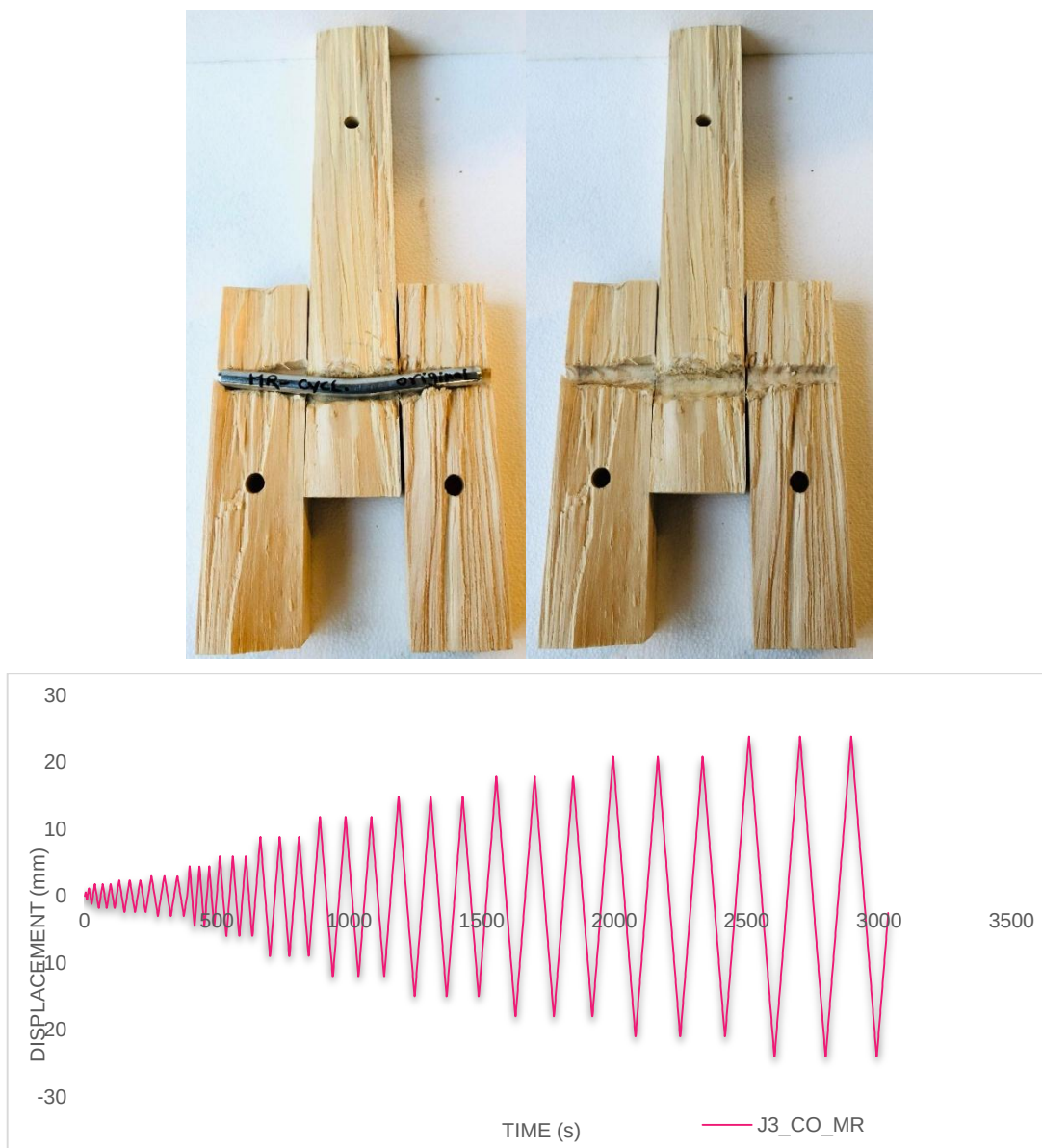


Figure 4:7 - Example of typical failure mode of original joints shown by J3_CO_MR, Bottom: Time-displacement curve of the same joint.

4.2 REINFORCED JOINTS

4.2.1 SCOTS PINE

A total of 3 joints were reinforced with scheme (8) that was chosen in light of results of monotonic testing, in order to measure the joints' ductility and energy dissipation when reinforced with STS while being subjected to cyclic loading. HBS screws were used for reinforcement, similar to monotonic tests. Figure 4:8. The reinforcement has increased the capacity and the ductility of the joints, as 2 of the joints have completed the 15 cycles without splitting. The average maximum force obtained for compression is 22.3 kN which represents an increase of 11.50% in compressive capacity compared to the original joints. Figure 3.76 shows the force-slip curve for one of the reinforced joints, J1_CR_SC, which

completed the 15 cycles with less splitting in comparison with the original joint shown in Figure 4:7 above, that completed only 11 cycles with brittle failure and splitting in the middle element throughout the length of the specimen, below and above the dowel.

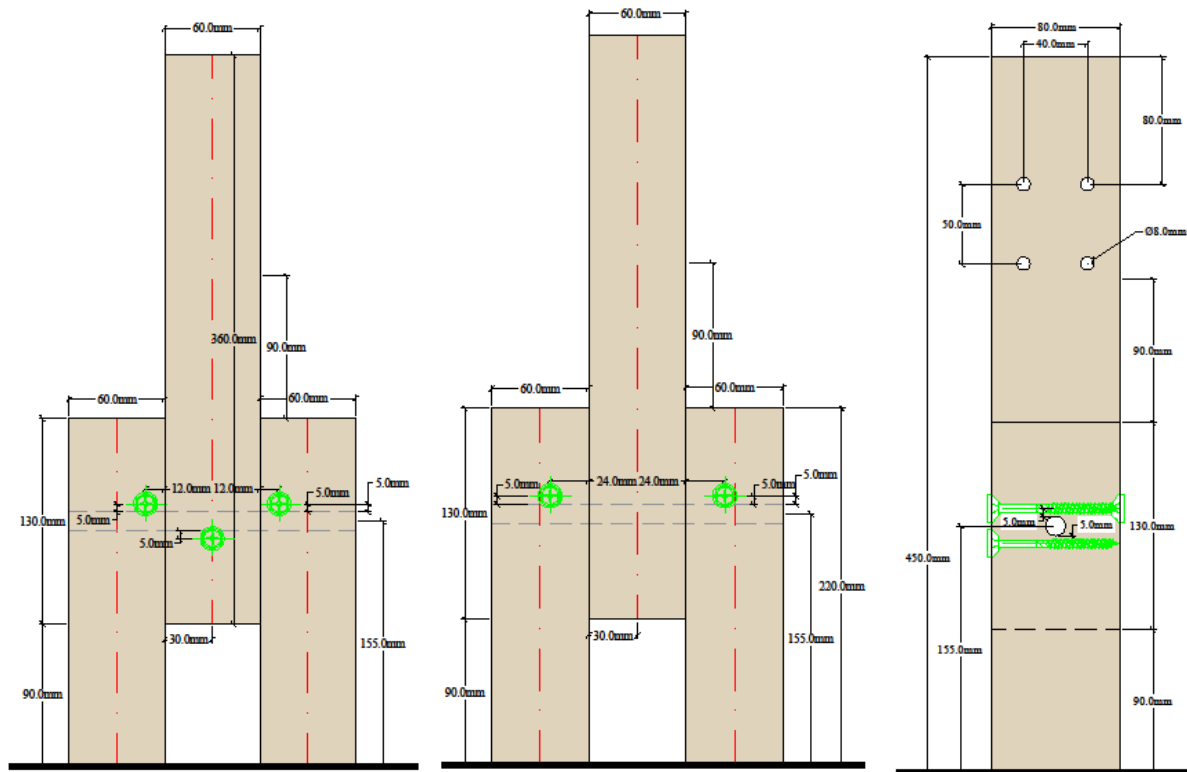


Figure 4:8 - Reinforcement scheme (8), Left: Front elevation, Middle: Rear elevation, Right: Side elevation.

The reinforced joint showed also splitting in the middle element, but the effect is much less, as the element was split only beneath the screw and the test was finished before the crack could prevail to the top. This geometry of the joint plays an important role in preventing the splitting of the elements in terms of end distance tolerance and requirements. In these joints, the end distance parallel to the grain to the screw in the middle element is small, but the necessity of comparison with FRP reinforcement in this work has the geometry adopted was not adjusted.

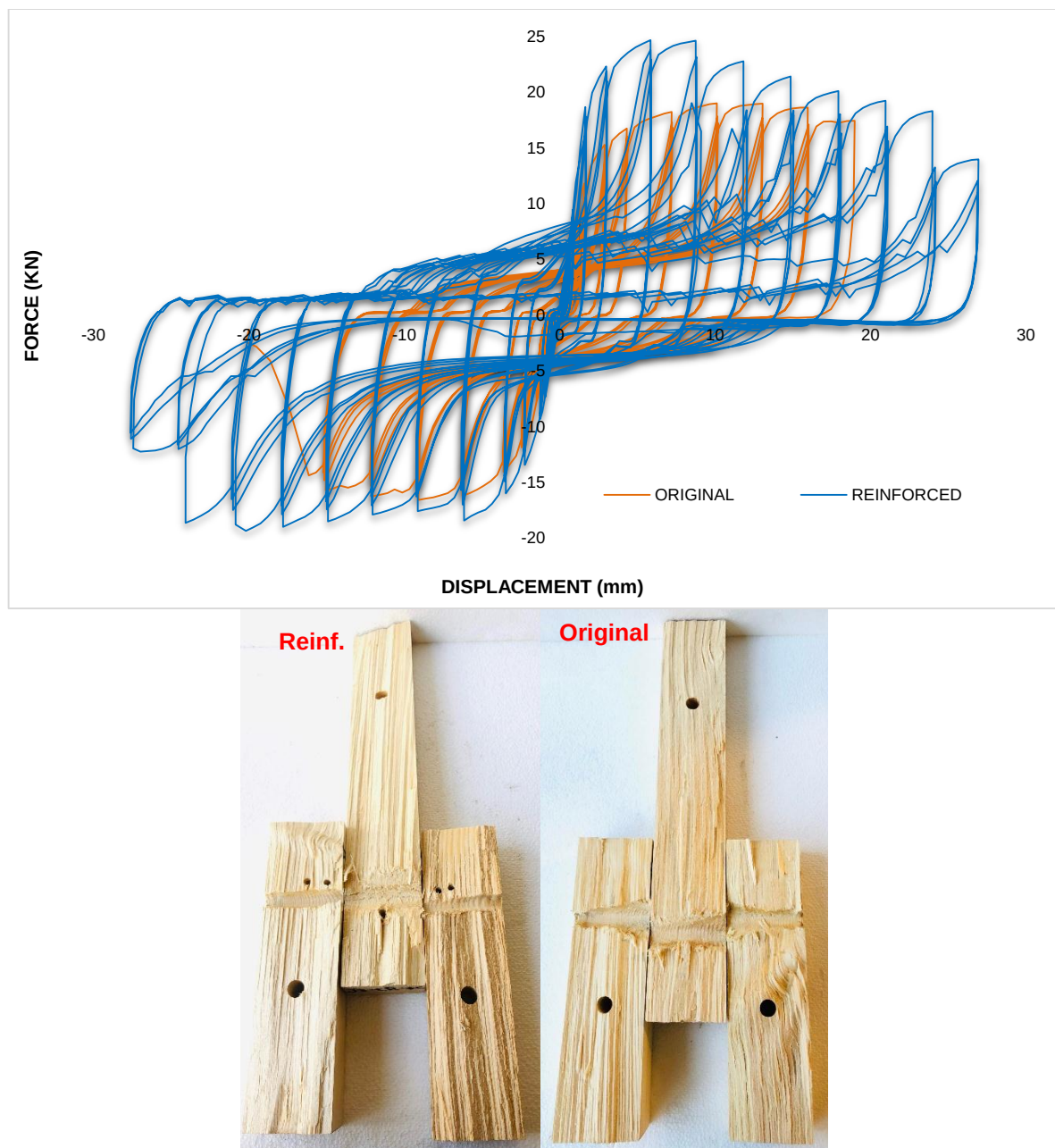


Figure 4:9 - Top: Force-slip curve of a reinforced and original joints, Bottom: The failure in the two joints after testing.

The failure obtained is mode (g) ^[3], which corresponds to embedment failure that is encountered usually when the joint is not ductile and the failure occurred before the dowel reaches its yield capacity. Nevertheless, although the embedment strength in the original joint seems to be higher, yet the capacity and the ductility of the joints have increased with reinforcement as it can be seen from the capacity curves.

4.2.2 MARITIME PINE

The same procedure was followed for maritime pine joints. A total of 3 joints were reinforced with HBS screws, following scheme (8) for reinforcement shown in Figure 4:10 similar to scots pine joints. Unlike original joints, the 3 reinforced joints have completed all 15 cycles with no splitting in the elements. The failure obtained is mode (j) ^[3], with embedment in all 3 elements and one plastic hinge in the dowel in the middle element. Table 4-1 - Amplitudes for the reversed cycles ^[7].

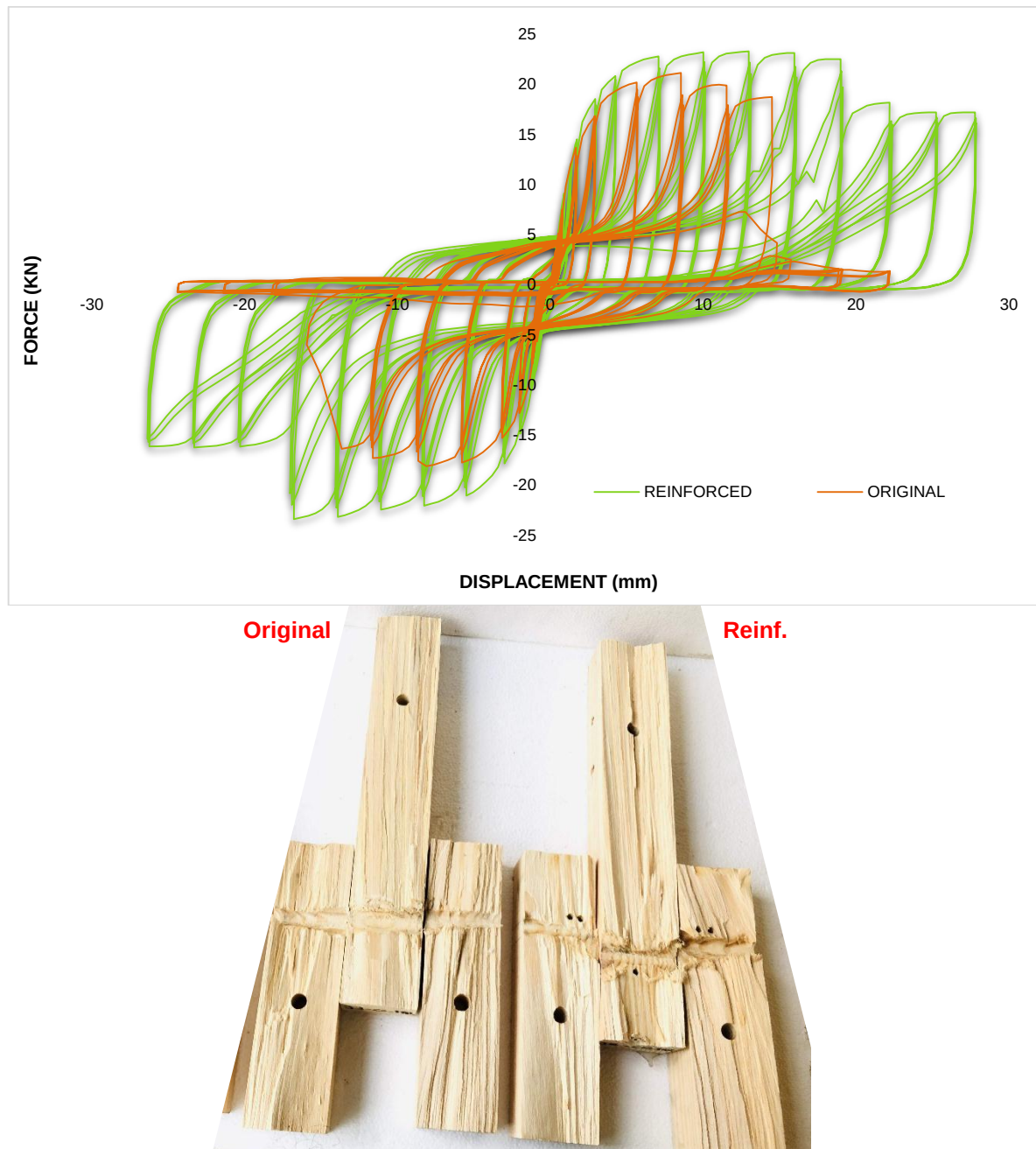


Figure 4:10 - Top: Force-slip curve of a reinforced and original joints, Bottom: The failure in the two joints after testing

The capacity of the joints compared to the original have not increased significantly, 2.66 %, yet the effect of the reinforcement can be seen in terms of ductility. The pinching effect has reduced considerably as well as the force impairment.

4.3 REPAIRED JOINTS

4.3.1 SCOTS PINE

Referring to the failure pattern in the original joints, splitting was the dominant case. Therefore, repairing scheme (5) with inclined STS would not be effective nor suitable for this case. Similar to the case of repairing maritime monotonic joints, scheme (6) for repairing was adopted for the cyclic joints. The contribution of the two screws placed in the middle element, above and below the dowel, is more suitable for cyclic joints, as the splitting tends to prevail much faster under cyclic loading. Furthermore, positioning the screws at a distance of 10 mm from the dowel, opposed to the repaired monotonic joints, was carried out deliberately in order to stop the splitting. Figure 4:11 shows scheme (6) schematic for cyclic joints. A total of 3 joints were chosen to be repaired with this scheme and tested for cyclic loading, joints 2, 3 and 4. Joint 1 was excluded from repairing due to the total failure of the joint.

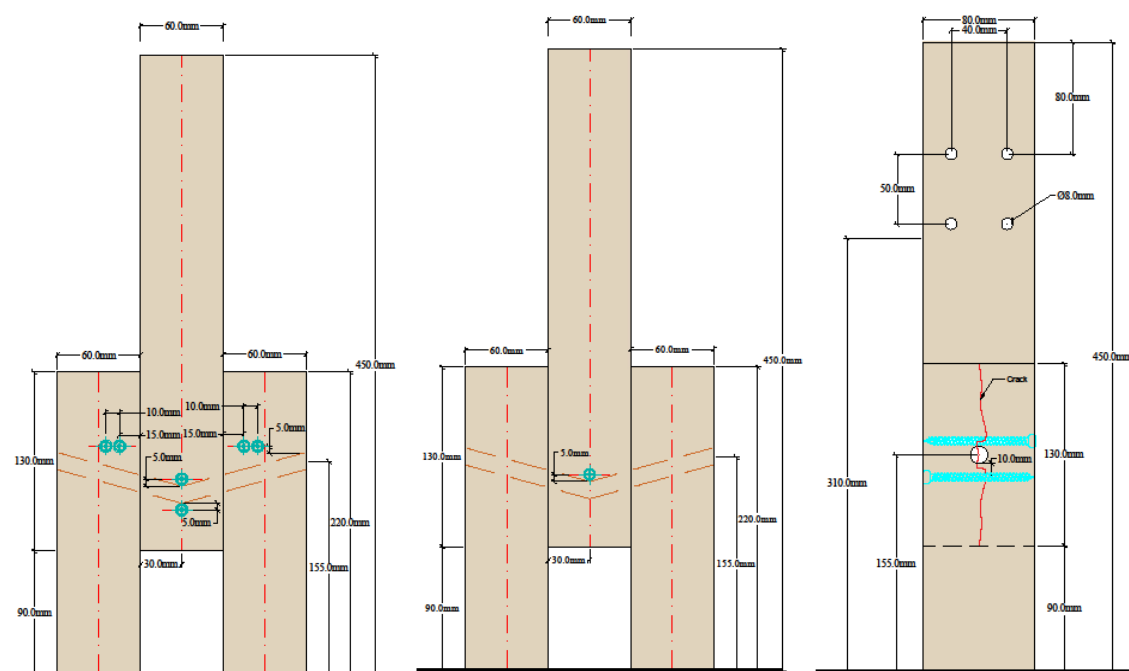


Figure 4:11 - Repairing scheme (6), Left: Front elevation, Middle: Side elevation, Right: Rear elevation.

The failure encountered with this technique corresponds to mode (h) ^[3] with embedment in the middle element although the dowels had developed a plastic hinge in the middle elements that are not fully developed. The joints have sustained more cycles than the original joints. It can be noticed from the force-slip curves for both the original joints and the repaired, that the ductility of the joints have increased

significantly, as well as the stiffness. Regarding the pinching effect on the repaired joints, it is expected to increase due to the repetition of the test. Nevertheless, a large percentage of the capacity was restored and the impairment of strength was decreased. Figure 4:12 shows the force-slip curves obtained by each joint original and after repairing, and Figure 3.80 shows the failure on the joints after testing.

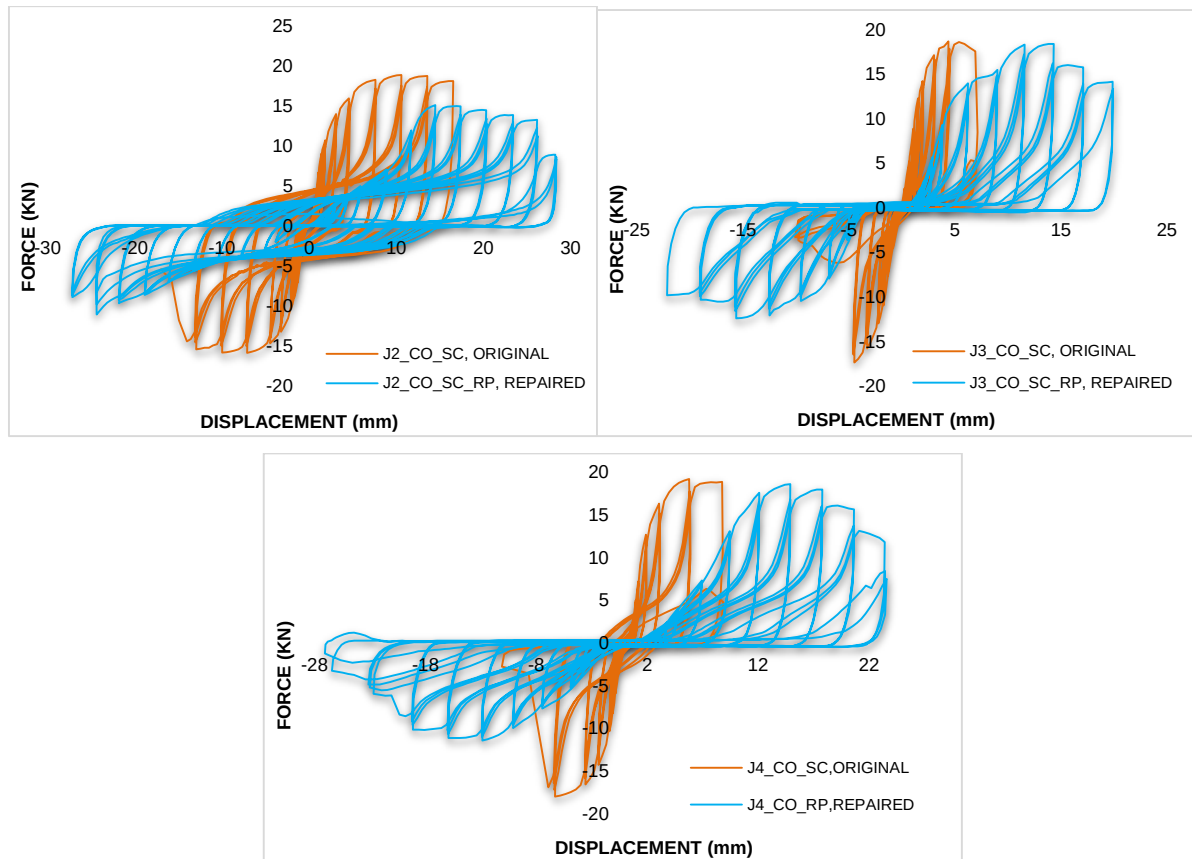


Figure 4:12 - Top: Force-slip curves of repaired joints vs. the original



Figure 4:13 - Left: Dowel's deformation, Right: Screw deformation.

As mentioned above that the screws were placed at a distance from the dowels and that is for a purpose of controlling the splitting of the element. The crack on the middle element has not developed after repairing.

4.3.2 MARITIME PINE

Similar to scots pine joints, out of 6 original joints, 3 were chosen to be repaired. The same scheme for repairing was used on joints 2, 5 and 6. The joints were chosen based on their status after the first test. For example, the original joint no. 3, J3_CO_MR, had an ultimate failure and the crack on the middle element had reached the top of the element which caused total separation.

The joints have all completed the 15 cycles within the protocol, yet the results of these 3 three joints, unlike scots pine joints, are different in terms of failure. Joints J2_CO_MR_RP, had lower capacity after testing despite that the original joint after the first test did not experience any damage in terms of splitting and cracking, only failed on embedment after the 15 cycles. However, the capacity of the repaired joint has increased abruptly in the last cycle, after a plateau for the first cycles, to reach the capacity of the original joint in the last cycle in compression and tension. The sudden increase in the joint's capacity is due to the contact of the dowel with the screw that is above the dowel. The screws were placed at a distance from the dowel as in the previous joints, but the large pinching resulted in more movement of the dowel. This contact have caused a rigid-plastic behavior for the screw that resulted in its deformation. A similar behavior in terms of failure and screws' deformation was encountered also for joint and J5_CO_RP, Yet different results were obtained in terms of capacity and ductility. The original joint sustained 10 full cycles ad failed on the 11th with a failure mode (g) [3]. On the other hand, after repaired, the joint completed all 15 cycles before failure, and the failure mode obtained after repairing was mode (j) [3], with a hinge developing in the dowel within the middle element.

Regarding the third tested joint, the middle element had developed a crack below the dowel after the first test and the failure mode obtained was mode (g) [3] as well. The original joint had only completed 8 full cycles before failing on the 9th. After STS were introduced as a solution for repairing, the joint for the second test has completed the test, 15 full cycles, before failure was reached. The resulting failure mode was (j) [3] with a fully developed hinge in the middle element. Over and above, the screws were able to stop the crack from extending and prevented full splitting of the timber element. Figure 4:15 shows the force-slip curves compared to the original joints. Figure 4:15 and Figure 4:16 shows joints 5 and 6 in comparison with the original, and the deformation of the screws in joints 5 and 2.

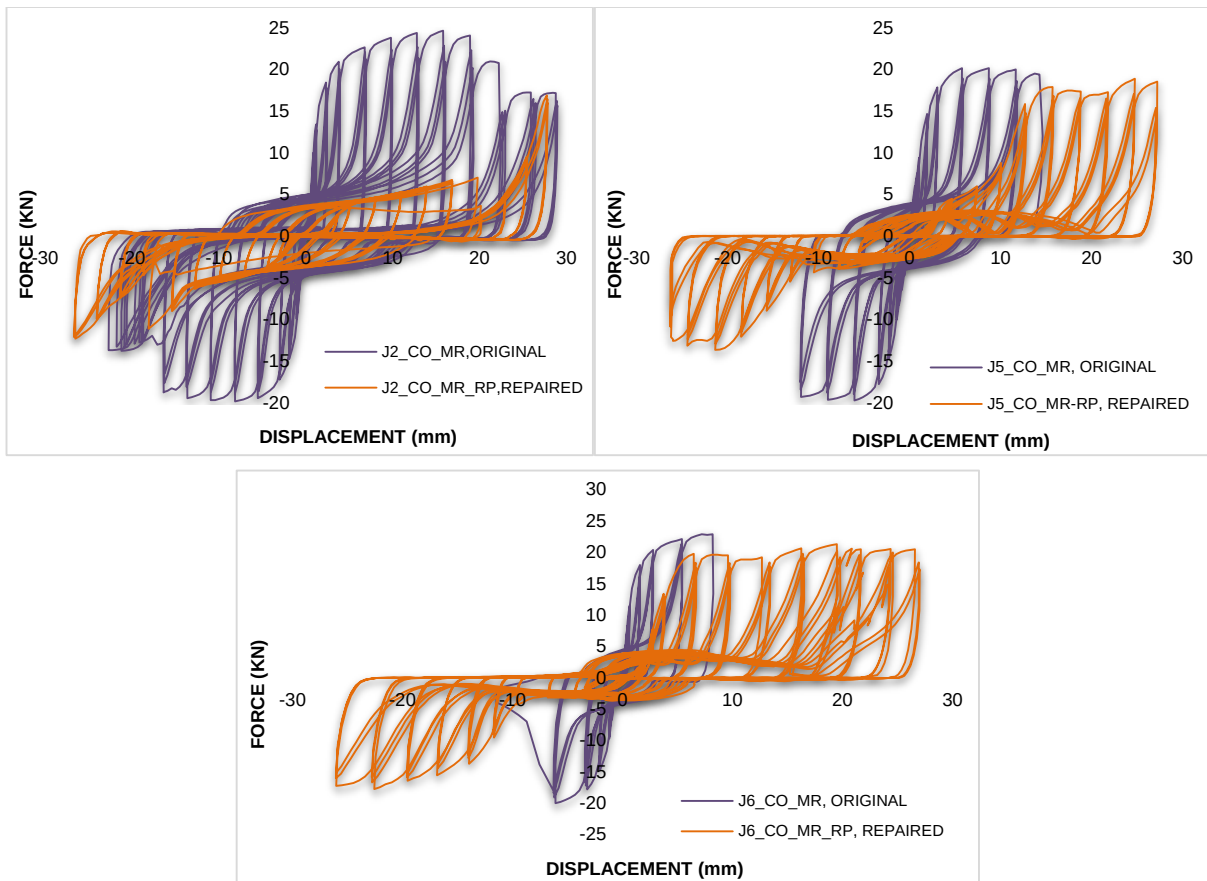


Figure 4:14 - Force-slip curves of repaired joints vs. the original.



Figure 4:15 - Joints 5 and 6 opposed to original.

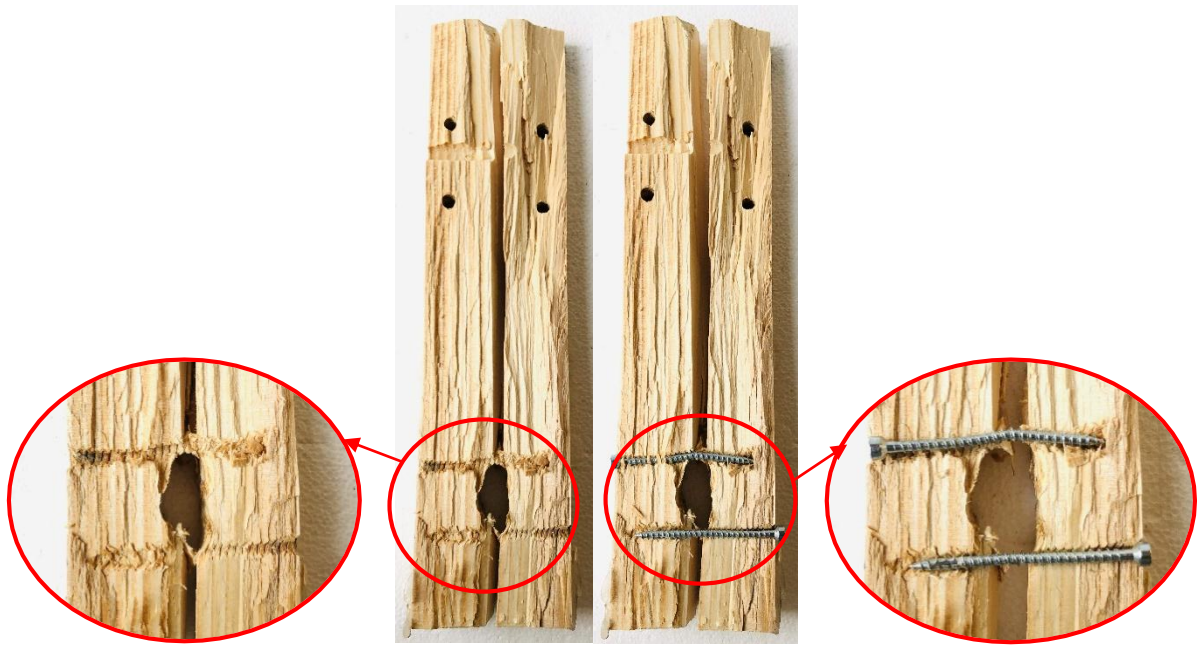


Figure 4:16 - Deformation of the STS in joint 5, and its embedment in the timber element.

CHAPTER 5

5. ANALYSIS AND DISCUSSION

5.1 DENSITY

After stabilizing the timber elements and prior to testing, a destructive density test was carried out according to the specifications provided within the International Standards ISO 3130. Three joints of Scott's Pine with 12 mm dowel, three from Scott's Pine joints with 8 mm dowel, and finally three Maritime Pine joints were chosen to provide the density samples. Two density test samples, 20 X20 X25, were taken from each element in every joint. Thus, a total of 54 specimen were taken from the selected samples after removing the dowels and the screws. The samples were measured as a first step and then placed in an oven in the - Laboratory of Structures of the University of Minho (LEST) – at a temperature of $[\pm 100.5^{\circ}\text{C}]$.

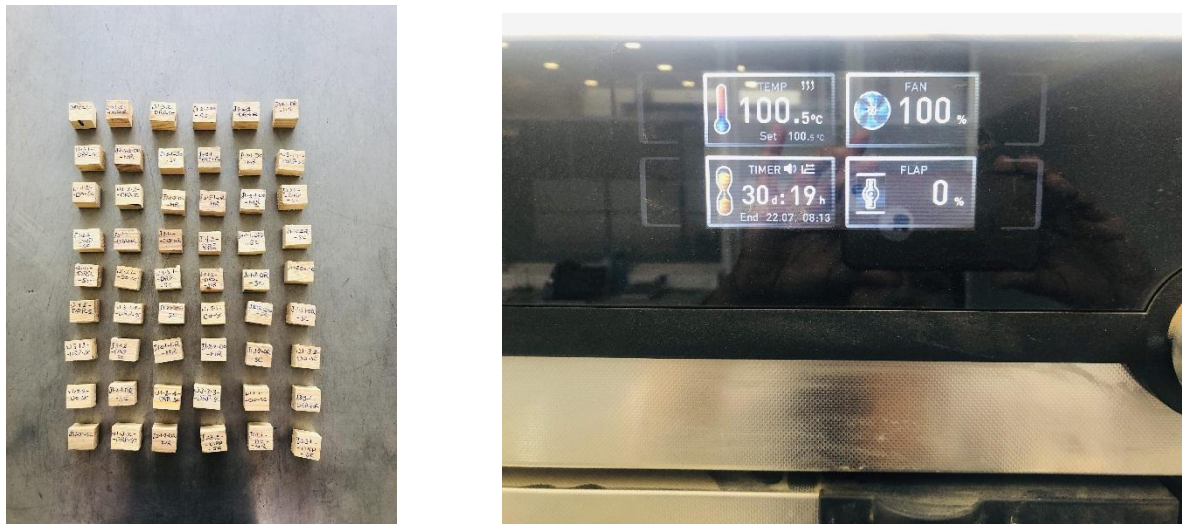


Figure 5:1 - Density test samples.

The specimen were weighed and measured every 6 hours until a stabilization was obtained. According to the standards, the density samples are considered stabilized when the variation in the mass between two consecutive readings is less than 0.5%. The moisture content obtained after stabilization was 10 % and the density was obtained accordingly following the expression below:

$$W = \frac{m_1 - m_2}{m_2} \cdot 100 \quad (5-1)$$

Where:

m_1 is the density before drying.

m_2 is the mass after drying; and

W is the water content.

Table 4.1 shows the final density values for all wood specimen in this work accordingly.

Table 5-1 - Mean density values.

Wood Specimen	ρ (kg/m ³)
Scott's Pine (8 mm dowel joints)	471.96
Scott's Pine (12 mm dowel joints)	433.31
MARITIME PINE	556.06

5.2 MECHANICAL PROPERTIES OF TESTED JOINTS

5.2.1 REINFORCEMENT OF SELF-TAPPING SCREWS (STS)

5.2.1.1 PROCEDURE

Analyzing the dynamic behavior of the joints under cyclic loading is an essential step in estimating the mechanical properties of dowel-type joints. The embedment strength of the timber elements, the yield properties of the dowel as well as the ability to dissipate energy are all factors that contributes to the dynamic behavior of the joint subjected to seismic loading.

The entire testing procedure to determine the dynamic behavior under seismic loading that is proposed according to EN 12512 – new version is conducted in two main phases. The first phase is monotonic testing, and the second phase is cyclic testing. To start cyclic testing, the amplitudes of the cycles and the displacement rates are ought to be determined beforehand. These amplitudes are a function of the monotonic yield displacement of the joints that is obtained from the monotonic capacity curves, and as mentioned earlier in chapter 3, the monotonic tests were carried out according to EN 26891. However, the procedure in the new proposal quantifies the yield displacement, which is referred to as the elastic displacement in other occasions, by determining the turning point between the elastic and plastic phase. This point along with the corresponding force are determined by equating the area under the force-slip curve to the area under an ideal elastic-plastic curve. This curve is referred to as the Equivalent Energy Elastic-Plastic curve (EEEP). The elastic part of this curve represents the stiffness of the joint. Referred to as $K_{EEEP,m}$ and $K_{EEEP,c}$ for monotonic and cyclic tests, respectively. The stiffness is determined by a line that representing the elastic modulus that corresponds to 10% and 40% of the maximum force. Then the plastic phase that is represented by a horizontal line, is found by equating area under this ideal curve with the area under the original force-slip curve. The intersection of these two lines defines the limit between the elastic and the plastic phase and is referred to as the equivalent energy elastic-plastic yield force, $F_{y,EEEP,m}$ and $F_{y,EEEP,c}$ for monotonic and cyclic tests, respectively. Consequently, the

displacement corresponding to this force is the equivalent energy elastic-plastic yield displacement, $V_{y,EEEEP,m}$ and $V_{y,EEEEP,c}$ for monotonic and cyclic tests, respectively. Therefore, in order to obtain the EEEP curve for the cyclic tests, the EEEP curve for monotonic tests should be determined in advance, as the yield displacement of the monotonic tests sets the amplitudes for cyclic tests, as mentioned earlier. Regarding the EEEP curve for monotonic tests, it is obtained by determining the ultimate displacement reached by the test piece, $V_{u,m}$, which according to the standards is limited by either the displacement at failure, the displacement that corresponds to 80% of the maximum estimated force, or 30 mm, whichever occurs first. In addition to the maximum force, $F_{max,m}$, that corresponds to the maximum force obtained for a value of displacement less than the ultimate displacement described above. The resulting final EEEP monotonic curve is shown in Figure 5:2 below.

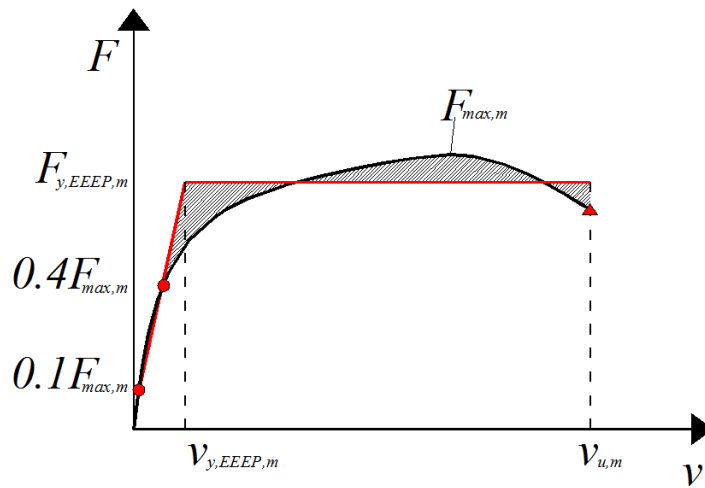


Figure 5:2 - Equivalent Energy elastic-plastic curve (EEEEP) for monotonic tests [7].

Following this phase, the EEEP curve for the cyclic tests as well should be obtained. Regarding cyclic tests, the results are in a form of a force-displacement curve representing each step with its specified amplitude that includes 3 cycles each. Thus, in order to determine the EEEP curve for cyclic tests, the results of these cycles are ought to be translated into another linear curve that is referred to as the Load Envelope Curve (LEC). This curve is obtained by connecting the maximum forces reached in each step. Since each step consists of 3 fully reversed cycles, then 3 load envelope curves (LEC) can be obtained for each joint, both in compression and tension. Nevertheless, only the two LEC curves that correspond to the first and the third cycles are considered.

These curve are a measurement of the force impairment of the joints. In this new version of the standard EN 12512, the force impairment of the joints is referred to as the strength degradation factor, $K_{deg}(v)$, which is calculated following this expression:

$$k_{deg}(v) = \frac{F_3(v)}{F_1(v)} \leq 1 \quad (5-2)$$

To calculate the degradation of the force in each step, the forces $F_1(v)$ and $F_3(v)$ of the first and third cycles, respectively, should be determined at the exact value of displacement, for every point along the two curves. In this work, these values were obtained by linear interpolation using MATLAB software. However, The EEEP for cyclic tests, EEEP,c, is finally calculated for the LEC that corresponds to the first cycle, 1st LEC, following the same procedure for the monotonic. The ultimate cyclic displacement in this case, $V_{u,c}$ is determined by means of the minimum between the displacement that corresponds either to failure, 80% of the maximum force of the 1st LEC or 30 mm.

This procedure aims at quantifying then ductility and the energy dissipation of the joints, in order to fully comprehend the dynamic behavior of mechanically fastened timber joints under seismic loading. The ductility is obtained by means of cyclic tests based on the analogy that the behavior of the joint under cyclic loading is a representation of the actual behavior under seismic loading. Hence, the estimated seismic response of a structure depends strongly on the ductility. This quantity is acquired by defining the ratio between the ultimate displacement and the yield displacement from cyclic loading. The yield displacement, $V_{y,EEEE,c}$, is defined by the limit between the elastic and plastic phases from the EEEP curve similar to monotonic tests. As for the ultimate displacement, the value to measure the ductility is a value lower than the cyclic ultimate displacement, $V_{u,c}$, that is defined earlier. This new value is referred to as the equivalent energy elastic-plastic ultimate cyclic displacement, $V_{u,EEEE,c}$. This value is defined as the minimum of the ultimate displacement, $V_{u,c}$, or the displacement that corresponds to the minimum value of degradation factor, $K_{deg,min}$. This parameter is limited by the condition $K_{deg,min} \geq 0.6$. Consequently, the final value for the design strength degradation factor is calculated as follows:

$$k_{deg} = \min[k_{deg}(v)] \geq k_{deg,min} \quad (5-3)$$

Thus, the value of ductility estimated for the joints is then calculated with the following equation:

$$D_{EEEE,c} = \frac{V_{u,EEEE,c}}{V_{y,EEEE,c}} \quad (5-4)$$

Based on the procedure explained above, the final EEEP,c curve for cyclic loading is shown in Figure 4.3 below.

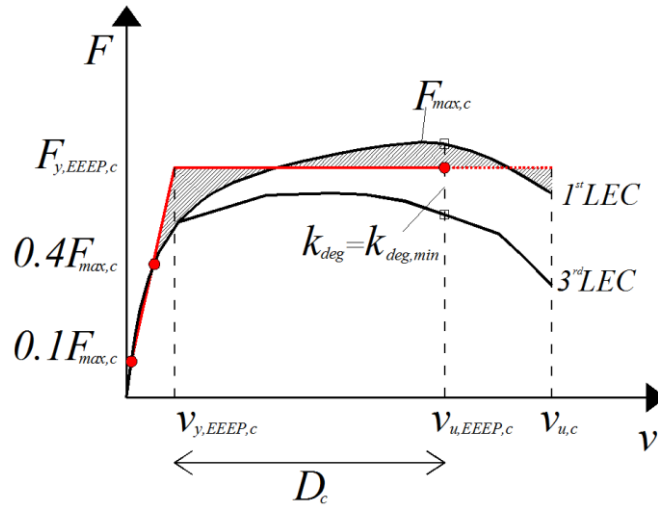


Figure 5:3 - Equivalent Energy elastic-plastic curve (EEEEP) for cyclic tests [7].

Following the quantification of ductility, another important property that influences the dynamic behaviour of dowel-type joints under seismic loading is the ability to dissipate energy. This ability is measured by the mechanism of damping. Damping capacity is a flexible parameter that can change according to the type of load and its magnitude, as well as the damage present already in the system. Rigidity of the joint is reduced by the reduction of damping during the steps when the amplitudes for each step are decreasing between the 1st and the 3rd cycles. However, to measure the damping of a joint, it is calculated by means of a ratio of the dissipated energy and the potential energy in each fully reversed cycle. This parameter is referred to as the viscous damping coefficient, and it is calculated as follows:

$$v_{eq,c} = \frac{E_d}{4\pi \cdot E_p} \quad (5-5)$$

The value of the dissipated energy in each cycle, E_d , is limited by the area of the full cycle, and the potential energy, E_p , for that cycle is measured as a function of the maximum force reached in each cycle.

$$E_p = \frac{1}{2} F_{max} \cdot v \quad (5-6)$$

Where v is the displacement that corresponds to the maximum force value. This procedure was followed for all monotonic and cyclic tests within this experimental campaign, and the parameters detailed above are obtained accordingly. The EEEP curves were formed for each joint using MATLAB software, as well

as the dissipated energy for each cycle. All the results for both Scott's and Maritime pine, are presented and compared in the following section.

5.2.1.2 SCOTT'S PINE

The procedure elaborated above was carried out for all Scott's pine joints for monotonic tests as a first step, then from the monotonic EEEP curve, the values of the yield force and the yield and ultimate displacement were specified. For the original joints, the coefficient of variation of the maximum force obtained from testing and the yield force is slightly different, 7.95% and 7.41%, respectively, with only 5.58% difference between the average maximum force and the average yield force obtained by the energy curve. Nonetheless, the variation in ultimate displacement is higher with a variation in the yield displacement of over 60%. This indicates that the stiffness of the joints differs considerably. Table 5-2 gives the values of the maximum force as well as the yield force and displacements obtained by the energy curves.

Table 5-2 - maximum capacity and EEEP,m values for original Scott's pine joints.

ORIGINAL JOINTS				
Joint Tag	F_{max} (kN)	V_u (mm)	$F_{y,EEEE,m}$ (kN)	$V_{y,EEEE,m}$ (mm)
J1_O_SC	16.33	11.41	15.48	0.33
J2_O_SC	18.03	17.50	17.08	0.29
J3_O_SC	17.71	17.59	16.67	0.31
J4_O_SC	18.98	21.38	17.80	0.14
J5_O_SC	17.68	20.64	16.72	0.17
J6_O_SC	20.57	21.03	19.13	0.14
J7_O_SC	17.04	28.03	16.13	0.88
J8_O_SC	16.67	30.01	15.68	0.49
J9_O_SC	20.34	29.23	18.98	0.58
J10_O_SC	18.80	27.49	17.83	0.60
Average	18.22	22.43	17.15	0.39
CoV %	7.95%	27.24%	7.41%	60.89%

Regarding the reinforced joints, it can be noticed that the variation in the maximum force have decreased considerably, as well as the variation of the ultimate displacement, which gives a value of 27.24% for the original joints and the value has decreased to only 3.18% for the reinforced joints. Similarly, the coefficient of variation for the yield displacement obtained by the original joints has decreased drastically, as it gave previously a value of 60.89% and the value with the reinforcement is 12.31%. Keeping in mind that the coefficient of variation is a statistical value that depends on the number of the samples involved in the survey, therefore, in order to obtain more accurate results, more joints should be reinforced and added to the study for more robust conclusions. However, the

maximum force of the joints have increased by 14.90% from the original capacity. Table 4.2 shows the results for reinforced joints.

Table 5-3 - maximum capacity and EEEP_m values for reinforced Scott's pine joints

REINFORCED JOINTS				
Joint Tag	F _{max} (kN)	V _u (mm)	F _{y,EEEP,m} (kN)	V _{y,EEEP,m} (mm)
J1_R8_SC	20.87	30.01	19.29	0.44
J2_R8_SC	21.17	30.01	19.73	0.37
J3_R8_SC	22.17	30.01	20.34	0.47
Average	21.40	30.01	19.79	0.42
CoV %	3.18%	0.01%	2.67%	12.31%

With regards to the repaired joints, the yield displacement varies by 20.24%, yet the values are much higher than the values obtained by the original joints and the reinforced joints, and the maximum force of the joints has increased of an average of 126.02% of the original capacity. The variation of the yield force, 6.21%, is larger than the variation of the maximum force and that can be explained by the behaviour of the joint by means of force-slip curve. The stiffness of the joints has increased significantly, thus, the elastic modulus is no longer obtained by the slope of 10% and 40% of the maximum force.

The joints were repaired with the STS placed inclined at an angle of 30°. As mentioned earlier, the fully threaded VGZ screws have a very high withdrawal capacity. Thus, the tensile forces at the beginning of the test were resisted by the high tensile strength of STS. Therefore, the elastic modulus to obtain the EEEP curve was found at a slope of 30% and 70% of the maximum force. Figure 5:4.

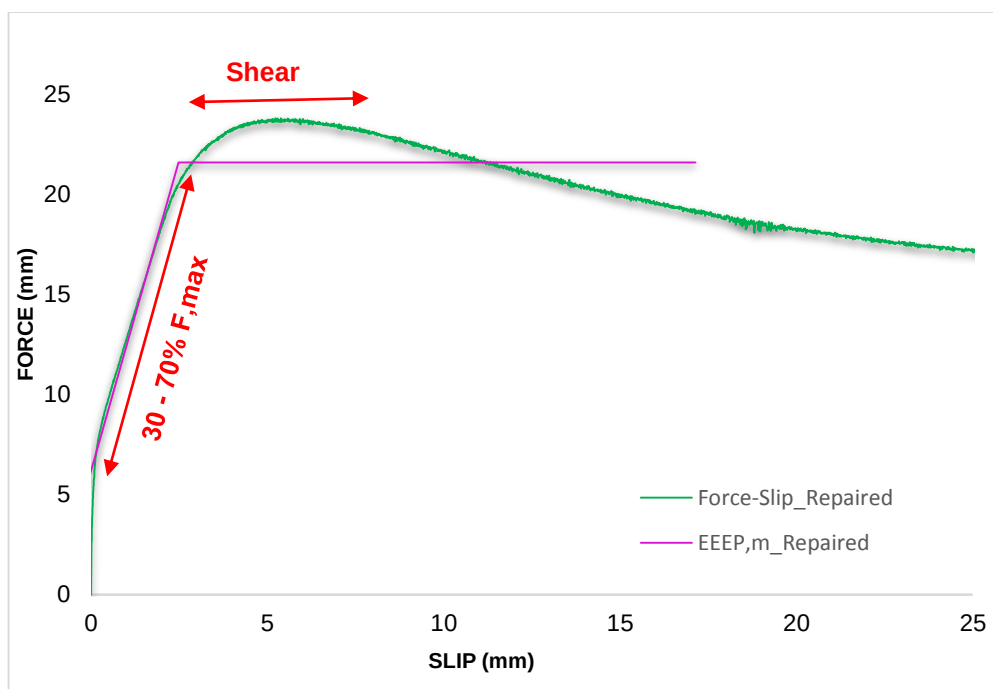


Figure 5:4 - Force-slip curve of a joint repaired with inclined STS.

Figure 5:5 shows the EEEP,m curves for the average of the original, reinforced and repaired joints, and the slope of the energy curves gives a clear indication that the highest stiffness and capacity is achieved with the repaired joints, but the reinforcement provides higher ductility. Table 5-4 gives the values obtained by the energy curve of the joints.

Table 5-4 - maximum capacity and EEEP,m values for repaired Scott's pine joints.

REPAIRED JOINTS				
Joint Tag	$F_{,max}$ (kN)	V_u (mm)	$F_{y,EEEE,m}$ (kN)	$V_{y,EEEE,m}$ (mm)
J7_O_SC_RP	23.82	17.55	21.56	3.28
J8_O_SC_RP	22.73	16.79	20.56	2.98
J10_O_SC_RP	25.51	16.23	23.24	4.34
Average	24.02	26.86	21.79	3.52
CoV %	5.82%	3.92%	6.21%	20.24%

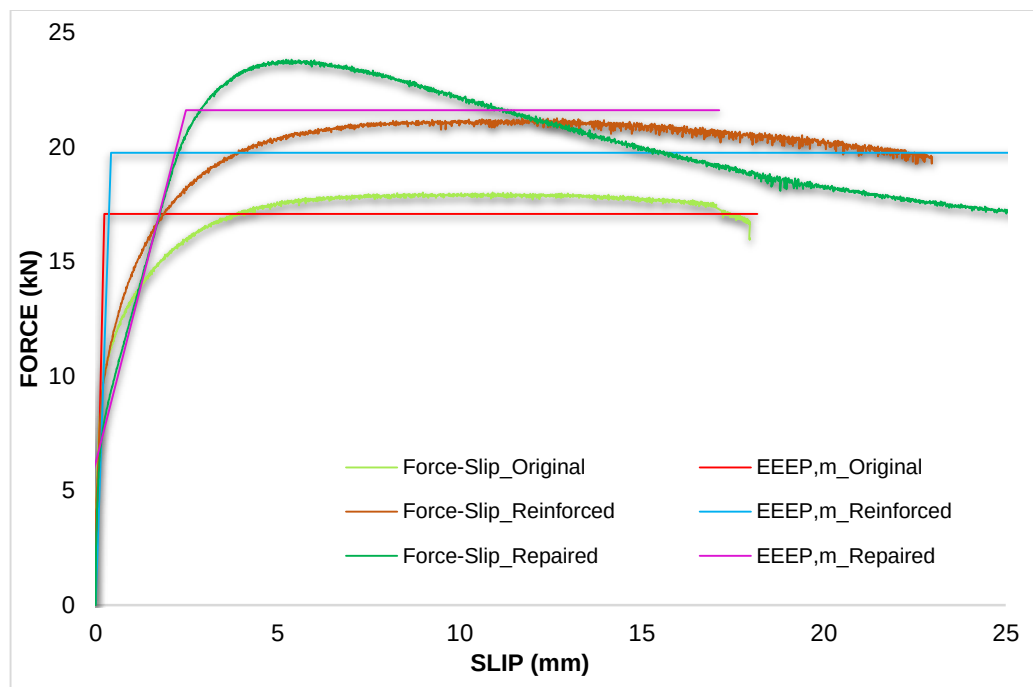


Figure 5:5 - Force-slip curves for original, reinforced and repaired Scott's Pine joints.

5.2.1.2.1 DUCTILITY

The quantification of ductility, as explained in 5.3.1, is carried out by means of the EEEP,c for cyclic tests. The 1st and 3rd LEC curves were obtained for both tension and compression and the ductility was calculated in both cases. The design strength degradation factor, K_{deg} , was calculated in each case and the corresponding forces in the 1st and the 3rd LEC, $F_1(v)$ and $F_3(v)$, respectively were specified within each curve. Table 5-5 shows the values obtained for the original joints.

Table 5-5 - EEEP,c results for original Scott's Pine joints.

ORIGINAL JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEEP,c}$ (kN)	$V_{y,EEEEP,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEEP,c}$ (mm)	$D_{EEEEP,c}$
J1_CO_SC	19.60	2.859	20.18	17.31	0.86	12.81	4.48
J2_CO_SC	17.45	2.26	18.74	16.55	0.88	13.56	5.99
J3_CO_SC	17.40	2.19	18.60	12.57	0.68	5.41	2.47
J4_CO_SC	17.89	2.11	19.20	16.84	0.88	5.776	2.74
J5_CO_SC	20.78	2.29	21.89	18.63	0.85	12.731	5.55
J6_CO_SC	17.82	2.01	18.61	16.30	0.88	15.48	7.69
Average	18.49	2.29	19.54	16.37	0.84	10.96	4.82
CoV %	7.48%	13.02%	6.64%	12.41%	9.52%	39.01%	41.62%

ORIGINAL JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CO_SC	-19.20	-1.224	-18.92	-17.05	0.90	-13.43	10.98
J2_CO_SC	-15.00	-2.05	-15.48	-13.93	0.90	-12.96	6.33
J3_CO_SC	-14.03	-2.32	-17.35	-16.04	0.93	-4.49	1.94
J4_CO_SC	-16.45	-2.18	-18.00	-16.34	0.91	-6.32	2.90
J5_CO_SC	-17.35	-2.55	-17.75	-15.91	0.90	-11.72	4.60
J6_CO_SC	-15.61	-2.46	-15.71	-14.16	0.90	-15.06	6.13
Average	-16.28	-2.13	-17.20	-15.57	0.91	-10.66	5.48
CoV %	-11.28%	-22.51%	-7.84%	-8.03%	1.15%	-39.87%	58.50%

The design strength degradation factor is higher in tension than in compression in all joints, which means that the joints have more stiffness in compression. This indicates that most of the joints are more ductile in tension than in compression. However, the coefficient of variation for the ductility in tension is also higher, 58.5%, which makes the values inconclusive. Since the ductility depends directly on the ultimate energy displacement, which has also have a very high coefficient of variation in both compression and tension, therefore the high variation in ductility values is expected. Figure 5:6 shows the 1st LEC curves for all 6 original joints.

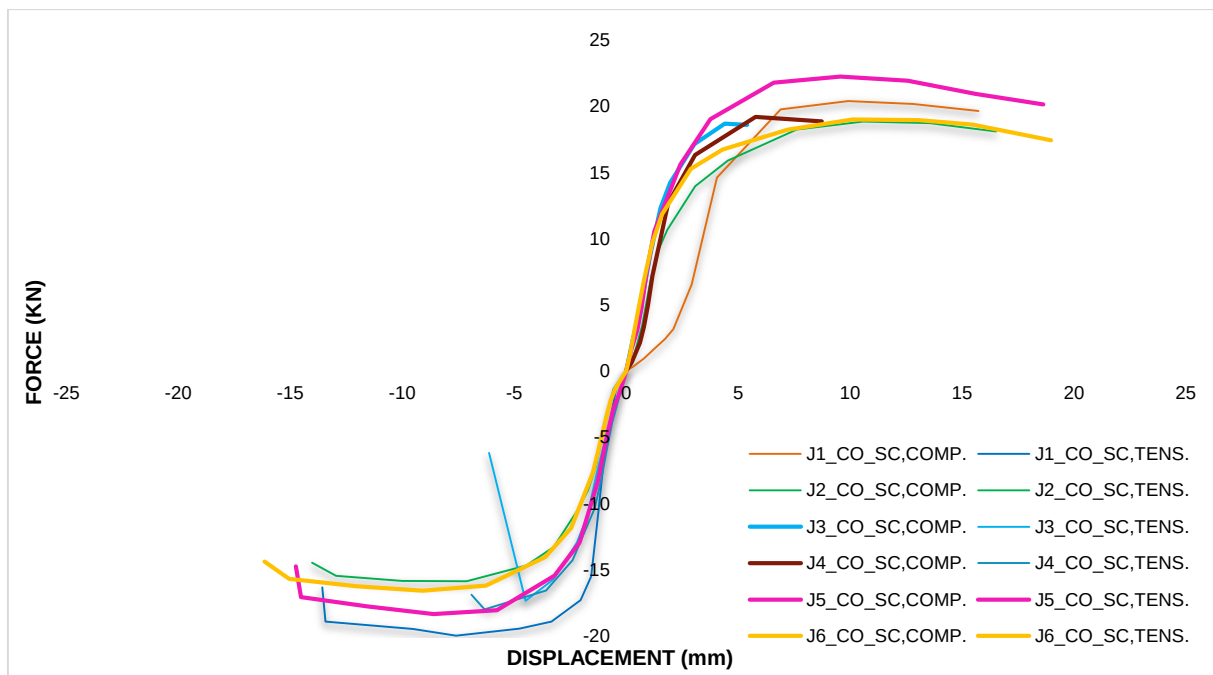


Figure 5:6 - 1st LEC curves for Scott's pine original joints.

The LEC curve for the first joint above, J1_CO_SC, shows the effect of the pinching in the results, and it was excluded when equating the area under the graph with the EEEP curve. Despite the initial displacement in the joint, yet the joint had the highest stiffness in compression. Therefore, the elastic

modulus of the EEEP,c curve was obtained with a slope of 30% and 70% F_{max} . Figure 5:7. Based on the LEC curve of the 3rd joint, J3_CO_SC, it can be clear from the curve that the joint was brittle, which is the case for this joint as it failed a total failure by splitting in the middle element, and was unsuitable for repairing after.

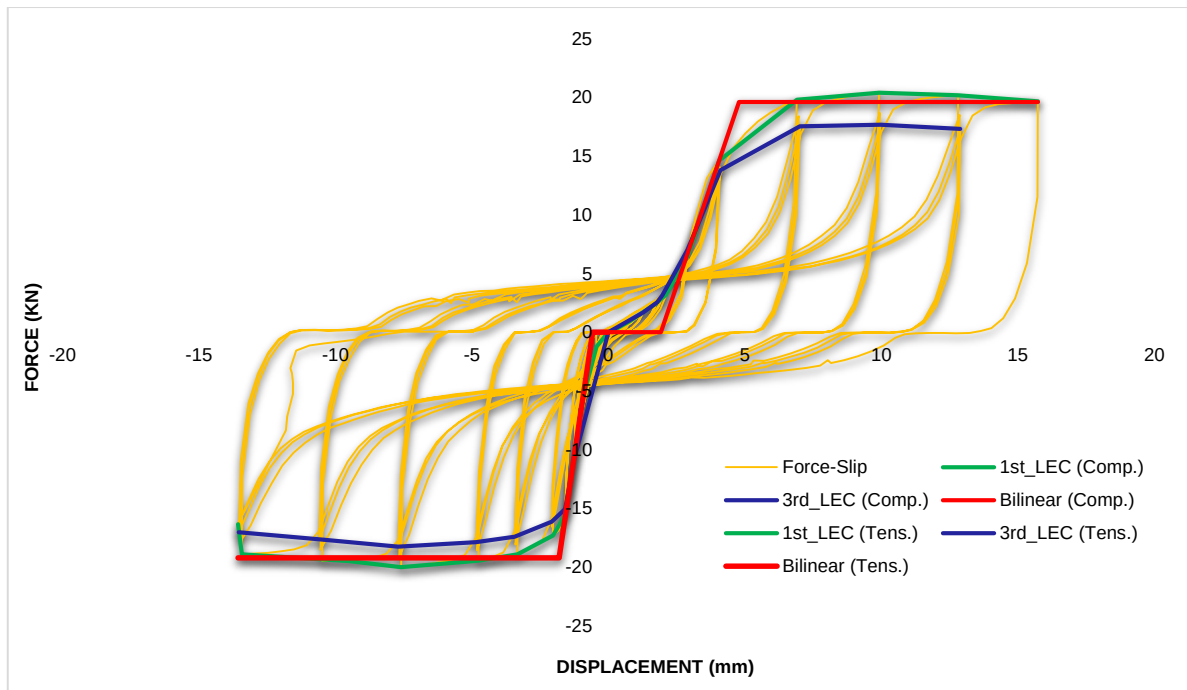


Figure 5:7 - EEEP,c and LEC curves for the 1st and the 3rd cycles for the first joint.

Regarding the reinforced joints, the design strength degradation factor has decreased from the original, which indicates an increase in the capacity of the joints. Referring to Table 4.5 below, the capacity of the joints has in fact increased 7.13% in compression and 6.49% in tension. Figure 5:8. The ductility of the joints have increased significantly from the original joints, as the value increased to 40.35% with a coefficient of variation (CoV) of 18.52% in compression, and a value of 32.52% with a CoV of 47.62%.

Table 5-6 - EEEP,c results for reinforced Scott's Pine joints

REINFORCED JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CR_SC	22.21	2.25	17.98	11.97	0.67	20.97	9.31
J2_CR_SC	20.96	2.46	17.70	10.91	0.62	20.98	8.52
J3_CR_SC	16.56	2.40	15.19	11.26	0.74	15.43	6.42
Average	19.91	2.37	16.96	11.38	0.67	19.13	8.08
CoV %	14.90%	4.60%	9.07%	4.75%	9.34%	16.74%	18.52%

REINFORCED JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CR_SC	-17.74	-2.52	-18.63	-12.31	0.66	-24.07	9.55
J2_CR_SC	-17.95	-2.25	-18.41	-12.37	0.67	-24.90	11.06
J3_CR_SC	-16.52	-2.78	-19.16	-16.40	0.86	-10.41	3.74
Average	-17.40	-2.52	-18.73	-13.69	0.73	-19.80	8.12
CoV %	-4.44%	-10.57%	-2.07%	-17.12%	15.02%	-41.11%	47.62%

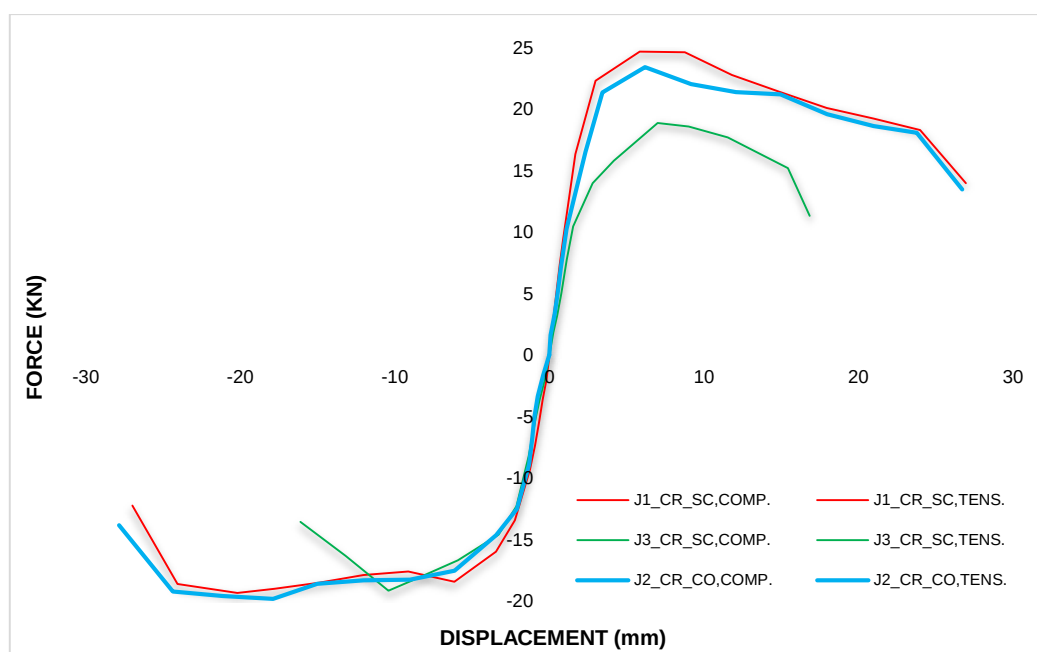


Figure 5:8 - 1st LEC curves for Scott's pine reinforced joints.

The recovered capacity in the repaired joints is 89.59% in compression, with a CoV of 9.38%, which is slightly higher than the original joints CoV, and 39.33% in tension with CoV of 10.22%. The high capacity of the joints with repairing was expected, as the position of two screws in the middle element, below and above the dowel, has stopped the elements' splitting and helped restore part of the capacity. The strength degradation factors in compression and tension to values between 0.6 and 0.85, and its higher in compression. The ductility in tension gave very low values with a very high CoV. The high value of the CoV of ductility in tension, 81.57%, is due to the very high stiffness in joint J2_CO_SC that can be seen in Figure 5:9 in comparison to the other 2 joints.

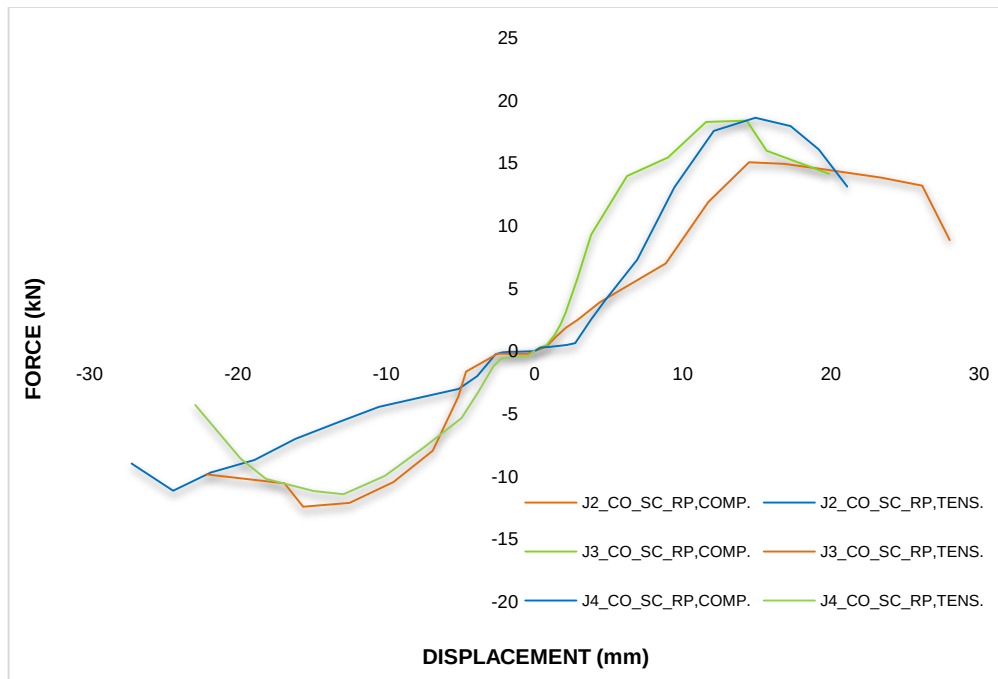


Figure 5:9 - 1st LEC curves for Scott's pine repaired joints.

Since the ductility is a ratio between the ultimate and yield displacement, the large value of $V_{y,EEEP,c}$ of this joint, 19.27 mm, and a variation of 59.98% in compression, and 21.94 mm in tension with a CoV of 92.39%, has caused the values of ductility to be significantly low, which corresponds to brittle failures which is the case in this joint. Although the original joint sustained all 15 cycles without any cracks. The joint failed before the screws could reach their maximum yield capacity. Due to its high stiffness, the elastic slope was obtained by connecting 30% and 70% instead of 10% and 40%. Figure 5:10 shows the joint's 1st and 3rd LEC before and after repairing, and Table 4.6 shows the values discussed above and the corresponding variations.

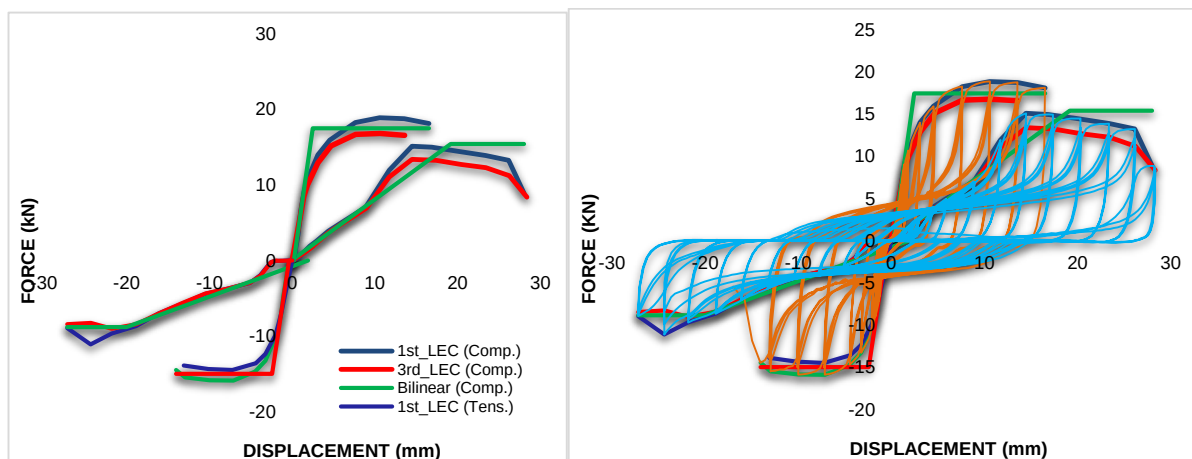


Figure 5:10 - Left: EEEP,c, 1st and 3rd LEC curves of the original and the repaired joints, Right: The curves with the original force-slip cyclic curve.

Table 5-7 - EEEP,c results for repaired joints

REPAIRED JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J2_CO_SC_RP	15.40	19.27	1.90	1.57	0.83	2.15	0.11
J3_CO_SC_RP	15.97	4.93	18.40	15.22	0.83	14.32	2.91
J4_CO_SC_RP	18.33	11.68	17.97	15.30	0.85	17.29	1.48
Average	16.56	11.96	12.75	10.70	0.84	11.26	1.50
CoV %	9.38%	59.98%	73.74%	73.88%	1.65%	71.26%	93.20%

REPAIRED JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J2_CO_SC_RP	-8.83	-21.94	-0.07	-0.04	0.60	-1.79	0.08
J3_CO_SC_RP	-10.85	-4.74	-0.31	-0.19	0.60	-2.65	0.56
J4_CO_SC_RP	-9.94	-5.17	-0.45	-0.37	0.84	-1.29	0.25
Average	-9.87	-10.61	-0.28	-0.20	0.68	-1.91	0.30
CoV %	-10.22%	-92.39%	-70.17%	-83.71%	20.38%	-35.99%	81.57%

5.2.1.3 MARITIME PINE

Regarding Maritime Pine joints, the timber elements in general have higher density than the Scott's Pine elements, therefore, higher capacity. The joints have a maximum force for the original joints with a variation of variation of 9.61% and the yield force of 9.03%, which is slightly higher than the variation between the Scott's pine joints. Nonetheless, the CoV of the reinforced joints, 1.00% and 1.69%, for the maximum force and the yield force, respectively, is much less than CoV obtained for the Scott's Pine joints, yet the capacity for these joints have increased by 15.24%. This indicates that the reinforcement gave better results with Maritime Pine joints. However, the yield slip values have increased with reinforcement which projects the contribution of the reinforcement in increasing the overall capacity of the connections. Regarding the ultimate displacement, the values have increased by 42% of the original average ultimate slip, with a CoV of only 0.02% opposed to a CoV of 23.15% for original joints. Hence, giving the increase in the ultimate slip, the effect of the reinforcement is more dominant by means of ductility. Table 5-8 shows the results for original and reinforced joints.

Table 5-8 - EEEP,m results for original and reinforced Maritime Joints.

ORIGINAL JOINTS				
Joint Tag	F_{max} (kN)	V_u (mm)	$F_{y,EEEE,m}$ (kN)	$V_{y,EEEE,m}$ (mm)
J1_O_MR	21.39	10.54	19.38	0.42
J2_O_MR	19.89	20.86	18.65	0.03
J3_O_MR	19.09	17.11	17.92	0.21
J4_O_MR	18.33	17.95	17.06	0.19
J5_O_MR	20.27	21.89	19.00	0.14
J6_O_MR	16.07	16.09	15.01	0.18
Average	19.17	17.41	17.84	0.20
CoV %	9.61%	23.15%	9.03%	64.48%

REINFORCED JOINTS				
Joint Tag	F_{max} (kN)	V_u (mm)	$F_{y,EEEE,m}$ (kN)	$V_{y,EEEE,m}$ (mm)
J1_R8_MR	22.65	30.00	21.41	0.53
J2_R8_MR	22.38	30.01	20.97	0.47
J3_R8_MR	22.83	30.01	21.68	0.49
Average	22.62	30.01	21.35	0.50
CoV %	1.00%	0.02%	1.69%	6.48%

The repairing technique for these joints is different as mentioned earlier. The joints failed mainly by splitting, therefore, the screws to repair were placed perpendicular to the grain and to the dowel direction in order to stop the split from spreading. However, the maximum force obtained with a CoV of 5.61%, is an average of 17.98 kN which represents 93.76% of the original capacity. This can be taken as a considerable improvement. Over and above, the repairing technique was successful in increasing the joints' ductility. The ultimate displacement, which is an average of 30mm, increased by again 42% compared to the original that had an average of 17.41mm and varied by 23.15%. Figure 5:11 shows the EEEP,m of the average of the original joints, as well as the reinforced and the repaired, where the evolution of stiffness and ductility can be seen. Table 5-9 shows the results for repaired joints.

Table 5-9 - EEEP,m results for repaired joints

REPAIRED JOINTS				
Joint Tag	F_{max} (kN)	V_u (mm)	$F_{y,EEEE,m}$ (kN)	$V_{y,EEEE,m}$ (mm)
J3_O_MR_RP	18.43	30.02	16.59	0.71
J4_O_MR_RP	18.68	30.00	17.79	0.80
J6_O_MR_RP	16.82	30.01	15.76	1.24
Average	17.98	30.01	16.71	0.92
CoV %	5.61%	0.02%	6.10%	30.70%

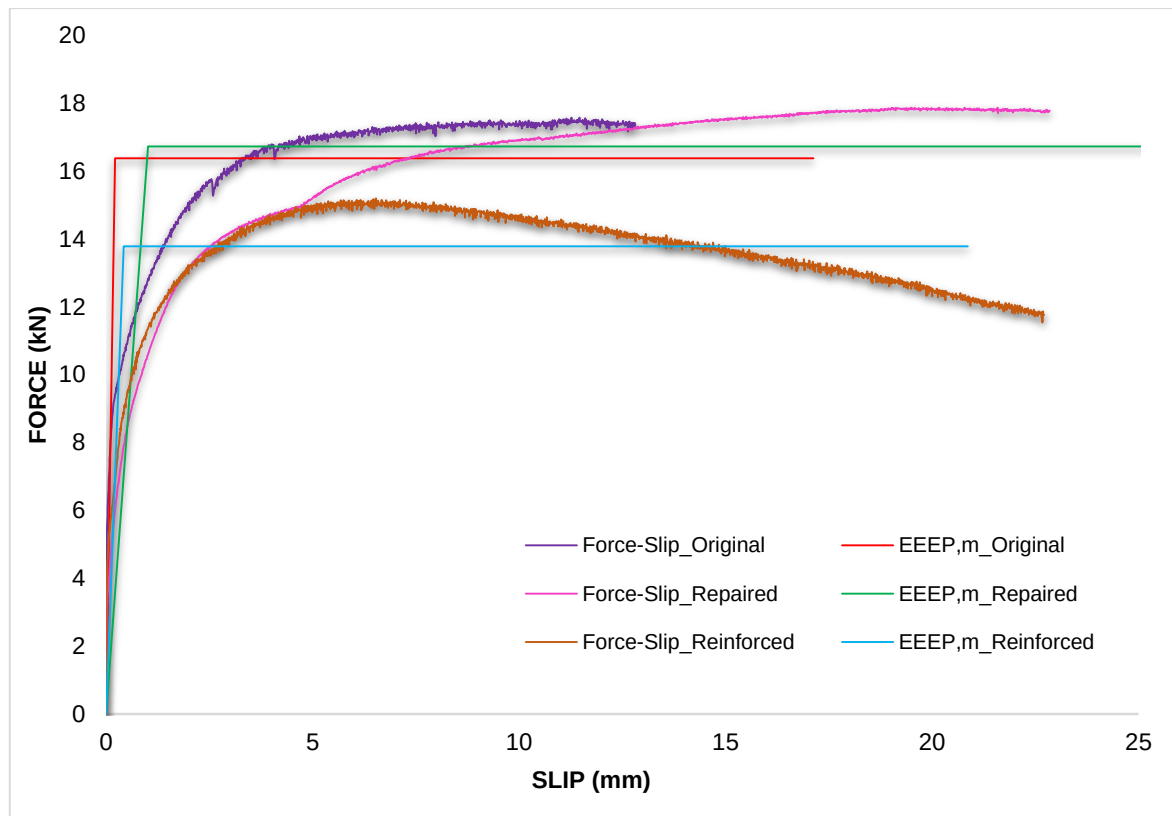


Figure 5:11 - Force-slip curves for original, reinforced and repaired joints.

5.2.1.3.1 DUCTILITY

Following the same procedure as Scott's Pine joints, cyclic tests were conducted for 6 original joints, 3 reinforced and 3 that are repaired. The design strength degradation factor is higher in tension, similar to Scott's Pine joints but with a higher CoV in compression, 15.04%. The ultimate displacement vary by large CoV of 56.21% in compression and 62.06% in tension. This could be a result of joint 6, J6_CO_MR that has much less ductility and much less ultimate displacement. The failure in that joint was brittle with splitting in the middle element. The capacity of the joints is higher in compression than in tension, with an average of 21.1 kN and a CoV of 9.07% that is higher than the tensile yield force CoV that is equal to 4.76%. Table 5-10 gives the values obtained by EEEP,c for the 6 original Maritime Pine.

Table 5-10 - EEEP,c results for original Maritime Pine joints.

ORIGINAL JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CO_MR	22.60	2.224	20.59	14.93	0.73	20.29	9.12
J2_CO_MR	22.21	1.56	20.05	15.01	0.75	22.95	14.66
J3_CO_MR	18.39	2.01	19.37	11.62	0.60	12.70	6.31
J4_CO_MR	23.22	2.24	25.58	22.17	0.87	11.434	5.11
J5_CO_MR	19.39	2.42	19.97	17.66	0.88	11.678	4.82
J6_CO_MR	20.79	1.05	17.96	16.27	0.91	1.593	1.52
Average	21.10	1.92	20.59	16.27	0.79	13.44	6.92
CoV %	9.07%	27.03%	12.68%	21.59%	15.04%	56.21%	65.23%

ORIGINAL JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CO_MR	-19.29	-2.009	-17.24	-12.58	0.73	-20.15	10.03
J2_CO_MR	-18.83	-1.88	-18.78	-16.29	0.87	-16.36	8.70
J3_CO_MR	-16.95	-2.03	-18.13	-16.09	0.89	-8.12	4.00
J4_CO_MR	-18.38	-2.26	-20.88	-19.01	0.91	-8.54	3.77
J5_CO_MR	-19.05	-2.85	-19.34	-17.59	0.91	-11.90	4.17
J6_CO_MR	-17.83	-1.57	-4.81	-4.03	0.84	-0.79	0.50
Average	-18.39	-2.10	-16.53	-14.26	0.86	-10.98	5.20
CoV %	-4.76%	-20.64%	-35.51%	-38.19%	7.93%	-62.06%	67.88%

Table 5-10 shows the 1st LEC curves for the 6 joints. The high ductility of joint 1, J1_CO_MR, can be seen in the LEC curve for the joint as well as from the ductility value from the table above. The high CoV of ductility values caused the CoV to increase significantly so that the values are deemed inconclusive. That is also evident in the LEC of the joints of the 1st cycle, as the variations can be seen in the behaviour of the joints. The first joint has high ductility in both compression and tension, with joint 6 has the lowest ductility which is noticeable from the joints behaviour shown by the 1st LEC.

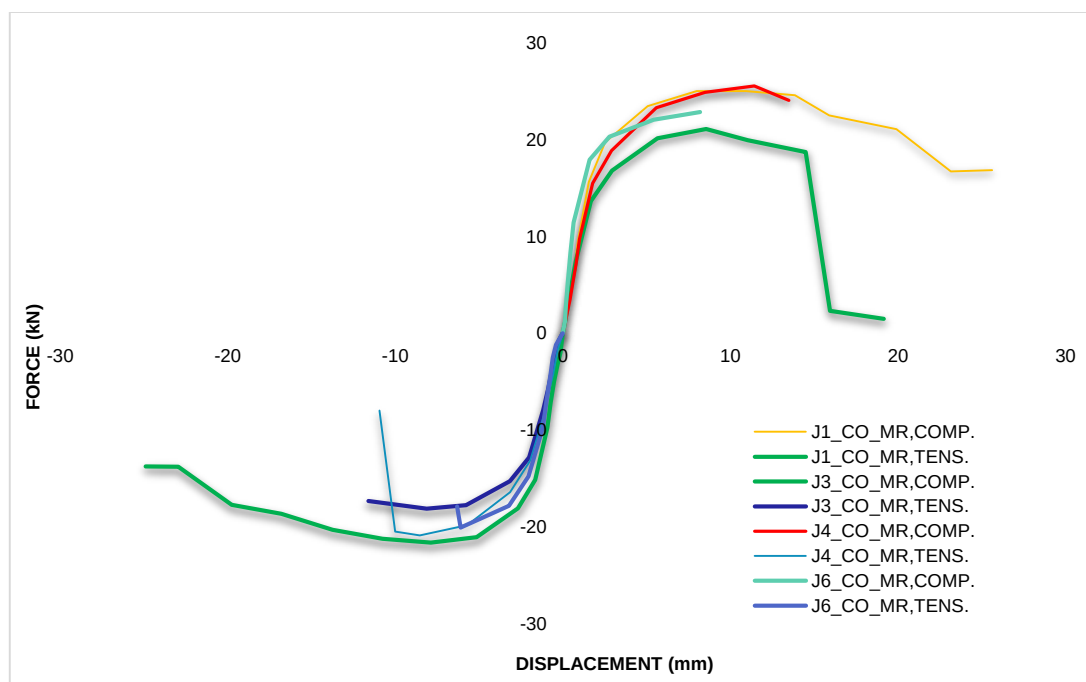


Figure 5:12 - 1st LEC curves for Maritime pine original joints.

With regards to the reinforcement by means of cyclic testing for Maritime Pine joints, the capacity has increased by only 3.83% in compression with a CoV of 1.87%, but 11.50% in tension, and the CoV for the values is slightly higher, 3.97%. The original joints had a tendency to fail under tension, yet with reinforcement, this capacity has slightly increased. This increase can be also seen as the strength degradation factor has decreased from the original joints by 5.91% in tension, with a CoV of 13.17% in reinforced joints, and 7.93% for original joints. The ductility however have increased significantly, by 38.64% from the original ductility with a CoV of 23.86% opposed to a CoV of the ductility in the original joints of 67.88%. This elevation in the quantity of ductility is following an increase in the ultimate displacement around 44.55%. That is with regards to tension. As for compression, the ultimate displacement has also increased significantly by 37.71%, which lead to an increase of 38.51% in the ductility with a CoV that is equal to 8.99% that is much lower than the variation between the original joints, 65.23%. Table 5-11 shows the values for the discussed parameters.

Table 5-11 - EEEP,c results for reinforced Maritime Pine joints.

REINFORCED JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CR_MR	21.78	2.07	18.10	15.95	0.88	22.20	10.75
J2_CR_MR	21.63	1.69	22.36	17.60	0.79	21.02	12.43
J3_CR_MR	22.41	2.03	21.46	14.57	0.68	21.52	10.61
Average	21.94	1.93	20.64	16.04	0.78	21.58	11.26
CoV %	1.87%	10.69%	10.88%	9.45%	12.95%	2.74%	8.99%

REINFORCED JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J1_CR_MR	-21.20	-2.39	-23.40	-20.89	0.89	-16.77	7.02
J2_CR_MR	-21.31	-2.79	-22.98	-15.81	0.69	-21.25	7.61
J3_CR_MR	-19.83	-1.98	-18.07	-15.14	0.84	-21.36	10.77
Average	-20.78	-2.39	-21.48	-17.28	0.81	-19.80	8.47
CoV %	-3.97%	-16.96%	-13.81%	-18.21%	13.17%	13.23%	23.86%

It can be seen from the 1st LEC of the joints in Figure 5:13, that the increase in ductility is prevalent more in tension. The stiffness for the joints is around the same values as the slope of elastic modulus was obtained by connecting 10% and 40% of the maximum force in all 3 joints.

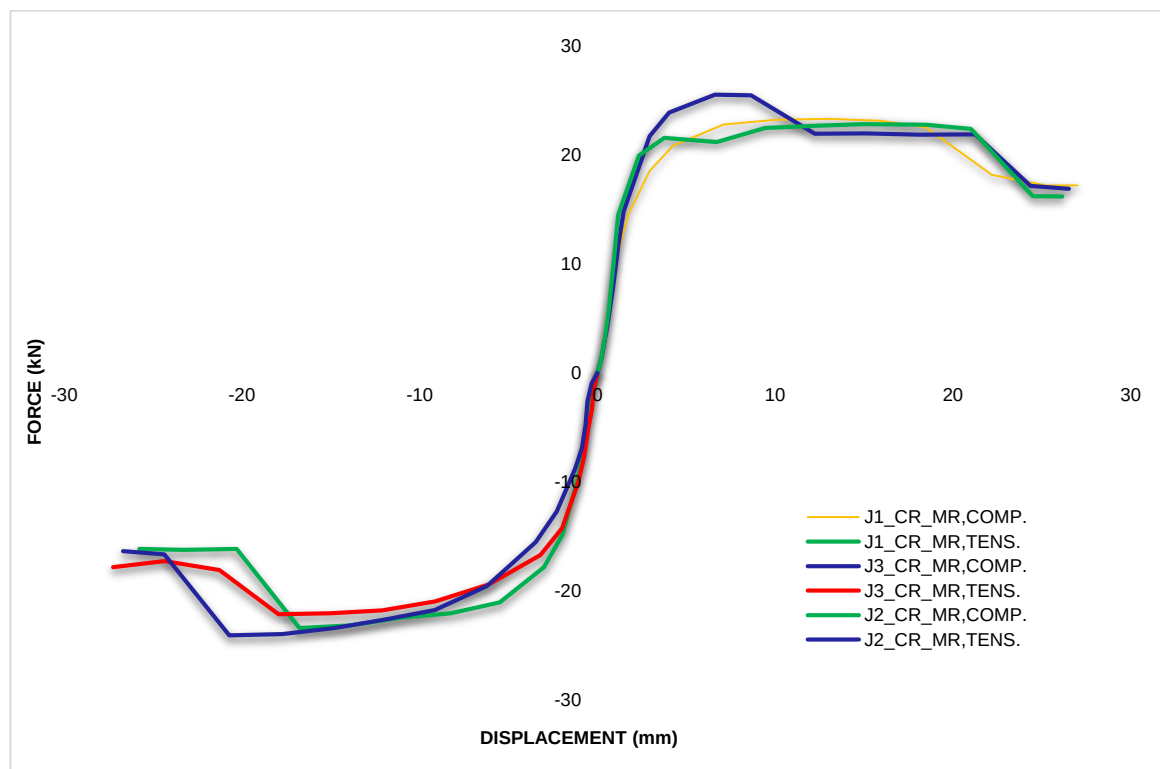


Figure 5:13 - 1st LEC curves for Maritime pine reinforced joints

The repairing for cyclic joints was mainly to stop the splitting from spreading, thus two STS were placed at the top and bottom of the dowel in each of the joints, in addition to 2 screws on each of the side elements. As presented earlier in chapter 3, that the pinching effect in these joints have allowed the dowel to become in contact with the STS during the reversed cycles, leading the STS to form a support for the dowel and deforming in a plastic-rigid behaviour. Figure 5:14 below shows the behaviour under

cyclic loading resulting from this contact between the dowels and the screws, not to mention that the joint is repaired and it has experienced a partial failure beforehand.

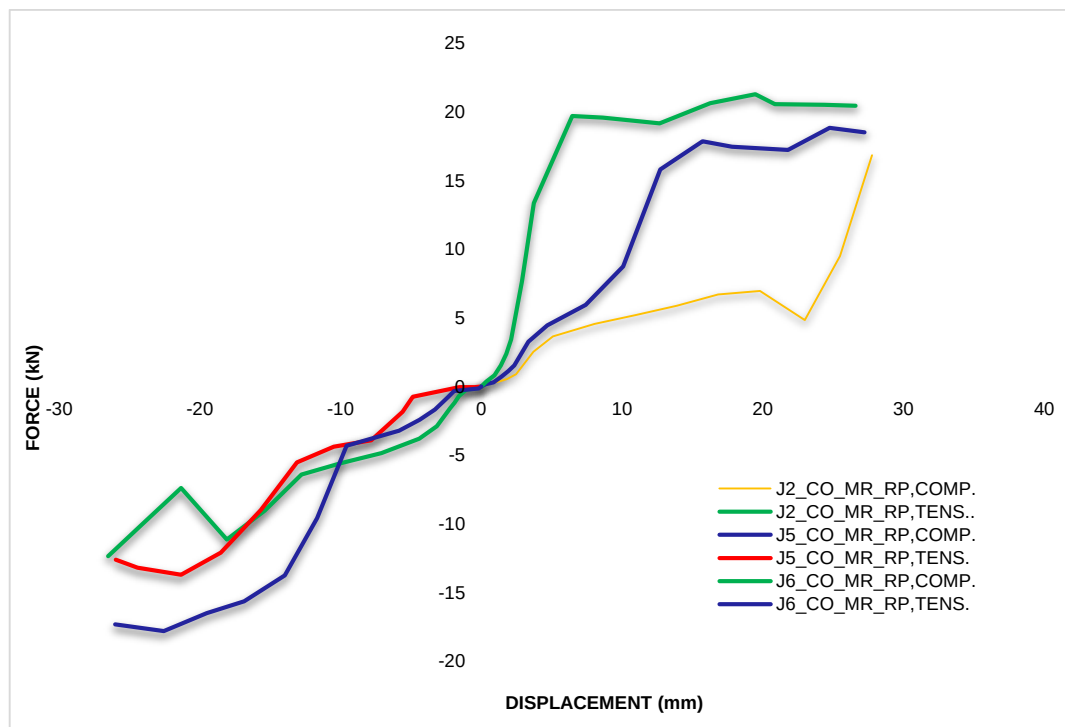


Figure 5:14 - 1st LEC curves for Maritime pine repaired joints

Referring to the distorted behaviour of the second joint, J2_CO_MR_RP, the force increases abruptly after it reached the maximum and decreased, and that is believed to be caused by the contact with the STS. In general, the stiffness of the joints have increased opposed to the original joints and the repaired. Referring to Table 4.10 below, the coefficient of variation, CoV, are very high, and that is again caused by the very different behaviour of the 3 joints under loading. However, the strength degradation here is less than the values obtained for the original and reinforced joints with an average of 0.64 in tension with a CoV of 10.30%, and an average of 0.71, with a CoV of 17.29% in compression. The lowest value of degradation is obtained once again for the 2nd joint. The ductility values are lower than the values obtained by the reinforced and original joints, while the stiffness of the joints have increased considerably. Therefore, for joint 5 and 6, the elastic slope was obtained by means of 30% and 70% of the maximum force. Table 5-12 shows the values obtained by the EEEP,c curves, and Figure 5:15 shows the 1st and 3rd LEC for the repaired joint 2 as opposed to the original.

Table 5-12 - EEEP,c results for reinforced Maritime Pine joints.

REPAIRED JOINTS_COMPRESSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J2_CO_MR_RP	5.66	5.18	6.87	4.12	0.60	18.79	3.63
J5_CO_MR_RP	17.74	12.72	0.78	0.54	0.70	1.49	0.12
J6_CO_MR_RP	20.10	4.14	20.45	17.27	0.84	26.59	6.42
Average	14.50	7.35	9.37	7.31	0.71	15.62	3.39
CoV %	53.42%	63.70%	107.48%	120.46%	17.29%	82.22%	93.20%

REPAIRED JOINTS_TENSION							
Joint Tag	$F_{y,EEEE,c}$ (kN)	$V_{y,EEEE,c}$	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$
J2_CO_MR_RP	-8.16	-10.33	-10.47	-6.28	0.60	-17.23	1.67
J5_CO_MR_RP	-12.73	-16.82	-0.06	-0.04	0.60	-1.68	0.10
J6_CO_MR_RP	-16.62	-7.62	-0.20	-0.14	0.71	-0.69	0.09
Average	-12.50	-11.59	-3.58	-2.15	0.64	-6.53	0.62
CoV %	-33.88%	-40.82%	-166.99%	-166.14%	10.30%	-142.00%	146.55%

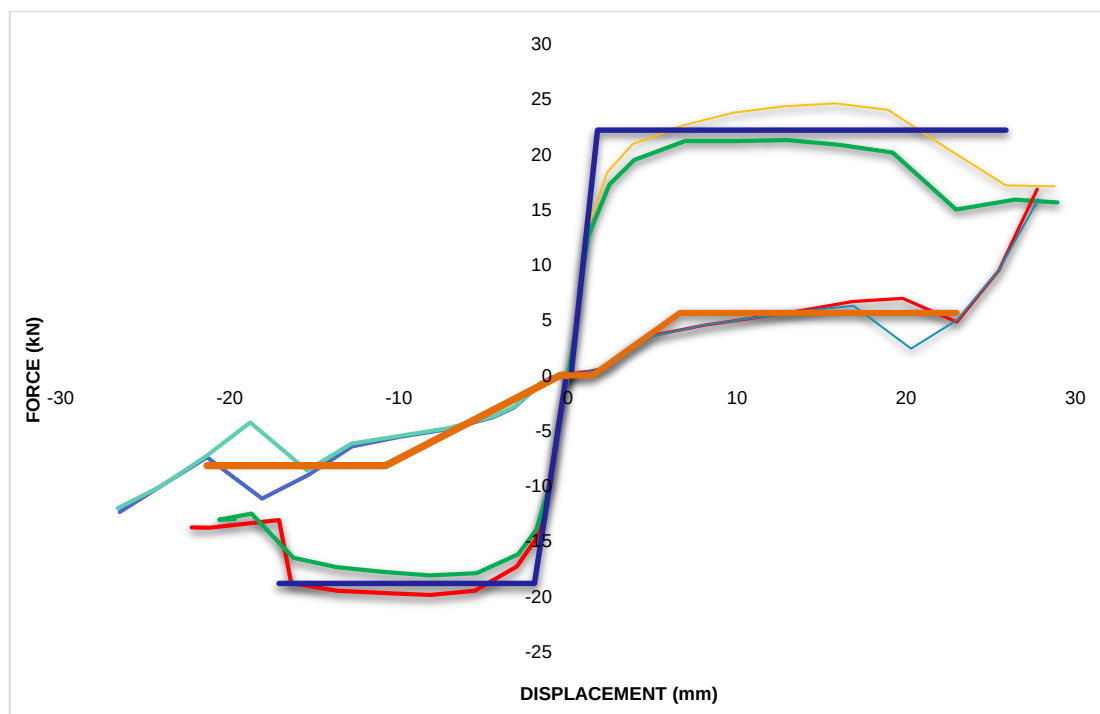


Figure 5:15 - EEEP,c, 1st LEC and 3rd LEC curves for Maritime pine repaired joints opposed to the original

It can be noticed that the regained capacity is 25.49%, but the stiffness has significantly increased.

5.2.1.1 ENERGY DISSIPATION

Regarding the energy dissipation, as explained in the procedure above, that it is measured by calculating the viscous damping coefficient, which is a function of the dissipated energy, E_d , and the potential energy, E_p , of each full cycle. The dissipated energy was obtained using MATLAB software, and the potential energy and the viscous damping were obtained by equations (5-5 and 5-6). Then the total dissipated energy by each connection was measured with respect to accumulative displacement. Table 5-13 shows the average values for original, reinforced and repaired joints, for both Scott's Pine and Maritime Pine. And Figure 5:16 shows the total energy dissipated in terms of accumulative displacement for Scott's pine.

Table 5-13 - Dissipated energy and viscous damping ratio for Scott's Pine and Maritime Pine.

MARITIME PINE			SCOTT'S PINE		
ORIGINAL JOINTS			ORIGINAL JOINTS		
Joint Tag	E_d (KN.mm)	ϑ_{eq}	Joint Tag	E_d (KN.mm)	ϑ_{eq}
J1_CO_MR	108.65	0.10	J1_CO_SC	71.28	0.18
J2_CO_MR	115.81	0.09	J2_CO_SC	73.27	0.10
J3_CO_MR	53.53	0.12	J3_CO_SC	15.18	0.06
J4_CO_MR	69.60	0.09	J4_CO_SC	19.64	0.08
J5_CO_MR	53.75	0.09	J5_CO_SC	102.33	0.12
J6_CO_MR	26.10	0.08	J6_CO_SC	80.78	0.09
Average	71.24	0.10	Average	60.42	0.11
CoV %	48.81%	13.27%	CoV %	58.12%	40.14%
REINFORCED JOINTS			REINFORCED JOINTS		
Joint Tag	E_d (KN.mm)	ϑ_{eq}	Joint Tag	E_d (KN.mm)	ϑ_{eq}
J1_CR_MR	123.97	0.09	J1_CR_SC	159.63	0.16
J2_CR_MR	141.17	0.12	J2_CR_SC	134.51	0.11
J3_CR_MR	156.06	0.11	J3_CR_SC	61.83	0.08
Average	140.40	0.11	Average	118.66	0.12
CoV %	11.44%	13.81%	CoV %	42.81%	30.85%
REPAIRED JOINTS			REPAIRED JOINTS		
Joint Tag	E_d (KN.mm)	ϑ_{eq}	Joint Tag	E_d (KN.mm)	ϑ_{eq}
J2_CO_MR_RP	52.81	0.14	J2_CO_SC_RP	100.37	0.09
J5_CO_MR_RP	55.07	0.06	J3_CO_SC_RP	30.86	0.06
J6_CO_MR_RP	70.60	0.06	J4_CO_SC_RP	46.93	0.18
Average	59.49	0.08	Average	59.39	0.11

CoV %	16.28%	55.21%	CoV %	61.27%	58.39%
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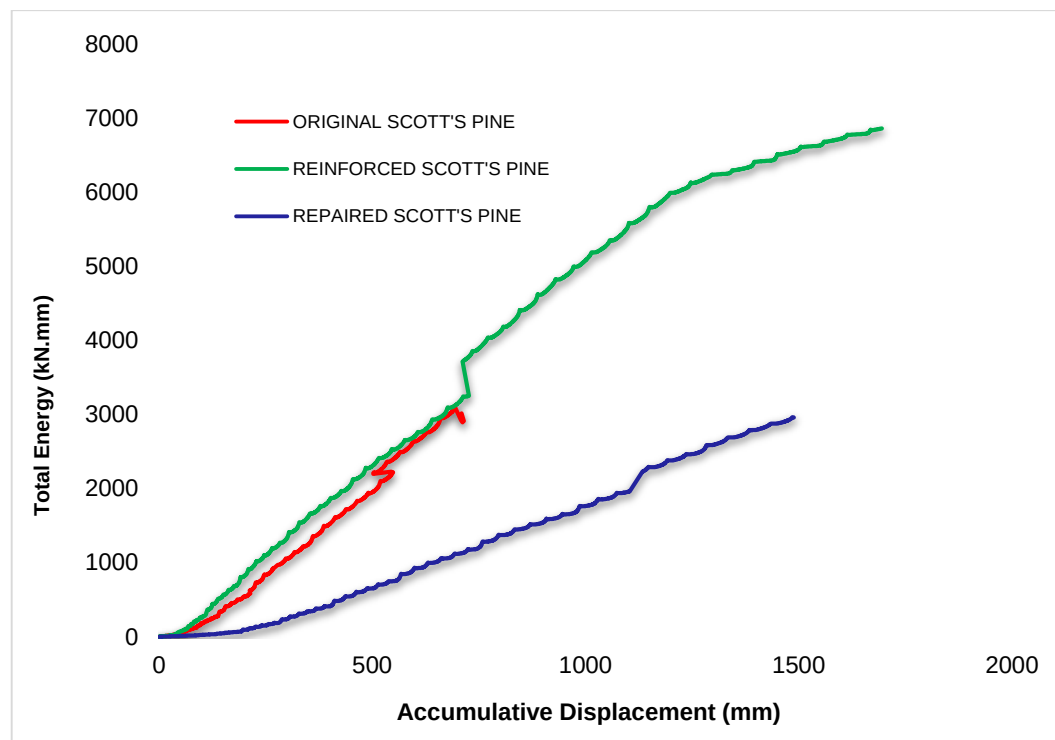


Figure 5:16 - Total Energy of original and reinforced and repaired Scott's Pine joints.

The variation of the Maritime Pine joints is less than the CoV of the Scott's pine with respect to original, reinforced and repaired joints. However, the viscous damping ratio for Maritime pine is higher than the one obtained with Scott's Pine joints by 15.19% for the original joints, 15.49% for the reinforced joints and 0.18% for the repaired joints. Regarding Scott's Pine samples, it can be noticed that the total dissipated energy is significantly higher than the original and repaired joints, as it reaches a value of around 7000 kN.mm, for an accumulative displacement more than 1500 mm. While the original joints dissipate only 3000 kN.mm in total for an accumulative displacement around 750 mm. Regarding the repaired joints, the total dissipated energy is slightly lower than the original, yet for an accumulative displacement of 1500 mm, which is twice the displacement.

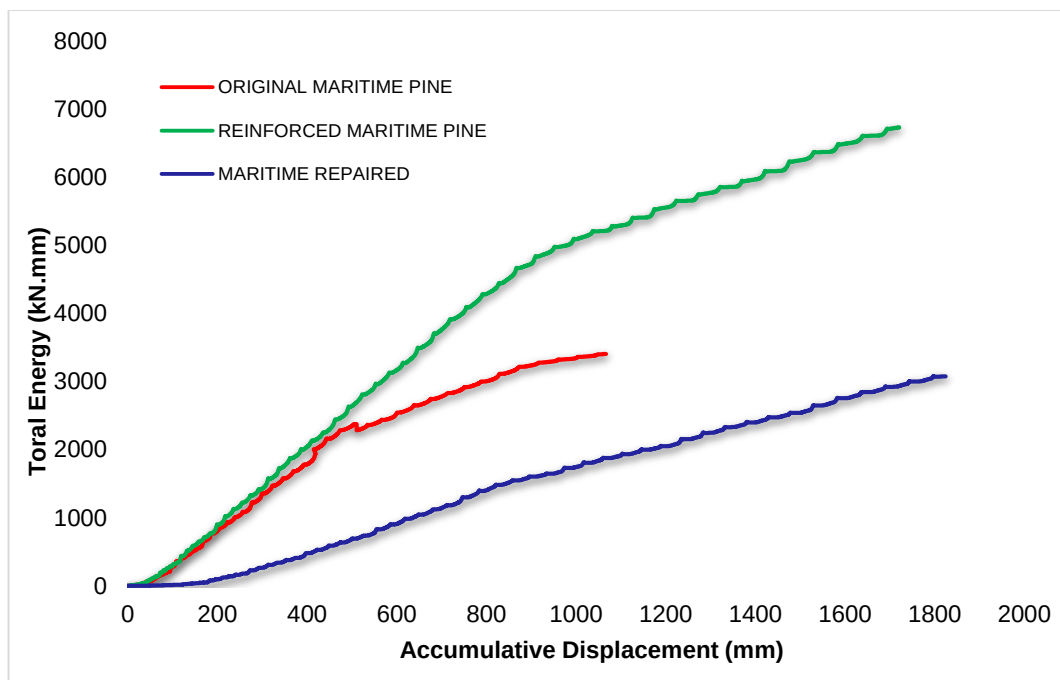


Figure 5:17 - Total Energy of original and reinforced and repaired Scott's Pine joints.

The same behaviour in terms of energy dissipation is encountered for the Maritime Pine joints, Figure 5:17, as the reinforced joints dissipates more energy than the original and reinforced joints. However, the energy dissipated by the repaired joints is less than the energy dissipated by Scott's Pine joint in higher accumulative displacement, which is a result of a lower damping coefficient for the Maritime Pine repaired joints, 0.08. Nevertheless, the total energy dissipated by the reinforced joints showed similar amount as the Scott's pine reinforced joints, 7000 kN.mm, despite of the amount of dissipated energy by joint in Maritime Pine is higher. This can be a result of the high CoV between the values of the Scott's Pine, 42.81%, is much higher than the CoV between the Maritime reinforced joints, 11.44%.

5.2.2 REINFORCEMENT WITH FIBER GLASS (GFRP)

5.2.2.1 PROCEDURE

The results of reinforcement of dowel-type timber joints in double shear, reinforced with fiberglass (GFRP), processed and discussed within this project, are a result of an experimental campaign that was carried out by (Matos,2011) as a part of a master thesis. The study included a total of 20 joints of Scott's and Maritime Pine, both original and reinforced with GFRP. The joints were tested under monotonic and cyclic loading according to EN 26891 – 1991 ^[8] and EN 12512 – 2001 ^[6], respectively. As mentioned, earlier that the work on (Matos, 2011) is included within this work for a purpose of comparison between the two reinforcement techniques, GFRP and STS. However, the previous results were obtained according to EN12512- 2001, and as specified in section 5.2.1.1, the procedure has changed in the proposal for the new version of the standards. Since the testing and material

characterization with STS were conducted following the new proposal's procedure, then in order to gain a mutual ground for a valid comparison, therefore, more reliable results, the data obtained from (Matos, 2011) was processed according to the new proposal [7].

The testing procedure on EN 12512 – 2001 differs in the definition of the steps for the cycles and the corresponding amplitudes for each step, as well as the displacement rates. Figure 5:18 shows the time-displacement curve for these amplitudes.

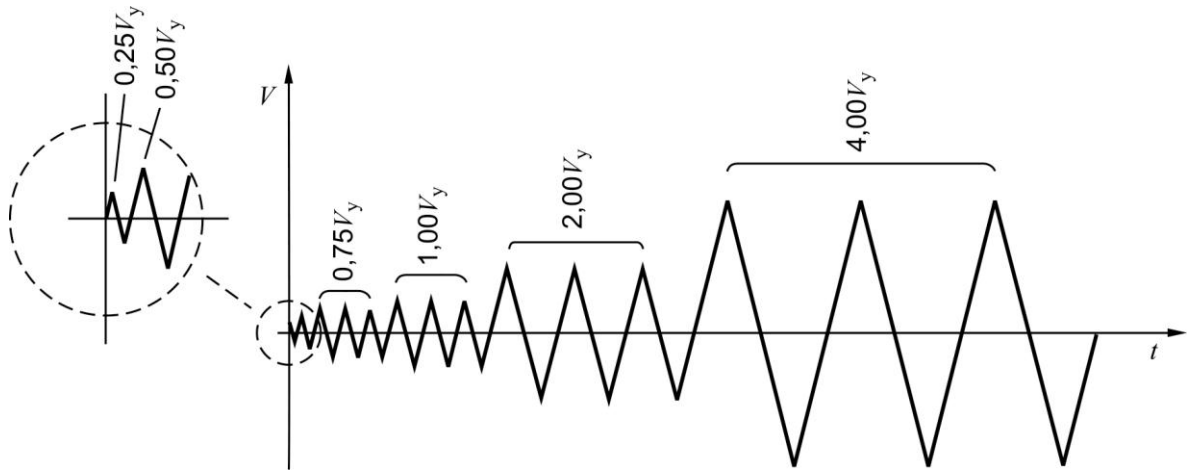


Figure 5:18 – Procedure for cyclic testing according to EN 12512 – 2001.

Similar to the approach in the new proposal, the yield slip, V_y , is obtained from monotonic tests according to EN 26891 – 1991, and the amplitudes are also a function of the yield slip. However, giving that the amplitudes that the cyclic test were based on are different, then the new amplitudes from the new standards were inapplicable and to process further, the results of the cyclic tests were processed with these amplitudes specified in Figure 5:18 above. Regarding the ductility, it was specified as the ratio between the ultimate displacement and the yield displacement. The force corresponding to the yield slip is the yield force and in the previous method these values were taken at the intersection of the elastic and plastic phases on the monotonic curve itself. While the new proposal gives these values in term of energy, as explained above. Thus these values after re-analysing the data with the new approach have differed.

As for the force impairment, it was measured for each amplitudes by means of solely the maximum values obtained in each 1st and 3rd cycles for each amplitude. The viscous damping coefficient was calculated for half cycle with the following equation:

$$v_{eq,c} = \frac{E_d}{2\pi \cdot E_p} \quad (5-7)$$

5.2.2.2 SCOTT'S PINE

Following the procedure of the new version proposal of EN 12512, the EEEP,c curves for cyclic loading for the Scott's Pine joints, both original and reinforced, were derived, and the corresponding parameters were calculated accordingly. The estimated maximum force, F_{est} , for these joints from the previous study was equal to 18.97 kN. The yield forces obtained in compression are less than the maximum forces in tension with a CoV of 10.82% and 14.62%, respectively. The yield slip of the energy curve, $V_{y,EEEE,c}$ also displayed a higher values in tension with a CoV of 33.07% opposed to a CoV of 38.99% in compression. The ductility for the joints were obtained with both approaches, as well as the force impairment and the degradation factor. The results are given in Table 5-14 below.

Table 5-14 – EEEP,c and ductility with for the original Scott's Pine joints.

ORIGINAL JOINTS_COMPRESSION										
Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
S01.U	15.28	1.36	24.52	3.17	2.39	0.75	0.32	0.24	18.02	2.62
S02.U	14.03	3.05	10.87	15.65	13.61	0.87	10.87	3.57	3.57	1.03
S03.U	15.41	1.41	15.59	17.43	14.92	0.86	10.61	7.54	11.07	1.67
S04.U	18.51	3.32	23.52	19.35	16.43	0.85	10.83	3.26	7.09	1.82
S05.U	17.01	2.70	23.24	17.21	13.92	0.81	23.24	8.61	8.61	5.62
Avg	16.05	2.37	19.55	14.56	12.26	0.83	11.17	4.64	9.67	2.55
CoV %	10.82%	38.99%	30.82%	44.64%	45.87%	5.69%	72.70%	73.51%	55.84%	70.86%

ORIGINAL JOINTS_TENSION										
Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
S01.U	-17.99	-5.13	-22.35	-21.20	-17.69	0.83	-22.35	4.36	4.36	2.84
S02.U	-11.92	-2.53	-11.62	-13.20	-11.34	0.86	-11.62	4.60	4.60	4.14
S03.U	-16.35	-3.89	-21.77	-17.30	-14.53	0.84	-10.66	2.74	5.60	1.44
S04.U	-16.57	-3.00	-23.36	-17.10	-14.75	0.86	-23.31	7.76	7.78	1.15
S05.U	-16.82	-2.45	-23.31	-16.60	-13.50	0.81	-23.31	9.51	9.51	3.04
Avg	-15.93	-3.40	-20.48	-17.08	-14.37	0.84	-18.25	5.79	6.37	2.52
CoV %	-	-	-	-	-	2.37	-	47.63	34.77	48.76
	14.62%	33.07%	24.42%	16.63%	16.00%	%	35.68%	%	%	%

Referring to the values displayed above, the ductility obtained with EEEP,c curves, $D_{u,EEEE,c}$ is around the same range of the ductility obtained by the old method, D. While in the new method, the force impairment is calculated in terms of degradation percentage at each point of the LEC curves. This can be seen with comparing the values of the degradation factor, K_{deg} , and ΔF for these joints in Table 5-14 above. The K_{deg} values varied around an average of 0.84 for tension, with CoV of 2.37%, and an average of 0.83 for compression, with a CoV of 5.69%. These values in percentage are much higher than the values of ΔF which were on an average of 2.55% and 2.52% for compression and tension, respectively. However, the variation of these values is much higher, which condemns the values as inconclusive

compared to the values of k_{deg} . The inconsistent values of ΔF opposed to the K_{deg} factor reflects the inaccuracy of this method in estimating the strength degradation of the joints. Figure 5:19 shows the 1st LEC for the 5 joints in comparison with the original joints from this study.

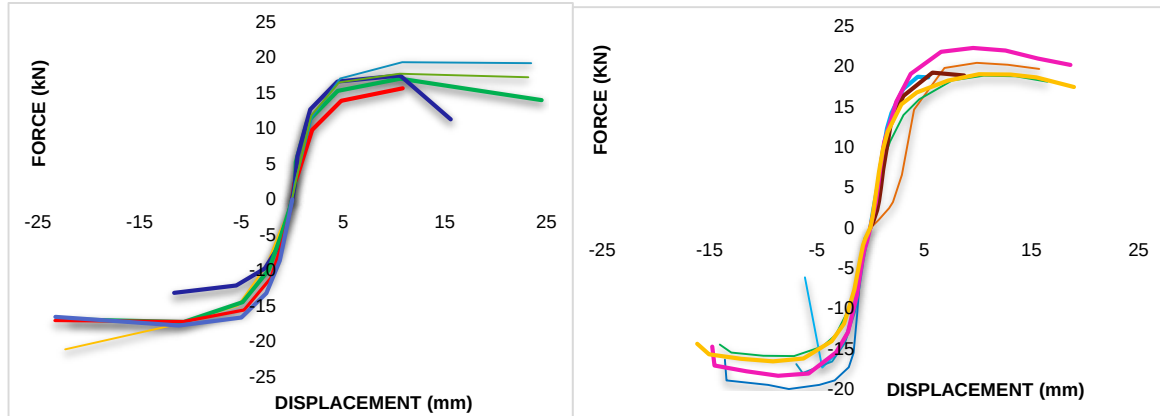


Figure 5:19 – Left: 1st LEC curves of original Scott's Pine joints from previous study, Right: 1st LEC from current study.

It can be seen from the LEC curves for both cases that the joints of the current study have higher capacity, and less stiffness and ductility. The energy ultimate displacement, $V_{u,EEEE,c}$, for the joints from the previous study was slightly higher with a percentage of increase of 1.91% but the variation between the values is 72.70%. This variation could be caused by the fact that some of the joint had a brittle behavior and failure.

On the other hand, after analyzing the data of the reinforced joints with the new method and obtaining the energy curves, $EEEE,c$, for cyclic tests as well, the values of the energy ultimate displacement increased by 52.74%, with a CoV of 45.98%. Despite that the CoV is high, this increase indicates an increase in the joints' ductility, which can be seen in Figure 5:20. This value is also significantly high when compared to the joints reinforced with STS, as the increase is 19.09%.

As for yield force, the values have increased from the original joints by 36.88% and 35.21% for compression and tension, respectively, with a CoV of 9.10% and 6.91%. The strength degradation factor values, however, were less than the values obtained for the original joints with a decrease of 14.8% with a CoV of 11.53 for compression. Nonetheless, the values for tension were slightly higher the K_{deg} values for the original joints with a CoV of 3.15% opposed to 2.37%. This indicates that the reinforcement effect was more evident than in tension. Table 5-15.

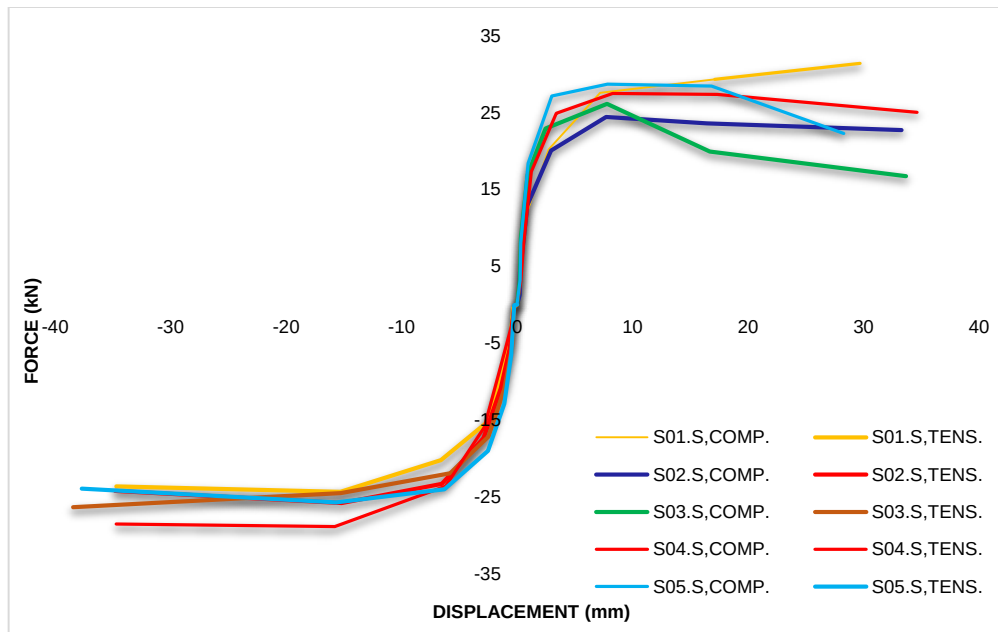
Table 5-15 – $EEEE,c$ and ductility values for Scott's Pine joints reinforced with GFRP

REINFORCED JOINTS_COMPRESSION										
Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	K_{deg}	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
S01.S	28.18	2.15	29.70	31.40	21.82	0.69	29.70	13.84	13.84	14.07
S02.S	22.90	1.32	33.30	22.72	16.66	0.73	33.30	25.28	25.28	11.44

S03.S	23.14	0.72	16.69	19.76	14.79	0.75	16.69	23.06	23.06	5.30
S04.S	26.26	1.58	34.62	25.58	15.35	0.60	30.57	19.38	21.94	2.62
S05.S	26.66	0.95	28.29	28.71	23.76	0.83	7.95	8.36	29.74	7.46
Average	25.43	1.34	28.52	25.63	18.48	0.72	23.64	17.98	22.77	8.18
CoV %	9.10%	41.41%	24.89%	18.06%	21.94%	11.53%	45.98%	38.40%	25.55%	56.39%

REINFORCED JOINTS_TENSION										
Joint Tag	F_{y,EEEP,c}	V_{y,EEEP,c}	V_{u,c} (mm)	F_{1(v)} (kN)	F_{3(v)} (kN)	Kdeg	V_{u,EEEP,c} (mm)	D_{EEEP,c}	D	ΔF (%)
S01.S	-22.79	-3.15	-34.70	-24.39	-20.06	0.82	-15.31	4.86	11.02	10.42
S02.S	-24.26	-2.88	-34.81	-25.82	-22.64	0.88	-15.19	5.27	12.07	11.44
S03.S	-24.19	-2.22	-38.45	-5.66	-4.96	0.88	-0.44	0.20	17.30	6.75
S04.S	-27.41	-4.83	- 34.71	-28.57	-23.55	0.82	-34.71	7.18	7.18	4.21
S05.S	-24.29	-1.63	- 37.69	-25.76	-21.71	0.84	-15.65	9.60	23.13	9.91
Average	-24.59	-2.94	-36.07	-22.04	-18.58	0.85	-16.26	5.42	14.14	8.55
CoV %	-6.91%	- 41.12%	- 5.11%	- 42.12%	- 41.57%	3.15%	- 74.91%	64.05%	43.77%	35.04%

As for the ductility, both with the now and the old methods, the values have increased considerably as the ductility I terms of energy calculations, $D_{EEEP,c}$ has increased to around 74.18% with a CoV of 25.55% in compression, and only 6.86% with a CoV of 64.05% in tension. Regarding the ductility in terms of ultimate displacement, D , it increased by 57.52% with a CoV of 25.55% in compression, and an increase of 54.96% in tension with a CoV of 43.77%. The energy based method did not detect an increase in the ductility for tension opposed to the old method as the values of $D_{EEEP,c}$ have decreased with the reinforcement, which is not the case with reference to Figure 5:20.

Figure 5:20 – 1st LEC for Scott's Pine joints reinforced with GFRP.

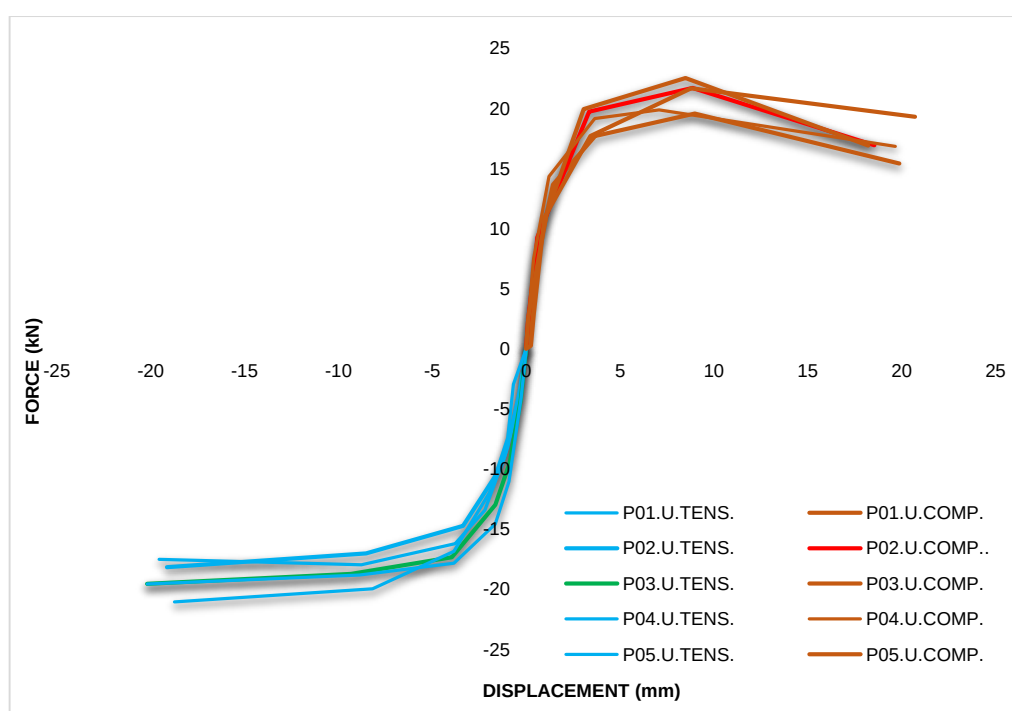
5.2.2.3 MARITIME PINE

Regarding the Maritime Pine joints from the previous study, the same procedure was followed in order to analyze the results with the new method, Equivalent Energy Elastic-Plastic Curve (EEEP). In terms of ductility, similar to Scott's Pine joints, the values of the two methods were close, as the values are higher in compression than in tension. The ductility in terms of ultimate displacement, $V_{u,c}$, which is the old method, are 21.53% higher than the values of ductility in terms of energy, $D_{EEEP,c}$, with a CoV of 63.20% in $D_{EEEP,c}$ values as well as in tension which is equal to 96.95%. K_{deg} values are within the range of the values obtained by the original Scott's Pine joints, yet the difference from the force impairment obtained by only the minimum of the amplitudes for each joint, ΔF , is large. Table 5-16.

Table 5-16 – EEEP,c and ductility values for original Maritime Pine joints.

ORIGINAL JOINTS_COMPRESSION										
Joint Tag	$F_{y,EEEP,c}$	$V_{y,EEEP,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEP,c}$ (mm)	$D_{EEEP,c}$	D	ΔF (%)
P01.U	17.51	1.38	19.87	15.42	13.10	0.85	19.87	14.36	14.36	4.71
P02.U	19.20	1.35	18.54	16.94	14.37	0.85	18.54	13.69	13.69	8.26
P03.U	19.94	2.17	18.23	16.90	14.45	0.86	18.23	8.39	8.39	6.27
P04.U	18.25	1.10	19.67	16.85	13.64	0.81	19.67	17.94	17.94	6.28
P05.U	19.64	1.37	20.71	0.42	0.25	0.60	0.26	0.19	15.17	6.99
Avg	18.91	1.47	19.40	13.30	11.16	0.79	15.31	10.92	13.91	6.50
CoV %	5.33%	27.65%	5.23%	54.34%	54.87%	13.97%	55.14%	63.20%	25.04%	19.85%

ORIGINAL JOINTS_TENSION										
Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
P01.U	-17.16	-2.18	-19.56	-17.97	-16.15	0.90	-8.78	4.03	8.97	10.14
P02.U	-16.61	-2.17	-19.14	-18.17	-15.50	0.85	-19.14	8.81	8.81	9.71
P03.U	-18.23	-1.73	-20.19	-18.74	-16.93	0.90	-9.33	5.41	11.70	17.13
P04.U	-18.48	-1.54	-20.19	-1.34	-1.19	0.89	-0.09	0.06	13.15	14.50
P05.U	-19.25	-2.16	-18.74	-7.43	-6.62	0.89	-0.98	0.45	8.67	9.25
Average	-17.95	-1.96	-19.56	-12.73	-11.28	0.89	-7.66	3.75	10.26	12.15
CoV %	-5.90%	-	-	-	-	2.23 %	-	96.95 %	19.95 %	28.71 %

Figure 5:21 – 1st LEC for original maritime Pine joints.

The reinforcement effect on these joints according to analysis based on the energy method is similar to the original joints in term of the variation of the values of static ductility and the ultimate displacement obtained by the EEEP curve. The static ductility, $D_{EEEE,c}$, is in the same range of the ductility obtained by means of the ultimate displacement $V_{u,c}$, which was originally used to calculate the ductility before the introduction of the new energy method. The values are around the same range, yet the CoV of the static ductility and consequently the energy ultimate displacement, $V_{u,EEEE,c}$ is more than 100% in both cases compression and tension, while the CoV for the ductility, D , is 39.05% in tension while the value is higher than the static ductility value. Which indicates that these values are once again inconclusive.

Table

Table 5-17 – EEEP,c and ductility values for maritime Pine joints reinforced with GFRP

Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
P01.S	23.49	2.30	18.73	25.06	21.05	0.84	18.73	8.15	8.15	3.78
P02.S	32.60	2.90	15.29	34.85	28.72	0.82	8.29	2.86	5.26	1.44
P03.S	30.49	1.57	16.86	18.34	13.19	0.72	0.90	0.57	10.75	1.52
P04.S	32.68	0.71	16.81	2.12	1.81	0.86	0.15	0.21	23.75	0.31
P05.S	28.07	3.17	18.05	27.43	23.52	0.86	7.12	2.25	5.70	3.05
Average	29.47	2.13	17.15	21.56	17.66	0.82	7.04	2.81	10.72	2.02
CoV %	13.03%	47.22%	7.71 %	57.36 %	59.36 %	7.05 %	106.23 %	113.54 %	70.93 %	68.58 %

Joint Tag	$F_{y,EEEE,c}$	$V_{y,EEEE,c}$	$V_{u,c}$ (mm)	$F_1(v)$ (kN)	$F_3(v)$ (kN)	Kdeg	$V_{u,EEEE,c}$ (mm)	$D_{EEEE,c}$	D	ΔF (%)
P01.S	-19.54	-2.11	-16.91	-21.62	-19.12	0.88	-16.91	8.01	8.01	0.40
P02.S	-21.77	-2.95	-15.17	-23.00	-20.67	0.90	-6.12	2.07	5.14	4.73
P03.S	-21.01	-1.54	-10.82	-21.97	-18.87	0.86	-10.82	7.01	7.01	2.86
P04.S	-25.79	-3.15	-15.04	-14.84	-8.31	0.56	-1.22	0.39	4.78	3.91
P05.S	-24.65	-5.74	-14.25	-15.57	-9.04	0.58	-2.38	0.41	2.48	5.74
Average	-22.55	-3.10	-14.44	-19.40	-15.20	0.76	-7.49	3.58	5.48	3.53
CoV %	11.51%	52.01%	15.54%	19.96%	39.49%	22.56 %	86.33%	102.56 %	39.05 %	57.91 %

The odd values of the static ductility for these joints can be seen from the LEC curves of the joints in Figure 5:22, as the joints had projected less ductility than the original joints which is not the case, giving the increase in the load-carrying capacity of the joints with the reinforcement. This can be related directly to the discordance of the values of the energy yield displacement, $V_{y,EEEE,c}$, as the large variation in the values that are shown above, 106.23% in compression and 86.33% in tension, can be seen in the stiffness difference between the joints. However, the stiffness has evidently increased compared to the original joints. The huge variation in the values can be a result of the adaptation of the new method with the values if the cyclic tests that were performed following the previous version of the standard, that specifies different values for the cyclic amplitudes. These values are a function of the yield displacement which is linked directly to the ductility along with ultimate displacement.

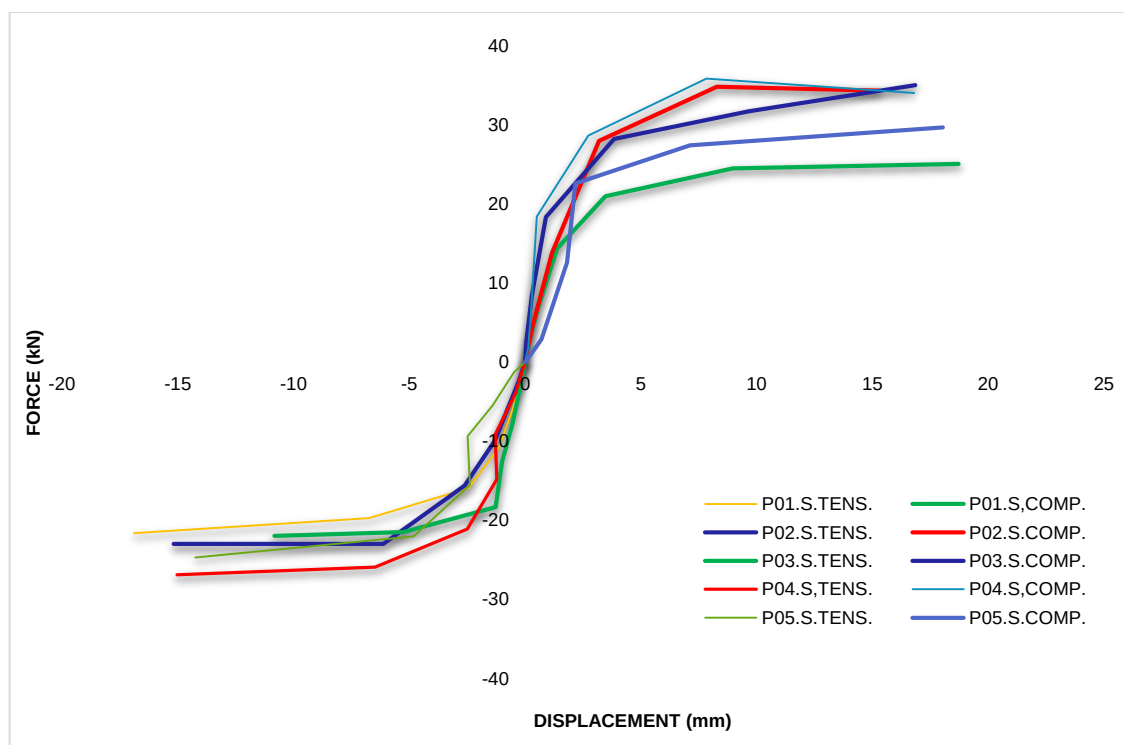


Figure 5:22 – 1st LEC curves for Maritime Pine joints reinforced with GFRP.

5.2.3 COMPARISON OF REINFORCEMENT TECHNIQUES

5.2.3.1 SCOTT'S PINE REINFORCED JOINTS

The effect of the STS reinforcement on the load-carrying capacity of Scott's Pine joints is negligible compared to the effect of GFRP reinforcement. The increase of the capacity when the joints are reinforced with STS is 5.99% of the original joint's capacity, yet for GFRP, the capacity has increased by a percentage of 8.52% of the original capacity. This indicates that the joints have improved by 29.69% with GFRP than with STS. Table 5-18.

Table 5-18 – Reinforcement techniques effect on load-carrying capacity of Scott's Pine joints

STS			
REINFORCED		ORIGINAL	
Average F_{max}	19.38	Average F_{max}	18.22
CoV %	10.63	CoV %	7.95
capacity %	5.99%		

GFRP			
REINFORCED		ORIGINAL	
Average F_{max}	22.37	Average F_{max}	20.46
CoV %	5.11	CoV %	9.13
capacity %	8.52%		

With regards to effect of reinforcement technique on static ductility, $D_{EEEE,c}$, the values for ductility have increased significantly in compression by 73.59%, but in tension, the values show that the ductility has decreased with GFRP reinforcement, which is not the case looking at the LEC for the 1st and the 3rd cycles for the original and the reinforced joints. Figure 5:23. It is possible that these misleading values for the joints with GFRP reinforcement are due to the fact that the procedure to begin the cyclic tests was different based on the previous version of the standards. Hence, and due to the fact that the tests were from a previous study and cannot be redone, the EEEEP,c curves were obtained based on these values, which are to be different if the procedure was done entirely according to the new standards.

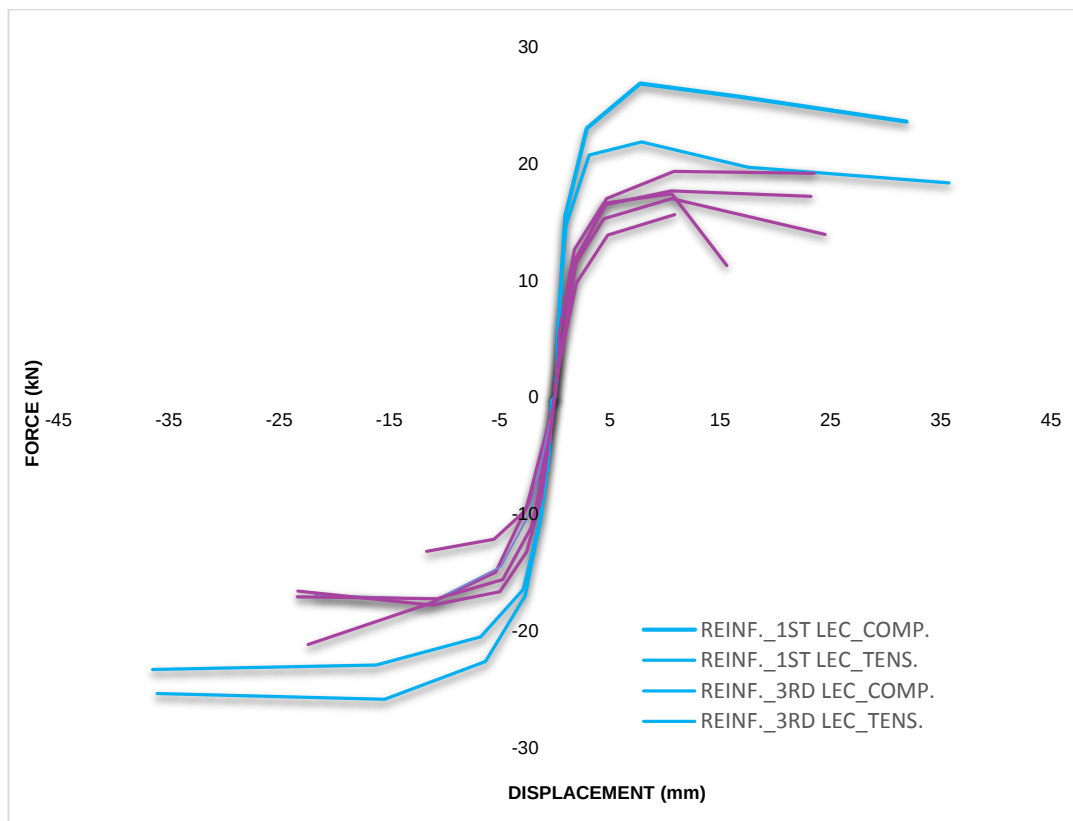


Figure 5:23 – 1st and 3rd LEC for original and GFRP reinforced Scott's pine joints.

Over and above, compared to STS reinforced joints, the ductility and capacity of the joints have increased significantly with GFRP, yet, the values of the static ductility for STS reinforced joints have shown higher percentages of increase in ductility. This further verifies the effect of building the EEEEP,c

curves based on the cyclic amplitudes obtained by the previous methodology for cyclic tests. The high CoV between the values can be a factor as well in effecting the results and having a better judgement. Table 5-19 gives the values of the ductility, and Figure shows the effectiveness of each reinforcement technique.

Table 5-19 – Static ductility values for GFRP and STS reinforced joints

GFRP REINFORCED JOINTS					
REINFORCED			ORIGINAL		
	COMPRESSION	TENSION		COMPRESSION	TENSION
Average D,EEEE,c	15.47	4.74	Average D,EEEE,c	4.08	5.03
CoV %	56.37%	70.95%	CoV %	77.21%	59.98
COMPRESSION			TENSION		
Increase %	73.59%		-6.24%		
STS REINFORCED JOINTS					
REINFORCED			ORIGINAL		
	COMPRESSION	TENSION		COMPRESSION	TENSION
Average D,EEEE,c	8.08	8.12	Average D,EEEE,c	4.82	5.48
CoV %	18.52	47.62	CoV %	41.62	58.50%
COMPRESSION			TENSION		
Increase %	40.35%		32.51%		

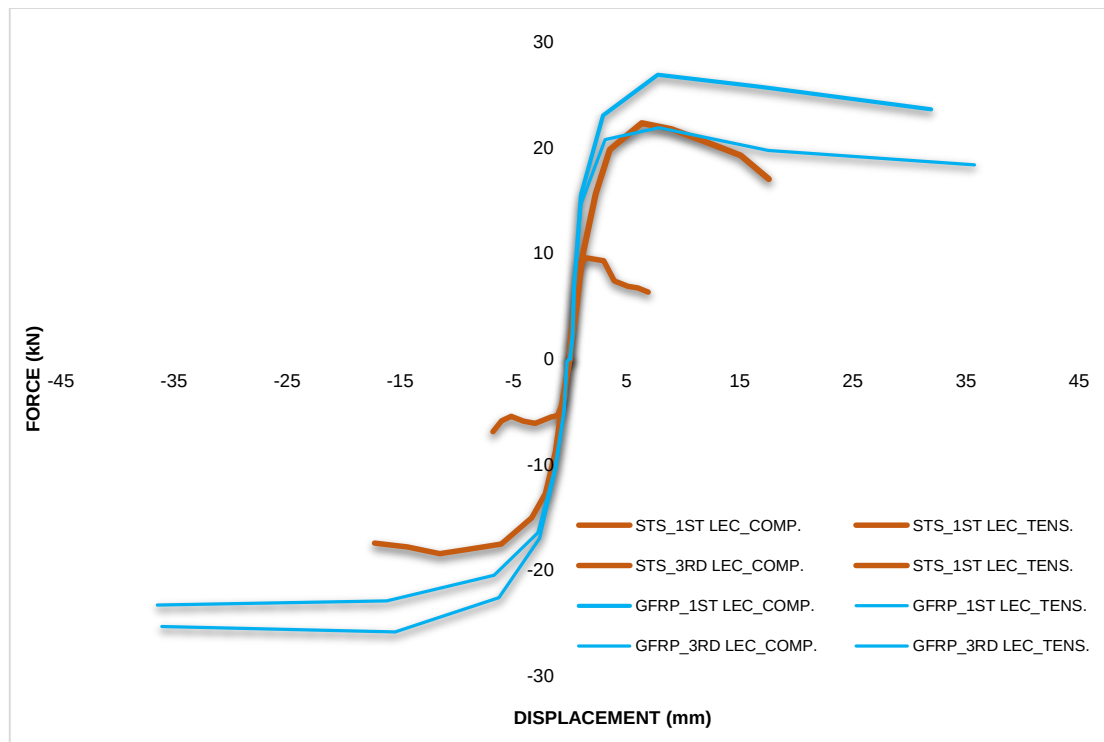


Figure 5:24 – 1st and 3rd LEC for Scott's Pine joints reinforced with STS vs. GFRP.

It can be noticed from the LEC curves that the ductility of GFRP reinforced joints is much higher than the joints that are reinforced with STS, as well as the overall load-carrying capacity. Referring to the curves, what can be also pointed out is the strength degradation of both techniques between the 1st and the third curves. With GFRP reinforcement, the strength impairment is much less evident than with STS.

5.2.3.2 MARITIME REINFORCED JOINTS

The case of Maritime joints by means of reinforcement techniques effectiveness is different from Scott's Pine case. However, regarding the efficiency in improving the load carrying capacity of the joints, GFRP have proved again to have provide higher effect on increasing the joints' capacity, as the capacity of the tested Maritime joints reinforced with GFRP has increased by 22.95 % opposed to 15.24% for STS reinforced joints. Which means that the capacity of the joints have improved an additional of 33.59% with GFRP reinforcement. Table 5-20.

Table 5-20 - Reinforcement techniques effect on load-carrying capacity of Maritime Pine joints

STS			
REINFORCED		ORIGINAL	
Average F_{max}	22.62	Average F_{max}	19.17
CoV %	1.00%	CoV %	9.61%
capacity %		15.24%	

GFRP			
REINFORCED		ORIGINAL	
Average F_{max}	24.13	Average F_{max}	18.59
CoV %	5.26%	CoV %	8.20%
capacity %		22.95%	

This effect can be seen in the 1st and 3rd LEC of the joints with GFRP and STS. As the increase in the capacity is more evident in compression than in tension, yet less than the effect of GFRP reinforcement on Scott's Pine joints. Figure 5:25.

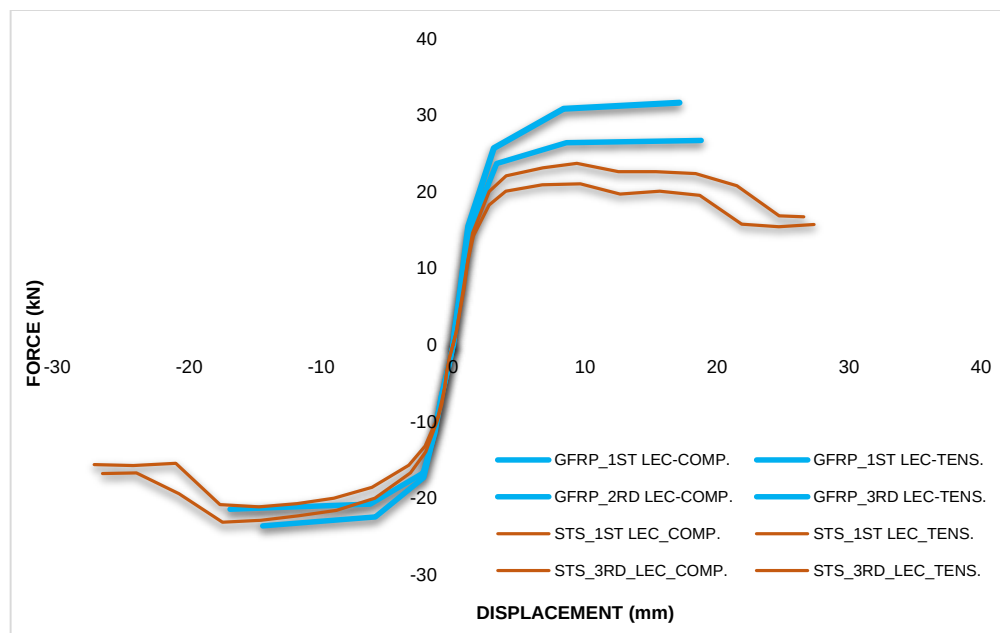


Figure 5:25 - 1st and 3rd LEC for Maritime Pine joints reinforced with STS vs. GFRP.

The strength degradation of these joints is less than the case of Scott's Pine for both GFRP and STS, yet its more obvious with STS as the force impairment is much less between the 1st and the 3rd LEC. As for the ductility, the curves show that the static ductility has improved more in the case of STS reinforcement. This can be seen also referring to the percentages of the increased ductility for both cases. For STS joints, the calculated $D_{EEEP,c}$ show that the ductility has improved after reinforcement by around 38% for both compression and tension, Which reflects the homogeneity in the results obtained with STS opposed to GFRP. Once again, with GFRP reinforcement, the CoV between the values of static ductility makes the values unreliable in this case, hence, inconclusive.

Table 5-21 - Static ductility values for GFRP and STS reinforced joints

GFRP REINFORCED JOINTS					
REINFORCED			ORIGINAL		
	COMPRESSION	TENSION		COMPRESSION	TENSION
Average $D_{EEEP,c}$	2.57	3.21	Average $D_{EEEP,c}$	9.45	3.35
CoV %	104.27%	97.96%	CoV %	72.44%	93.93%
COMPRESSION			TENSION		
Increase %	-267.88%		-4.36%		

STS					
REINFORCED			ORIGINAL		
	COMPRESSION	TENSION		COMPRESSION	TENSION
Average $D_{EEEP,c}$	11.26	8.47	Average $D_{EEEP,c}$	6.92	5.20
CoV %	8.99%	23.86%	CoV %	65.23%	67.88%
COMPRESSION			TENSION		
Increase %	38.54%		38.61%		

CHAPTER 6

6. CONCLUSIONS

The two reinforcement techniques have projected different effects by means of the joints' load-carrying capacity, ductility and strength degradation, depending on many factors other than the technique itself, such as the different densities and the mechanical properties of the different wood species. For example, in the case of Maritime Pine joints, from the LEC curves obtained, it can be concluded that the ductility for these joints has improved more with STS reinforcement. It can be noticeable as well that the stiffness in this case has also increased with STS reinforcement rather than GFRP.

Referring to the EEEP,c curves for STS reinforced joints, many joints have shown an increased stiffness so that the slope of elastic modulus is only obtainable with 30% and 70% of the maximum force. This effect was not encountered regarding GFRP, as all the tested joints had an elastic range of 10% and 40% of the maximum force. However, for the case of Scott's Pine joints, the opposite was evident for the joints, as the difference on the increased capacity is minimal in the case of STS reinforcement, but is significantly high for the joints that were reinforced with GFRP, as well as the ductility. It is important to keep in mind though that the load carrying capacity of the original Scott's Pine joints tested within the current work scope, that were later reinforced with STS was higher prior to reinforcement than the Scott's Pine joints tested on the previous study. This can indicate that the effect of the GFRP is in fact higher than the calculated value, but it decreased due to the difference in capacities of the original joints. Similarly for the static ductility parameter, the values did not coincide with the LEC curves in both cases, as for the Scott's Pine joints, the LEC curves have shown that the ductility with GFRP is much higher than the one obtained with the original joints and with reinforcement with STS, yet the values have projected that the ductility has decreased in tension which was not the case.

On the other hand, for Maritime pine joints, the values have shown a decrease in the values with reinforcement of less than a half, but with LEC curves of the 1st and 3rd cycles, the decrease was rather minimal. From the previous work, the ductility were obtained as a function of the ultimate displacement, $V_{u,c}$ instead of $V_{u,EEEP,c}$, that is calculated by means of the strength degradation factor with EEEP,c curves. This has made a huge difference in the values for both cases with very high CoV in the static ductility values, $D_{EEEP,c}$. In addition to the yield slip, V_y , that was obtained for these joints following the old method by defining the displacement that corresponds to the intersection of the elastic and the plastic modulus from the monotonic tests, while the other value of yield slip, $V_{y,EEEP,c}$, is obtained from the equivalent energy curve. However, the values of ductility in both cases differ by high CoV, with the values in tension are more comparable than in compression.

The application of EC5 [3] equations and approach for estimating the capacity of dowel-type joints, that follows the theory for the European Yield Model (YEM) and Johannes's theory, has reflected upon the importance of the slenderness of the dowels in obtaining the developed failure modes, as the estimated maximum forces prior to testing with 12 mm dowels have shown higher capacities in most cases than the actual capacities obtained by testing. Which proves that the equations were developed for a certain ratio of slenderness of the dowel. This was further examined within this work with the case of the joints

with 8 mm dowels. Although the estimation of the failure mode has shown that the expected failure is of mode (k) [3], and the joints have failed with mode (j) [3], yet the failure has improved with reinforcement opposed to the case of the 12 mm dowels. This indicates that with increasing the dowel's slenderness, more accurate the application of the equation will be.

6.1 SUGGESTIONS FOR FUTURE WORK

- Regarding the drastic variation in the static ductility values that were obtained with the EEEP,c curve for the joints from the previous study, is caused by the influence of overlapping two different methods for determining the ductility of the joints and the overall capacity in general. The EEEP,c curves for the joints reinforced with GFRP as well as the original joints prior to reinforcement application, were obtained based on the cyclic tests results that were conducted from a previous study, as mentioned earlier, and with different amplitudes for the cycle. Due to the fact that those tests cannot be redone, these values were ought to be used in order to compare the results. Thus, it is recommended for future work, that the procedure from the proposal of the new version of EN 12512, that is based on the energy equivalent elastic-plastic curves, to be performed on joints reinforced with GFRP as a full scale procedure with the recommended amplitudes, in order to have a better judgement on the effectiveness of the results.
- Despite of the remarkable results that the application of inclined STS for repairing purposes, repairing dowel-type joints generally in practice can be a rather complicated process due to many factors involved in the degradation process and various scenarios that can result in the failure of the joints. This is tied to the fact that wood is a natural material, thus it's behaviour is complicated in so many ways and is subjected to many changes that can induce more complications with time. Hence, in order to effectively repair an existing dowel-type joint that had degraded over the course of time, many parameters that should be taken into consideration in practice, were absent in this case. The timber elements that formed the joints in this project are considered sound. The degradation on the wood was only in a form of natural defects such as knots and resin bags, and only the damage caused by testing was the main parameter. Therefore, to have a better judgement on the effectiveness of STS in repairing existing dowel-type joints, a more comprehensive campaign on a real case study should be carried out. However, for cases that are similar to this case, fully threaded STS have shown rather promising results with repairing.
- A more indepth experimental campaign is recommended in order to test the effect of the dowel's slenderness on the failure mode of dowel-type joints with mechanical fastenrs. It is sugessted that this work to be continued with testing cyclic test jonts with 8 mm dowels in order to conclude on this issue and to improve the equations provided by EC5 in order to suit the case of less slender dowel's. This is rather an important step as the certain ratio of slenderness of the dowel

that can guarantee a rigid-plastic behaviour cannot always be achieved in practice. Which adds to the complexity of the design of these joints.

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8. ANNEXES

Annex 1:

Amplitudes for the reversed cycles ^[7]

Pattern	Step	Number of Cycles	Amplitude (%)	Amplitude [mm]	Displacement rate (mm/s)	Time [s]
1	1	1	0.2	0.6	0.5	4.8
	2	1	0.4	1.2	0.5	9.6
	3	3	0.6	1.8	0.5	43.2
	4	3	0.8	2.4	0.5	57.6
	5	3	1	3	0.5	72
2	6	3	1.5	4.5	1	54
	7	3	2	6	1	72
	8	3	3	9	1	108
	9	3	4	12	1	144
	10	3	5	15	1	180
	11	3	6	18	1	216
	12	3	7	21	1	252
	13	3	8	24	1	288
	14	3	9	27	1	324
	15	3	10	30	1	360