Experimental Reinforcement of Wood Beams

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Master's Thesis in Civil Engineering – Constructions Branch

Instituto Superior de Engenharia do Porto – October 5th 2013

Acknowledgments

Firstly, I would like to thank my tutor, Eng. Duarte Lopes for all the guidance and assistance given throughout the development of this thesis.

I would also like to thank Eng. Joana Almeida from Sika company (www.sika.pt) and Eng. Mário Alvim from Alto Perfis Pultrudidos company (www.alto.pt), for all the materials provided for the experimental work.

I also cannot forget the valuable support of Eng. Arnaldo Pinto and Eng. Fátima Andrade from the Laboratório de Ensaio de Materiais do Departamento de Engenharia Mecânica and Eng. Duarte Lopes from the Laboratório de Física das Construções do Departamento de Engenharia Civil, in Instituto Superior de Engenharia do Porto for providing the space and equipment necessary to conduct the experiments.

Lastly, I would like to show my gratitude to my family and friends for all the support given.

ABSTRACT

Rehabilitation is becoming more and more usual in the construction sector in Portugal. The introduction of newer construction materials and technical know-how of integrating different materials for achieving desired engineering goals is an important step to the development of the sector. Wood industry is also getting more and more adapted to composite technologies with the introduction of the so called "highly engineered wood products" and with the use of modification treatments.

This work is an attempt to explain the viability of using stainless steel and glass fibre reinforced polymer (GFRP) as reinforcements in wood beams.
This thesis specifically focuses on the flexural behaviour of Portuguese Pine unmodified and modified wood beams. Two types of modification were used: 1,3-dimethylol-4,5-dihydroxyethyleneurea (DMDHEU) resin and amid wax.

The behaviour of the material was analysed with a nonlinear model. The latter model simulates the behaviour of the reinforced wood beams under flexural loading. Small-scale beams (1:15) were experimented in flexural bending and the experimental results obtained were compared with the analytical model results.

The experiments confirm the viability of the reinforcing schemes and the working procedures. Experimental results showed fair agreement with the nonlinear model. A strength increase between 15% and 80% was achieved. Stiffness increased by 40% to 50% in beams reinforced with steel but no significant increase was achieved with the glass fibre reinforcement.

Key words: Beams, wood, modification, reinforcement, GFRP, experimental

RESUMO

A reabilitação vem-se tornando cada vez mais usual no setor da construção em Portugal. A introdução de novos materiais de construção e o conhecimento técnico de integração de diferentes materiais para atingir os objectivos desejados são importantes para o desenvolvimento do setor por um lado. Por outro lado, a indústria da madeira está também a adaptar-se às tecnologias de compósitos, com a introdução dos chamados "produtos derivados de madeira" e com o uso de tratamentos de modificação química para aumentar as suas performances.

O presente estudo investiga a viabilidade da utilização do aço e de polímeros reforçados com fibra de vidro como soluções de reforço em vigas de madeira numa escala 1:15.
Este trabalho foca-se especificamente no comportamento à flexão de vigas de madeira de pinho Português, vigas de madeira modificada com 1,3-dimetilol-4,5-dihydroxyethyleneurea (DMDHEU) e vigas de madeira modificada com cera de amido.

Na análise do comportamento foi usado um modelo não-linear. Este último simula o comportamento das vigas de madeira à flexão. As vigas com escala 1:15 foram ensaiadas à flexão de 3 pontos.

Os resultados confirmam a viabilidade dos sistemas de reforço e dos procedimentos de trabalho usados. Estes demonstram uma significativa relação com o modelo analítico não linear comparado. Um aumento de força máxima de rotura entre 15% e 80% foi alcançado nos resultados experimentais. Um aumento de rigidez entre 40% e 50% foi alcançado nas vigas reforçadas com aço mas nenhum aumento significativo foi verificado nas vigas reforçadas com fibra de vidro.

Palavras-chave: Reforço, madeira modificada, PRFV, flexão

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Notations

A'	Transformed cross sectional area
A_r	Cross sectional area of the reinforcement
A_w	Cross sectional area of wood
b	Width
b_r	Width of the reinforcement
b_w	Width of wood
C_1 ; C_2	Internal compressive forces
E ; MOE	Modulus of Elasticity
E_{frp}	Modulus of Elasticity of fibre reinforced polymer
E_r	Modulus of Elasticity of reinforcement
E_{st}	Modulus of Elasticity of steel
E_w	Modulus of Elasticity of wood
e_c	Distance to the application point of the internal compressive force
e_t	Distance to the application point of the internal tensile force
f	Frequency
f_c	Ultimate compressive strength
f_{cu_frp}	Ultimate compressive strength of fibre reinforced polymer
f_{cu_st}	Ultimate compressive strength of steel
F_{max}	Maximum load
f_t	Ultimate tensile strength
f_{tu_frp}	Ultimate tensile strength of fibre reinforced polymer
f_{tu_st}	Ultimate tensile strength of steel
h	Height
h_r	Height of the reinforcement
h_w	Height of wood
I	Moment of Inertia
I'	Moment of Inertia of the transformed cross section
I_w	Moment of Inertia of wood
l	Length of the beam
l_s	Span length of the beam
m	Homogeneity factor
M	Mass

MOE_{dyn}^{long}	Dynamic Modulus of Elasticity
MOE_{dyn_i}	Dynamic Modulus of Elasticity before reinforcement
MOE_{dyn_r}	Dynamic Modulus of Elasticity after reinforcement
MOR	Modulus of Rupture
M_u	Ultimate moment of resistance
na	Neutral axis
T_1 ; T_2	Internal tensile forces
w	Section moduli
W_o	Weight
x	Distance to the centre of gravity
z_p ; z_3	Depth of compressive plasticization
ε_c	Compressive strain
ε_{c_el}	Elastic compressive strain
ε_{c_max}	Compressive strain at the outermost compressive wood fibre
ε_{c_pl}	Plastic compressive strain
$\varepsilon_{el\_c\_frp}$	Elastic compressive strain limit of fibre reinforced polymer
$\varepsilon_{el\_c\_st}$	Elastic compressive strain limit of steel
$\varepsilon_{el\_t\_frp}$	Elastic tensile strain limit of fibre reinforced polymer
$\varepsilon_{el\_t\_st}$	Elastic tensile strain limit of steel
$\varepsilon_{pl\_c\_st}$	Plastic compressive strain limit of steel
$\varepsilon_{pl\_t\_st}$	Plastic tensile strain limit of steel
ε_r	Strain at the outermost fibre of the reinforcement
ε_t	Tensile strain
ε_{t_el}	Elastic tensile strain
ε_{t_max}	Tensile strain at the outermost composite fibre
ε_{t_w}	Tensile strain of wood
δ	Mid-span deflection
ρ	Density
σ_c	Compressive stress
$\sigma_{c,w}$	Compressive stress of wood
σ_t	Tensile stress
$\sigma_{t,w}$	Tensile stress of wood

Abbreviations

C-Glass	Corrosion-glass
CFRP	Carbon fibre reinforced polymer
CV	Coefficient of variation
D-Glass	Dielectric-glass
DMDHEU	1,3-dymethylol-4,5-dihydroxyethyleneurea
E-Glass	Electrical-glass
EBR	Externally bonded reinforcement
FFT	Fast Fourier transforms
FRP	Fibre reinforced polymer
GFRP	Glass fibre reinforced polymer
Glulam	Glued laminated wood
MOR	Modulus of Rupture
NSM	Near surface mounted reinforcement
PAN	Polyacrylonitrile
PITCH	Synthetic pitch
RH	Relative humidity
RTL	Radial tangential longitudinal
S-Glass	Strength-glass
STDEV	Standard deviation
UV	Ultraviolet

1. INTRODUCTION

Wood beams used in buildings that are subject to flexural deformation can benefit from the application of reinforcements such as steel or composite materials. Reinforcements are often applied to enhance the load bearing capabilities or stiffness properties of structural wood.
Increased stiffness without a need of increasing the depth of the beam may result in substantial space savings and material savings.

Wood is a biological material and is readily degraded by bacteria, fungi and termites. So far, wood preservation processes were the best way to prevent and/or reduce its deterioration by agents. Nowadays, traditional wood preservation and the use of tropical species are under political and consumer pressure. The environmental awareness, the increasing demand for high and constant quality and the increasing prices and availability of tropical hardwood species has led to the up-scaling and the market introduction of a number of wood modification techniques. The modification of wood with 1,3-dymethylol-4,5-dihydroxyethyleneurea (DMDHEU) has attracted increasing attention over the past years (Hill, 2006).

This work deals with the reinforcement of small scale (1:15) *Pinus Pinaster* Ait. wood beams. Stainless steel plates and glass fibre reinforced polymer (GFRP) sheets were glued to the tensile face with an epoxy resin. Three types of beams were tested: unmodified wood, DMDHEU modified wood and wax modified wood. The flexural behaviour and the strength variation of the unmodified and modified wood beams were assessed to.

1.1. Organization

This thesis is organized in 7 chapters with the first one being the introduction chapter where initial considerations are made and the objectives are defined.
In chapter II, the rehabilitation state of art is presented, more specifically, the structural retrofitting of wood structures. The most common reinforcement methods are summarized and a brief introduction on the wood modification theme is given.
At the beginning of chapter III, the mechanical properties of the materials used in the experiment are listed. Throughout the chapter, the reinforcement model schemes and procedures are described as well as the tests configurations.
In chapter IV, the analytical model is explained as well as all the assumptions and base formulations.

In chapter V all the results are presented, compared and discussed accordingly with the reinforcements and type of wood used.
The conclusions are taken in chapter VI and future research is suggested.
Finally, chapter VII lists all the references used in this thesis.

1.2. Goals and methodology

The aim of this work is to evaluate the behaviour of reinforced wood beams in bending.
As specific goals, the following three topics were assessed:

- Strengthening effect comparison between stainless steel and GFRP reinforcement.
- Flexural behaviour comparison between unmodified and modified reinforced wood beams.
- Stiffness variation before and after the reinforcement in unmodified and modified wood beams.

To reach all objectives, the following methodology was used:

- The strength and stiffness of the reinforced beams were obtained by a three point bending test.
- The dynamic Modulus of Elasticity (MOE_{dyn}) was assessed by acoustic test.
- The theoretical flexural behaviour of the beams with the different reinforcing materials (stainless steel and GFRP) was studied using an analytical model.

This study has the following limitations:

- The study does not consider any creep effects or any other long term effects. It also excludes the effects of moisture content.
- Only the effect of short term loading was studied.
- Only one wood species has been used.
- Only one type of FRP has been used.

2. LITERATURE REVIEW

2.1. Rehabilitation

In the recent years, the rehabilitation theme has been largely discussed in Portugal. The European economic crisis and the saturation of the new constructions' market were the main reasons. Recognizing this fact, construction companies pointed out the search for alternative solutions in the construction sector.

Between 2005 and 2011, the permits for new construction of residential buildings decreased by 17.2% (from 64.4% to 47.2%). In comparison with new construction, the rehabilitation works didn't decrease as much, which resulted in a relative increase in the overall sector percentage as seen in Figure 2.1.

In 2011 there were 27.790 finished constructions in Portugal, in which 6.930 corresponded to alteration, expansion and reconstruction works, meaning around 25% of the concluded works were building rehabilitations. In comparison to 2010, in 2011 there was a 3.1% increase of rehabilitated buildings; most of them (70.3%) corresponded to expansion works, according to INE (2012).

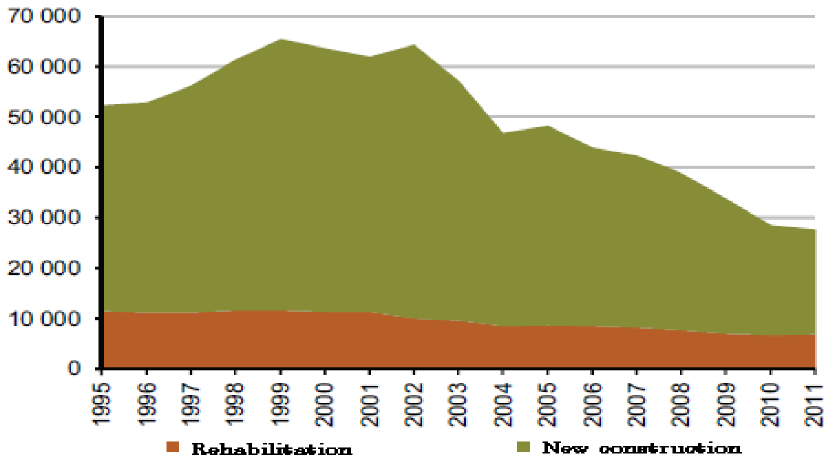


Figure 2.1 – New constructions and rehabilitation works in Portugal between 1995-2011 (INE, 2012)

The major Portuguese cities like Lisbon and Porto have started a new wave of building rehabilitation in their historical areas as the municipal entities try to bring back the population to the centre of the cities. These areas in the centres of the cities have a highly degraded habitation park. The structures of these buildings are a mix of masonry (vertical members) and wooden (horizontal members) elements which need interventions to comply with the new European standards, codes and safety regulations.

2.2. Retrofitting

Retrofitting is the process of modifying systems inside buildings or even the structure itself at some point after its initial construction and occupation. Typically this is done with the expectation of improving amenities for the building's occupants and/or improving the performance of the building.

In general, the following building structures need to be evaluated and retrofitted (Lu, 2010):

- The buildings whose serviceability or strength cannot meet the requirements of structural codes or regulations, due to misuse, irregular maintenance, aging of materials and structures.
- The buildings that have quality or safety problems due to design flaws or deficiency in construction quality.
- The buildings in which structural damages are caused by disasters such as earthquakes, strong winds, fires, etc.
- The historic buildings and memorial buildings that need to be rehabilitated and protected.
- The buildings that will be reconstructed, or have additional stories built.
- The buildings whose structural members may be changed during renovation, which may influence the performance of whole structural system.

Applications can be classified broadly into two types (Bank, 2006):

- *Strengthening*, when the original structure's strength or ductility is increased. This increase may be needed to make the structure compatible with existing building codes (mainly in case of seismic retrofitting) or due to changes in the structure use.
- *Repair* is the other type of retrofitting. In this case, an existing and deteriorated structure is retrofitted. The load capacity or ductility is brought back to the loads for which it was initially designed (and hence is, in fact, a type of strengthening). Repair is needed when the original structure has deteriorated due to environmental effects, such as corrosion of steel reinforcing in concrete structures or when the original structure has been damaged in service or was not properly constructed.

In the past few decades, studies have focused on different methods and materials to reinforce wood structures, mainly by using steel, aluminium and fibre reinforced polymers. The addition of reinforcements can increase the strength and stiffness of wood beams and is a means of both avoiding the brittle tension failure and producing a more consistent failure mode (Gilfillan et al., 2005). Table 2.1 shows a summary of the results obtained in several studies conducted on the reinforcement of wood beams with fibre reinforced polymers or steel.

Table 2.1 – Results from studies on the reinforcement of wood beams with steel or FRPs

Material	Type and % of reinforcement		ΔMOR[1]	ΔMOE[2]	Author
Glued laminated wood - Glulam	GFRP[3] (sheet)	0,04	21,5	2,3	Dorey & Cheng (1996a)
	GFRP[3] (sheet)	0,08	32,6	6,7	Dorey & Cheng (1996b)
	GFRP[3] (strip)	1,10	61,5	---	Lindyberg et al. (1998)
	GFRP[3] (strip)	3,30	119,0	---	Lindyberg et al. (1998)
	CFRP[4] (strip)	0,90	100,0	---	Blaß & Romani (2000)
	GFRP[3] (strip)	1,00	57,0	8,4	Poulin (2001)
	GFRP[3] (strip)	3,00	95,0	17,0	Poulin (2001)
	GFRP[3] (strip)	1,40	20,0	---	Haiman & Zagar (2002)
	GFRP[3] (strip)	3,50	30,0	---	Haiman & Zagar (2002)
	GFRP[3] (strip)	1,00	44,6	7,7	Dagher et al. (2010)
Natural solid wood	Steel (plate)	---	45,0	48,0	Stern & Kumar (1973)
	CFRP[4] (sheet)	0,08	42,3	22,3	Borri et al. (2002)
	CFRP[4] (sheet)	0,12	60,3	29,2	Borri et al. (2002)
	GFRP[3] (sheet)	0,87	---	27,9	Fiorelli & Dias (2002)
	CFRP[4] (strip)	0,87	---	29,6	Fiorelli & Dias (2002)
	Steel (sheet)	16,00	50,0	---	Batista & Demachi (2004)
	Steel (rod)	2,00	300,0	20,0	Gesualdo & Lima (2004)
	CFRP[4] (sheet)	0,12	52,0	25,0	Dias et al. (2006)
	CFRP[4] (strip)	0,14	54,0	7,0	Dias et al. (2006)
	CFRP[4] (strip)	0,42	18,7	---	Balseiro (2007)
	GFRP[3] (rod)	0,32	21,2	32,1	Yusof & Saleh (2010)
	GFRP[3] (rod)	1,27	24,8	60,4	Yusof & Saleh (2010)

Legend: [1]Increase in resistance (%); [2]Increase in stiffness, modulus of elasticity (%); [3]Glass fibre reinforced polymer; [4]Carbon fibre reinforced polymer.

It is noted in Table 2.1 that, even with small reinforcement percentages used in Dorey & Cheng (1996a), Dorey & Cheng (1996b) and Borri et al. (2002), the fibre reinforced polymers (further referred to as FRPs) systems allow a substantial strength increase, 20% up to 60%. In fact, the placement of FRPs, in addition to increasing the mechanical properties of the section, still has the virtue of confining the wood on which it is applied, limiting the occurrence of cracks in wood with defects which would lead to premature rupture (Fiorelli, 2002).
Studies conducted with steel, sheets in Batista & Demachi (2004), rods in Gesualdo & Lima (2004) and plates in Stern & Kumar (1973), also showed a significant increase in strength.
Overall, the increases in stiffness are not as significant as the increases in strength although Yusof & Saleh (2010) achieved increments in stiffness of around 30% to 60% by using GFRP rods inserted and glued into grooves cut along the bottom of the beams.

In the next part of this chapter, reinforcing materials and methods are described with focus on steel and fibre reinforced polymers.

2.2.1. Steel reinforcement

Steel is a widely used material in the building construction sector. It can be used as a strengthening material, most commonly in concrete structures, and individually in structural components such as beams, columns, frames and cables. On one hand, steel has a high ratio of strength to weight, high bending compression and tensile strengths, stiffness and ductility. On the other hand, steel has some disadvantages as its chemical corrosion propensity and weak fire resistance (Owens et al., 1992).
A great advantage of steel is its versatility. It can be manufactured in different strengths, shapes and product forms. This versatility also makes steel an economically sound material since it can be tailored to any given situation (Engström, 2010).
Steel is known as a ductile material and its failure can be predicted when large deformations are seen, before the member collapses. It also has good fracture toughness and it behaves as an elastic material, with a high stiffness, until the yield limit is reached. When yield limit is reached, it can withstand large plastic deformations. However, if the structural component has been exposed to fatigue, where cyclically variable stresses can cause defects and cracks, steel can break in a brittle failure mode.

Steel is the most homogeneous material used in construction. It is an isotropic material, i.e. it has the same properties in all directions, as opposed to wood. Thereby, steel has about the same strength in tensile as in compression. The shear strength however, is somewhat lower (approximately 1/3 lower). Figure 2.2 shows the idealized stress-strain relationship for steel, where: f_{tu_st} - ultimate tensile strength of steel; f_{cu_st} - ultimate compressive strength of steel; $\varepsilon_{pl\_c\_st}$ - plastic compressive strain limit of steel; $\varepsilon_{el\_c\_st}$ - elastic compressive strain limit of steel; $\varepsilon_{el\_t\_st}$ - elastic tensile strain limit of steel; $\varepsilon_{pl\_t\_st}$ - plastic tensile strain limit of steel.

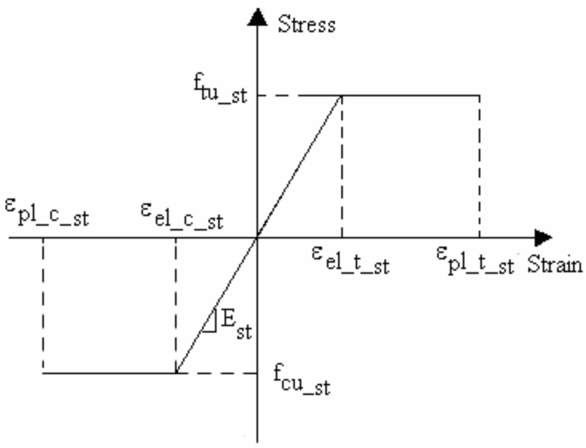


Figure 2.2 – Idealized stress-strain relationships for steel (Jacob & Barragan, 2007)

The most common techniques to reinforce wood elements with steel are:

- *Strengthening with steel sections*: There are many possible solutions. The wooden beam can be braced with U, I or H steel sections (Figure 2.3a). When the beam's height is not a problem, one can also insert a beam to support the existing beam (Figure 2.3b). The installation of a sufficient number of connectors (lag screws) has the effect of locking together the wood and steel, resulting in increased inertia which is greater than the sum of the inertias of the two beams. Figure 2.3c shows an example of the support of a wood beam by suspension. A steel beam perpendicular to the frame's initial span is fixed into the walls. The wood beams are then secured to this beam, which has a sufficient inertia, by means of stirrups (www.constructalia.com).

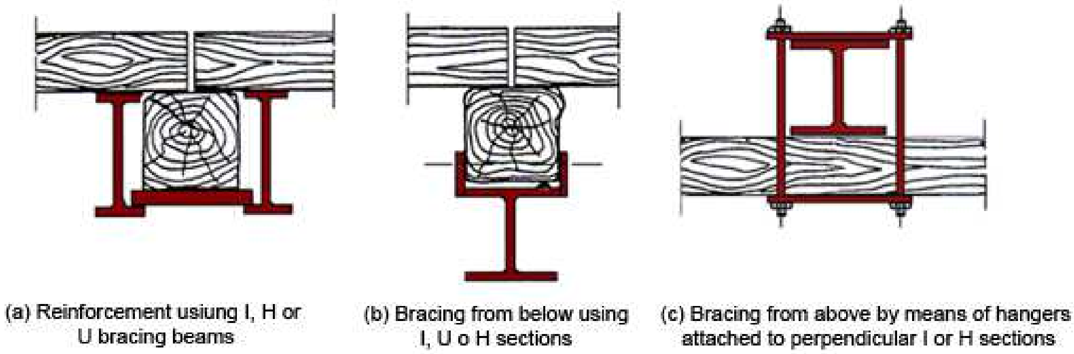


Figure 2.3 – Reinforcement of wooden beams by addition of steel sections (http://www.constructalia.com)

- *Strengthening with steel plates*: Some of the ways in which this technique can be performed are shown in Figure 2.4. Flitch beams are a classic example of a steel reinforced wooden beam which has been researched in detail during the early half of the 1970s (Stern & Kumar, 1973; Coleman & Hurst, 1974). In this method, steel plates are laminated between wood elements by mechanical or adhesive connection (Coleman & Hurst, 1974). Adhesive methods for joining reinforcements to wood are less cumbersome than drilling and fastening, but may require more time to complete since there is a period of curing during which essential cross linking takes place (Alam et al., 2010). This vertically laminated (flitch) beam may be used with original construction to make possible a larger span than would be possible with an all-wood beam of equal depth, Figure 2.4a. Nevertheless, this flitching method is time consuming. Another possibility is to reinforce a wood beam with steel plates on the top and bottom faces, Figure 2.4b. Despite not being so common, its strength and stiffness can be equal to that of a steel beam with equal depth. Ordinarily, it would be simpler and more effective to use a steel beam instead of reinforcing a wood beam in this manner (Stalnaker & Harris, 1997).

Reinforcing a wood beam with a steel plate on the bottom face only however, is a more effective way of adding to the strength or stiffness of a wood beam already installed in a structure, Figure 2.4c.

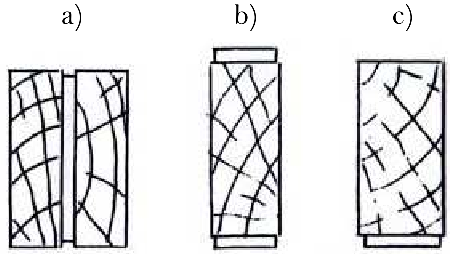


Figure 2.4 – Wood beams reinforced with steel plates (Stalnaker & Harris, 1997)

- *Replacement of the damaged wood section with steel prosthetic*: This method consists in the removal of the damaged wood section and placement of steel prosthetic connected to the sane wood element by mechanical means (i.e. clamped or haunched connections) as seen in Figure 2.5. If the damage depth on the upper or bottom sides of the beam is larger than 1/3 of its height, it should be strengthened with clamps. Otherwise, if the damage depth is larger than 3/5 of the height, the beam-end should be replaced by a new one (Lu, 2010). Nevertheless, this technique is intended to achieve a better performance on areas of interconnection between different structures (i.e., walls-floors, walls-roofing), to ensure a good overall performance of the building. Typically, this repair option is used in wooden roof structures not only because they often present supports in advanced stages of decay, but also to better ensure the consolidation of the joists and thereby global stiffening of the building (Cabrita et al., 2010).

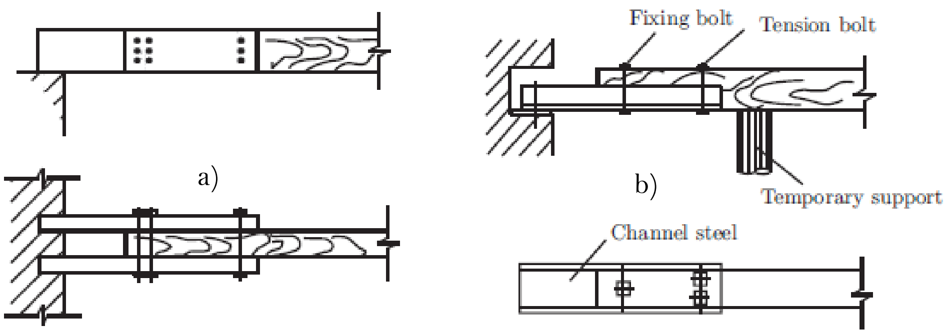


Figure 2.5 – Strengthening with: a) clamped connection; b) haunched connection (Lu, 2010)

- *Strengthening with bottom-bracing steel tension rods*: There are diverse strengthening forms with bottom-bracing steel tension rods. One simple form is shown in Figure 2.6. It can be applied to strengthen a shaky beam with small cross section, which has deficient bearing capacity or excessive deflection. Before strengthening, it is necessary to check whether the ends of the beam are corroded or moth-eaten. The steel tension rod can only be fixed well if it is of good quality material (Lu, 2010).

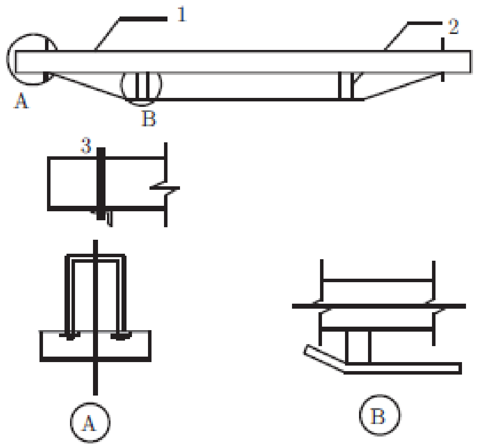


Figure 2.6 – Strengthening with bottom-bracing steel tension rods: 1 – wood beam; 2 – brace rod; 3 – steel tension rod (Lu, 2010)

- *Strengthening with steel rods:* The wooden beams can be reinforced by routing grooves along the centre of the beam axis on both the tensile and compressive faces or only on the tensile face of the beam. Steel rods are slotted and glued along the grooves as seen in Figure 2.7.

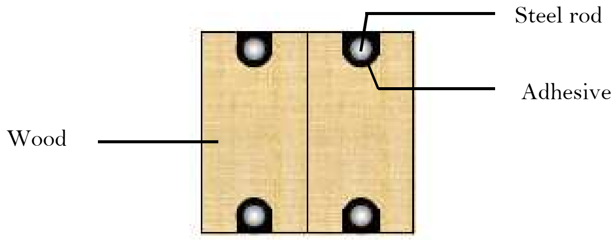


Figure 2.7 – Strengthening with steel rods (Alam et al., 2012)

- *Strengthening with flat steel hoops:* Strengthening with flat steel hoop is suitable for wooden beams subjected to longitudinal splitting damage (Figure 2.8). The flat steel hoop should be in accurate dimension and uniform configuration, and adhered to the beam. The lofting should be in full size, while the bolts need to be fastened and fixed individually with no loosing of the steel hoops. It must be noted that there should be a gap after the bayonet bolt closes. Only in this way can the bolt be fastened tightly and well adhered to the beam. The cracks on the beam should be stuffed (Lu, 2010).

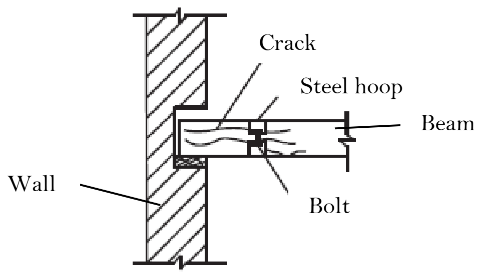


Figure 2.8 – Strengthening with flat steel hoops (Lu, 2010)

2.2.2. Fibre reinforced polymers (FRP)

Fibre reinforced polymers are engineered materials where strengthened fibres are joint in a resin matrix to create a material with extraordinary properties. The FRPs were most commonly used in aerospace and military applications. An increased choice in raw materials, which is better, cheaper and with simpler manufacturing methods have enabled FRPs to be competitive in more application areas (Peters, 1998). Today, FRP is already used on concrete or wooden structures like bridges, mainly to strengthen weakening structural parts. According to Meier (1997), the first bridge to be reinforced with carbon FRP was the *Kattenbusch* bridge (Germany) between 1986 and 1987. Another pioneer example was the reinforcement with glass FRP of the *Ibach* bridge in Switzerland in 1991 and its success might have contributed for the subsequent use of FRPs in bridge repairs like the *Oberriet Rhine* bridge (1996) and *Furstenland* bridge (1996) in Switzerland, amongst many others worldwide. In Portugal, studies like Juvandes (1999), Silva (1999) and Dias (2000) were conducted in the late 1990s early 2000s to validate the use of carbon FRPs as a reinforcing system in general. More specifically, the repair and reinforcement of bridges was studied in Juvandes et al. (1998) and Oliveira & Figueiras (1999) in the case of the *Nossa Senhora de Guia* bridge in *Ponte de Lima* (Portugal).

Figure 2.9 – Bridge strengthening with FRP (Ibach bridge, 1991, Switzerland) (Cress, 2000)

Structural composites like FRPs are a mixture of two or more components, one of them being a long and stiff fibre and the other an adhesive, resin or matrix, between the former fibres. Fibres are generally stronger and stiffer than the matrices. Fibres also show anisotropic behaviour, which means that they have different properties in different directions.

The fibres may be placed in one direction, making it a unidirectional composite. However, fibres may also be woven or bonded in many directions and the composite becomes bi- or multidirectional.

The manufacturing of composites can be made by a number of different methods: Hand lay-up, pultrusion, filament winding, and moulding. The composites mechanical properties are dependent on the fibres, matrix, fibre amount and fibre direction. Also the volume and size of the fibres will affect the mechanical properties. The fibre content by volume is normally 30% - 60%, depending on materials, manufacturing process and desired properties (Carolin, 2003). The composites can also be tailored to have specific properties, for example temperature-resistance and electrical conductivity, by a correct choice of fibres and matrices (Persson & Wogelberg, 2011).

Some of the commonly cited advantages of FRPs over more conventional materials like steel include (ISIS, 2006):

- High weight-to-strength ratios;
- Outstanding durability in a variety of environments;
- Ease and speed of installation, flexibility, and application techniques;
- Electromagnetic neutrality;
- The ability to tailor mechanical properties by appropriate choice and direction of fibres;
- Outstanding fatigue characteristics (carbon FRP); and low thermal conductibility.

However, FRP materials also have a number of potential disadvantages. Foremost among these disadvantages is the initial material cost of FRPs which can be several times that of steel. However, when the cost of a structure is considered over its entire life cycle, the improved durability offered by FRP materials can make them the most cost-effective material in many cases (ISIS, 2006).

Figure 2.10 – Assorted FRP products currently used for rehabilitation or reinforcement (ISIS, 2006)

Figure 2.11 shows the idealized stress-strain relationship for FRP where: f_{tu_frp} - ultimate tensile strength of FRP; f_{cu_frp} - ultimate compressive strength of FRP; $\varepsilon_{el\_c\_frp}$ - elastic compressive strain limit of FRP; $\varepsilon_{el\_t\_frp}$ - elastic tensile strain limit of FRP; E_{frp} – Modulus of Elasticity of FRP.

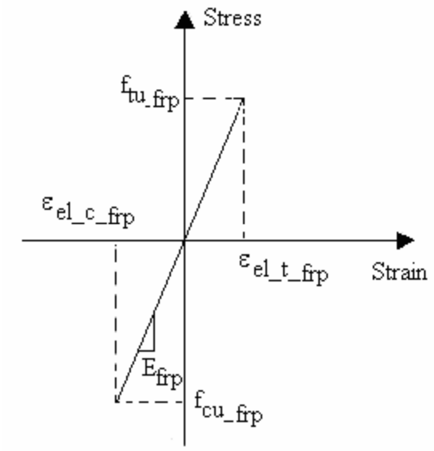


Figure 2.11 – Idealized stress-strain relationships for FRP (Jacob & Barragan, 2007)

2.2.2.1. Matrices

While the fibres' function is to carry the load, the matrices' main functions are to keep the fibre in place, transfer the loads, protect the fibres and carry inter-laminar shear.

The polymer matrix generally accounts for 30-40% of a FRP composite material. The polymer matrix also protects the fibres from the environment and mechanical abrasion (Jacob & Barragan, 2007).

Matrix materials for FRPs can be grouped into two broad categories: Thermoplastics and thermosetting resins. Thermoplastics include such polymer compounds as polyethylene, nylon, and polyamides, while thermosetting materials include epoxies and vinyl esters.

Almost exclusively, thermosets are currently used in structural engineering applications (ISIS, 2006). These polymers generally have good thermal stability at service temperatures, good chemical resistance, and display low creep and relaxation properties in comparison with most thermoplastics. However, thermosets cannot be reversibly softened and will deteriorate irreversibly at elevated temperatures, so FRP components made from thermoset matrices must be bent or formed during the manufacturing process. Three specific types of thermosetting resins are commonly used in the manufacture of infrastructure composites: polyesters, vinyl esters, and epoxies. Table 2.2. shows some properties of these thermosetting polymer resins.

Table 2.2 – Approximate properties of thermosetting polymer resins (Bank, 2006)

	Density [g/cm³]	MOE [GPa]	Tensile strength [MPa]	Max elongation [%]
Polyester	1,2	4,0	65	2,5
Vinyl ester	1,12	3,5	82	6,0
Epoxy	1,2	3,0	90	8,0
Phenolic	1,24	2,5	40	1,8

2.2.2.2. Fibres

Many different types of fibres are available for use and all have their respective advantages and disadvantages. In civil engineering applications, the three most commonly used fibre types are glass, carbon (graphite), and to a lesser extent, aramid. The suitability of the various fibres for specific applications depends on a number of factors including the required strength, stiffness, durability considerations, cost constraints, and the availability of component materials.

Glass fibre is made by creating long fibres from melted glass. They are the most inexpensive, and consequently, the most commonly used fibres in structural engineering applications. There are several different grades available, but the most common are E-glass (low electrical conductivity) and the more expensive, but stronger, S-glass (higher mechanical properties). Glass fibres are characterized by their high strength, moderate modulus of elasticity and density, and by their low thermal conductivity. Glass fibres are often chosen for structural applications that are not weight critical (glass FRPs are heavier than carbon or aramid) and that can tolerate the larger deflections resulting from the comparatively low elastic modulus of the glass fibres (ISIS, 2006).

Carbon fibres can have properties that vary widely, and so several classes of carbon fibres are available, differentiated based on their elastic moduli. Although considerably more expensive than glass fibres, carbon fibres are beginning to see widespread use in structural engineering applications such as pre-stressing tendons for concrete and structural FRP wraps for repair and strengthening of reinforced concrete beams, columns, and slabs. Their steadily increasing use can be attributed to their steadily decreasing cost, high elastic modulus and available strengths, low density (low weight), and their outstanding resistance to thermal, chemical, and environmental effects. Carbon fibres are an ideal choice for structures which are weight and/or deflection sensitive (ISIS, 2006). It has high resistance to fatigue but is sensitive to impact and has a sudden and brittle failure. It is also a very conductive material and can create galvanic cells when it comes in to contact with metals (Jacob & Barragan, 2007).

Aramid fibres are manufactured from a synthetic compound called aromatic polyamide in a process called extrusion and spinning. Two stiffness grades are readily available: 60 GPa and 120 GPa (ISIS, 2006). Aramid fibres are characterized by a distinct golden color, high strength and moderate elastic modulus (lower than carbon fibres), low density and very high impact resistance. It is also very resistant to heat and chemicals. In addition, FRPs manufactured from aramid fibres have low compressive and shear strengths as a consequence of the unique anisotropic properties of the fibres. Aramid fibres are also susceptible to degradation from exposure to ultraviolet radiation and/or moisture, which makes it less desirable for beam strengthening purposes (Jacob & Barragan, 2007).

Table 2.3 shows the major properties of commonly used fibres.

Table 2.3 – Fibre properties (Jacob & Barragan, 2007)

Fibres	Diameter [µm]	Density [g/cm³]	MOE [GPa]	Tensile strength [MPa]	Elongation [%]
E-Glass	8-14	2,54	72,4	3450	1,8-3,2
C-Glass	-	2,49	68,9	3160	1,8
S-Glass	10	2,49	85,5	4590	5,7
D-Glass	-	2,14	55,0	2500	4,7
PAN Carbon	7-10	1,67-1,9	57-228	1720-2930	0,3-1,0
Pitch Carbon	10-11	2,02	345	1720	0,4-0,9
Rayon Carbon	6,5	1,53-1,66	41-393	620-2200	1,5-2,5
Kevlar-29	12	1,44	62	2760	3-4
Kevlar-49	12	1,48	131	2800-3790	2,2-2,8
Kevlar-149	-	1,47	179	3620	1,9

The figure bellow shows the stress-strain relationship of some typical types of fibres.

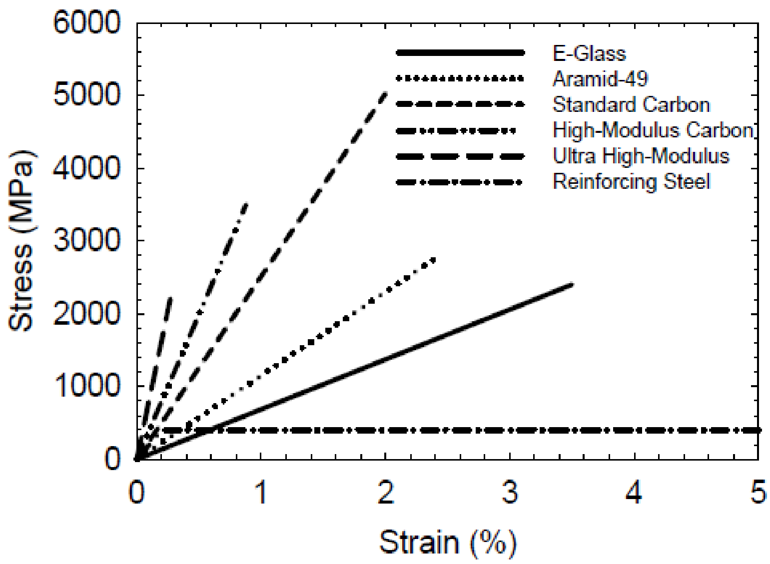


Figure 2.12 – Stress-strain properties of typical fibres (ISIS, 2006)

According to Marques (2008), the FRP reinforcing techniques are:

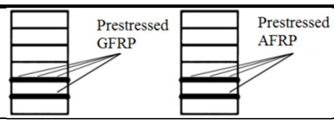
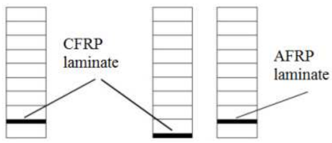
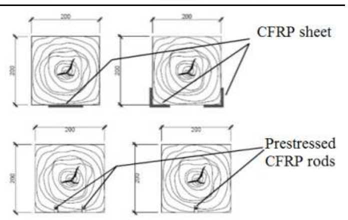
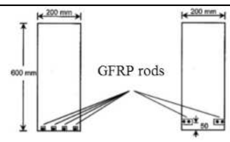
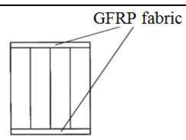
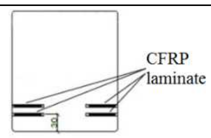
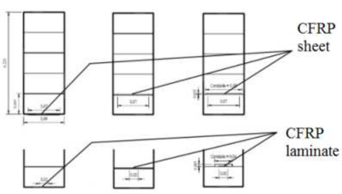
Externally Bonded Reinforcement (EBR) – consists on bonding the composite to the surface of the wood element with an adhesive resin. This technique is mainly intended to increase the load bearing capacity of structural elements, with an increase of ductility, but with no substantial change of the element's stiffness.

Near Surface Mounted Reinforcement (NSM) – consists on inserting and bonding the composite to notches previously made on the wood surface. This technique requires a preparation work of the substrate resulting in more time consumption than the EBR technique but with the advantage of protecting the reinforcement.

Internal Bonding – Consists on a bonding technique applied on the glulam production phase. In general, the composite can be glued on a sandwiched face between the lamellas of a beam. This solution has, on one hand, aesthetic benefits and on the other hand, an increase to the protection of the reinforcement.

Table 2.4 summarizes case studies of FRP reinforced wood beams.

Table 2.4 – Compilation of studies about FRP reinforcement of wood beams (Barbosa, 2008)

Author	Wood	FRP system	Reinforcement technique	Scheme
Dolan et al (1997)	Glulam	GFRP	Internal prestressed	
		AFRP	Internal prestressed	
Blaß & Romani (2000)	Glulam	CFRP laminate	External (EBR)	
		CFRP laminate	Internal	
		AFRP laminate	Internal	
Borri et al (2002)	Solid	CFRP fabric	External (EBR)	
			Corner external (EBR)	
			Prestressed external (EBR)	
			Corner prestressed (EBR)	
		CFRP rods	Prestressed (NSM)	
Gentile et al (2002)	Solid	GFRP rods	Near Surface (NSM)	
Lopez-Anido & Xu (2002)	Glulam	GFRP fabric	External (EBR)	
Parisi & Piazza (2003)	Solid	CFRP laminate	Near Surface (NSM)	
Dias et al (2006)	Glulam	CFRP sheet	External (EBR)	
			Internal	
			Internal with cavity	
		CFRP laminate	External (EBR)	
			Internal	
			Internal with cavity	

The existing codes and guidelines for the design of FRP strengthening systems are mainly focused for concrete or masonry structures and can be found in Canada, United States of America, Italy, United Kingdom, Switzerland and Japan. The following list summarizes these design codes and guidelines:

Table 2.5 – List of Design Codes and Guidelines for FRP composites in Structural Engineering (www.iifc-hq.org)

Country	Year	Design Code or Guideline
Canada	2007	CSA-S806-02
	2010	CSA-S807-10
	2001	ISIS Design Manual no. 3
	2001	ISIS Design Manual no. 4
	2001	ISIS Design Manual no. 5
	2006	ISIS Product Certification
	2006	ISIS Durability Monograph
United States of America	2006	ACI 440.1R-06
	2004	ACI 440.4R-04
	2008	ACI 440.2R-08
	1997	AC125
	2001	AC178
United Kingdom	2004	TR55
	2003	TR57
Japan	1995	BRI
	1997	JSCE
	2001	JSCE
Italy	2004	CNR-DT 200/2004
Switzerland	2001	SIA Norm 166
International	2001	*fib* bulletin no. 14
	2006	*fib* bulletin no. 35
	2007	*fib* bulletin no. 40
	2008	ISO 10406-1
	2008	ISO 10406-2
	2013	ISO 14484:2013

2.2.3. Combined glued laminated wood

The glued-laminated wood (glulam) elements are constituted by wood lamellas, graded and selected, juxtaposed, grain oriented in the longitudinal direction. The lamellas are strongly connected by suitable adhesive. The ability to make connections between lamellas, end to end (longitudinal) and overlapping (transverse) makes it theoretically possible to produce elements of any cross section, length and shape. As a result of the production process and the smaller section of the lamellas compared to the solid wood equivalent, natural defects in the wood are more scattered by the section of the element, making the material more homogeneous. The lower dimensions evidenced by lamellas ensure an even better drying, which results in a better control of moisture content of the set. They are, for all this, less dependent from localized defects (knots, resin pockets, etc.) decreasing the variability of their resistant characteristics, leading to higher values of average strength and stiffness than those in solid wood (Dias et al., 2006).

Figure 2.13 – Glued laminated wood (Boise Cascade, 2013)

The strength of an element is mainly conditioned by the external lamellas, according to the stresses normally developed in the cross section (Figure 2.14).

The placement of better quality lamellas in the extreme fibres (outside), while the lower quality ones are in the center, not only allows controlling the desired resistance, as well as a better and more rational use of wood. Thus emerged beams with lamellas of wood with distinct resistances (Combined Glulam).

Figure 2.14 shows the flexural stress diagram of a combined glulam where: h - height; E_1 – Modulus of Elasticity of material 1; E_2 – Modulus of Elasticity of material 2.

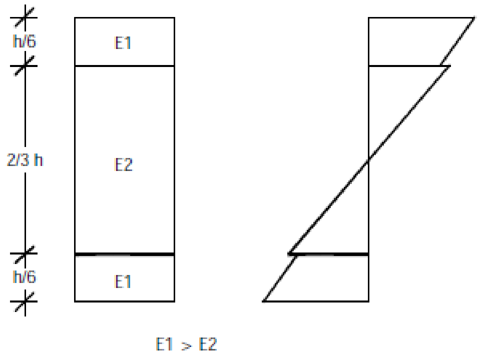


Figure 2.14 – Flexural stress diagram of a combined Glulam (Lopes, 2005)

In the following (Table 2.6), extracted from EN 1194, the combinations of wood lamellas required for obtaining each class of Glulam are shown.

Table 2.6 – Lamella grade according to the desired Glulam grade (EN 1194:1999)

Desired Glulam grade	GL24	GL28	GL32
Homogeneous Glulam	C24	C30	C40
Combined Glulam: Exterior/interior	C24/C18	C30/C24	C40/C30

2.2.4. Wood-concrete systems

The practice of replacing old wooden structures with concrete structures has obvious drawbacks. The weight gain by replacing a wooden floor with concrete could penalize the overall security of the building.

In addition to the increase of the vertical loads on the walls, this increase in weight gives a proportional increase of seismic forces. This type of intervention also leads to a mischaracterization of the old buildings, which may represent an irreversible loss of asset and architectural value.

A solution that increasingly gains more followers is the conversion of flooring systems in a mixed wood-concrete structure, resulting in a set with excellent structural and aesthetic features.

The use of this technique can capitalize all existing equipment, since the beams still have an important structural function, while the floorboards are used as natural formwork for the concrete slab.

The connection between the two materials may be performed in different ways, but the simplest refers to the use of easy to apply metal fasteners (nails, screws, washers, steel bars, etc.).

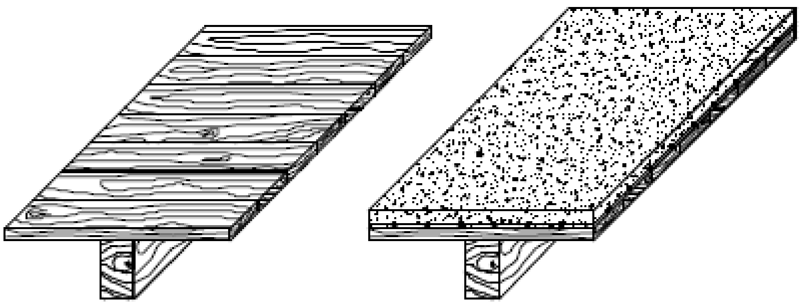


Figure 2.15 – Transformation of a traditional wooden floor into a composite wood-concrete floor (Branco & Cruz, 2002)

The transformation in a mixed set takes advantage of the best properties of the two materials, combining strength, stiffness and fire protection afforded by the concrete and the tensile strength and lightweight of wood material.

Concrete or light-concrete is used in compression, providing strength and rigidity. Wood is used in tensile, thus eliminating the use of concrete as passive load only (Ceccotti, 1995). Concrete-wood mix structures are an efficient solution with high stiffness and lightweight at the same time.

The original bearing capacity may be doubled, its bending stiffness increased three to four times and in-plane stiffness can be regarded as infinite, according to Lopes (2005).

The transformation of the floors in composite slabs has other advantages in addition to increased resistance: reduction of vibration; acoustic insulation (60 dB) and fire protection (F30, F60 and F90) (Natterer et al., 1998).

But the field of application of composite wood-concrete systems is not limited to the rehabilitation of old buildings. This solution has a huge potential, particularly in the pre-fabrication of new buildings. The combination of features such as low weight / resistance, the reduced frequency of vibration and excellent thermal and acoustic insulation with an industrialized process, leads to a competitive constructive solution.

There are various known applications in new structures, especially in bridges and footbridges: Lao River bridge (Italy), Crestawald bridge (Switzerland), Vihantasalmi (Finland) (Figure 2.16); and building slabs: Neuchâtel museum (Switzerland), Montalcino restaurant (Italy), Triesenberg school (Finland), Swiss National Expo 2002 (Switzerland), Casa da Renda (Portugal) (Fontes & Branco, 2004), etc…


Figure 2.16 – Vihantasalmi bridge, Finland

The mixed wood-concrete floors are traditionally dimensioned with simplified equations and very high safety factors in relation to those for other structures (Cruz, 2000). The dimensioning is often performed disregarding the contribution of the wooden structure.

The methods normally applied are distinguished according to the rigidity of the connection between the two materials. If the connection is rigid, the Bernoulli hypothesis can be accepted, making the calculation very simple. Homogenizing the section of one material, wood or concrete, into the other, is sufficient to obtain loads and deformations of the section and allows the application of the basic equations of the mechanics of materials. The section is no longer considered plane when the connection is not rigid. The appearance of small horizontal slips between the two materials makes it necessary to quantify the relative slip between the two materials. The relation between the slip and the force that originates it is translated by a slip coefficient.

With the EN 1995-1-1 (2004), the design of wood-concrete mixed sections is facilitated with the use of simplified equations based on the calculation of the effective flexural stiffness and stress distribution, as a function of stiffness of the connection between the two materials (Branco & Cruz, 2002).

When executing this system, Cardoso (2010) describes some recommendations:

- The usage of dried wood is highly recommended. If this is not possible, it is important to assure that the cracks do not affect the connectors and that the behaviour of the connectors is minimized under the proximity of a crack. Whenever possible, the wood material should be put in the work site before the execution in order for it to be influenced by the humidity of the location.
- It is recommended to use anti-corrosion metal connectors so that the water in the concrete and in the wood itself does not degrade the connectors.
- The reinforcement of the concrete must be well thought out especially when a concrete slab with a slightly larger thickness is desired. The electro-welded mesh reinforcement should also be placed in the lower part of the concrete where stresses may crack the concrete in the non-visible wooden floor-concrete or concrete-formwork connections.
- The shoring should be provided not only to create a counter-deflection but also to withstand the loads imposed by the fresh concrete.
- During the execution it is necessary to protect the wood from the water contained in the concrete and that can be guaranteed by the use of waterproof insulations or waterproofing additives to reduce the water/cement ratio in the concrete.

2.2.5. Strengthening with new wooden elements

The introduction of wooden elements is widely used in rehabilitation/strengthening interventions on wood structures, as it allows the maintenance of a structure with similar characteristics to the existing structure. Several examples of interventions done in Porto (Portugal) are described in Costa et al. (2007a; 2007b; 2007c; 2007d) and Ilharco et al. (2007a; 2007b; 2007c).

Numerous possibilities can be used, i.e., coupling of new wooden parts to reinforce degraded elements with or without their replacement; adding new wooden beams parallel to the span of the old element; adding uprights to create middle support points; etc…
Yet for this to happen, it is very important that the new wood elements are from the same wood species with natural features such as strength and stiffness similar to the existing wood. Appleton (2003) states that it is appropriate to adopt old dried woods of good quality. In turn, the moisture content at application must be compatible with the wood structure to avoid physical incompatibility problems.

Still, it is not always possible to get the same wood as the existing; either there are no commercial sections available or not enough time to dry the wood to ensure a similar performance (Zoreta, 1986). This limitation can be mitigated by taking advantage of wood originated from the demolition of old buildings or storing the new wood pieces in the building where they will be installed to acquire moisture equilibrium with the environment (Dias, 2008).

2.2.5.1. Fixation of new wood elements without removing the degraded part

The addition of new pieces of wood to strengthen the beams, as illustrated in Figure 2.17, is a solution commonly used in the rehabilitation of elements with large fissures and with localized damages. The main goal of the addition is to increase inertia in cases where the wood section is insufficient to withstand the existing loads in acceptable conditions (Dias, 2008).

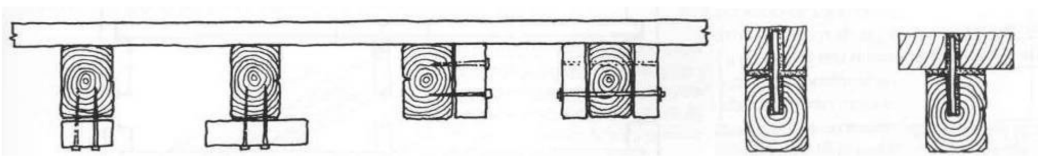


Figure 2.17 – New wooden elements or beams connected to the older structure (Arriaga et al., 2002)

The simplest solution consists in the coupling of new wood pieces in one or both sides of the old structure (Figure 2.18), connecting them with nails, bolts, plates or metal straps. It is also possible to use epoxy resins to strengthen the connections, which has the advantage of filling in any irregularities on the beam surface (Arriaga et al., 2002).

A good design rule indicates that the new wood elements must have the same height as the existing elements (Cruz, 1993) and its width, when two elements are used, is at least half the width of the existing beam (Appleton, 2003). The structural design of the connectors must be taken care of in compliance with the regulations of the EN 1995-1-1, 2004 (EC5), particularly with respect to spacing, distance between the ends of the part and the wood section reduction that entails.

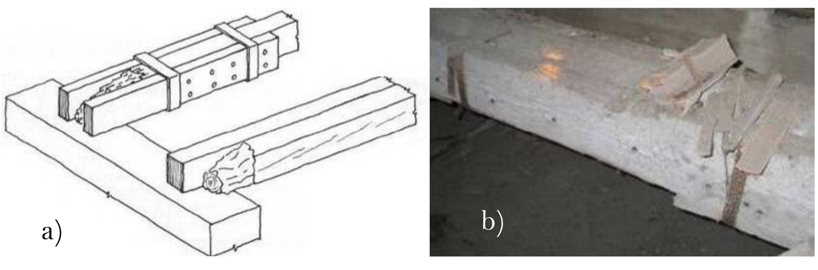


Figure 2.18 – a) Attachment of new wood pieces to the old structure (Arriaga et al., 2002); b) New wood elements linked to the degraded element through metal straps in Convento de Corpus Christi, Vila Nova de Gaia (Costa et al., 2007d)

This reinforcement solution is usually aimed at restoring the resistant capacity of broken or weakened elements with large fissures (Duarte, 2004). It has the advantage of not involving removal operations which are usually time-consuming and require the shoring of the floor. However, it has the disadvantage of needing a more careful wood preservative treatment, as new wooden elements will be in contact with decayed elements that are subject, in most cases, to the attack of biotic agents.

2.2.5.2. Replacement of the degraded parts with new wood elements

This solution involves replacing the damaged parts of the beam with new wood parts. Unlike the previous solution, and since the area that supports the beam on the wall is removed, it is necessary to execute a temporary shoring of the structure.

The connection of the new wooden element to the sound beam can be made through pieces of wood or metal plates mechanically fastened together with steel bolts, nails or screws on each side of the beam as seen in Figure 2.19. The connection to the wall can be reinforced with metal elements inserted in the wood and bolted to the wall.

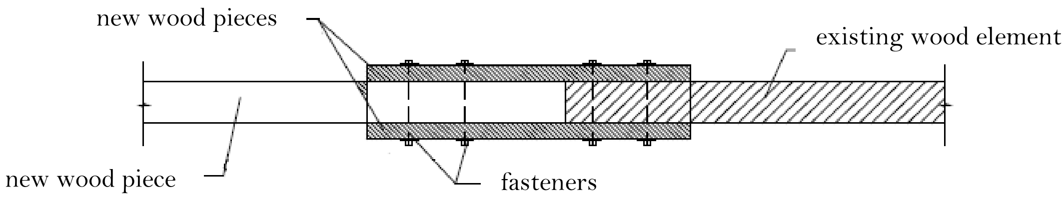


Figure 2.19 – Replacement of a degraded part of a wood beam with new wood parts (top view) (Dias, 2008)

In Arriaga et al. (2002), a complete description is made of two researches carried out by Mettem et al. (1993) and Landa (1997; 1999), where the flexural and stiffness efficiency of the connections between old elements and new is evaluated.

Mettem et al. (1993) indicates that solutions with oblique splicing fixed by mechanical means have very low efficiencies and may only be used on beam areas with low bending stresses, as is the case of support areas. However, Arriaga et al. (2002) states that in that study, situations of high shear stresses with low bending stresses were not analyzed, and that the tests were only performed in pure bending.

Among the three reinforcement solutions studied by Mettem et al. (1993), the most effective presented a splicing along a vertical oblique plane with metal bolts connecting the two parts,

Figure 2.20a. The less effective solution had an oblique splicing on the upper part and oak wood spikes as the connection between parts, Figure 2.20b.

In turn, Landa (1997; 1999) concluded that the most effective solutions are the ones in which the wood parts are glued together. Of the three studied, the most effective solution was the one with a splicing along the vertical oblique plane (Figure 2.20c) and the less effective was the one with box-shaped splicing and vertical spikes connection (Figure 2.20d).

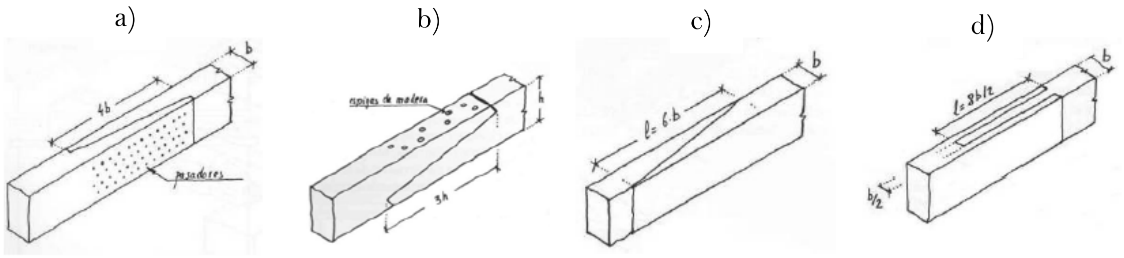


Figure 2.20 – a) Splicing with oblique vertical cut with metallic bolt (Mettem et al., 1993); b) Splicing with oblique cut from the superior face and wood spikes (Mettem et al., 1993); c) Glued splicing with oblique vertical cut (Landa, 1997; 1999); d) Glued box splicing with spikes (Landa, 1997; 1999)

These types of rehabilitation solutions, using notches and holes, have the disadvantage of requiring skilled workers for its accomplishment and also, the implementation of this type of solution on beams of circular cross section becomes a very difficult task.

2.3. Wood modification

The last decade's developments in the area of wood modification have accelerated considerably. This acceleration is due to a number of circumstances like the environmental awareness, the increasing demand for high and constant quality and the increasing prices and availability of tropical hardwood species. This has led to the up-scaling and the market introduction of a number of wood modification techniques.
Modification can be done without added chemicals by heat treatment only, or with the aid of added chemicals (Homan & Jorissen, 2004).
Most fast grown wood species tend to deteriorate rapidly under biological and physical influences. In particular the sapwood of most species has a low durability. The most important biological decay is caused by fungi. Many wood species from temperate and boreal forests have insufficient durability for the intended applications. Until 2000, the problem was solved partly by using biocides (containing for example: creosote, arsenic, zinc, copper, chromium, etc.) and partly by using tropical hardwoods. The recently approved Biocidal Products Regulation (EU) 528/2012 limits the use of biocides. Since both the traditional wood preservation and the use of tropical species are under political and consumer pressure, the wood industries are seeking alternatives. The use of home grown species with enhanced qualities could be the solution to this problem.

In summary the broad range of possible wood modification treatments can be divided into three categories related to the mode of action of the chemicals (Homan & Jorissen, 2004): Lumen filling with a substance; Bulking to fill the cavities in the cell wall; Modifying the chemical structures of cell wall components (lignin, cellulose and hemi-cellulose).

Wood modification can change important properties of the wood including biological durability, dimensional stability, hardness and UV-stability. Strength and stiffness properties are, however, mostly reduced due to the modification treatments.

The modification of wood with use of 1,3-dymethylol-4,5-dihydroxyethyleneurea (DMDHEU) has attracted increasing attention over the past years (Hill, 2006).
Modification of wood with DMDHEU improves the dimensional stability (Yun et al., 2013) and induces high resistance against fungi and insects by depositing the chemical into the cell wall (i.e. a bulking effect) and by reacting with the functional groups of wood polymers, e.g. mostly hydroxyl groups of cellulose, hemicelluloses and lignin (Verma et al., 2009; 2010).

The modification also exhibits limited water uptake, resistance to biological attack (Dieste et al., 2009), good performance to resist weathering (Pfeffer et al., 2012) and reduction of the UV sunlight absorption by the lignin (Norrstrom, 1969).

Although many advantages result from DMDHEU modification, some degree of strength loss is induced during the modification. A reduction in the bending strength of DMDHEU-treated wood has been previously reported (Ashaari et al., 1990).

3. EXPERIMENT

3.1. Materials

3.1.1. Wood and modified wood

The natural wood used in this experiment was extracted from pure sapwood part of the stem of the Portuguese Pine tree (*Pinus Pinaster* Ait.). The mechanical properties of wood present a large variability due to its heterogeneous structure, presence of defects and moisture contents. In Table 3.1, average values of physical and mechanical properties for the Portuguese Pine tree are shown.

Table 3.1 – Average values of physical and mechanical properties of *Pinus Pinaster* Ait (LNEC, 1997)

Designation	Flexural strength (MPa)	Modulus of Elasticity (GPa)	Density (kg/m³)	Grade (according to EN 338:1995)
Pinus Pinaster Ait	18	12	580	C18

The information regarding the modification processes (DMDHEU resin and amid wax) can be accessed in Lopes (2013). The wood properties used in this work were also taken from Lopes (2013) and are listed in Table 3.2.

Table 3.2 – Properties of unmodified and modified wood (Lopes, 2013)

Modification	MOE [GPa]	f_c [MPa]	f_t [MPa]	ε_c [‰]	ε_t [‰]
Unmodified	11	50	80	4,55	7,27
DMDHEU	11	75	60	6,82	5,45
Wax	11	65	85	5,91	7,73

Legend: f_c – Ultimate compressive strength; f_t – Ultimate tensile strength; ε_c – Compressive strain; ε_t – Tensile strain

3.1.2. Epoxy adhesive

In this experiment, Sikadur®-30 was used as an adhesive component. It is a thixotropic adhesive mortar based on a 2-component solvent free epoxy resin. Sikadur®-30 is used primarily to bond structural reinforcements to other substrates. It can also be used to bond and fill a wide variety of building and construction materials. Sikadur®-30 is supplied in factory proportioned units comprising the correct quantities of Part A (Resin) and Part B (Hardener). The properties of the adhesive can be seen in Table 3.3

Table 3.3 – Technical data of Sikadur®-30 (Sika, 2011)

Properties	Information
Component A	White paste
Component B	Black paste
Component A+B	Light grey when mixed
Mix ratio	A : B = 3:1 (parts by weight & volume)
Density	A + B (at +23°C): 1,65 kg/dm³ ± 0,1 kg/dm³
Shrinkage*	0,04%
Coefficient of thermal expansion	$2{,}5\text{x}10^{-5}$ /°C (-10°C to +40°C)
Static Elastic Modulus**	11200 N/mm² (at +23°C)
Adhesive strength*	> 4 N/mm² (over concrete)
Pot Life*	Approx. 90 min. (at +20°C)
Open time *	Approx. 110 min. (at +20°C)

* According to FIP – Fédération Internationale de Précontrainte

** According to ISO 527

3.1.3. Stainless steel

The stainless steel plates were acquired from *Dobra – Corte e Quinagem de Chapa, Lda* company. The properties of the stainless steel used in the analysis were taken from EN1993-1-4 and are shown in Table 3.4.

Table 3.4 – Stainless steel properties (EN 1993-1-4, 2006)

Designation	Tensile strength (MPa)	MOE (GPa)	Density (Kg/m³)
Stainless steel	640	200	7700

3.1.4. Glass fibre reinforced polymer (GFRP)

Glass fibre reinforced polymer was provided by *Alto Perfis Pultrudidos, Lda* company. Table 3.5 shows the properties of the glass fibre sheet that consisted of unidirectional fibres aligned along the longitudinal direction.

Table 3.5 – Glass fibre reinforced polymer properties (Alto, 2013)

Designation	Tensile strength (MPa)	MOE (GPa)	Density (Kg/m³)
GFRP	1825	73	1800

3.2. Material cross section features

The wood beams used for the experiment were modeled in a 1:15 scale with dimensions of 20x10x200mm RTL. Figure 3.1 demonstrates the cross section dimensions used in modelling.

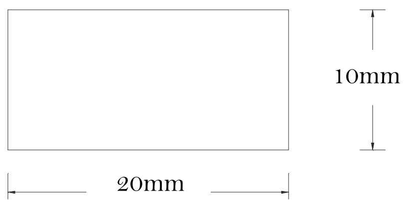


Figure 3.1 – Cross section dimensions of an unreinforced wood beam

The steel reinforcement was designed to have a 6,0x0,5mm² cross section at mid-span in order to achieve a 1,5% reinforcement ratio. The original plates had dimensions of 20x0,5mm², and were submitted to a disk sawing machine to narrow down the width at mid-span. An average cross section of 6,2x0,5mm² was the outcome of the manual process and thus these dimensions were used for the analytical model presented in chapter 4.

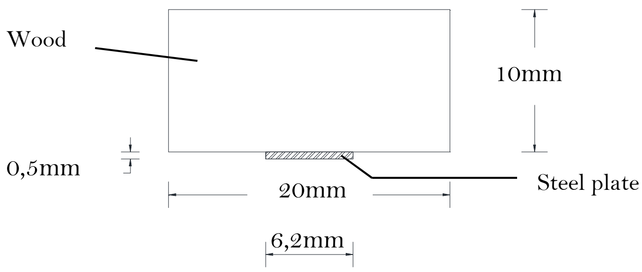


Figure 3.2 – Cross section dimensions of a steel plate reinforced wood beam

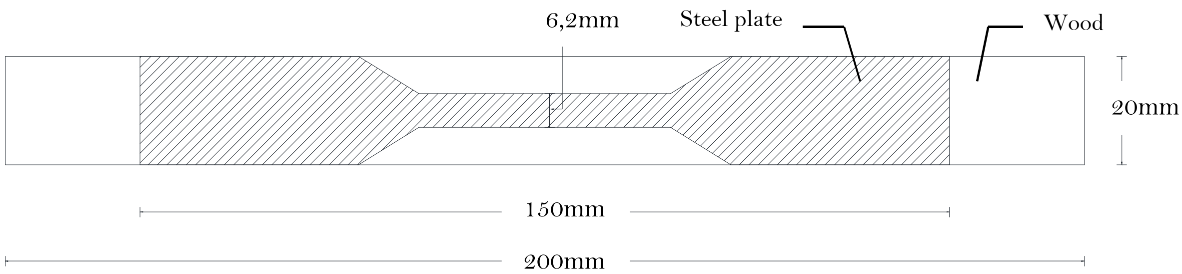


Figure 3.3 – Bottom view of a steel plate reinforced wood beam

The GFRP sheet used for the reinforcement was cut into 20x0,4mm² cross sections. The original idea was to use the same reinforcement percentage ratio at mid-span and design as the stainless steel plates but, to assure that no fibres were damaged in the process, a 4% reinforcement ratio was used instead.

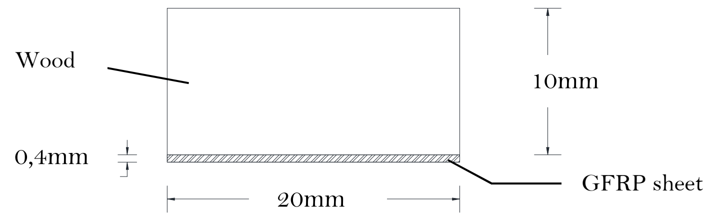


Figure 3.4 – Cross section dimensions of a GFRP reinforced wood beam

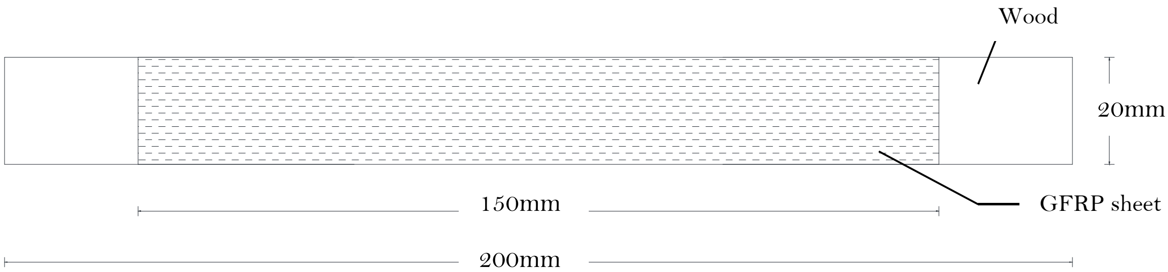


Figure 3.5 – Bottom view of a GFRP reinforced wood beam

The following table presents some cross section properties of unreinforced and reinforced wood beams.

Table 3.6 – Cross section properties

		Control	Steel	GFRP
Reinforcement thickness	[mm]	-	0,5	0,4
Cross section % ratio¹	[%]	-	1,5	4
Gravity centre²	[mm]	5,0	6,15	6,09
Transformed area³	[mm^2]	200	256,36	253,09
Moment of Inertia	[mm^4]	1666,67	2879,81	2801,81
ΔI⁴		1	1,73	1,68

Legend: 1 - Reinforcement percentage ratio of the cross section; 2 - Gravity centre measured from the top; 3 - Transformed cross section in wood; 4 - Inertia variation

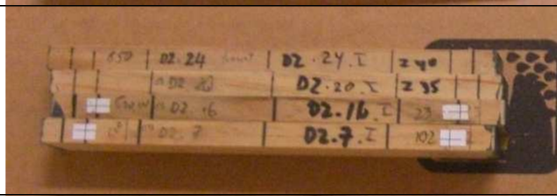
Although different percentages of reinforcement were used, the transformed area and inertia in both cases was identical.

3.3. Test configurations

In order to have some statistically reliable data, a total of 25 wood beams were tested until failure under monotonic loading in a three point bending configuration with load applied at mid span. The unreinforced beams results were provided by a previous study conducted by Lopes (2013) with the same wood origin and were used for this study as a control set of data. Eight unmodified wood beams were tested, 4 of them reinforced with stainless steel plates and the other 4 with GFRP sheets. The same procedure was used on 8 DMDHEU modified wood beams (4 reinforced with stainless steel and 4 reinforced with GFRP sheets). As for the wax modified wood beams, there were a total of 9 beams tested in which 4 were reinforced with stainless steel plates and 5 with GFRP sheets.

The designation of each specimen is listed in the table below:

Table 3.7 – Assignment of the wood beam specimens

Modification	Reinforcement	Designation	
Unmodified	Stainless Steel	0. 4I 0. 7I 0. 10I 0. 17I	
	Glass Fibre	0. 20G 0. 21G 0. 11G 0. 12G	
DMDHEU	Stainless Steel	D2. 24I D2. 20I D2. 16I D2. 7I	
	Glass Fibre	D2. 22G D2. 21G D2. 9G D2. 11G	
Wax	Stainless Steel	WA. 17I WA. 2I WA. 3I WA. 4I	
	Glass Fibre	WA. 7G WA. 5G WA. 13G WA. 10G WA. 24G	

3.4. Beam fabrication

The unreinforced wood beams were fabricated in Germany under certain specifications and further information can be accessed in Lopes (2013).

The reinforcement of the beams was performed at *Instituto Superior de Engenharia do Porto (ISEP)* at *Laboratório de Física das Construções* of the *Departamento de Engenharia Civil* and the procedure is explained in the following sections.

3.4.1. Surface preparation

No surface preparation was conducted, although studies have concluded that it is beneficial in terms of adhesion, to proceed to a surface treatment by using a steel brush over the wood surface (Dias et al., 2006). The same study also concludes that, contrary to what happens in concrete, there are no advantages in applying a primer coating to improve the wood-reinforcement bond.

3.4.2. Adhesive preparation

The two components adhesive was supplied by Sika, the details of which can be seen in Table 3.3. The components A and B were mixed in the ratio prescribed by the manufacturer (3:1). Both were mixed for approximately 5 minutes until a more consistent light grey coloured substance was visible. The component A is basically an epoxy resin, and component B is a hardener, which on mixing will start reactions that are responsible for hardening. Due to the material's pot life, care had been taken to assure that the gluing process was done within the initial setting time of the adhesives.

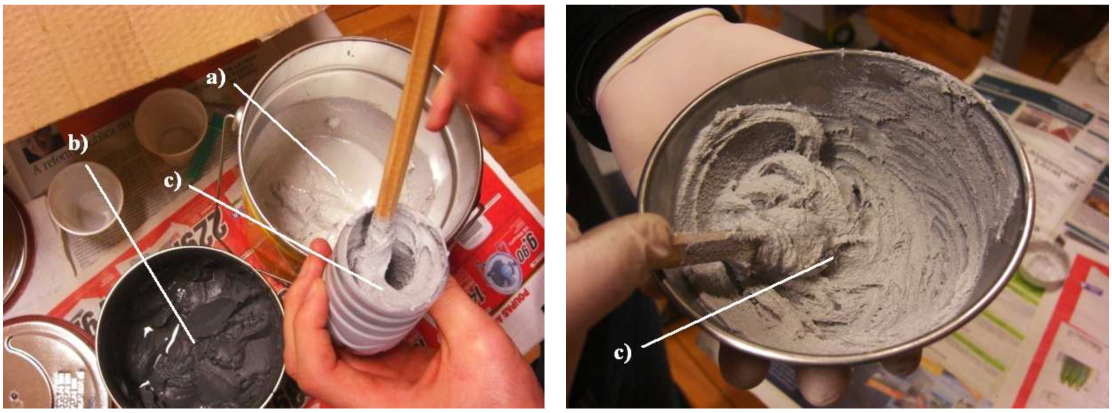


Figure 3.6 – Preparation of the adhesive; a) Component A; b) Component B; c) Final mixture

3.4.3. Adhesive application

The adhesive was applied on the surface of the reinforcement and an effort was made to ensure that it covered the entire area homogeneously.
The Sikadur®-30 was stiff and not very viscous. It should be noted that the viscosity of the adhesive plays a very important role in the workability which in-turn affects the overall quality of the process.

3.4.4. Curing

The reinforced beams were left for curing for 7 days. Weights were placed on top of the reinforced beams in order to ensure the reinforcements were held in place. Figure 3.7 shows some specimens with the reinforcements in place and finished.

Figure 3.7 – Wood beams reinforced with stainless steel, 20x10x200mm RTL

3.5. Mechanical testing

3.5.1. Dynamic Modulus of Elasticity

The 25 beams were tested according to the ASTM E1876-97 (1998) in order to obtain the dynamic Young's modulus, before and after reinforcement. The beams were weighted with a scale to receive the bulk density of each beam. The accuracy of the scale was 0.01g. The beams' mid-span cross section was also measured with an extensometer.

The beams were simply supported at the ends with flexible foam rubber supports. These supports were used to simulate the free-free support condition. Resonance frequencies were determined from induced vibrations in the fibre direction (longitudinal) generated by impact at mid-span of the beam with a rubber hammer. The sound pressure was registered in the support end of the beam by a microphone connected to the GrindoSonic MK5 machine (Figure 3.8). From the sound pressure the frequency spectrum was established with Fast Fourier Transforms (FFT). The dynamic longitudinal elastic modulus was calculated from the standard solution of the wave equation for longitudinal vibrations of a slender rod with free-free support conditions, using equation (1) (Ohlsson & Perstorper, 1991);

$$MOE_{dyn}^{long} = 4 \times \pi^2 \times l^4 \times f^2 \times \rho \times \frac{b \times h}{4{,}72^4 \times I} \times \frac{1 + I \times \frac{49{,}48}{l^2 \times b \times h}}{1000000} \qquad (1)$$

Where, MOE_{dyn}^{long} - Dynamic longitudinal Modulus of Elasticity [MPa]; l – length of beam [mm]; f – frequency [KHz] ; ρ – density [Kg/m^3]; b – cross section width [mm]; h - cross section height [mm]; I – moment of inertia [mm^4].

Figure 3.8 – GrindoSonic MK5 Industrial (http://www.grindosonic.com/)

The GrindoSonic MK5 Industrial equipment was available at the *Laboratório de Física das Construções* of the *Departamento de Engenharia Civil* of *ISEP*.

3.5.2. Static MOE / Strength

The bending strength and static Modulus of Elasticity (MOE_{st}) testing was done using the SHIMADZU AG-X100KN machine at *Instituto Superior de Engenharia do Porto*, *Departamento de Engenharia Mecânica*, *Laboratório de Ensaio de Materiais*. Beams were tested in three point bending configuration (according to DIN 52186), as shown in Figure 3.9 and Figure 3.10.
The information recorded during tests comprised of beam deflections at mid-span (80mm from the support) as well as the corresponding loads.

Three point bending configuration corresponds to a load-support arrangement where a vertical load is applied to a simply supported horizontal beam such that a constant bending moment is obtained. An upper loading cell thrusts the beam down against the static roller supports. This is basically a deflection based test and it allows strain increments even after linear elastic range and further in the cracked stage too. This helps us to follow the post linear behaviour as well as cracked behaviour.

Flawless specimens had 20x10x200mm RTL. The clear distance between the supports was fixed to 160mm and the load was applied at mid-span (80mm from support).
In the test procedure, the loading was done at a speed of 6 N.s^{-1}. The loadings were keenly followed in order to identify key events which led to failure. The data was collected each quarter second and stored in Excel format.

The following figure shows the arrangement of the test where the reinforced wood beam was placed with its radial face over the static roller supports.

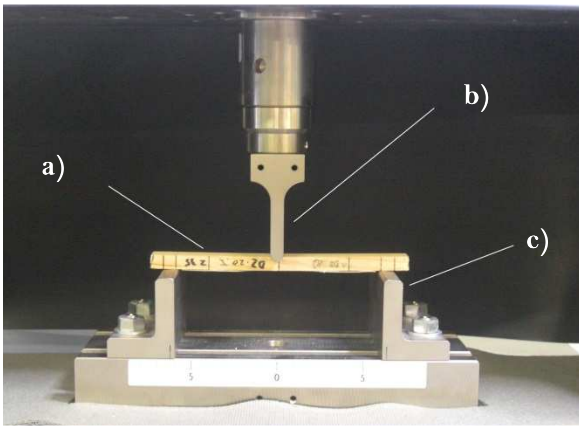


Figure 3.9 – Arrangement of the three-point bending test at a SHIMADZU AG-X100KN machine where: a) wood beam; b) load cell; c) static roller supports

Figure 3.10 represents the structural arrangement of the test. The wood growth rings were set to be parallel and perpendicular to the faces according to the following figure.

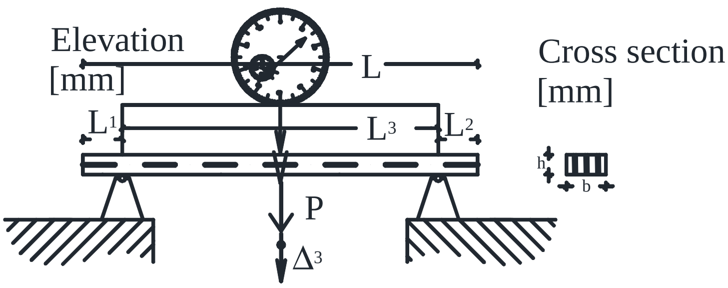


Figure 3.10 – Structural arrangement of a three-point bending test (Lopes, 2013)

With $L_1 = L_2 = 0{,}10xL$; $L_3 = 0{,}80xL$

The following equation is used to calculate the deflection of a beam loaded in 3 point bending configuration.

$$\Delta_3 = \frac{P \times L_3^3}{48 \times E_w \times I'} \qquad (2)$$

With, Δ_3 – deflection at mid span; P – load; L_3 – span length; E_w – Modulus of Elasticity of wood; I' – Transformed moment of inertia (according to section 4.3, equation (11)).

4. ANALYTICAL MODEL

An analytical model was used to predict the flexural behaviour of the specimens. Several studies like Blaß & Romani (2000), Fiorelli & Dias (2002), Jacob & Barragan (2007) and Balseiro (2007), where the same analytical model was used, confirm the viability of the model. The model assumes that the wood specimen fails when the stresses in the outermost fibre in the tensile zone reach the tensile limit after some plasticization in the compression side. It is assumed that the tensile failure of the wood occurs while the cross section is in a linear-elastic-ideal plastic state.

A more detailed explanation of the analytical model used in this work can be found in Triantafillou & Deskovic (1992), Blaß & Romani (2000), Fiorelli & Dias (2002; 2004) and Balseiro (2007).

4.1. Tensile failure in wood

This failure mode is brittle as wood does not have plastic behaviour under tensile loads. This can even introduce cracks along the grain direction which will result in catastrophic destruction of the cross section (Jacob & Barragan, 2007).

The wood tensile limit state is considered to be attained when the maximum tensile stress is equal to its tensile strength. The beam is considered to be failed when the outermost fibre in tensile reaches the tensile limit.

Two failure modes can be identified in the tensile zone based on the degree of plasticization at the compression side:

- *Failure of the wood on tensile zone while the cross section is in a linear elastic state*: This occurs when the stress in the outermost fibre in the tensile zone reaches its tensile limit and the compression zone is still in the linear elastic range. This usually happens with unreinforced beams whose tensile strength is lesser than the compressive strength. The failure is brittle.
- *Failure of the wood while the cross section is in a linear-elastic-ideal plastic state*: This occurs when the stresses in the outermost fibre in the tensile zone reaches the tensile limit only after some plasticization in the compression side. But still the compressive zone has not reached the ultimate compressive strain. This is the most common type of failure and is typical to beams which are lightly reinforced in only tensile zone or in both tensile and compressive zones. Even though there is some ductility in the global behaviour, the fibres locally fail in tensile which makes the failure mode brittle.

Figure 4.1 shows the failure modes and the corresponding stress/strain state, where: ε_c – compressive strain; ε_{c_el} – elastic compressive strain; ε_t – tensile strain; ε_{t_el} – elastic tensile strain; σ_c – compressive stress; f_c – ultimate compressive stress; σ_t – tensile stress; f_t – ultimate tensile stress; ε_{c_pl} – plastic compressive strain; NA – neutral axis; Z_p – depth of compressive plasticization.

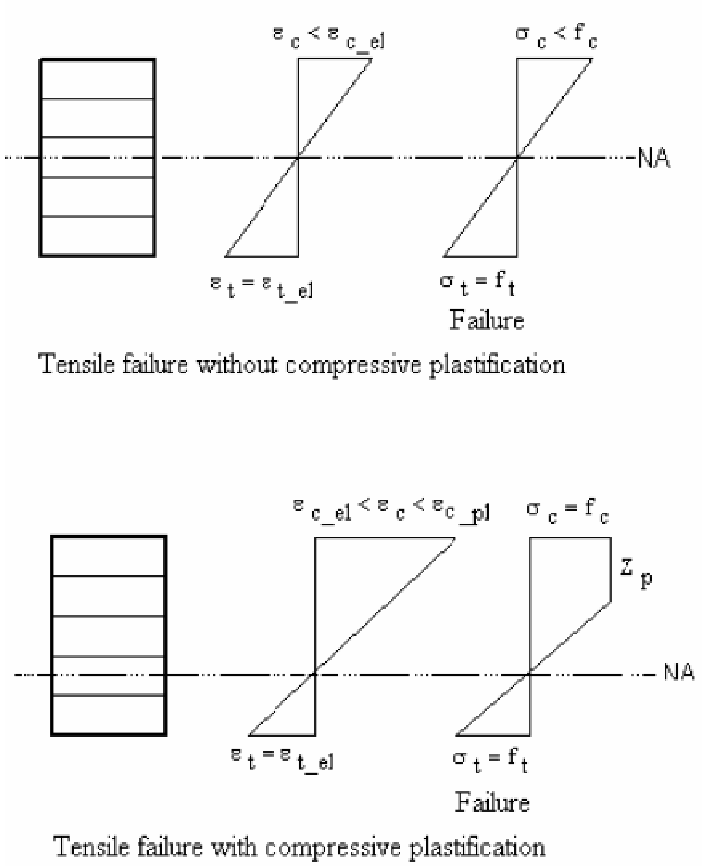


Figure 4.1 – Tensile failure modes – stress/strain states (Jacob & Barragan, 2007)

4.2. Assumptions and simplifications

The analytical model was designed assuming that tensile failure of the wood occurred while the cross section is in a linear-elastic-ideal plastic state.

The stress strain relationship is simplified to be linear-elastic in tensile side, and linear elastic-perfect-plastic in compression, ignoring the nonlinearities.

It is assumed that failure occurs when the outermost fibre reaches the maximum allowable stress or strain state. The maximum strain was obtained via Hooke's law and the values are expressed in Table 3.2 of section 3.1.

The decrease in cross sectional resistance due to reduction in cross sectional area resulting from yielding of outermost fibres is not taken into account.

The MOE of wood is assumed to be the same in both compression and tensile.

The adhesive contribution is not taken into account.
The presence of knots and other defects in wood is not considered in the model.

4.3. Formulation

For the calculation of the bending moment of an unreinforced beam, the following equations were used:

Wood area:

$$A_w = b_w \times h_w \tag{3}$$

With: A_w – wood area; b_w – wood width; h_w – wood height.

Wood inertia:

$$I_w = \frac{b_w \times {h_w}^3}{12} \tag{4}$$

With: I_w – wood inertia; b_w – wood width; h_w – wood height.

Ultimate moment of resistance:

$$M_u = \frac{\sigma_{t,w} \times I_w}{h_w/2} \tag{5}$$

With: M_u – ultimate moment of resistance; $\sigma_{t,w}$ – wood tensile stress; I_w – wood inertia; h_w – wood height.

Maximum load:

$$F_{max} = \frac{4 \times M_u}{l_s} \tag{6}$$

With: F_{max} – maximum load; M_u – ultimate moment of resistance; l_s – span length.

Deflection at mid span:

$$\delta = \frac{F_{max} \times l_s^3}{48 \times E_w \times I} \tag{7}$$

With: δ – deflection; F_{max} – maximum load; l_s – span length; E_w – Modulus of Elasticity of wood; I – wood inertia.

For the calculation of the bending moment of a reinforced beam, the literature recommends the use of the method of homogenization of the cross section, also known as the transformed cross section method. In general, this method considers the composite beam is transformed into one of the composite beam materials with equivalent stiffness (Silva et al., 2004).

According to Batista (1996), it is necessary that the neutral axis, in both transformed and actual cross section, stays in the same position and, in addition, Parker (1978) and John & Chilver (1961) reported that the ability to resist the bending moment must be the same.

In this study, the equivalent cross section area is considered to be constituted only by wood. An example of a transformed cross section scheme of a wood beam reinforced with a steel plate can be seen in Figure 4.2

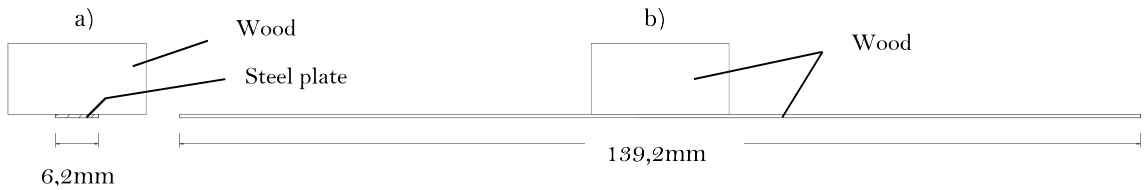


Figure 4.2 – a) Cross section of a wood beam reinforced with a steel plate ; b) Transformed cross section of a wood beam reinforced with a steel plate

Transformed cross section area:

$$A' = A_w + m \times A_r \tag{8}$$

With: A' - transformed cross section area; A_w – wood area; m – homogeneity factor; A_r – reinforcement area.

$$m = {}^{E_r}/_{E_w} \tag{9}$$

With: m – homogeneity factor; E_r – Modulus of Elasticity of the reinforcement; E_w – Modulus of Elasticity of wood.

The following figure is a scheme that represents the cross section, where: x – gravity centre; h_w – wood height; h_r – reinforcement height.

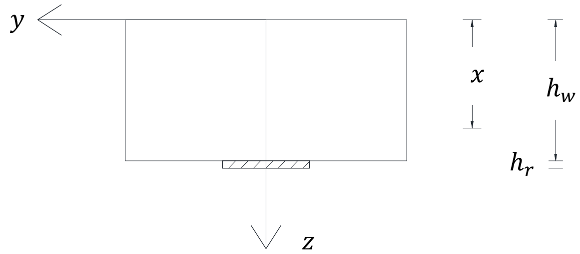


Figure 4.3 – Cross section scheme

Gravity centre of the transformed cross section (from the top):

$$x = \frac{A_w \times {h_w}/{2} + m \times A_r \times \left(h_w + {h_r}/{2}\right)}{A'} \tag{10}$$

With: x – gravity centre; A_w – wood area; h_w – wood height; m – homogeneity factor; A_r – reinforcement area; h_r – reinforcement height; A' - transformed cross section area.

The moment of inertia of the transformed cross section can be obtained with:

$$I' = \frac{b_w \times {h_w}^3}{12} + A_w \times \left(x - \frac{h_w}{2}\right)^2 + m \times \left(\frac{b_r \times {h_r}^3}{12} + A_r \times \left(h_w + \frac{h_r}{2} - x\right)^2\right) \tag{11}$$

With: I' - transformed cross section moment of inertia; b_w – wood width; h_w – wood height; A_w – wood area; x – gravity centre; m – homogeneity factor; A_r – reinforcement area; b_r – reinforcement width; h_r – reinforcement height.

Using the Excel software and considering that the failure mode was accordingly to the linear-elastic-ideal plastic phase model presented in section 2.3.1 and schematized in Figure 4.4, a neutral axis that results in the internal forces equilibrium was found using equations (12), (13) and (14):

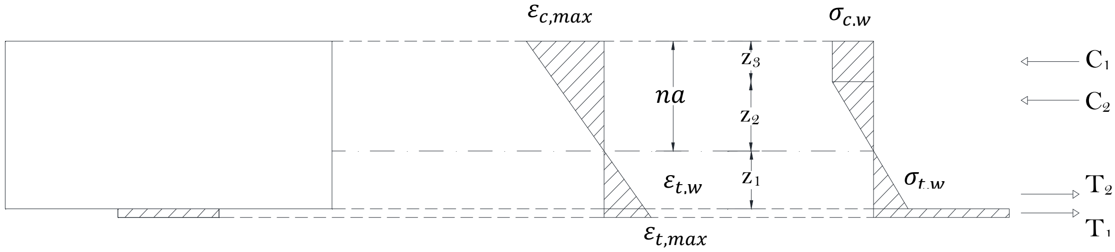


Figure 4.4 – Internal loads at the mid-span of the cross section

$$\sigma_{c,w} \times b_w \times z_3 + \sigma_{c,w} \times \frac{b_w}{2} \times (na - z_3) = \sigma_{t,w} \times \frac{b_w}{2} \times (h_w - na) + A_r \times E_r \times \varepsilon_r \tag{12}$$

$$z_3 = \frac{(\sigma_{c,w} + \sigma_{t,w}) \times na - \sigma_{c,w} \times h_w}{\sigma_{t,w}} \tag{13}$$

$$\varepsilon_r = \frac{\varepsilon_{t,w} \times \left(h_w + {h_r}/{2} - na\right)}{(h_w - na)} \tag{14}$$

Where: $\sigma_{c,w}$ – wood compressive stress; b_w – wood width; z_3 – depth of compressive plasticization; na – neutral axis; $\sigma_{t,w}$ – wood tensile stress; h_w – wood height; A_r – reinforcement area; E_r – reinforcement Modulus of Elasticity; ε_r – reinforcement strain; $\varepsilon_{t,w}$ – wood tensile strain; h_r – reinforcement height.

After finding the neutral axis, new correlations are proposed:

$$na = z_2 + z_3 \tag{15}$$

$$z_1 = h_w - na \tag{16}$$

Where: na – neutral axis; z_1 – depth of the elastic tensile wood section; z_2 – depth of the elastic compressive wood section; z_3 – depth of compressive plasticization; h_w – wood height.

The maximum strain in the outmost superior fibre is:

$$\varepsilon_{c,max} = \varepsilon_{t,w} \times \left(\frac{h_w}{na} - 1\right) \tag{17}$$

The maximum strain in the outmost inferior fibre is:

$$\varepsilon_{t,max} = \left(h_w + \frac{h_r}{2} - na\right) \times \frac{\varepsilon_{t,w}}{h_w - na} \tag{18}$$

Stress in the reinforcement is:

$$\sigma_r = E_r \times \varepsilon_{t,max} \tag{19}$$

The internal forces that balance the cross section were obtained with the following equations:

Compressive forces in wood

$$C_1 = z_3 \times b_w \times \sigma_{c,w} \tag{20}$$

$$C_2 = \frac{z_2}{2} \times b_w \times \sigma_{c,w} \tag{21}$$

$$C_{total} = C_1 + C_2 \tag{22}$$

Tensile force in wood

$$T_2 = \frac{z_1}{2} \times b_w \times \sigma_{t,w} \tag{23}$$

Tensile force in the reinforcement

$$T_1 = \sigma_r \times A_r \tag{24}$$

$$T_{total} = T_1 + T_2 \tag{25}$$

If $C_{total} = T_{total}$, then the neutral axis was well calculated.

The application point of the total compressive and tensile forces can be determined by geometrical considerations as schematized in Figure 4.5.

Figure 4.5 – Application point of the internal forces

$$e_c = \frac{C_1 \times {}^{z_3}/_2 + C_2 \times (z_3 + {}^{z_2}/_3)}{C_{total}} \tag{26}$$

$$e_t = \frac{T_1 \times \left(h_w + {}^{h_r}/_2\right) + T_2 \times \left(z_3 + z_2 + \frac{2}{3} \times z_1\right)}{T_{total}} \tag{27}$$

The rupture moment can be calculated by:

$$M_u = C_{total} \times (e_t - e_c) = T_{total} \times (e_t - e_c) \tag{28}$$

Maximum load and deformation of a beam at 3 point bending:

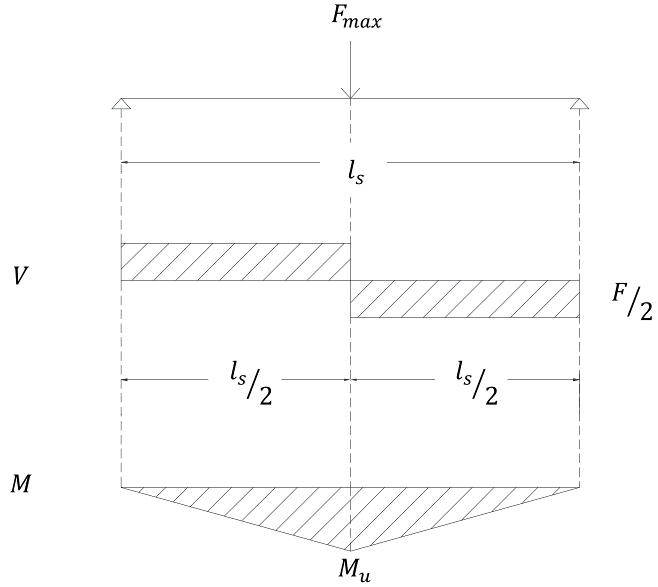


Figure 4.6 – Shear and Moment diagrams

Where F_{max} – maximum load; M_u – ultimate moment of resistance; l_s – span length; δ – deflection; E_w – Wood modulus of elasticity; I' – Transformed cross section moment of inertia.

$$F_{max} = \frac{4 \times M_u}{l_s} \tag{29}$$

$$\delta = \frac{F_{max} \times l_s^3}{48 \times E_w \times I'} \tag{30}$$

5. RESULTS

This section deals with the results from the Dynamic Modulus of Elasticity test and the 3-point bending test, as well as the results obtained in the analytical model. The results are plotted as load-deformation graphs, tables and bar charts.

5.1. Analytical results

The following table shows the transformed cross section properties and factors according to the type of reinforcement used.

Table 5.1 – Transformed cross section properties

		Un-reinforced	Steel reinforced	GFRP reinforced
Homogeneity factor		-	18,18	6,64
Area in wood	[mm^2]	200,00	256,36	253,09
Gravity centre	[mm]	5,00	6,15	6,09
Moment of Inertia	[mm^4]	1666,67	2879,81	2801,81
ΔI		1	1,73	1,68

Although the homogeneity factor of the wood beam reinforced with steel is nearly three times higher than the GFRP reinforced, the transformed areas are identical. The moment of inertia increased by 73% with the steel reinforcement and by 68% with the GFRP reinforcement.

The following table shows the theoretical results obtained with the equations from section 4.3.

Table 5.2 – Linear-elastic-ideal plastic model results

Type of wood		Unmodified (O)		DMDHEU (D2)		Wax (WA)	
Reinforcement		Steel	GFRP	Steel	GFRP	Steel	GFRP
Neutral axis (na)	[mm]	7,63	7,43	6,25	6,17	7,05	6,90
z_1	[mm]	2,37	2,57	3,75	3,83	2,95	3,10
z_2	[mm]	1,48	1,60	4,69	4,79	2,25	2,37
z_3	[mm]	6,14	5,83	1,56	1,37	4,80	4,54
$\varepsilon_{c,max}$	[‰]	2,26	2,51	3,27	3,39	3,23	3,47
$\varepsilon_{t,max}$	[‰]	8,04	7,84	5,82	5,74	8,38	8,23
C_1	[N]	6141,4	5829,4	2341,4	2057,3	6237,6	5897,0
C_2	[N]	742,0	802,0	3516,3	3595,2	1465,2	1539,0
C_{total}	[N]	6883,4	6631,5	5857,7	5652,5	7702,8	7436,0
T_1	[N]	4983,8	4578,2	3607,2	3351,6	5197,2	4804,2
T_2	[N]	1899,6	2053,2	2250,4	2300,9	2505,6	2631,8
T_{total}	[N]	6883,4	6631,5	5857,7	5652,5	7702,8	7436,0
e_c	[mm]	3,46	3,33	2,19	2,14	3,00	2,90
e_t	[mm]	9,96	9,87	9,67	9,60	9,85	9,76
M_u	[N.m]	44,8	43,4	43,9	42,2	52,8	51,0
F_{max}	[N]	1120	1084	1096	1054	1319	1276
deflection (δ)	[mm]	3,0	3,0	3,0	2,9	3,6	3,5

The depth of the neutral axis was deeper on the unmodified wood beams and lower on the DMDHEU modified wood beams.

The unmodified wood beams presented the highest values for the compressive plasticization depth (z_3) with around 60% of the cross section whereas in the DMDHEU modified it only accounts for around 15% of the cross section.

The results of the wax modified beams fall in between the other two types of beams.

In terms of strains, the wax modified beams seem to produce higher maximum strains.

The model led to no considerable differences between both types of reinforcement in the same type of wood beam.

The DMDHEU modified specimens are subjected to lower values of internal forces than the other specimens. That may be explained by the significant tensile strength reduction ($f_{t,0}$) that occurs from the modification process (Lopes, 2013).

The maximum loads obtained show higher values for wax modified beams (1319N with steel and 1276N with GFRP) and no significant differences between unmodified and DMDHEU modified beams.

The unmodified and DMDHEU modified, reinforced with steel or GFRP, theoretically present the same deflection (around 3mm) but the wax modified beams can reach nearly 20% more (around 3,5mm).

5.2. Control beams

The experiments regarding the unreinforced unmodified and modified wood beams were conducted in Lopes (2013) from which the following results were recorded and used as control results for this study.

Table 5.3 – Average maximum loads of the control beams without reinforcement obtained with specimens with 20.10.200mm RTL (Lopes, 2013)

Modification		Unmodified (O)	DMDHEU (D2)	Wax (WA)
Average maximum load	[N]	890	550	1090

The unmodified wood beams sustained an average maximum load of 890N while the DMDHEU modified wood beams only averaged 62% of that value (550N). The wax modified wood beams sustained 22% more load than the unmodified wood (1090N).

5.3. Steel reinforced beams

The following figure shows the load-deformation graph that resulted from the 3 point bending test done to the 4 unmodified wood beams reinforced with steel plates.

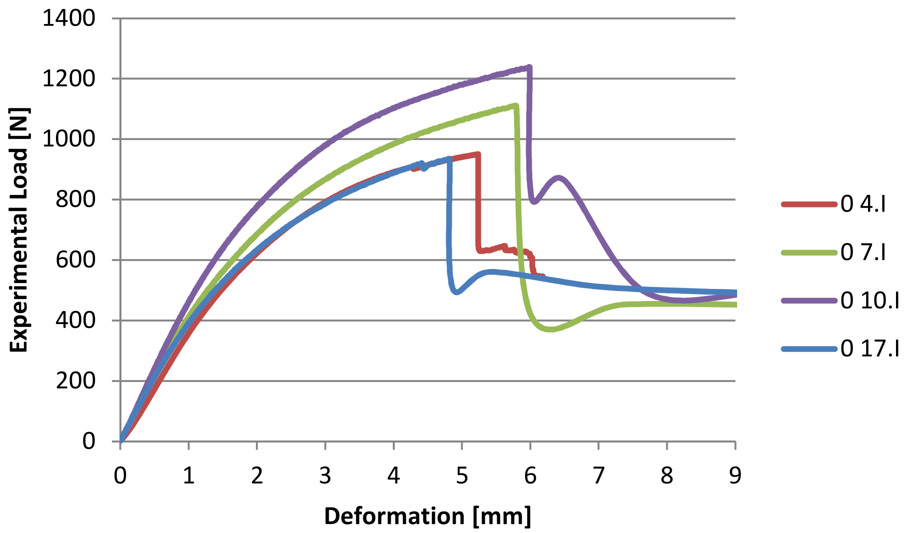


Figure 5.1 – Load-Deformation graph of unmodified wood beams (O) reinforced with steel plates

The following table shows the maximum load and deflection results of each of the unmodified wood beams reinforced with steel.

Table 5.4 – Maximum load and deflection results for unmodified wood beams reinforced with steel

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
Unmodified (O)	Steel	4.I	950	5,2
		7.I	1110	5,8
		10.I	1238	6,0
		17.I	934	4,8
		Average	**1058**	**5,5**
		STDEV	144	0,5
		CV (%)	14	9,8

The unmodified wood beams (O) reinforced with stainless steel plates presented a ductile behaviour (Figure 5.1) and endured an average maximum load of 1058N and deformations of 4,8mm to 6mm.

The discussion of these results is made in section 5.6.3 in Table 5.23 and Table 5.25.

The following figure shows the load-deformation results of the 3 point bending test done to the 4 DMDHEU modified wood beams reinforced with steel plates.

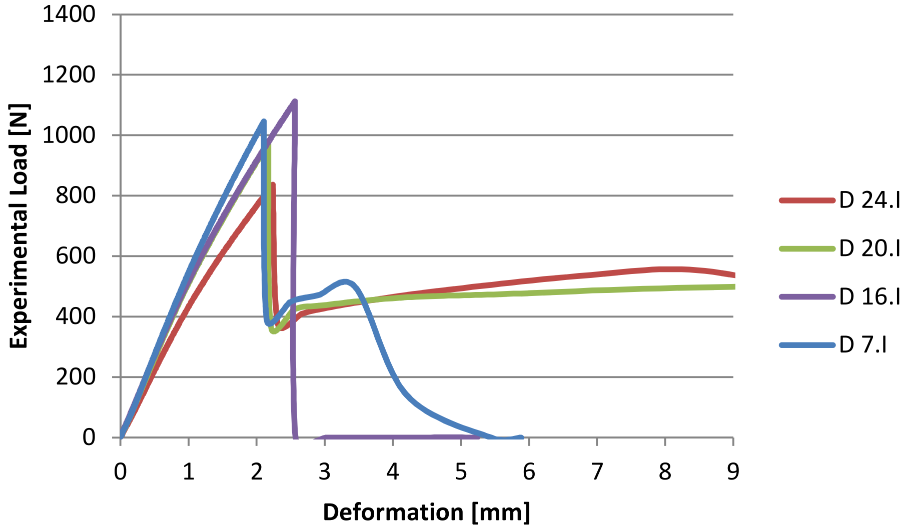


Figure 5.2 – Load-Deformation graph of DMDHEU modified wood beams (D2) reinforced with steel plates

The following table shows the maximum load and deflection results of each of the DMDHEU modified wood beams reinforced with steel.

Table 5.5 – Maximum load and deflection results for DMDHEU modified wood beams reinforced with steel

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
DMDHEU (D2)	Steel	24.I	836	2,2
		20.I	973	2,2
		16.I	1112	2,6
		7.I	1046	2,1
		Average	**992**	**2,3**
		STDEV	118	0,2
		CV (%)	12	9,0

The DMDHEU modified wood beams (D2) presented a fragile behaviour with a linear response until failure, as seen in Figure 5.2. The beams averaged 2mm deformations with an average maximum load of 992N.

The discussion of these results is made in section 5.6.3 in Table 5.23 and Table 5.25.

The following figure shows the load-deformation graph obtained of the 3 point bending test done to the 4 wax modified wood beams reinforced with steel plates.

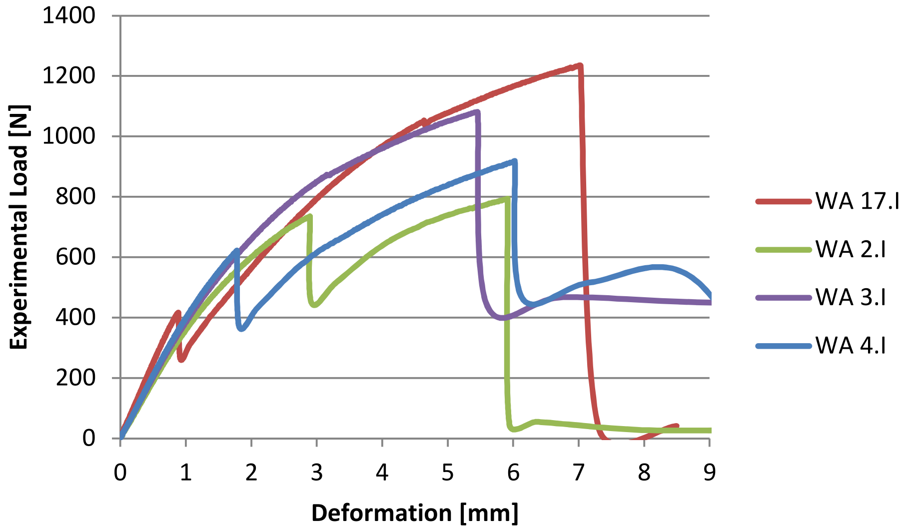


Figure 5.3 – Load-Deformation graph of wax modified wood beams (WA) reinforced with steel plates

The following table shows the maximum load and deflection results of each of the wax modified wood beams reinforced with steel.

Table 5.6 – Maximum load and deflection results for wax modified wood beams reinforced with steel

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
Wax (WA)	Steel	17.I	1235	7,0
		2.I	793	5,9
		3.I	1080	5,5
		4.I	918	6,0
		Average	**1006**	**6,1**
		STDEV	192	0,7
		CV (%)	19	10,8

The wax modified wood beams (WA) presented a ductile behaviour, supporting an average maximum load of 1006N and deformations of 5,5mm to 7mm. It is also visible on the load-deformation graph (Figure 5.3) that 3 of the 4 reinforced beams suffered a load decrease between 1mm and 3mm deformations that corresponded to the de-bonding of the steel plate. The discussion of these results is made in section 5.6.3 in Table 5.23 and Table 5.25.

5.4. Glass fibre reinforced polymer reinforced beams

The following figure shows the load-deformation graph of the 3 point bending test done to the 4 unmodified wood beams reinforced with GFRP sheets.

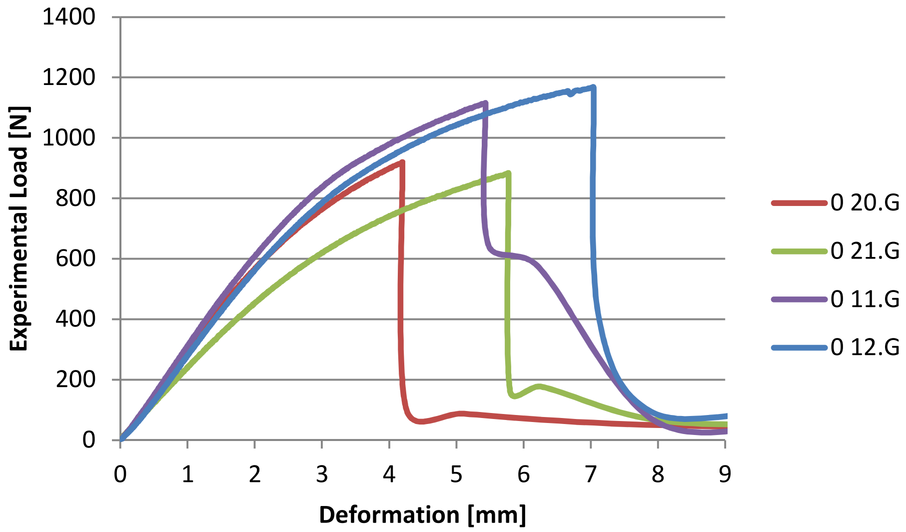


Figure 5.4 – Load-Deformation graph of unmodified wood beams (O) reinforced with GFRP

The following table shows the maximum load and deflection results of each of the unmodified wood beams reinforced with GFRP.

Table 5.7 – Maximum load and deflection results for unmodified wood beams reinforced with GFRP

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
Unmodified (O)	GFRP	20.G	920	4,2
		21.G	883	5,8
		11.G	1115	5,4
		12.G	1167	7,0
		Average	**1021**	**5,6**
		STDEV	141	1,2
		CV (%)	14	20,8

The unmodified wood beams (O) reinforced with GFRP presented a ductile behaviour (Figure 5.4) and endured an average maximum load of 1021N with deformations around 4mm up to 7mm.

The discussion of these results is made in section 5.6.3 in Table 5.24 and Table 5.26.

The following figure shows the load-deformation results of the 3 point bending test done to the 4 DMDHEU modified wood beams reinforced with GFRP sheets.

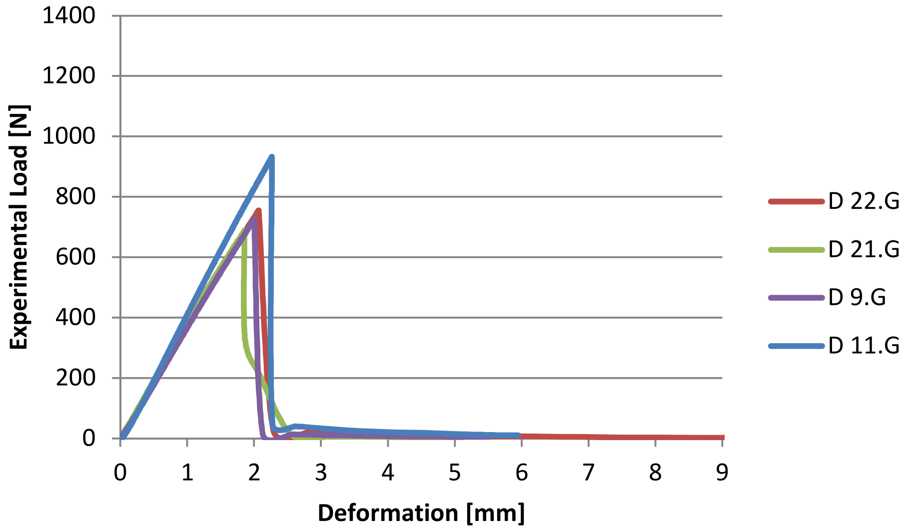


Figure 5.5 – Load-Deformation graph of DMDHEU modified wood beams (D2) reinforced with GFRP

The following table shows the maximum load and deflection results of each of the DMDHEU modified wood beams reinforced with GFRP.

Table 5.8 – Maximum load and deflection results for DMDHEU modified wood beams reinforced with GFRP

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
DMDHEU (D2)	GFRP	22.G	755	2,1
		21.G	689	1,9
		9.G	726	2,0
		11.G	933	2,3
		Average	**776**	**2,1**
		STDEV	108	0,2
		CV (%)	14	8,3

The DMDHEU modified wood beams (D2) had a fragile behaviour (Figure 5.5) and reached failure at an average maximum load of 775N and 2mm deformations.

The discussion of these results is made in section 5.6.3 in Table 5.24 and Table 5.26.

The following figure shows the load-deformation graph of the 3 point bending test done to the 5 wax modified wood beams reinforced with GFRP sheets.

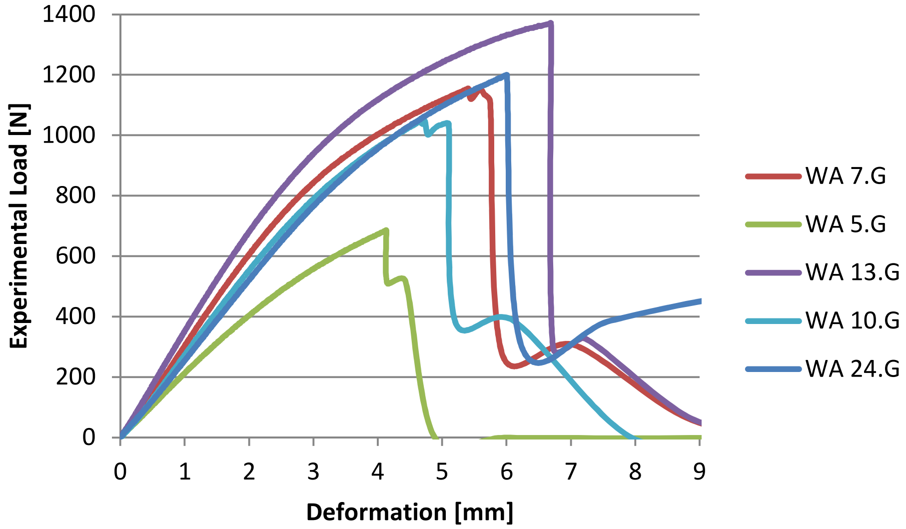


Figure 5.6 – Load-Deformation graph of wax modified wood beams (WA) reinforced with GFRP

The following table shows the maximum load and deflection results of each of the wax modified wood beams reinforced with GFRP.

Table 5.9 – Maximum load and deflection results for wax modified wood beams reinforced with GFRP

Modification	Reinforcement	Specimen	Maximum Load [N]	Deflection [mm]
Wax (WA)	GFRP	7.G	1155	5,6
		5.G	686	4,1
		23.G	1371	6,7
		10.G	1200	5,0
		24.G	1091	6,0
		Average	**1100**	**5,8**
		STDEV	110	0,7
		CV (%)	11	12,0

The wax modified wood beams (WA) presented a ductile behaviour (Figure 5.6), supporting an average maximum load of 1100N and deformations of 5mm to 6,7mm. The WA 5.G specimen prematurely reached failure at a load of 685N and was not considered in terms of average calculations.

The discussion of these results is made in section 5.6.3 in Table 5.24 and Table 5.26.

5.5. Stiffness

The beams' stiffness was obtained dynamically and statically according to the procedures described in section 3.5.1 and 3.5.2 respectively. To obtain the final results, additional calculations were made and are specified in this section.

The static MOE was obtained on the experimental bending test of the reinforced beams and the results are shown in Table 5.10 for unmodified wood beams, Table 5.11 for DMDHEU modified wood beams and Table 5.12 for wax modified wood beams.

The calculations were made according to equation (31), where: l (span) = 160mm; b – width; h – height; ΔL – difference between two load values taken from the linear elastic part of the load-deformation graphs; Δd – difference between two deformation values taken from the linear elastic part of the load-deformation graphs that correspond to the two load values taken for ΔL.

$$MOE = \frac{l^3}{4 \times b \times h^3} \times \frac{\Delta L}{\Delta d} \qquad (31)$$

Table 5.10 – Static MOE of unmodified wood beams (O) reinforced with steel and GFRP

Reinforcement	Unmodified (O)	b [mm]	h [mm]	ΔL/Δd [N/mm]	MOEst [Mpa]	Average MOEst [Mpa]
Steel	4.I	20,25	11,42	385,65	13094,02	**14459,44**
	7.I	20,25	11,55	420,71	13807,43	
	10.I	20,22	11,45	494,18	16672,04	
	17.I	20,19	11,27	402,59	14264,28	
					STDEV	1551,69
					CV (%)	10,73
GFRP	20.G	20,13	11,12	303,94	11244,29	**10792,20**
	21.G	20,24	11,10	240,90	8911,68	
	11.G	20,24	11,19	333,59	12045,21	
	12.G	20,19	11,02	289,40	10967,62	
					STDEV	1334,36
					CV (%)	12,36

The average static MOE for unmodified wood beams reinforced with steel was approximately 14,5 GPa whereas with the GFRP reinforcement was 10,8 GPa.

The discussion of these results is made in section 5.6.2.

Table 5.11 – Static MOE of DMDHEU modified wood beams (D2) reinforced with steel and GFRP

Reinforcement	DMDHEU (D2)	b [mm]	h [mm]	ΔL/Δd [N/mm]	MOEst [Mpa]	Average MOEst [Mpa]
Steel	24.I	20,58	11,51	464,81	15167,29	**16566,57**
	20.I	20,61	11,64	561,73	17696,58	
	16.I	20,77	11,80	531,95	15962,17	
	7.I	20,54	11,72	563,17	17440,24	
					STDEV	1206,03
					CV (%)	7,28
GFRP	22.G	20,69	11,59	392,07	12463,76	**12666,39**
	21.G	20,55	11,61	376,15	11976,97	
	9.G	20,68	11,68	390,95	12148,89	
	11.G	20,57	11,55	435,67	14075,94	
					STDEV	961,07
					CV (%)	7,59

The DMDHEU modified wood beams reinforced with steel and with GFRP averaged a static MOE of 16,6 GPa and 12,7 GPa respectively.
The discussion of these results is made in section 5.6.2.

Table 5.12 – Static MOE of wax modified wood beams (WA) reinforced with steel and GFRP

Reinforcement	Wax (WA)	b [mm]	h [mm]	ΔL/Δd [N/mm]	MOEst [Mpa]	Average MOEst [Mpa]
Steel	17.I	20,01	11,51	486,40	16323,76	**13773,34**
	2.I	20,24	11,38	336,74	11559,87	
	3.I	20,22	11,33	381,79	13293,87	
	4.I	20,01	11,58	422,26	13915,86	
					STDEV	1970,98
					CV (%)	14,31
GFRP	7.G	20,15	11,12	326,73	12075,31	**13058,06**
	5.G	20,19	11,19	206,65	7480,07	
	13.G	20,17	11,09	351,34	13077,73	
	10.G	20,20	10,52	277,34	12075,60	
	24.G	20,19	9,88	285,30	15003,62	
					STDEV	1380,41
					CV (%)	10,57

Note: Specimen WA 5.G was not taken into account for the average calculation

The average results obtained for the static MOE of the wax modified wood beams were 13,8 GPa with steel reinforcement and 13,1 GPa with GFRP reinforcement.
The discussion of these results is made in section 5.6.2.

The dynamic MOE (MOE_{dyn}) was calculated according to equation (1) presented in section 3.5.1. For practical reasons the equation is also shown below:

$$MOE_{dyn}^{long} = 4 \times \pi^2 \times l^4 \times f^2 \times \rho \times \frac{b \times h}{4{,}72^4 \times I} \times \frac{1 + I \times \frac{49{,}48}{l^2 \times b \times h}}{1000000} \tag{1}$$

Where, MOE_{dyn}^{long} - Measured dynamic longitudinal Young's modulus [MPa]; l – length of beam [mm]; f – frequency [KHz]; ρ – density [Kg/m^3]; b – cross section width [mm]; h - cross section height [mm]; I – moment of inertia [mm^4].

The results of the dynamic MOE before reinforcement (MOE_{dyn_i}) are shown in Table 5.13 for unmodified wood beams, Table 5.14 for DMDHEU modified wood beams and Table 5.15 for wax modified wood beams.
As for the dynamic MOE after reinforcement (MOE_{dyn_r}), the results are shown in Table 5.16 for the unmodified wood beams, Table 5.17 for the DMDHEU modified and Table 5.18 for the wax modified.

The density (ρ) of the unreinforced wood beams was calculated according to equation (32) where W_o – weight; b – width; h – height; l – length.

$$\rho = \frac{Wo}{b \times h \times l} \tag{32}$$

Table 5.13 – MOE_{dyn} of unmodified wood beams (O) before reinforcement (MOE_{dyn_i})

Reinforcement	Unmodified (O)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m^3]	I [mm^4]	MOE_{dyn_i} [Mpa]
Unreinforced	4.I	20,25	10,31	22,94	1,212	549,39	1849,35	11721,78
	7.I	20,25	10,22	23,62	1,214	570,66	1801,34	12429,50
	10.I	20,22	10,21	23,88	1,374	578,36	1793,40	16167,93
	17.I	20,19	10,15	21,86	1,226	533,36	1759,35	12010,12
							Average	**13082,33**
							STDEV	2077,49
							CV (%)	15,88
Unreinforced	20.G	20,13	10,11	22,43	1,343	551,07	1733,47	15007,18
	21.G	20,24	10,15	22,17	1,096	539,58	1763,71	9710,22
	11.G	20,24	10,14	23,70	1,344	577,39	1758,50	15655,41
	12.G	20,19	10,19	23,81	1,300	578,65	1780,24	14536,98
							Average	**13727,44**
							STDEV	2717,12
							CV (%)	19,79

According to the calculations, the average dynamic MOE for unmodified wood beams before applying the steel reinforcement was 13,1 GPa and for the unmodified wood beams before the GFRP reinforcement was 13,7 GPa.
The discussion of these results is made in section 5.6.2.

Table 5.14 – MOE_{dyn} of DMDHEU modified wood beams (D2) before reinforcement (MOE_{dyn_i})

Reinforcement	DMDHEU (D2)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m³]	I [mm⁴]	MOE_{dyn_i} [Mpa]
Unreinforced	24.I	20,58	10,59	33,38	1,065	765,80	2036,82	11964,85
	20.I	20,61	10,57	32,67	1,190	749,84	2028,25	14681,70
	16.I	20,77	10,72	35,47	1,163	796,53	2132,26	14486,96
	7.I	20,54	10,69	33,89	1,273	771,73	2090,99	16910,01
							Average	**14510,88**
							STDEV	2022,20
							CV (%)	13,94
Unreinforced	22.G	20,69	10,59	33,30	1,184	759,90	2047,70	14674,19
	21.G	20,55	10,55	32,78	1,180	755,99	2010,89	14609,02
	9.G	20,68	10,71	34,08	1,180	769,36	2117,08	14431,54
	11.G	20,57	10,72	34,83	1,286	789,76	2111,73	17562,82
							Average	**15319,39**
							STDEV	1499,13
							CV (%)	9,79

The average dynamic MOE for DMDHEU modified wood beams before reinforcing with steel was 14,5 GPa and 15,3 GPa for the beams before the GFRP reinforcement.
The discussion of these results is made in section 5.6.2.

Table 5.15 – MOE_{dyn} of wax modified wood beams (WA) before reinforcement (MOE_{dyn_i})

Reinforcement	Wax (WA)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m³]	I [mm⁴]	MOE_{dyn_i} [Mpa]
Unreinforced	17.I	20,01	10,02	44,74	0,975	1115,71	1677,53	16290,05
	2.I	20,24	10,21	43,94	0,785	1063,15	1795,17	9691,12
	3.I	20,22	10,21	43,96	0,857	1064,68	1793,40	11586,97
	4.I	20,01	10,12	43,55	0,860	1075,30	1728,25	11984,51
							Average	**12388,16**
							STDEV	2787,09
							CV (%)	22,50
Unreinforced	7.G	20,15	10,16	43,61	0,981	1065,09	1761,06	15319,71
	5.G	20,19	10,30	44,09	0,759	1060,07	1838,51	8887,15
	23.G	20,17	10,11	44,17	1,016	1083,03	1736,91	16879,92
	10.G	20,20	10,30	44,61	0,992	1072,05	1839,42	15343,22
	24.G	20,19	9,49	41,44	0,998	1081,40	1437,98	18426,72
							Average	**16492,39**
							STDEV	1481,84
							CV (%)	8,99

The average dynamic MOE of the wax modified wood beams before the steel reinforcement was 12,4 GPa and before the GFRP reinforcement was 16,5 GPa. Specimen WA 5.G was not taken into consideration to the calculation of the average.
The discussion of these results is made in section 5.6.2.

For the MOE_{dyn} after reinforcement (MOE_{dyn_r}), additional calculations were made in order to obtain the density and the moment of inertia of the composite beam, taking into account, all the different materials involved by using equation (33) and (34).

$$\rho_{composite} = \frac{d_{wood} \times A_{wood} + d_{reinf} \times A_{reinf} + d_{epoxy} + A_{epoxy}}{A_{wood} + A_{reinf} + A_{epoxy}} \quad (33)$$

With: d_{steel} = 7700 Kg/m³; d_{gfrp} = 1800 Kg/m³; d_{epoxy}= 1650 Kg/m³

$$I' = \frac{b_w \times h_w^3}{12} + A_w \times \left(\frac{h_w}{2} - c_g\right)^2 + m_1 \times \frac{b_r \times h_r^3}{12} + m_1 \times A_r \times \left(h - \frac{h_r}{2} - c_g\right)^2 + m_2 \times \frac{b_{ep} \times h_{ep}^3}{12} + m_2 \times A_{ep} \times \left(h_w + \frac{h_{ep}}{2} - c_g\right)^2 \quad (34)$$

With: m_1 – homogeneity factor of the reinforcement; m_2 – homogeneity factor of the adhesive.

Table 5.16 – MOE_{dyn} of unmodified wood beams (O) reinforced with steel and GFRP (MOE_{dyn_r})

Reinforcement	Unmodified (O)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m³]	I [mm⁴]	MOE_{dyn_r} [Mpa]
Steel	4.I	20,25	11,42	34,22	1,568	671,48	3522,75	16303,12
	7.I	20,25	11,55	34,80	1,580	699,16	3590,80	16887,92
	10.I	20,22	11,45	35,01	1,700	704,42	3519,31	20011,87
	17.I	20,19	11,27	33,27	1,603	658,47	3387,91	17122,95
							Average	**17581,47**
							STDEV	1656,53
							CV (%)	9,42
GFRP	20.G	20,13	11,12	26,56	1,419	655,66	3386,67	13554,63
	21.G	20,24	11,10	26,36	1,214	638,93	3381,39	9707,73
	11.G	20,24	11,19	28,15	1,408	682,26	3457,74	13731,82
	12.G	20,19	11,02	28,00	1,364	664,04	3309,10	12908,52
							Average	**12475,67**
							STDEV	1878,91
							CV (%)	15,06

The average dynamic MOE for unmodified wood beams reinforced with steel was 17,6 GPa and for the GFRP reinforced was 12,5 GPa.

The discussion of these results is made in section 5.6.2.

Table 5.17 – MOE_{dyn} of DMDHEU modified wood beams (D2) reinforced with steel and GFRP (MOE_{dyn_r})

Reinforcement	DMDHEU (D2)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m³]	I [mm⁴]	MOE_{dyn_r} [Mpa]
Steel	24.I	20,58	11,51	44,75	1,402	872,21	3679,52	16692,81
	20.I	20,61	11,64	44,14	1,524	859,99	3764,58	19073,16
	16.I	20,77	11,80	46,88	1,490	903,22	3926,86	18686,38
	7.I	20,54	11,72	44,67	1,577	879,55	3837,65	20603,13
							Average	**18763,87**
							STDEV	1609,74
							CV (%)	8,58
GFRP	22.G	20,69	11,59	38,16	1,270	839,37	3863,39	12897,23
	21.G	20,55	11,61	38,40	1,280	840,66	3864,91	13067,11
	9.G	20,68	11,68	38,47	1,260	845,27	3945,82	12597,79
	11.G	20,57	11,55	39,44	1,345	855,04	3813,05	14820,64
							Average	**13345,69**
							STDEV	1002,26
							CV (%)	7,51

The average dynamic MOE for DMDHEU modified wood beams reinforced with steel was 18,8 GPa and reinforced with GFRP was 13,3 GPa.

The discussion of these results is made in section 5.6.2.

Table 5.18 – MOE_{dyn} of wax modified wood beams (WA) reinforced with steel and GFRP (MOE_{dyn_r})

Reinforcement	Wax (WA)	b [mm]	h [mm]	Wo [g]	f [KHz]	ρ [Kg/m³]	I [mm⁴]	MOE_{dyn_r} [Mpa]
Steel	17.I	20,01	11,51	55,78	1,280	1228,66	3502,04	19564,95
	2.I	20,24	11,38	55,12	1,116	1170,73	3475,73	14497,37
	3.I	20,22	11,33	54,88	1,173	1171,62	3441,26	16155,34
	4.I	20,01	11,58	55,22	1,194	1188,55	3566,46	16285,26
							Average	**16625,73**
							STDEV	2121,80
							CV (%)	12,76
GFRP	7.G	20,15	11,12	48,37	1,063	1120,60	3388,65	13002,26
	5.G	20,19	11,19	48,40	0,863	1111,90	3451,99	8399,03
	23.G	20,17	11,09	48,91	1,088	1138,12	3365,66	13906,65
	10.G	20,20	10,52	46,12	0,979	1085,43	2250,38	13337,43
	24.G	20,19	9,88	43,23	0,993	1105,15	1878,38	15718,19
							Average	**13991,13**
							STDEV	1210,38
							CV (%)	8,65

The average results of the dynamic MOE for wax modified wood beams reinforced with steel were 16,6 GPa and 14,0 GPa with the GFRP reinforcement. Specimen WA 5.G was not considered in the calculation of the average.

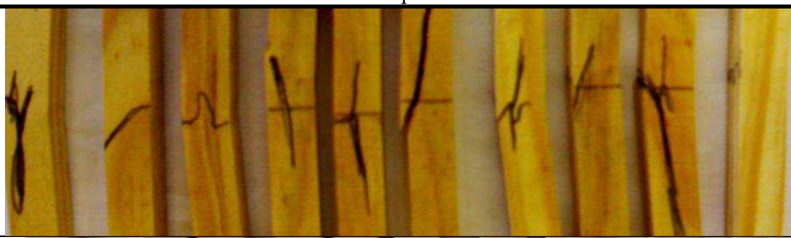
The discussion of these results is made in section 5.6.2.

5.6. Discussion

5.6.1. Failure patterns

The failure patterns are described on ASTM D143-94:2000. The experimental failure mode patterns obtained on the control specimens were extracted from Lopes (2013) and are shown in Table 5.19.

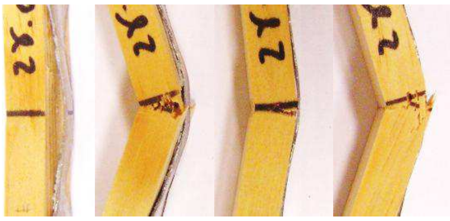
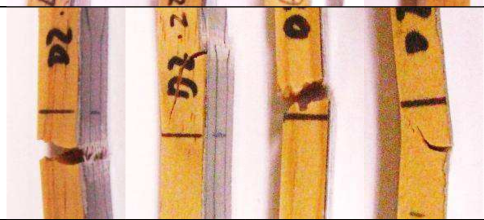
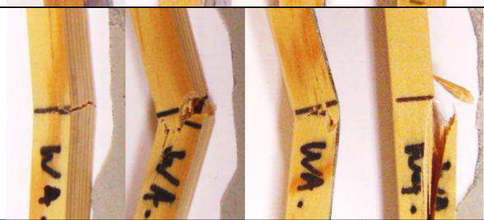
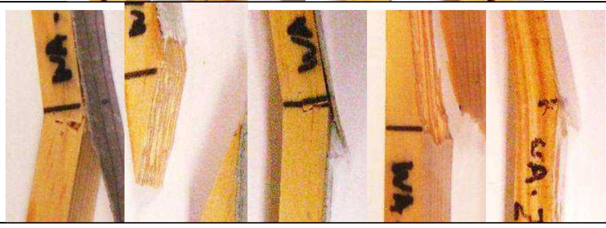
Table 5.19 – Experimental failure mode patterns obtained for 20.10.200mm RTL unreinforced specimens (Lopes, 2013)

Modification	Failure pattern
Unmodified	
DMDHEU	
Wax	

The failure pattern observed in static bending for unreinforced unmodified wood beams was simple tension and for unreinforced DMDHEU modified wood beams was typical brash tension. As for the unreinforced wax modified wood beams, the failure pattern observed was simple and cross-grain tension (Lopes, 2013).

The experimental failure mode patterns obtained on the reinforced specimens are shown in Table 5.20.

Table 5.20 – Experimental failure mode patterns obtained for 20.10.200mm RTL reinforced specimens

Modification	Reinforcement	Failure pattern
Unmodified	Steel	
	Glass fibre	
DMDHEU	Steel	
	Glass fibre	
Wax	Steel	
	Glass fibre	

The unmodified wood beams reinforced either with steel or with GFRP presented simple tension failure.

The DMDHEU modified wood beams reinforced with steel presented cross-grain tension failure, while the beams reinforced with GFRP presented brash tension failure as well as cross-grain tension.

The wax modified beams presented cross-grain tension, brash tension and simple tension failure. In this type of beams a de-bonding of the reinforcement caused a premature failure.

Overall, the type of failure was unchanged before and after reinforcement and the only major differences verified were deformation-wise.

5.6.2. Stiffness

Table 5.21 shows the average stiffness results obtained in the specimens subjected to reinforcement with steel plates. The static MOE before reinforcement (MOEst_i) was taken from Lopes (2013). The coefficient of variation is expressed between brackets.

Table 5.21 – Average stiffness values for MOE_{dyn} before (MOE_{dyn_i}) and after reinforcement with steel (MOE_{dyn_r}) and for the static MOE before (MOEst_i) and after reinforcement with steel (MOEst_r)

Modification		MOE_{dyn_i} [GPa]	MOE_{dyn_r} [GPa]	Variation [%]	MOEst_i [GPa]	MOEst_r [GPa]	Variation [%]
Unmodified	O	13,1 *(16%)*	17,6 *(9%)*	35	9,5	14,5 *(11%)*	52
DMDHEU	D2	14,5 *(14%)*	18,8 *(9%)*	30	11,6	16,6 *(7%)*	43
Wax	WA	12,4 *(22%)*	16,6 *(13%)*	36	-	13,8 *(14%)*	-

According to calculations, in steel reinforced beams, the average increase in dynamic stiffness (MOE_{dyn}) was approximately 35% for unmodified wood (O), 30% for DMDHEU modified wood (D2) and 36% for wax modified wood (WA) as schematized in Figure 5.7.

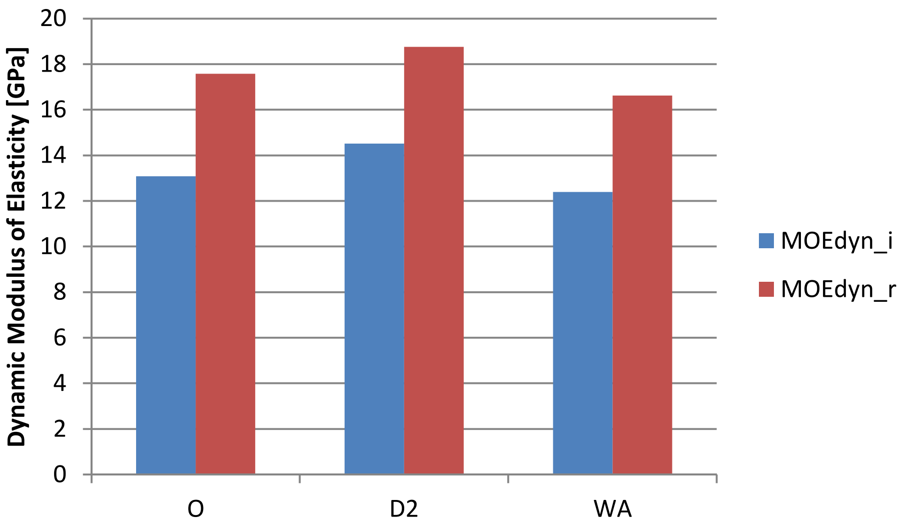


Figure 5.7 – Comparison between Dynamic Modulus of Elasticity before reinforcement (MOE_{dyn_i}) and after reinforcement with stainless steel plates (MOE_{dyn_r}) in the different types of beams (O – unmodified wood; D2 – DMDHEU modified wood; WA – wax modified wood)

The static MOE average results of the steel reinforced specimens show that there was an increase in stiffness of 52% in unmodified wood (O) and 43% in DMDHEU modified wood (D2), as seen in Figure 5.8. It was not possible to compare values of the wax modified specimens due to the lack of reference values. No relevant studies in which to compare these stiffness increments with were found in the literature.

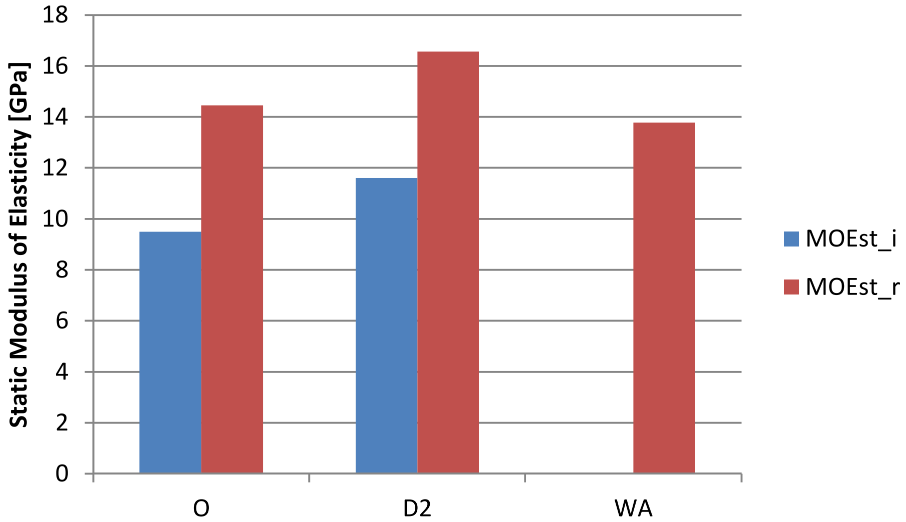


Figure 5.8 – Comparison between static Modulus of Elasticity before reinforcement (MOEst_i) and after reinforcement with stainless steel plates (MOEst_r) in the different types of wood beams (O – unmodified wood; D2 – DMDHEU modified wood; WA – wax modified wood)

Table 5.22 shows the average stiffness results obtained in the specimens subjected to reinforcement with GFRP sheets. The coefficient of variation is expressed between brackets.

Table 5.22 – Average stiffness values for MOE_{dyn} before (MOE_{dyn_i}) and after reinforcement with GFRP (MOE_{dyn_r}) and for the static MOE before (MOEst_i) and after reinforcement with GFRP (MOEst_r)

Modification		MOE_{dyn_i} [GPa]	MOE_{dyn_r} [GPa]	Variation [%]	MOEst_i [GPa]	MOEst_r [GPa]	Variation [%]
Unmodified	O	13,7 *(20%)*	12,5 *(15%)*	-8	10,5	10,8 *(12%)*	3
DMDHEU	D2	15,3 *(10%)*	13,4 *(8%)*	-13	12,0	12,7 *(8%)*	5
Wax	WA	16,5 *(9%)*	14,0 *(9%)*	-15	-	13,1 *(11%)*	-

According to the calculations, the dynamic stiffness (MOE_{dyn}) decreased after reinforcing with GFRP on an average of 8% in unmodified wood (O), 13% in DMDHEU treated wood (D2) and 15% in wax treated wood (WA), as seen in Figure 5.9.

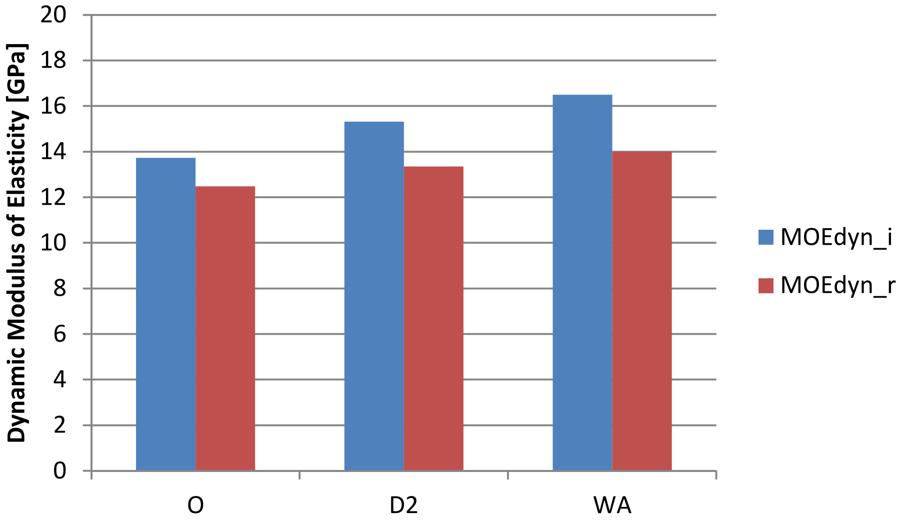


Figure 5.9 – Comparison between Dynamic Modulus of Elasticity before reinforcement (MOE_{dyn_i}) and after reinforcement with GFRP (MOE_{dyn_r}) in the different types of beams (O – unmodified wood; D2 – DMDHEU modified wood; WA – wax modified wood)

The results obtained for the GFRP reinforced beams seem to suggest that the formula used to calculate the dynamic MOE (MOE_{dyn}) is not appropriate for this study as the loss of MOE_{dyn} after reinforcement goes against its purpose. No specific formula for composite beams was found in the literature.

Despite the formula results, it is clear that the frequencies increased after reinforcement (as seen in section 5.5) and that seems to suggest by itself and by analyzing equation (35), that the MOE_{dyn} should have increased.

$$f = \frac{1}{2\pi} \times \sqrt{\frac{E \times I}{m}} \qquad (35)$$

As for the static MOE results of the beams reinforced with GFRP, no significant increases were achieved (Figure 5.10). The unmodified wood beams reinforced with GFRP had an average increase of 2% and the DMDHEU modified wood beams had an average increase of around 5%. It was not possible to compare values of the wax modified wood beams due to the lack of reference values.

When comparing these results with other studies like Dorey & Cheng (1996a; 1996b), Poulin (2001) and Fiorelli & Dias (2002), it is possible to conclude that even with a 4% GFRP reinforcement ratio, the stiffness improvement obtained in this study can be ignored. Fiorelli & Dias (2002) achieved an increase in stiffness of 28% with a 0,87% GFRP reinforcement ratio. Dorey & Cheng (1996a;1996b) achieved an increase in stiffness of 2% with a 0,04% reinforcement ratio and 7% with a 0,08% ratio. Poulin (2001) achieved an increase in stiffness of 17% with a 3% reinforcement ratio.

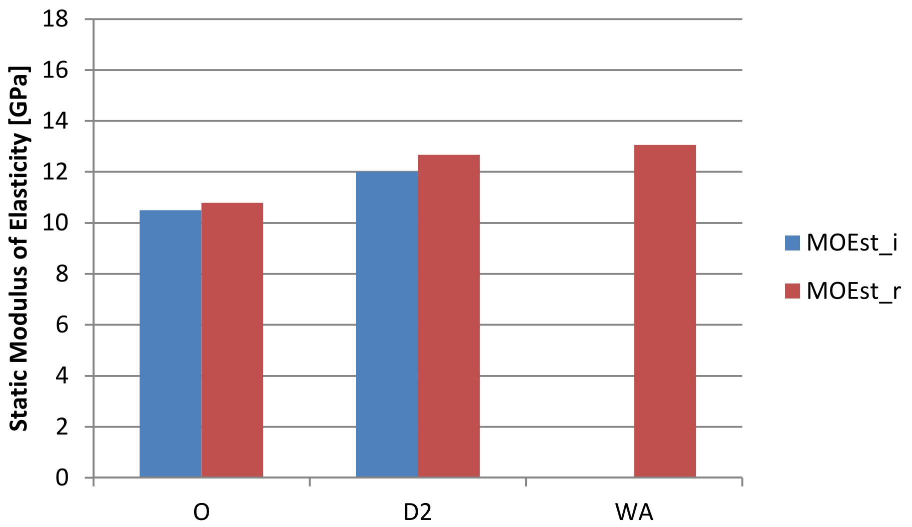


Figure 5.10 – Comparison between static Modulus of Elasticity before reinforcement (MOEst_i) and after reinforcement with GFRP (MOEst_r) in the different types of wood beams (O – unmodified wood; D2 – DMDHEU modified wood; WA – wax modified wood)

For both types of reinforcements, static MOE results are less scattered in the DMDHEU modified specimens than in the other specimens. It seems to suggest that this modification process can diminish the variability of the wood properties, as seen in Lopes (2013), even after reinforcement procedures are applied.

5.6.3. Strength

The comparison of the maximum loads was done in two ways:

- Comparing the experimental with the analytical results
- Comparing the experimental with the reference results.

The first comparison was intended to verify the similarity of the experimental values with the theoretical study carried out, and thus infer its legitimacy.

The second comparison aimed to determine the contribution of each type of reinforcement on the resistance of the different types of wood beams.

The following tables (Table 5.23 and Table 5.24) show a comparison between the analytical and average experimental maximum loads of the reinforced beams. The coefficient of variation is expressed between brackets.

Table 5.23 – Comparison between analytical and average experimental loads of beams reinforced with steel

Modification		Analytical Load [N]	Experimental Load [N]	Variation [%]
Unmodified	O	1119,85	1058,00 *(14%)*	-6%
DMDHEU	D2	1096,34	991,67 *(12%)*	-10%
Wax	WA	1319,24	1006,47 *(19%)*	-24%

Table 5.24 – Comparison between analytical and average experimental loads of beams reinforced with GFRP

Modification		Analytical Load [N]	Experimental Load [N]	Variation [%]
Unmodified	O	1084,46	1021,03 *(14%)*	-6%
DMDHEU	D2	1054,19	775,47 *(14%)*	-26%
Wax	WA	1275,85	1100,39 *(11%)*	-14%

From Table 5.23 and Table 5.24 it is possible to conclude that the analytical results are 6% to 26% higher than the experimental results. This can be explained with the fact that the analytical model considered ideal conditions, i.e. no wood defects, perfect dimensions, fixed mechanical properties, which led to higher values of maximum load. Balseiro (2007) achieved theoretical results 68% lower than experimental and Fiorelli & Dias (2002) 14% lower. The possible reasons for the difference between the theoretical results of this study and the studies previously mentioned might be the 1:15 scale used in this study and the percentage of wood fibres in the specimens. Despite the differences, it is accepted as a valid model.

The following tables (Table 5.25 and Table 5.26) show a comparison between the average reference and experimental loads obtained in the 3 point bending test. As previously mentioned, the reference values were taken from Lopes (2013).

Table 5.25 – Comparison between average reference and experimental loads of beams reinforced with steel

Modification		Reference Load [N]	Experimental Load [N]	Variation [%]
Unmodified	O	890,00	1058,00 *(14%)*	19%
DMDHEU	D2	550,00	991,67 *(12%)*	80%
Wax	WA	1090,00	1006,47 *(19%)*	-8%

Table 5.26 – Comparison between average reference and experimental loads of beams reinforced with GFRP

Modification		Reference Load [N]	Experimental Load [N]	Variation [%]
Unmodified	O	890,00	1021,03 *(14%)*	15%
DMDHEU	D2	550,00	775,47 *(14%)*	41%
Wax	WA	1090,00	1100,39 *(11%)*	1%

Overall, the reinforcements increased the maximum load capacity of the beams, as seen in Table 5.25 and Table 5.26.

The unmodified wood beams strength increased by an average of 15% when reinforced with GFRP and 19% with steel.

The DMDHEU modified beams had the highest percentage of increase in resistance, with a 41% increase with the GFRP reinforcement and an 80% increase with the steel reinforcement.

The wax modified wood beams had a decrease of 8% in their resistance when reinforced with steel plates and that can be explained by the reinforcement detachment that happened during the test, causing a sudden shift of load share between the materials which led to a premature failure. The same problem occurred with the GFRP reinforcement that also detached from the wood beam and ultimately led to no significant increase in resistance.

The de-bonding issues may have their origin on a chemical incompatibility at a cellular level as the wax treatment modifies the wood cells.

The results obtained with the GFRP reinforcement can be backed up by studies like Dorey & Cheng (1996a; 1996b), Lindyberg et al. (1998), Poulin (2001), Haiman & Zagar (2002) and Fiorelli & Dias (2002) in which increases in resistance between 20% and 120% were obtained with GFRP reinforcement ratios ranging from 0,04% to 3,5%. As for the steel plate reinforcement results, no comparable studies were found in the literature apart from Stern & Kumar (1969), where a flitch beam system was experimented and a 45% increase in resistance was obtained.

On another note, it is not viable to compare rupture tensions (MOR) before and after reinforcement because the values are inconclusive. The explanation resides in the fact that the section moduli ($w = \frac{b \times h^2}{6}$) after reinforcement greatly increases accordingly with the homogeneity factor inherent to the formed composite and that the increase in maximum load isn't high enough to counter the increase of the section moduli.

$$MOR = \frac{M_u}{w} \qquad (36)$$

Ultimately it results in lower values of MOR after reinforcement thus the only conclusive comparisons are the ones provided by the ultimate load or rupture moment.

6. CONCLUSIONS AND FUTURE RESEARCH

Except for the wood beams modified with wax, reinforcing has resulted in considerable increases in the maximum load. With the steel reinforcement, the increase in the resistance was around 20% for unmodified wood beams and 80% for 1,3-dymethylol-4,5-dihydroxyethyleneurea modified wood beams. The glass fibre reinforcement provided increases around 15% for the unmodified wood beams and 40% for 1,3-dymethylol-4,5-dihydroxyethyleneurea modified wood beams.
Even though the highest increments in resistance were registered on the specimens modified with 1,3-dymethylol-4,5-dihydroxyethyleneurea, the low strain allied with the fragile behaviour makes it a non-solution for structural engineering purposes.
Wood beams modified with wax failed prematurely due to the detachment of the reinforcing materials caused by the possible chemical incompatibility between the adhesive and the wax.

In terms of the static Modulus of Elasticity, the highest increases were obtained with the steel reinforcement, 52% in the unmodified specimens and 42% in the 1,3-dymethylol-4,5-dihydroxyethyleneurea modified wood specimens. No significant increase was achieved with the glass fibre reinforcement. The wax modified wood beams' stiffness could not be analyzed due to the lack of reference values.

The dynamic Modulus of Elasticity results were inconclusive due to the lack of a proper formula for composite specimens in the literature. Even so, data collected shows that the eigen-frequencies increased after reinforcement.

The amount of GFRP reinforcement in the cross section (4%) compared to the steel reinforcement (1,5%) made it possible to conclude that with lower reinforcement percentage ratios, the steel reinforcements achieved higher resistances. Even so, selection of reinforcement material is basically demand driven. Both reinforcing systems (steel and glass fibre) have their advantages and disadvantages, so the selection criteria should be able to find a balance between the specific strength/stiffness needs of a particular design/load case and the economical/production issues.

An extensive study considering different materials (wood species, reinforcements and adhesives), geometries and reinforcing methods can be done in order to obtain the best optimizations. The study may result in more economical cross sections without compromising the strength/stiffness properties.

The change in moisture content and changes in temperatures which are inevitable in case of exposed structures can cause expansion/contraction resulting in volume changes in the cross section. If this volume change is not occurring in a synchronised way among the constituent materials (wood, adhesive, reinforcement), it can result in residual stresses and eventually de-lamination under sustained loading. So the effect of alternating moisture content/temperature on the bond properties needs further focus.

The adhesives contribution to the composite beam should also be studied and, in the specific case of wood subjected to modification processes, compatibility tests should be conducted.

Wood modification is still an area of study that needs to be researched and in the 1,3-dymethylol-4,5-dihydroxyethyleneurea case, studies should be carried out in an attempt to alter the brittle behaviour to ductile behaviour after reinforcement.

As future research, the long term behaviour of beams reinforced with steel and glass fibre reinforced polymer should be studied with particular attention to the fatigue performance, creep behaviour, and relaxation.

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