A retrofit technique for timber beams using wooden pegs – Guidelines and experimental validation

Técnica de reforço de vigas de madeira com uso de cavilhas de madeira – Orientações e validação experimental

Alice Sartori
Hélder S. Sousa
Jorge M. Branco

Abstract

This work presents the analysis of a retrofit method for existing floor or roof timber beams, using only timber elements. The retrofit technique applies timber planks connected on the lateral sides of the existing beam by means of timber pegs. Based on literature recommendations and carpenters' experience, a guideline was prepared with the description of each step of implementation of the technique for onsite applications.

The guidelines reflect recommendations that derive from an experimental campaign regarding the application of this technique. That experimental campaign consisted of both double shear tests on smaller size specimens and 4-point bending tests on structural size elements. The results of the experimental campaign were analyzed considering different visual grading parameters and geometry of the retrofitted elements. The outputs of the experimental campaign validated the use of the proposed guidelines and further evidenced that this technique allowed for an increase of performance of the retrofitted element.

Resumo

Este trabalho apresenta a análise de um método de reforço para vigas de piso ou cobertura em madeira, usando apenas elementos de madeira. A técnica de reforço utiliza tábuas de madeira ligadas às faces laterais da viga existente por meio de cavilhas de madeira. Com base em recomendações existentes e na experiência de carpinteiros, foram elaboradas diretrizes descrevendo cada etapa da implementação desta técnica em aplicações no local.

As diretrizes refletem os resultados de uma campanha experimental realizada neste âmbito. Essa campanha experimental consistiu em testes de corte duplo em provetes de menores dimensões e testes de flexão de 4 pontos em elementos de tamanho estrutural. Os resultados da campanha experimental foram analisados considerando diferentes parâmetros de classificação visual e geometria dos elementos reforçados. Os resultados da campanha experimental validaram o uso das diretrizes propostas e evidenciaram ainda que essa técnica permitiu um aumento no desempenho do elemento reforçado.

Keywords: Timber structures / Wooden pegs / Guidelines / Retrofit / Material compatibility

Palavras-chave: Estruturas de madeira / Cavilhas de madeira / Diretrizes / Reforço / / Compatibilidade material

Alice Sartori

ISISE, IB-S, Dept. Civil Engineering
University of Minho,
Guimarães, Portugal
alice.sartori@hotmail.com
AOF – Augusto de Oliveira Ferreira & ca lda

Hélder S. Sousa

ISISE, IB-S, Dept. Civil Engineering
University of Minho
Guimarães, Portugal
sousa.hms@gmail.com

Jorge M. Branco

ISISE, IB-S, Dept. Civil Engineering
University of Minho
Guimarães, Portugal
jbranco@civil.uminho.pt

SARTORI, A. [*et al*.] – A retrofit technique for timber beams using wooden pegs – Guidelines and experimental validation. **Revista Portuguesa de Engenharia de Estruturas**. Ed. LNEC. Série III. n.º 13. ISSN 2183-8488. (julho 2020) 7-16.

1 Introduction

Timber structures have been widely used in historic buildings especially in roofs, floors and frameworks. Focusing strictly on timber floors, the structures in need of retrofit intervention, especially due to the lack of bending capacity and in-plane stiffness [1], consist in a considerable number of examples. Speaking in general about the present panorama, retrofitting technique on timber structure, in case of floors, can mainly focus either on the floor itself or on the structural elements composed by the beams. Today, the use of different materials can be used in strengthening interventions, as new timber elements, concrete layers, steel elements and FRPs. Each technique and approach display advantages and disadvantages. For instance, techniques focused on the retrofitting of the floorboards lead to an increased thickness which could imply problems of levelling, while the introduction of material other than timber can lead to incompatibility and irreversibility issues. Focusing on this last aspect, the increasing interest in conservation and compatibility of materials in intervention actions led to a higher concern about strengthening methods which could consider only timber elements. This kind of intentions is compound by the effective loss of centuries of knowledge regarding timber, in favour of material considered more modern. Because of the consequential lack in the present literature [2] of design rules for timber fastener, is thus forced the use of design rules formulated for steel dowels [3] or a derivation of what developed for steel-to-timber and steel-to-concrete approaches. This encouraged several experimental campaigns carried out by different experts in the last decades in order to overcome this lack and loss of expertise [1] [4] [5] [6].

The present work is the result of the cooperation between the Civil Engineering Department of the University of Minho and the company Augusto de Oliveira Ferreira & Ca. Lda. (AOF) and aims to study a retrofitting technique, based on compatibility concerns, validated through an experimental campaign. More precisely, the technique adopted is an evolution of the traditional method which considers the use of planks on the extrados of the beam. In this case, the work will focus on the study of the strengthening technique carried out with the application of timber planks on the lateral sides of the beams by means of a dry connection with timber pegs. This technique can be used either along the full length of the element or on located section subjected to damage or decay. The case study here presented considers 70 years old Chestnut (*Castanea sativa* Mill.) timber beams strengthened with planks of Pine (*Pinus pinea* L.) connected by means of Massaranduba (*Manilkara spp*) fasteners. With the aim of verifying the safety of the retrofitting technique and compare the results obtained with what is provided by the codes, the analysis and experimental campaign will focus on both local and global scales considering both double shear and 4-point bending tests.

2 Proposed technique – Guidelines

The procedure to follow in order to apply retrofitting planks on the side of the timber beams is extremely similar to the one adopted for the application of the planks on the extrados of the beam. Nevertheless, some peculiar arrangements should be attended.

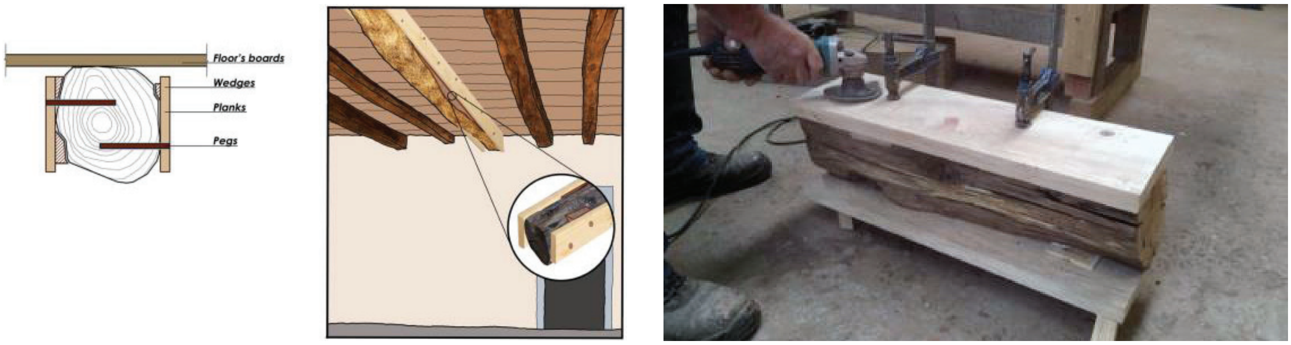


Figure 1 Scheme of the technique and construction of the tests

Dividing the procedure in different steps, the first considers the installation of a propping system followed by visual inspection, possibly with the help of non-destructive tests (NDT), in order to approximately estimate the depth of decay and the need of intervention on either the entire length of the beam or simply in correspondence of critical cross-sections. The second step focus on the cleaning of the existing structure where all the natural irregularities are maintained as long as proved not decayed. Metal elements are to be removed, if possible, without bringing more damage to the structure. The third step sees the application of the retrofitting planks which are maintained in place by means of clamps during the procedure. In case of an application along the entire length of the beam, the planks are to be inserted in the bearing wall. Wedges are then inserted in the next step to provide a smooth and regular surface for the installation of the fastener. The wedges are costume-made *in situ* depending on the presented irregularities and are to be preferably of the same timber species of the planks. Dimension and shape of the wedge are parameters that may influence the choice to use, or not, adhesive such as white glue. It is firmly recommended to avoid the use of very rigid glues as the intention is only to maintain the wedge in position prior to the placement of the wooden pegs. In the following step, the timber planks are cored and the peg is inserted right after. It is recommended to proceed always symmetrically and progressively from one end of the beam to the other and to intervene on both sides at the same time alternating the sides of application. The penetration length is not defined as it depends on the cross-section of the beam itself. As recommendation, the hole should be at least 2 cm longer than half of the width of the beam, such that an overlap of the area of interest of the pegs is guaranteed and the formation of an internal failure line due to a non-uniform distribution of the stresses is avoided. As for the wedges, white glue can be used to keep in position the pegs for the time necessary for installation. Once the installation of the pegs is finished, the propping system can be removed. Because of its possible dry nature, this technique does not require an additional time to set as its influence on the structure is immediate. Figure 1 shows the final result of the technique's installation.

3 Experimental campaign

3.1 Double shear test

For the experiments on the local scale, a total of 30 sample with different cross-sections (shapes and dimensions) were prepared from old chestnut beams with average length of 2.80 m in order to consider as much as possible the geometric irregularities along its length. To provide the presence of a considerable irregularity consistent with the beam itself, a total length of the specimens equal to 70 cm was considered. The dimensions of the cross--section varied from a minimum of 17×8 cm^2 to a maximum of 20×16.2 cm^2. Reinforcing pine planks were then added by means of 6 Massaranduba pegs (3 per each side) with a diameter of 25 mm and a variable length depending on the cross-section of the sample. Even though this technique is based on the concept of a dry connection, if considered necessary it is possible to use glue during the application in order to maintain the fasteners in position during the whole process. Because the possible structural influence of the glue is a variable to be considered, both scenarios were taken into account and a sample with white glue was compared with unglued specimens in order to verify if and how the presence of an adhesive could influence the structural response of the tests. The specimens were built following the guidelines exposed previously. In order to prepare the experimental campaign, the mechanical characteristics of the three timber species were identified with different approaches. Destructive tests for density and bending strength were carried out on samples of old Chestnut retrieved from parts of the elements to be used in the experimental campaign. The mechanical properties of pine planks were identified following the information provided by the manufacturer and provided by reference codes [7], while the values for Massaranduba were determined through previous experimental campaigns available on literature [8].

Eventually the panorama provided 18 glued samples divided in 4 groups and 12 unglued samples divided in 3 groups, division made considering strictly the presence of adhesive and the different shape of cross-section present. Figure 2 shows their grouping along with the

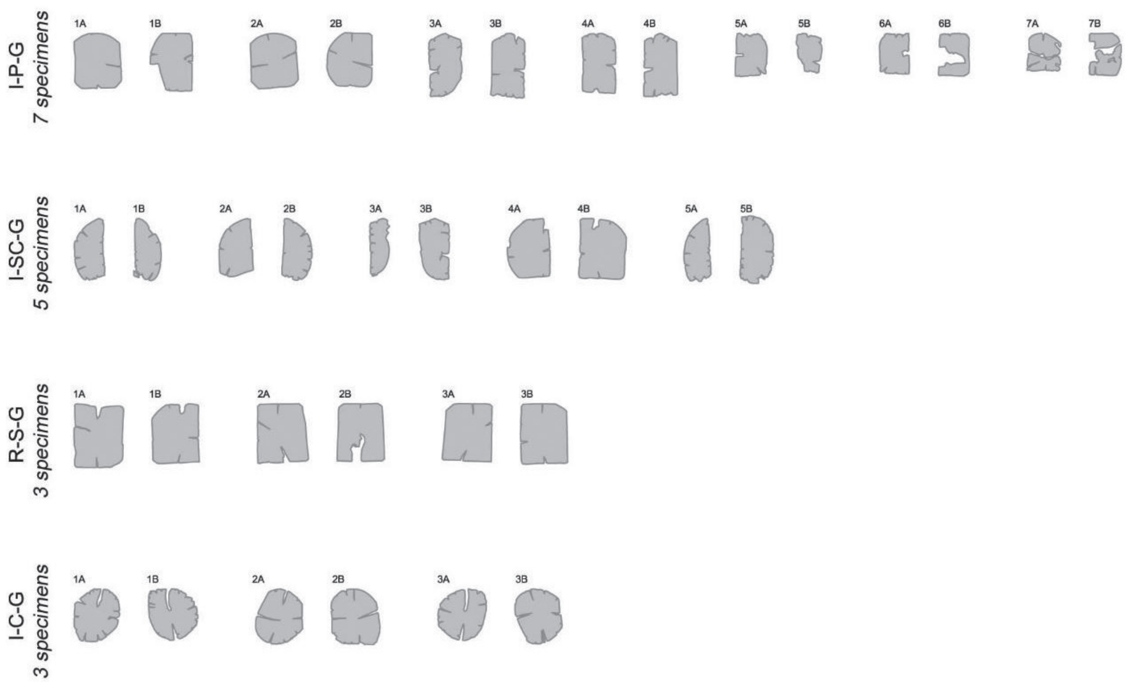


Figure 2 Example of grouping of the double shear tests (glued elements)

cross-section of both the end of the specimen. The labelling of the specimens is based on three groups of letters, where the first refers to if the cross-section is regular (R) of irregular (I), the second letter refers to the geometry of the cross-section: circular (C), squared (S), semi-circular (SC) or with parallel edges (P). The last letter explains if the sample is glued (G) or unglued (NG), the nomenclature is then completed by a progressive numeration of the sample.

The procedure from [9] was adopted for all the samples and the setup is shown in Figure 3. A 200 kN actuator was used for each test. The samples were confined laterally with the use of metal blocks avoiding out of plane horizontal displacement. Two LVDTs were then applied on the front and on the back of the test specimen in order to record the relative vertical displacement between both the reinforcement planks and the middle element (Figure 3). Starting from the displacement recorded by the two LVDTs, the mean value was then considered and used to plot the load/displacement graphs. During the tests, values of minimum and mean vertical load and slip were monitored and recorded for all samples. Table 1 shows these values while Figure 4 shows the overlap of the envelope curves of the two types of tested sample, where is showed a higher maximum load for the glued samples (111.5 kN *versus* 97 kN) with at the same time a less uniform performance when compared with the unglued one. In fact, the obtained standard deviation was equal to 29.2 kN for the unglued sample and 14.2 kN for the glued elements. This allowed to evaluate the coefficient of variation (CoV) which was found equal to 15.7% for glued elements, while unglued elements presented a value of CoV = 9.0% proving how the unglued samples provided a more uniform behaviour.

Figure 3 Sample R-S-NG-1 front (right) and back (left)

Values of stiffness for both glued and unglued samples were then calculated. Figure 5 shows the envelopes respectively for glued and unglued samples, where the chromatic differentiation is ruled by the criterion based on the cross-section's shape used to originally cluster the samples.

After, from the mean values for glued and unglued specimens the CoV were calculated and compared with the scope of verifying a possible pattern in the structural behaviour. As already outlined, the unglued samples showed a more uniform response, with a double stiffness recorded in the glued samples (16.3 kN mean value for the glued samples against a 9.3 kN for the unglued) and a CoV of 45.2% and 28.1% for respectively glued and unglued elements. This is clearly graphically visible in the envelopes' representation as in the values evaluated through the CoV.

Table 1 Data collected from double shear tests

Type	Parameter	Min	Max	Mean	CoV (%)
Glued	Vertical load (kN)	60.5	111.5	90.5	15.7
	Slip (mm)	12.2	25.6	15.7	21.8
Unglued	Vertical load (kN)	67.8	97.0	81.4	8.9
	Slip (mm)	12.5	18.4	14.0	8.9

In terms of failure, a higher amount of brittle events of the connection was recorded in the unglued samples while an example of more ductile behaviour was performed by the glued specimens. It is important to underline that if along the peg's length a void was present between the pine plank and the chestnut beam, the peg evidenced a higher tendency to break with a fragile behaviour.

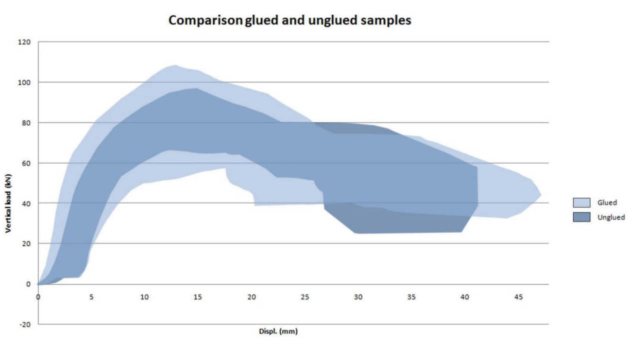
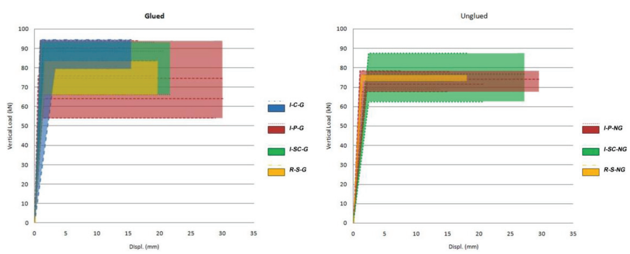


Figure 4 Envelope of load displacement curve



a) b)

Figure 5 Bilinear envelope curves for: (a) glued and (b) unglued samples

An analysis regarding the presence of wedges in the samples was made in order to identify a possible correlation with the obtained results. Regarding the glued samples, it was possible to notice a light influence when the wedges are considered, as the higher the presence of wedges the higher the mean vertical load recorded. Nevertheless, this correlation is not strong enough to be considered univocally predominant in the structural response of the samples. On the other hand, no substantial difference is noticeable in the case of unglued specimens which display an almost constant response.

3.2 4-point bending test

Both the 4-point bending test were built following the procedure explained by the guidelines. Equal in length (230 cm) the two elements displayed some irregularities regarding the geometry of the cross-section and the location of the timber pegs, more in particular, the glued sample presented an average cross-section of 17 × 10.5 cm while the unglued beam displayed an average dimension of 20 × 22 cm. Dimension in thickness and type of pine planks applied as reinforcement were identical for both samples, while the height of the planks was matching the height of the sample itself (3 × 17 cm for B-I-G-1 and 20 × 17 for B-I-NG-1). Figure 6 shows the design followed for the construction of the two reinforced beams which were assembled in the carpentry. Similarly to the tests on a local scale, the use of both dry and glued connection was adopted, for this reason the two real sized beams tested in bending were defined as B-I-G-1 for the glued element and B-I-NG-1 for the unglued sample. The construction of the samples was made maintaining the orientation that the beam had in its original structure, while the position of the pegs in the strengthening elements was chosen by the carpenter himself. This approach was chosen in order to replicate as much as possible what would be displayed on the construction site, as this technique is mainly based on tradition and experience. In order to monitor the vertical displacement of the central element in chestnut with respect the pine planks, a total amount of 4 LVDTs were placed: 2 in the middle of the span and aligned with cross section III, the other two were located on the right end and aligned with the peg present between the internal cross-sections I and II (Figure 7). More precisely, the LVDTs A and C were connected to the chestnut element, while the B and D to the external planks. The extrados of the sample was also smoothed in order to provide a plain surface so that the vertical load could be uniformly applied on the specimen. Pegs were also numbered progressively from P1 to P6, while the actuator was identified as P7.

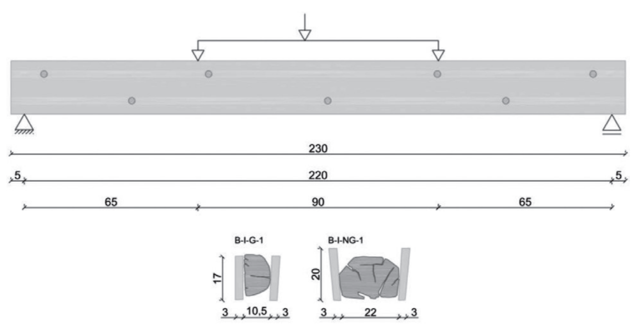


Figure 6 Set up of the 4-point bending test, measurements in cm

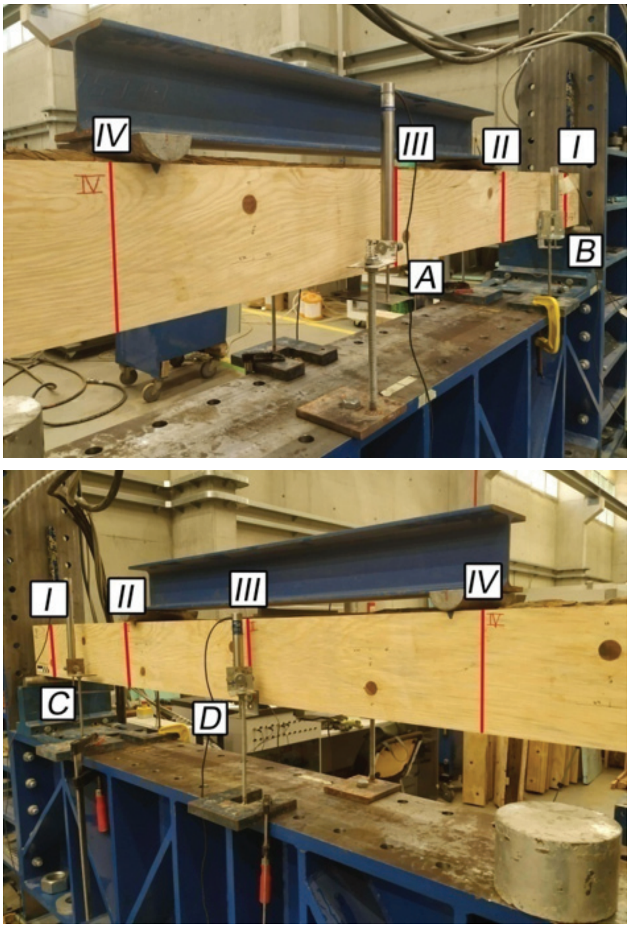


Figure 7 B-I-G-1: location of the LVDTs on the front (left) and back (right)

Both tests were carried out using a 200 kN actuator and the adopted protocol was ruled by [10]. Both beams were simply supported and loaded symmetrically along their length in two points located at 65 cm from the lateral supports. The predicted maximum displacement was evaluated with the equation of global modulus of elasticity for a 4-point bending test [10]:

$$E_{m,g} = \frac{3al^2 - 4a^3}{2bh^3\left(2\frac{w_2 - w_1}{F_2 - F_1} - \frac{6a}{5Gbh}\right)} \tag{1}$$

where: a is the distance between the two point of application of the load in mm; l is the spacing between the two supports in mm; $F_2 - F_1$ is the range of vertical load considered in kN; $w_2 - w_1$ is the range of vertical displacement considered in mm; G is the shear modulus; b is the base of the sample in mm and h is the height of the sample in mm.

From the values found with the destructive tests carried out on the chestnut samples, it was possible to evaluate the estimated maximum displacement for the beams substituting the modulus of elasticity and range of vertical load in equation (1), a value of w_{max} equal to 50 mm was found, thus 0.4 w_{max} is equal to 20 mm.

The test focused firstly on the glued sample and then on the element B-I-NG-1 and in a second moment on the element B-I-NG-1. With the data collected at the end of the tests was possible to plot the vertical load/displacement graphs for each of the 4 LVDTs present (Figure 8 shows the example for the unglued element). The presence of a mean initial deflection equal to 3.7 mm for the glued sample and 4.1 mm for the unglued beam was recorded on the vertical axis and recorded by the LVDTs as a consequence of the initial three cycles of the test's protocol.

The modulus of elasticity was evaluated with equation (1) while the value of the bending strength was calculated with equation (2) [10]:

$$f_m = \frac{3Fa}{bh^2} \tag{2}$$

where F is the maximum vertical load recorded in N; a is the distance between the two point of application of the load in mm; b is the base of the sample in mm (all the 3 elements are considered); h is the height of the sample in mm (all the 3 elements are considered).

It is important to underline that, considering the difference of mechanical properties between glued and unglued specimens noted during the experiments on the local scale, lower results were expected in matter of vertical load and modulus of elasticity regarding the unglued beam as well. Nevertheless, this element presented considerable higher dimensions in the cross section than the glued one, although a remarkable presence of decay was highlighted by the survey and visual grading initially carried out.

The values obtained from the data analysis of the 4-point bending test of B-I-G-1 and B-I-NG 1, show an apparent better result of the unglued sample respect the glued one with a maximum vertical load equal to 60.5 kN for the unglued beam against 95.8 kN recorded in the glued sample. The calculation of the bending strength, however highlighted a different result: once the influence of the geometrical dimension is neglected the glued beam (25.5 N/mm^2 and 21.5 N/mm^2 respectively for glued and unglued sample) display a strength 18 % higher than the unglued element. In this way the tendency observed on a local scale with the double shear test that pointed out as able to reach higher results the glued specimens is confirmed. The existing literature [11] provided a reference value equal to 8000 N/mm^2 for the modulus of elasticity which was evaluated through visual grading based on the presence and quantity of cracks, biological decay and wanes. Tests carried out allowed to collect values for the modulus of elasticity equal to 5289 N/mm^2 for B-I-G-1 and 6995 N/mm^2 for B-I-NG-1. The two reinforced beams provided a value of modulus of elasticity lower than the threshold assumed by the codes, and therefore a higher stiffness than the one expected by an unreinforced element.

The failure mode was observed in both tests in order to identify, if possible, a similar pattern or an analogous behavior between the samples. Considering the case of the glued sample, the failure occurred on the front plank between the pegs P4 and P3 near the right support of the vertical load initiated by the presence of a group of knots in the intrados of the beam. This caused the formation of an oblique crack that connected the two fasteners and partially the fibre in tension. This phenomenon appeared mirrored in the plank on the back of the beam. When the failure occurred the sudden loss of

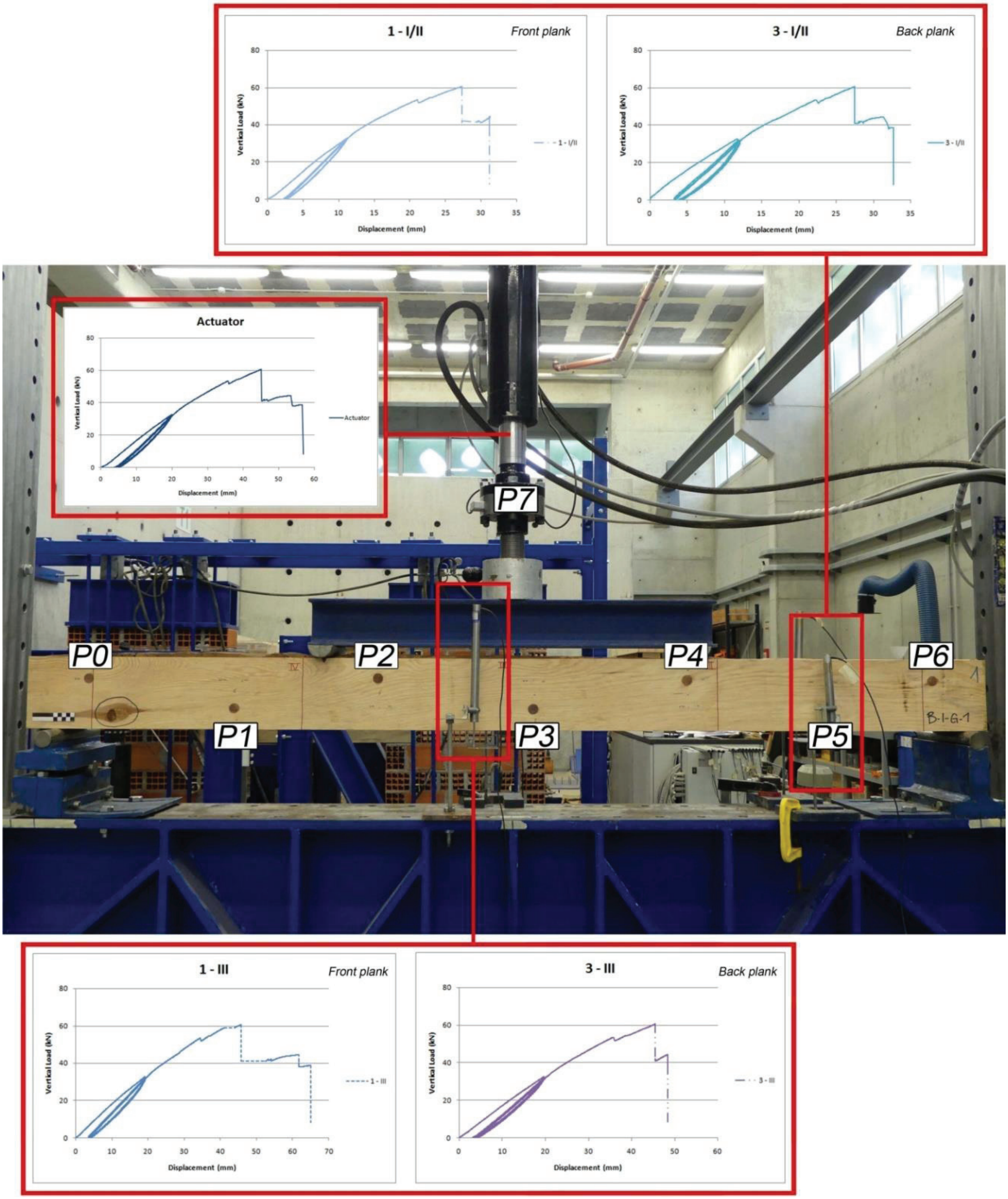


Figure 8 B-I-G-1: graphs of the LDVTs and the actuator

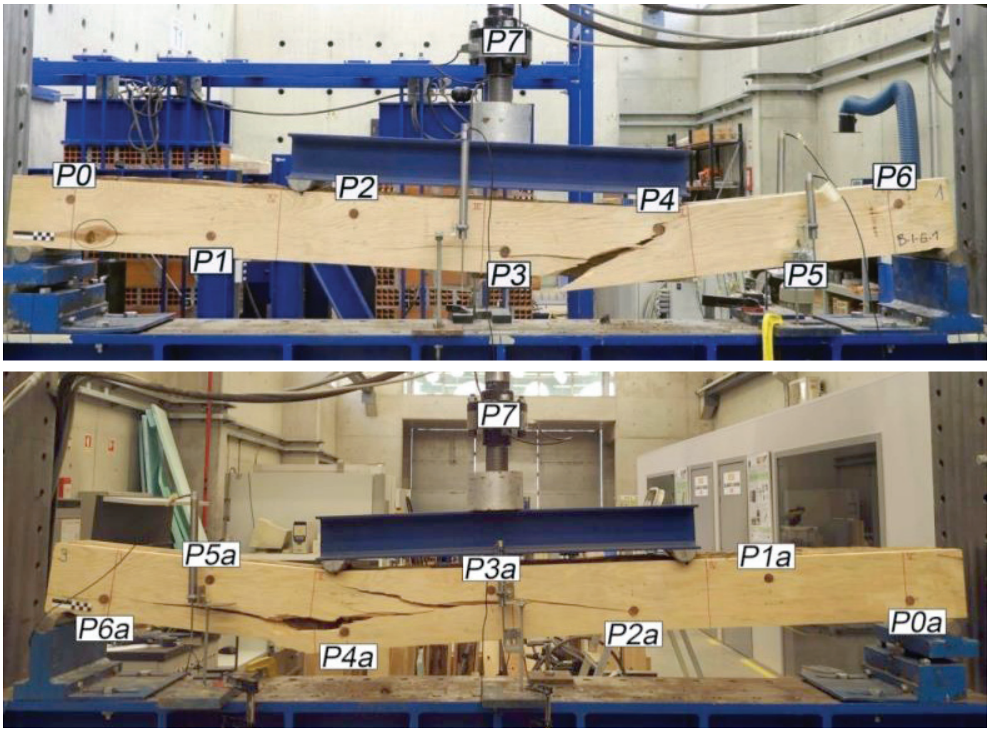


Figure 9 B-I-G-1: failure on the front plank (up) and the back plank (bottom)

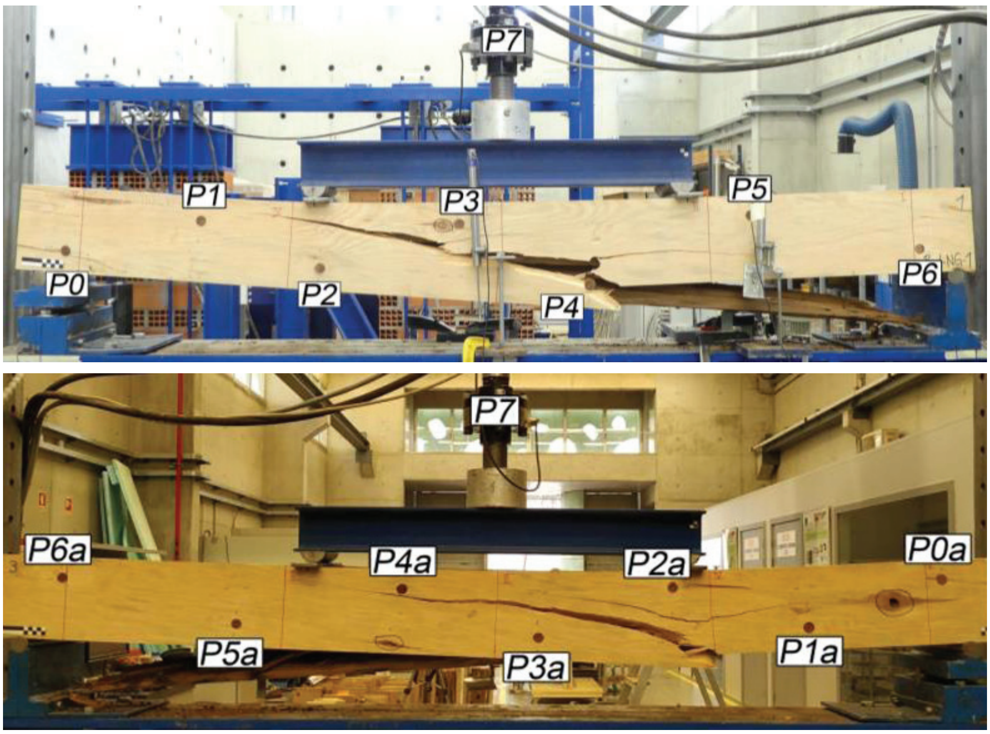


Figure 10 B-I-NG-1: failure on the front plank (up) and the back plank (bottom)

energy caused an abrupt horizontal displacement. On the back of the sample, was noticed the consequently failure of the corresponding peg on the other side of the element due to the presence of a knot in the chestnut beam.

The test on performed B-I-NG-1 displayed an initial failure of the chestnut member due to a consistent decay located in the right end of the beam which caused the sudden failure of the back plank through the formation of a symmetrical crack located on the side of the beam which expands from P4a to the intrados under P2a. This caused a twist along the cross-section which eventually lead also the front plank to collapse and the formation of a crack parallel to the fracture occurred in the chestnut element, which from P4 almost reached P1. The analysis conducted on the element pointed out the decay as reason of the failure as its presence weakened the cross-section to the point that was impossible to distribute the internal stresses and a sudden brittle failure occurred.

It is important to underline that in both the bending tests carried on the beams, none of the pegs experimented failure or significant deformation. This means that, even though able to connect the planks to the main beam and provide a composite cross-section capable with uniform behaviour, the thickness of the pegs could be optimized. Figures 9 and 10 show the failure modes of the two 4-point bending tests carried out and previously explained.

4 Conclusions

Considering all the analysis carried out, the observation done and the results presented, it is possible to assert that the reinforcing technique studied in this work gave satisfying results and was able to provide a higher performance to the strengthened timber beams. This result lead to the conclusion that a satisfying retrofitting can be provided for historical timber structure where this technique may be applied.

Differences were observed between samples with completely dry connections and where glue was used. A more uniform response was recorded in the unglued samples when subjected to double shear tests, while glued elements reached higher vertical load and performed in a more ductile way. More in detail, for the double shear test the ultimate load recorded for the specimens with glued fasteners was equal to 111.5 kN, 15% higher than the unglued elements which performed with a maximum load of 97 kN. Values of stiffness are almost double in case of glued connection when compared with the dry fasteners (respectively 16 N/mm^2 against 9 N/mm^2). More important than quantifying the results of the glued specimens as higher than the other sample, the evaluation of the coefficient of variation allowed to evaluate more precisely the structural behaviour of the local scale specimens. A more constant output for the unglued samples, characterized also by a more regular cross-section, was in fact recorded. Was possible to evaluate more precisely the structural behaviour of the specimens studied on a local scale through the evaluation of the coefficient of variation which shows a more constant output for the unglued specimens, characterized by a more regular cross-section.

Both tested beams behaved well under bending stresses with higher performance for B-I-G-1 (glued) than B-I-NG-1 (unglued) with bending strength of 25.5 N/mm^2 against 21.5 N/mm^2. These values confirm the general behaviour monitored on the local scale and the influence that the use of an adhesive has on the structural response. Nevertheless, because of the distinctive ability of this method to adapt to the existing structure, it is not possible nor is the objective of this work to establish if is better to consider the use of adhesive or not. Even if strictly related with the dimensions of the cross-section, a decreased modulus of elasticity compared with the one estimated through the destructive tests carried out on clear samples was found. The glued beam had a MOE = 5289 N/mm^2 while the unglued element provided a value equal to 6120 N/mm^2. The value extrapolated from the DTs was found equal to 7092.5 N/mm^2, in line with the visual grading performed on the tests subjected to bending (chestnut, class III with MOE = 8000 N/mm^2).

On a general level, none of the defects proper of a timber element such as wane, crack or knots seems to affect the final results of both local and global tests for this technique. On the other hand, wedges showed an apparent influence on the mechanical properties of the element subjected to retrofit. Most probably this tendency is related with location of the peg itself: when fasteners were installed in correspondence of a void between the plank and the beam, a brittle failure was more likely to occur. As the minimum spacing between the pegs in strictly related with the geometry of the beam itself, it was impossible to provide a clear and constant value especially of the pegs located along the cross-sections.

According to the obtained results, the proposed retrofit solution posed as a viable solution for use in existing timber elements even with irregular cross-sections.

Acknowledgements

This work was partly financed by FEDER funds through the Competitivity Factors Operational Programme – COMPETE and by national funds through FCT Foundation for Science and Technology within the scope of the project POCI-01-0145-FEDER-007633. The Project POCI-01-0145-FEDER-032031-UM.1.19, designated by "TimQUAKE – Desempenho estrutural de ligações e estruturas de madeira sob sismos", financed by national funds through FCT and co-financed by the "Fundo Europeu de Desenvolvimento Regional (FEDER)" through the "Programa Operacional Competitividade e Internacionalização (POCI)" is acknowledged. A particular acknowledgement to AOF – Augusto de Oliveira Ferreira & Ca. Lda. for the assistance and availability in the building of the samples and for sharing their experience and knowledge.

References

[1] Valluzzi, M. R.; Garbin, E.; Modena, C. – "Flexural strengthening of timber beams by traditional and innovative techniques". *Journal of Building Appraisal*, vol. 3, n.º. 2, pp. 125-143, 2007.

[2] EN 1995-1-1 :2004 – Eurocode 5: *Design of timber structures – Part 1-1: General - Common rules and rules for buildings*. European Committee for Standardization, 2008.

[3] O'Loinsigh, C.; Oudjene, M.; Ait-Aider, H.; Fanning, P.; Pizzi, A.; Shotton, E.; Meghlat, E.M. – "Experimental study of timber- to-timber connections using wood dowels". *Construction and Building Materials*, vol. 36, pp. 245-250, 2012.

[4] Gubana, A. – "State-of-the-Art Report on high reversible timber to timber strengthening interventions on wooden floors". *Construction and Building Materials*, vol. 97, pp. 2-13, 2015.

[5] Gubana, A.; Melotto, M. – "Experimental tests on wood-based in-plane strengthening solutions for the seismic retrofit of traditional timber floors". *Construction and Building Materials*, vol. 191, pp. 290-299, 2018.

[6] Valluzzi, M.R.; Garbin, E.; Benetta – "In-plane strengthening of timber floors for the seismic improvement of masonry buildings". *World Conference on Timber Engineering*, 2010, Trentino, Italy, 6 pp.

[7] EN 338 – *Structural timber – Strength classes*. European Committee for Standardization, 2009

[8] Branco J.M.; Teodorescu, I.; Pereira, B.; Voiculescu, M.; Tapusi, D.; Ferreira, F. – *Avaliação experimental de ligações com cavilhas de madeira*. CREPAT, Aveiro, Portugal, 29-30 June 2017. ISBN: 978-989-20-7623-2. pp. 333-344, 2017.

[9] EN 26891 – *Timber structures - Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics*. European Committee for Standardization, 1991.

[10] EN 408 – *Timber structures – Structural timber and glues laminated timber – Determination of some physical and mechanical properties*. European Committee for Standardization, 2012.

[11] UNI 11119 – *Load-bearing structures - On site inspections for the diagnosis of timber members*. Italian Standards, 2004