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Repair of Beam End Joints Using Steel Rods and Wood Prosthesis in Heritage Buildings: Implantation in the Structure of the Zabala Palace in Ordizia (Basque Country, Spain)

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Abstract: This paper presents the methodology developed in the repair of three oak beam ends in a protected heritage building: the Zabala Palace in Ordizia (Basque Country, Spain). It describes the structural assessment, design, calculation and execution process, as well as the experimental tests carried out in the laboratory to verify and validate the structural capacity of the repair method. The intervention consisted of cutting and removing the beam ends degraded by fungi and replacing them with wooden prostheses. These elements were connected to the beams by means of threaded steel rods and epoxy resin. Calculations based on standards and the literature were verified by laboratory tests where aspects such as the fluidity, filling and pull-out resistance of four commercial epoxy resins were tested. Once the epoxy resin was selected, three samples of the reinforcement design were also flexure tested. The results of the different tests show capacities much higher than those resulting from the application of the calculation procedures in the current bibliography and standards. The implemented solution allowed the conservation of most of the original patrimonial timber, following the criteria of minimum intervention.

Keywords: timber structures; repair; wood prosthesis; epoxy resin; steel rods; architectural heritage

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1. Introduction

A significant number of existing and historic buildings have wooden structures. Throughout the world, there are numerous examples of traditional wooden building typologies (farmhouses, villa houses, palaces, etc.), with different types of structural systems and elements [1–7]. Due to their cultural, esthetic, historical, technological and/or social values, some of these buildings are considered heritage, constructions that we must maintain and conserve as a society. According to international organizations such as ICOMOS [8], a timber heritage structure is considered to be any type of construction built "wholly or partially in timber that have cultural significance". Intervention on a heritage structure must be carried out in a respectful manner, keeping the intervention to the minimum necessary (ensuring safety and durability) and with the least damage to the integrity and authenticity of its character-defining elements [9,10].

Although wooden constructions historically have a high natural durability, biological deterioration can reduce their load-bearing capacity or even cause the partial or total collapse of the building. In solid wood, some of the most common damages are caused by woodworms, termites and fungi [11–15], which may appear when a certain moisture content is exceeded (depending on the species of tree and insect/fungi). This deterioration often affects structural elements embedded in walls or in contact with stone supports, such as wooden beam end joints [16,17]. These hidden elements are usually places with poor ventilation and where moisture can accumulate, due to external water filtration, internal vapor condensation or capillary action. Therefore, as they are usually located in hidden parts of the building, this type of biological deterioration in beam ends is generally difficult to detect unless a comprehensive inspection of the structure is carried out.

Regarding intervention in this type of pathology, nowadays there are more and more techniques and materials to repair timber structures [16–18], although not all the systems are sufficiently tested nor have they been fully verified in real interventions. For biologically deteriorated beam ends, one of the most common techniques is to replace the damaged part with wooden prosthesis connected by means of steel or fiber-reinforced polymer (FRP) rods or plates [19–27]. This article deals with the application of this technique in a complex structural joint of a highly protected heritage building in use and proposes a methodology of action that includes different phases: preliminary study, design of the intervention, structural calculation, experimental verification and constructive execution.

The building object of the intervention is the Zabala Palace, which is in the municipality of Ordizia, in the Basque Country (Spain). Ordizia is a settlement of great historical (it was founded in 1256 by King Alfonso X) and heritage value (the medieval village is declared a Heritage Site). The Zabala Palace is located outside its historic center, in one of the *arrabales* or first expansions of the medieval nucleus and near one of its gates. It was built at the end of the 16th century by Domingo de Zabala, an aristocrat of great historical relevance who held the position of accountant and advisor to Kings Felipe II and Felipe III. Its main section, located in the southeast area of the complex (Figure 1, left), has a rectangular plan and consists of a ground floor, two residential upper floors and an attic. Its facade is arranged along several symmetrical axes, where its main decorative detail stands out: two geminated segmental arches without mullion and with a large baroque coat of arms.

Figure 1. Image of the main facade. Aerial photo of the building, showing the corner of the courtyard with damage.

As well as other historical heritage buildings in the Basque Country [28–30], the Zabala Palace has a structure composed of masonry perimeter load-bearing walls and an interior structure of wooden posts, beams and joists. In the central part, the palace has a Renaissance courtyard, where a row of sandstone Doric columns support the wooden beams and the loads of the upper floors. Before the intervention, pathologies of biological origin were visible throughout the courtyard. The most serious damage was in the support of the northwest corner of the courtyard (Figure 1, right), in which three beam ends were badly decayed.

This article deals with the intervention carried out in these three wooden beams located in an outdoor environment. The structural joint of this corner is complex, since three beams converge at the same point with little support surface. In addition, this point is exposed to external humidity and rain, and a downspout runs through it as well. Two of the members are facade beams (Figure 2, B1 and B2) and the third beam (Figure 2, B3) allows the support of the joists in continuity with one of the facade beams (Figure 2, B1). These beams support the weight of the two upper floors, which were to be kept in use during the intervention. Due to the great decay of the beam ends, the elements were secured with props by the property before the preliminary inspection.

Figure 2. General view of the corner. Detail of the support of the three beams on the stone column.

As part of the comprehensive intervention project commissioned by the OREKA Arkitektura studio, the aim of this work was to carry out a structural diagnosis of the beams of this support, assessing the damage in detail, and to design and calculate a structural intervention respectful of and in accordance with the heritage value of the building. For this purpose, a methodology was followed that included theoretical calculations and laboratory tests in the process of developing the design of the solution.

2. Materials and Methods

The work carried out is divided into several tasks: field work and inspection of the existing structure, design of the intervention proposal, structural calculation, experimental verification and elaboration of the execution planning of the repair work.

For the assessment of this heritage structure, the specifications of the Annex I of ISO 13822:2010 [10] were followed, as well as the recommendations of the International Scientific Committee for the Analysis and Restoration of Architectural Heritage Structures of ICOMOS [8,9]. In the first phase, information on the original construction was searched in the historical archives, as well as other documents related to later transformations (refurbishments, previous repairs, etc.) or historical events that caused structural damage.

During the field work, a visual and instrumental diagnosis was carried out to determine the scope and extent of the damage and to limit the intervention to the strictly necessary. Given that the preliminary inspection detected that much of the damage was on the hidden face of the beams (on which the facade walls are built), IML Resi-300 micro-driling instrument (IML Instrumenta Mechanik Labor, Wiesloch, Germany) was used to determine the spread of the hidden decay. During the on-site inspection, the wood elements were also analyzed to determine the species, and the moisture content was measured at different points of the three beams using an Exotek MC-380XCA microprocessor-controlled moisture meter (Exotek Instruments, Fichtenberg, Germany).

After conducting the on-site inspection and analyzing the available documentation, the structural intervention design was developed. This process involved discussions and collaboration with both the architectural studio and the property owners. As a result, the key objectives and guidelines that the intervention should aim to follow and achieve, to the greatest extent possible, were established:

- The building is a protected heritage building, so the intervention must have the strictly necessary scope (minimum intervention criteria), and it must protect its character-defining elements. Furthermore, the intervention must follow the criteria of the Heritage Service of the Provincial Council of Gipuzkoa, as well as the considerations of international charters and documents [8–10]: incremental or step-by-step approach, use of compatible and sufficiently tested materials, where possible use of reversible measures, etc.
- The upper floors in the intervention area should not be evacuated. If possible, the building will be kept in use during the execution.
- The execution time (including the shoring system) should be reduced to the minimum necessary.
- Fulfilling all the above, the intervention should be as economically reduced as possible.

According to the experience obtained in previous works and following the technical specifications provided by the Heritage Service of the Provincial Council of Gipuzkoa, it was decided to use wooden prostheses connected by means of threaded steel rods and epoxy resin. The general calculation of loads and stresses in the elements was carried out using the regulations based on the Spanish Technical Building Code [31–33] and Eurocode 5 [34]. The calculation of threaded steel rods with epoxy resin was based on recognized literature [35–37] and scientific publications [38,39], as well as on documentation received from different courses and congresses. For this task, a spreadsheet specifically created for sizing this type of reinforcement was used.

To verify the calculation model of the embedded rods and the feasibility of the solution, an experimental campaign in three phases was conducted. The first phase involved assessing the suitability of different resins for injection, fluidity and filling. Four commercial resins from three different suppliers were tested. The aim was to determine whether the resins could be injected using only a single entry hole and a single exit hole. Secondly, a series of pull-out tests were conducted to determine the withdrawal strength of the threaded rods bonded with epoxy resin in oak timber blocks. Three specimens were prepared for each resin, with 12 mm diameter rods bonded into 14 mm diameter holes with a depth of 120 mm. In the third phase of the experimental verification, three samples with dimensions similar to those of the design were tested in flexure, to obtain the flexural capacity provided by each steel rod reinforcement to the element. In this case, 16 mm diameter rods, 20 mm square-shaped grooves and 40 mm depth oak covers were used.

Finally, the design of the intervention was settled and, once all possible options had been discussed with the carpenter, the construction plan was agreed upon and executed.

3. Results

3.1. Field Work and Inspection of the Structure

Regarding the study of the historical documentation on the building, no information was found on relevant transformations or previous repair works in the intervened area. The only important previous work, which is not documented, is the installation of the downspout. During the visual inspection, it was observed that the corner was already shored up by the property and that the three beam ends showed severe degradation caused by wood decay fungi (Figure 3). It was determined that the cause of this damage was leaks from the downspout and the water trap formed at the joint of the beams resting on the stone column. The on-site inspection also served to verify that the species of the three beams was oak wood.

In the instrumental diagnosis (Figure 4), the moisture meter and micro-drilling were used to determine the area and extent of degradation hidden by fungi in the three beams. To be able to measure with the necessary accuracy, some of the damaged finishes had to be removed at some points by manual means, avoiding vibrations or levers and causing the least possible damage to the healthy parts. In the three members, a gradual improvement in the condition of the wood was observed as the beams extended away from the corner. The micro-drill tests determined that the non-recoverable extent of degradation in beams 2 and 3 was slightly less than 500 mm in length from the beam end. Beam 1 showed a greater degradation, which was around 700–750 mm in length from the beam end. Thus, this study served to determine the length of the beam to be sectioned, removed and replaced by a wooden prosthesis of the same species.

Figure 3. Beam ends damaged by fungi and the downspout.

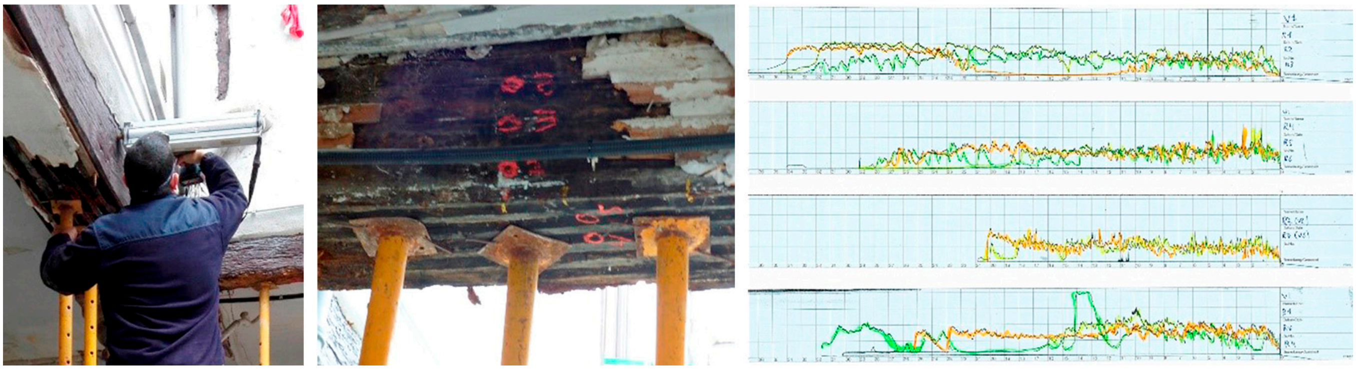

Figure 4. On-site inspection, marking of tests on the beams and several graphs obtained using the micro-drilling technique, in which the areas damaged by fungi appear with lower values.

3.2. Structural Intervention Proposal

In recent decades, the most used solutions to repair degraded wooden beam ends have been their complete replacement with new wooden elements or their reinforcement with UPN steel profiles fixed to the sides and bolted with screws or threaded rods. In this case, neither option was considered suitable. Regarding the first option, special shoring was required to support the slabs and courtyard facades, and it was difficult to replace the entire beams without damaging the facades. The reinforcement with UPN profiles was also considered inappropriate from a heritage point of view, as it altered the way the historic element was perceived, especially considering that this joint is perfectly visible from the large window of the ground floor restaurant. Moreover, being an outdoor joint, this solution could create new water traps at the junction between elements.

At this stage, the optimal approach for restoring the original appearance and structural capacity of the degraded elements with minimal intervention was to manufacture and install wood prostheses. The technique aimed to reduce both the overall intervention and the impact on the original wooden elements. After consulting with the contractor and the carpenter regarding their technical and operational capabilities, fully vertical cut (instead of an angle cut) timber prostheses were designed. This solution minimized the loss of material in the original element and facilitated the execution process for the builder.

The connection was designed to ensure the structural continuity of the prostheses and the original beams consisted of steel threaded rods bonded with epoxy resin for the bottom-edge tensile stress, and fully threaded screws inserted at 45-degree angle for transmission of the shear stress. The relative position of the reinforcement and number of rods were adjusted on every beam according to its degradation length, load and geometry (Figure 5). The rods were concealed within grooves, which were subsequently covered with oak wood covers to maintain the esthetic integrity of the structure with a fully compatible material and to protect the steel elements from fire.

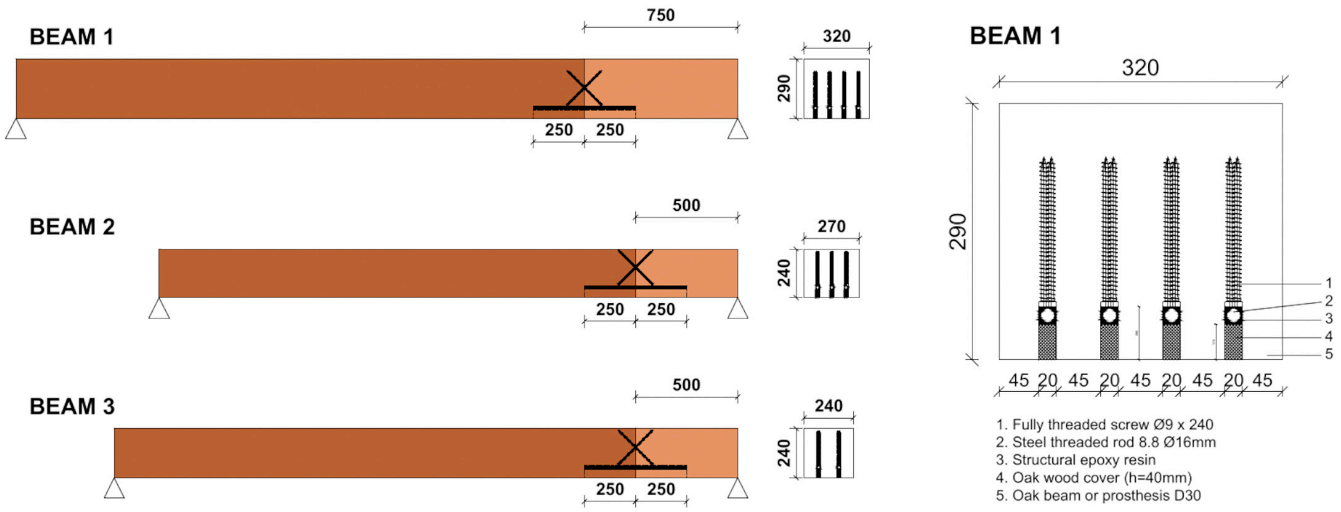

Figure 5. Design of the prostheses of beams B1, B2 and B3. Detail of beam B1.

3.3. Structural Analysis and Calculation

For the general calculation of the intervention, the formulations and regulations of the Spanish Technical Building Code [31–33] and Eurocode 5 [34] were followed. Based on the information provided by the architectural team, the loads considered in the calculation were a floor self-weight of 2.67 kN/m^2, a facade self-weight of 4 kN/m^2, a partition self-weight of 1 kN/m^2 and a live load (residential use) of 2 kN/m^2. The stresses in the sections were determined by means of classical formulas of mechanics of materials, more specifically using the homogenized section method. The pull-out resistance of the steel rods glued with epoxy resin was determined using the methods reported in the technical bibliography [35–39]. Following these formulations, moment and shear forces were calculated for each beam at the position of the cut. Using the cross-sectional parameters of the beam and a predetermined number of rods of specific diameter, the tensional stress of the rods was assessed for each load scenario, as well as their required embedment length within the wood. The following spreadsheets (Figures 6 and 7), created by the authors specifically for this project, illustrate the design and verification performed for the design solution.

Table 1 provides a summary of the key parameters and calculation results for the design verification of the reinforcement of the three beams. It includes geometrical parameters (h, b and L), characteristic permanent and live linear loads (Q_p and Q_l), the position of the cut x, the design moment M_d, and the inserted rods. In addition, the table presents the estimated axial force on each rod $F_{ax,rod}$, the resulting stress on the timber–resin bonding area $F_{ax,rod}/A_{net}$ and the verification design ratio n_d, calculated following the technical references [35–39]. With this particular design, and according to the calculation models, the governing failure mode in the reinforcement of the three beams was the timber–resin bond.

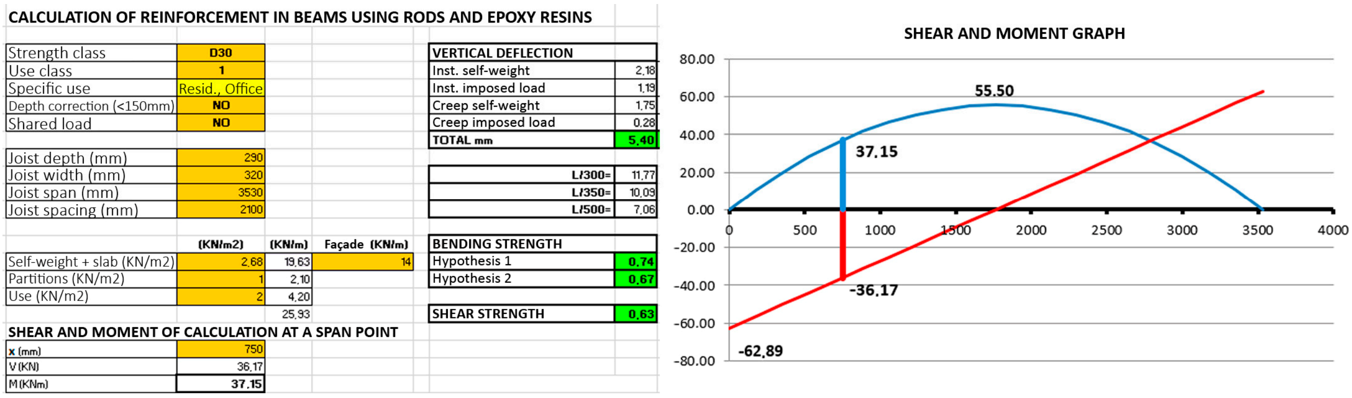

Figure 6. Spreadsheet. Input of basic data and resultant forces.

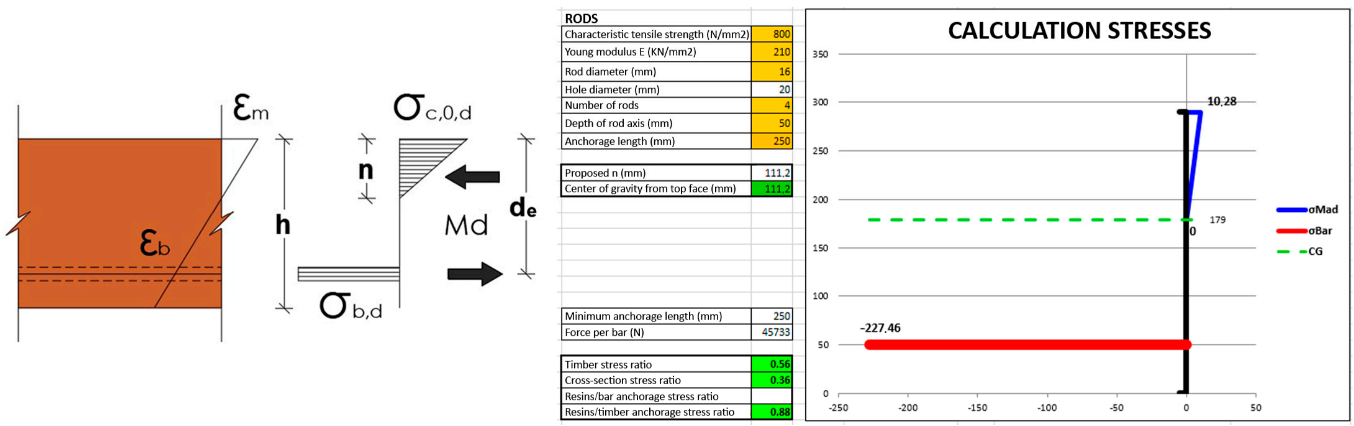

Figure 7. Spreadsheet. Calculation of rods and anchorage length.

Table 1. Main parameters and results of the design verifications of the intervention on the beams.

Element	h [mm]	b [mm]	L [m]	x [mm]	Q_p [kN/m]	Q_l [kN/m]	M_d [kNm]	rod [mm]	$F_{ax,rod,d}$ [N]	$F_{ax,rod,d}/A_{net}$ [N/mm^2]	n_d [-]
Beam 1	290	320	3.53	750	21.73	4.2	37.17	4 × Ø16	45,733	2.91	0.88
Beam 2	240	270	2.83	500	18.67	2.5	16.87	3 × Ø16	35,220	2.24	0.68
Beam 3	240	240	3.5	500	7.36	4.0	6.57	2 × Ø16	20,221	1.29	0.39

3.4. Experimental Verification

As mentioned above, the first phase of the experimental verification studied the suitability of four different resins for injection, fluidity and filling. The test specimen manufactured for this purpose consisted of a timber block with four grooves into which the rods were inserted. These rods were separated from the wood surface by wire spacers and sealed with a wooden strip glued with epoxy resin (Figure 8).

To assess the fluidity of the resin, holes were drilled along the wooden caps at distances of 50 cm, 75 cm and 100 cm from the injection point (Figure 9, left). As the resin overflowed each of the holes, they were sealed with a wooden dowel (Figure 9, center) and the injection continued. The results indicated that all four resins were suitable for injection through holes separated 100 cm apart, which was a distance significantly longer than required for the intervention. It is noteworthy to mention that one of the resins was found to be excessively fluid and was subsequently discarded, as it would partially run out through the holes if injected from below. After the fluidity tests were completed, the test sample was sectioned into four parts to verify that the space between the rod and the wood surface was adequately filled (Figure 9, right).

Figure 8. Sample preparation for testing. Fluidity and filling test of resins.

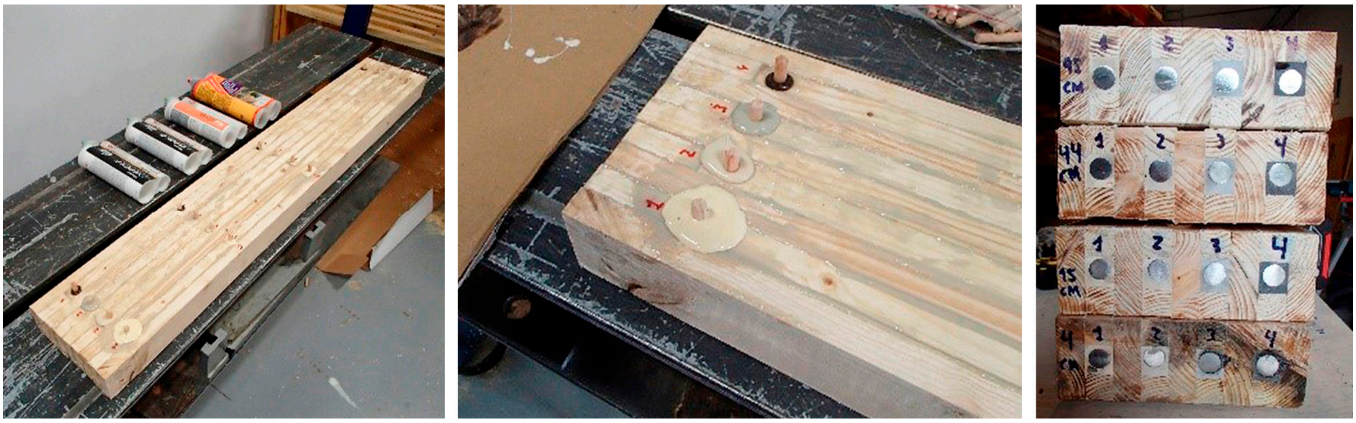

Figure 9. Fluidity and filling test of resins.

In the second phase, the pull-out tests revealed substantial differences between the four resins (Table 2), even between the two from the same supplier. Epoxy resin 1 showed a fairly constant pull-out resistance with values between 68 and 71 kN. Epoxy resin 3 also showed high pull-out resistance, although with slightly more dispersed values (66–76 kN). Epoxy resins 2 and 4 showed a lower pull-out resistance, also with a considerable variance between 34 kN and 46 kN and 52 kN and 64 kN, respectively (Figure 10). All samples showed a predominant timber failure around the bonding line.

Table 2. Results of the pull-out tests.

Resin	Sample	d [mm]	d_h [mm]	l_a [mm]	A_{net} [mm^2]	F_{max} [N]	F_{max}/A_{net} [N/mm^2]	Mean Value [N/mm^2] (COV %)
1	1-1 1-2 1-3	12	14	120	5278	70,759.53 68,161.19 69,047.45	13.41 12.91 13.08	13.13 (1.91)
2	2-1 2-2 2-3	12	14	120	5278	43,225.12 46,045.04 34,271.91	8.19 8.72 6.49	7.80 (14.93)
3	3-1 3-2 3-3	12	14	120	5278	66,328.25 65,804.55 75,644.02	12.57 12.47 14.33	13.12 (7.99)
4	4-1 4-2 4-3	12	14	120	5278	52,369.69 57,798.02 63,870.91	9.92 10.95 12.10	10.99 (9.92)

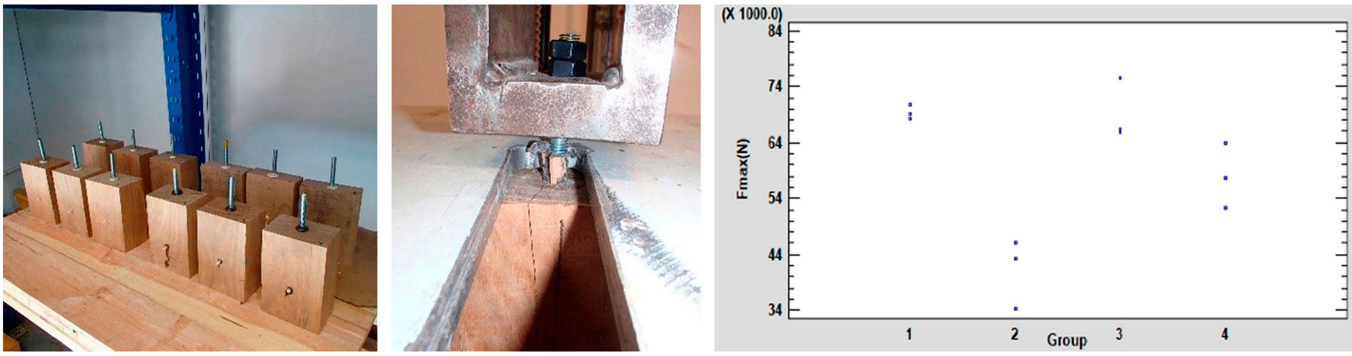

Figure 10. Samples, test and results of extraction of rods glued with 4 types of resins (1–4).

After the two initial phases of tests, it was decided to use epoxy resin number 3, because it showed a high pull-out resistance and one of the best fluidity and filling capabilities. After the choice of the resin, the third phase of the experimental verification was carried out on timber elements with beam-size dimensions. Three samples were manufactured and flexural tested (Figure 11). These test beams had the height and other characteristics of the original ones, but only one single rod in the central axis could be inserted since it was not possible to obtain historical oak elements with the same width as the beams in the project. Likewise, a test configuration with a span shorter than the original beams was designed to reproduce, in a controlled manner, the same stress that the rods would have in the real situation. Thus, the bending capacity provided by each rod to the prosthesis was obtained (Figure 12).

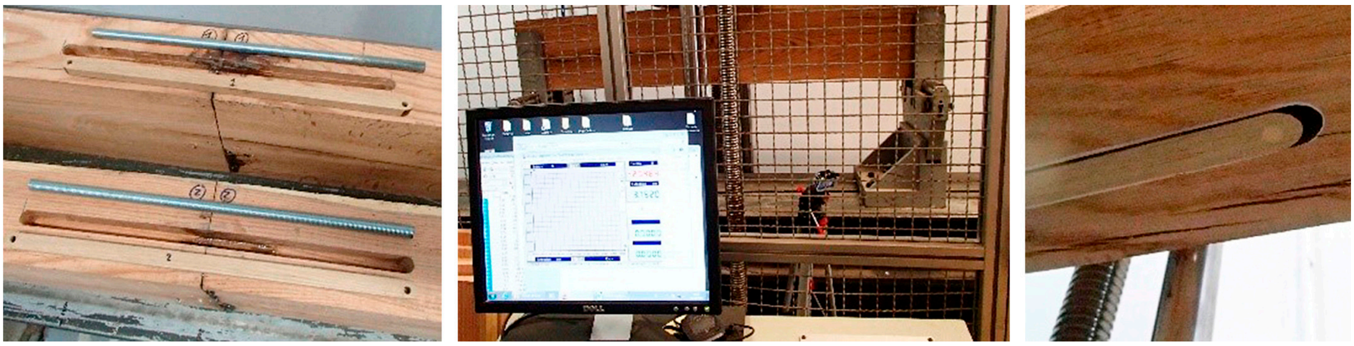

Figure 11. Samples and flexural test.

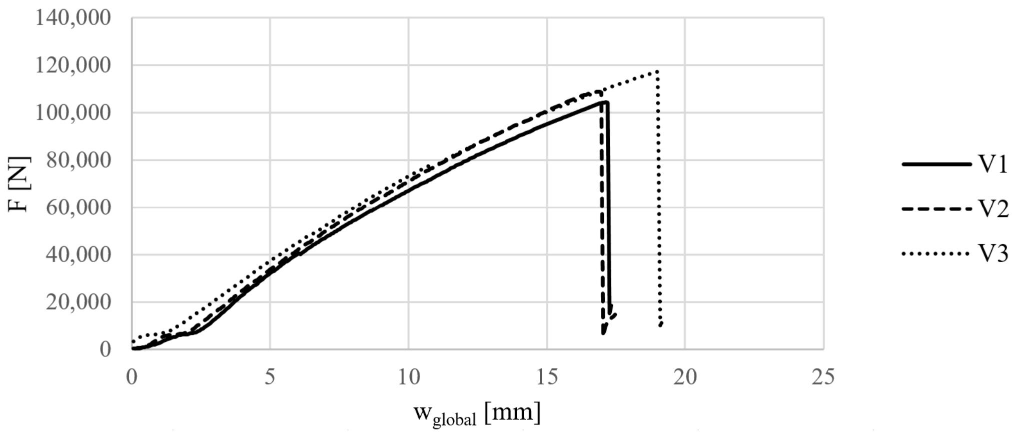

Figure 12. Load–displacement curves of the flexural tests.

The three tested samples exhibited predominant timber failure around the bonding line, consistent with the failure mode observed in the pull-out tests and predicted by the theoretical models. The results (Table 3) were satisfactory, and the solution was validated, as the bending capacity achieved in the tests exceeded the design requirement by more than 200%, even before accounting for safety factors related to load increase and long-term strength reduction.

Table 3. Results of the flexural tests.

Sample	d [mm]	d_h [mm]	l_a [mm]	A_{net} [mm^2]	F_{max} [kN]	M_{max} [kNm]	$F_{ax, rod}$ [N]	$F_{ax, rod}/A_{net}$ [N/mm^2]	Mean Value [N/mm^2] (COV %)
V1	16	20	250	15,708	104.31	30.25	148,293	9.44	
V2	16	20	250	15,708	108.70	31.52	154,509	9.84	9.96 (5.92)
V3	16	20	250	15,708	117.15	33.97	166,524	10.60	

3.5. Intervention and Execution of Reinforcements

Once the experimental tests were completed, the designed solution was executed using the selected resin. The structural reinforcement was carried out without any incidents or unforeseen issues. The first step was to adequately shore up the structure. In this case, four high-strength props were used: two for the front beam and one for each of the remaining beams. After shoring the beams, their ends were cut according to the recommendations derived from the micro-drilling analysis (Figure 13). Two of the three beams still showed shallow degradation on the upper side of the cutting plane, which was consistent with the results of the micro-drilling study and was considered acceptable.

Figure 13. Shoring and cutting of beam ends.

Once the beam ends were cut and removed, the new prostheses, made of the same species of oak wood (*quercus robur*) to ensure compatibility, were presented and glued to the beams (Figure 14). The purpose of gluing the heads is to ensure the correct seating of the two pieces. As mentioned above, it was decided to make a vertical cut instead of an inclined cut to minimize the loss of original material and to facilitate the execution process. Once the pieces were bonded and secured, both parts were grooved, and the fully threaded screws were inserted to support the shear stress. Given that it was in an outdoor environment, moisture content control of the intervened elements was necessary (the moisture content of the prostheses and beams should be almost the same).

After inserting the screws, the threaded rods were placed with the wire spacer, which also served to hold the rods in place before the resin was injected (Figure 15). The epoxy resin used was in cartridge format. Subsequently, the oak cover was installed, and the resin was injected through the entry hole until resin began to come out of the exit hole (Figure 16). Thus, the injection was stopped, and wooden dowels were placed over the two holes. Finally, the mixture was allowed to cure according to the specifications of the manufacturer, leaving the element available to apply the necessary finish treatments.

Figure 14. Placement of the wooden prosthesis. Grooving and insertion of fully threaded screws.

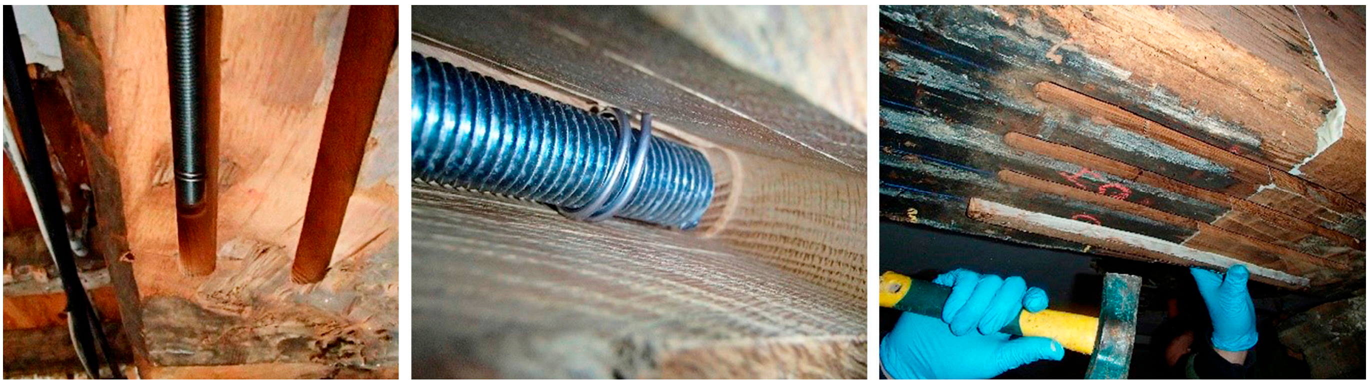

Figure 15. Insertion of rods and placement of glued wooden covers.

Figure 16. Injection of resins, placement of dowels and final result without finish treatment.

4. Conclusions

The intervention carried out was successful and the objectives established in the project were effectively met. The reinforcement system with threaded rods and epoxy allowed the restoration of the structural integrity and load-bearing capacity of the structure with minimal intervention, while preserving the appearance and historical authenticity of the structure and ensuring compliance with modern building safety standards.

The system proved to be both effective and relatively simple for an experienced carpenter to implement. The cartridge-format resin, despite its higher price, was found to be cleaner and faster than resins that require manual mixing, and the overall cost impact was minimal due to the small amount of resin used in the reinforcement, making the price difference negligible within the total project budget.

The experimental tests revealed that, when properly executed, this type of solution can achieve a significant mechanical capacity, showing a resulting bending strength considerably higher than what was predicted with the available formulations and models found in the literature.

Despite its advantages, this technique is not without limitations. Although initial tests showed that the different epoxy resins adequately filled the spaces between the inner surfaces of the grooves and the rods, ensuring complete filling of the grooves remains challenging. While the filling process can be monitored during execution through the observation of the resin overflow from the exit hole, achieving complete assurance that the entire groove is adequately filled is yet difficult. Special attention must be paid to the grooving finish, as anatomical imperfections (such as cracks or dead knots on the inner wooden surfaces) can cause the resin to flow in unintended directions, thereby compromising the effectiveness of the reinforcement. These factors, together with the need for humidity and temperature control, underline the importance of careful execution and monitoring throughout the application of this technique.

In this respect, it is considered convenient to continue advancing in this research line, until standards such as Eurocode 5 include standardized construction guidelines and calculation procedures that make the verification and execution of this type of reinforcements available to all competent technicians. Thus, the methodology developed in this complex real case is expected to be further improved and developed in the coming years.

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