NATIONAL RESEARCH COUNCIL

ADVISORY COMMITTEE
ON TECHNICAL RECOMMENDATIONS FOR CONSTRUCTION

Guidelines for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures

Timber structures

Preliminary study

CNR-DT 201/2005

CONTENTS

1. FOREWORD

The present document adds to the series of documents recently issued by the CNR on the structural use of fibre-reinforced polymer (FRP) composites, started with the publication of CNR-DT 200/2004. The topic of the present document is the use of externally bonded systems for strengthening timber structures.
This is an informative document. The intended objectives of this document are to disseminate, within the professional community, the current knowledge on the use of FRPs for strengthening of timber structures, and to provide the necessary information to identify cases where the use of FRPs for strengthening applications is appropriate and safe.
It is well-known that the current state of the art on this topic is not as advanced as that on concrete and masonry structures. Accordingly, no international guidelines on the use of FRPs for strengthening of timber structures are available at present. In this context, the present document represents the first step towards the development of specific design guidelines. These guidelines will be issued in the near future, once the theoretical and experimental studies currently underway internationally, will produce a more complete and consolidated understanding of the subject.
This document will then be useful to identify any still unsolved problems , therefore allowing the scientific community to focus on them over the next years, and will become a reference document on this topic.
The current version of this document contains some basic concepts on FRP strengthening and on related specific problem~~s~~, on strengthening of structural members under combined bending and axial force, with particular reference to strengthening of wooden floors and their stiffening under in-plane loads. Some qualitative aspects of debonding issues and of strengthening of joints are also addressed.
Finally, three Appendices are included, which present several examples of FRP strengthening interventions on structures (Appendix A), the main technical references (Appendix B) and the design code references on the subject (Appendix C).

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1.1 PUBLIC HEARING

After its publication, this document CNR-DT 201/2005 was subject to a public hearing. Following the public hearing, some modifications and/or integrations have been made to the document including corrections of typos, additions of subjects that had not been dealt with in the original version, and elimination of others deemed not to be relevant.
This Technical Document has been approved as a final version on 18/06/2007, including the modifications derived from the public hearing, by the "Advisory Committee on Technical Recommendation for Construction", whose members are:

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1.2 SYMBOLS

General notations

$(.)_d$ design value of quantity (.)
$(.)_f$ value of quantity (.) for the FRP
$(.)_R$ value of quantity (.) as a resistance
$(.)_S$ value of quantity (.) as a demand

Uppercase Roman letters

A cross-sectional area of timber
A_f cross-sectional area of the FRP
B width of the cross-section of the timber beam
E modulus of elasticity in tension of timber

E_f modulus of elasticity in tension of the FRP
H depth of the cross-section of the timber beam
M_{Rd} design value of flexural capacity
M_{Sd} factored bending moment
N_{Sd} factored axial force
T_g glass transition temperature of the resin

Lowercase Roman letters

b width of the cross-section of the FRP
h depth of the cross-section of the FRP
f_{cd} timber design compressive strength
$f_{c,el}$ timber yield compressive strength
f_{cu} timber ultimate compressive strength
f_{tu} timber tensile strength
p_{frp} ratio of the depth of the FRP from the top of the section to the depth of the beam cross-section
n ratio of the modulus of elasticity in tension of the FRP to the modulus of elasticity in tension of timber
k ratio of the ultimate strain to the yield strain in compression
k_φ in-plane rotational stiffness of a couple of connectors joining a board to the beam

Lowercase Greek letters

η ratio between the timber tensile strength and compressive strength
ρ_{frp} ratio between the cross-sectional area of timber and FRP
ξ ratio of the depth of the neutral axis from the top to the depth of the beam cross-section
ε_s longitudinal strain of the top fibres of the section
ε_i longitudinal strain of the bottom fibres of the section
ε_{tu} timber ultimate tensile strain
$\varepsilon_{c,el}$ timber yield compressive strain
ε_{cu} timber ultimate compressive strain

2 AIMS AND APPLICATIONS

2.1 INTRODUCTION

This document specifies the criteria and conditions for the most technically up to date use of FRPs in strengthening existing timber structures.
Structural timber is included by the technical codes among the construction materials suitable for load-bearing applications. This material has a long application history, as witnessed by the high durability of well-designed and built structures. In many Italian cities, there are several examples of buildings with centuries-old timber roofs or floors that are still fully functional and require only routine maintenance. In recent years, the ever-increasing need for greater resistance and stiffness of timber and glued-laminated timber structures has led to an intense increase in the experimental activity being carried out in the field of composite timber-FRP structures.
Even though experimental tests in the field of timber-FRP structures have been carried out for over 15 years, the application of this technology is still under development, with the current state of the art being in continuous evolution.
Prior to the application of composite materials for structural use, various strengthening techniques, with different degrees of efficiency, were tested in the past, with the aim of increasing the strength and stiffness of timber structures. These included the use of elements made from timber materials, the use of steel and aluminium bars or plates with and without pre-tensioning, up to the more recent use of laminated timber elements reinforced by steel and carbon bars, inserted during manufacturing.
While all these strengthening techniques obtained significant and useful results, only a few of them have been diffused and/or commercialised, and almost none of them have achieved the *status* of being universally recognised or adopted.
This can be attributed to various causes, including manufacturing difficulties and high costs, the limited versatility of the techniques studied and the need for specialised staff. A further reason is the limited dissemination of the technical knowledge needed to safely and effectively use these techniques in design, with special regard to the long-term behaviour, which has been worsened in Italy by the long unavailability of national design codes on timber structures.
In this context, fibre-reinforced composite materials offer several evident advantages, being easily applicable and extremely versatile for both retrofitting of existing structures and design of new ones. These attractive characteristics have favoured a rapid and wide diffusion of the FRP strengthening techniques for reinforced concrete and masonry structures. These techniques are now included among the tools of many designers and are proving to be effective for the rapid solution of numerous problems.
A similar process has not occurred for timber structures due to two main factors. The first is the lack of any specific guidelines for timber structures. Until recently no national design guidelines were available for design of timber structural elements. The second factor is the lack of confidence in timber as a structural material, stemming from technical ignorance and groundless prejudices. This has limited the use of timber to roofing and to constructions situated in specific geographical areas (mainly the mountains). It is therefore inevitable that a smaller market absorbs innovations more slowly, due to the more limited possibilities of experimentation and to the reduced number of investments.
From a pure engineering perspective, the use of FRP materials in combination with timber presents several advantages due to the compatibility and complementarity of their characteristics. For example, the low weight of timber, which is one of its most appreciated features, is unaffected by the FRP strengthening. Moreover, the most evident deficiencies of timber, such as the pronounced mechanical heterogeneity due to the presence of numerous defects, can be mitigated by the synergy with another structurally efficient material such as the FRP composite.

The combination timber-FRP has also some limitations due to the nature of the materials involved, including the different behaviour of the two materials in relation to temperature and humidity variations, and the different fire behaviour. Another issue is related to the connection techniques (adhesive bonding, mechanical joints). The issues mentioned above justify the need to carry out further detailed studies on the compatibility of the two materials and on the possible strengthening applications, including reliability and durability aspects.
The number of studies being carried out in Italy on this topic is increasing, and significant developments are occurring in writing of design guidelines.
This document represents in itself an important step towards the correct diffusion and use of the techniques already available, and the development and testing of new applications.

2.2 FIELDS OF APPLICATION

2.2.1 Foreword

Almost all the static problems relative to timber structures can theoretically be solved through the appropriate application of FRP materials.

In practice, the most common and less problematic approach is the "improvement" type of interventions, applicable to situations otherwise unsolvable without difficult design compromises. In most cases, this type of intervention compensates for the high cost of the FRP materials.

The expression "strengthening of timber structures with FRPs" may be intended as the restoration of the performance of a deteriorated element – (i.e. rehabilitation) - and the upgrade of the structural performance of an undamaged structural element.

2.2.2 Solutions of Proven Effectiveness

The most widespread application is undoubtedly the strengthening of timber elements under prevalent bending, including single beams, floor beams and single elements of more complex structures such as trusses and frames. The strengthening system can consist of plates or sheets of various materials, applied according to the criteria that offer the best advantages in terms of strength, deformability and ductility.
Another application is the strengthening of structures subjected to in-plane actions. A typical application is on timber or mixed floors. Such structures, rather widespread in many European countries and historically dominant on the Italian territory, have lightweight, good resistance, acoustic isolation and compartmentalization characteristics but offer limited stiffness and modest effectiveness in the transmission of horizontal in-plane forces. This last requisite, important for a correct design in seismic areas, can be easily satisfied, without affecting the weight or thickness of the structure, by connecting the wooden planking or the existing slab with a rapidly applicable grid, made of two or more rows of FRP plates laid orthogonally to each other.
A further application is the strengthening of the joints between timber elements. Composites with different microstructures can be used according to their function. The desired results are mainly the following: reduction of the failure risk due to tensile stresses perpendicular to the fibres, reduction of the distance between the joints and between the joints and the edges, improvement of the ultimate behaviour of the connection and improvement of the hysteretic dissipation capacities of the connection under cyclic loading. There are many cases occurring in practice including the transmission of axial, bending and shear forces.

2.2.3 Solutions of Unproven Effectiveness

When strengthening timber elements under compression with FRPs, some important

contraindications are worth noting. While FRP wrapping of concrete columns gives adequate confinement, the same results cannot be achieved with the same guarantee of long-term effectiveness for timber elements, due to the bulking and shrinkage movements caused by thermo-hygrometric variations, that can subsequently compromise the confinement effect.
Due to the aforementioned reasons and to the particular mechanical behaviour and collapse modes of structural timber elements, to be treated later, any strengthening intervention based on wrapping configurations of the FRP system (wrapping of the timber elements) is of dubious effectiveness. Moreover, such interventions often have strong aesthetic impact and limited reversibility, which is completely unacceptable in rehabilitation of existing structures, classified as part of the Cultural Heritage.

The FRP strengthening interventions addressed in this document do not give a significant increase to the shear resistance of timber.
In the case of traditional timber trusses, it is generally not recommendable to strengthen the structural joints by bonding FRP plates, as this would unacceptably constraint the relative displacement of the members, which is an essential condition for the correct working of the truss.

2.3 RELATED PROBLEMS

2.3.1 Foreword

The procedures briefly described in § 2.2 may present several difficulties in execution, which require specific solutions. The following section comments on the most relevant ones, whereas a more detailed analysis is presented in the subsequent chapters. The choice of the type of strengthening and its geometry and the choice of the type of connection cannot be operated separately. Instead, they must occur simultaneously, within a single phase identifiable as the "design of the strengthening intervention".

2.3.2 Types of Connection

The FRP materials can be connected to timber using various types of connection, including nails, studs, bolts, screws or adhesive. Each of these solutions has different effects on the connection behaviour and in particular on the transfer of stresses, which influences the stiffness of the composite element and the exploitation of the single materials.

The most common type of connection between the FRP strengthening and the timber members is adhesive bonding, given its high versatility and the compatibility between commonly used adhesives and the polymer matrix of the composite. In order to obtain the best results, several aspects regarding the timber-adhesive interface must be taken into account. Lacking or inadequate preparation of the surfaces can seriously compromise the effectiveness of the strengthening intervention.
The following requirements are fundamental for an effective bonding:

- the adhesive should sufficiently "soak" the timber surface and have a rheological behaviour similar to that of the adherends;
- the timber surface should be carefully prepared, in order to result as uniform and regular as possible;
- the thickness of the adhesive layer should be the optimal one according to the indications of the Manufacturer.

The first requirement can be easily fulfilled by choosing a type of adhesive that is compatible with the timber. The second requirement can create serious difficulties, especially when strengthening existing structures, especially if they are extremely deteriorated and have highly uneven surfaces.

2.3.3 The "timber" component

The heterogeneity and anisotropy, characteristics of the timber element, influence both the nature and the disposition of the connection in relation to the stresses that it must bear and transfer. Considering that the timber fibres are orientated (usually in a direction similar to that of the element axis but not in every region of the same element), timber members exhibit a limited resistance to tensile and compressive stresses orthogonal to the fibre direction. These stresses should therefore be minimised with a proper design. In design, other timber characteristics including the species of wood, presence of imperfections, humidity, type of selection and the state of conservation should be investigated.

The schematization of a timber element as a "homogenous beam with a constant cross section", which is usually adopted in the design calculations, is overly simplified and cannot be acritically used for elements with an excessive presence of imperfections. Moreover, the presence of severe fibre irregularities, excessive knots and evident form imperfections may induce an unpredictable response of the FRP-timber connection.

The resisting profiles of timber structural elements are described in the following sets of guidelines UNI EN 338, UNI 11035 and UNI 11119, which should be referred to for further details.

2.3.4 The "FRP" Component

FRP composite materials have a microscopic structure that has been optimised in order to obtain a well defined constitutive behaviour. From a technological perspective, such materials are particularly suited for the fabrication of thin plane elements, commonly known as plates.
Elements made with fibres oriented in a single direction are ideal in absorbing tensile or compressive stresses in the same direction, but they are inadequate for absorbing stresses orthogonal to the fibre direction.
Conversely, compositions with fibres disposed in different directions, usually made by combining more layers with fibres having different orientations, are suitable to bear in-plane actions of any direction.

2.3.5 Durability

Durability of strengthening is an important aspect, strictly but not exclusively connected to adhesive bonding. The adhesive with the best performance characteristics at the time of application, does not necessarily maintain the same characteristics over time, especially under harsh environmental conditions. Considering that timber is subjected to significant deformations connected to humidity variations, an adhesive can only be considered compatible when it adapts to these deformations, thus avoiding or limiting at the minimum the risk of debonding. Furthermore, in the case of pre-tensioned strengthening, the long term effectiveness of the intervention assumes that the strengthening has the capability to maintain the imposed stress state, achievable only when an adequate bonding has been carried out.
The problem of durability can also be dealt with by providing protection to the strengthening system. For instance, the environmental conditions under which the strengthening will be placed can be controlled or, if this is not be possible, it is possible to adopt an appropriate geometry of the connection or other construction details in order to protect it adequately. The same result can be achieved through the choice of materials possessing a good resistance to physical or chemical environmental attacks. For example, in the case of the strengthening of a timber roofing, an

adequate environmental isolation can be sufficient to protect the strengthening system from temperature and humidity excursions. Whereas in the case of the strengthening of a bridge deck, it may be more appropriate to select FRPs with better chemical resistance and adhesives compatible with the particular service class (as defined in the code EN 1995-1-1).

2.3.6 Behaviour in case of fire

Independently from the thermal resistance of the fibres, related to their chemical composition, that of FRPs is always governed by the corresponding resistance of the polymer matrix. In fact, resins tend to lose a large part of their mechanical characteristics at around 100°C and therefore are highly sensitive to the risk of fire. In addition, regardless from the type of strengthening, the adhesive used in composite-timber bonding has a glass transition temperature always well under 100°C. The aforementioned problems can be solved by taking advantage of the high thermal isolation capacity of timber and by studying effective protection solutions. The latter include the use of an appropriate geometry that adequately covers the strengthening with another layer of timber, thus protecting the FRP and completing the timber structure from the structural, functional and aesthetic points of view. It is worth noting that, in cases where the importance of strengthening for the load-carrying capacity is greater, the need for adequate protection becomes more crucial. This is due to the fact that the collapse of the polymer matrix or the adhesive could imply a drastic and immediate reduction in the load bearing resistance of the strengthened element.
For further details, see CNR-DT 200/2004 ("Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures. Materials, RC and PC structures, masonry structures").

2.3.7 Ultimate behaviour of the strengthened element

The last significant problem relates to the Ultimate Limit State (ULS) of the strengthened element, i.e. its behaviour at incipient structural collapse.
Firstly, it is worth highlighting that the collapse of a timber member generally is triggered where natural defects (knots, excessive fibre inclination) are located, in a brittle fashion. Secondly, in most practical cases, not only flexure and axial force but also shear can be responsible of the collapse (stocky beams, moment-resisting joints), and also other specific phenomena such as *rolling shear* (in timber-based panels), cracking (in the presence of cuts and borings) and bearing (in mechanical joints).

Composite materials have a nearly linear elastic behaviour up to failure, that consequently appears to be brittle. In elements under bending, by locating the strengthening in the tension zone, it is also possible to mobilize sufficient strains in the compression region, so that localised plasticization of the timber occurs before the tension region fails.

3 GENERAL DESIGN AND EXECUTION CRITERIA

3.1 INTRODUCTION

When the strengthening concerns a new structural element, the quality of the materials used can be easily verified and their workability is guaranteed by an industrial process assisted by the most modern quality control criteria. Whereas, when the intervention concerns an existing structural element, it is appropriate to follow the applicability criteria discussed in the subsequent paragraphs.

For what not specifically indicated in the present document, the basic concepts for the strengthening design and special problems are reported in CNR-DT 200/2004.

The interventions on buildings of historical, cultural and artistic interest are regulated by the code UNI 11138, recently published, which should be consulted for further details. It defines the logical sequence for the activities related to the design and execution of the strengthening, as follows:

- preliminary evaluation of the state of conservation;
- design;
- control of the effectiveness;
- execution methodology and techniques;
- periodic inspections.

The essential requirement for the aforementioned activities is the preliminary check of the opportunity of carrying out the intervention in the spirit of conservation and protection of the historical and artistic values.

Furthermore it is worth noting that the preliminary analysis of the state of conservation is also to be intended as a technical-economic evaluation of the available alternatives to the strengthening. These could also include the substitution or reconstruction of more or less extensive parts of the structural complex.

In cases where the conservation of an original structural element is not dictated by artistic or cultural issues, but simply by technical-economical ones, the element must have sufficient residual performance in order to justify its conservation, despite being deteriorated/damaged. This allows the original function of the element to be preserved, so that it can conserve the same static role it was originally constructed for.

3.2 PRELIMINARY EVALUATION OF THE STATE OF CONSERVATION

The preliminary evaluation of the state of conservation, as indicated in the current code, is an independent activity from the design of the strengthening intervention. It can also be commissioned for merely cognitive reasons, in order to evaluate the static adequacy. The aim of this activity is to define the static behaviour of the building, the state of conservation of the single members and their complete characterisation also from the historical-architectonic perspective. Every preliminary operation that is part of the evaluation of the state of conservation is to be considered fundamental in defining the technical aspects of the intervention.
Placing the structure in its historical context, also through an analysis of the material, generally gives precious indications on the structural type and the constructional techniques adopted. Material characterisation is also needed to determine the mechanical characteristics of the elements, upon which the design of the strengthening is necessarily based.

Characterisation of the type and degree of degradation and the study of its causes can be important, not only to identify the static capacity of the elements to be strengthened but also to identify the conditions under which the strengthening will be carried out. It also identifies the possible agents that may cause deterioration of the strengthening system and the possible causes of long term deterioration of its effectiveness. Diagnosis of the state of the conservation of timber structures should follow UNI 11119.

The evaluation of the actual state of stress in the structure is one of the most important aspects in determining the type of intervention. Even though the specific circumstances to be evaluated contextually to the intervention are described in detail in the following section, it is appropriate to anticipate some general considerations on the evaluation of the existing static conditions.

The structural analysis can be carried out on various levels of detail according to what is required. In order of increasing refinement, these levels pertain to the following schemes:

- global static scheme of the structure, possibly including all the other elements, made of different materials, that could interfere with the timber structure;
- static scheme of the various sub-structures (trusses, floors etc);
- static scheme of each member;
- static scheme of the external and internal restraints (connections).

In the particular case of FRP strengthening, the preliminary evaluation of the state of conservation should also include those aspects that are not directly related to the structural analysis, such as the hiding or the destruction of typical characters of the timber elements, such as particular working signs or superficial marking, fixed joints, etc., which are an integral part of the historical and cultural value of the construction.

It is worth highlighting the importance of an accurate preliminary evaluation of the state of conservation. The extension to the entire structural system of interventions designed from the analysis of a few elements is in this case unacceptable, as it generally leads to serious inadequacies and/or significant over-dimensioning.

3.3 REQUIREMENTS FOR THE STRENGTHENING APPLICABILITY

Accessibility is clearly a fundamental condition for the applicability of the strengthening on the structural element, or even only to carry out most of the preliminary inspection procedures. The ideal situation appears to be the one in which the structure can be secured and the element temporarily removed. This has evident advantages, including the possibility to apply direct and non-destructive investigation methods and to proceed to the strengthening application according to the most appropriate procedures, identified during the evaluation phase. However the success of the intervention is not excluded in cases where it is not possible to temporarily remove the elements that require strengthening, provided that this is taken into account during the design phase. Therefore, accessibility and the practically feasible strengthening procedures need to be evaluated.

In general, given the simplicity of application of most of the techniques used, strengthening can be carried out in presence of any load conditions, with the obvious exception of those interventions that could compromise the structural stability. It is also worth highlighting that any strengthening applied without preliminary unloading of the structure is effective only for further loads and does not produce any release on the existing stresses. On the other hand, complete unloading of the structure can in some cases increase the difficulty of the strengthening procedures, or even make the intervention impossible.

The conditions of the timber, to which the FRP will transfer the stresses, should also be carefully

evaluated, to judge on the applicability of the strengthening. These conditions depend on the volume of timber involved and on its position, in relation to the type of strengthening to be carried out. The volume of timber involved should be made up of material in good conditions, free from bacteria attacks, fungi and any other form of biotic or abiotic attacks. Moreover, the humidity of such volume should be similar to the average humidity of the existing element, in equilibrium with the thermo-hygrometric environmental service conditions, in order to avoid any possible shrinkage or bulking, responsible for anomalous self-balanced stresses in the intervention region.
The transfer of stresses between the FRP and the support should be such to avoid stress concentrations. Suitable devices can be used to involve larger volumes of timber in the stress transfer.

3.4 CHOICE OF THE STRENGTHENING TYPE

Once the preliminary evaluation phase has been carried out and the operating conditions that characterise the intervention established, the choice of the most suitable type of strengthening is a decision-making moment of primary importance in the design of an intervention, be it one of retrofitting or upgrading of the structure. Such evaluation, in general, should take into account several aspects, related to different issues and not only the static and technological ones. In the following, it is assumed for brevity that the compatibility of the strengthening with the conservation of the artistic and historical values of the structure has already been evaluated.

The following factors based on engineering aspects should be taken into account:

- the geometric characteristics of the timber structure to be strengthened;
- the material characteristics;
- the geometric and typological characteristics of the strengthening system;
- the static conditions, before and after the intervention.

Other factors include:

- visibility or invisibility of the strengthening;
- material compatibility;
- the thermo-hygrometric conditions to which the strengthening will be exposed;
- exposure to chemically or physically aggressive environments.

Economic factors and factors connected to the implementation of the strengthening should also be taken into consideration, including:

- evaluation of the intervention time;
- provisional works required;
- availability of qualified staff;
- costs related to the realization of the intervention.

Another important aspect to be considered when designing the strengthening intervention is the evaluation of the possible and practical alternatives. The fact that a particular type of FRP intervention can be carried out, does not necessarily mean that this is the best possible solution. The possibility that the best intervention involves the use of timber only is not to be excluded.
According to a general approach, when strengthening timber structures, it is always important to carefully evaluate the advantages and disadvantages that any form of intervention will entail.

The aforementioned code UNI 11138 requires that the strengthening effectiveness be evaluated both during the design phase and upon completion of the intervention, in accordance to the following times and procedures:

- during the design phase, using traditional verification methodologies;
- during the testing phase, on types of interventions similar to the one to be carried out;
- through the use of previously obtained significant experimental results.

With these premises, this document can define the theoretical basis for FRP strengthening interventions.

3.5 DURABILITY REQUIREMENTS

When considering the durability of the strengthening, it is important to take into account the difference between the strain properties of the existing timber material, of the composites, and of the different types of connectors and/or adhesives to be used. For example, an excessive stiffness of the adhesive, with subsequent inability to sustain the timber strains, especially those due to hygrometric variations, can seriously increase the existing cracking state, even provoking new cracks.

The fundamental requirements listed below should be followed in order to ensure the maximum durability of any type of strengthening intervention.

- <u>Limitation of stresses in service conditions</u>
  During the design phase, the correct evaluation of the static conditions following the strengthening is necessary in order to ensure admissible stress states for all the materials involved.
- <u>Careful choice of the strengthening material</u>
  The physical and mechanical differences among the various types of fibres, resins, and FRPs cannot be ignored. No material can be considered the optimal one. For instance, while carbon fibres are less sensitive to environmental conditions and *creep* phenomena, aramidic fibres are more resistant to impulsive loads. Glass fibres adapt more easily to timber strains, due to their low elastic modulus and high ultimate strain, therefore reducing the risk of debonding phenomena and premature collapse.
- <u>Correct design of the connection between materials</u>
  During both the design and execution phases, priority should be given to the realization of the connections, taking into due account the constitutive behaviour of the materials used.
- <u>Protection of the strengthening intervention</u>
  The careful choice of the materials can ensure the effectiveness of the strengthening also under exceptional conditions. This however, is not sufficient to guarantee an acceptable durability in absence of appropriate protection measures. In fact, chemical-physical or mechanical factors (impacts, vandalism, etc.), and thermal factors (high temperatures, fires) contribute to the degradation of a fibre-reinforced material. Nevertheless, by adopting particular construction details, the characteristics of timber allow an easy protection against these conditions without the need for using other materials.
- <u>Correct execution</u>
  A correct design phase must be followed with precision. The use of FRPs for structural purposes, in particular for strengthening of timber structures, requires specialised staff. The global effectiveness of the intervention could be compromised even if only one phase of the execution is not carried out properly.
- <u>Suitable planning of the environmental conditions</u>
  The presence of adequate microclimatic conditions, for example a good ventilation system, is a necessary requirement for the durability of the intervention. Also the limitation of the

hygrometric excursions of timber favours a good conservation of the strengthening-timber interface.

3.6 EXECUTION RECOMMENDATIONS

Cracking in wood is usually a sign of a well seasoned material and, generally speaking, is admitted by the classification codes for structural uses (UNI 11035-2). The filling of shrinkage cracks should be limited to when it is vitally necessary. The same concept applies to the use of bars glued to the timber to stop further widening of the cracks. The use of such techniques can, in fact, create undesired stress states, by preventing the natural movements of the timber material.

It is appropriate to carry out strengthening interventions that will not induce long term timber degradation, either mechanical or biological. In particular, any type of intervention should allow the timber to exchange humidity with the external environment, thus avoiding undesirable water deposits. It is recommended that:

- the beams ends be ventilated, especially for beams supported on exterior walls, avoiding that they are directly embedded into the wall;
- ventilation be favoured also on the upper parts of the beams, whenever possible, in order to avoid, or at least to obstruct, the condensation and stagnation of water.

4 MATERIALS: CHARACTERISTICS AND CONSTITUTIVE MODELS

4.1 INTRODUCTION

The physical-mechanical characteristics of fibre reinforced composite materials and of their single phases (fibres and polymer matrices) are described in detail in chapter 2 of the CNR-DT 200/2004, which can be referred to for more details.
This chapter describes the properties of solid and glued-laminated timber, highlighting the typical characteristics and the behavioural differences of these materials when compared to other construction materials. The characteristics of adhesives used for wood are also dealt with.

4.2 TECHNOLOGICAL PROPERTIES OF TIMBER

Wood can be classified as a natural fibre-reinforced material, in which the fibres are made of cellulose and the matrix is lignin. The fibres resist to tension, and the lignin to compression.

The cellulose fibres have a good affinity with water and therefore they interact with the environmental humidity, constantly tending to achieve a state of hygroscopic equilibrium with it. This entails a continuous exchange between the water molecules contained in the surrounding environment and those in the wood. The external water amount can be quantified through the “air relative humidity” parameter, while the internal water amount through the “moisture content” parameter, defined as the percentage ratio between the mass of moisture contained in the wood and its mass at the anhydrous state. The aforementioned process is innate to the physical-chemical nature of the wood and occurs during the whole life of a timber element.
This has two important mechanical consequences, both connected to the moisture content of the timber: the variation in size (swelling and shrinkage); the variation in the physical-mechanical characteristics (volumic mass, resistance, instantaneous and long-term deformability).

Size variations are usually not negligible and depend on both the type of wood species and the considered anatomical direction (radial, tangential, longitudinal). Figure 4-1 shows an example of these variations.

Figure 4-1 highlights how the size variation process is circumscribed to the so called “hygroscopic interval”, that ranges from the anhydrous state (moisture content equal to 0%) to the saturation point (moisture content about 30%, the shaded area in Figure 4-1). Considering that the structural timber has an equilibrium moisture content usually within the range of 10% - 18%, and that the total volumetric variation within the entire hygroscopic field can vary, according to the wood species, from 6% up to more than 16%, it is evident that the entity of the size variations due to the humidity variations, even if modest, cannot be neglected from a technical perspective.
The dependence on the humidity of the other mechanical characteristics of the timber (strength, modulus of elasticity) is generally significant. These characteristics attain their maximum values in the anhydrous state, while they tend to decrease towards the saturation point. Beyond this point, they are more or less constant (Figure 4-2)

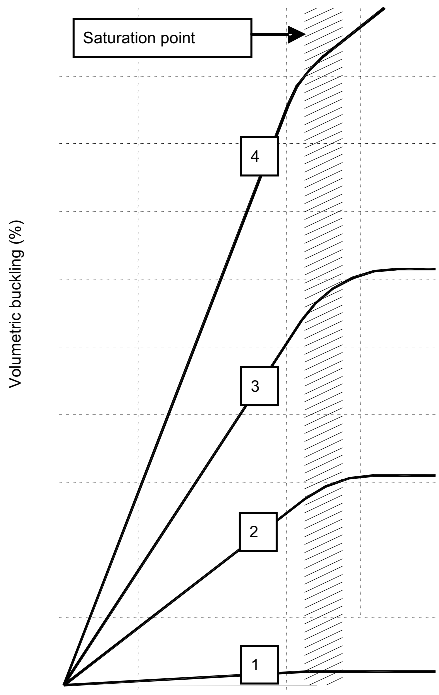


Figure 4-1 – Swelling phenomena: volumetric (4), longitudinal (1), radial (2) and tangential (3), versus wood moisture content
(a similar diagram can be obtained for the symmetric shrinkage phenomenon).

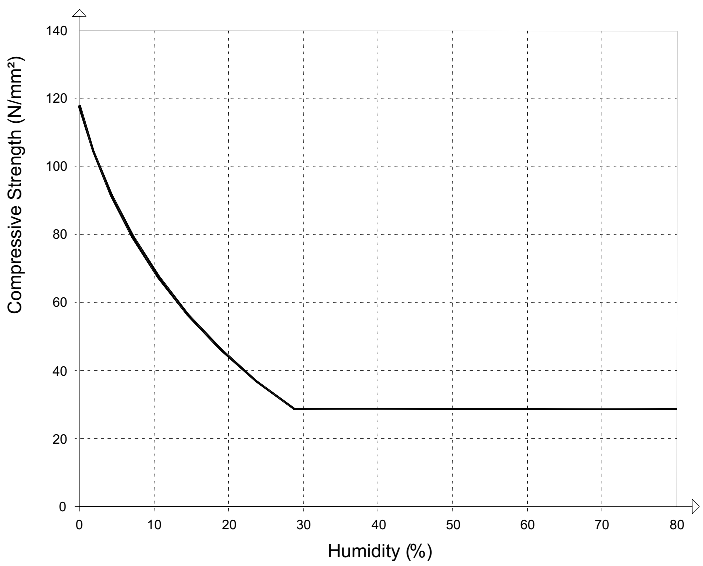


Figure 4-2 – Compressive strength parallel to the fibres versus wood moisture content (conifer).

The mechanical properties of timber, under the same moisture content, vary depending also on other factors, including:

- the volumic mass;
- the presence of defects or alterations (knots, ring shakes etc);
- the fibre inclination in relation to the axis of the structural member.

Figure 4-3 shows an example of the trend of several mechanical characteristics of wood with the variation of the inclination angle of the fibres.

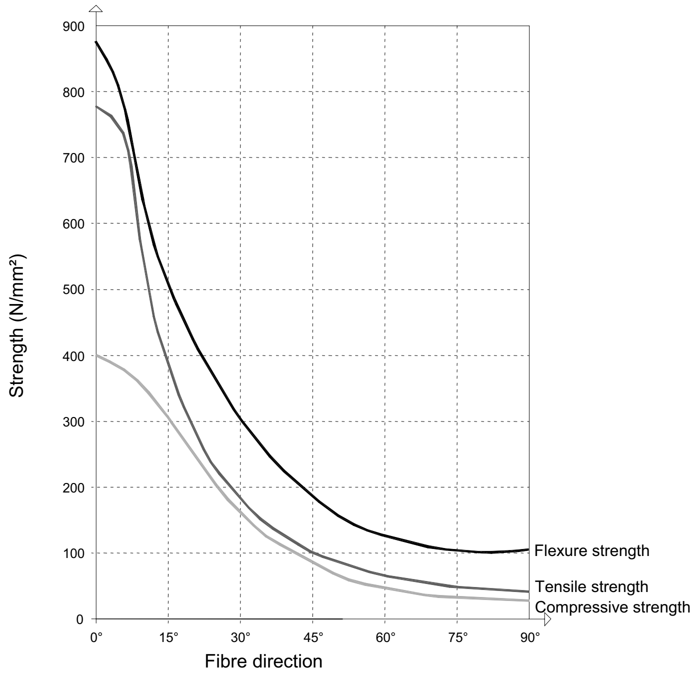


Figure 4-3 – Example of the relationship between the main mechanical strength characteristics and the fibre inclination.

The number and complexity of the factors have suggested, in the structural engineering of the timber constructions, the adoption of an approach based on the combined use of the following three measures:

- classification of the timber elements according to resistance criteria;
- establishment of characteristic values for the main mechanical properties of the classified elements (resistance class, characteristic performance profiles);
- design of the elements based on calculation rules especially conceived to use the aforementioned characteristic values.

These measures, within the field of new constructions, are currently being used in the European context as follows (semi-probabilistic Limit States Method):

- rules for classification of structural timber according to the codes EN 14081, UNI EN 518 and UNI EN 519;
- profiles of characteristic values determined according to the code UNI EN 338 (resistance class), and to the codes UNI EN 408 (test methods) and UNI EN 384 (determination of the characteristic values);
- profiles of characteristics values determined according to the codes UNI 11035-1 and UNI 11035-2 for Italian timber;
- calculation rules determined according to the code EN 1995 (Eurocode 5).

For further details on strengthening existing structures, see UNI 11138 and 11119.

4.3 CONSTITUTIVE CHARACTERISTICS AND MODELS OF SOLID AND GLUED-LAMINATED TIMBER

Generally, the constitutive behaviour of timber can be described through various models characterised by a different degree of approximation (Table 4.1). The assumptions common to all the models are listed hereafter.

- Behaviour in tension
  The most commonly accepted model for the uni-axial behaviour in tension is the elastic-linear up to failure model, characterised by a rectilinear diagram with a constant slope (arctg E_w). Once the ultimate strength is given, it is therefore immediate to obtain the corresponding strain and vice-versa.
- Behaviour in compression
  The typical stress-strain diagram derived by a uni-axial compression test usually has a initial branch with a constant slope (sometimes continuously variable, O'Halloran model). The tangent at the origin for negative strains (compression) has the same slope of that relative to positive strains (tension).
- Modulus of elasticity
  This modulus is simple to be determined, as it represents the slope of the linear range of the stress-strain diagrams in all the considered models (Table 4.1).

In relation to the plastic behaviour in compression, the three constitutive models presented in Table 4.1 deserve specific considerations.

It can be quantitatively stated that the tensile strength, evaluated by tests on clear wood specimens, has average values of 80-100 MPa, approximately the double of the average values in compression, which are estimated as 40-50 MPa. This notable difference is no longer valid for structural timber, for which statistical considerations on the presence and incidence of defects lower the values above to 10-40 MPa for the tensile strength and 25-40 MPa for the compressive strength. It is worth noting how the presence of serious defects can influence the tensile strength of the material more than the compressive strength, so that the current codes consider the two values to be very close.

The analysis of the behaviour up to collapse of a timber element under bending shows that the mode of failure essentially depends on:

- the ratio between the values of the tensile and compressive strengths;
- the non-linearity of the behaviour at the ultimate limit state of timber in compression;
- the amount of material stressed in tension, a parameter that is directly proportional to the probability of involving local defects able to trigger a premature failure.

It is also worth noting that, at present, current codes consider the bending resistance as an independent characteristic, specific to the resistance class, but not directly deducible from the tensile nor from the compressive characteristic strengths.

Table 4-2 shows the main collapse modes of timber under uniaxial bending, where η indicates the ratio between the tensile and compressive strengths, f_{tu}/f_{cu}, of the material used. In the technical practice, the modes of failure here described have different probabilities of occurring.

It can be stated that, for solid wood, the most common type of failure occurs when the limit value of stress or, to be more precise, the ultimate value of the fibre strain, is reached in the tension zone, most of the times in the presence of a plasticization of the compression zone. The corresponding collapse modes are, then, those labelled by numbers 2 and 3 in Table 4-2. On the contrary, the mode of collapse 1 is relatively rare and indicative of particularly defective materials. Furthermore this is considered the least desirable mode due to its brittle characteristics and to the modest mobilised strains. Finally, collapse mode number 4, unusual in elements for structural use, but typical in green timber, i.e. timber with a moisture content higher than that relative to the point of saturation (§**4.2**), would be preferable, because it is characterized, under the same conditions, by large strains and a progressive loss of the load-bearing capacity.

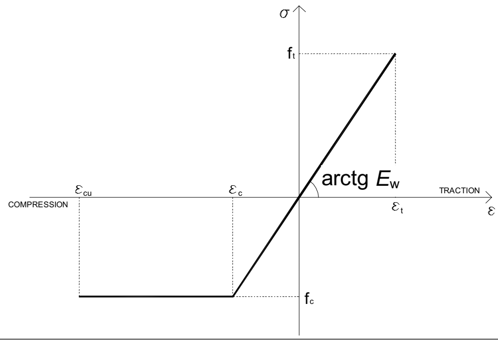
Table 4-1 - Constitutive models for solid timber and glued-laminated timber

1. Elastic-plastic model

Analytical Definition	*Diagram*

$$\sigma = E_w \cdot \varepsilon \quad \text{per} \quad \varepsilon_c < \varepsilon < \varepsilon_t$$
$$\sigma = f_c \quad \text{per} \quad \varepsilon < \varepsilon_c$$

The model is very simple, limiting the diagram to a triangle-rectangle. Despite the simplification, the model is widely used.

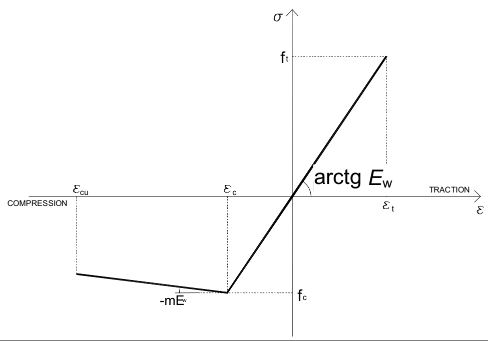


2. Bazan Model

Analytical Definition	*Diagram*

$$\sigma = E_w \cdot \varepsilon \quad \text{per} \quad \varepsilon_c < \varepsilon < \varepsilon_t$$
$$\sigma = f_c + m \cdot E_w \cdot (\varepsilon - \varepsilon_c) \quad \text{per} \quad \varepsilon < \varepsilon_c$$

This model is more complete than the previous one, without losing the advantages of simplicity related to linearity. A difficulty is represented by the definition of the coefficient m<0 that characterises the slope of the plastic branch with negative hardening (*softening* behaviour).



3. O'Halloran Model

Analytical Definition	*Diagram*

$$\sigma = E_w \cdot \varepsilon \quad \text{per} \quad 0 < \varepsilon < \varepsilon_t$$
$$\sigma = A \cdot |\varepsilon|^n + E_w \cdot \varepsilon \quad \text{per} \quad \varepsilon_{cu} < \varepsilon < 0$$

This model, among those reported herein, gives the best description of the actual behaviour of timber. Its definition requires the specification of two parameters A>0 and n>0.

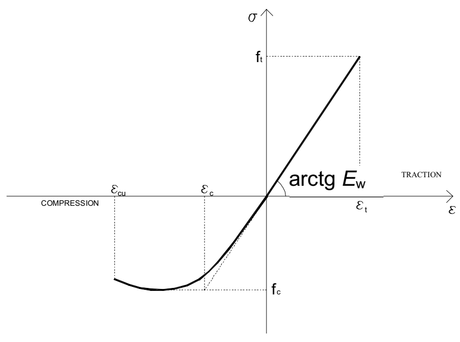
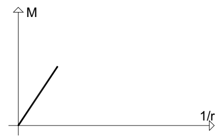
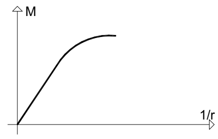
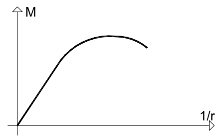
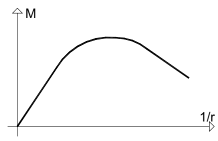


Table 4-2 – The main collapse modes of timber under uniaxial bending.

	Description of the collapse	Condition	Diagram
1	Brittle failure in the tension zone with a linear relationship between bending moment and curvature up to failure.	$\eta < 1$	
2	Failure in the tension zone following plasticization in the compression zone and downward shifting of the neutral axis. The bending moment-curvature relationship differs from the linear trend.	$\eta \geq 1$	
3	Failure in the tension zone but with larger ductility due to plasticization of the cross section. The bending moment-curvature diagram shows a slight descending trend (*softening*).	$\eta > 1$	
4	Failure in the compression zone with large ductility due to the considerable plasticization of the cross-section. The bending moment-curvature diagram shows an evident descending trend.	$\eta >> 1$	

Since the modes of failure for glued-laminated timber are directly connected to the quality of the wood lamellae, the accuracy in the choice of the lamellae and the control of the various manufacturing phases make its behaviour qualitatively different from that of solid timber. The bending tests up to failure highlight how difficult it is to attain the plasticization state at the edge in compression. Glued-laminated timber therefore has a more marked tendency for brittle failure than solid timber, with the failure mode being 1 or 2 rather than 3 (Table 4-2).

The main objective of the design of structural elements made of FRP strengthened timber is to couple the ductile behaviour, which can be improved by "forcing" the η ratio onto higher values, with a higher ultimate strength, achievable by reducing the influence of the defects on the global behaviour of the element. This result can be obtained, at least for elements in bending, by interventions aimed at improving the behaviour at the tension edge through the attainment of a higher tensile strength. The η ratio is therefore artificially increased, so to allow a plasticization (even if only modest) at the compression edge. Such circumstance allows at the same time to achieve a more ductile collapse and to take full advantage of the materials that constitute the cross section. The collapse mode is shifted, also in the case of the glued-laminated timber, towards the type 3 indicated in Table 4-2.

4.4 MAIN CHARACTERISTICS OF ADHESIVES FOR TIMBER

The adhesives used for the on-site bonding of timber are essentially epoxy based. They have the same chemical base as most of the products used for the polymeric matrix of FRPs, and they are described in greater detail in chapter 2 of CNR-DT 200/2004.

This chapter deals with the description of the problems related to the use of structural wood adhesives, some of them being discussed in further detail in the following chapter.

Adhesives have a particular role in transferring stresses between wood and FRPs, which are materials with a different visco-elastic behaviour. It is fundamental for structural wood adhesives that they be specifically formulated and tested with suitable methodologies, able to evidence their good adhesion (in terms of shear strength) and compatibility with different wood species and

moreover their durability towards thermo-hygrometric cycles coherent with the assigned service class. The characterisation methods used for this purpose will be described below.
Table 4-3 shows an example of a technical data sheet for three different products used in bonding timber structures. These are a tixothropic adhesive, a fluid adhesive (ideal for bonding conifer timber) and a *primer* to be used with the aforementioned adhesives in bonding more solid/dense species (chestnut and oak).

Table 4-3 – Example of technical datasheet for different types of adhesives/*primer*.

	Tixothropic Adhesive	Fluid Adhesive	*Primer* for solid wood
Component	A B	A B	A B
Conservation	24 months in the original closed packaging at a temperature ranging between +5°C and +30°C		
Mixing Ratio	A : B = 2 : 1	A : B = 4 : 1	A : B = 1 : 1
Specific weight of the mixture	1,59 g/cm^3	1,01 g/cm^3	1,08 g/cm^3
Brookfield viscosity of the mixture	220 Pa.s (Helipath F - revs 5)	11.400 cPs (Rotor 3 – revs 5)	9.400-9.600 cPs (Rotor 5 - revs10)
Pot life (a +10 °C) (a +23 °C) (a +30 °C) (a +30 °C):	 150 mins 60 mins 30 mins	 60 mins 40 mins 20 mins	 / 30-40 mins /
Cure time (a +10 °C) (a +23 °C) (a +30 °C)	 14-16 hrs 4-5 hrs 2 h 30' - 3 h	 90 mins 50 mins 30 mins	 / 4 - 5 hrs /
Application Temperature	from +10 °C to +35 °C	from +10 °C to +30 °C	from +10 °C to +30 °C
Complete Hardening	7 days	7 days	12-24 hrs
Adhesion: compressive shear strength (fir)[1]	9 N/mm^2 (after 7 days at +23 °C)	9 N/mm^2 (after 7 days at +23 °C)	not applicable
Tensile strength (ASTM D 638)	18 N/mm^2	30 N/mm^2	not applicable
Tensile Elongation (ASTM D 638)	1%	1,2%	not applicable
Bending Resistance (ISO 178)	30 N/mm^2	60 N/mm^2	not applicable
Elastic Modulus in bending (ISO 178)	4.000 N/mm^2	2.000 N/mm^2	not applicable
Compressive strength (ASTM D 695)	45 N/mm^2	70 N/mm^2	not applicable
Elastic Modulus in compression (ASTM D 695)	3.000 N/mm^2	5.000 N/mm^2	not applicable

Apart from the mechanical aspects, the fracture surfaces of bonded joints can give important indications during the evaluation phase of adhesives. Three main failure modes can be defined for bonded joints:

- Cohesive failure
  This failure occurs inside one of the two materials that are jointed. It is a type of failure observed in "ideal" conditions (cohesive wood failure). It can also occur within the adhesive (cohesive failure of the adhesive), due to a specific design choice or to a cohesion defect (mixture defects, excessive presence of inerts).

- Adhesive failure
  This failure occurs at the interface between the adhesive and the adherend when the adhesion

forces are less than the cohesive ones of the adherends. The fracture surfaces are usually smooth. This type of failure evidences either an incorrect application of the adhesive (e.g. pot life not respected) or inadequate preparation of the surfaces (dust, grease etc).

- <u>Mixed failure</u>
  This failure is partially cohesive and partially adhesive. A cohesive failure is usually observed in wood, whereas an adhesive one in the immediate proximity.

Adhesives can be considered to be mechanically isotropic. They are generally thermosetting materials with a visco-elastic behaviour. Their strain-stress constitutive model (**Figure 4-4**) depends on the chemistry of the adhesive and on its final formulation (content and distribution of inerts etc). The glass transition temperature, T_g, above which most of the mechanical properties (in particular the modulus of elasticity) decrease dramatically, is often below 100°C.

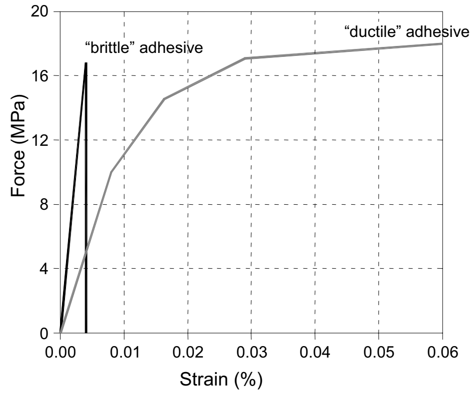


Figure 4-4 –Stress-strain diagram of an adhesive.

As evidenced by the values of the modulus of elasticity in **Table 4-3**, the rigidity of epoxy adhesives is generally lower than that of wood in the longitudinal direction. On the other hand, it is similar to the rigidity in the direction transverse to the fibres. Hence, adhesives can perform differently, both from a qualitative and quantitative standpoint, depending on the volume and the characteristics of the product used for the joint.

5 BONDING BETWEEN TIMBER AND FRP

5.1 INTRODUCTION

There are many factors determining the efficiency of a bonded joint. They depend on the adhesive used, the design of the joint and other aspects that take into account the characteristics of the constituent materials: wood and FRP.
The first part of this chapter deals with specific problems related to the wood-adhesive interface, including the criteria for the selection of products and the preparation of the surface to be bonded. The mechanical behaviour of the joints is discussed in the second part, taking into account the transmission of internal shear forces and the specific problems related to bonding with adhesives.

5.2 ADHESIVE BONDING

5.2.1 Foreword

The efficiency of bonding with adhesives is due to various factors. Assuming that the joint has been correctly designed and realized, they can be summarised as follows:

- <u>Wetting ability of the adhesive in relation to the surface</u>
  It depends on the substrate (working of the surface, type of surface treatment, "ageing" and chemical modification of the surface in general, porosity, etc.), and on the adhesive (viscosity, density, chemical affinity with the substrate, etc.).
- <u>Bulk properties of adhesive after complete hardening</u>
  These are most important when the thickness of the bond line is appreciable, that is frequent in on-site interventions and in general when no pressure is applied to the parts to be bonded. The cohesive properties of the adhesive are in fact crucial in order for the whole bonded surface to participates in the transmission of the stresses through the interface. To such purpose, the maximum thickness of the bondline should be as indicated by the manufacturer.
- <u>Particularly severe environmental conditions</u>
  These can include elevated thermal distortions, such as those produced by fires and repeated hygrometric variations, inducing deformations in the timber that are not subsequently followed by similar variations in the adhesive volume, thus leading to additional coactive stress at the interface between the adhesive and the wood.

In order to maintain the reliability of a bonded joint during its service life, it is therefore important to undertake appropriate precautions, with the conditions of the bonded surface being adequate, and moreover to make use adhesives specific for structural applications on wood.

In relation to the first aspect, it is important to remember that any kind of surface treatment should obtain a clean surface, i.e. not contaminated by either extraneous particles (dust) or waste (grease, oil, etc.). Moreover, any previous treatment (such as impregnation, use of additives, etc.) should be removed or its chemical compatibility with the adhesive should be checked. Common treatments include decontamination with solvents and mechanical preparation of the surfaces to be bonded (abrasion by sandblasting, planing, sandpapering, etc.), in order to increase their roughness. However, it is important:

- to avoid localized temperature increase (formation of burns due to the use of unsuitable or worn-out tools), considering that high temperatures can provoke the deactivation of bonding;

- to remove shavings and dust, preferably by means of suction rather than blowing;
- to completely clean the surfaces from any traces of lubricants caused by the dragging of cutting tools (such as chains of saws) by chipping or milling.

In some cases, in particular for high density wood species, it can be useful to apply to the surface a *primer* with a chemical formulation similar to that of the adhesive.

It is important that any surface treatment be carried out immediately prior to the application of the reinforcement, in order to avoid any contamination by the dust and the dirt that may be present on the construction site.

The aspects related to the suitability of an adhesive for structural use on wood should be aimed at identifying the relationship between the bulk properties of the product and those of the wood substrate, which however should be in a good, or at least reliable, state of preservation. This relationship is usually achieved by measuring the shear strength at the interface between wood and adhesive, with joints in normal moisture conditions. However, for the complete evaluation of a structural adhesive, it is also important to verify the effects related to the predictable hygrometric distortions that can occur during the service life of a joint.

5.2.2 Existing standards

For the applications on wood it is useful to classify adhesives based on specific laboratory tests.

The Italian (and European) standard dealing with the evaluation of structural wood adhesives is UNI EN 302-1, which includes a description of a shear test by tension. Other international standards for the evaluation of the shear characteristics of the adhesives also exist (for example the ASTM D3165), even if they do not specifically refer to wood. Other standards, such as the ASTM D905, the ASTM D3931 and the ISO 6238 are based on shear tests by compression. It is worth noting that all of them refer to thin joints and/or realized by application of pressure.

The latter conditions cannot be taken into account when dealing with FRP and in the structural on-site bonding of timber, where the thickness of the adhesive is greater than one millimetre and bonding is not carried out by applying pressure. It is therefore essential to use a shear strength evaluation technique that is specific of “thick joints”.

The reference standard for classifying structural wood adhesives is UNI EN 301. It deals with two classes, the first for exterior use and the second for interior use, described in detail as follows.

- <u>Type I</u>
  Suitable for exposure to all types of weather, to climatic conditions with more than 85% relative humidity, and service temperatures higher than 50°C. In the case of FRP, such conditions could exist for exterior structures or for interior members having the ends supported by exterior masonry walls.

- <u>Type II</u>
  Suitable for interior applications and covered exteriors, with service temperatures lower than 50°C. In the case of FRP, such conditions could exist for structures for interiors or for exterior elements with heads supported by interior masonry walls.

From a qualification perspective, the main difference between the two types of adhesive is that tests for the first type should include a series of thermo-hygrometric cycles at high temperature, while for the second one cycles are carried out at room temperature.

The EN 1995-1-1 (Eurocode 5) defines three service classes for timber structures (**Table 5-1**) depending on the hygrometric service conditions. The same code recommends the use of Type I adhesives for all the service classes and Type II only for classes 1 and 2, when exposures to temperatures higher than 50°C are not prolonged.

Table 5-1 – Service classes defined by the Eurocode 5 and examples of application.

Service classes	**1**	**2**	**3**
Description	Climatic conditions of 20°C, and relative humidity higher than 65% only for a few weeks per year	Climatic conditions of 20°C, and relative humidity not higher than 85% except for a few weeks per year	Climatic conditions worse than for Class 2
Average moisture content in timber	about 12%	always < 18%	also > 18%
Typical examples of timber structures	- interiors - warmed and conditioned environments, with limited hygrothermal variations	- covered exteriors - not conditioned environments (shelters, cold roofs, terraces) or humid ones (swimming pools) - beam ends on interior walls, well ventilated and drained	- exteriors - bridges, columns, piles - beam ends on exterior walls, also for heated environments

5.2.3 Compatibility between adhesives and timber

The structural products usable on timber and applied on site should be evaluated on the basis of the shear method with "thick joint" specimens, through the definition of a coefficient of mechanical compatibility between adhesive and wood, $k_{a,w}$. The specimens (Figure 5-1), prepared by bonding the radial faces of two planks of timber, are tested up to collapse under the action of a shear force applied parallel to the fibres.

The specimens are subdivided in two groups. Those of the first group, G_{con}, are kept in a *standard* environment (T = 20 °C, φ = 65% relative humidity) up to complete hardening of the adhesive. They represent the bonding of timber in standard conditions. The second group, G_{inv}, is subjected to several cycles, which include the repeated alternating of phases of complete soaking (maximum increment of volume) and quick drying at a temperature close to the glass transition temperature, T_g, of the adhesive. The tests in complete soaking conditions identify the minimum mechanical performance of the joints representing cases where the timber is accidentally subjected to unusual working conditions.
Both groups of specimens are then tested again (after the determination of the shear strength of the joint), with a further series of tests carried out on one of the two adherends, in order to evaluate the shear strength of solid wood (the same as used for preparing the joint). This allows for the direct comparison of the performance of the adhesive related to that of wood, on the same specimen, both under normal and wet conditions.

By following the aforementioned procedure it is possible to accurately test the products, therefore accurately and reliably identifying their differences in performance.
Tests carried out on several commercially available products for on site interventions on timber have evidenced substantial differences among them, with some having a marked tendency to delamination, and others not generating any premature ruptures. These latter specimens have generally shown strength values comparable to those of solid wood, even when subjected to repeated hygrothermal cycles.
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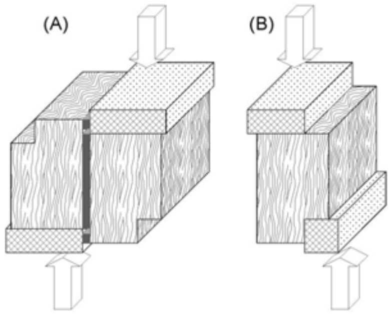


Figure 5-1 – Specimen fo tests on thick joints.

The coefficient of mechanical compatibility between adhesive and wood, $k_{a,w}$, can be expressed through the following relationship:

$$k_{\mathrm{a,w}} = \eta_{\mathrm{dry}} \cdot \eta_{\mathrm{wet}}. \tag{1.1}$$

In (1.2) :

- η_{dry} is the comparison coefficient in a *standard* environment, i.e. the ratio of the mechanical shear performances of adhesive and wood, both relative to specimens conditioned in a climatic chamber (T = 20 °C,: φ = 65% relative humidity);
- η_{wet} is the analogous coefficient which relates the mechanical shear performances of adhesive and wood, both measured at the wet state after specimens have been subjected to thermohygrometric cycles.

The coefficients η_{dry} and η_{wet} are defined by the following relationships, respectively:

$$\eta_{\mathrm{dry}} = \frac{\tau_{\mathrm{G,std}}}{\tau_{\mathrm{L,std}}}, \tag{1.3}$$

$$\eta_{\mathrm{wet}} = \frac{\tau_{\mathrm{G,inv}}}{\tau_{\mathrm{L,inv}}}, \tag{1.4}$$

in which:
- $\tau_{L,std}$ and $\tau_{G,std}$ are the shear strengths of the solid wood and of the adhesively bonded specimens, respectively, in a normal environment;
- $\tau_{L,inv}$ and $\tau_{G,inv}$ are the shear strengths of the solid wood and of the adhesively bonded specimens, respectively, after the exposure to thermo-hygrometric cycles.

The coefficient of mechanical compatibility, $k_{a,w}$, includes contemporarily, in the form of a ratio, the results obtained for the wood and for the bonded joint, thus giving a quantitative measurement of the performance of the adhesive in relation to that of the same wood upon which it is applied.

The coefficient $k_{a,w}$ can be used when comparing different adhesives, but it can also be used for classification purposes by using an approach similar to that of UNI EN 301, and making direct reference to the Service Classes provided by the Eurocode 5. For example, a Type I adhesive (according to the Eurocode 5 to be used in Service Class 3) should be characterised by the coefficients: $\eta_{dry} \geq 1$ and $\eta_{wet} \geq 0{,}8$.

It is also possible to obtain the corresponding values for the other two Service Classes. Even if Service Class 3 considers full exposure to the weather, such service conditions should be carefully examined in cases of FRP/timber joints. It is highly recommended to provide an adequate protection against the direct climatic action.

5.2.4 Mechanical behaviour and failure modes

When evaluating the mechanical behaviour of a bonded joint, it is worth considering the modes of transmission of shear stresses and the modes of failure. The qualitative shapes of the typical load-displacement diagrams in a shear test indicate that the behaviour of the solid wood and of the bonded joint are qualitatively very similar to each other. When they are beyond a plastic phase, which could be more or less extended depending on the reached load values or the adhesive type, a sudden collapse of the joints occurs.

In reference to the debonding failure modes, previously described in chapter 4, an adhesive failure is an indication of poor bonding. In order to consider bonding successfully executed, the failed surface must show a significant number of fibres still attached to the adhesive layer (cohesive failure in the wood). Failure occurring within the adhesive layer evidences a good adhesion between the substrate and the adhesive. However, this type of failure needs to be carefully considered, because it can be due to poor characteristics of the adhesive, with a possible risk of debonding resulting from the differential movements between the substrate and the adhesive.

5.3 USE OF MECHANICAL CONNECTORS

In addition to bonding, FRP can be applied to timber using either traditional mechanical devices or mixed type connections.

The main problem of mechanical connections is represented by the internal structure of the composites. In fact, due to their nature, they present very low strengths for concentrated loads and forces inclined with respect to the fibre direction. Both the aforementioned circumstances occur when using mechanical connectors. Therefore, their best use is in combination with bonded connections in order to:

- improve the Ultimate Limit State behaviour of the bonded connection;
- "sew" the connection, by absorbing the tensile stresses perpendicular to the bonding plane.

With reference to the first item, several aspects need to be specified. As described in EN 1995-1-1 and highlighted by Figure 5-2, different types of connection have different behaviour under service conditions and therefore it is recommended not to use them in the same joint. Furthermore, if the stiffness is very different, such as between adhesives and mechanical connectors, the two types of connection cannot be considered to be acting together.

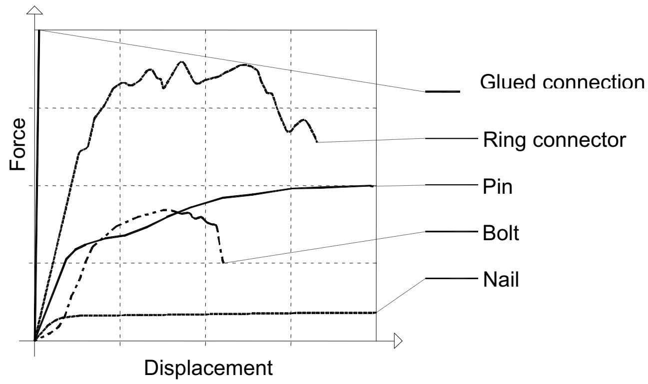


Figure 5-2 – Load-slip tests in timber-timber connections.

Adhesive connections, as long as the adhesive is effective, give a stiff joint because they can limit the relative movements between the two surfaces in contact. This is no longer the case when the strengthened element reaches its ultimate limit state. When the adhesive fails, the presence of mechanical connectors could result decisive for the stress transmission from the timber element to the reinforcement, conferring ductile characteristics to the collapse mode. The design of connectors should therefore be carried out independently from the presence of bonding, by assigning them the full design stresses.

Unlike timber, for which the introduction of mechanical connectors does not create problems different from those already dealt with in the available guidelines, this type of connection for FRP could induce fibre interruptions and the arising of localized stress states. If necessary, the latter should be verified by specific tests and its action should be related to the strength characteristics of the FRP used. While plates and thin sheets of FRP need a uniform stress distribution, prerogative of bonded connections, the effectiveness of the connection of thick sheets or pultruded elements could be improved by using mixed type connections.

Substantially different situations could arise in the case of connections for the "sewing" of joints, especially in the strengthening interventions where the stress state transferred among the elements involves several components. The example of an element loaded in bending and strengthened with a sheet having not negligible bending stiffness, represented in Figure 5-3, is considered. The reinforcement contrasts the flexural deformation of the beam, and therefore at its ends tensile stresses arise perpendicularly to the adhesive layer. Their values could reach levels sufficient to cause the premature debonding of the sheet.
Given the inability of the adhesive to absorb such stresses, it is therefore important to adopt other elements able to transfer the same stresses to the timber without involving the bonding surface. These elements include the mechanical connectors, which should obviously be designed by their pullout resistance and can be metallic screws and nails (with improved bond) or FRP nails injected with resins.

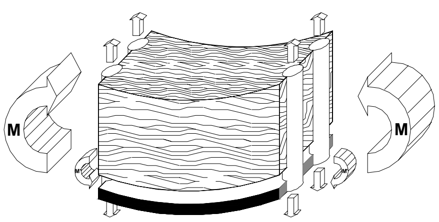
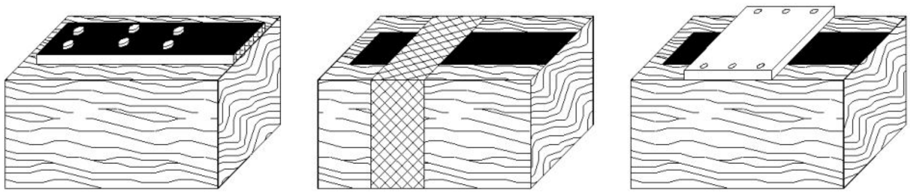


Figure 5-3 – Transverse sewing of bonded connections.

The above can be considered valid for any type of reinforcement applied externally to the examined member, with the use of this technique requiring appropriate calculation models. The sealing of the connection should not be considered always necessary when designing a strengthening intervention, however its use makes the calculation model closer to the actual conditions, leaving to the adhesive layer the only function to transfer the shear forces.
Under no circumstance should the presence of the connectors influence the calculation of the resistance to debonding of the bonded connection.
There are several sewing types of connections; some of them, particularly common, are shown in Figure 5-4.



(a) (b) (c)

Figure 5-4 – Sewing types of connections: (a) with connectors; (b) wrapping with FRP bands; (c) connectors applied to secondary elements.

Although the stress analysis in the mixed joints has not yet been fully understood, the possibility to check the effectiveness of a strengthening system during an experimental simulation, as set out in UNI 11138, opens the way to their use for improving the ultimate behaviour of a structure.

6 STRENGTHENING IN BENDING, AND IN COMBINED BENDING AND AXIAL FORCE

6.1 INTRODUCTION

The strengthening of timber beams can be necessary for several reasons including:

- the increase of live loads, possible in historical buildings for changes of destination of use or upgrading to meet new requirements;
- the reduction of the resistant cross sections following deterioration (attacks of biological agents such as fungi, insects, etc.) or traumatic events (i.e. fires);
- the excess of deflection in the members.

Strengthening techniques that use FRPs allow the existing structure not to be dismantled, therefore aiding in preserving the built historical heritage, with a consequential reduction in costs and time.

Even though timber is generally an anisotropic material, its behaviour can be described as orthotropic in three directions: longitudinal, tangential and radial. The constitutive law is commonly considered to be elastic-perfectly plastic (model 1, **Table 4-1**).

In a cross section in bending, the different behaviour of the material under tension or compression, once the plasticization process of the compression zone has started, determines a progressive shift of the neutral axis towards the tension zone (**Figure 6-1**). This in turn determines a reduction of the size of the tension zone and an increase of the tensile stresses.
Strengthening in the tension zone with FRP materials, characterized by considerable strength and stiffness in tension, yields an increase in the load bearing capacity of the timber members under bending moment, and an increase in their ductility, as highlighted in **Table 4-2**.
Needless to say that the essential requirement for the effectiveness of the strengthening is that the transfer of stresses from the timber to the composite material is guaranteed.

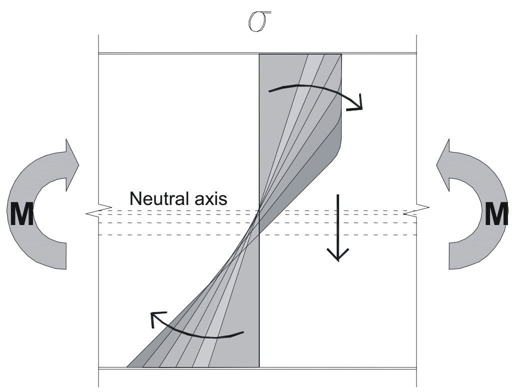


Figure 6-1 – Shifting of the neutral axis in the plasticization phase.

In relation to the ULS, the reinforcement in the tension zone by means of composite materials appears to be particularly effective when:

- the timber has an elastic-plastic behaviour in compression;
- the ultimate tensile strain is less than the compressive one.

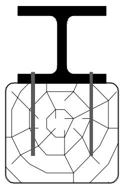
An elastic-plastic behaviour under compression does not occur in situations where a premature failure of timber occurs within the elastic range. This is frequent in the case of low grade timber, with several and large defects, including knots and significant fibre inclinations. These defects, especially in the tension zone, represent the weak points from which failure could be triggered.

Reference is subsequently made to combined bending and axial force, including pure bending and pure axial force as particular cases.

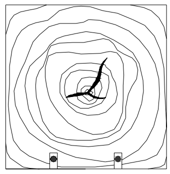
6.2 TECHNIQUES FOR FLEXURAL STRENGTHENING

In principle, the FRP can be applied to the tension zone, the compression zone, or both.

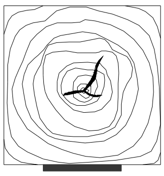
Strengthening in the tension zone can be preferably realized by using FRP bars and plates (Figure 6-2) bonded to the external surface of the beam (possibly with the addition of mechanical connectors) or inside special slots cut into the beam.



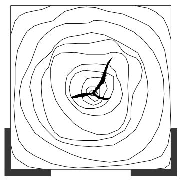
a) Application of pultruded profiles in the compression zone connected by mechanical devices



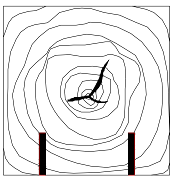
b) Application of bars in the tension zone



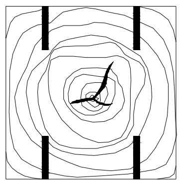
c) Application of external plates in the tension zone



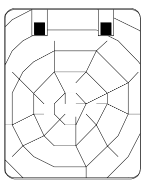
d) Application of plates in the tension zone



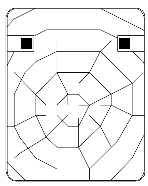
e) Application of internal plates in the tension zone



f) Application of internal plates in the tension and compression zones



g) Application of bars in the compression zone



h) Application of bars in the compression zone

Figure 6-2 – Possible applications of FRP strengthening.

In the case of lateral slots (Figure 6-2, examples g), h), the reduction of the chords for calculation of the tangential stresses should be taken into account. Strengthening in the compression zone can be made by applying bars, plates and pultruded profiles. In the case of elements not appropriately confined within the cross section, the connection of the strengthening with the timber substrate should be guaranteed also through the use of mechanical devices (nails or screws). In the absence of such devices, in accordance with CNR-DT 200/2004, the use of FRP plates externally applied to the compression zone is not recommended.

The insertion of the reinforcement in slots inside the cross section doubles the bonding surface. Debonding at the ends of the beam is prevented when the strengthening is completely inserted into the transverse cross section, and hence is confined by the section.

The external application of strengthening with FRP plates or sheets can prevent the propagation of possible pre-existing cracks starting from defects (knots, excessive fibre inclination), which could cause a considerable decrease of the load bearing capacity of the timber beams.

6.3 COLLAPSE MODES BY DELAMINATION OF ELEMENTS STRENGTHENED IN BENDING

As follows, the collapse modes by delamination of elements strengthened in bending are described qualitatively. There are no theoretical and/or experimental studies currently available to establish reliable guidelines for the design and/or the verification of the mentioned elements against delamination.
The collapse modes by delamination of timber elements strengthened with FRP in the tension zone include:

- Delamination caused by local failures in the timber
  The possible presence of knots or other defects, such as large shrinkage cracks, mainly in the tension zone, could determine the collapse by delamination. It is more likely that collapse starts from a knot within the timber when the shear force prevails over the bending moment. This collapse mode has been observed in the laboratory, especially during three- or four-point bending tests, in which shear represents the dominant internal force; instead, it is uncommon in cases of distributed loading.
  In the case of strengthening with plates, the collapse can occur due to:
  - delamination at the element ends, caused by the concentration of shear stresses in the anchorage zones (Figure 6-3);
  - delamination in intermediate zones of the timber beam, caused by cracks or strong wood fibre inclination with respect to the beam longitudinal direction.

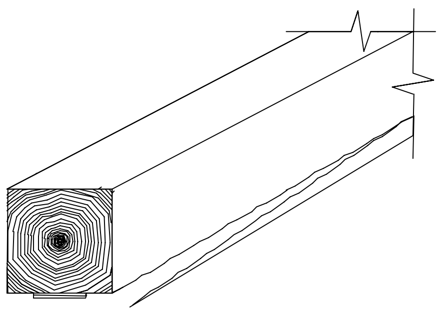


Figure 6-3 – Delamination at the ends with the detachment of timber parts.

- Delamination caused by irregularities in the timber surface
  The non planarity of the bonding surface induces a state of tensile stress due to the curvature, which can cause the detachment of the reinforcement. This could be prevented by lightly planing the support, therefore eliminating any irregularities. Obviously, it is necessary to remove any incoherent materials produced during planing.
  In the case of strengthening with plates, collapse can occur due to the following modes:

- rupture in tension of the timber material due to defects, knots, and excessive inclination of the tension fibres in the zones of maximum bending moment;
- detachment of portions of timber due to the slots cut in the element.

In the case of reinforcement in the compression zone, the collapse can occur due to the following modes:

1. all flexural collapse modes of the unstrengthened timber, when the collapse is due to failure in the tension zone;
2. buckling of the reinforcement and crisis of the connection.

For the strengthening systems which include the application of FRPs both in the tension and in the compression zones, the considerations reported above are all valid.

6.4 ANALYSIS OF THE BEHAVIOUR AT THE ULTIMATE LIMIT STATES FOR COMBINED COMPRESSION AND UNIAXIAL BENDING

6.4.1 Foreword

The design at the Ultimate Limit State requires the sizing of the FRP reinforcement in such a way that the representative point in the plane (*N, M*) of the design internal force is inside the resistance (design) region of the cross section.

In the case of flexural strengthening timber elements with FRP plates or sheets, considering their insignificant contribution in terms of stiffness, the reinforcement accomplishes its action mainly in ultimate conditions, whereas in service conditions its effectiveness is to be considered insignificant.

In service conditions, with reference to the FRP reinforcement, only stress verifications according to what is reported in § 4.2.3.2 of the CNR-DT 200/2004 are required.

In relation to the ULS, the procedure of verification/design is based on the following hypotheses:

- plane cross sections remain plane;
- perfect FRP/timber bond;
- linear elastic behaviour of the timber in tension parallel to the fibre up to collapse;
- elastic-plastic behaviour of the timber in compression parallel to the fibre (**Table 4-1**);
- linear elastic behaviour of the FRP up to collapse;
- crisis of the element in bending caused always by the attainment of the ultimate strains of the timber material, both in the compression and in the tension zones.

In Figure 6-4, the normal stress and strain diagrams acting on a rectangular cross section strengthened by a single plate are reported as an example. In situations different from the one considered above, they require appropriate modifications and/or additions.

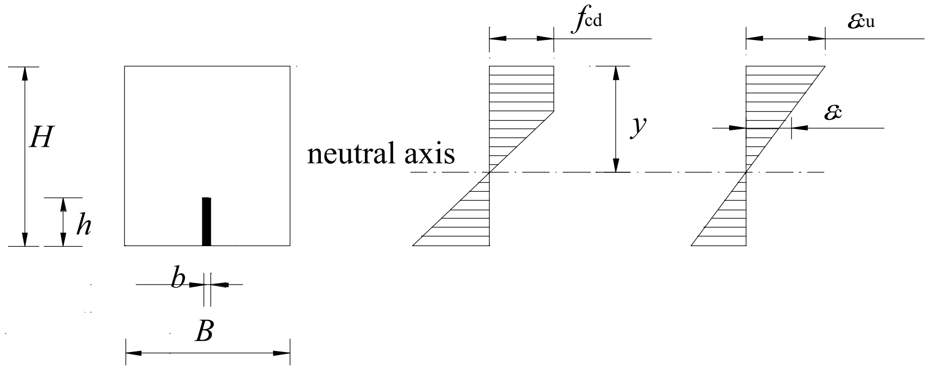


Figure 6-4 – Stress and strain diagrams for a rectangular cross section strengthened by a single plate.

In the verification, the buckling phenomena are neglected. The related check for stability must be made without taking into account the contribution of the FRP strengthening.

6.4.2 Verification procedure

Reference is made to the timber cross section of Figure 6-4, with area A, strengthened by a plate with area A_f, located at the distance d_f from the top edge in compression ($d_f = H - h/2$).

The possible collapse conditions correspond to the attainment of one of the following situations:

1 - ultimate strain of the timber in tension with the whole cross section under tension;
2 - ultimate strain of the timber in tension with the compression zone in the elastic range;
3 - ultimate strain of the timber in tension with the compression zone in the plastic range;
4 - ultimate strain of the timber in compression with the FRP under tension;
5- ultimate strain of the timber in compression with the FRP under compression.

The limit regions identified by the strain diagrams corresponding to the situations above are represented in Figure 6-5.

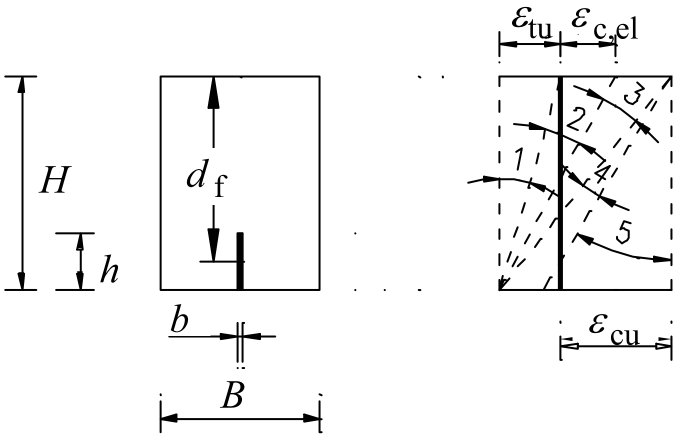


Figure 6-5 – Limit regions.

Each diagram of the limit strains, which is included in one of the above identified five regions, corresponds to an (N, M) point of the boundary of the resistance region of the strengthened cross section.

The governing equations of the problem are equilibrium of forces in the direction of the axis of the element and equilibrium of moments about the parallel to the neutral axis through the centroid of the FRP.

The verification consists in satisfying the following inequality:

$$M_{Sd} \leq M_{Rd}\,(N_{Sd}) \tag{6.1}$$

in which M_{Sd} is the design bending moment and M_{Rd} is the bending resistance, which is a function of the design axial force, N_{Sd}.
The following dimensionless quantities can be introduced:

- N_1, equal to the ratio between the axial force N_{Sd} and the quantity $B \cdot H \cdot f_{cu}$;
- M_1, equal to the ratio between the moment M_{Rd} and the quantity $B \cdot H^2 \cdot f_{cu}$;
- ξ, equal to the ratio between the distance y of the neutral axis from the edge in compression and the depth H of the cross section (Figure 6-4).

The position of the neutral axis, ξ, can be determined from the equilibrium of forces in the direction of the axis of the element. Subsequently, the bending capacity can be calculated from the equilibrium of moments about the axis passing by the centroid of the FRP plate and parallel to the neutral axis.

6.4.3 Expressions of N_1 and M_1 for the different limit regions

Within the five limit regions identified in Figure 6-5, the two dimensionless quantities N_1 and M_1 assume the expressions reported hereafter, where the introduced symbols have the following meaning:

- ε_s longitudinal strain of the fibres at the top edge;
- ε_{tu} ultimate strain of the timber in tension;
- ε_i longitudinal strain of the fibres at the bottom edge;
- ξ ratio between the distance of the neutral axis from the top fibres and the depth of the cross section;
- η ratio between the ultimate strength in tension and the ultimate strength in compression of the timber;
- ρ_{frp} ratio between the cross-sectional area of the timber and that of the FRP reinforcement;
- n ratio between the modulus of elasticity in tension of the FRP reinforcement and the modulus of elasticity in tension of the timber;
- p_{frp} ratio between the distance of the FRP reinforcement from the top fibres and the depth of the cross section of the beam;
- $\varepsilon_{c,el}$ strain of the timber in compression at the elastic limit;
- ε_{cu} ultimate strain of the timber in compression;
- k ratio between the ultimate strain and the strain at the elastic limit in compression.

- Zone 1. Attainment of the ultimate strain of the timber in tension with the whole cross section in tension (tension with small eccentricity):

  - $0 \leq \varepsilon_s \leq \varepsilon_{tu}$ (top fibres); (6.2)
  - $\varepsilon_i = \varepsilon_{tu}$ (bottom fibres); (6.3)

- $-\infty \le \xi \le 0$ (position of the neutral axis); (6.4)
- $N_1(\xi) = \frac{\eta}{2} \cdot \left(2 \cdot \xi - 1 + \frac{2 \cdot \rho_{\text{frp}}}{1-\xi} \cdot n \cdot (\xi - p_{\text{frp}}) \right);$ (6.5)
- $$M_1(\xi) = \frac{\eta}{2 \cdot (1-\xi)} \cdot \xi^2 \cdot \left(\frac{1}{2} - \frac{1}{3} \cdot \xi \right) + \eta \cdot \frac{1-\xi}{2} \cdot \left(\frac{1}{2} - \frac{1-\xi}{3} \right) + \\ + \eta \cdot \frac{n}{1-\xi} \cdot (p_{\text{frp}} - \xi) \cdot \rho_{\text{frp}} \cdot \left(p_{\text{frp}} - \frac{1}{2} \right). \qquad (6.6)$$

- Zone 2. Attainment of the ultimate strain of the timber in tension with the compression zone in the elastic range (tension with large eccentricity):

- $0 \le \varepsilon_{\text{s}} \le \varepsilon_{\text{c,el}}$ (top fibres); (6.7)
- $\varepsilon_{\text{i}} = \varepsilon_{\text{tu}}$ (bottom fibres); (6.8)
- $0 \le \xi \le \frac{1}{1+\eta}$ (position of the neutral axis); (6.9)
- $N_2(\xi) = \frac{\eta}{2} \cdot \left(2 \cdot \xi - 1 + \frac{2 \cdot \rho_{\text{frp}}}{1-\xi} \cdot n \cdot (\xi - p_{\text{frp}}) \right);$ (6.10)
- $$M_2(\xi) = \frac{\eta}{2 \cdot (1-\xi)} \cdot \xi^2 \cdot \left(\frac{1}{2} - \frac{1}{3} \cdot \xi \right) + \eta \cdot \frac{1-\xi}{2} \cdot \left(\frac{1}{2} - \frac{1-\xi}{3} \right) + \\ + \eta \cdot \frac{n}{1-\xi} \cdot (p_{\text{frp}} - \xi) \cdot \rho_{\text{frp}} \cdot \left(p_{\text{frp}} - \frac{1}{2} \right). \qquad (6.11)$$

- Zone 3. Attainment of the ultimate strain of the timber in tension with the compression zone in the plastic range:

- $\varepsilon_{\text{c,el}} \le \varepsilon_{\text{s}} \le \varepsilon_{\text{cu}}$ (top fibres); (6.12)
- $\varepsilon_{\text{i}} = \varepsilon_{\text{tu}}$ (bottom fibres); (6.13)
- $\frac{1}{1+\eta} \le \xi \le \frac{k}{k+\eta}$ (position of the neutral axis); (6.14)
- $N_3(\xi) = \frac{\eta}{2} \cdot \left(\frac{1}{\eta^2} \cdot (\xi - 1) + \xi \cdot \left(\frac{2}{\eta} + 1 \right) - 1 + \frac{2 \cdot \rho_{\text{frp}}}{1-\xi} \cdot n \cdot (\xi - p_{\text{frp}}) \right);$ (6.15)
- $$M_3(\xi) = \frac{1-\xi}{2 \cdot \eta} \cdot \left(\frac{1}{2} - \xi + \frac{2 \cdot (1-\xi)}{3 \cdot \eta} \right) + \frac{1}{2} \cdot \left(\xi - \frac{1-\xi}{\eta} \right) \cdot \left(1 - \xi + \frac{1-\xi}{\eta} \right) + \\ + \eta \cdot \frac{1-\xi}{2} \cdot \left(\frac{1}{2} - \frac{1-\xi}{3} \right) + n \cdot \eta \cdot \left(\frac{p_{\text{frp}} - \xi}{1-\xi} \right) \cdot \rho_{\text{frp}} \cdot \left(p_{\text{frp}} - \frac{1}{2} \right). \qquad (6.16)$$

- Zone 4. Attainment of the ultimate strain of timber in compression with the reinforcement in tension:

- $\varepsilon_{\text{s}} = \varepsilon_{\text{cu}}$ (top fibres); (6.17)

- $\varepsilon_{\mathrm{cu}} \cdot \dfrac{1-p_{\mathrm{frp}}}{p_{\mathrm{frp}}} \leq \varepsilon_{\mathrm{i}} < \varepsilon_{\mathrm{tu}}$ (bottom fibres); (6.18)
- $\dfrac{k}{k+\eta} \leq \xi \leq p_{\mathrm{frp}}$ (position of the neutral axis); (6.19)
- $N_4(\xi) = \xi \cdot k \cdot \left(\dfrac{1}{k} - \dfrac{1}{2 \cdot k^2} - \dfrac{1}{2 \cdot \xi^2} + \dfrac{1}{\xi} - \dfrac{1}{2} \right) + \rho_{\mathrm{frp}} \cdot k \cdot n \cdot \left(1 - \dfrac{p_{\mathrm{frp}}}{\xi} \right);$ (6.20)
- $$M_4(\xi) = \frac{\xi}{2 \cdot k} \cdot \left(\frac{1}{2} - \xi + \frac{2 \cdot \xi}{3 \cdot k} \right) + \frac{1}{2} \cdot \left(1 - \frac{1}{k} \right) \cdot \left(1 - \xi + \frac{\xi}{k} \right) \cdot \xi + \\ + \frac{k}{6 \cdot \xi} \cdot (1-\xi)^2 \cdot \left(\frac{1}{2} + \xi \right) + n \cdot \frac{\rho_{\mathrm{frp}} \cdot k}{\xi} \cdot \left(p_{\mathrm{frp}} - \xi \right) \cdot \left(p_{\mathrm{frp}} - \frac{1}{2} \right). \quad (6.21)$$

- Zone 5. Attainment of the ultimate strain of timber in compression at the top edge with the reinforcement in compression:

- $\varepsilon_{\mathrm{s}} = \varepsilon_{\mathrm{cu}}$ (top fibres); (6.22)
- $0 \leq \varepsilon_{\mathrm{i}} < \varepsilon_{\mathrm{cu}} \cdot \dfrac{1-p_{\mathrm{frp}}}{p_{\mathrm{frp}}}$ (bottom fibres in tension); (6.23)
- $0 \leq \varepsilon_{\mathrm{i}} < \varepsilon_{\mathrm{cu}}$ (bottom fibres in compression); (6.24)
- $p_{\mathrm{frp}} \leq \xi \leq \dfrac{k}{k-1}$ (position of the neutral axis); (6.25)
- $N_5(\xi) = \xi \cdot k \cdot \left(\dfrac{1}{k} - \dfrac{1}{2 \cdot k^2} - \dfrac{1}{2 \cdot \xi^2} + \dfrac{1}{\xi} - \dfrac{1}{2} \right);$ (6.26)
- $M_5(\xi) = \dfrac{\xi}{2 \cdot k} \cdot \left(\dfrac{1}{2} - \xi + \dfrac{2 \cdot \xi}{3 \cdot k} \right) + \dfrac{1}{2} \cdot \left(1 - \dfrac{1}{k} \right) \cdot \left(1 - \xi + \dfrac{\xi}{k} \right) \cdot \xi + \dfrac{k}{6 \cdot \xi} \cdot (1-\xi)^2 \cdot \left(\dfrac{1}{2} + \xi \right).$ (6.27)

7 STRENGTHENING OF SLABS AND BRACING FOR IN PLANE ACTIONS

7.1 INTRODUCTION

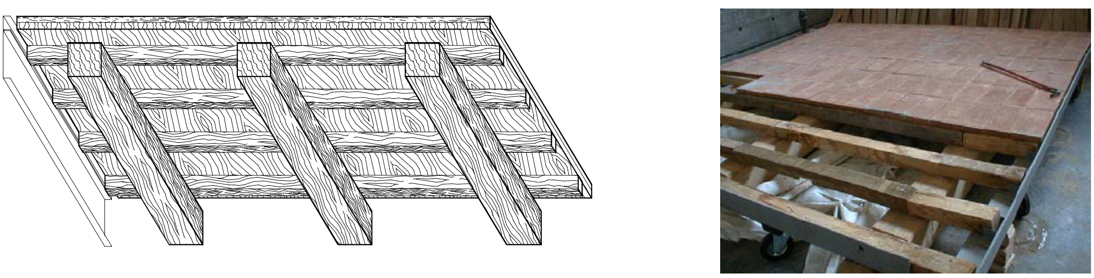
Since ancient times, lightness and workability have contributed to make timber the main material for the realization of slabs in buildings, ranging from those that are not very expensive with residential destination, often entirely made of timber, to the most important public buildings with a masonry framework. During the last century, more modern technological solutions, such as the reinforced concrete floor slab, the common reinforced concrete and hollow tiles mixed floor, and the modern slab with *predalles* have been used. The modest degree of safety offered to the stability of the overall structure in the case of seismic actions, is one on the main factors which have contributed to strongly limit the diffusion of timber floors, although their presence in the construction panorama has never really disappeared.

The widespread diffusion of timber floors in existing buildings and in particular those of historical and monumental importance, has led to the development of new technologies for their retrofitting.

A timber floor has two main parts, with different structure and function: the frame and the slab.

- The frame is made of one or more orders of beams, orthogonal to each other (generally, a primary frame, made of beams, and a secondary frame, made of joists). The frame has the static function of resisting the vertical actions due to: the self weight, the weight of the floor elements and of the completing parts at the intrados and the extrados, the weight of any partition walls, and the action of live loads.
- The floor, in its simpler configuration, is made of a plane element realized by one or more layers of timber planks placed aside each other (Figure 7-1(a)), or even realized by tiles supported by the joists (Figure 7-1(b)). The slab has the static function of resisting the vertical loads which are directly acting on it and to distribute them among the elements of the frame; moreover it also increases the lateral stiffness of the frame, transferring the horizontal actions to the vertical elements of the structure.

Afterwards, the transverse stiffening function will be exclusively examined and the problem of its possible enhancement through the use of FRP will be addressed.



(a) (b)

Figure 7-1 – Timber floor types: (a) slab with timber plank; (b) slab with tiles.

7.2 PREMISES FOR THE STRENGTHENING

One of the primary objectives, which can be pursued for a timber floor in seismic zones, is the enhancement of its strength and stiffness characteristics. In this context, the use of FRP for stiffening may be an efficient tool and valid alternative to traditional techniques, should the latter appear to be inadequate.

In the seismic behaviour of a structure, the role of the slab is to transfer the inertia forces to the vertical structural elements. It is thus important to know the actual stiffness of the slab in its plane. The evaluation of the horizontal forces acting on the single vertical elements and that of the deformation performance of the structure are both based on the value of this stiffness (Figure 7-2). The situation of a slab having a “sufficient” stiffness under in-plane loading in comparison to its bending stiffness has two objectives:

- to allow the introduction of several simplifications in the calculation of the global behaviour of the structure, preserving the validity of the results obtained in terms of forces and deformations;
- to ensure that the slab deformations are sufficiently small, in order to neglect their effects on the force distribution among the vertical elements.

It can be stated that a simple timber floor cannot guarantee an adequate extensional stiffness, so that stiffening interventions for in plane actions are required. This is carried out in order to obtain also an improvement of the global deformation performance of the structure. The effectiveness of the intervention should be evaluated in relative terms, with reference to the initial geometry and constructional aspects.

(a)

(b)

(c)

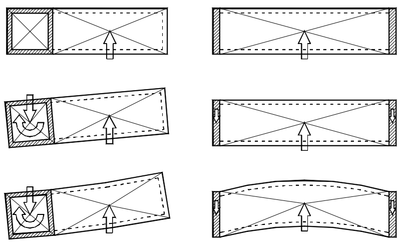


Figure 7-2 – Deformed configurations of typical slabs (a), subjected to in plane actions, in the hypothesis of infinitely rigid slab (b) and of deformable slab (c).

In order for a strengthening intervention for in plane actions to be feasible, the floor to be strengthened should be structurally adequate. In fact, this type of intervention does not significantly influence the flexural stiffness nor the load bearing capacity of the floor. These should then be examined separately, and, if needed, suitably upgraded. Moreover it is worth noting that the intervention does not alter the dead loads acting on the floor therefore, a sufficient load bearing capacity of the slab after the intervention is automatically guaranteed if it was guaranteed before the intervention.

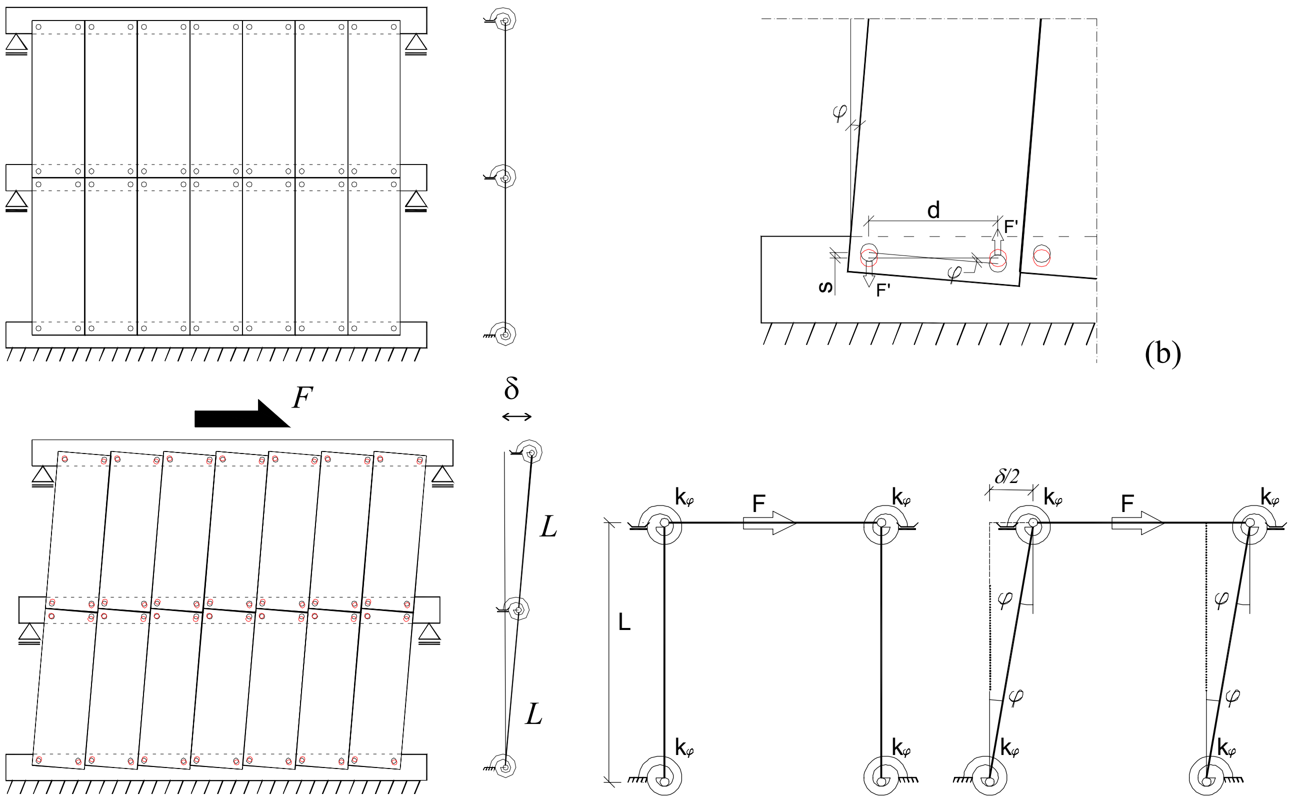
7.3 BEHAVIOUR OF THE UNSTRENGTHENED FLOOR UNDER IN PLANE ACTIONS

The behaviour of a floor subjected to in plane actions is not easy to determine, due to the high degree of disconnection between the various members of the floor.

For the sake of simplicity, reference can be made to a floor made of a single layer of planks, regularly distributed and connected to the joists of the frame by two connectors (nails or screws) at each end. It is also assumed that the horizontal displacements of the floor are restrained along one side, whereas on the opposite side a displacement is impressed parallel to the side, which corresponds to an in-plane shear force acting on the floor.

The behaviour of the floor is shown in Figure 7-3. Assuming that, for small displacements, no problems of penetration between the planks can arise, the only resistance opposed to the deformation is offered by the connectors which link the planks to the beams. Each single plank can be modelled as pinned to the beams and also restrained at each end by a spring having a finite rotational stiffness about an axis orthogonal to the floor plane.

Once the geometrical parameters and the mechanical characteristics of the materials constituting the system are known, under the assumption that the single planks are undeformable, the rotational stiffness of the two springs can be easily estimated through the geometrical construction illustrated in Figure 7-3(b):



(a) (c)

Figure 7-3 – (a) Scheme of the floor subjected to in plane actions; (b) Stiffness due to a single couple of connectors; (c) Model of a single span of the floor.

Following a rotation, φ, of the floor in its plane, the sliding, s, of the single connector is given by the following relationship:

$$s = \frac{d}{2} \cdot \varphi = \frac{F'}{k_{\mathrm{ser}}}, \tag{3.1}$$

where k_{ser} (according to what indicated in the code EN 1995-1-1) depends upon the type of connector and the volumic mass of the timber.
By evaluating the moment M' of the couple of forces F', which are mobilized by the sliding mentioned above, it is possible to determine the rotational stiffness, k_φ:

$$M' = F' \cdot d = \left(\frac{d^2}{2} \cdot k_{\mathrm{ser}} \right) \cdot \varphi = k_\varphi \cdot \varphi . \tag{3.2}$$

The principle of virtual work allows to express the relationship between the story shear force , F, and the corresponding story displacement, δ:

$$F \cdot \delta = 2 \cdot \sum_{\text{planks}} k_\varphi \cdot \varphi^2 \, , \tag{3.3}$$

$$\varphi = \frac{\delta}{L} \, , \tag{3.4}$$

$$F = \left(\frac{2 \cdot \sum_{\text{planks}} k_\varphi}{L^2} \right) \cdot \delta = k_{\text{tot}} \cdot \delta \, , \tag{3.5}$$

where k_{tot} is the overall stiffness of the floor, which can be used as a parameter to evaluate the efficiency of a strengthening intervention, using the model proposed in Figure 7-3(c).

7.4 STRENGTHENING FOR IN PLANE ACTIONS

The intervention for stiffening the floor in its plane, described hereafter, consists in the application on the floor plank of bracing crosses realized with FRP strips, as indicated in Figure 7-4(a).

It is possible to analyse the effectiveness of the intervention by the simplified scheme reported in Figure 7-4(b).
The reinforcement can be modeled as a diagonal tie, pinned to the structure, and considered as a spring with axial stiffness, k_δ, equal to:

$$k_\delta = \frac{E_f \cdot A_f}{D} \, , \tag{3.6}$$

E_f being the elastic modulus of the FRP in the direction of the diagonal, A_f the area of the resisting cross section of the latter and D the length of the diagonal.
The diagonal in compression is not accounted for as the compressive strength of the FRP elements is neglected.
In the case of a floor made of tiles, the tiles can give a not negligible contribution to the stiffness of the floor in its plane being able to transfer compressive forces, provided that this is clearly demonstrated.
With reference to Figure 7-4(b), following the application of an horizontal force, F, the deformation of the structure, similar to that considered for the unstrengthened floor (Figure 7-3(c)), includes also the extension of the tie-reinforcement, function of the angular rotation, φ. The corresponding coefficient of linear dilation, ε, is:

$$\varepsilon = \frac{\delta \cdot cos\, \alpha}{D} \, . \tag{7.7}$$

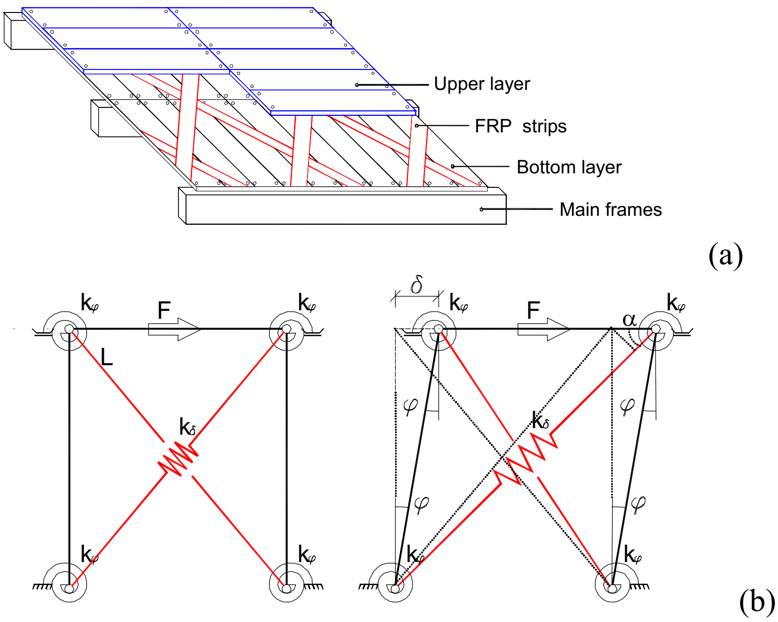


Figure 7-4 – Scheme (a) and calculation model (b) of a reinforced typical floor.

The axial force in the tie-reinforcement, N_{frp}, is:

$$N_{\mathrm{frp}} = k_{\delta} \cdot \varepsilon \cdot D = \frac{E_{\mathrm{f}} \cdot A_{\mathrm{f}}}{D} \cdot \delta \cdot \cos\alpha = \left[\frac{E_{\mathrm{f}} \cdot A_{\mathrm{f}}}{D} \cdot \cos\alpha \right] \cdot \delta . \tag{3.7}$$

Applying the principle of virtual work, it is possible to define the overall stiffness of the floor, $k_{\mathrm{tot,rinf}}$, to be compared to that of the equivalent unstrengthened system, k_{tot}, (Figure 7-3(c)):

$$F = \left[\frac{E_{\mathrm{f}} \cdot A_{\mathrm{f}}}{2 \cdot D} \cdot \cos^2\alpha \right] \cdot \delta = k_{\mathrm{tot,rinf}} \cdot \delta . \tag{3.8}$$

It is immediate to realize the larger magnitude of such stiffness contribution, also for modest amounts of reinforcement, with respect to that offered by the rotational springs, used to model the nailed joints. Therefore, in presence of the FRP reinforcement, this can be entirely given the function of connecting the floor to the frame beams.

Despite its simplicity, the described model gives important indications on the characteristics of the reinforcement in order to guarantee the necessary stiffness to the floor in its plane.
The main characteristics are described as follows.

- The reinforcement can be placed diagonally, with a behaviour similar to that of the metallic braces, exploiting only its tensile strength.
- The FRP should be unidirectional, with an adequate axial stiffness but sufficiently thin and deformable under out of plane actions.

Further considerations can derive from practical aspects related to the realization of the intervention, including:

- The more suitable materials for strengthening are those with glass fibres, due to their modest elastic modulus which allows them to follow the natural movements of the timber and those of the floor without causing excessive stresses at the bonded interface.
- The types of product more suited to the required mechanical characteristics are sheets, either pre-impregnated or impregnated *in situ*. These are available in rolls, from which strips of appropriate length can be easily obtained.
- In order to prevent buckling of the reinforcements in compression, to make the floor behaviour in the two main directions more uniform, and to protect the strengthening system and improve its bond, it is appropriate that, at the extrados, the intervention be completed by a second slab. The planks of this slab are directed perpendicularly to those of the first slab, and are connected to the first slab by means of nails or screws (Figure 7-4(a)). Before laying the second slab, the top surface of the FRP strengthening should be covered with adhesive, therefore increasing the degree of connection between the strengthening and the slab and between the two orders of planks. At the end of the intervention, the extrados surface of the strengthened slab will be totally similar to that without the strengthening, consenting the ordinary operations of surface finishing. In the case of slabs made with tiles, the role of protection and prevention of buckling of the composite fibres can be performed by a layer of mortar.

8 JOINTS AND THEIR STRENGTHENING

8.1 INTRODUCTION

The possibility of exploiting FRPs for new types of connections, in place of steel, and for upgrading existing traditional ones, has been the object of a limited number of experiments. General design and calculation models are currently not available. This topic seems to offer various possibilities for future development and hence it is deemed appropriate to complete the present document with some general concepts and design indications related to it.

The following will be dealt with:

- FRP strengthening of traditional connections;
- Joints made by FRP connectors.

Although both subjects are related to the connections between timber elements, the problems are clearly distinct and cannot be dealt with together.

8.2 FRP STRENGTHENING OF TRADITIONAL JOINTS

Reference is made to both the traditional connections “of carpentry” and the mechanical ones. A strengthening intervention should never substantially alter the original static scheme of the structure, unless the inadequacy of such scheme and the consequent need for its substantial modification are demonstrated.

In particular, an inappropriately evaluated increase of rotational stiffness in favour of a connection can induce not negligible and not admissible alterations of the stress state in the connected elements. The design of the strengthening cannot ignore the overall analysis of the whole structure, in order to correctly evaluate the consequences of the intervention and mitigate, if this is the case, its undesired effects.

It is possible to classify the types of connection strengthening into two distinct categories, based on their objective (Figure 8-2):

- strengthening aimed at upgrading the load bearing capacity;
- strengthening aimed at upgrading the dissipative capacities, in relation to cyclic actions, such as the seismic actions.

In the first case, the basis of the strengthening design is the verification of the tensile stresses perpendicular to the fibres, and of the bearing stresses, which are often the cause of the joint collapse. The problem is particularly significant for the joints realized with a few connectors of large diameter, as these joints are exposed to a higher risk of brittle collapse.

The second case is relative to beam-to-column joints of glued-laminated timber structures, when their dissipative behaviour needs to be upgraded. This objective can be pursued by interposing FRP sheets or plates between the timber lamellae.

Unidirectional, bidirectional or multi-directional FRPs can be used. The choice of the fibre type is conditioned by the required behaviour. Glass fibres are used for a greater deformability of the connection, whereas carbon fibres in order to achieve greater stiffness and strength.

(a)

(b)

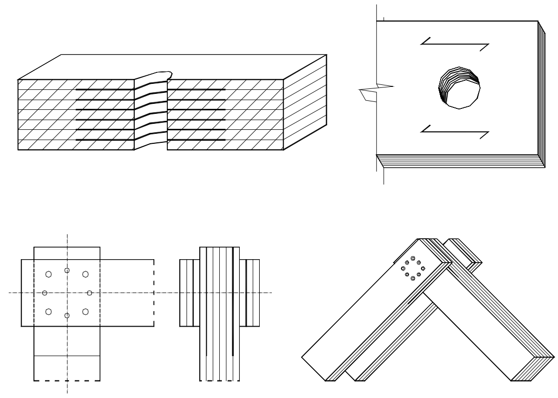


Figure 8-1 – Examples of connection strengthening: (a) detail of an element made of glued-laminated timber strengthened near the connector; (b) beam-to-column connection in glued-laminated timber.

8.3 JOINTS MADE BY FRP CONNECTORS

The types of joints which can be made with FRP bars (Figure 8-2) are similar to those made with steel bars. The results valid for the steel bars can be extended to FRP bars.

The main advantages of FRPs are:

- resistance to physical and chemical attacks for applications in aggressive environments or in absence of adequate protection (carbon fibres are particularly recommended);
- easy on site workability, also with traditional tools;
- light weight;
- wide range of available properties of FRPs, with reference to deformability, strength and fatigue behaviour.

The designer should evaluate for each case the advantages which can be obtained from the use of FRP bars in place of steel bars.

(a)

(b)

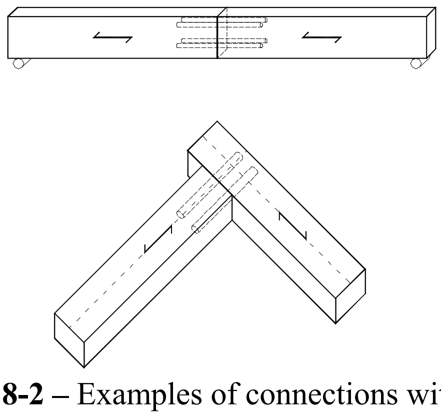


Figure 8-2 – Examples of connections with FRP bars: (a) head-to-head joint; (b) L joint.

9 APPENDIX A: APPLICATIONS TO EXISTING STRUCTURES

9.1 PALAZZO NOBILI, LUCCA

The beams of one of the caisson floors (**Figure 9-1**) of Palazzo Nobili, previously the Bank of Italy in Lucca and now a private residence, were strengthened (2005) with the aim of increasing the fatigue resistance that had been weakened by a localised insect attack on the surface and fungi on one of the edges.
The strengthening used a CFRP plate inserted into the lower edge.

Figure 9-1 – Strengthening intervention in Palazzo Nobili (Designer: dr.ing. M. Martinelli - Legno Più S.r.l.).

9.2 SIAZ BUILDING, TREVI (PG)

The strengthening project, carried out in 2003, used CFRP plates on the structural members (**Figure 9-2**) of a wide timber floor of a building belonging to the SIAZ company in Trevi (PG).
The floor, on the first level of the building used for storing corn, is characterised by a triple row of timber beams (fir), still well preserved and subject to particular rules set out by the Monuments and Fine Arts Service.
The strengthening included CFRP plates applied to both tension and compression areas, bonded with epoxy resin.

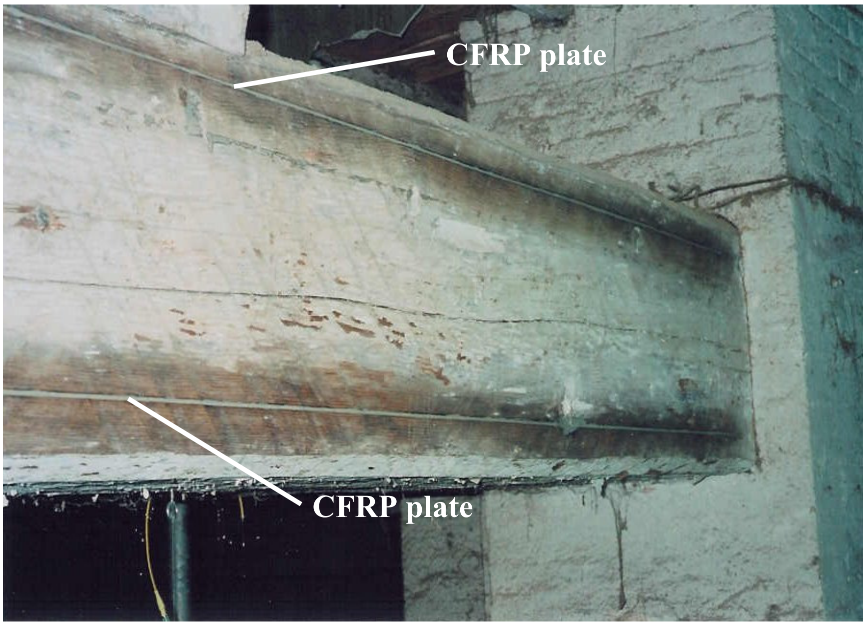


Figure 9-2 – Strengthening intervention on the main beam of the SIAZ building (Designers: ingg. A. Giannantoni and F. Menghini, in collaboration with prof. A. Borri).

9.3 RESIDENTIAL BUILDING, SPOLETO (PG)

This strengthening, part of a project on a civil residence in Azzano, Spoleto (PG), was carried out on a floor with a double timber row and terracotta paving tiles in 2003 (**Figure 9-3**).
The strengthening was needed due to the excessive deformation of the secondary row of the floor, using pre-impregnated CFRP plates inserted into purposely carved slots in the intrados of the secondary timber members.

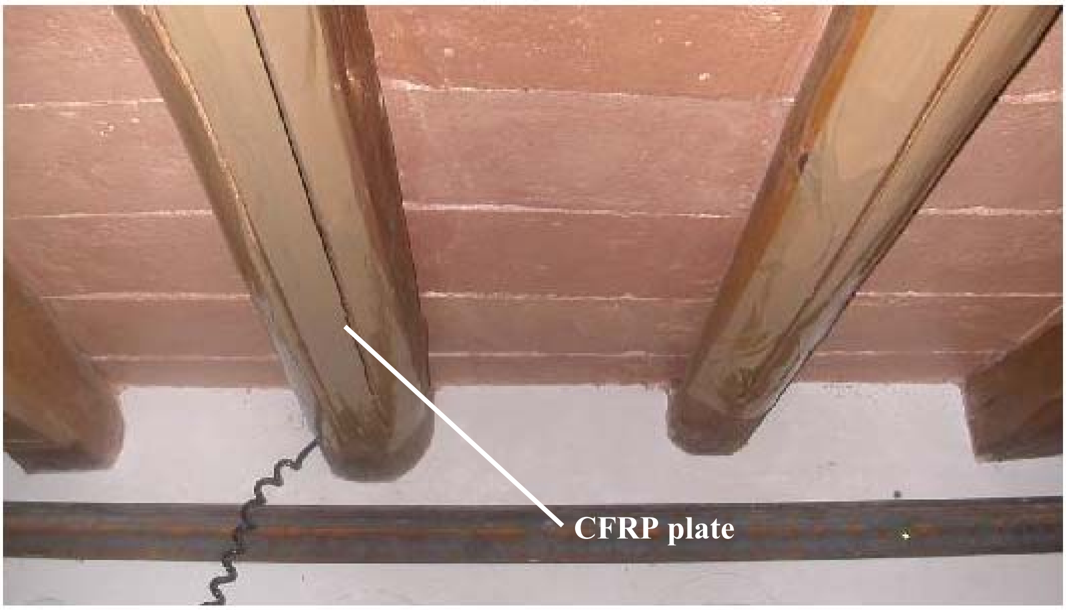


Figure 9-3 –Strengthening intervention on the secondary beams (Designer: ing. A. Giannantoni, in collaboration with prof. A. Borri).

9.4 PALAZZO COLLICOLA, SPOLETO (PG)

This strengthening (2004) was carried out on an old timber floor in Palazzo Collisola in Spoleto, home of the Modern Art Gallery and part of the town hall. It was part of a static intervention being carried out to improve the seismic resistance of the structure.
The floor had a main row of beams made of two wooden beams and a secondary row made of smaller timber beams.
The strengthening was carried out by applying a pultruded GFRP beam to the extrados of the main beams, by removing the existing paving and the concrete layer.
The underside of the GFRP beam was also cut on site at the location of the smaller beams in order to avoid to dismantle them.
The GFRP beam was connected to the timber one using GFRP shear connectors and epoxy resin.

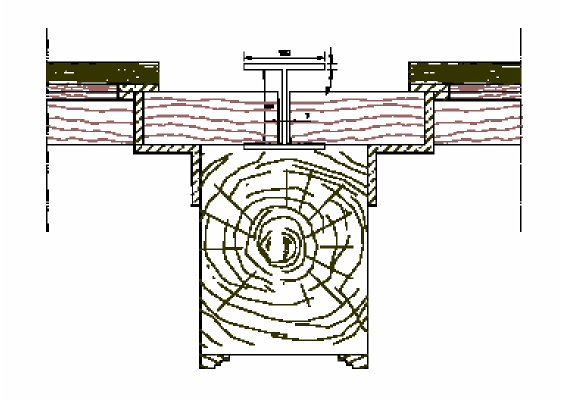


Figure 9-4 –Strengthening intervention on the main beams (Designers: ingg. A. Giannantoni e F. Menghini, in collaboration with prof. A. Borri).

9.5 HISTORICAL BUILDING, LUCCA

This strengthening project was carried out on the beam of an old timber floor of an historical building in Lucca in 2004.
It was needed due to the cracking generated by previous work on part of the building above the floor. The innovative aspects of this strengthening project included the use on the extrados of a CFRP covered timber beam, connected to the beam with connectors to increase rigidity. On the intrados, a CFRP sheet was applied to increase the flexural capacity.
The strengthening was then completed by covering the intrados with a thin timber lamina and, at the extrados, by replacing the covering that had been removed prior to the intervention. (Figure 9-5).

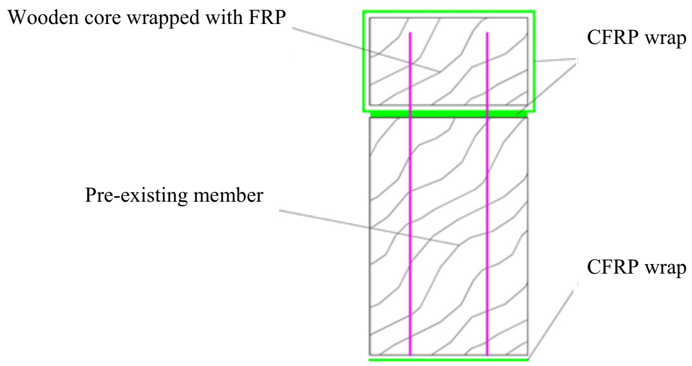


Figure 9-5 – Strengthening intervention on the main beam (Designers: ing. G. Ussia – Ardea Progetti & sistemi S.r.l.).

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11 APPENDIX C: CODES AND STANDARDS

CNR-DT 200/2004: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures. Materials, RC and PC structures, masonry structures.

UNI 11035-1:2003. Legno strutturale. Classificazione a vista di legnami italiani secondo la resistenza meccanica: terminologia e misurazione delle caratteristiche.

UNI 11035-2:2003. Legno strutturale. Regole per la classificazione a vista secondo la resistenza e i valori caratteristici per tipi di legname strutturale italiani.

UNI 11118:2004. Beni culturali. Manufatti lignei. Strutture portanti degli edifici. Criteri per l'identificazione delle specie legnose.

UNI 11119: 2004. Beni culturali. Manufatti lignei. Strutture portanti degli edifici. Ispezioni in situ per la diagnosi degli elementi in opera.

UNI 11138:2004. Beni culturali Manufatti lignei Strutture portanti degli edifici. Criteri per la valutazione preventiva, la progettazione e l'esecuzione di interventi.

UNI EN 301:1993. Adesivi fenolici e amminoplastici per strutture portanti in legno. Classificazione e requisiti prestazionali.

UNI EN 302/1:1993. Adesivi per strutture portanti in legno. Determinazione della resistenza del giunto al taglio a trazione longitudinale.

UNI EN 302/2: Adesivi per strutture portanti in legno. Determinazione della resistenza alla delaminazione (metodo di laboratorio).

UNI EN 338:2004. Legno strutturale. Classi di resistenza.

UNI EN 384:1997. Legno strutturale. Determinazione dei valori caratteristici delle proprietà meccaniche e della massa volumica.

UNI EN 408:2004. Strutture di legno - Legno massiccio e legno lamellare incollato - Determinazione di alcune proprietà fisiche e meccaniche.

UNI EN 518:1997. Legno strutturale. Classificazione. Requisiti per le norme di classificazione a vista secondo la resistenza.

UNI EN 519:1997. Legno strutturale. Classificazione. Requisiti per il legno classificato a macchina secondo la resistenza e per le macchine classificatrici.

EN 1995-1-1:2004 Eurocode 5: Design of timber structures. Part 1-1: General - Common rules and rules for buildings.

UNI ENV 1995-2:1999. Progettazione delle strutture in legno, parte 2: ponti, edizione 1999 (sostituita da EN 1995-2/2004).

EN 14081-1 "Timber structures - Strength graded structural timber with rectangular cross-section - Part 1: General requirements"

PREN 1998-1/2003: Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.

DIN 1052/2004: Bemessungsregeln für Holzkonstruktionen.

ASTM D905:1998. Standard Test Method for Strength Properties of Adhesive Bonds in Shear by Compression Loading.

ASTM D3165:1991. Standard test method for strength properties of adhesives in shear by tension loading of single-lap-joint laminated assemblies.

ASTM D3931:1980. Standard Practice for determining strength of gap-filling adhesive bonds in shear by compression loading.

ISO 6238:1987. Adhesives. Wood-to-wood adhesive bond. Determination of shear strength by compression loading.

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