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

Metallic structures

Preliminary study

CNR-DT 202/2005

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CONTENTS

1 FOREWORD

The present document adds to the series of documents recently issued by the CNR on the structural use of fiber-reinforced polymer (FRP) composites, started with the publication of CNR-DT 200/2004.
This is an informative document, whose intended objectives are disseminating, within the Technical-Professional community, the knowledge acquired on the use of FRPs in the static consolidation of metallic structures as well as identifying the strengthening interventions that are effectively appropriate and safe.
It is also well-known that the current state-of-the-art on this topic is not as advanced as that of concrete or masonry constructions. In fact, adequate solutions are currently available only for a few specific applications.
No more than the UK and USA guidelines (CIRIA 2004, ICE 2001, Schnerch et al. 2006) on the use of FRP for strengthening metallic structures have been drawn up until now.
The present study represents the first step towards the definition of specific design guidelines that will be issued once all the theoretical studies and experiments have given a more complete and universally accepted understanding of the subject.
The results of this study will be useful in identifying any problems that still remain unresolved, therefore allowing the scientific community to concentrate upon them over the next few years.
The current version of this document makes use of some of the basic principles and rules adopted in the document CNR-DT 200/2004, that is relevant to reinforced concrete and masonry structure, while integrating and completing with specific aspects of FRP strengthening interventions with metal structures. In particular, the use of FRP in the flexural and tensile strengthening of structural metal elements is discussed. In addition, several qualitative aspects of debonding as well as the strengthening of fatigue sensitive elements are also dealt with.
The document includes two Appendices, dealing with several examples of FRP strengthening interventions on real structures (Appendix A) and main bibliographic references (Appendix B).
The Document derives from the studies developed within the context of important Italian research projects financed by CNR and MIUR.
It is important to highlight that a guideline, by its nature, is not a binding regulation, but merely represents an aid for practitioners interested in the field of composites. Nevertheless, the responsibility of the operated choices remains with the designer.
Finally, the readers not familiar with the technological and mechanical properties of fiber-reinforced composite materials may consult the first Section and the Appendix A of the CNR-DT 200/2004.

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1.1 PUBLIC HEARING

After its publication, this document CNR-DT 202/2005 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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2 INTRODUCTION

2.1 GENERAL CONSIDERATIONS

The building patrimony of Italy includes various examples of metallic structures, especially used for industrial purposes.
As in the case of other structural types, the need exists to rehabilitate metal structures due to either design defects or degradation of the load bearing elements, as well as variation in their use.
In several situations, strengthening with Fiber Reinforced Polymer composites (FRP) gives better results. This technical document deals specifically with this reinforcement technique.
A significant part of the Italian architectural and historical patrimony is made up of metallic structures. They have had a fundamental role in the development of the industrialization and contributed to structural theories and studies relative to the strength of materials.
The first structures were made of cast iron and/or forged iron (worked or puddled), but subsequently they rapidly developed along with the evolution of mineral iron forging techniques.
The main cultural reason for restoring the older metallic structures, guaranteeing a structural functionality, is based on the need to preserve not only their historical origins but also the scenery value of the places in which they are located. It is therefore for these reasons that the strengthening should be carried out with the aim of maintaining the original design idea.
The most common type of damage for these structures is connected to the characteristics of the materials. Cast iron often fails upon impact due to its brittleness and cracks when subject to temperature changes, that produce tensile stresses.
Tensile elements made of forged iron often present reductions in the cross sectional area due to corrosion.
Bridges built in the second half of the 19th century and at the beginning of the 20th century are among the oldest and most widespread metal structures, in particular railway bridges are currently still being used. Lack of adequate maintenance, corrosion, as well as fatigue, are the main causes of their degradation.
The increase of traffic volume often requires the strengthening of existing bridge structures. The associated costs of strengthening interventions are often smaller than those needed to either dismantle or reconstruct the bridge, not only from the economical point of view but also from the social one.
The strengthening techniques currently used for either restoring or increasing the load bearing capacity of a metallic structure are based on the application of steel plates, either by bolting or welding. This however has several drawbacks. Steel plates add further loads to the structure and are susceptible to corrosion and fatigue. Often, it can be difficult to weld the reinforcement to the substrate (as an example for a cast iron substrate).
Strengthening of existing metallic structures subjected to traffic loading should be designed in the light of drastically reducing the traffic disruption.
The use of FRP overcomes several of the difficulties associated with the use of traditional reinforcing techniques and materials. FRPs have an elevated resistance/weight ratio, much greater than the steel one. They are also much more resistant to corrosion, if not completely unaffected, as well as extremely easy to be handled.
The use of such materials in strengthening metallic structures is not as developed as its concrete or masonry counterparts. Studies have focused on only some of the possibilities, including:

- strengthening of riveted elements against fatigue crack propagation;
- strengthening of tensile elements, or tensile zone of flexural elements, to reduce the stresses in serviceability conditions due to either the increase in loads or corrosion. Pre-stressing of the FRP lamina prior to bonding reduces the in service tensile stresses without requiring the preliminary application of temporary preloaded struts to support the flexural elements.

The use of FRP is convenient also in strengthening older metallic constructions, since the mechanical properties of the FRP integrate well with the cast iron ones. The high tensile strength of FRP in the fiber direction as well as the use of the pre-stressing technique compensate the low tensile strength of cast iron.
FRPs can be applied to steel structures either by bonding in situ FRP sheets and thermosetting resin or pultruded laminas, eventually prestressed. Careful attention should be given to the effectiveness of the bonding technique.
With reference to the fiber choice, carbon is often the most adequate due to the fact that it presents values of the Young's modulus of elasticity similar or greater than that of steel. The same cannot be said for neither glass fiber nor aramid fibers.
The use of carbon fibers however requires an insulating element, to be placed between the composite and the substrate, in order to avoid galvanic corrosion. This is not necessary when glass or aramid fibers are used.
The studies and tests, reported in the literature, highlight that the use of FRPs in strengthening metallic structures is versatile and reliable as well as in constant increase.

2.2 TOPICS

The aim of the Document is to represent the state-of-the-art on the design, installation and monitoring of externally bonded FRP systems for strengthening existing metallic structures. This study represents the first step towards the definition of design guidelines.
The Document deals with:

- basis of strengthening and special issues;
- strengthening of tensile elements;
- flexural strengthening;
- debonding strength;
- strengthening of fatigue sensitive elements;
- installation, monitoring and maintenance.

There are also two Appendices, including several national and international applications of FRP strengthening of steel structures (Appendix A) and the main references on the topic (Appendix B).
The remaining topics are approached according to the usual style of technical documents published by the CNR. The approach of the Eurocodes is adopted; statements are divided between *Principles* and *Application Rules*. Each statement is marked by a progressive numbering, with the principles being marked by the label (P). *Principle* statements include the following:

- *general statements and definitions of mechanical-structural nature.*
- *recognized needs and/or analytical models accepted by the scientific community, whose value is universally deemed to be pre-eminent with respect to possible alternatives, unless otherwise explicitly stated.*

Application Rules are *procedures of widely recognized value, following the Principles and satisfying their needs.*

2.3 SYMBOLS

General notations

$(.)_a$ value of quantity (.) for the adhesive
$(.)_d$ design value of quantity (.)
$(.)_f$ value of quantity (.) for the fiber-reinforced composite
$(.)_k$ characteristic value of quantity (.)
$(.)_R$ value of quantity (.) as resistance
$(.)_s$ value of quantity (.) for the metallic substrate
$(.)_S$ value of quantity (.) as demand

Uppercase Roman letters

A_s cross section area of the metallic substrate
A_f cross section area of FRP reinforcement
E_s Young's modulus of elasticity of the metallic substrate
E_f Young's modulus of elasticity of FRP reinforcement
G_a shear modulus of adhesive
I moment of inertia of transformed section
I_s moment of inertia of the metallic substrate about its centroidal axis, parallel to the reinforced beam neutral axis
I_f moment of inertia of the FRP reinforcement about its centroidal axis, parallel to the reinforced beam neutral axis
M_{Rd} flexural capacity of FRP-strengthened member
M_{Sd} factored moment
N_{Sd} factored axial force
S_e static moment about neutral axis of the section part subtended by this axis
T temperature

Lowercase Roman letters

f_{ak} adhesive characteristic strength
f_{fd} design strength of FRP reinforcement
f_{fk} characteristic strength of FRP reinforcement
f_{yk} characteristic yield strength of metallic substrate
f_{yd} design yield strength of metallic substrate
f_{uk} characteristic tensile strength of metallic substrate
$f_{sk,sup}$ upper characteristic value of the yielding stress for ductile material, or failure stress for brittle material
b_f width of FRP reinforcement
d distance from extreme compression fiber to centroid of tension reinforcement
t_f thickness of FRP reinforcement
t_s thickness of the metallic substrate
x_e distance from extreme compression fiber to neutral axis

Uppercase Greek letters

K_{eff} stress intensity factor
$K_{eff,th}$ threshold value of K_{eff} for crack propagation
Y non-dimensional stress intensity factor range

Lowercase Greek letters

α_f coefficient of thermal expansion of the reinforcement
α_s coefficient of thermal expansion of the metallic substrate

γ partial factor
γ_{Rd} partial factor for resistance models
ε_0 initial strain prior to strengthening
$\varepsilon^{(t)}$ tensile strain
$\varepsilon^{(c)}$ compression strain
ε_{fa} strain at adhesive-reinforcement layer
ε_{sa} strain at metallic substrate-reinforcement layer
χ curvature
η conversion factor
η_a environmental conversion factor
η_l conversion factor for long-term effects
σ_0 metallic substrate normal stress prior to strengthening
σ_f FRP reinforcement normal stress
σ_{fp} FRP reinforcement prestress
σ_s metallic substrate normal stress
σ_{sp} compressive stress in metallic substrate due to the FRP prestress
τ_{fk} FRP shear strength characteristic value

3 BASIS OF STRENGTHENING AND SPECIAL ISSUES

3.1 BASIC REQUIREMENTS

(1) P The purposes of the FRP strengthening of metallic structures include:

- increasing or restoring the tensile strength;
- increasing or restoring the flexural strength;
- increasing the fatigue resistance.

Recent studies have also highlighted the possibility of increasing or restoring the load bearing capacity of thin compressed elements. Nevertheless, the number of studies currently available, from both the theoretical and experimental point of view, is still few, in order to define reliable design procedures.

(2)P The FRP strengthening described in this document consist of the application of either pultruded laminates or impregnated sheets to the external surface of the structural elements.

(3)P The strengthening design should be aimed at guaranteeing a tensile stress state within the FRP strengthening. In such a case, the FRP strengthening can be fully functional in addition to the tensile stress present within the metal support element. The use of FRP in a compressive zone is currently not recommended due to the lack of adequate models and tests relative to the delamination phenomena in the presence of compressive stresses.

(4)P FRP-strengthened metallic structures, subject to cyclic stresses, including those deriving from thermal cycles can deteriorate over time. In order to reduce this potential failure mode, additional mechanical connectors can be applied to the strengthened area.

(5)P FRP strengthening should be applied to structural elements with adequate characteristics. If the metallic substrate is in an advanced corroded state, the oxides must be completely removed, taking great care to restore the original profile of the support.

(6)P The FRP reinforcement should be selected in order to be compatible with the environmental conditions (temperature, humidity, UV radiations, etc.) to which it will be exposed, avoiding the formation of galvanic currents.

(7)P In the case of FRP strengthening of fatigue-sensitive elements as well as when using the prestressing technique creep phenomena of the adhesive should be taken into account. In particular cases, when considered to be necessary, it is possible to integrate the FRP-metal joint with permanent mechanical connectors.

(8)P The FRP strengthening of existing metallic structures can be carried out only when the original structure fulfill the ultimate limit state (ULS) requirements with reference to the quasi-permanent load combination (as defined in the current design Codes) and using the values of the partial factors for exceptional loading condition.

(9)P The design should take into account the possible accidental damage or degradation of the FRP reinforcement under usual working conditions, incorporating eventual protection systems.

(10)P In the case of an historical structure or monument being strengthened, a specific justification should be given to the need for an immediate intervention and full compatibility of the reinforcement with the existing structure, in accordance with the consolidated Restoration principles. The requirements set out in (8)P should not be applied to this type of structures.

3.2 PARTIAL COEFFICIENTS

3.2.1 General considerations

(1)P The values of the materials and products properties used for the strengthening should be determined in accordance to the indications set out in the Document CNR-DT 200/2004.

(2)P The design strength of a FRP strengthened metallic element can be expressed as follows:

$$R_{\rm d} = \frac{1}{\gamma_{\rm Rd}} \cdot R\left\{X_{\rm d,i}; a_{\rm d,i}\right\} = \frac{1}{\gamma_{\rm Rd}} \cdot R\left\{\eta \cdot \frac{X_{\rm k,i}}{\gamma_{\rm m,i}}; a_{\rm d,i}\right\}, \tag{2.1}$$

where $R\{\cdot\}$ is a suitable function for the specific mechanical model being considered (e.g. flexure, shear, etc) and $\gamma_{\rm Rd}$ is a partial factor covering uncertainties of the assumed model. The arguments of the function $R\{\cdot\}$ are typically the design values $X_{\rm d,i}$ of the i-th materials used for strengthening, or the existing materials, and the nominal values $a_{\rm d,i}$ of the i-th geometrical parameter involved in the model. In addition, η is a conversion factor that takes into account, in a multiplicative manner, special design issues, while $\gamma_{\rm m,i}$ is the partial coefficient of the i-th material and product.

3.2.1.1 Partial factors of the metallic substrate (γ_s)

(1)P If the substrate is made of a metal provided of specific design Code, the partial factors should then be applied.

(2)P In general, the partial factor of a ductile metal (steel, "old" iron,) is different to that of a brittle metal (cast iron). In the case of brittle metals, with reference to the failure consequences, an higher value of partial factors should be adopted.

(3) It is well known that a brittle metal structure is designed with the "admissible" stresses method (i.e. in the elastic field of response). Verification through the limit state method, as set out in this document, requires the use of a partial factor of the material. In the case of a ductile substrate, the indications set out in the existing Codes could be used, while for "old" iron as well as cast iron specific tests are required.

3.2.1.2 Partial factors of the FRP strengthening (γ_f)

(1)P The partial factors of FRPs depend on both the installation method as well as the quality of the application. Table 3-1 suggests values of partial factors for the ultimate limit state, as they are set out by the CNR-DT 200/2004:

Table 3-1 – ULS partial factors γ_f.

Type A Application	Type B Application
1.10	1.25

Type A and Type B applications are defined hereafter.

Type A Application	Strengthening system with certification of each component as well as the final product to be applied to a given support.
Type B Application	Strengthening systems certified for each component only.

(2) For the serviceability limit states, the value of γ_f is equal to 1.

3.2.1.3 The adhesive/interface partial factor (γ_a)

(1)P The partial factors of the adhesive/interface used to join the FRP to the substrate, depend on both the installation method and the quality of the application. Particular attention should be given to the evaluation of the thickness of the adhesive layer in the definition of the partial factor. In a type A application the adhesive thickness should be controlled. At the ultimate limit states the values listed in Table 3-2 are suggested.

Table 3-2 – ULS partial factors γ_a.

Type A application	Type B application
1.20	1.50

3.2.1.4 Partial factors for resistance models (γ_{Rd})

(1) For ULS, values to be assigned to the partial factors γ_{Rd} are reported in Table 3-3.

Table 3-3 – Partial factors γ_{Rd}.

Resistance model	γ_{Rd}
Bending/Combined bending and axial load	1.00
Shear/Torsion	1.00
Delamination	1.20
Fatigue	1.20

3.3 SPECIAL DESIGN PROBLEMS AND RELEVANT CONVERSION FACTORS

(1) Hereafter some reference values to be assigned to the conversion factor $\eta=\eta_a\cdot\eta_l$, that affects both the durability and behavior of the FRP materials, are reported.

3.3.1 Environmental reduction factor (η_a)

(1)P Mechanical properties (e.g. tensile strength, ultimate strain, and Young modulus) of FRP systems degrade under specific environmental conditions such as: alkaline environment, moisture (water and saline solutions), extreme temperatures, thermal cycles, freeze and thaw cycles, and ultraviolet radiations (UV).

(2) Effects of alkaline environment. The damage of the resin due to alkaline environment is typically more dangerous than the one due to moisture. The resin shall complete its curing process prior to be exposed to alkaline environment.

(3) Effects of moisture. The main effects of moisture absorption concern the resin and they can be summarized as follows: plasticization, reduction of glass transition temperature, strength and stiffness (the latter less significant). The absorption of moisture depends on the type of resin, the composition and quality of the laminate, the thickness, the curing conditions, the resin-fiber

interface, and the working conditions. In a marine environment, where osmotic effects may cause the presence of air pockets in the resin, it is suggested to use protective coatings.

(4) Effects of extreme temperatures and thermal cycles. The primary effects of temperature concern both viscous response of resin and composite. As the temperature rises, the Young modulus of elasticity of the resin lowers. If the temperature exceeds the glass transition temperature, the performance of FRP materials significantly decreases. In general, thermal cycles do not have detrimental effects on FRP; however, they may cause the formation of micro-fractures in systems with an high resin Young modulus. For typical temperature levels in civil infrastructures, undesired performance can be avoided by choosing a system where the glass transition temperature is always higher than the maximum operating temperature of the structure or component being strengthened.

(5) Effects of freeze and thaw cycles. In general, exposure to freeze and thaw cycles does not have an impact on FRP performance, whereas it lowers the resin performance as well as the fiber-resin interface. For temperatures below 0 °C, polymeric based-resin systems may improve their performance by developing higher strength and stiffness. The effects of the degradation induced by freeze and thaw cycles may be magnified by the presence of moisture.

(6) Effects of ultraviolet radiations (UV). Ultraviolet radiations rarely degrade the mechanical performance of FRP-based systems, although this may cause some resins to have a certain degree of brittleness and surface erosion. In general, the most harmful effect linked to UV exposure is the penetration of moisture and other aggressive agents through the damaged surface. FRP-based systems may be protected from such damages by adding fillers to the resin or by providing appropriate coatings.

(7) Table 3-4 summarizes the values to be assigned to the environmental reduction factor η_a, depending upon fiber/resin type and exposure conditions. Such values represent conservative estimates based on the durability of different fiber types. Values as reported in this table may be increased by 10% (however $\eta_a \le 1$ shall always be satisfied) whenever protective coatings are used. Such coatings need to be maintained on the strengthened structure for its entire life and need to be experimentally tested and proven to be effective in protecting the FRP system from the environmental exposure.

Table 3-4 – Environmental reduction factor η_a for different exposure conditions and FRP systems.

Exposure conditions	Type of fiber/resin	η_a
Internal	Glass/Epoxy	0.75
	Aramid/Epoxy	0.85
	Carbon/Epoxy	0.95
External	Glass/Epoxy	0.65
	Aramid/Epoxy	0.75
	Carbon/Epoxy	0.85
Aggressive environment	Glass/Epoxy	0.50
	Aramid/Epoxy	0.70
	Carbon/Epoxy	0.85

3.3.2 Reduction factors for long-term effects (η_l)

(1)P Mechanical properties (e.g. tensile strength, ultimate strain, and Young modulus of elasticity) of FRP-based systems degrade due to creep, relaxation, and fatigue.

(2) Effects of creep and relaxation. For FRP-based systems, creep and relaxation depend on the properties of both resins and fibers. Typically, thermosetting resins (unsaturated polyesters, vinyl esters, epoxy and phenolic resins) are less viscous than thermo-plastic resins (polypropylenes, nylon, polycarbonates, etc.). Since the presence of fibers lowers the resin creep, such phenomena are more pronounced when the load is applied transversely to the fibers or when the composite has a low volume ratio of fibers.
Creep may be reduced by ensuring low serviceability stresses. CFRP, AFRP, and GFRP systems are the least, moderately, and most prone to creep rupture, respectively.

(3) Fatigue effects. The performance of FRP systems under fatigue conditions also need to be taken into account. Such performance depends of the matrix composition and, moderately, on the type of fiber. In unidirectional composites, fibers usually have few defects; therefore, they can effectively delay the formation of cracks. The propagation of cracks is also prevented by the action of adjacent fibers.

(4) In order to avoid failure of FRP strengthened members under continuous stress or cyclic loading, values of the reduction factor for long term effects, η_1, are suggested in Table 3-5. In the case of combined continuous or cyclic loading, the overall reduction factor may be obtained as the product of the pertaining reduction factors.

Table 3-5 – Reduction factor for long-term effects η_1 for different FRP systems.

Loading mode	Type of fiber/resin	η_1
Continuous (creep and relaxation)	Glass/Epoxy	0.30
	Aramid/Epoxy	0.50
	Carbon/Epoxy	0.80
Cyclic (fatigue)	All	0.50

3.3.3 Impact and explosive loading

(1) The behavior of FRP systems subjected to impact or explosive loading is not completely understood yet. First indications suggest choosing AFRP (more resistant to impact) and/or GFRP systems rather than CFRP.

3.3.4 Vandalism

(1)P FRP composite materials are particularly sensitive to cuts and incisions produced by cutting tools.

(2) Particular protection systems need to be used for FRP strengthened members open to the public where vandalism could be an issue. The safety of the structural member shall be checked, assuming that the FRP system is no longer in place. Design shall be verified using the combination for quasi-permanent loads while the material partial factors at ULS shall be considered for exceptional loading.

3.3.5 Strengthening limitations in case of fire

(1)P FRP materials are particularly sensitive to high temperatures that may take place during fire. When the room temperature exceeds the glass transition temperature of the resin (or the melting temperature in the case of semi-crystalline materials) both strength and stiffness of the installed FRP system are reduced. In the case of FRP applied as an external reinforcement to steel members, exposure to high temperature produces a fast degradation of the bond between the FRP system and the support. As a result, degradation of the strengthening effectiveness and debonding of FRP composite may take place.

(2) During fire exposure, mechanical properties of FRP strengthened members may be improved by increasing the thickness of protective coatings. It is suggested to use coating capable of reducing the spreading of flames as well as smoke production. It is also recommended to use protective coating systems provided with official certificates. Further specifications on the application of protective coating systems are reported in CNR-DT 200/2004.

(3)P In order to prevent collapse of the FRP strengthened structure, as long as further information on the actual behaviour of coatings and resins under fire exposure is not available, it is recommended to keep low the FRP contribution to the member capacity.

(4) It is suggested that the combination for exceptional loading (fire), as defined by the current building code, takes as a reference the situations hereafter listed, where the design value of the effect of indirect thermal loading is indicated by the symbol E_d.

- Exceptional loading with FRP strengthening still in place ($E_d \neq 0$), when the strengthening system has been designed to withstand fire exposure. Applied loads need to be considered at SLS and load factors in compliance with frequent loading conditions. In this case, all loads acting on the structure for the frequent combination are to be considered. The member capacity, reduced to take into account the duration of fire exposure, shall be computed with the partial factors pertaining to exceptional situations, according to the current codes (for FRP $\gamma_f = 1$).
- Situation following an exceptional event ($E_d = 0$), when the strengthening system is no longer in place. Applied loads need to be considered for quasi-permanent loading conditions. The member capacity, reduced to take into account the duration of fire exposure, shall be computed with the partial factors pertaining to exceptional situations.

4 STRENGTHENING OF TENSILE ELEMENTS

4.1 INTRODUCTION

(1)P This chapter deals with the strengthening of steel tensile elements by FRP materials. The strengthening can be applied for the restoring of the load bearing capacity of partially corroded elements (Figure 4-1) as well as for the upgrading of the failure load of undamaged elements (Figure 4-2).

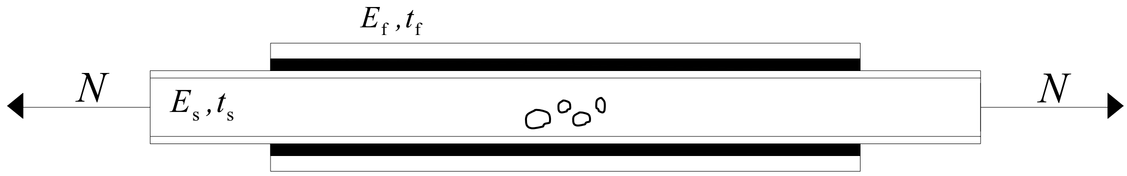


Figure 4-1 – Tensile element with partially corroded cross section and symmetric reinforcement.

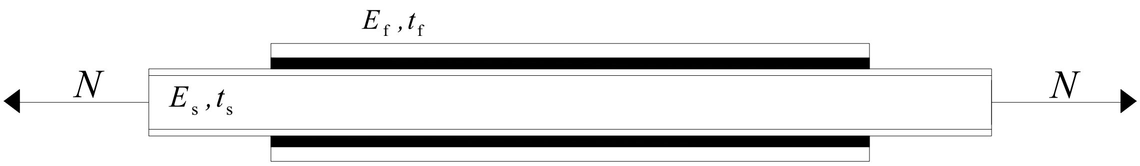


Figure 4-2 – Undamaged tensile element with symmetric reinforcement.

(2) In the following, reference is made to double-symmetric strengthening since this configuration avoids the presence of secondary bending moments.

(3)P The failure mode of tensile elements strengthened by FRP materials can be associated to:

- failure of the metallic substrate;
- failure of the FRP reinforcement;
- failure due to delamination.

The following recommendations refer to the first two failure modes. Delamination must be checked according to the information given in Section 6.

(4)P The evaluation of the structural capacity of a reinforced section is based on the following hypotheses:

- cross section of the reinforced element remains plane;
- perfect bond between the strengthening and the substrate;
- linear elastic behaviour of the materials (isotropic for the metallic substrate and orthotropic for the FRP reinforcement).

4.2 RESTORING OF THE LOAD BEARING CAPACITY OF DAMAGED ELEMENTS

(1)P On the conservative side, restoring of the load bearing capacity of damaged elements, not subjected to fatigue (Section 7 deals with fatigue), is performed assuming that the stresses across the damaged section are bridged by the FRP materials. The strengthening must be designed such that:

$$2 \cdot A_{\mathrm{f}} \cdot \frac{f_{\mathrm{fk}}}{\gamma_{\mathrm{f}}} \cdot \eta \geq A_{\mathrm{s}} \cdot f_{\mathrm{sk,sup}}\,, \tag{3.1}$$

where:

- A_{f} is the cross section area of the FRP;
- f_{fk} is the lower characteristic value of the composite tensile strength;
- γ_{f} is the partial factor of the reinforcement material (§3.2.1.2);
- η is the conversion factor (§3.3.1);
- A_{s} is the cross section area of the metallic substrate;
- $f_{\mathrm{sk,sup}}$ is the upper characteristic value of the yielding stress (f_{y}) for ductile material, or failure stress (f_{u}) for brittle material.

Note that in (3.1) the model partial factor, γ_{Rd}, is not present since it is assumed to be equal to 1 (Table 3-3, § 3.2.1.4).

(2) If a more accurate determination is not available, the upper characteristic value of the yielding or failure stress can be evaluated by multiplying the nominal characteristic value by 1.35.

4.3 UPGRADING OF THE LOAD BEARING CAPACITY OF UNDAMAGED ELEMENTS

(1) The normal stress, σ_{s}, in the metallic substrate, taking into account the mismatch between the thermal expansion coefficient of the strengthening and the substrate, is given by:

$$\sigma_{\mathrm{s}} = \frac{\left[N_{\mathrm{Sd}} + 2 \cdot E_{\mathrm{f}} \cdot A_{\mathrm{f}} \cdot \left(\alpha_{\mathrm{f}} - \alpha_{\mathrm{s}}\right) \cdot \Delta T\right] \cdot E_{\mathrm{s}}}{2 \cdot E_{\mathrm{f}} \cdot A_{\mathrm{f}} + E_{\mathrm{s}} \cdot A_{\mathrm{s}}}\,. \tag{3.2}$$

(2) The normal stress , σ_{f}, in the reinforcement is given by:

$$\sigma_{\mathrm{f}} = \frac{\left[N_{\mathrm{Sd}} + E_{\mathrm{s}} \cdot A_{\mathrm{s}} \cdot \left(\alpha_{\mathrm{s}} - \alpha_{\mathrm{f}}\right) \cdot \Delta T\right] \cdot E_{\mathrm{f}}}{2 \cdot E_{\mathrm{f}} \cdot A_{\mathrm{f}} + E_{\mathrm{s}} \cdot A_{\mathrm{s}}}\,, \tag{3.3}$$

where:

- N_{Sd} is the design load;
- E_{f} and A_{f}, are the modulus of elasticity and the cross section area of the strengthening, respectively;
- α_{f} is the coefficient of thermal expansion of the reinforcement;
- α_{s} is the coefficient of thermal expansion of the substrate;
- ΔT is the change in temperature after the strengthening application;
- E_{s} and A_{s}, are the modulus of elasticity and the cross section area of the metallic substrate, respectively.

(3) The check of the strengthening element requires that:

$$\sigma_s \le \frac{f_{sk}}{\gamma_s \cdot \gamma_{Rd}},$$
$$\sigma_f \le \frac{f_{fk}}{\gamma_f \cdot \gamma_{Rd}} \cdot \eta, \tag{3.4}$$

where f_{sk}, f_{fk}, γ_s, γ_f, γ_{Rd}, η, are defined in § 4.2.

4.4 SERVICEABILITY LIMIT STATE

(1)P In service condition, the stresses in the composite, evaluated with reference to the quasi-permanent load combination, must be $\sigma_f \le \eta \cdot f_{fk}$, where f_{fk} is the characteristic failure stress and η is the conversion factor, given by § 3.3.

(2)P In the metallic substrate, the design checks required by the current Building Code must be performed.

5 FLEXURAL STRENGTHENING

5.1 FAILURE MODES

(1)P Failure modes occurring in beams are listed below:

- tension failure of either the metallic beam (fracture or yielding depending upon the type of base material) or FRP (Figure 5-1(a) and Figure 5-1(b));
- compression failure of the metallic beam (yielding or local buckling) (Figure 5-1 (c));
- delamination ((i) at the interface between the composite reinforcement and the metallic beam, (ii) inside the composite reinforcement, (iii) inside the metallic beam);
- shear failure by local buckling at the beam supports;
- global buckling (either axial or lateral-torsional) of the metallic beam.

In any case, only the composite reinforcement applied in the tension zone must be taken into account.

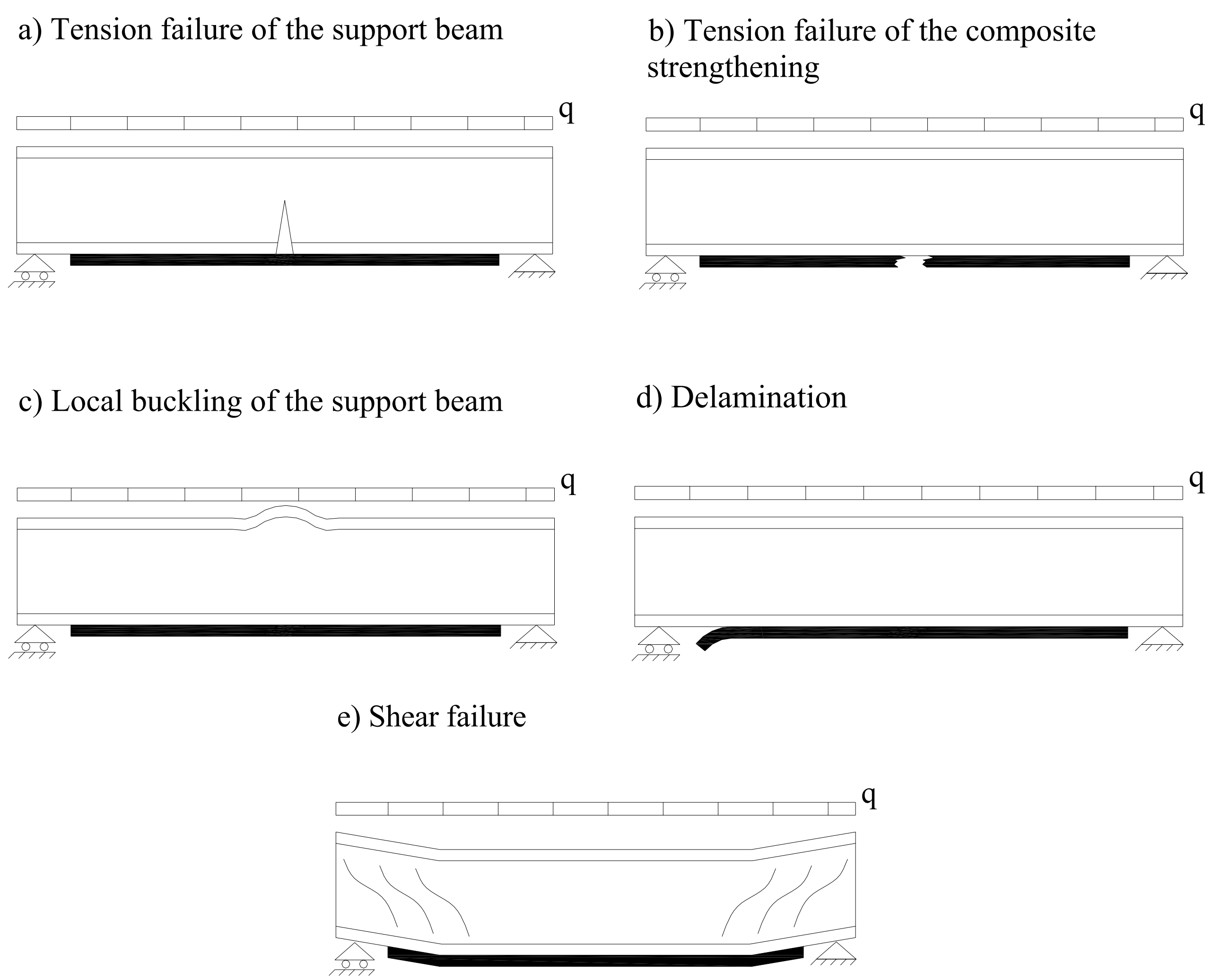


Figure 5-1 – Failure modes of a metallic beam strengthened with FRP and loaded in flexure.

In the following, the possible failure modes are briefly described, giving also some information regarding the design checks against tension, compression or delamination failure mode.

1) Tension failure of either the metallic beam or FRP

Some ancient metallic members are made of cast iron, which is a material exhibiting brittle failure when subject to tension. In such a case, the design is usually addressed at reducing normal stresses produced by service dead and live loads, in order to increase safety against failure at the tensile side of the metallic member. This is a fundamental difference with respect to the case of strengthening reinforced concrete beams, where it is usually accepted the tensile flexural cracking.

With reference to cast iron beams, the use of pre-tensioned FRPs may be found particularly useful, if not mandatory, since in this case a reduction of the tensile stresses in the substrate is achieved without a preliminary reduction of the stresses due to live loads.

In the case of metallic beams exhibiting ductile failure (wrought iron, steel) the composite strengthening has the function of reducing stresses in the metallic beam under service loads (in order to increase the fatigue resistance), as well as increasing the ultimate load bearing capacity of the composite strengthened beam, with respect to the un-reinforced situation, allowing the development of post-elastic deformation capacity of the metallic beam thanks to the FRP high tensile strength. In fact, even if tensile failure of FRP is brittle, the development of inelastic deformation in the metallic beam can result in a ductile composite system, until the delamination of the strengthening FRP system takes place.

2) Compression failure of the metallic beam

Compression failure of the metallic beam occurs with different modes according to the type of the substrate material and geometry of the cross section. Some metals, such as cast iron, are characterized by strong non linearity in tension, with gradual stiffness reduction. On the contrary, wrought iron and steel exhibit clear yielding. Once the type of stress-strain response is known, the geometry of the cross section will dictate the type of failure mode. In fact, depending on the local slenderness and the ratio between the Young modulus and the elastic limit stress, local buckling may occur either before or after some inelastic deformation takes place. In the case of steel beams, modern design Codes, such as Eurocode 3 (EN 1993-1-1), give rules for establishing the type of the compression failure mode. In the case of cast or wrought iron, specific investigation is required to ascertain the type of failure mode in compression. However, an approximate estimate of the buckling stress may be obtained by using classical formulae of metallic beams, using inertia properties of an homogeneous beam equivalent to the actual composite one.

3) Delamination

The analysis of the composite section, made up of the metallic beam and FRP, is usually carried out under the assumption of a perfect bond between the metallic beam and the strengthening FRP. But, this is not the actual situation, due to the adhesive always being made of a thin but finite and deformable layer of material. Strong stress and strain concentration normally occurs at the interface between the metallic beam and the FRP, where discontinuities occur, such as at the end of the strengthening FRP or in correspondence of cracks in the metallic beam. This stress concentration is at the origin of failure by delamination, which will be deeply described later on Section 6.

4) Shear failure

Since the flexural strengthening implies an increase of the acting vertical loads, it is necessary to check the beam against both shear and local buckling failure close to the supports.

5) Global buckling of the beam

The composite metal-FRP beam will be characterized by better inertia properties in relation to the original bare metallic beam. This will improve its resistance against global buckling phenomena. Reference is made to recommendation given at Section 3.1.

(2)P The design of the composite metal-FRP beam must adequately consider fatigue effects. Fatigue failure may occur not only in the metallic beam, but also in the FRP strengthening or within the adhesive. Criteria for checking safety against failure by fatigue are given in Chapter 7 of these Guidelines.

5.2 BASIS OF DESIGN

(1)P Analysis of stress and strain state on the cross section of a metallic beam strengthened with FRP must be carried out taking into account of the initial state immediately prior to the strengthening. Stresses and strains in the metallic beam are obtained by superposition of those produced by loads acting before strengthening and those produced on the whole composite FRP-metal beams by loads acting subsequently to the strengthening application. For serviceability limit state checking, the initial stresses can be summed up to those produced on the composite FRP-metal beam after the application of the strengthening FRP system. For ultimate limit state, the initial strains can be summed up to those produced on the composite FRP-metal beam after the application of the strengthening FRP system.

(2)P In the case of flexural strengthening of existing beams characterized by corrosion and/or damage of different nature geometrical dimensions used in calculations must be derived by *in situ* measurements.

(3) Analysis of the stress-strain state of a FRP strengthened metallic beam and subject to bending moments is carried out on the basis of the following main assumptions:

- perfect bond (i.e. no slip) between the metallic beam and the FRP strengthening system;
- plane sections remain plane after deformation;
- negligible thickness of the strengthening system with reference to the section depth;
- negligible contribution of the adhesive layer to the stiffness of the member.

The meaning of the first two hypotheses is well known and established. The third assumption allows for the FRP strengthening system to be assimilated to one single fiber, at the same level of the metallic beam fiber to which it is glued. The last hypothesis is justified by the very small value of the Young modulus of the adhesive (resin) material with respect to the metallic beam one (about 1/100), in addition to the small thickness of the adhesive layer.

5.3 SERVICEABILITY LIMIT STATES

5.3.1 Design checks at time of reinforcement application

(1)P Analysis of the beam behavior at SLS must consider the initial stress, σ_0, acting in the metallic beam immediately before strengthening.

(2)P Linear elastic response of all the materials is assumed at SLS.

5.3.2 Metallic beam design check

(1)P The metallic beam must be checked against all the serviceability limit states suggested by existing design Codes and/or standards of practice.

5.3.3 FRP design check

(1)P Stresses induced by the quasi-permanent load combination in the strengthening FRP system must be adequately limited.

(2) In the case of plane stress states, the following limitation should be applied:

$$\left(\frac{\sigma}{f_{\mathrm{fk}}}\right)^2 + \left(\frac{\tau}{\tau_{\mathrm{fk}}}\right)^2 \leq \eta^2 . \tag{3.5}$$

Symbols used in Equation (3.5) have the following meaning:
- σ and τ are the normal and shear stresses, respectively, acting at the generic point of the composite FRP strengthening system;
- f_{fk} is the characteristic value of the tension strength of the composite FRP strengthening system;
- τ_{fk} is the characteristic value of the shear strength of the composite FRP strengthening system, measured in the direction perpendicular to the beam axis;
- η is the conversion factor (Section 3.2).

5.3.3.1 Local stresses

(1) Both normal and shear local stresses can be computed using available formulations for the analysis of composite sections in the linear-elastic range. The composite FRP-metal beam can be reduced to an equivalent homogeneous beam, by means of coefficients given by the ratio between the Young modulus of the i-th material and a reference material constituting the composite cross section.

5.4 ULTIMATE LIMIT STATES

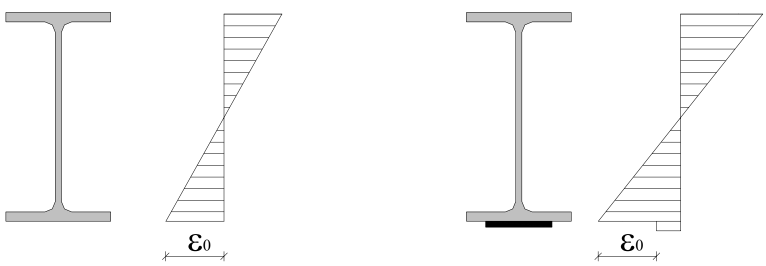
5.4.1 Design checks at time of reinforcement application

(1)P The ultimate limit state is checked by verifying the following Equation:

$$M_{\mathrm{Sd}} \leq M_{\mathrm{Rd}} , \tag{3.6}$$

where M_{Sd} is the bending moment produced by the design load combination and M_{Rd} is the design value of the flexural capacity.

(2)P The initial strain, ε_0, at the extreme fiber of the beam cross section, prior to strengthening, must be considered (Figure 5-2).



Prior to strengthening After strengthening

Figure 5-2 – Initial deformations.

5.4.2 Flexural capacity

5.4.2.1 Basis of design

(1)P The ultimate limit state is reached when the strain acting at the extreme fiber of the cross section equals a limit value. The latter must be established on the basis of the failure modes previously described. For instance, limit values of strains to be considered for a metallic beam strengthened in the tension zone are the following:

$$\varepsilon_{\mathrm{d}}^{(\mathrm{t})} = \min\left\{\varepsilon_{\mathrm{sd}}^{(\mathrm{t})};\varepsilon_{\mathrm{fd}}^{(\mathrm{t})} + \varepsilon_0\right\}, \tag{3.7}$$

$$\varepsilon_{\mathrm{d}}^{(\mathrm{c})} = \varepsilon_{\mathrm{sd}}^{(\mathrm{c})}, \tag{3.8}$$

$\varepsilon_{\mathrm{sd}}^{(\mathrm{t})}$ and $\varepsilon_{\mathrm{fd}}^{(\mathrm{t})}$ being the design values of the tension strain capacity of the substrate and strengthening FRP, respectively, ε_0 is the initial strain previously defined, $\varepsilon_{\mathrm{sd}}^{(\mathrm{c})}$ is the design value of the compression strain capacity of the substrate.
The smallest bending moment corresponding to the fulfillment of the condition (3.7) or (3.8) is the flexural capacity of the strengthened beam.

(2) Design values of strain capacity are calculated starting from the characteristic values, applying the partial and conversion factors introduced at Section 3.2.

(3) The design flexural capacity, M_{Rd}, can be computed using the following procedure:

a) assume that one of the materials composing the cross section has reached, at the extreme fiber of the cross section, its strain capacity, as specified in (1)P;
b) fix a first value of the neutral axis position; this will fix the whole strain state on the cross section because of the assumption of a linear strain distribution (plane sections remain plane);
c) evaluate stresses at each fiber of the cross section, based on the appropriate stress-strain relationship and applying appropriate conversion factors for the FRP strengthening system (Section 3.3);
d) check equilibrium of resultant forces in the direction of the beam axis; for example, with reference to Figure 5-3, the following Equation must be satisfied:

$$F_1 + F_2 + F_3 = F_4 + F_5; \tag{3.9}$$

e) if Equation (3.9) is not satisfied then go back to step b) and repeat steps b), c) and d) with iteration on the neutral axis position, until Equation (3.9) is satisfied;
f) once determined, the neutral axis position allowing satisfaction of Equation (3.9), the design value of the flexural capacity associated to the considered failure mode can be easily computed. For example, with reference to Figure 5-3, the following Equation applies:

$$M_{\mathrm{Rd}} = \sum F_{\mathrm{i}} \cdot d_{\mathrm{i}}, \tag{3.10}$$

where the partial factor γ_{Rd} has been set equal to 1 (Table 3-3, § 3.2.1.4).

(4) The stress-strain relation for the FRP strengthening system is always linear up to failure. The relevant Young modulus, to be used in the procedure above outlined, is the one measured for the FRP system in the direction parallel to the beam axis. Since the FRP strengthening system is often

placed with fibers aligned parallel to the direction of the beam axis, thus maximizing its efficiency, the Young modulus of the FRP strengthening system often coincides with the modulus measured parallel to the fiber direction.
The stress-strain relation of the metallic beam depends upon the type of material, being different in the case of a ductile metal (wrought iron, steel) or in the case of a brittle metal (cast iron), as will be discussed in Sections 5.4.2.2 and 5.4.2.3.

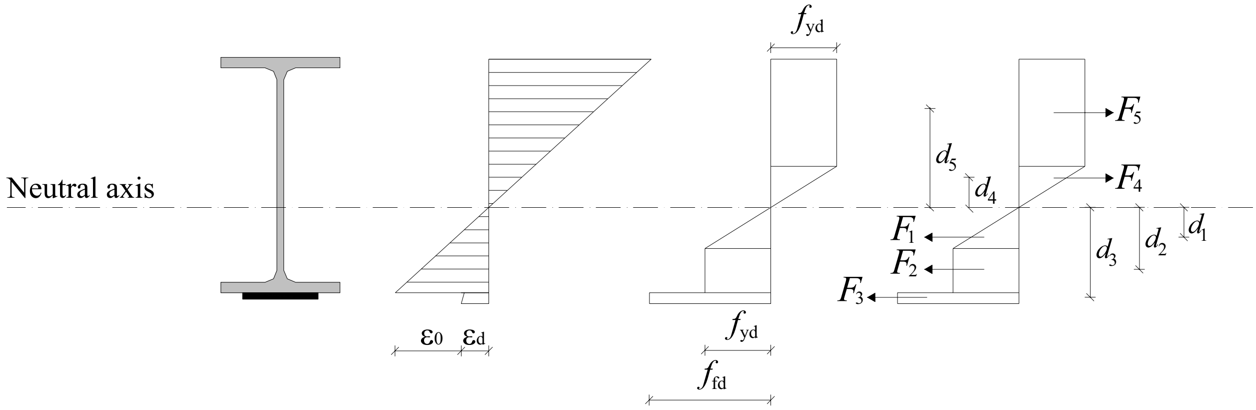


Figure 5-3 – Procedure for computing the flexural capacity.

5.4.2.2 Ductile metallic beam

(1) If the metallic beam is made of a ductile material (wrought iron, steel), exhibiting clear yielding when subject to axial loading, then the stress-strain relation to be used in the evaluation of the flexural capacity can be assumed as elastic-perfectly plastic. The design value of the yield stress can be computed according to the information provided at Section 3.2.

(2) In the case of a metallic beam made of a ductile material, it is possible to reduce the number of possible failure modes. Using definitions suggested by Eurocode 3, if the cross section of the metallic beam is of class 1 or class 2, then failure will occur due to excessive strain reached in the FRP strengthening system. If the cross section is of class 3, then failure will occur due to either yielding in the metallic beam or excessive strain in the FRP strengthening system. If the cross section is of class 4, then failure will occur because of either local buckling of the compression flange of the metallic beam or excessive strain reached in the FRP strengthening system. Hence, limit values of strain in the metallic beam and in the FRP strengthening system can be set as follows:

- class 1 or 2 cross section:
$$\begin{cases} \varepsilon_d^{(t)} = \varepsilon_{fd}^{(t)} + \varepsilon_0 \\ \varepsilon_d^{(c)} = \infty \end{cases},$$

- class 3 cross section:
$$\begin{cases} \varepsilon_d^{(t)} = \min\left\{\varepsilon_{yd}; \varepsilon_{fd}^{(t)} + \varepsilon_0\right\} \\ \varepsilon_d^{(c)} = \varepsilon_{yd} \end{cases},$$

- class 4 cross section:
$$\begin{cases} \varepsilon_d^{(t)} = \varepsilon_{fd}^{(t)} + \varepsilon_0 \\ \varepsilon_d^{(c)} = \varepsilon_d^{(LB)} \end{cases},$$

where ε_{yd} is the design value of the yielding strain of the substrate (assumed to be equal both in tension and compression), $\varepsilon_d^{(LB)}$ is the design value of the peak compressive strain inducing local buckling and other symbols have the meaning already defined in previous Sections. The peak compressive strain corresponding to local buckling can be computed as the ratio of the peak stress inducing local buckling, $\sigma_{d}^{(LB)}$, and the Young modulus, E_s, of the substrate.

5.4.2.3 Brittle metallic beam

(1) In the case of a brittle metal (cast iron), the stress-strain relation is non-linear in tension and linear in compression. Design limit values of stress and strain must respect the information given at section 3.2.

(2) In the case of a metallic beam made of a brittle material, there is no available classification of the cross section allowing *a priori* selection of the type of failure mode. Then, all the limits associated with potential failure modes will be checked.

(3) In the case of a metallic beam made up of a brittle material, already cracked at the time of strengthening, the FRP reinforcement system may be used to bridge the tensile stresses over the crack location. Particular caution is required to avoid local delamination of the FRP strengthening, in correspondence of the cracks in the metallic beam. In fact, there is no validated method for checking safety against local delamination.

6 EVALUATION OF THE DEBONDING STRENGTH

6.1 INTRODUCTION

(1) This chapter deals with the delamination failure mode of the FRP reinforcement. Debonding of the reinforcement from the metallic substrate and the relevant drastic reduction of the bearing capacity of the structural element is called “delamination”. Delamination is due to both the peeling and shear stress in the adhesive layer. Moreover, stresses are present in the FRP reinforcement too and they can lead to the composite delamination. From a theoretical point of view, delamination failure could also take place in the metallic substrate. Nevertheless, the usual hierarchy of resistance (for increasing resistance) is the following: adhesive joint, reinforcement and substrate. It is worth noting that this hierarchy of resistance is not the usual one achieved in RC or masonry structures where the weakest element is given by the substrate where delamination failure takes place. In the literature two different models have been proposed to analyze the delamination problem:

- a conventional stress approach, based on the elastic analysis of the stress field;
- a fracture mechanics approach, based on linear elastic fracture mechanics concepts.

The present chapter deals only with the stress-based approach since the fracture mechanics one, although more appropriate from a theoretical point of view, requires additional theoretical studies and experimental validations.

(2)P As shown in Figure 6-1, delamination takes place in high-stresses concentration zones, often produced by discontinuities in the reinforcement or cracks in the metallic substrate.

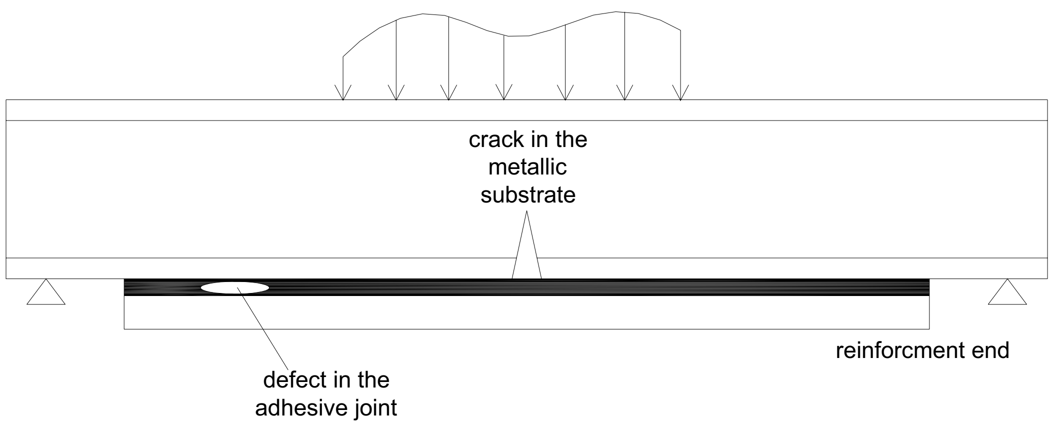


Figure 6-1 – Discontinuity examples in the metallic substrate, adhesive and FRP strengthening.

(3)P Delamination of a strengthened element can take place in the reinforcement, in the substrate or at the interface. In a strengthened element with a metallic substrate, the possible delamination failure modes are listed below, as shown in Figure 6-2:

- delamination at the substrate - adhesive or adhesive - FRP interface;
- cohesive failure of the adhesive;
- delamination in the strengthening material.

As stated above, differently from what usually occurs in strengthened RC structures, delamination does not take place in the metallic substrate but in the adhesive (cohesive failure), at the interface (interface failure) or in the reinforcement (FRP delamination). Nevertheless, if the substrate is made of wrought iron, delamination can also take place in the substrate similarly to what occurs in RC or masonry structures.

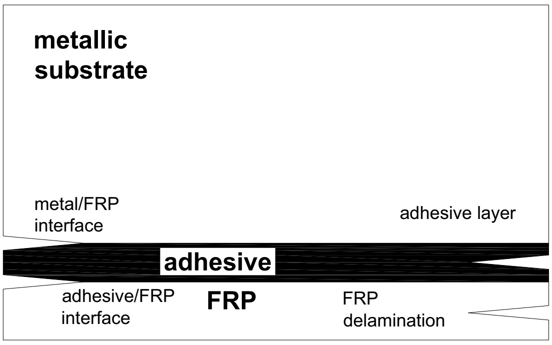


Figure 6-2 – Types of debonding in FRP strengthened metallic structures.

(4)P Due to the mismatch between the thermal expansion coefficient of the substrate and the FRP reinforcement, high stress concentration takes place in the adhesive layer. It is worth noting that in such cases the design process is governed by the delamination failure mode. Temperature effects on the delamination strength must therefore be taken into account.

(5)P Particular environmental conditions can deteriorate the mechanical properties of the interface and then promote the delamination process. In particular, this can be due to thermal cycles as well as the presence of water solution with high salts concentration (de-icing salts).

(6)P The stress-based approach, as well as the fracture mechanics approach, makes use of linear elastic relationships. The relevant delamination check is then valid only in zones with elastic behaviour such as the strengthening ends. If inelastic strain takes place in the metallic substrate, no reliable methods are available to evaluate the delamination strength.

6.2 STRESS-BASED APPROACH

(1)P Delamination strength is conventionally computed evaluating the maximum stresses (peeling and shear) in the adhesive layer through a linear elastic analysis. Figure 6-3 shows the qualitative behaviour of the peeling and shear stress at the strengthening end.

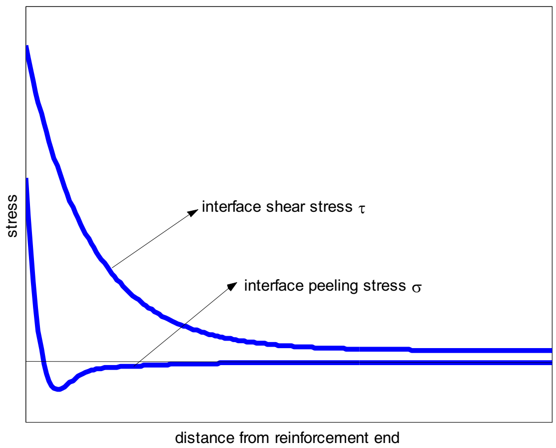


Figure 6-3 – Qualitative behavior of the shear and peeling stresses at the interface.

(2)P The main assumptions of the stress based approach are:

- cross section of the strengthening element remains plane;
- shear strain in the metallic beam and strengthening is neglected;
- perfect bond between different materials;
- linear-elastic stress-strain relationships;
- constant shear and peeling stresses through the thickness of the adhesive layer.

(3)P The shear stresses in the adhesive layer are function of the lack-of-fit strain, $\Delta\varepsilon_{fs}$, developing between the metallic beam and the strengthening if no adhesive joint was present. The peeling stresses are function of the lack-of-fit curvature, $\Delta\chi_{fs}$, developing between the metallic beam and the strengthening if no adhesive joint was present. The lack-of-fit strain and curvature can be evaluated as follows:

$$\begin{aligned} \Delta\varepsilon_{fs} &= \left(\varepsilon_{fa,t} - \varepsilon_{fa,0}\right) - \left(\varepsilon_{sa,t} - \varepsilon_{sa,0}\right) \\ \Delta\chi_{fs} &= \left(\chi_{f,t} - \chi_{f,0}\right) - \left(\chi_{s,t} - \chi_{s,0}\right) \end{aligned}, \tag{4.1}$$

where:

- $\varepsilon_{fa,t}$ is the longitudinal strain at the adhesive-FRP interface after the adhesive joint setup;
- $\varepsilon_{sa,t}$ is the equivalent longitudinal strain at the metal-adhesive interface;
- $\varepsilon_{fa,0}$ is the longitudinal strain at the adhesive-FRP interface at the adhesive joint setup ($t = 0$);
- $\varepsilon_{sa,0}$ is the equivalent longitudinal strain at the metal-adhesive interface at the adhesive joint setup ($t = 0$);
- $\chi_{f,t}$ is the bending curvature in the strengthening after the adhesive joint setup;
- $\chi_{s,t}$ is the equivalent bending curvature in the metallic beam;
- $\chi_{f,0}$ is the bending curvature in the strengthening at the adhesive joint setup ($t = 0$);
- $\chi_{s,0}$ is the equivalent bending curvature in the metallic beam at the adhesive joint setup ($t = 0$).

The above quantities must be evaluated under the assumption that no adhesive joint is present between the beam and the reinforcement.

(4) The subscript "0" is used for strains and curvatures developing at the time of adhesive joint setup. They are due to the loads applied before the adhesive joint setup or to other initial strains (prestrain). For instance, in a metallic beam prestrain can be due to the precambering produced by props or jacks. In a pultruded strip prestarin can be applied by a pretension prior to bonding. Similarly, after the formation of the adhesive joint, the longitudinal strain and the bending curvature in the substrate must be evaluated in relation to the total applied loads and, eventually, to the temperature change. With reference to the strengthening, the relevant quantities are simply due to the prestrain applied to the composite.

6.2.1 Evaluation of shear stresses in the adhesive layer

(1)P At a stress discontinuity, the lack-of-fit strain, $\Delta\varepsilon_{fs}$, can be approximated by the following linear expression:

$$\Delta\varepsilon_{fs}\left(x\right) = \Delta\varepsilon_0 + \Delta\varepsilon_1 \cdot x , \tag{4.2}$$

where $\Delta\varepsilon_0$ e $\Delta\varepsilon_1$ are function of the structural scheme of the strengthened element.

(2) For a beam strengthened with FRP on the tensile side (Figure 6-4), assuming, for the sake of simplicity, that the metallic beam is unstressed at the time of strengthening ($\varepsilon_{sa,0} = 0$) and no other strains are applied to the reinforcement at the time $t = 0$ ($\varepsilon_{fa,0} = 0$) and after the adhesive joint setup ($\varepsilon_{fa,t} = 0$), $\Delta\varepsilon_0$ e $\Delta\varepsilon_1$ are evaluated as follow:

$$\Delta\varepsilon_0 = -\frac{\Delta M(0)}{E_s \cdot I_s} \cdot y_s = -\frac{p \cdot a \cdot \left(l + \frac{a}{2} \right)}{E_s \cdot I_s} \cdot y_s , \tag{4.3}$$

$$\Delta\varepsilon_1 = -\frac{\Delta T(0) \cdot x}{E_s \cdot I_s} \cdot y_s = -\frac{(p \cdot l) \cdot x}{E_s \cdot I_s} \cdot y_s , \tag{4.4}$$

where $\Delta M(0)$ and $\Delta T(0)$ are, respectively, the change of bending moment and shear force at $x = 0$ after the adhesive joint setup (in the present example they are equal to the total applied loading); p, a, l are showed in Figure 6-4; E_s and I_s are the modulus of elasticity and the moment of inertia of the metallic beam, respectively; y_s is the distance of the centroid of the beam from the adhesive interface (Figure 6-4). Note that the above differences were evaluated by a linear approximation of the bending moment $\Delta M(x)$ in the substrate close to the stress discontinuity and then assuming:

$$\Delta M(x) = \Delta M(0) + \Delta T(0) \cdot x . \tag{4.5}$$

If non linear bending moment is present, the above approximation does not lead to significant errors. In fact, stress concentrations are limited to a short distance close to the relevant discontinuity.

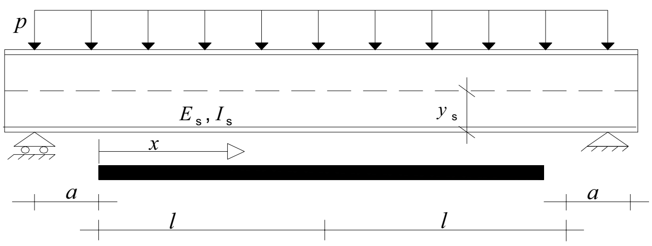


Figure 6-4 – Evaluation of $\Delta\varepsilon_{fs}$ for a metallic beam under a distributed load p and reinforced by an FRP plate.

(3) In the case of a symmetrically strengthened metallic element subject to a tensile load P and a temperature change ΔT (Figure 6-5), if the metallic beam is unstressed at the time of strengthening ($\varepsilon_{sa,0} = 0$) and no strain is imposed to the strengthening at $t = 0$ ($\varepsilon_{fa,0} = 0$), then $\Delta\varepsilon_0$ and $\Delta\varepsilon_1$ are given by:

$$\Delta\varepsilon_0 = \left(\alpha_f - \alpha_s\right) \cdot \Delta T - \frac{\Delta N(0)}{E_s \cdot A_s} = \left(\alpha_f - \alpha_s\right) \cdot \Delta T - \frac{P}{E_s \cdot A_s}, \tag{4.6}$$

$$\Delta\varepsilon_1 = 0, \tag{4.7}$$

where α_f and α_s are the thermal expansion coefficient of the composite and substrate respectively; $\Delta N(0)$ is the axial load change in the steel element, at $x = 0$ after the adhesive joint setup (note that in this case it is equal to the total applied load); A_s is the cross section of the metallic element.

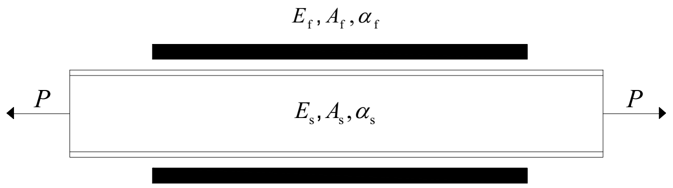


Figure 6-5 – Evaluation of $\Delta\varepsilon_{fs}$ for a tensile metallic element symmetrically strengthened and subject to an axial load P and a temperature change ΔT.

(4) The maximum value of the shear stress in the adhesive layer (at $x = 0$), τ_{max}, is evaluated by the following expression:

$$\tau_{max} = \frac{1}{b_f} \cdot \left[\left(\Delta N_f(0) + \frac{\Delta\varepsilon_0}{f_2} \right) \cdot \lambda + \frac{\Delta\varepsilon_1}{f_2} \right], \tag{4.8}$$

where:

$$\lambda = \sqrt{\frac{f_2}{f_1}}, \tag{4.9}$$

$$f_1 = \left(\frac{t_a}{G_a \cdot b_f} \right), \tag{4.10}$$

- for a metallic beam strengthened on the tensile side:

$$f_2 = \left(\frac{1}{E_\mathrm{f} \cdot A_\mathrm{f}} + \frac{1}{E_\mathrm{s} \cdot A_\mathrm{s}} + \frac{(y_s + t_f/2 + t_a) \cdot y_s}{E_\mathrm{s} \cdot I_\mathrm{s}} \right); \tag{4.11}$$

- for a tensile element symmetrically strengthened:

$$f_2 = \left(\frac{1}{E_\mathrm{f} \cdot A_\mathrm{f}} + \frac{2}{E_\mathrm{s} \cdot A_\mathrm{s}} \right); \tag{4.12}$$

where:

- b_f is the strengthening width;
- t_a is the adhesive thickness;
- t_f is the strengthening thickness;
- G_a is the adhesive shear modulus;
- E_f and A_f, are the modulus of elasticity and the cross section area of the strengthening, respectively;
- E_s, A_s, I_s and y_s are the quantities above defined.

The quantity $\Delta N_\mathrm{f}(0)$ in eq. (4.8) is the axial load change in the composite at the discontinuity (x = 0) and it is function of the discontinuity type (Figure 6-1). As an example, without an external load applied directly to the strengthening end, one has: $\Delta N_\mathrm{f}(0)=0$.

6.2.2 Peeling stress evaluation in the adhesive layer

(1)P The curvature lack-of-fit, $\Delta\chi_\mathrm{fs}$, can be evaluated from eq. (4.5), as follows:

$$\Delta\chi_\mathrm{fs}(x) = \Delta\chi_0 + \Delta\chi_1 \cdot x . \tag{4.13}$$

(2)P The maximum value of the peeling stress in the adhesive layer (at x = 0), σ_max, is evaluated by the following expression:

$$\sigma_\mathrm{max} = \frac{1}{b_\mathrm{f}} \cdot \left[+ \frac{a_3 \cdot C_1 \cdot \lambda^2}{a_1 \cdot \lambda^4 + a_2} - 2 \cdot \beta^2 \cdot C_4 \right], \tag{4.14}$$

where:

$$a_1 = \frac{t_\mathrm{a}}{E_\mathrm{a} \cdot b_\mathrm{f}}, \tag{4.15}$$

$$a_2 = \frac{1}{E_\mathrm{f} \cdot I_\mathrm{f}} + \frac{1}{E_\mathrm{s} \cdot I_\mathrm{s}}, \tag{4.16}$$

$$a_3 = \frac{y_f}{E_\mathrm{f} \cdot I_\mathrm{f}} - \frac{(y_s + t_f/2 + t_a) - y_f}{E_\mathrm{s} \cdot I_\mathrm{s}}, \tag{4.17}$$

$$\beta = \left(\frac{a_2}{4 \cdot a_1} \right)^{0.25}, \tag{4.18}$$

where:

- b_f is the strengthening width;
- t_a is the adhesive thickness;
- t_f is the strengthening thickness;
- E_a is the modulus of elasticity of the adhesive;
- E_f and I_f, are the modulus of elasticity and the moment of inertia of the strengthening, respectively;
- E_s and I_s, are the modulus of elasticity and the moment of inertia of the substrate, respectively.

The constants C_1 and C_4 in eq. (4.14) reads:

$$C_1 = \Delta N_f(0) + \frac{\Delta \varepsilon_0}{f_2}, \tag{4.19}$$

$$C_4 = +\frac{1}{\beta} \cdot \left(\Delta T_f(0) + \frac{\Delta \chi_1}{a_2} + \frac{a_3 \cdot \Delta \varepsilon_1}{a_2 \cdot f_2} + \frac{\lambda \cdot a_3 \cdot C_1}{a_1 \cdot \lambda^4 + a_2} \right) + C_3, \tag{4.20}$$

where:

$$C_3 = \left(\Delta M_f(0) + y_f \cdot \Delta N_f(0) \right) - \left[-\frac{\Delta \chi_0}{a_2} - \frac{a_3 \cdot \Delta \varepsilon_0}{a_2 \cdot f_2} + \frac{a_3 \cdot C_1}{a_1 \cdot \lambda^4 + a_2} \right]. \tag{4.21}$$

The quantities $\Delta N_f(0)$, $\Delta T_f(0)$ and $\Delta M_f(0)$ are function of the discontinuity type (Figure 6-1). As an example, without external loads applied directly to the strengthening end,: $\Delta N_f(0) = \Delta T_f(0) = \Delta M_f(0) = 0$.

6.2.3 Design check of the delamination strength

(1) As long as the maximum value of the peeling stress, σ, and shear stress, τ, are known with reference to the relevant load combination at the ULS, the check of the delamination strength can be performed comparing the principal value, σ_I, with the design adhesive resistance:

$$\sigma_I = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2} \right)^2 + \tau^2} \le \frac{f_{ak}}{\gamma_a \cdot \gamma_{Rd}} \cdot \eta. \tag{4.22}$$

(2) The characteristic value of the adhesive resistance, f_{ak}, can be evaluated through experimental tests on a *standard* specimen, such as the single shear lap specimen (ISO 4587:2003(E)). In eq. (4.22) the partial factor of the adhesive, γ_a, is given in Section 3.2.1.3, the conversion factor, η, in Section 3.3, and the model partial factor, γ_{Rd}, in Table 3-3 of Section 3.2.1.4.

7 STRENGTHENING OF FATIGUE SENSITIVE ELEMENTS

7.1 FATIGUE FAILURE MODES

(1)P In metallic structures strengthened by FRP materials, the fatigue failure modes are the following:

- fatigue delamination of the FRP reinforcement;
- fatigue damage of the metallic substrate.

7.2 FATIGUE DELAMINATION OF THE STRENGTHENING

(1)P The ends of the FRP reinforcement and the locations where geometric discontinuities (cracks) takes place are the zones of the adhesive joint most sensitive to fatigue damage, because of stress concentration.

(2) It is recommended to locate the strengthening ends in zones of low stress in the beam (as an example close to the beam supports) in order to reduce the stress concentration in the adhesive joint.

(3) No accurate and reliable model (such as the *S-N* curve) is currently available for the evaluation of the fatigue resistance of the adhesive joint. If no experimental data on the fatigue resistance are available, the fatigue limit can be assumed, approximately, equal to 20 ÷ 30% of the static failure strength.

7.3 FATIGUE DAMAGE OF THE METALLIC SUBSTRATE

(1) In the following, reference is made to riveted composite steel sections. This kind of joint technology was widely used in the first half of the XX century. Several structures realized with this technique are still in service and they frequently show fatigue damage. The proposed design approach makes use of the fracture mechanics concepts.

7.3.1 Introduction

(1)P Fatigue crack propagation in the metallic substrate is function of the effective stress range, $\Delta\sigma_{\text{eff}}$:

$$\Delta\sigma_{\text{eff}} = \sigma_{\max} - \sigma_{\min,\text{eff}}, \tag{5.1}$$

where the symbols are illustrated in Figure 7-1 and eqs. (5.2) and (5.3).

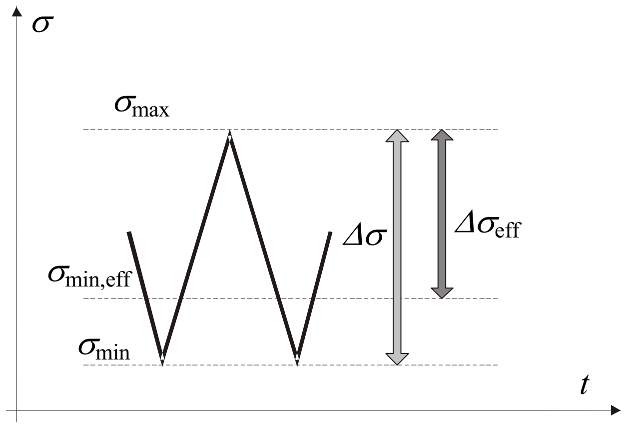


Figure 7-1 – The effective stress range.

(2)P The minimum effective stress, $\sigma_{min,eff}$, is function of the stress ratio, R:

$$R = \frac{\sigma_{min}}{\sigma_{max}} . \tag{5.2}$$

(3) The minimum effective stress, $\sigma_{min,eff}$, can be evaluated by the following expression:

$$\sigma_{min,eff} = \sigma_{max} \cdot \max\left[R; \frac{1}{1+\alpha} \cdot \left(1 + \frac{R \cdot \sigma_{max}}{\sigma_0} \right) \right], \tag{5.3}$$

where:

- α is the plastic constraint factor ($\alpha = 1$ for plane stress; $\alpha = 3$ for plane strain);
- $\sigma_0 \approx ½ \left(f_{yd} + f_{td} \right)$ is the mean value between the yielding and failure stress.

(4) The effective stress intensity factor range, ΔK_{eff}, is evaluated as:

$$\Delta K_{eff} = \Delta\sigma_{eff} \cdot \rho \cdot Y \cdot \sqrt{\pi \cdot a} = \left(\sigma_{max} - \sigma_{min,eff} \right) \cdot \rho \cdot Y \cdot \sqrt{\pi \cdot a} , \tag{5.4}$$

where a is the crack length and Y is the non-dimensional stress intensity factor.

(5) The non-dimensional stress intensity factor range, Y, can be evaluated without taking into account the reinforcement since it is assumed that the patch does not cover the crack. It can be evaluated either by a simplified analytical procedure or in a numerical way. By means of the simplified analytical procedure the factor Y is computed, as shown in Figure 7-2, replacing the structural element by a cracked plate of appropriate dimensions.

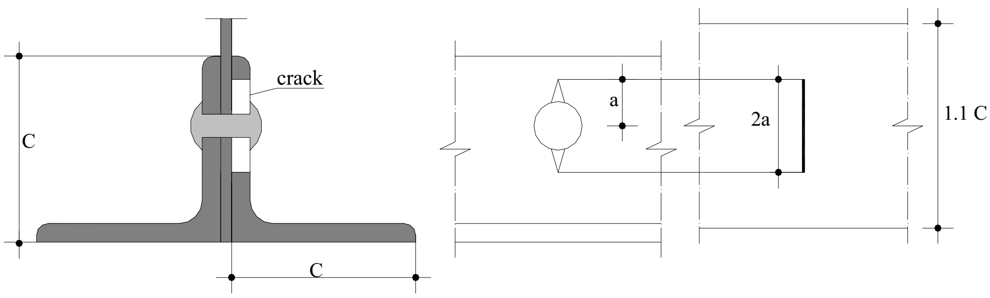


Figure 7-2 – Evaluation of the non-dimensional stress intensity factor *Y*.

(6)P Stopping crack propagation is achieved if the effective stress intensity factor range, ΔK_{eff}, is lower than the corresponding threshold value, $\Delta K_{eff,th}$:

$$\Delta K_{eff} \le \frac{\Delta K_{eff,th}}{\gamma_s} , \tag{5.5}$$

where the partial factor γ_s must be assumed according to § 3.2.1.4.

(7) Modern steel shows a threshold level $\Delta K_{eff,th}$ between 90 N/mm$^{3/2}$ (R = 0.7) and 190 N/mm$^{3/2}$ (R < 0.15). Older steels show greater $\Delta K_{eff,th}$. As an example, for R = 0.7 a $\Delta K_{eff,th}$ between 145 N/mm$^{3/2}$ e 180 N/mm$^{3/2}$ was recorded for older steels.

7.3.2 Basis of design

(1)P Following are the objectives that are pursued with the FRP strengthening of a fatigue damaged metallic riveted structure:

- the stress range $\Delta\sigma$ is lowered compared to the un-strengthened configuration;
- compressive stresses are introduced in the metallic substrate if prestressed laminas are adopted;
- crack bridging is achieved if the crack is covered by the composite patch;

A "global" effect can be associated to the reduction of $\Delta\sigma$ and to the application of compressive stresses to the strengthened element. Moreover, a "local" effect is associated to the crack bridging.

(2) The reduction of the stress range $\Delta\sigma$ can be estimated through the parameter ρ, defined as:

- structural element under tensile loading $\rho = \dfrac{E_s \cdot A_s}{E_s \cdot A_s + 2 \cdot E_f \cdot A_f}$, (5.6)

- structural element under flexure load $\rho = \dfrac{I_s \cdot y_{g1}}{I \cdot y_s}$, (5.7)

where:

- $E_s \cdot A_s$ is the axial stiffness of the substrate;
- $E_f \cdot A_f$ is the axial stiffness of the reinforcement;
- I_s is the moment of inertia of the metallic cross section;
- y_{g1} is the distance from the centroid of the strengthened section to the adhesive interface (Figure 7-3);
- I is the moment of inertia of the strengthened cross section;
- y_s is the distance from the tensile side to the centroid of the unstrengthened metallic section (Figure 7-3).

The value $\rho \cdot \Delta\sigma_{eff}$ must then be used in the evaluation of the effective stress range in the strengthened element (eq. (5.4)), where $\Delta\sigma_{eff}$ is the stress range of the unstrengthened element.

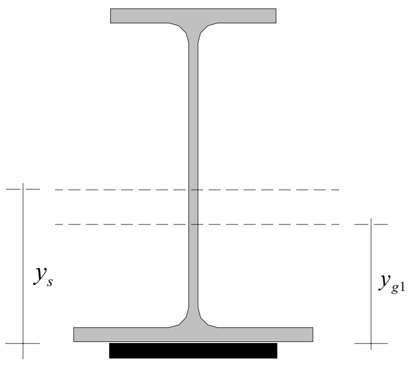


Figure 7-3 – Geometric parameters of the strengthened section.

(3) From a general point of view, a small decrease of the stress range $\Delta\sigma$ is achieved since the contribution of the FRP strengthening to the extensional or flexural stiffness of the reinforced beam is marginal.

(4) Moreover, it is not possible to obtain an efficient, or even partial, crack bridging due to the presence of rivets or bolts in the structural element. Figure 7-4 shows a possible strengthening configuration of a tension flange of a metallic beam where an efficient crack bridging is not possible due to the rivet location.

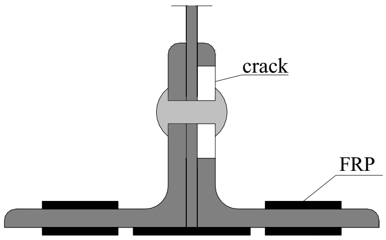


Figure 7-4 – An example of fatigue reinforcement configuration.

(5)P The application of prestressed lamina prior to bonding is a very important issue (or mandatory in several cases) for fatigue problems, since the compressive stresses, σ_{sp}, introduced in the metallic beam by the pretension reduce significantly the stress ratio R:

$$R = \frac{\sigma_{\min} - \sigma_{sp}}{\sigma_{\max} - \sigma_{sp}}. \quad (5.8)$$

The reduction of the stress ratio produces a decrease of the crack propagation rate up to, eventually, crack stop.

(6)P Experimental evidence shows that fatigue crack propagation is confined to the metallic substrate and does not propagate to the FRP material. On the other hand, crack propagation produces a partial strengthening delamination, as shown in Figure 7-5. At present no accurate and reliable models are available to quantify the phenomenon.

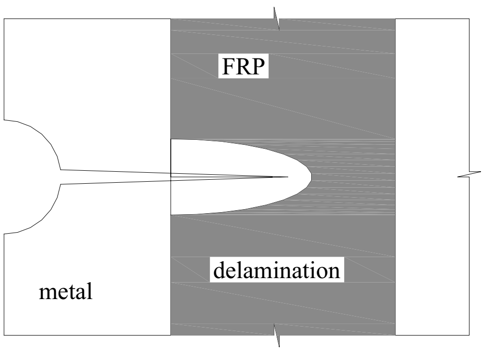


Figure 7-5 – Partial fatigue delamination of the strengthening.

(7)P The design of fatigue damaged metallic structural element, strengthened by FRP material, can be carried out by the *S-N* curves or the fracture mechanics law. At the moment *S-N* curves are available with reference to few structural details.

7.3.3 Design of the fatigue reinforcement

(1) The proposed method follows the fracture mechanics concepts to evaluate the required pretension stress level of the FRP lamina, σ_{fp}, in order to block fatigue crack propagation. The method considers global effects only, neglecting the local effect due to crack bridging.

(2) With reference to the flow chart reported in Figure 7-6, the proposed technique allows the evaluation of the stress level in the substrate, σ_{sp}, in order to block crack propagation, as long as the stress field, the mechanical properties of the materials and the substrate strengthening geometry are known. The condition to be satisfied is: $\Delta K_{eff} = \frac{\Delta K_{eff,th}}{\gamma_s}$.

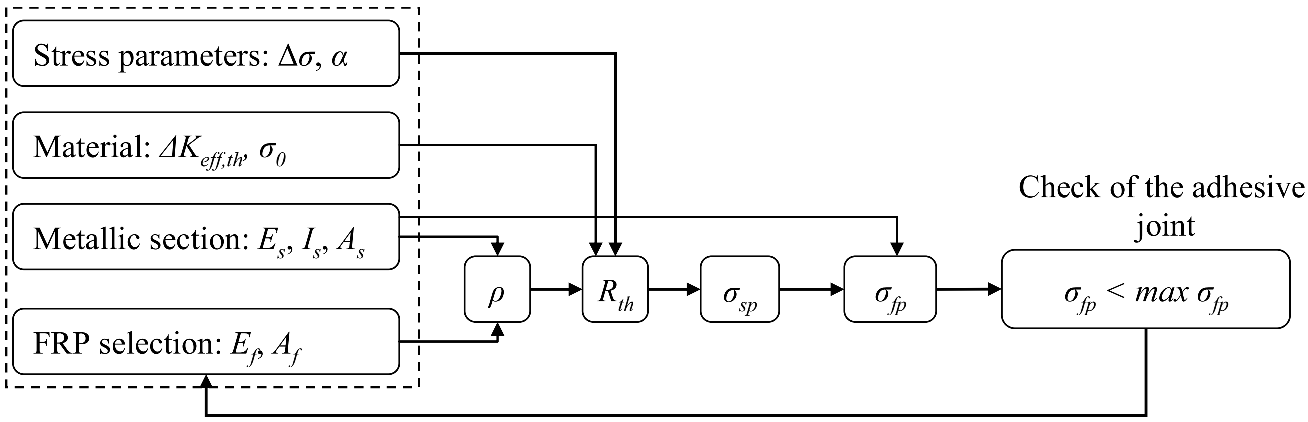


Figure 7-6 – Iterative procedure for the fatigue design of prestressed FRP lamina.

(3) The parameter R_{th} can be evaluated as:

$$\Delta\sigma = \frac{\eta \cdot (1 - R_{th}) \cdot \sigma_0 \cdot \alpha}{2 \cdot R_{th} \cdot \gamma_{Rd}} \cdot \left(1 - \sqrt{1 - \frac{\Delta K_{eff,th}}{\gamma_s} \cdot \frac{4 \cdot R_{th} \cdot (1+\alpha)}{\alpha^2 \cdot \sigma_0 \cdot \rho \cdot \sqrt{\pi \cdot a}}} \right). \tag{5.9}$$

In eq. (5.9) the partial factor of the substrate, γ_s, is given in Section 3.2.1.1, the conversion factor, η, in Sections 3.3.1 and 3.3.2, and the model partial factor, γ_{Rd}, in Table 3-3 and Section 3.2.1.4.

A conservative estimation of the parameter $\Delta K_{eff,th}$ is 100 N/mm$^{3/2}$. The plastic constraint factor can be assumed equal to: $\alpha = \sqrt{2 \cdot \sqrt{2}} = 1.68$.

(4) Finally, the compressive stress, σ_{sp}, to be applied to metallic element in order to obtain a stress ratio R_{th}, is computed by the following relationship:

$$\sigma_{sp} = \frac{\rho}{\gamma_{Rd} \cdot (1 - R)} \cdot (\sigma_{min} - R_{th} \cdot \sigma_{max}). \tag{5.10}$$

(5) The pretension stress, σ_{fp}, in the reinforcement can be evaluated by standard Strength of Material relationships.

(6)P It is recommended to check the adhesive joint at the release of the pretension of the FRP plate in order to avoid delamination or damage of the adhesive layer.

8 INSTALLATION, MONITORING AND MAINTENANCE

8.1 GENERAL CONSIDERATIONS

(1)P Strengthened structures require inspection and monitoring, (i) during and immediately after installation of the strengthening system and (ii) during the lifetime at regular intervals. In case of FRP strengthened structures, such monitoring activity is particularly important, since no reliable information is available about the long-term behaviour of the FRP strengthening system.

8.2 RECOMMENDATIONS FOR THE INSTALLATION

8.2.1 General considerations

(1)P The correct installation of FRP strengthening systems depends upon several factors, among which:
- storage of the materials;
- substrate preparation;
- preparation of the strengthening materials;
- environmental conditions (temperature, humidity);
- job site management (cleanliness, protection measures against accidental events).

(2) Strengthening materials (fibers, resins, composites) must be stored under the conditions specified by the producer.

(3) Preparation of the strengthening system must strictly follow the information given by the producer. For example, adhesive resins are often obtained by the mixture of two parts, whose quantitative ratio must strictly follow the information provided by the producer.

(4) Temperature and humidity conditions can determine good FRP/ metallic element adhesion. In particular, the temperature of both the structure and the environment influences the time required for the development of a given percentage of the final strength in the adhesive joint. Producers give information about the temperature ranges allowing a correct development of the chemical reactions needed to reach the strength of the adhesive joint and such prescriptions must be strictly satisfied. Excessive humidity may adversely affect the final strength of the adhesive joint and its durability. Therefore, the adhesive joint must be carefully protected against contact with water, during the time required for strength development.

(5)P Appropriate management of the job site is important for a good final quality of the strengthening system. In particular, attention must be paid to cleaning the substrate surface, avoiding the presence of particles that adversely affect the quality of the FRP/substrate bond.

8.2.2 Preparation of the substrate surface

(1) The correct preparation of surfaces is fundamental for the good adhesive bond of the composite FRP system to the metallic substrate. Preparation involves treating the substrate surface in the following steps:

- Remove coatings (e.g. painting), slag and other corrosion products.

This operation may be carried out using wire brushing or grid blasting in order to mechanically remove slag. Special care is required in the case of brittle members (cast iron), avoiding percussions as well as the use of hammer drills.

- Degrease with solvent
Suitable solvents, carefully selected and used, can be adopted to remove grease and avoid widespread contamination of the bonding surface.

- Abrasion
Dry or wet sand-blasting might be used to expose a chemically active surface prior to bonding. Debris should be removed using only water.

- Drying the surface
If the surface is wet at the end of the abrasion and the cleaning phase, then it must be immediately dried in order to avoid the quick formation of oxide layers on the exposed surface.

- Chemical etching
In the case of the formation of oxide layers, such as for galvanized or stainless steel, acid etching and the following neutralization of etching products are required. This is not required in the case of steel and cast iron.

- Primer
A first layer of adhesive resin should be applied as soon as possible after the cleaning of the substrate surface, and always within two hours after cleaning. In some cases, the use of primer is required to make the existing metallic surface compatible with the FRP material.

- Fill irregularities
Small irregularities on the metallic surface can be filled using the primer and the subsequent resin layer. Larger irregularities, such as in the case of strongly corroded or highly irregular metallic members, can be filled using an *ad hoc* resin layer.

8.2.3 Structural detailing

(1) An end anchorage length not smaller than 200 mm is suggested to be used for the FRP strengthening, unless mechanical anchorages are provided and adequately certified.

(2) When using carbon fibers, direct contact with the metallic surface should be avoided by interposing an insulating material layer, such as a layer of GFRP, in order to avoid galvanic corrosion.

(3) Whenever the provided anchorage length is insufficient and it is not possible to increase it, then a special end detailing of the FRP strengthening must be studied, in order to reduce normal and shear stresses in the adhesive layer close to the FRP strengthening terminal parts. Figure 8-1 shows some examples of local detailing appropriate in such a case. Figure 8-1(a) and Figure 8-1(b) show the reduction of the FRP plate thickness, while Figure 8-1(c) represents a case of mechanical anchorage.

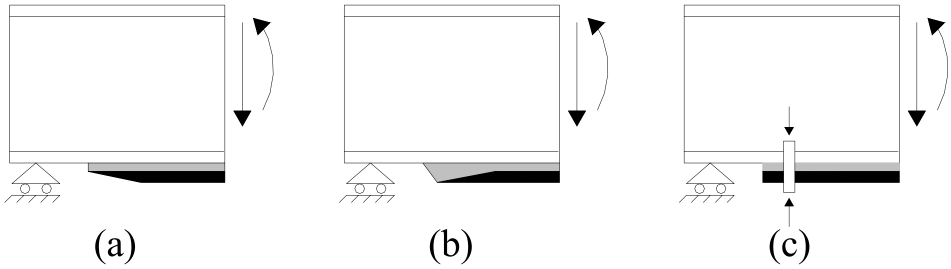


Figure 8-1 – Typical end detailing of the FRP strengthening system, allowing for a reduction of the debonding stresses

(4) Special care in the detailing is required when existing riveted or bolted connections are given. Preferred solutions to be adopted in the case of flexural strengthening of a riveted or bolted beam are shown in Figure 8-2.

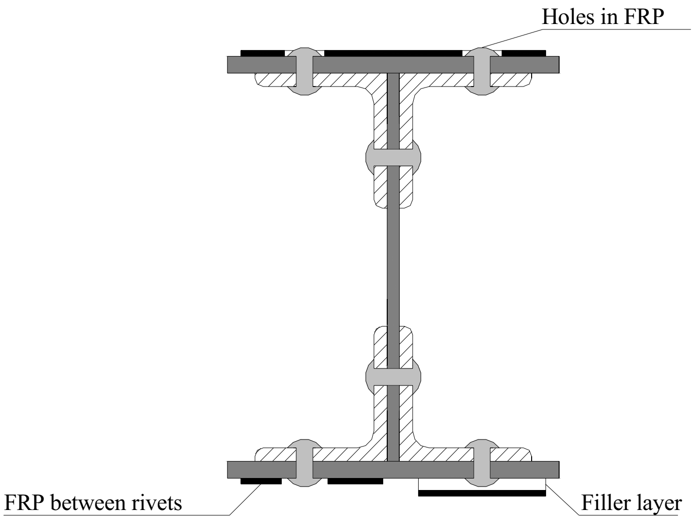


Figure 8-2 – Flexural strengthening of a riveted or bolted beam.

8.3 QUALITY CONTROL DURING INSTALLATION

8.3.1 General considerations

(1) Quality control during installation of the FRP strengthening system may include semi-destructive or non-destructive tests.

(2) If pre-cured FRP strengthening is used, qualification tests on the composite materials should have already been carried out by the producer. In the case of FRP strengthening systems cured *in situ*, laboratory tests are required. In any case, specimens should be prepared in order to evaluate strength against delamination.

(3) In order to carry out semi-destructive tests, additional special testing zones (*witnesses*) strengthened with FRP must be prepared. The total area of *witnesses* should be not less than 0.5% of the actual strengthened area, and in any case not less than 0.1 m^2. *Witnesses* zones should be subdivided in rectangular areas of dimensions 500 mm x 200 mm. The FRP strengthening system should be applied in these special "testing" zones using the same materials and procedures adopted

for the actual strengthening system. In addition, *witnesses* must be exposed to the same environmental conditions as those affecting the actual strengthening system. If more than one rectangular special "testing" zone is required, then they must be uniformly distributed. In any case, the *witnesses* must be selected in such a way that the subsequent execution of the semi-destructive tests does not affect the response of the strengthened member.

8.3.2 Semi-destructive tests

(1) Pull-off tests and/or additional tests among those listed hereafter may be carried out. Semi-destructive tests shall be carried out on *witnesses* and, where possible, in non-critical strengthened areas at the rate of one test for every 5 m^2 of the strengthened area, and in any case, not less than 2 for each type of test.

(2) Pull-off test. The test is suitable for assessing the correct preparation of the substrate surface and to verify compatibility of the selected adhesive resin with the metallic substrate. It is carried out by using a 20 mm thick circular steel plate with a diameter of at least 40 mm. The FRP reinforcement is cut with a core drill rotating at a speed of at least 2500 rpm and particular care shall be taken in order to avoid heating of the FRP system.

(3) Shear tearing test. The test is particularly suited to check the quality of the bond between the FRP and the substrate. It may be carried out only when it is possible to pull a portion of the FRP system in its plane located close to an edge debonded from the substrate.

(4) Torsion tearing test. The test is carried out using a special device that can apply a torque moment.

8.3.3 Non-destructive tests

(1) Different types of non-destructive tests have been developed, aiming at identifying various types of defects. In particular, non-destructive tests are used to identify the presence of voids in the adhesive layer. There is no type of non-destructive test that can identify a weak bond originating by rust or grease on the metallic surface.

(2) Non destructive tests may be used to characterize the uniformity of FRP application starting from adequate two-dimensional survey of the strengthened surface with a different spatial resolution as a function of the strengthening area (see Table 8-1).

Table 8-1 – Minimum resolution for defects thickness to be identified with non destructive tests.

Shear stress transfer at interface	Example	Non destructive test	Surface mapping grid	Minimum resolution for defects thickness
absent	wrappings, with the exception of the overlapping area in single-layer application	Optional	250 mm	3.0 mm
weak	central area of very extensive plane reinforcement	Optional	250 mm	3.0 mm
moderate	central area of longitudinal flexural strengthening	Suggested	100 mm	0.5 mm
critical	anchorage areas, overlapping areas between layers, shear strengthening, interface areas with connectors, areas with large roughness or cracks in the substrate	Required	50 mm	0.1 mm

(3) High-frequency ultrasonic testing. Defects in the bond, local delamination or presence of voids in the adhesive layer can be identified by means of ultrasonic tests because they affect the transit time of ultrasonic waves. Specialist equipment and personnel are required.

(3) Stimulated acoustic testing. Similar to the previous tests, they use lower frequency pulses.

(4) Acoustic emission tests. The technique is based on the acoustic emission (AE) method and allows the assessment of damage inside a structural member subject to loads by listening to and recording the noise generated by either the formation of cracks or delamination phenomena that propagates as elastic waves. The technique belongs to the passive systems, which activates if anomalous conditions occur. Such a test is particularly suitable for detecting defects in the application of FRP composites as well as the occurrence of delamination from the substrate.

(5) Thermographic tests. They measure small temperature differences on the surface of a structure thus identifying interruptions to the heat flows caused by voids within the composite or adhesive. It can be either a passive or active system, by exploiting either natural daily or seasonal variations or artificially heating up and cooling down the surface. These tests are effective only for FRP systems with low thermal conductivity and can not be applied to carbon or metallic FRP strengthening systems unless special precautions are taken. The heat developed during the test shall be smaller than the glass transition temperature of the FRP system.

8.3.4 Personnel qualification

(1) Personnel in charge of the tests shall have one of the three qualification levels specified in Table 8-2, according to UNI EN 473 and UNI EN 45013.

Table 8-2 – Qualification levels to perform semi-destructive and non-destructive tests.

Level 1	Proper knowledge of tests equipment; performing tests; recording and classifying test results according to a given procedure; writing a report on test results.
Level 2	Choosing the way of performing the test; defining the application limits of the test for which the level 2 technician is certified; understanding test specifications and translating them into practical test instructions suitable to the in-situ working conditions; adjusting and calibrating test equipments; performing and controlling the test; interpreting and evaluating test results according to the specifications to comply with; preparing written test instructions for level 1 personnel; performing and supervising all level 1 functions; training personnel of level 1; organizing test results and writing the final report.
Level 3	Be in charge of a laboratory facility; establishing and validating test techniques and procedures; interpreting specifications and procedures; having the skill to evaluate and understand test results according to existing specifications; having a sufficient practical knowledge of materials, production methods and installation technology of the system to be tested to be able to choose appropriate methods, establish techniques and collaborate in the definition of acceptance criteria when they are not pre-established; be knowledgeable in different application fields; being able to lead personnel of level 1 and 2.

(2) The personnel qualification is also ruled according to UNI EN 45013 – General criteria regarding Certification Body proposed to Personnel Qualification.

8.4 INSPECTION AND MAINTENANCE

(1) Composite materials do not require special maintenance interventions. If the layer of surface resin is lost due to abrasion or environmental degradation, it can be replaced using a compatible resin.

(2) The existing metallic structure must be submitted to normal inspection and maintenance operations. In particular, it is recommended to renew the paint before its failing, in order to avoid grit-blasting of the structure, which will require adequate protection of the FRP.

9 APPENDIX A

9.1 REVIEW OF STRENGTHENING INTERVENTIONS CARRIED OUT ON EXISTING STRUCTURES

There are several examples of the FRP strengthening to restore the load bearing capacity of bridges and other metallic structures throughout Europe, in Italy as well as in the USA. They include several different application methods: *wet lay up* and bonding of pultruded (eventually prestressed) lamina.
Several representative cases will now be discussed, dealing with the main types of strengthening carried out on bridges, arches, bridge girders as well as compressed elements.
These include examples of bridges that are currently being used in the United Kingdom, *Hythe* (Luke 2001), *Tickford* (Lane 2000), *King Street* (Farmer et al. 2001), *Acton* (Moy et al. 2002) as well as the *Slattocks Canal* bridge (Luke 2001). Further noteworthy examples include the strengthening of the cast iron struts of an Underground tunnel in London (Moy et al 200, Leonard 2000), the longitudinal and transversal bridges girders on the *Interstate Highway*- 704 (Miller 2001), *State Route* 82 (Chacon et al. 2004) in *Delaware*, (USA) as well as on the *State Highway* 92 in *Iowa* (Phares et al. 2003).
The only example of a similar application in Italy is a cast iron pedestrian bridge in Venice, the *Corona* bridge, strengthened by AFRP (Zerbo 2001, Ceriolo & Di Tommaso 2002, Zaffaroni et al. 2002, Bastianini et al. 2004).

9.1.1 Hythe Bridge

The Hythe Bridge, built in 1874 over the River Thames, is a two spans bridge with a clear span equal to 7.8 m. It was realized by 8 reversed T cast iron girders and transversal masonry arches that sustain the traffic loading. The main girders were prestressed by pultruded laminas (two per each edge of the girders) and CFRP sheets. The compressive stress level was selected in order to remove the tensile stresses in the cast iron girders for a given traffic volume (vehicle weights up to 400 kN). The prestressed technique used: anchored as well as movable terminals, jacks and a fixed framework. The fixed terminals were used to guarantee the adhesion of the FRP to the existing structure, taking into account the stress concentration at the edges of the plate. Stresses were transferred to the FRP by the bolts connected to the movable terminals bonded to the FRP. The use of the jacks as well as the moveable terminals gave a correct strengthening alignment of the cast iron struts. These FRP interventions were carried out prior to the adhesive curing.

Figure 9-1 – Hythe Bridge (England).

9.1.2 Tickford Bridge

The Tickford Bridge was built of cast iron by Thomas Wilson in 1810 and is located near Newport Pagnell (UK). It is an elegant structure, made up of six girders with a span equal to 18.29m, sustaining a deck with six arches having spans of 7.2m. The structure subjected to the intense vehicular traffic crossing Newport Pagnell and was strengthened using pre-impregnated CFRP sheets. The FRP system was selected since it can be rapidly applied to the curved surfaces of the arches. In order to prevent galvanic corrosion, a polyester sheet was placed between the substrate and the FRP. A total of 14 layers of FRP were laminated on the substrate for a final thickness equal to 10mm. The whole intervention used materials that had the same color as the existing structure, thus making it unnoticeable.

9.1.3 Slattocks Canal Bridge

The *Slattocks* traffic bridge was built in 1936 over the Rochdale canal (UK). It is a single span bridge with a span equal to 7.6m supported by twelve double "T" laminated girders. Two CFRP laminas, 4mm thick and 100mm wide, were applied to the bottom side of the existing girders. This increased the load bearing capacity of the bridge from 170 kN to 400 kN.

Figure 9-2 – Slattocks Canal Bridge (England).

9.1.4 Acton Bridge

The Acton Bridge has a timber deck with steel girders and is part of the London Underground. The aim of the strengthening was to reduce by about 25 % the stress level due to the service loads. This increased the fatigue resistance of the metallic structure, subject to intense load cycles induced by traffic volume. The bridge was strengthened by applying pultruded CFRP elements to the tension flanges of the girders. Monitoring of the strengthening under service conditions confirmed its effectiveness.

9.1.5 King Street Bridge

The King Street Bridge is a railway bridge built in England in 1870. It was subsequently adapted for vehicular traffic. It is characterized by six metallic girders that sustain masonry arches with span equal to 2m long. In 1900, the metallic girders' spans were reduced from 8.9m to 5.9m by inserting an additional support. FRP strengthening restored the original design of the bridge. Two laminas, 170mm length and 33mm thick, were applied to the bottom side of the girders. Pre-impregnated CFRP laminas were applied longitudinally, while GFRP laminas transversally, in order to prevent galvanic corrosion. The strengthening was applied without having to move the existing structure by using screws.

9.1.6 Corona Bridge

The Corona Bridge, 1850, was built from three cast iron arches, with spans equal to 4m. It was the first example of a cast iron pedestrian bridge built in Venice. It was restored in 2001 in order to stop any further damages as well as to reduce the vulnerability of the structure to impacts loading. The arches and their decorative openings were strengthened with aramid tri-axial sheets and mono-directional strips, respectively. It is worth noting that the strengthening was applied to the lateral bridge arches for purely aesthetical reasons, with the central arch being strengthened on both sides. The resin was grey in color, in order to make the strengthening unnoticeable.

Figure 9-3 – Arch of Corona Bridge after the FRP strengthening.

9.1.7 Compressed structure of a London Underground tunnel

Another example of FRP structural strengthening is the reinforcement of a London Underground tunnel. The structure, consisting of 18 cast iron struts with cross-shaped sections, was strengthened by bonding 26 layers of high modulus CFRP to the flanges of each member. The effectiveness of the repair was initially tested on elements with the same characteristics as the structure to be strengthened. The results highlighted that the FRP strengthening would increase the load bearing capacity of the structure by approximately 40%.

9.1.8 Christina Creek Bridge

The *Christina Creek* Bridge (I-704) in Delaware (USA) is characterized by a mixed steel – concrete structure that has been recently FRP strengthened. The strengthening was proposed by the Department of Transport of the University of Delaware, with a purely scientific aim, studying a completely intact structure. The final goal was to study the effectiveness of a FRP strengthening on a composite steel – concrete structure, that are widely diffused in the USA. The other aims of the strengthening included verifying the durability of the adhesive when exposed to environmental agents as well as fatigue. The bridge is characterized by three simply supported girders, with a span equal to 35m. A set of six CFRP pultruded laminas, 1.5m length, was bonded in parallel to the bottom side of the tension flanges. In order to ensure a perfect match of the laminas as well as to make them easier to apply, a staggered joint was created. Moreover, the strips was beveled to 45° at the reinforcement end in order to reduce stress concentration. They were bonded to the beams with bi-component adhesive, using an air gun.
In order to prevent galvanic corrosion, a GFRP sheet was placed between the CFRP lamina and the substrate.

Figure 9-4 – CFRP pultruded laminates applied using removable clamps to bridge I-704 in the USA (Miller et al. 2001).

The results of this study highlighted that following the application of CFRP pultruded laminas, there was a 11.6% increase in stiffness and a 10% decrease in deformation.

Table 9-1 – Summary of the Structural Strengthening of Metallic Bridges.

Strengthening Context	Type	Material	Strengthening Method	Final Result	Date
Tickford Bridge 1810, Newport Pagnell, England. Vehicle Bridge	Concrete arches and Structure	Cast Iron	Bonding of pre-impregnated CFRP laminas with polyester underlay	Reduction of stresses on the original material	1999
Hyhte Bridge 1874, Oxford, England. Vehicle Bridge	Reversed T girders and masonry arches	Cast Iron	Pre-stressing of CFRP laminas	Elimination of tensile stress on the girders due to variation of loading conditions. Traffic up to 40 t.	2000
Slattocks Canal Bridge 1936, England. Vehicle Bridge	T girders and concrete deck	Steel Early	Bonding of CFRP laminas	Increase of the load bearing capacity, traffic up to 40t	2000
King Street. Bridge 1870, England. Vehicle Bridge	T girders and masonry arches	Cast Iron	Bonding of pre-impregnated CFRP and GFRP laminas, longitudinally and transversally, respectively. Use of temporary struts to support to deck, thus allowing load transfer of some of the permanent loading to the reinforcement.	Increase of the load bearing capacity, traffic up to 40t	2000
Acton Bridge London Underground, England. Railway Bridge	T girders and timber deck	Steel Early	Bonding of CFRP laminas	Reduction of strain on the original material, increase in service life and fatigue resistance.	2000
Corona Bridge 1851, Venice, Italy. Pedestrian Bridge	Cast iron arches	Cast Iron	Bonding of aramid tri-axial laminas and mono-directional strips	Reduction of vulnerability to thermal shocks and impacts loading	2002

%

Christina Creek Bridge I-704, Delaware, USA. Vehicle Bridge	Composite steel-concrete girders and deck	Steel	Bonding of CFRP laminas	Pilot project to analyze the fatigue resistance as well as sensitivity to environmental agents.	2000
Ashland Bridge Delaware, USA. Vehicle Bridge	Composite steel-concrete girders and deck	Steel	Bonding of CFRP laminas	Stress reduction on the original material to increase the fatigue lifetime	2002
7838.5S092 Bridge Pottawattamie County, Delaware, USA. Vehicle Bridge	Composite steel-concrete girders and deck	Steel	Bonding of CFRP laminas	Stress reduction on the original material to increase the fatigue lifetime.	2003

9.2 CURRENT STATE OF ART: EXPERIMENTAL TEST RESULTS

Studies carried out on the external FRP strengthening of metallic structures have focused on improving performance levels, including the load bearing capacity, stiffness, ductility, environmental durability, as well as fatigue resistance.

Tests carried out on different types of metallic structural elements, including steel bridges girders and columns, briefly described in the previous Sections, have highlighted the effectiveness of FRP strengthening. The metallic substrate preparation as well as the adhesive properties are important factors to be taken into account. The tests carried out can be divided into groups based on their function: debonding analysis, bending and fatigue behavior as well as analysis of strengthened compressed elements.

For each of these topics, there are suggested references.

9.2.1 Failure due to delamination

The failure modes of a FRP strengthened element have been classified by Buyukozturk et al. (2003), as follows: failure due to buckling of the compressed beam flange, failure due to shear buckling of the web, failure of the FRP strengthening and composite debonding. Failure of the resin has been the subject of numerous studies with several precautionary measures being identified. Failure of the composite-to-metal joint may occur in zones with geometrical discontinuities, where a strong concentration of stresses in the adhesive layer determines a significant load transfer from the metallic substrate to the FRP lamina. This typically occurs at the edge of the composite strengthening (where it is interrupted) and/or in correspondence to cracks in the structural element that has been repaired through the application of FRP laminas. One of the most widely used solutions for avoiding debonding of the composite was proposed by Vinson & Sierakowski (1987). The reinforcement ends are beveled to 45° and tapered as used in aeronautical engineering (de Bruyne 1944, Hart-Smith 2001). Liu et al (2001) suggested to transversally wrap the laminas along the longitudinal axis in correspondence to the FRP extremity.

Bonding durability is conditioned by galvanic corrosion, which often occurs from the galvanic interaction of two different materials: carbon and steel (Aylor 1993, Bellucci 1991, 1992). A practical solution consists of adding a layer of GFRP between the metallic substrate and the FRP (West 2001, Tavakkolizadeh & Saadatmanesh 2001). For pultruded elements, an insulating layer can be directly added to the bonding surface of the metallic substrate (McKnight 1994, Karbhari & Shulley 1995). It also seems appropriate to avoid using hydrolysis resins, which releases hydroxyl ions in structures that are exposed to either de-icing solutions or marine environments with alkaline solutions (pH > 7) (Buyukozturk & Gunes 2000, Tucker & Brown 1989, Boyd et al. 1991, Tavakkolizadeh & Saadatmanesh 2001).

9.2.2 Flexural strengthening

Tests were carried out on beams subjected to bending moments in order to verify the possibility to strengthen damaged sections of metallic structures as well as to increase the load bearing capacity of undamaged elements. Pultruded and pre-impregnated CFRP, bonded with either epoxy-based adhesives or bi-component resins (for steel or cast iron beams) have been used as well as multidirectional *on-site* aramid impregnated wraps (for cast iron beams).
Sen et al (2001) tested artificially damaged steel beams. Damage was introduced by loading the tension flange of a steel beam beyond the yielding point in order to produce permanent deformation. The tests are then carried out using CFRP laminas, with various thicknesses, and steels with different yielding points.
Beams artificially damaged at the midspan section, by cutting the tension flange, were used in three-point bending tests carried out by Liu et al. (2001).
Bending tests were also carried out on damaged beams from a dismantled existing bridge. The results are reported in Mertz & Gillespie (1996).
Photiou et al. (2004) performed tests on strengthened metallic beams, using a hybrid system of carbon and glass *prepregs* bonded to the steel substrate with an adhesive layer. The layer of glass between the CFRP and the steel not only has the role of preventing corrosion but it also allows for higher failure resistance due to the smoothing of the shear stresses through the adhesive layer. The system was applied to a beam subjected to four-point bending test. The section, a box type, was artificially damaged by taking out half of the thickness of the tension flange.
Flexure tests of integral FRP-strengthened metallic beams (undamaged) have been carried out by Moy (2002), Moy & Nikoukar (2002). The study analyses the behavior during the curing period in correspondence to defined loading cycles.
The results highlight the substantial loss of any benefit, should the structure be loaded prior to the adhesive has completely cured.

9.2.3 Fatigue strengthening

Tests have been carried out by Gillespie et al. (1996), Miller et al. (2001), Bassetti et al. (2000), Nozaka (2002), Jones & Civjan (2003), Matta et al. (2004) and Gillespie et al. (1996). Miller verifies the adhesive layer fatigue resistance, by comparing FRP behavior to that of a normal fatigue sensitive steel details, classified by the American Guidelines as category C'. The results highlight that the bonded FRP has a greater resistance than the details of category C'. It can therefore be assumed that the CFRP –steel bonding does not constitute the most sensitive detail, in terms of fatigue durability, when the adhesive is subjected to only elastic deformations.

9.2.4 Strengthening of compressed elements

The load bearing capacity of struts with a circular hollow section can be increased by CFRP wraps as proposed by Carolin (2003).
The problem was also studied by Liu et al. (2003) with reference to concrete filled steel bridge piers that were strengthened with FRP.
Cecchi & Zerbo (2004) carried out an analysis of the stability of thin walls strengthened with FRP laminas applied along the flanges of HEA profiles.

10 APPENDIX B: REFERENCES

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