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**FRP STRENGTHENING OF MASONRY ARCHES:
ANALYSIS OF LOCAL MECHANISMS AND GLOBAL BEHAVIOUR**

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**FRP STRENGTHENING OF MASONRY ARCHES:
ANALYSIS OF LOCAL MECHANISMS AND GLOBAL
BEHAVIOUR**

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Sommario

Negli ultimi due decenni, i materiali compositi FRP (Fibre-Reinforced Polymers) sono stati adottati anche nel rinforzo di costruzioni murarie sia moderne sia antiche (edifici, ponti, torri), nonché di vari elementi strutturali (pareti, archi e volte, pile e colonne).

Il rinforzo di archi e volte in muratura con FRP può contribuire in modo significativo a migliorare la loro portanza in stato limite, attivando meccanismi locali a livello di materiali e interfaccia, ma comporta anche un cambiamento dei meccanismi di collasso della struttura originale, dal momento che il rinforzo impedisce la tipica rottura fragile causata dalla formazione di meccanismi a cerniera.

Nonostante un crescente numero di studi riguardanti il rinforzo di strutture murarie con FRP, le indagini sono tuttavia ancora limitate, se confrontate con applicazioni su calcestruzzo armato. Inoltre, pochi codici e raccomandazioni sono attualmente disponibili.

Partendo da questi punti, il presente lavoro affronta l'indagine sperimentale di tre meccanismi locali coinvolti nel collasso di archi rinforzati con FRP: il comportamento dell'interfaccia nel caso di tensioni normali alla superficie (distacco del composito osservato in strutture rinforzate all'intradosso) è stato investigato mediante l'esecuzione di un ampio numero di prove combinate eseguite su mattoni pieni in laterizio (prove a flessione, compressione, trazione indiretta e pull-off), allo scopo di calibrare sperimentalmente eventuali correlazioni fra i relativi parametri di resistenza e di osservare eventuali influenze del tipo di fibra e della presenza del primer sul comportamento nella prova di pull-off; il comportamento nel caso di azioni tangenziali è stato studiato mediante l'esecuzione di dieci Double-lap Shear Tests su mattoni pieni in laterizio, al fine di calibrare energia di frattura e legge bond-slip per rinforzo in fibra di carbonio e fibra di vetro (inoltre, è stata proposta una semplice legge bond-slip); il comportamento nel caso di azioni miste (quattordici prove eseguite) è stato investigato mediante l'adattamento di una prova sviluppata per applicazioni di FRP su calcestruzzo armato, nota come V-shape Peel Test, a mattoni in laterizio usati come substrato, allo scopo di riprodurre a scala locale le condizioni relative allo scorrimento a taglio sul giunto di archi con rinforzo estradosale. Infine, vari casi studio, riguardanti archi e volte in muratura, a dimensioni reali oppure scalate, sono stati raccolti da letteratura al fine di comparare i risultati sperimentali con i modelli interpretativi del comportamento globale della struttura a collasso attualmente disponibili.

Abstract

In the last two decades, FRP (Fibre-Reinforced Polymers) composite materials have been adopted for strengthening and repair of both modern and historic masonry constructions (buildings, bridges, towers) and structural components (walls, arches and vaults, piers and columns).

Strengthening of masonry brick arches and vaults with FRP laminates can contribute significantly in the improvement of their structural capacity at a limit state, by activating local mechanisms both at material and interface levels, but also modifies the collapse mechanisms of the original structures, as the reinforcement prevents the typical brittle failure due to the formation of hinge-mechanisms.

Despite the increasing number of specific studies on FRP reinforcement of masonry structures, investigations are still limited if compared to reinforced concrete applications. Moreover, few codes and recommendations are currently available.

Starting from these points, the present work deals with experimental investigations on three local mechanisms involved by the collapse of FRP-reinforced masonry arches: the interface behaviour in the case of stresses normal to the surface (FRP detachment observed in structures with intrados reinforcement) has been investigated through the execution of a large number of combined tests on solid clay bricks (flexural, compressive, splitting and pull-off tests) aimed at experimentally calibrate possible correlations among the corresponding strength parameters and to observed possible influences of the fibres type and the presence of primer on the pull-off behaviour; the bond behaviour has been investigated by performing ten Double-lap Shear Tests on solid clay bricks, aimed at calibrating fracture energy value and bond-slip law for both carbon FRP and glass FRP reinforcement (moreover, a simple exponential-based bond-slip law has been proposed); the mixed-mode behaviour (fourteen tested samples) has been investigated by adapting a test setup developed for FRP applications on reinforced concrete, known as V-shape Peel Test, to clay substrate, in order to reproduce on a local scale the conditions related to the shear sliding failure of arches with extrados reinforcement. Finally, several case studies, concerning real-scale or scaled brick masonry arches and vaults reinforced at their intrados or extrados, have been collected from literature in order to compare the experimental results to the available interpretative models of the global behaviour of the structure at failure.

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Chapter 1

Introduction

1.1 Background

As a matter of fact, innovative materials are often rapidly proposed in the current practice, even for restoration of historical constructions where preservation criteria have to be taken into account; on the other hand, variability and complexity of masonry structures and typologies make difficult to correctly deal with appropriate structural models and interventions, since they should be based on the knowledge of both existing and new materials, and their interaction under environmental and loading conditions.

In the last two decades, FRP (Fibre-Reinforced Polymers) composite materials have been adopted for strengthening and repair of both modern and historic masonry constructions (buildings, bridges, towers) and structural components (walls, arches and vaults, piers and columns). The choice of these techniques is mainly aimed to increase the ultimate capacity of masonry structures and components, often by altering mechanisms at collapse, which can involve further resisting phenomena.

Strengthening of masonry brick arches and vaults with FRP laminates can contribute significantly in the improvement of their structural capacity at limit state, by activating local mechanisms both at material and interface levels. Interventions on masonry arches and vaults by means of FRP laminates modify the collapse mechanisms of the original structures: the reinforcement prevents the typical brittle failure due to the formation of four hinges, and introduces additional strength, at a limit state, related to the local behaviour of materials and interfaces (Faccio and Foraboschi 2000, Briccoli Bati and Rovero 2000, Valluzzi *et al.* 2001, Barbieri *et al.* 2002, Foraboschi 2004, Briccoli Bati and Rovero 2008, Focacci 2008). Besides the available formulations regarding the global behaviour of the strengthened structure, an interpretation of the local phenomena, based on the calibration of analytical models through simple experimental tests, could be really significant.

Experimental studies carried out at the University of Padova allowed to characterize the behaviour of masonry barrel vaults, strengthened on their intrados or extrados by means of carbon (CFRP) and glass (GFRP) fibre-reinforced polymers (externally bonded textiles), subjected to loads applied to a quarter of the span. These studies identified the limit conditions that activate the collapse (Valluzzi *et al.* 2001).

In the case of intrados reinforcement, rupture is mainly related to the detachment of composite from masonry support. This mechanism is due to the stresses perpendicular to the interface, produced by the combination of tension force and the curved shape of the composite. A simplified model, that takes into account the structure geometry and the reinforcement properties, is commonly proposed (Valluzzi *et al.* 2001, Foraboschi 2004, Briccoli Bati and Rovero 2008) to estimate the collapse load. This model should be calibrated

on the results obtained by pull-off tests that could be performed on small samples, both in laboratory and on-site.

On the other hand, if the reinforcement is applied on the extrados, a really brittle collapse due to the shear sliding on a mortar joint can happen; in particular, for asymmetrical load condition, it occurs at the joint closest to the springer opposite to the loading point (Valluzzi *et al.* 2001, Foraboschi 2004, Briccoli Bati and Rovero 2008). In this case, it is supposed that the sliding resistance is related to a first contribution due to friction on the brick-mortar interface and a second contribution that involves the laminate-masonry interface along the perpendicular direction (a sort of dowel action). As the first contribution is supposed to be described by a simplified model based on the Coulomb law that could be calibrated by means of the rather standard triplet test, the second one requires a more complex modeling and experimental procedures to identify and characterize its behaviour.

Despite the increasing number of specific studies on FRP reinforcement of masonry structures, investigations are still limited if compared to reinforced concrete applications. Moreover, few codes and recommendations are currently available. Harmonization of test procedures and methods are still needed, in order to calibrate analytical and numerical models for design and assessment rules.

1.2 Aims and method of the present research

Starting from the above exposed background, the present work deals with experimental investigations on three local mechanisms involved by the collapse of FRP-reinforced masonry arches, namely the interface behaviour in the case of stresses normal to the surface (FRP detachment observed in structures with intrados reinforcement), of tangential stresses (plate-end or intermediate debonding) and of mixed actions (related to the shear sliding on a mortar joint observed in structures with extrados reinforcement).

The first mechanism has been investigated through the execution of a large number of combined tests on solid clay bricks (flexural, compressive, splitting and pull-off tests) aimed at experimentally calibrate possible correlations among the corresponding strength parameters. Moreover, the possible influence of the fibres type and the presence of primer on the pull-off behaviour have been included into the experimental program.

The bond behaviour has been investigated by performing ten Double-lap Shear Tests on solid clay bricks, aimed at calibrating fracture energy value and bond-slip law for both CFRP and GFP reinforcement; moreover, a simple exponential-based bond-slip law has been proposed. These tests have been modelled by Finite Elements techniques in order to carry out parametric analyses, based on experimental evidences, concerning the main parameters involved in the phenomenon.

The mixed-mode behaviour (fourteen tested samples) has been investigated by adapting a test setup developed for FRP applications on reinforced concrete, known as V-shape Peel Test, to clay substrate, in order to reproduce on a local scale the conditions related to the shear sliding failure of arches with extrados reinforcement.

Finally, several case studies, concerning real-scale or scaled brick masonry arches and vaults reinforced at their intrados or extrados, have been collected from literature in order to compare the experimental results to the available interpretative models of the global behaviour of the structure at failure.

1.3 Thesis Organization

The thesis is organized into eight chapters that, starting from the present introduction, face the main aspects of the application of FRP externally bonded textiles to masonry arches and barrel vaults: chapter 2 presents a state-of-the art review from both a research and a practical point of view; chapter 3 introduces the Data-Warehouse designed during this study in order to collect data on various research activities concerning the strengthening of masonry with composites materials; chapters 4, 5 and 6 expose the experimental activities performed to investigate the local mechanisms that involves the FRP-masonry ensemble subject to normal, tangential and mixed forces; chapter 7 deal with the currently available models developed to interpret the global structural behaviour; finally, chapter 8 summarize the main conclusions reached by the present work.

Chapter 2

State-of-the-Art

2.1 Mechanical behaviour of strengthened arches and vaults

It is known that stability and safety of curved masonry structures depend on their geometry, materials' properties and load conditions (Heyman 1982, Carbone *et al.* 2001).

The mechanical behaviour of the unstrengthened structures is well described on the basis of the Heyman's hypotheses: masonry does not resist to tension stresses (within the joints, the mortar could be absent, poor or deteriorated), masonry has indefinite compressive strength (stresses are usually lower than materials' strength and, although local crushing could be observed, it does not affect the overall equilibrium) and, finally, the low shear stresses assure that friction prevents sliding on mortar joints. On the basis of these assumptions, crisis is ruled by the formation of plastic hinges, which number and position could make the structure labile and activate collapse (Fig. 2.1 on the following page). When the resultant of the internal forces shifts from the internal core to the edge of the cross section, the section partialises and a local masonry crushing (hinge's centre) with mutual rotation of the adjacent faces could appear.

The application of a reinforcement resisting to tensile stress alters the brittle collapse mechanism of the unreinforced structure, preventing the formation of some hinges, because the edge of two adjacent faces are constrained not to split by the reinforcement's presence on that surface.

The reinforced structure, namely the two parts between the hinges, can undergo flexural performance, similarly to reinforced concrete beams (Triantafillou 1998a,b), as the line of thrust is allowed to be external to the arch thickness (Fig. 2.2 on the next page).

Therefore, crisis moves to a local level (Fig. 2.3 on the following page) and could be due to: (a) masonry crushing or fiber rupture (combined compressive and bending stresses on the cross section), (b) detachment of the adhesion system on the composite-masonry interface, (c) shear failure of joints, with sliding on the bricks-mortar interface.

Several experimental researches on brick masonry arches and barrel vaults (Valluzzi *et al.* 2001, Foraboschi 2004, Basilio 2007, Castori 2006, Briccoli Bati and Rovero 2008) pointed out that ultimate collapse is most likely due to the composite detachment (b) in case of intrados reinforcement, and to the shear sliding on a joint (c) close to the springer opposite to the loading point in case of extrados reinforcement. The shear disconnection provokes a particularly brittle failure in comparison with the reinforcement detachment, since it does not preserve the integrity of the loaded structure.

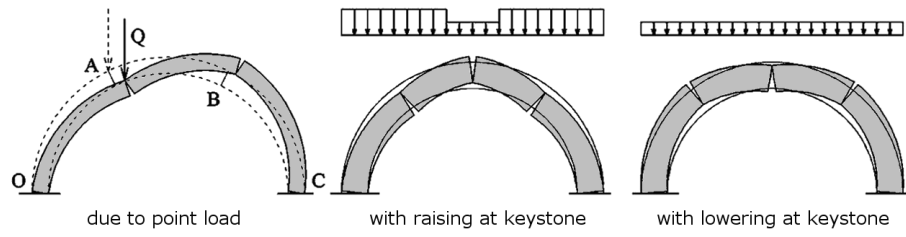


Figure 2.1: Various collapse mechanisms of unreinforced arches

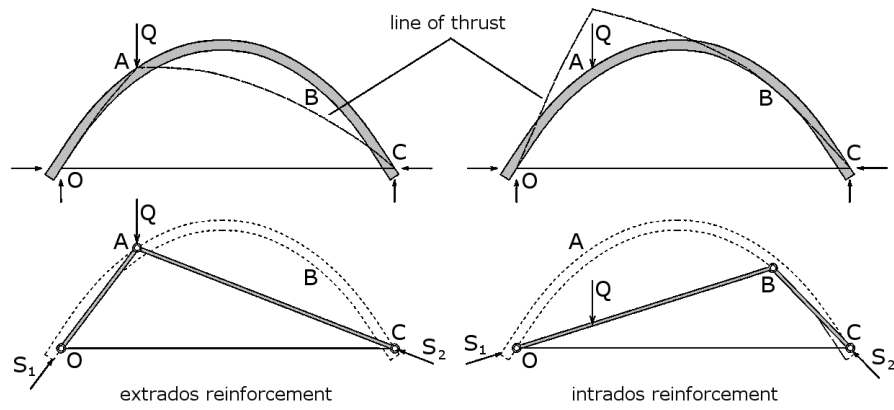


Figure 2.2: Mechanical behaviour of strengthened arches (Valluzzi *et al.* 2001)

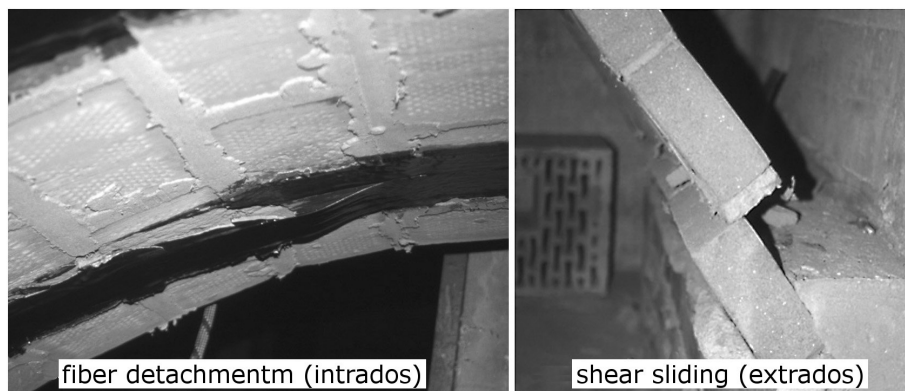


Figure 2.3: Collapse mechanisms for reinforced arches

2.1.1 Modelling of masonry crushing

As regards reinforced arches, it is possible to evaluate a quite good estimation of the ultimate strength of a masonry cross section, subject to combined compressive and bending stresses, by assuming a model based on the constitutive laws and stress and strain distributions shown in Figure 2.4. The maximum bending moment that the section could withstand, when masonry crushing occurs, can be evaluated on the basis of flexural tests performed on small-sized plain specimens (Valluzzi *et al.* 2001, see). The resulting design formulas are given in Equation (2.1), where M_{Rd} and N_{Rd} are resisting design bending moment and normal force, b and t are width and thickness of the masonry section, respectively, x is the neutral axis depth, f_k and $\varepsilon_{M,u}$ are design compressive strength and ultimate strain of masonry, E_f and A_f are elastic modulus and area of the reinforcement, and ω is the normalized area fraction.

A more simplified approach has been proposed by Foraboschi (2004), who affirmed to be adequate, on the basis of experimental evidences, adopting as stress-block a fixed compressed masonry area of one third of the arch thickness; this allows to estimate the compressive force and the tensile force acting on masonry and reinforcement, respectively, for any value of external load, and to identify the collapse as the compressive force uniformly distributed on that area equals the masonry compressive strength.

$$\frac{M_{Rd}}{bt^2 f_k} = \frac{1}{2} \omega \frac{1 - \frac{x}{t}}{\frac{x}{t}} + \frac{0.4}{\gamma_M} \frac{x}{t} \left(1 - 0.8 \frac{x}{t} \right) \quad (2.1a)$$

$$\frac{x}{t} = \frac{\gamma_M}{1.6} \left(\frac{N_{Rd}}{bt f_k} - \omega + \sqrt{\left(\omega - \frac{N_{Rd}}{bt f_k} \right)^2 + \frac{3.2}{\gamma_M} \omega} \right) \quad (2.1b)$$

$$\omega = \frac{\varepsilon_{M,u} E_f A_k}{f_k bt} \quad (2.1c)$$

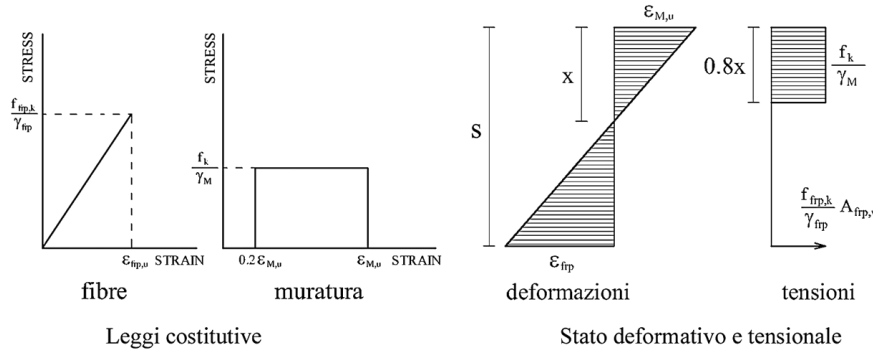


Figure 2.4: Constitutive laws and stress and strain distribution on the masonry cross section

2.1.2 Detachment of reinforcement at the composite-masonry interface

The application on a curved structure of a material designed to work in tension implies the presence of stresses perpendicular to the interface, produced by the spatial variation of the tensile force. In case of extrados reinforcement, these normal stresses are balanced by the underlying masonry, while in case of intrados reinforcement the equilibrium is ensured only by the adhesion properties of the interface. Equation (2.2) gives the relation among the normal stress σ_n , the tensile force on the reinforcement T , the intrados curvature radius R

and the composite width b_f . Therefore, the failure load can be defined as the load value that provokes a normal stress, on the critical section, comparable to the surface tensile strength measured by means of local pull-off tests (Valluzzi *et al.* 2001, Foraboschi 2004, Briccoli Bati and Rovero 2008).

$$\sigma_n = \frac{T}{b_f R} \quad (2.2)$$

2.1.3 Shear sliding on mortar joints

This mechanism is caused by the presence of shear stresses that are particularly high near the springers, in case of extrados reinforcement. The modeling of strength usually considers only the contribution due to the friction on mutual surfaces (Valluzzi *et al.* 2001, Foraboschi 2004, Briccoli Bati and Rovero 2008, Focacci 2008); this quantity is commonly evaluated by adopting a Coulomb-like law (Eq. 2.3) that assumes, during the post-critical phase, a proportionality between the shear strength V_{Rd} and the compressive force C controlled by the friction coefficient μ .

$$V_{Rd} = \mu C \quad (2.3)$$

Chapter 3

Collection of FRP-masonry experiences: Data-Warehouse

Within the framework of the RILEM Technical Committee 223-MSC (Masonry Strengthening with Composite Materials), an appropriate tool was designed in order to collect data on various research activities concerning the masonry strengthening with composites materials, as experimental work, analytical and numerical modelling, and cases study.

3.1 Main characteristics of the on-line tool

The system is basically composed by two distinct sites, one reserved to the TC members and the other open for external contributions from registered members, available at <https://contribution.rilem.isqweb.it>, that allow to manage a shared database (Fig. 3.1 on the next page).

This tool was structured to receive data belonging to researches and works concerning single structural components, as arches and vaults, columns and pillars, walls and panels, as well as global behaviour, and some particular aspects like bond, or Non-destructive Test applications and durability problems.

The main scope was to systematize results related to experimental works concerning studies on the main categories available in literature. Therefore, consistently with the current progress on these topics, some sections were fully developed (e.g. the activity layout “Experimental Work” and categories such as “Arch/Vault”, “Column/Pillar”, “Wall/Panel” and “Bond”), whereas, concerning other activities and categories, the possibility to insert generic information and descriptions and to add links and attachments was included (Fig. 3.2 on the following page).

The Data Base Management System (DBMS) allows to define some elements (Materials, Publications and Activity Layout) independently from their utilization in a specific project and, moreover, they can be picked up or copied entirely to be used for different projects, also among different users, being provided some restrictions on editing and deleting other user’s elements.

The DBMS allows to create a pdf-format summary of inserted data for any single Material and an entire Project. Moreover, it is possible to insert Definitions, to view charts (Fig. 3.3 on the next page) based on the inserted data and to share topics for discussion. Finally a Frequently Asked Questions (FAQ) section is available in the HELP section.

Basically, the system that allows to interactively manipulate data is based on the nesting tables concept, which permits a quicker work and a higher control on the process (Fig. 3.4 on page 11)



Figure 3.1: Access page of the DBMS for external contributors

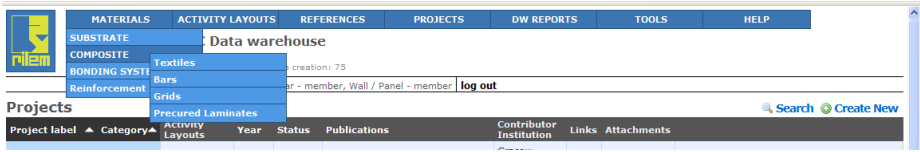


Figure 3.2: The menu bar on the upper side of the screen

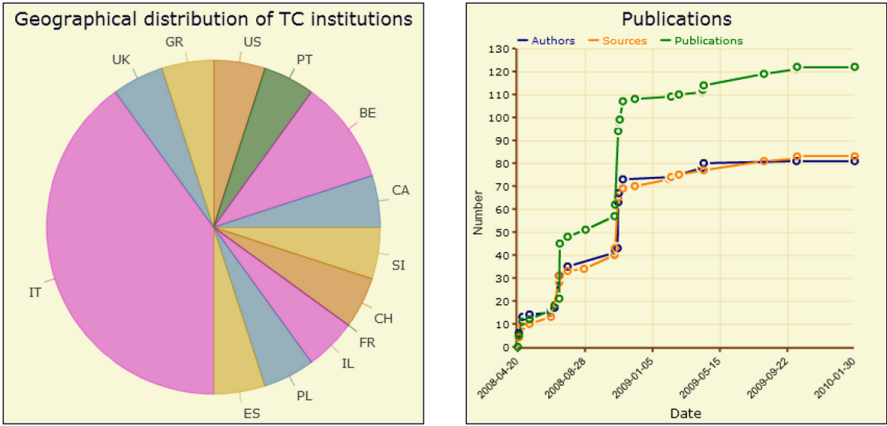


Figure 3.3: Examples of data charts

Projects

Project label

Category

Activity Layouts

Year

Status

Publications

Contributor Institution

Links

Attachments

UNIPD BARREL VAULTS (Clay Bricks single skin)

Arch / Vault

Experimental Work, Numerical Modeling

1998

CLOSED

2001 - Behaviour of brick masonry vault strengthened by FRP laminates, 2001 - Experimental analysis and modeling of masonry vaults strengthened by FRP, 1998 - Analisi sperimentale del comportamento di volte murarie rinforzate con FRP, ... (8)

University of Padova - IT

0

5

PDF Results Picture Edit Delete Show

Publications for UNIPD BARREL VAULTS (Clay Bricks single skin)

Authors

Year

Publication title

Source

Language

Links

Attachments

Panizza, M.

2006

Analisi sperimentale dei meccanismi di interazione locale di volte in murature rinforzate con tessuti FRP

University of Padova (Italy) - MS

Italian

0

0

Edit Show

Valdemarca, M.

1998

Analisi sperimentale del comportamento di volte murarie rinforzate con FRP

University of Padova (Italy) - MS

Italian

0

0

Edit Show

Authors for 1998 - Analisi sperimentale del comportamento di volte murarie rinforzate con FRP

Last name

First name INITIALS (e.g. M.J.)

Valdemarca

M.

1 Found

Valluzzi, M.R., Modena, C., Valdemarca, M.

2001

Behaviour of brick masonry vault strengthened by FRP laminates

ASCE Journal of Composites for Construction

English

0

1

Edit Show

Valluzzi, M.R., Modena, C., Garbin, E., ... (4)

2008

Bond Mechanism of Brick Masonry Vaults

Structural Faults + Repair - 2008

English

0

0

Edit Show

Valluzzi, M.R., Modena, C.

2001

Experimental analysis and modeling of masonry vaults strengthened by FRP

Structural Analysis of Historical Constructions - SAHC01

English

0

1

Edit Show

Ferrari, M.

1999

Modellazione di volte in muratura rinforzate con FRP

University of Padova (Italy) - MS

Italian

0

0

Edit Show

Valluzzi, M.R., Modena, C., Valdemarca, M.

1999

Volte murarie rinforzate mediante FRP

L'Edilizia

Italian

0

0

Edit Show

Valluzzi, M.R., Modena, C., Valdemarca, M.

1999

Volte murarie rinforzate mediante FRP. Analisi sperimentale e modellazione del comportamento

L'Edilizia

Italian

0

0

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Figure 3.4: Example of nested tables

3.2 Structure of the Data-Warehouse

The Data-Warehouse, whose structure is shown in Figure 3.6 on page 13 and whose internal relationship are shown in Figure 3.6 on page 13, is basically divided into five main sections: Materials, Activity Layouts, References, Projects and Results (Fig. 3.5 on the next page).

materials they are divided into three main groups of tables: Substrate, Bonding System and Composite; each of the former group is composed by more tables that contain all the relevant data related to the basic materials belonging to the appropriate group, such as bars, textiles, grids for Composite.

activity layouts they are divided into four main kinds: Experimental Work, Analytical Modeling, Numerical Modeling and Case Study, and were designed in order to connect materials' data with specific details concerning each experience.

references this group contains three tables, one for publications and the others dedicated to collect authors and sources.

projects it is the main table, designed to gather previously defined Activity Layouts and References, and to define specific groups of Results. Each Project belongs to a specific Category, namely "Arches/Vaults", "Bond", "Columns/Pillars", "Walls/Panels", "Global behaviour", "NDT" and "Durability".

results this table collects data directly connected to Projects, and its records cannot be defined without a corresponding record on Projects. The record structure depends on the project Category and on the related Activity Layouts.

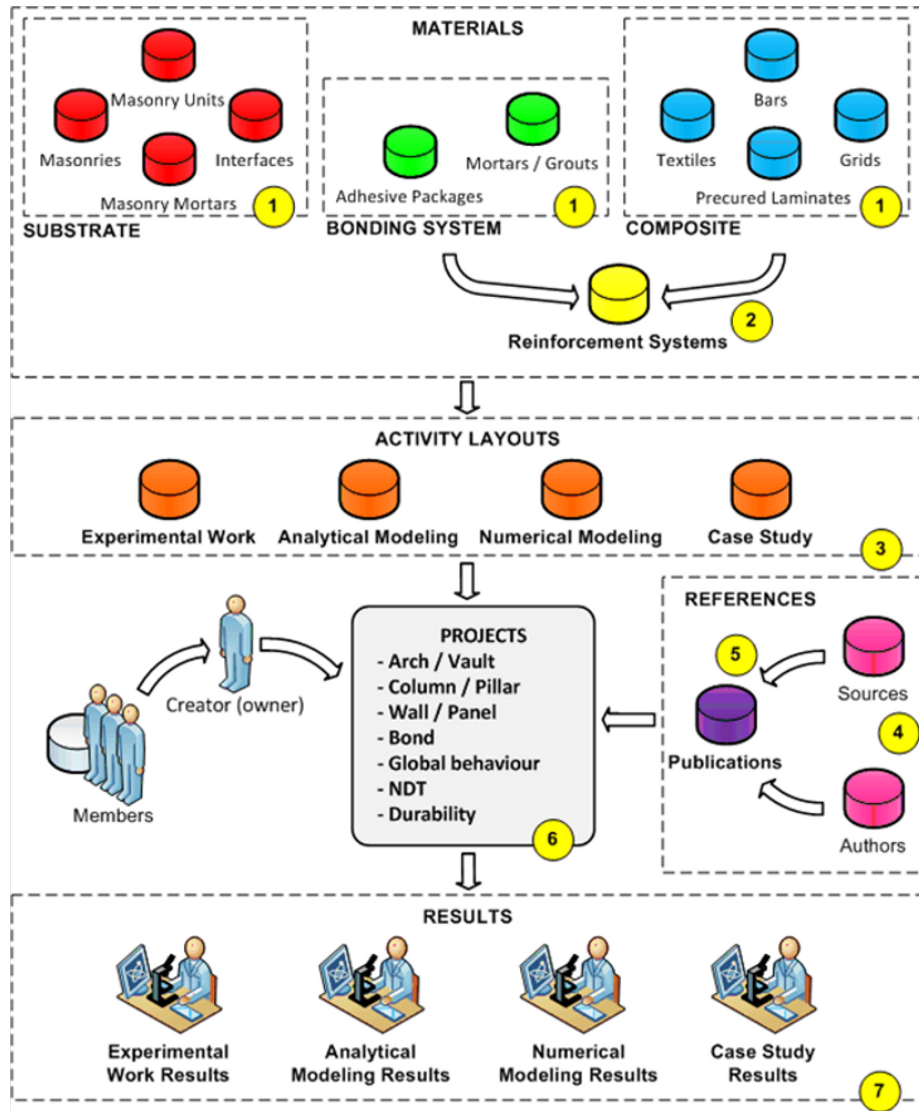


Figure 3.5: Structure of the Data-Warehouse

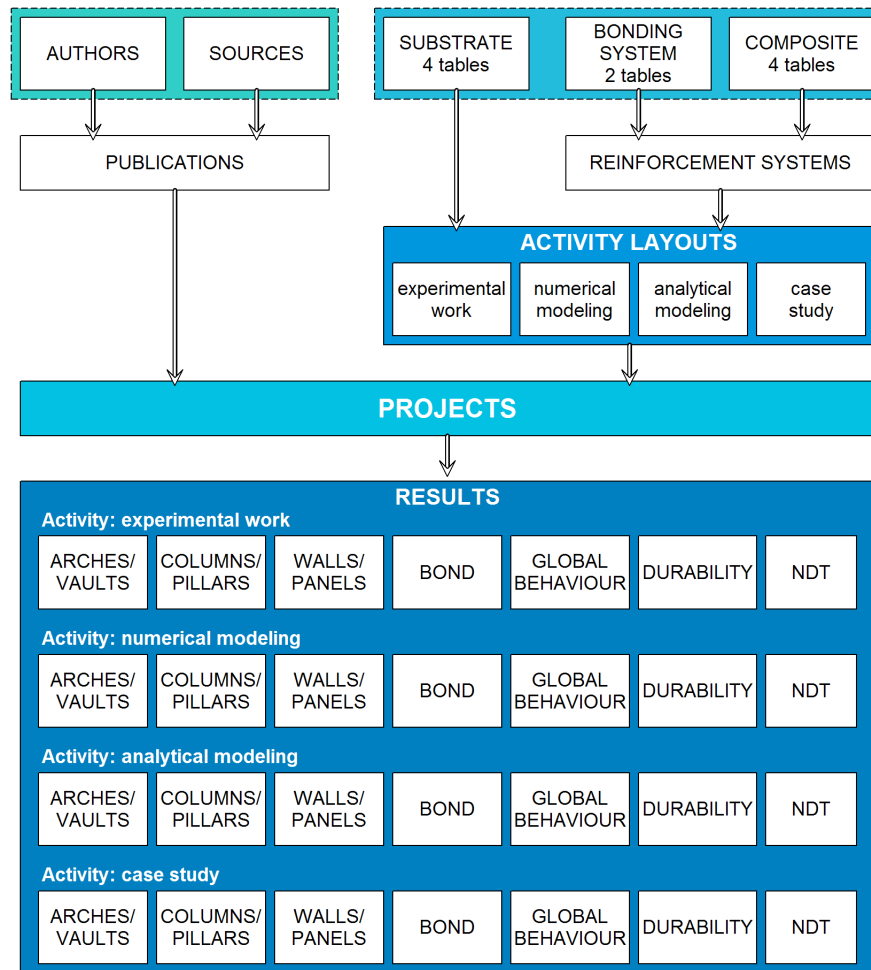


Figure 3.6: Structure of the data tables

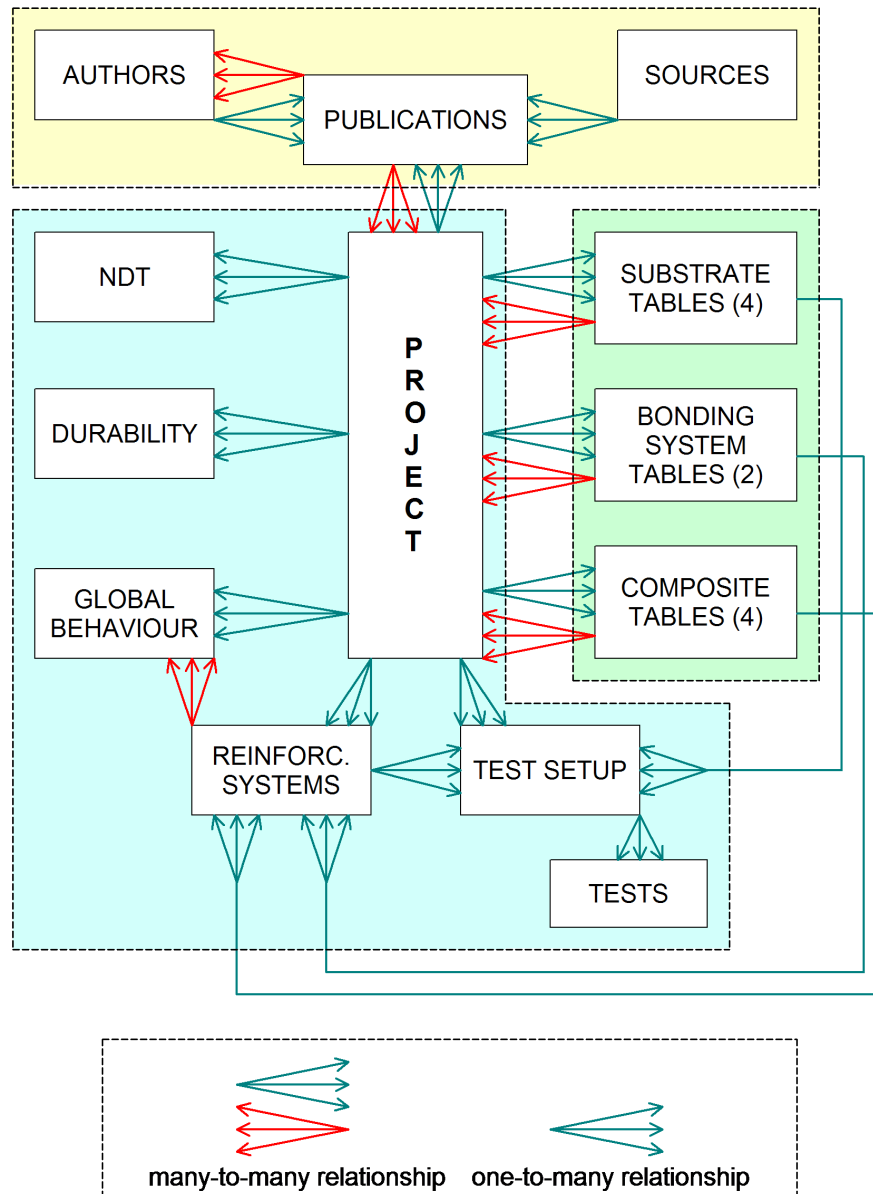


Figure 3.7: Relationships among the tables

Chapter 4

Interface behaviour in the case of normal actions

This chapter presents an experimental campaign focused on testing solid clay bricks, with externally applied FRP reinforcements. The adopted procedure, whose preliminary results have been reported in Panizza *et al.* (2009), consists in performing three types of mechanical test on the same clay element: more precisely, a three-points bending on the intact brick, a compressive test or a splitting tensile test on one of the two portion produced by the flexural failure, and two pull-off tests on the other clay portion, after the application of a layer of FRP textiles.

Two types of solid clay bricks have been used: extruded bricks and facing ones; four sets of extruded elements, three of which coming from the same manufacturer, and three sets of facing ones, two of which produced by the same factory, were used; carbon fibers (CFRP) and glass fibers (GFRP) have been applied by means of an epoxy matrix as reinforcement.

Therefore, three values of mechanical properties connected to the same masonry unit were obtained. On the basis of the first results, the possible relations between the pull-off strength and each of the other parameters (flexural, compressive and splitting tensile strength), were investigated, taking into account the variation of the clay substrate, the type of fibers (carbon or glass) and the presence, or not, of a layer of primer.

Moreover, the possible correlations among the average properties (flexural, compressive and splitting tensile strength) of the bricks tested during the present campaign and during previous experimental activities performed at the University of Padova, were investigated. The experimental matrix is reported in Table 4.1 on the following page.

4.1 Description of the experimental tests

4.1.1 Basic materials

Seven sets of solid clay bricks were used, namely four different sets of extruded bricks, three produced by Stabila Group and one by Solava SpA, and three sets of facing ones, two provided by Sant'Anselmo Furnace and one coming from San Marco Group (Tab. 4.2 on the next page).

The main difference between these two types of elements consists in the manufacturing process, being the facing bricks moulded and sanded on the surface, as they get applied without any plaster, and the other ones extruded (Fig. 4.1 on page 17); on the other hand, extruded bricks are generally more compact and stronger.

The MBrace[®] Wet lay-up System was applied, as reinforcement, on the surfaces of one

Table 4.1: Combination of the experimental tests

SERIES	Brick type	Flexural tests	Compressive tests	Splitting tests	Pull-off tests
S1	extruded	*	*		CFRP w/o primer
		*	*		CFRP with primer
		*	*		GFRP with primer
S2	facing	*	*		CFRP with primer
		*	*		GFRP with primer
S3	extruded	*	*		CFRP with primer
		*	*		GFRP with primer
		*		*	CFRP with primer
S4	facing	*	*		CFRP with primer
		*		*	CFRP with primer
S5	extruded	*	*		CFRP with primer
		*		*	CFRP with primer
S6	facing	*	*		CFRP with primer
		*		*	CFRP with primer
S7	facing	*	*		CFRP with primer
					No fibres, with primer
		*		*	GFRP with primer
					No fibres, with primer

of the two pieces of brick generated by the flexural failure, in order to perform pull-off tests: high strength carbon fibers (CFRP) and alkali-resistant glass fibers (GFRP). The main properties of the reinforcement system, obtained from producers datasheets, are reported in Table 4.3 on the next page.

Table 4.2: Solid clay bricks properties

Set code	Product name	Manufacturer	Type	Nom. dimensions
S1, S3, S5	11010R	Atesina Furnace (VR-IT)	extruded	240x115x60 mm
S2, S7	A001GL	Sant'Anselmo Furnace (VR-IT)	facing	250x120x55 mm
S4	"Vivo", rosa vivo	San Marco Group (VE-IT)	facing	250x120x55 mm
S6	A-005B	Solava SpA (AR-IT)	extruded	240x115x55 mm

4.1.2 Flexural tensile tests

According to UNI 8942/3 code, the flexural tests were performed adopting a three-point load scheme, by means of an apposite device. The distance between the support, l_s , was of 200 mm, fixed for all the samples. The flexural strength, f_t , was evaluated as conventional, assuming along the central cross section of the brick a triangular distribution for both the compressive (at the upper side) and the tensile stresses, as shown in Equation (4.1), where P_u denotes the failure load, b and h are the width and height of the specimen, respectively. The failure generally occurred near the central cross section, which is the most stressed, even if some irregular failure were observed, especially in the case of extruded bricks that revealed a slightly brittle behaviour (Fig. 4.2 on page 18). Results, in terms of mean strength for each set of samples, are given in Table 4.4 on page 18 and shown in Figure 4.3 on page 19.

Table 4.3: Reinforcement components properties

Adhesive MBrace[®] Saturant		
Characteristic compressive strength	>80	N/mm ²
Characteristic direct tensile strength	>50	N/mm ²
Maximum tensile strain	2.5	%
Tensile elastic modulus	>3000	N/mm ²
High-strength Carbon MBrace[®] C1-30		
Equivalent thickness	0.165	mm
Characteristic direct tensile strength	3430	N/mm ²
Maximum tensile strain	1.5	%
Tensile elastic modulus	230000	N/mm ²
Alkali-resistant Glass MBrace[®] G60-AR		
Equivalent thickness	0.230	mm
Characteristic direct tensile strength	1700	N/mm ²
Maximum tensile strain	2.8	%
Tensile elastic modulus	65000	N/mm ²



(a) Extruded brick



(b) Facing brick

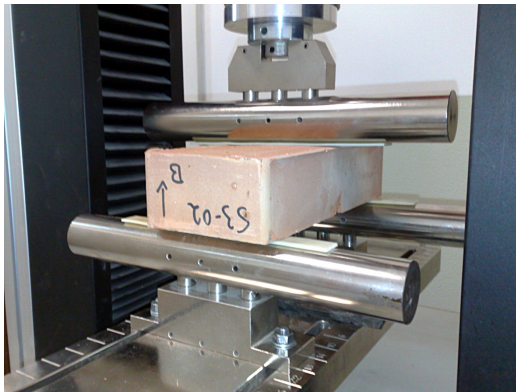
Figure 4.1: Types of solid clay bricks used during the experimental campaign

Facing bricks, compared to the extruded ones, generally presented a lower dispersion of the measured values, being the strength comparable to the other. The S3 set (extruded bricks), which showed the worst performance, presented a cooking not perfect to the core.

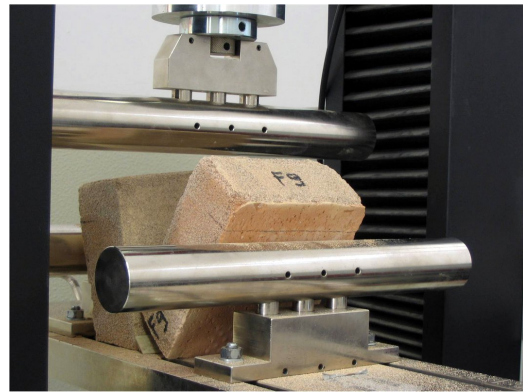
$$f_f = \frac{3P_{ul_s}}{2bh^2} \quad (4.1)$$

Table 4.4: Results of the flexural tests

SERIES	f_f N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1	4.67	0.75	16.1
S2	5.29	0.61	11.4
S3	2.97	0.66	22.3
S4	5.42	0.47	8.6
S5	3.89	1.01	26.0
S6	5.64	0.27	4.8
S7	4.75	0.57	12.0



(a) Sample ready for testing



(b) Typical brick failure

Figure 4.2: Performing of flexural tests

4.1.3 Compressive tests

Compressive tests were performed, by means of a hydraulic press commonly used to measure the strength of concrete specimens, on one of the two pieces of brick produced by the flexural failure of the intact element, being the other destined to pull-off tests; each sample was capped by a thin mortar layer in order to avoid uneven load distribution. According to EN 772-1 code, the compressive strength, f_c , was evaluated as conventional, assuming an uniform stress profile along each horizontal cross section, as shown in Equation (4.2), where P_u denotes the failure load, l and b are the length and the width of the specimen, respectively. All the tests presented a regular cubic failure (Fig. 4.4 on page 20). Results, in terms of mean strength for each set of samples, are given in Table 4.5 on the facing page and shown in Figure 4.5 on page 20. As expected, extruded bricks were stronger, their strengths covering a range from 30 to 60 N/mm², while facing bricks' values mostly varied from 15 to 25 N/mm². Two specimens of the S6 set, labelled as S1-01 and S1-02, were tested

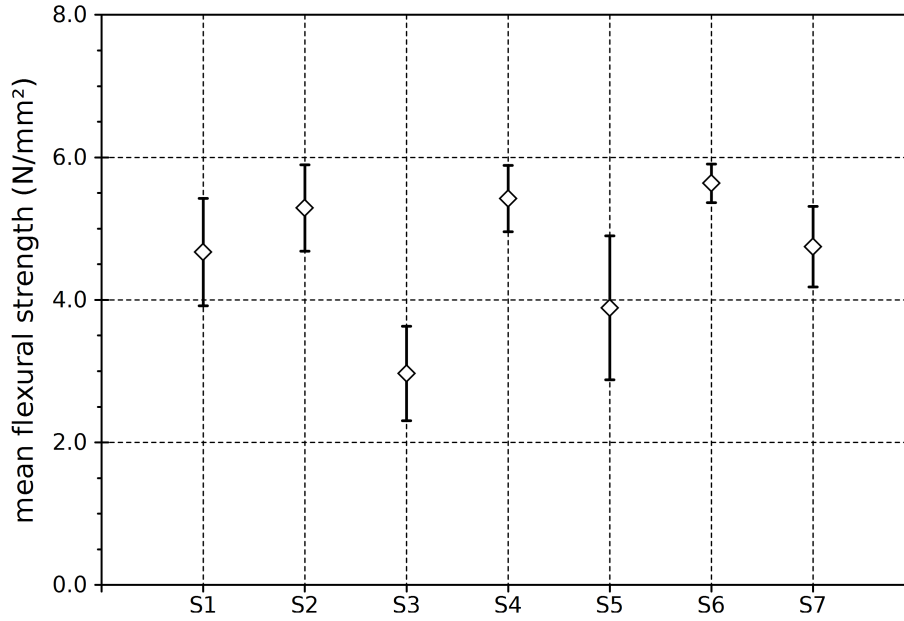


Figure 4.3: Results of flexural tests (average values)

before a complete drying of the moisture absorbed during the application of the mortar caps, therefore those values were excluded by the subsequent calculations.

$$f_c = \frac{P_u}{bl} \quad (4.2)$$

Table 4.5: Results of the compressive tests

SERIES	f_c N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1	35.36	3.19	9.0
S2	21.09	2.34	11.1
S3	33.29	2.39	7.2
S4	22.06	1.42	6.4
S5	38.39	3.18	8.3
S6	60.27	2.56	4.3
S7	16.51	1.22	7.4

4.1.4 Splitting tensile tests

Excluded S1 and S2 sets, some splitting tests were performed, according to UNI 8942/3 code, on one of the two pieces of brick (in place of compressive test) produced by the flexural failure of the intact element, being the other destined to pull-off tests; each sample was capped by a thin mortar layer in order to avoid uneven load distribution. The splitting tensile strength, f_s , was evaluated as conventional, as shown in Equation (4.3), where P_u denotes the failure load, b and h are the width and the height of the specimen, respectively. All the tests presented a regular failure localized on the loaded cross section (Fig. 4.6 on page 21), except for two samples belonging to the S3 set, which values were excluded by the subsequent calculations; in these cases, the unexpected rupture was probably due to an

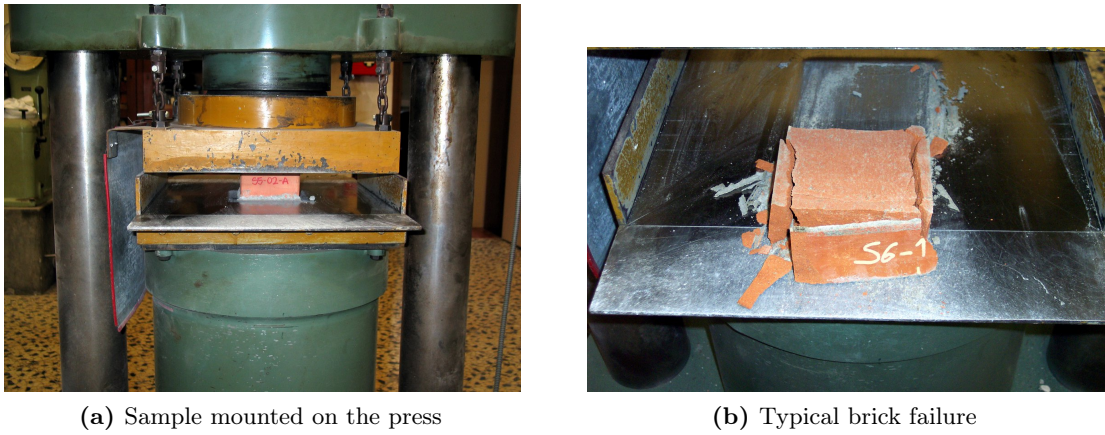


Figure 4.4: Performing of compressive tests

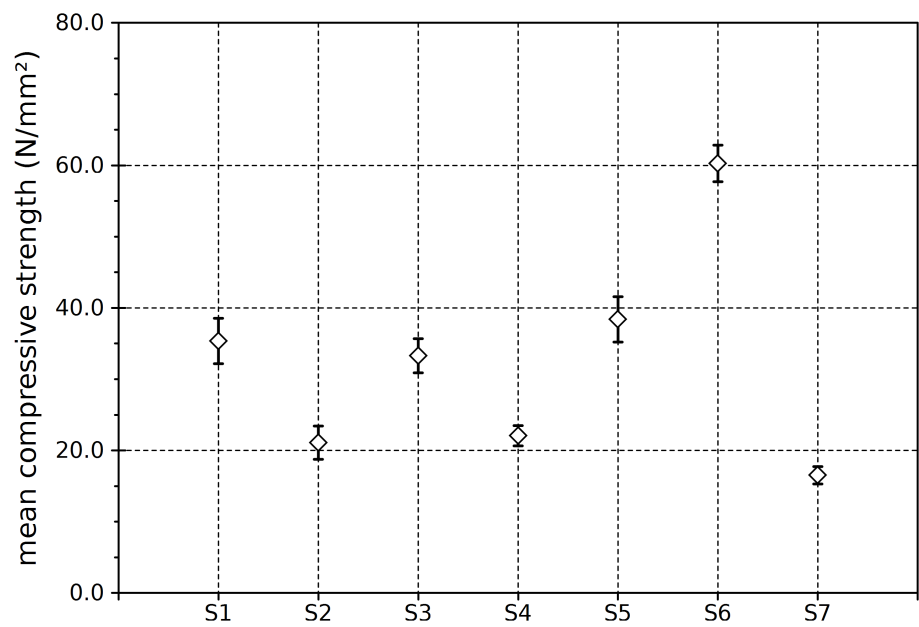


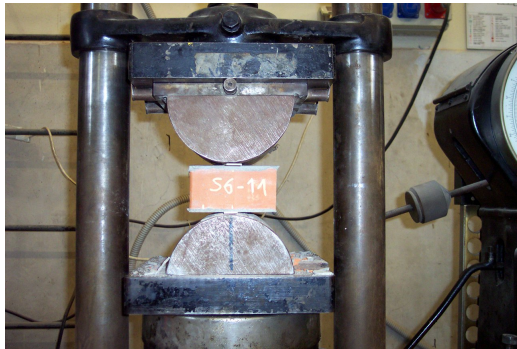
Figure 4.5: Results of compressive tests (average values)

imperfect regularization of the loaded surfaces. Results, in terms of mean strength for each set of samples, are given in Table 4.6 and shown in Figure 4.7 on the following page. Strength values were comparable for both extruded and facing bricks, excluding S3 set, which revealed an imperfect cooking, and S6 set, whose compressive strength (more than 60 N/mm²) was unusual and similar to high-strength concrete.

$$f_s = \frac{2P_u}{\pi b h} \quad (4.3)$$

Table 4.6: Results of the splitting tensile tests

SERIES	f_s N/mm ²	Stand. dev. N/mm ²	C. of V. %
S3	1.34	0.35	26.3
S4	4.02	0.23	5.8
S5	3.51	0.54	15.3
S6	5.89	0.38	6.5
S7	3.39	0.16	4.7



(a) Specimen on the test machine



(b) Failure of a brick

Figure 4.6: Performing of splitting tensile tests

4.1.5 Pull-off tests

Pull-off tests were performed on the two wider surfaces of the second clay piece originated by the flexural failure of each intact brick, being the other portion destined to either a compressive or a splitting tensile test, previously covered by composite reinforcement. This type of test, which is commonly used to verify the bond strength of overlay or repair materials applied on a different substrate (see UNI-EN 1015-12 199, ASTM C1583 2004), consists in cutting a circular area on the surface, deep into the substrate, and pulling off that area, normally to the surface, after the glueing of an apposite aluminum plate able to accommodate a pin. They were performed by means of a special portable tester, Matest E142, designed to work up to 16 kN (Fig. 4.8 on page 24).

The pull-off tensile strength, f_{p-o} , was evaluated as the mean stress on the area, according to ASTM C1583 (2004), as shown in Equation (4.4), where P_u denotes the failure load and r is the radius of the round cut. Two pull-off tests for each clay piece have been carried out,

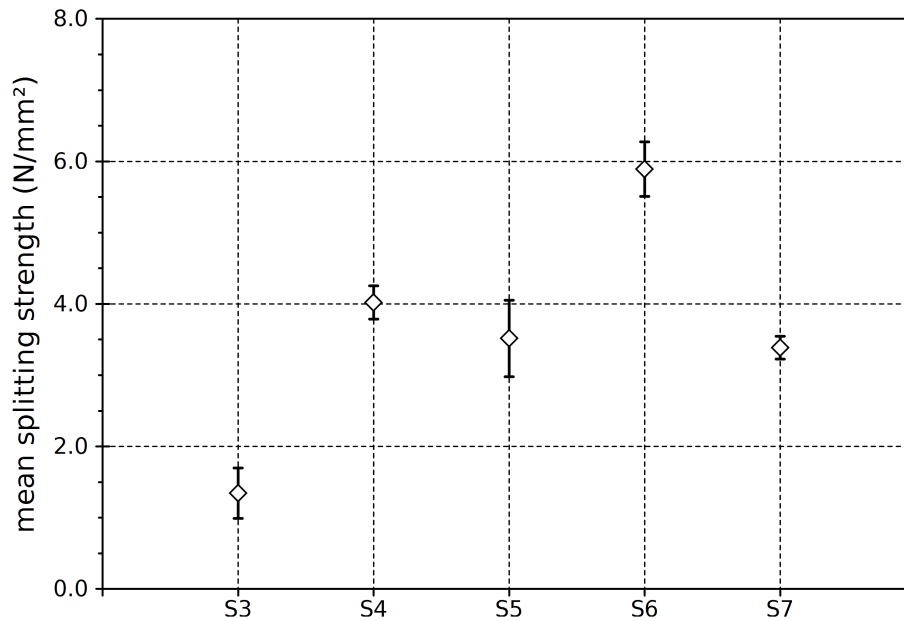


Figure 4.7: Results of splitting tensile tests (average values)

and the pertinent failures have been labelled as type A (failure in substrate), type B (bond failure at the support/overlay interface) and type C (failure in overlay or repair material) following ASTM C1583 (2004); failure type D (bond failure at epoxy/overlay interface) occurred only once, due to an incorrect application of the metallic plate.

The results of 166 pull-off tests are available: 14 concerning specimens reinforced with CFRP without applying any primer, 113 normally reinforced with CFRP, 25 with GFRP and, finally, on 14 specimens only primer and adhesive were applied, without inserting fibres. The common failure (Tab. 4.7 on page 24 and Fig. 4.10 on page 24), 155 tests, was localized into the clay support (type “A”), although in some cases it occurred at the interface between substrate and adhesive (type “B”) or within the reinforcement (type “C”): the latter event should be ascribed to an incorrect application, as it mainly affected samples reinforced with GFRP, while CFRP always broke as expected, except some samples without primer.

Results, in terms of mean strength for each set of samples, are given in Table 4.8 on page 25 and shown in Figure 4.11 on page 25. Extruded bricks showed better performances, in accordance with their compressive strength. It has to be underlined that only type “A” failures were involved in the calculations, since they refer to a property of the clay support, as well as the flexural, splitting and compressive strength; for this reason, no significant difference other than the failure mode was found related to the type of the applied reinforcement.

The summary of all the tests results is reported in Table 4.9

$$f_{p-o} = \frac{P_u}{\pi r^2} \quad (4.4)$$

Table 4.9: Results of the combined tests on clay bricks

SPECIMEN	kg/m^3	N/mm^2	N/mm^2	N/mm^2	Fibres	Primer	SIDE 1		SIDE 2	
							Fail.	f_{p-o} N/mm^2	Fail.	f_{p-o} N/mm^2
S1-01	1690	3.97	34.58		CFRP	Y	A	1.76	n.a.	n.a.
S1-02	1755	3.93	42.11		CFRP	Y	A	3.72	A	3.49

Table 4.9 – from last page

	ϱ	f_f	f_c	f_s	Fibres	Primer	Fail. (1)	f_{p-o}	Fail. (2)	f_{p-o}
S1-03	1686	4.63	33.46		CFRP	Y	A	3.04	A	4.52
S1-04	1710	6.18	36.47		CFRP	Y	A	2.90	A	2.90
S1-05	1677	5.05	38.16		CFRP	Y	A	2.07	A	2.40
S1-06	1697	4.47	34.72		CFRP	Y	A	2.88	A	2.89
S1-07	1763	5.30	33.30		GFRP	Y	C	1.85	A	3.68
S1-08	1697	5.03	33.53		GFRP	Y	C	0.61	A	3.47
S1-09	1722	4.08	40.78		GFRP	Y	C	1.29	A	3.32
S1-10	1669	3.67	31.29		GFRP	Y	n.a.	n.a.	n.a.	n.a.
S1-11	1719	4.29	32.87		CFRP	N	C	1.49	A	2.85
S1-12	1664	4.35	33.97		CFRP	N	A	3.78	A	3.10
S1-13	1673	5.77	34.40		CFRP	N	B	2.67	A	3.79
S2-01	1478	5.07	21.34		CFRP	Y	A	1.68	A	1.89
S2-02	1463	6.04	23.32		CFRP	Y	A	1.36	A	2.12
S2-03	1471	5.01	21.29		CFRP	Y	A	1.66	A	1.96
S2-04	1477	5.44	20.77		CFRP	Y	A	1.66	A	2.27
S2-05	1483	5.69	15.87		CFRP	Y	A	1.56	A	1.99
S2-06	1453	4.39	21.79		CFRP	Y	A	1.71	A	1.70
S2-07	1471	4.68	23.37		GFRP	Y	B	1.59	A	1.96
S2-08	1471	6.01	20.96		GFRP	Y	A	1.58	A	1.94
S3-01	1849	1.98	31.11		GFRP	Y	B	1.83	A	3.13
S3-02	1807	3.60	31.40		GFRP	Y	A	2.60	A	2.86
S3-03	1837	2.91	33.28		GFRP	Y	B	1.56	A	2.76
S3-04	1788	4.40	33.15		GFRP	Y	C	1.30	A	2.67
S3-05	1831	3.30		2.44	CFRP	Y	A	3.54	A	3.41
S3-06	1770	3.02		2.85	CFRP	Y	A	2.37	A	2.72
S3-07	1780	2.66		1.24	CFRP	Y	A	2.44	A	2.46
S3-08	1777	2.76		1.30	CFRP	Y	A	3.01	A	2.39
S3-09	1771	2.46		1.00	CFRP	Y	A	2.46	A	2.62
S3-10	1799	2.52		1.84	CFRP	Y	A	2.87	A	2.94
S3-11	1795	2.42	32.98		CFRP	N	B	2.90	A	2.55
S3-12	1788	3.58	37.79		CFRP	N	A	2.82	A	2.36
S4-01	1510	5.79		3.99	CFRP	Y	A	1.85	A	2.31
S4-02	1502	4.72		3.71	CFRP	Y	A	1.30	A	1.40
S4-03	1495	5.24		3.93	CFRP	Y	A	1.33	A	1.44
S4-04	1503	4.99		4.41	CFRP	Y	A	1.46	A	1.76
S4-05	1519	5.06		4.14	CFRP	Y	A	1.44	A	1.78
S4-06	1523	4.77		3.93	CFRP	Y	n.a.	n.a.	A	1.36
S4-07	1548	5.54	23.46		CFRP	Y	A	1.77	A	1.80
S4-08	1534	6.06	23.25		CFRP	Y	A	1.65	A	1.82
S4-09	1526	5.50	22.08		CFRP	Y	A	1.48	A	1.64
S4-10	1513	5.51	19.93		CFRP	Y	A	1.24	A	1.44
S4-11	1543	6.00	20.80		CFRP	N	A	1.17	A	1.54
S4-12	1543	5.88	22.84		CFRP	N	A	2.22	A	1.88
S5-01	1772	3.02	36.92		CFRP	Y	A	1.88	A	2.57
S5-02	1798	4.49	34.75		CFRP	Y	A	2.77	A	2.74
S5-03	1783	3.15	35.61		CFRP	Y	A	3.22	A	2.73
S5-04	1764	5.27	40.24		CFRP	Y	A	2.68	A	3.24
S5-05	1791	3.11	43.11		CFRP	Y	A	3.15	A	3.39
S5-06	1807	4.94	39.71		CFRP	Y	A	3.55	A	3.09
S5-07	1776	2.74		3.39	CFRP	Y	A	2.96	A	3.22
S5-08	1774	5.52		3.56	CFRP	Y	A	3.20	A	3.46
S5-09	1757	4.26		2.58	CFRP	Y	A	3.22	A	2.95
S5-10	1814	3.08		3.64	CFRP	Y	A	3.89	A	2.68
S5-11	1765	4.24		4.22	CFRP	Y	A	3.62	A	2.84
S5-12	1761	2.86		3.70	CFRP	Y	A	3.37	A	2.16
S6-01	1972	5.76	53.19		CFRP	Y	A	4.11	A	3.55
S6-02	1970	5.06	51.78		CFRP	Y	A	4.07	A	3.25
S6-03	1953	5.76	59.69		CFRP	Y	A	3.81	A	3.46
S6-04	1975	5.98	61.22		CFRP	Y	A	4.07	A	3.22
S6-05	1944	5.42	57.69		CFRP	Y	A	3.77	A	3.60
S6-06	1973	5.75	64.21		CFRP	Y	A	3.99	A	3.42
S6-07	1964	5.39	58.55		CFRP	Y	A	4.15	A	3.52
S6-08	1963	5.70		5.48	CFRP	Y	A	3.92	A	3.51
S6-09	1960	5.29		6.24	CFRP	Y	A	3.99	A	3.59
S6-10	1957	5.66		6.05	CFRP	Y	A	4.13	A	3.44
S6-11	1964	5.47		6.11	CFRP	Y	A	3.47	A	3.66
S6-12	1963	5.86		6.35	CFRP	Y	A	4.12	A	3.48
S6-13	1956	5.98		5.49	CFRP	Y	A	3.70	A	3.54
S6-14	1961	5.82		5.51	CFRP	Y	A	3.79	A	4.01
S7-01	1423	4.55	19.11		C(1)/No(2)	Y	A	1.59	A	1.41
S7-02	1419	4.78	15.24		C(1)/No(2)	Y	A	1.33	A	1.36
S7-03	1431	4.75	16.60		C(1)/No(2)	Y	A	1.48	A	1.29
S7-04	1427	5.32	16.20		C(1)/No(2)	Y	A	1.84	A	1.30
S7-05	1409	4.71	16.04		C(1)/No(2)	Y	D	1.31	A	1.38
S7-06	1418	4.07	16.13		C(1)/No(2)	Y	A	1.35	A	1.81
S7-07	1412	4.65	16.28		C(1)/No(2)	Y	A	1.38	A	1.74
S7-08	1431	5.42		3.52	G(1)/No(2)	Y	A	1.47	A	1.24
S7-09	1413	5.08		3.24	G(1)/No(2)	Y	A	1.33	A	1.72
S7-10	1430	4.85		3.49	G(1)/No(2)	Y	A	1.65	A	1.80
S7-11	1422	3.17		3.11	G(1)/No(2)	Y	A	1.57	A	1.11
S7-12	1419	5.06		3.45	G(1)/No(2)	Y	A	1.81	A	1.64
S7-13	1418	5.06		3.55	G(1)/No(2)	Y	A	1.55	A	1.50
S7-14	1417	4.98		3.36	G(1)/No(2)	Y	A	1.81	A	1.09

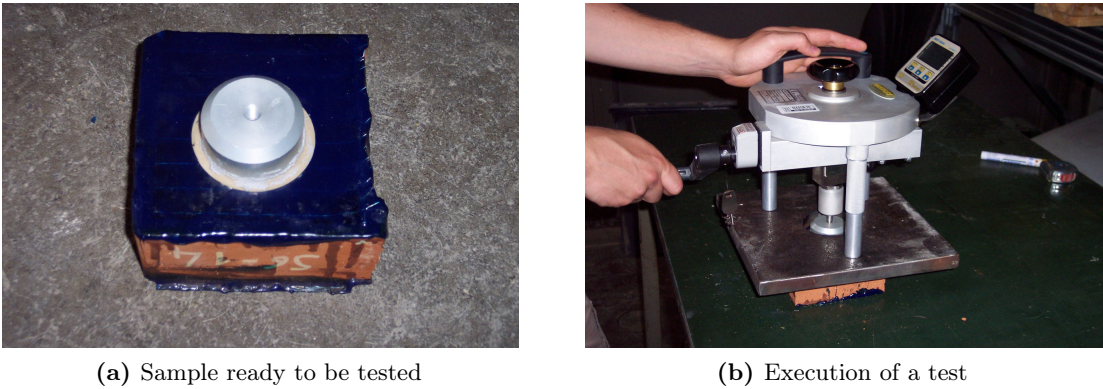


Figure 4.8: Performing of pull-off tests

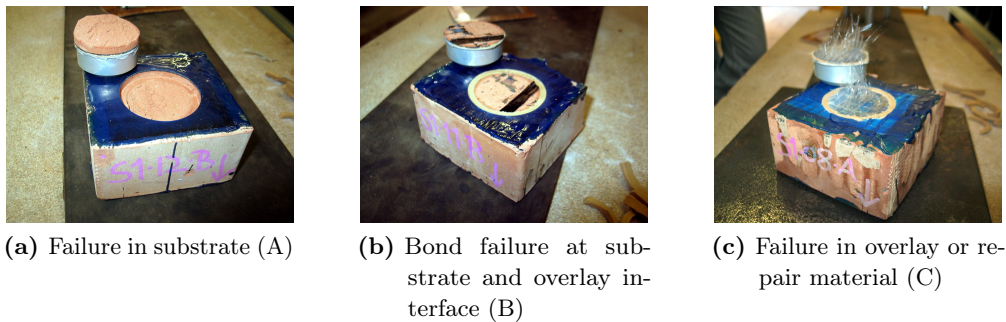


Figure 4.9: Observed failure of pull-off tests, labelled following ASTM C1583 (2004)

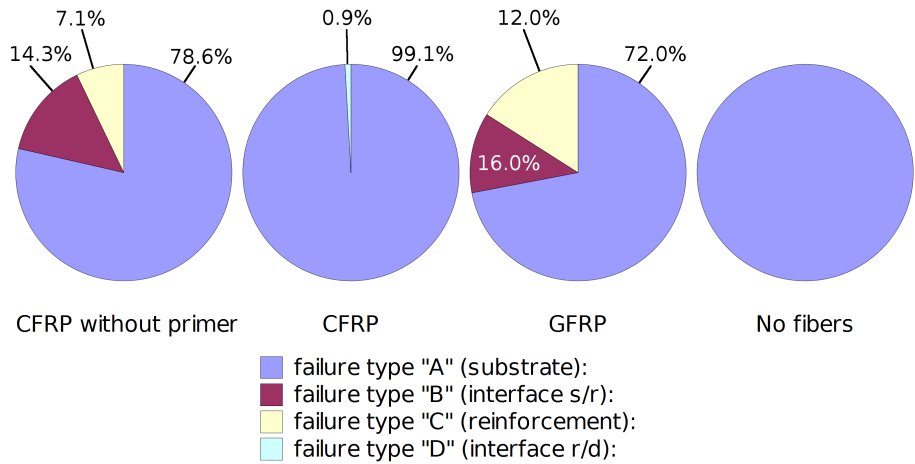


Figure 4.10: Failures of pull-off tests

Table 4.7: Observed failures of the pull-off tests

	CFRP without primer		CFRP with primer		GFRP with primer		No fibers with primer	
available tests:	14		113		25		14	
"A" (substrate):	11	(78.6%)	112	(99.1%)	18	(72.0%)	14	(100.0%)
"B" (interface s/o):	2	(14.3%)	0		3	(12.0%)	0	
"C" (reinforcement):	1	(7.1%)	0		4	(16.0%)	0	
"D" (interface e/o):	0		1	(0.9%)	0		0	

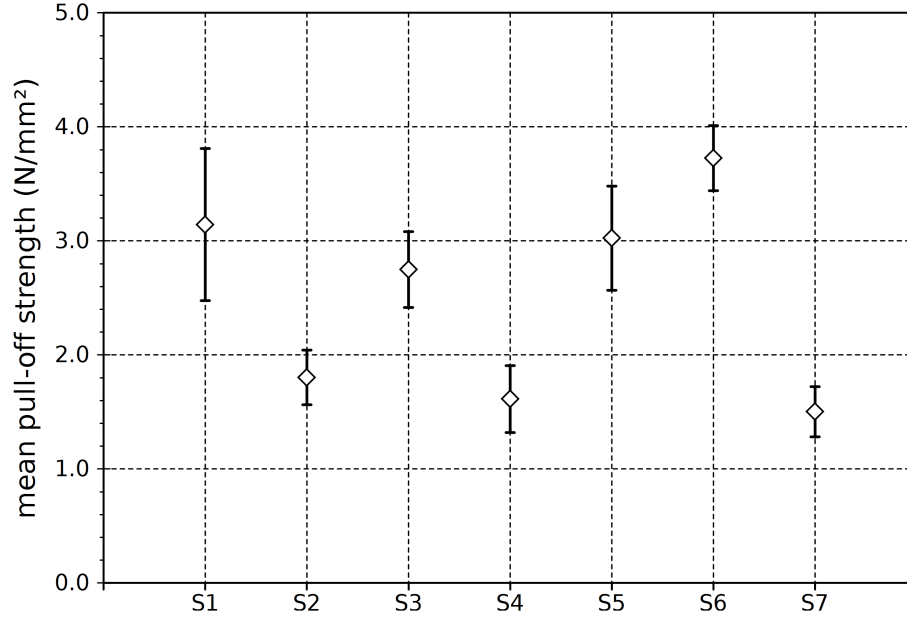


Figure 4.11: Results of pull-off tensile tests (average values)

Table 4.8: Results of the pull-off tensile tests

SERIES	f_{p-o} N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1	3.14	0.67	21.2%
S2	1.80	0.24	13.3%
S3	2.75	0.33	12.1%
S4	1.61	0.29	18.2%
S5	3.02	0.46	15.1%
S6	3.73	0.29	7.7%
S7	1.50	0.22	14.7%

4.2 Analysis of the results

4.2.1 Correlation between the pull-off and the flexural tensile strength

As previously highlighted, the facing bricks showed good flexural performance, compared to the extruded ones, despite their lower compressive strength; this fact could be imputed to their less brittle behaviour that permits a better stress distribution on the cross section, related to higher failure loads. On the other hand, the pull-off strength seems to be consistent with the compressive strength, since extruded bricks exhibited higher resistance. Plotting the pull-off versus the flexural tensile strengths, measured on the same clay element, extruded and facing bricks show similar trends, but higher values for the first ones (Fig. 4.12). These trends could be quite correctly described by a power-based regression, whose formulas are reported in Equation (4.5), where f_{p-o} is the pull-off strength and f_f the flexural one.

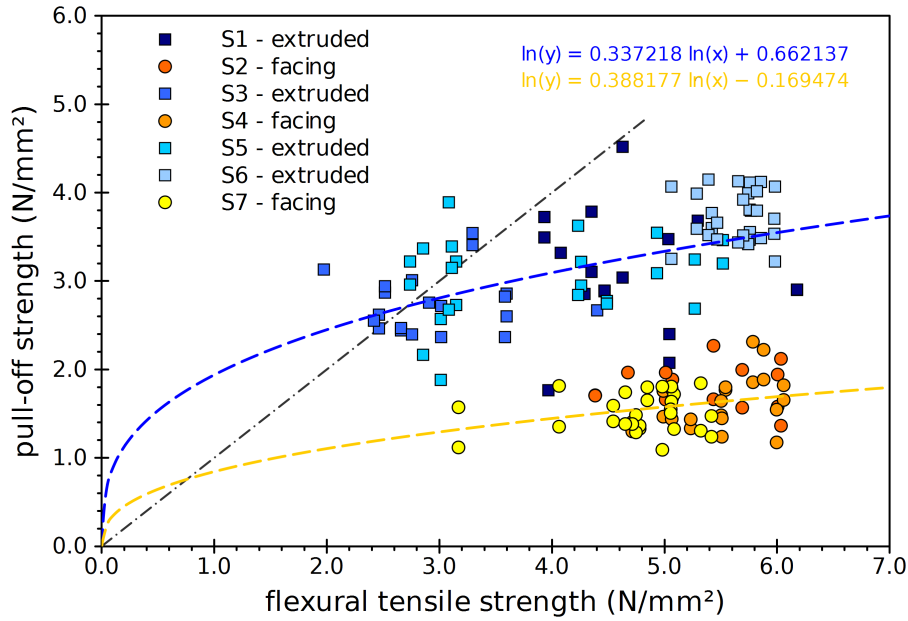


Figure 4.12: Pull-off strengths plotted versus flexural strengths

$$f_{p-o} = 1.939 f_f^{0.337} \quad \text{extruded bricks} \quad (4.5a)$$

$$f_{p-o} = 0.844 f_f^{0.388} \quad \text{facing bricks} \quad (4.5b)$$

4.2.2 Correlation between the pull-off and the compressive strength

The pull-off strength seems to be consistent with the compressive strength, since both the brick types show a similar trend. Plotting the pull-off versus the compressive strengths, measured on the same clay element, the behaviour of extruded and facing bricks could be depicted by the same curve (Fig. 4.13 on the facing page), as the regression functions separately evaluated are just slightly different. These trends could be quite correctly described by a power-based regression, whose formulas are reported in Equation (4.6).

The next step was to calibrate two regression functions based on the shape provided by two codes, ACI 318-99 (2004) and DM 09-01-96 (1996), which correlate the splitting tensile strength, in the first case, and the direct tensile one, in the latter, to the compressive strength of concrete (cylindric strength in the first case, cubic in the other). The appropriate

formulas are reported in Eq. (4.7) and (4.8), respectively, where f_s and f_t denote splitting and direct tensile strength, R_{ck} and f'_c cubic and cylindric compressive strength; it has to be noted that the function provided by the ACI 318-99 (2004) is intended to be applied on values expressed in PSI and it will not work using SI units, therefore the correct coefficient is also reported in order to facilitate a comparison. Clearly, the pull-off tensile strength (f_{p-o}) and the cubic compressive one (f_c) were used in place of the parameters indicated by the codes, in order to preserve only the shape of the two functions. Each type of regression function was calibrated three times, first on data coming from extruded bricks, then on data related to facing ones and, finally, on all the joined data. The results (Fig. 4.14 on the next page, and Eq. 4.9 and 4.10 on the next page) show not great differences among these curves. Finally, Figure 4.15 on the following page shows a comparison of the three regressions made on all the joined data (without distinguish among facing or extruded bricks).

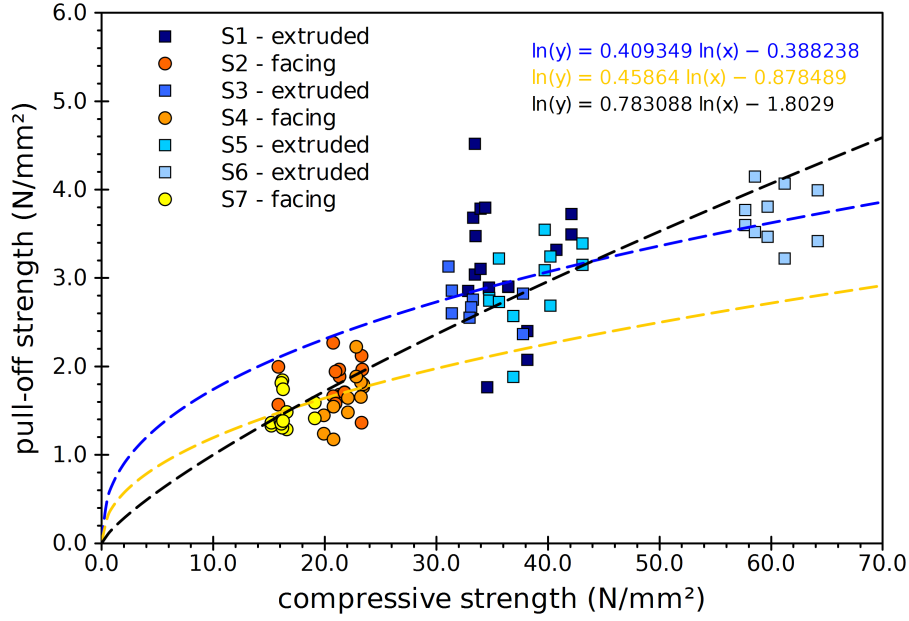


Figure 4.13: Pull-off strengths plotted versus compressive strengths

$$f_{p-o} = 0.409 f_c^{0.678} \quad \text{extruded bricks} \quad (4.6a)$$

$$f_{p-o} = 0.459 f_c^{0.415} \quad \text{facing bricks} \quad (4.6b)$$

$$f_{p-o} = 0.165 f_c^{0.783} \quad \text{all data} \quad (4.6c)$$

$$f_{ct} = 6.7 \sqrt{f'_c} \quad \text{ACI 318-99} \quad (4.7a)$$

$$f_{ct} = 0.56 \sqrt{f'_c} \quad \text{(SI units)} \quad (4.7b)$$

$$f_{ctm} = 0.27 \sqrt[3]{R_{ck}^2} \quad \text{DM 09-01-99} \quad (4.8)$$

$$f_{p-o} = 0.490 \sqrt{f_c} \quad \text{extruded} \quad (4.9a)$$

$$f_{p-o} = 0.371 \sqrt{f_c} \quad \text{facing} \quad (4.9b)$$

$$f_{p-o} = 0.456 \sqrt{f_c} \quad \text{all data} \quad (4.9c)$$

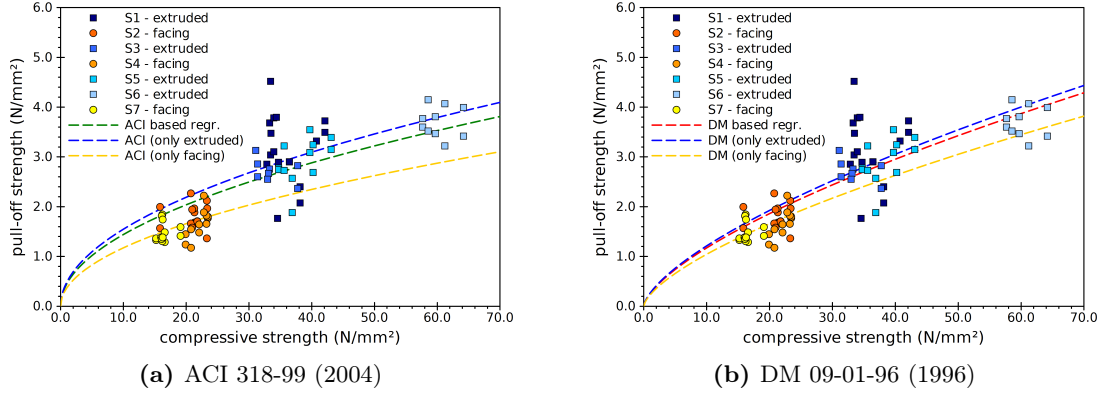


Figure 4.14: Regressions based on code provisions

$$f_{p-o} = 0.261 \sqrt[3]{f_c^2} \quad \text{extruded} \quad (4.10a)$$

$$f_{p-o} = 0.371 \sqrt[3]{f_c^2} \quad \text{facing} \quad (4.10b)$$

$$f_{p-o} = 0.253 \sqrt[3]{f_c^2} \quad \text{all data} \quad (4.10c)$$

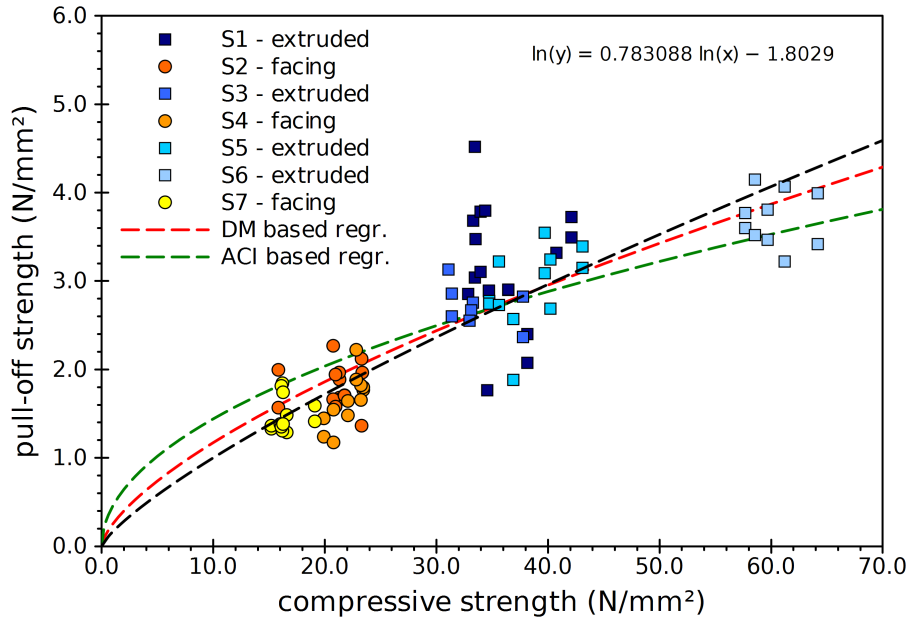


Figure 4.15: Comparison of regressions

4.2.3 Correlation between the pull-off and the splitting tensile strength

As in the case of the flexural tests, the facing bricks showed good splitting performance, compared to the extruded ones, despite their lower compressive strength; again, this fact could be probably imputed to their less brittle behaviour that permits a better stress distribution on the cross section, related to higher failure loads. On the contrary, the pull-off strength seems to be consistent with the compressive strength, since extruded bricks exhibited higher resistance. Extruded and facing bricks show similar trends, if the pull-off versus the splitting tensile strengths, measured on the same clay element, are plotted; but higher values for the

first ones (Fig. 4.16) were recorded. These trends could be quite correctly described by a power-based regression, whose formulas are reported in Equation (4.11), where f_{p-o} is the pull-off strength and f_s the splitting one. .

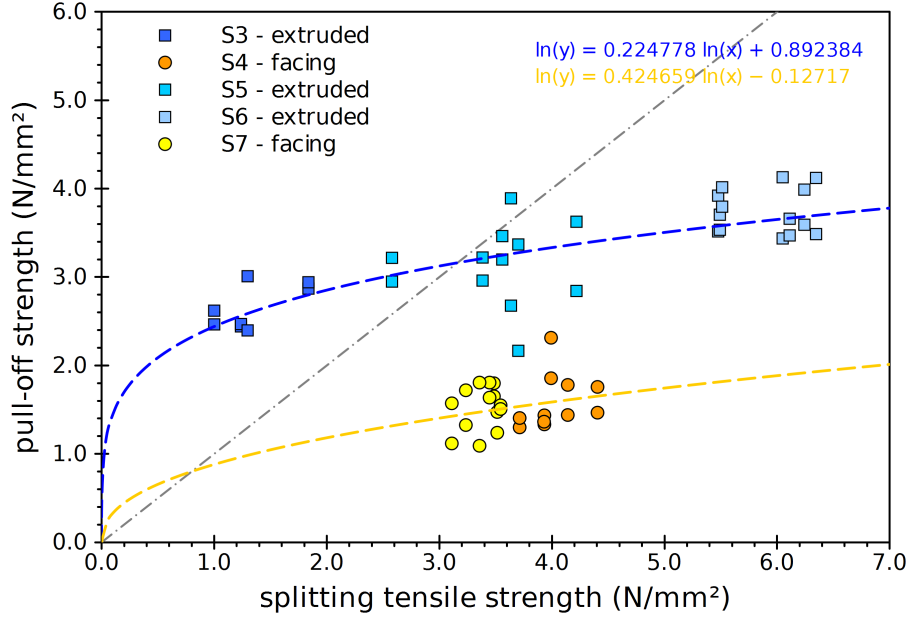


Figure 4.16: Pull-off strengths plotted versus splitting tensile strengths

$$f_{p-o} = 2.441 f_s^{0.225} \quad \text{extruded bricks} \quad (4.11a)$$

$$f_{p-o} = 0.881 f_s^{0.425} \quad \text{facing bricks} \quad (4.11b)$$

4.2.4 Correlation between the flexural tensile strength and the compressive strength

In this case, the available range of values is not particularly wide, but the different behaviour of extruded and facing bricks can be noticed: as previously observed, the latter elements exhibit better flexural performances in relation to their compressive strength. Therefore, the trend could be quite correctly described by two distinct power-based regression (Fig. 4.17 on the following page), whose formulas are reported in Equation (4.12), where f_f is the flexural tensile strength and f_c the compressive one.

$$f_f = 0.473 f_c^{0.602} \quad \text{extruded bricks} \quad (4.12a)$$

$$f_f = 1.647 f_c^{0.387} \quad \text{facing bricks} \quad (4.12b)$$

4.2.5 Correlation between the flexural and the splitting tensile strength

As both the parameters measure a tensile property of the material, the trend is almost proportional and no significant differences are noticeable between extruded and facing bricks. This trend could be quite correctly described by both a power-based regression or a linear one (Fig. 4.18 on the next page), whose formulas are reported in Equation (4.13) and Equation (4.14), power-based and linear regressions, respectively, where f_f is the flexural tensile strength and f_s the splitting tensile one.

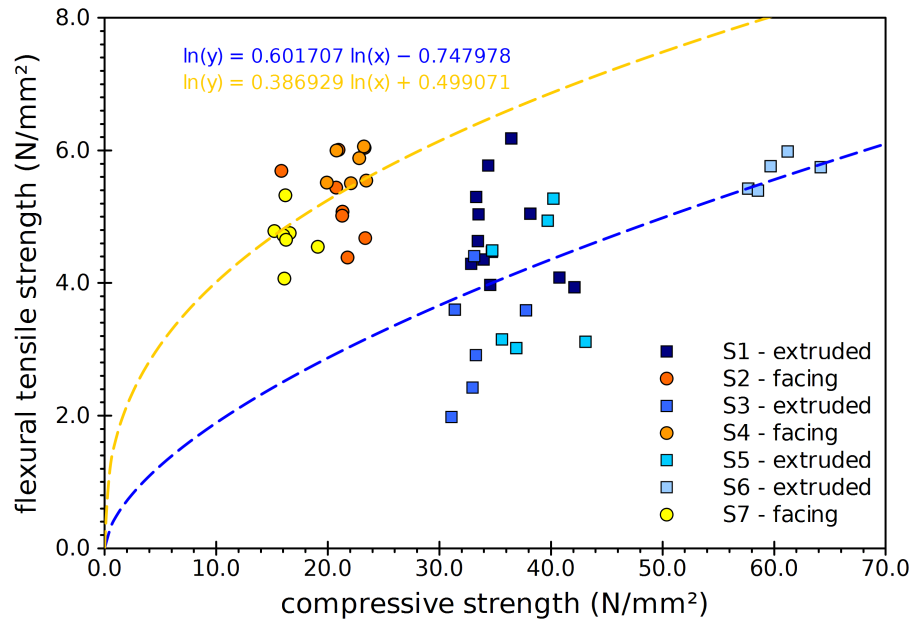


Figure 4.17: Pull-off strengths plotted versus splitting tensile strengths

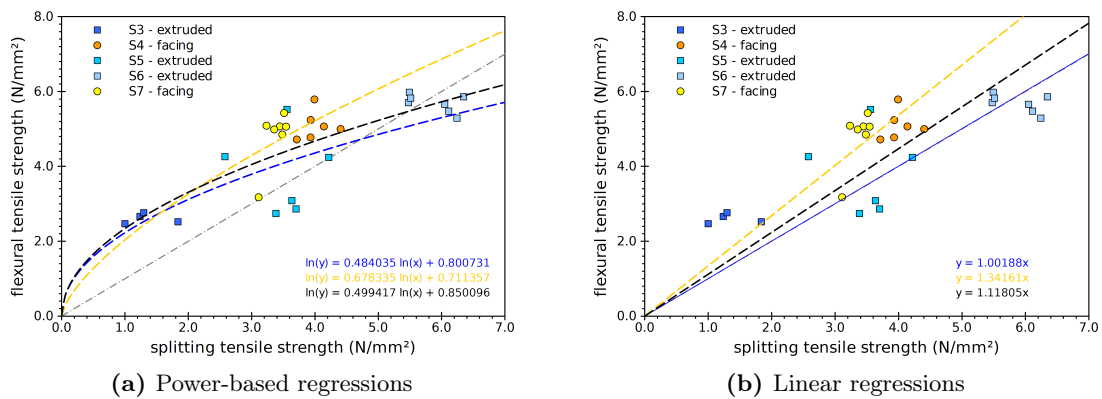


Figure 4.18: Flexural versus splitting tensile strength

$$f_f = 2.227f_s^{0.484} \quad \text{extruded bricks} \quad (4.13a)$$

$$f_f = 2.037f_s^{0.678} \quad \text{facing bricks} \quad (4.13b)$$

$$f_f = 2.340f_s^{0.499} \quad \text{all data} \quad (4.13c)$$

$$f_f = 1.002f_s \quad \text{extruded bricks} \quad (4.14a)$$

$$f_f = 1.342f_s \quad \text{facing bricks} \quad (4.14b)$$

$$f_f = 1.118f_s \quad \text{all data} \quad (4.14c)$$

4.2.6 Correlation among the average properties of solid clay bricks

Within this section, the possible relation among the average mechanical properties of solid clay bricks (tested during the present experimental campaign and collected by previous works carried out at the University of Padova) have been investigated and compared, where possible, to the relations deduced by punctual data, that means properties measured on the same element. It has to be noticed that pull-off tensile strength values are available only for the seven sets of elements which results have been above presented, whereas the relation between compressive and splitting tensile strength cannot be investigated on punctual data, since only one portion for each brick was available to these types of test, being the other piece originated by the flexural failure destined to pull-off testing.

The available data are given in Table 4.10 on the following page, 4.11 on the next page and 4.12 on the following page, where f_f , f_c and f_s indicate the flexural, compressive and splitting strength, respectively.

The average values of pull-off tensile strength plotted versus the flexural, compressive and splitting strength are compared to the regression functions calibrated on the punctual data (Fig. 4.19 on page 33, 4.20 on page 33 and 4.21 on page 34, respectively); it can be observed that these values appear to be consistent with the previously evaluated regressions.

Concerning the relation between flexural and compressive strength, the regressions based on average values, whose coefficients are reported in Equation (4.15) do not substantially differ from the ones based on punctual data (two data, namely UniPD02 and UniPD03, have been excluded due to their peculiar behaviour, that could be imputed to problems during testing), as shown in Figure 4.22 on page 34, where dotted lines describe previous regressions. It has also been confirmed that a function based on all data cannot correctly depict any relation.

Regarding the relation between splitting and compressive strength, three power-based regression have been evaluated, whose coefficients are reported in Equation (4.16), but only the one based on all the values without distinguishing among facing or extruded bricks seems to lead to a significant relation, as shown in Figure 4.23 on page 35.

Finally, flexural and splitting tensile strength have been compared; also in this case, the regressions based on average values, whose coefficients are reported in Equation (4.17) do not substantially differ from the ones based on punctual data (two data, namely UniPD02 and UniPD03, have been excluded due to their peculiar behaviour, as previously done), as shown in Figure 4.24 on page 36, where dotted lines describe previous regressions.

Table 4.10: Average values of flexural tensile strength

SERIES	f_f N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1 ES_A	4.67	0.75	16.1%
S2 FaV	5.29	0.61	11.5%
S3 ES_B	2.97	0.66	22.3%
S4 FaV	5.42	0.47	8.6%
S5 ES_C	3.89	1.01	26.0%
S6 ES	5.64	0.27	4.8%
S7 FaV	4.75	0.57	11.9%
UniPD 1999			
UniPD 2000	4.74	0.73	15.4%
UniPD 2002	5.97	0.49	8.3%
UniPD 2003	6.40	0.47	7.3%
UniPD 2004	5.46	0.66	12.1%
UniPD 2008	5.98	0.80	13.4%

Table 4.11: Average values of compressive strength

SERIES	f_c N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1 ES_A	35.36	3.19	9.0%
S2 FaV	21.09	2.34	11.1%
S3 ES_B	33.29	2.39	7.2%
S4 FaV	22.06	1.42	6.4%
S5 ES_C	38.39	3.18	8.3%
S6 ES	60.27	2.56	4.3%
S7 FaV	16.51	1.22	7.4%
UniPD 1999	8.66	0.99	11.5%
UniPD 2000	16.53	2.77	16.8%
UniPD 2002	16.91	2.02	12.0%
UniPD 2003	43.03	2.41	5.6%
UniPD 2004	50.94	3.97	7.8%
UniPD 2008	25.74	1.85	7.2%

Table 4.12: Average values of splitting strength

SERIES	f_s N/mm ²	Stand. dev. N/mm ²	C. of V. %
S1 ES_A			
S2 FaV			
S3 ES_B	1.34	0.35	26.3%
S4 FaV	4.02	0.23	5.8%
S5 ES_C	3.51	0.54	15.3%
S6 ES	5.89	0.38	6.5%
S7 FaV	3.39	0.16	4.7%
UniPD 1999	1.36	0.03	1.9%
UniPD 2000	2.28	0.29	12.8%
UniPD 2002	1.75	0.17	9.9%
UniPD 2003	4.94	0.82	16.6%
UniPD 2004	3.99	0.34	8.4%
UniPD 2008	4.02	0.45	11.3%

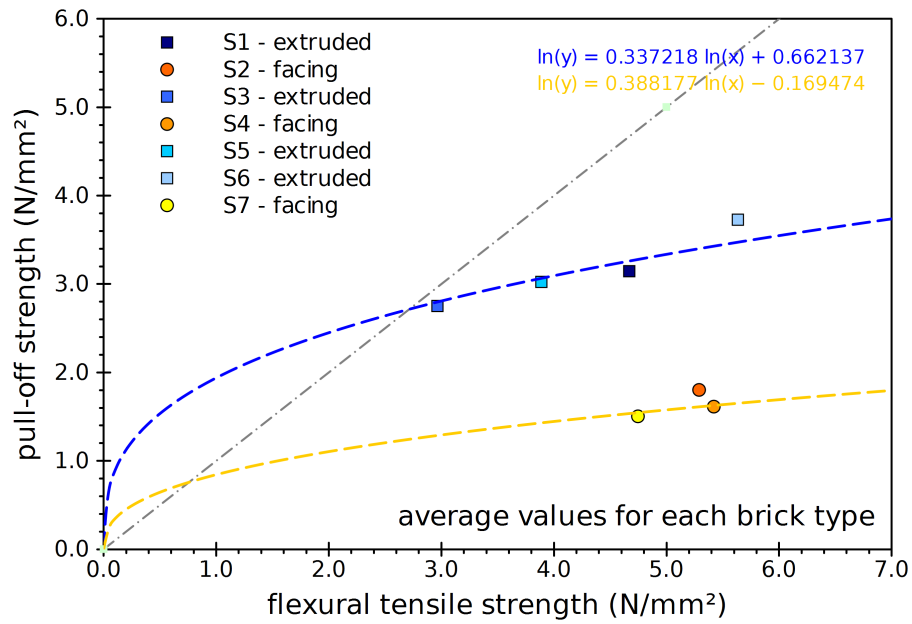


Figure 4.19: Average pull-off plotted versus flexural tensile strengths

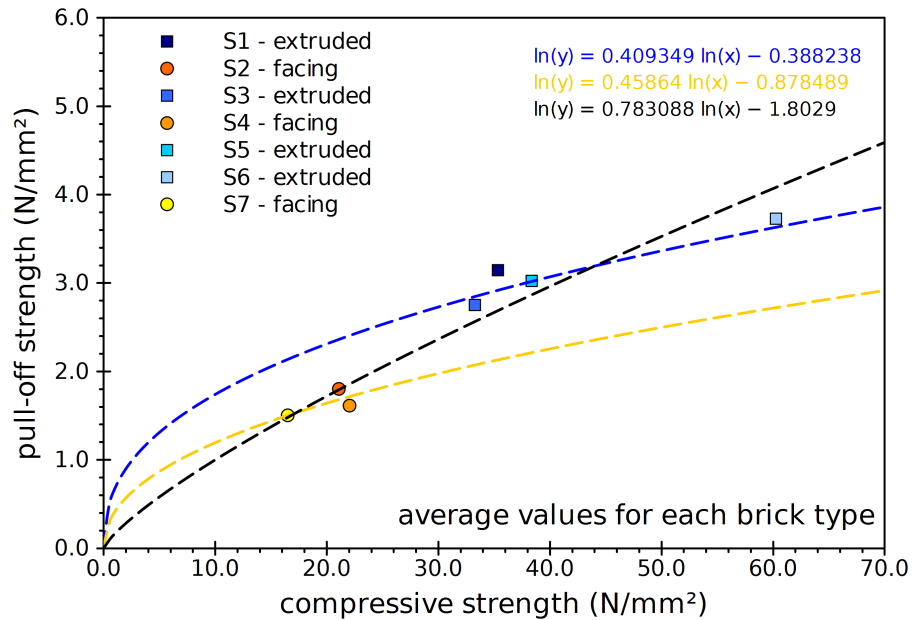


Figure 4.20: Average pull-off tensile plotted versus compressive strengths

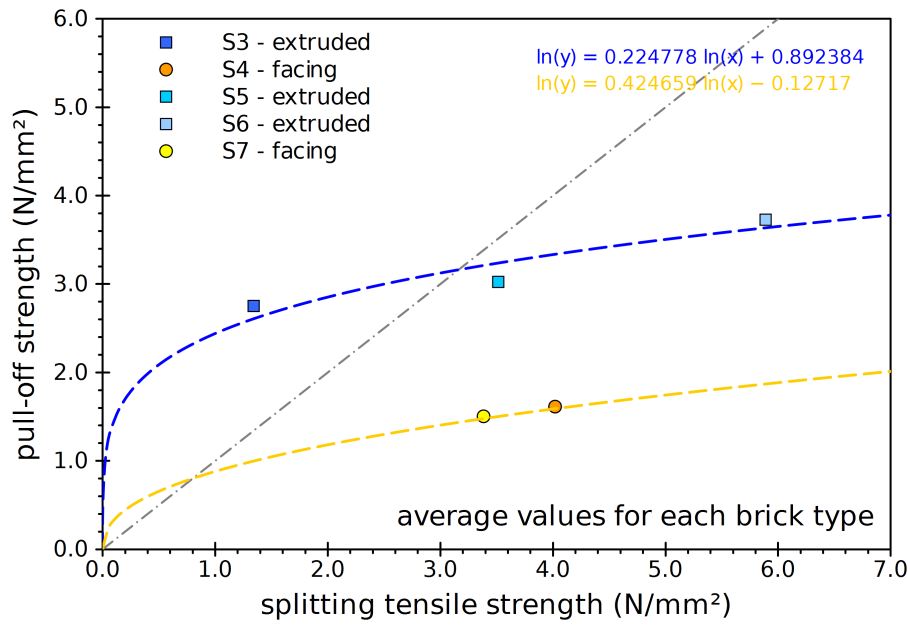


Figure 4.21: Average pull-off plotted versus splitting tensile strengths

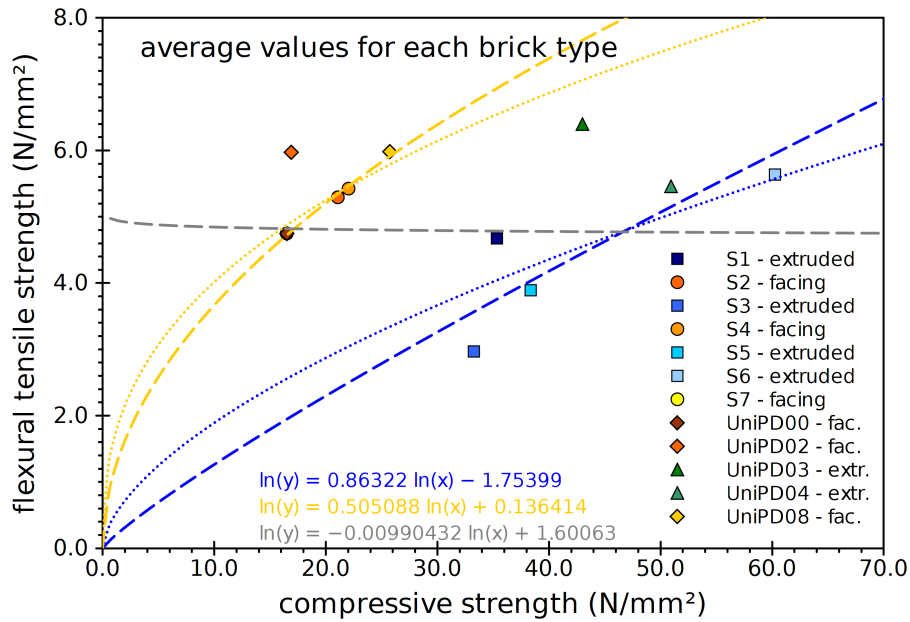


Figure 4.22: Average flexural plotted versus compressive strengths

$$f_f = 0.173 f_c^{0.863} \quad \text{extruded bricks} \quad (4.15a)$$

$$f_f = 1.146 f_c^{0.505} \quad \text{facing bricks} \quad (4.15b)$$

$$f_f = 4.956 f_c^{-0.010} \quad \text{all data} \quad (4.15c)$$

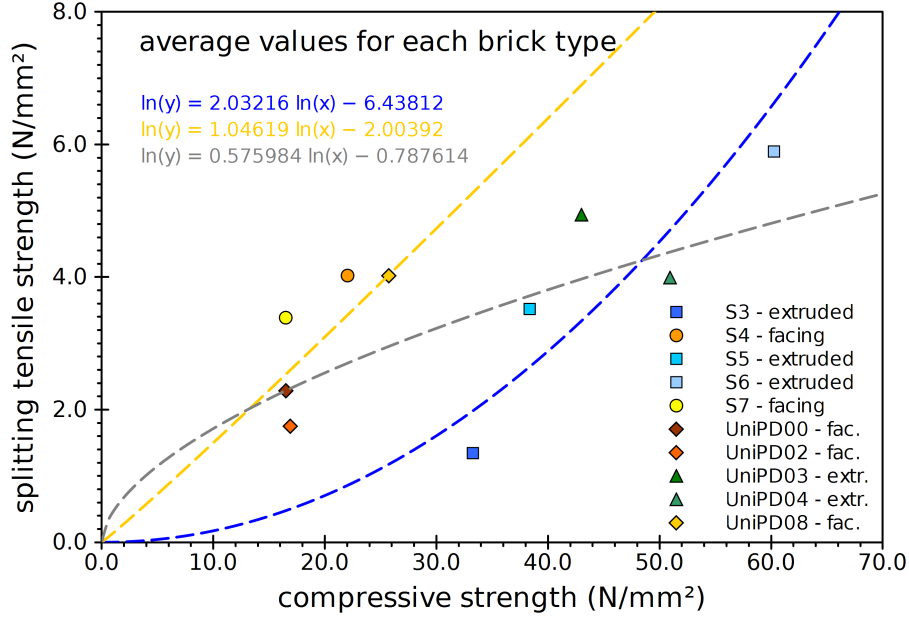


Figure 4.23: Average splitting plotted versus compressive strengths

$$f_s = 0.002 f_c^{2.032} \quad \text{extruded bricks} \quad (4.16a)$$

$$f_s = 0.135 f_c^{1.046} \quad \text{facing bricks} \quad (4.16b)$$

$$f_s = 0.455 f_c^{0.576} \quad \text{all data} \quad (4.16c)$$

$$f_f = 2.561 f_s^{0.448} \quad \text{extruded bricks} \quad (4.17a)$$

$$f_f = 3.548 f_s^{0.316} \quad \text{facing bricks} \quad (4.17b)$$

$$f_f = 2.828 f_s^{0.435} \quad \text{all data} \quad (4.17c)$$

4.3 Conclusive remarks

The results of experimental campaign herein presented have led to the following conclusions:

- if the reinforcement is correctly applied, the type of fibers should not have any influence on the strength measured by the pull-off test; only the failure mode could be influenced, as glass fibers caused some unwanted failures that should however imputed to contingent factors related to that available portion of reinforcement, such as an incorrect conservation or cleaning. Moreover, the absence of primer could increase the amount of unwanted failures.

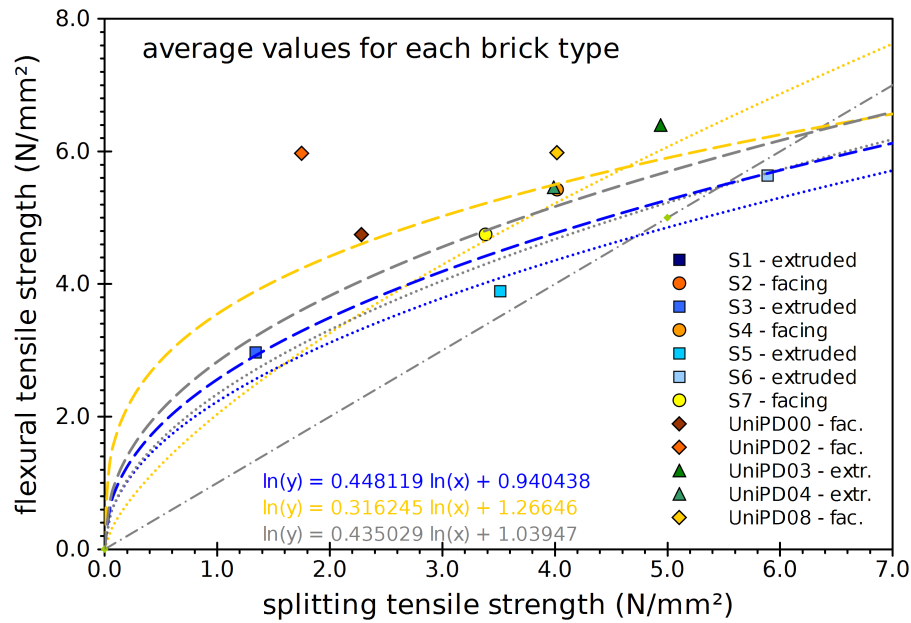


Figure 4.24: Average flexural plotted versus splitting tensile strengths

- The pull-off tensile strength could be correlated to the flexural, splitting and compressive strength of the bricks by means of power-based regressions; except the relation with the compressive strength, of which a single function seems to correctly depict the tendency, in the other cases extruded and facing bricks showed a different behaviour, although the trends were similar.
- A different behaviour of extruded and facing elements was observed also in the correlation between flexural and compressive strength, whereas a single power-based regression could be sufficient to describe the trend that relates flexural and splitting tensile strength.
- Finally, the correlations among average values of the bricks properties, where sufficiently evident, substantially confirmed the previously calculated trends.

Chapter 5

Interface behaviour in the case of tangential actions

In this chapter, the main results of ten Double-lap Push-pull Shear Tests performed on CFRP and GFRP reinforcement (five samples for each type) externally bonded on solid clay bricks are presented. Based on the experimentally acquired data, the fracture energy value, Gf , has been calibrated for both the reinforcements, following the commonly adopted interpretation of these tests. An analytical function has been proposed as bond-slip law, and fitted on the experimental results as well as a bilinear relation. A bi-dimensional FE model has been developed, by means of a commercial software, in order to compare its results to the experimental evidences, and carry out analyses of the test specimens aimed at investigating the influence of the following parameters: different modelling approaches, shape of the local bond-slip law, bonded length. Finally, an extensive collection of experimental tests available in literature, concerning stones, clay bricks and masonry prisms used as substrate for the reinforcement, has been made in order to calibrate the c_1 coefficient provided by the CNR DT-200 (2004) Guidelines, which correlates the predicted value of fracture energy to the square root of the product of compressive and tensile strength of the masonry substrate.

5.1 Literature review

5.1.1 Experimental testing of bond between FRP and substrate

In the last decade, the bond of composite laminates, applied on concrete substrate and affected by axial loading, has been deeply investigated. The characterization of the behaviour has been performed by means of different test setups, all aimed at generating a state of stress that could be assimilated to pure shear.

Among various possibilities (Fig. 5.1 on page 40 and 5.2 on page 40), three main types of test can be identified as the most commonly used: Single-lap Shear Test (Chajes *et al.* 1996, Täljsten 1997, Bizindavyi and Neale 1999, Mazzotti *et al.* 2009), Double-lap Pull-pull Shear Test (Lee *et al.* 1999, Nakaba *et al.* 2001), Double-lap Push-pull Shear Test (Camli and Binici 2007) and Beam-type Test (De Lorenzis *et al.* 2001).

On the other hand, few investigations concerning debonding on masonry-like substrate are available, such as Aiello and Sciolti (2006) that investigated bond on natural stones, Faella *et al.* (2009) that compared natural stones and clay bricks, Briccoli Bati *et al.* (2007), Grande *et al.* (2008) and Panizza *et al.* (2008a) that tested bond on solid clay bricks, Camli and Binici (2007) that used hollow clay blocks, Casareto *et al.* (2003) that studied bond on clay blocks and block masonry prisms, Basilio (2007), Subramanian *et al.* (2009) that

tested bond on brick masonry prisms, Capozucca (2010) that chose historic clay bricks as support. They adopted either Single-lap Shear Test or Double-lap Push-pull Shear Tests, where the last is also known in literature by different names, such as Double-shear Push Test or Near-end Supported Double-shear Test (Yao *et al.* 2005). It consists in loading in tension two reinforcement strips, symmetrically connected to the support, in order to create mainly shear stresses at the interface, while the brittle support is subjected to compressive stresses.

The Double-lap Push-pull Shear Test set-up is based on the assumption that the applied load is equally distributed on the two strips, but it is also particularly simple and suitable for the usual common available test machines.

Concerning the concrete substrate, the main aspects of the phenomenon are currently supposed to be clarified, and can be summarized as follows (Fig. 5.4 on page 41 and 5.3 on page 41):

- increasing the bonded length of the reinforcement, L_b , the failure axial load, P_u , will increase until a certain value of L_b is reached; if the bonded length exceeds that value, usually defined as effective bond length, L_e , this fact will not affect the ultimate load.
- The failure load is directly proportional to the reinforcement width, b_f .
- The fracture energy G_f and the reinforcement axial stiffness per unit width $E_f t_f$, given by the product of the Young's modulus E_f and the thickness of the reinforcement, show a positive correlation with the failure load.
- If the substrate axial stiffness, $b_c E_c t_c$, is sufficiently higher than the reinforcement one, it has no significant influence on the failure load.

5.1.2 Prediction of strength

During the last years, many predictive models have been developed to estimate the failure load of the composite-to-concrete bonded joint. Wide reviews of available strength or bond-slip models were given by Chen and Teng (2001), Lu *et al.* (2005), Karbhari *et al.* (2006), Ferracuti *et al.* (2007b).

Among them, the models of Tanaka (Eq. 5.1), Hiroyuki & Wu (Eq. 5.2), Maeda (Eq. 5.3), Khalifa (Eq. 5.4), reported in Chen and Teng (2001), Yang (Eq. 5.5), Sato (Eq. 5.6), Iso (Eq. 5.7), reported in Lu *et al.* (2005), express the failure load as the product of an area and a nominal average tangential stress, τ_u , different from model to model.

The models of Izumo (Eq. 5.8), Lu *et al.* (2005, see), Neubauer & Rost  sy (1) (Eq. 5.9), Chen & Teng (Eq. 5.10), both reported in Chen and Teng (2001), give other expressions for the failure load.

Finally, eleven ones provide an estimation of the fracture energy value G_f , which is correlated to the failure load (Eq. 5.11). In particular, the models of Monti (Lu *et al.* 2005), Lu *et al.* (2005), herein Lu Bilinear, Brosens & Van Gemert (Karbhari *et al.* 2006) and Italian Research Council (CNR DT-200 2004) utilize a bilinear bond-slip law (5.12); the models of Nakaba *et al.* (2001) and Savoia *et al.* (2003a) adopt a Popovics curve as bond-slip law (5.17); the models of Neubauer & Rost  sy (2) (Lu *et al.* 2005), Dai & Ueda (1) (2003), Dai & Ueda (2) (2005) and Lu *et al.* (2004), Precise and Simplified models, assume other types of bond-slip function. The corresponding formulas are reported from Equation (5.13) to (5.24).

Moreover, four models are reported, although not applied since they need, differently from the others, the knowledge of some parameters such as the value of the fracture energy

or the maximum bond stress: Täljsten (Täljsten 1996), Niedermaier, Yuan & Wu, Yuan et al. (Chen and Teng 2001). The related formulations are reported from (5.25) to (5.28).

In some cases, symbols have been changed, with respect to the source, in order to make them consistent; in other cases, some coefficients have been adapted to express lengths in mm, forces in N, stresses in N/mm².

The main parameters included in the above mentioned models, referring to the reinforcement, fibres and adhesive, and the support, are shown in Table 5.1 on page 42

5.1.3 Correlation of the interface fracture energy to the failure load

The interface fracture energy mode II, G_f , is defined as the definite integral of the tangential stress on the interface, τ , expressed as function of the mutual slip of composite and substrate, s (Eq. 5.29 on page 48).

One of the first analytical models of the concrete-composite bond strength was derived by Täljsten (1996), starting both from a linear approach, based on the beam theory, and from a non-linear approach, related to fracture mechanics. In the case of most of the epoxy adhesives commonly used, a simplified formulation, reported in Equation (5.25), was obtained.

Yuan (Chen and Teng 2001) proposed a modified constant value (Eq. 5.5 on page 43) that takes into account the width (b_f and b_c) ratio of the bonded materials.

In most cases, the constant value α_T , or α_Y , has a slight influence on the calculation. Many authors, such as Savoia et al. (2003a) and Dai et al. (2005), report the relation between ultimate load and fracture energy without any constant (see Eq. 5.11 on page 44).

It has also to be noticed that Equation (5.11), demonstrated in some cases (Wu *et al.* 2002, Dai *et al.* 2005), can be assumed in every case of regular interface law, as reported by Savoia (2003b).

5.1.4 Relation between local bond stress and mutual slip

Figure 5.5 on page 42 shows possible shapes of the bond-slip function: (a) cut-off, adopted by Neubauer and Rostásy (Chen and Teng 2001); (b) bilinear, assumed by some guidelines like fib Bulletin 14 (2001), CNR DT-200 (2004), and by Monti (Lu *et al.* 2005), Brosens and Van Gemert (Karbhari *et al.* 2006) and Lu (Lu *et al.* 2004); (c) rigid with linear softening, by Chen and Teng (2001); (d) a single function, as the Popovics curve chosen by Savoia *et al.* (2003a) and Nakaba *et al.* (2001), or an exponential curve obtained by Dai *et al.* (2005); (e) two different non-linear functions for ascending and descending branch, for instance the expressions adopted by Lu *et al.* (2004) or dai:2003.

Therefore, it is commonly assumed that bond of composite laminates exhibits a softening behaviour, with an ascending branch followed by a descending one, and presenting no residual stress for wider slip. In the following, $\tau_{\max}$ indicates the maximum bond stress reached by the interface, s_0 the corresponding mutual slip between substrate and reinforcement, and s_f the ultimate slip, where accounted.

The commonly adopted analyses of the interface behaviour is based on simple equilibrium and compatibility equations among bond stress τ , mutual slip s , strain ε and distance from the loaded end, x , reported in (5.30) (Fig. 5.6 on page 42); as usual, strain and slip refer only to the reinforcement, since the substrate is considered sufficiently stiffer to disregard its deformation (Täljsten 1997, Bizindavyi and Neale 1999, Yuan *et al.* 2004, Dai *et al.* 2005).

$$\tau_u = 6.13 - \ln(L_b) \quad (5.1a)$$

$$P_u = b_f L_b \tau_u \quad (5.1b)$$

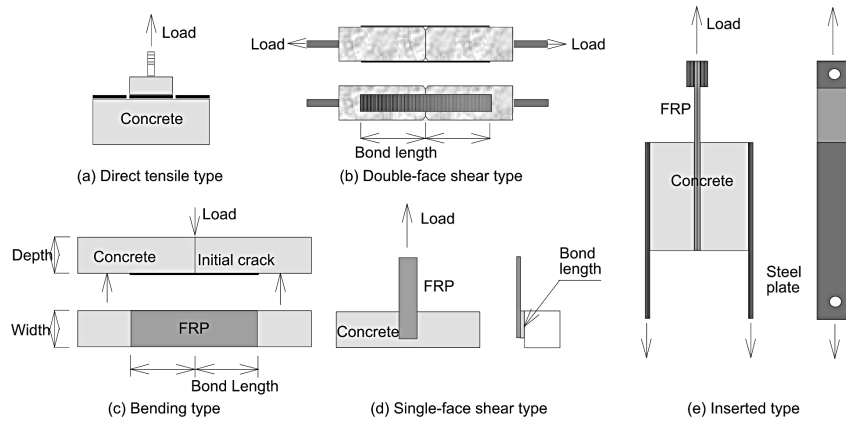


Figure 5.1: Different bond specimens (Nakaba *et al.* 2001)

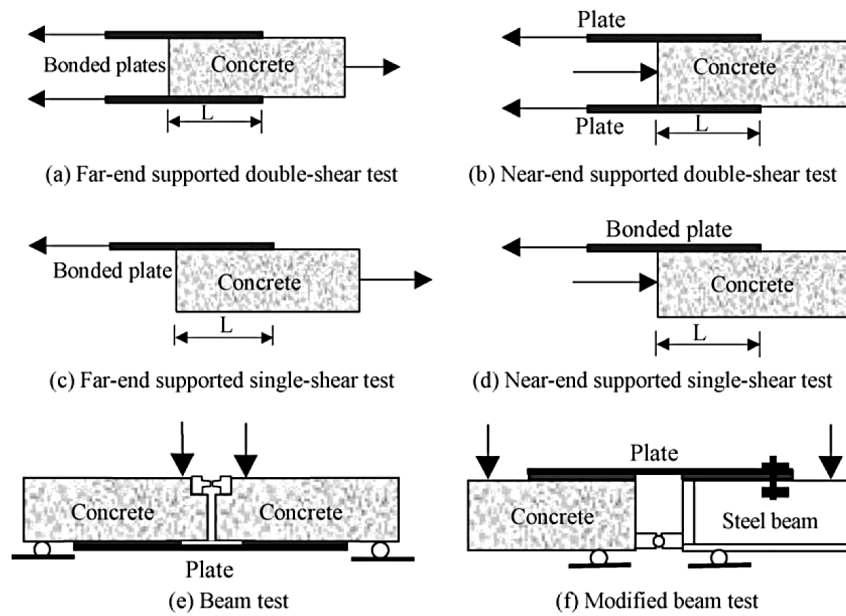
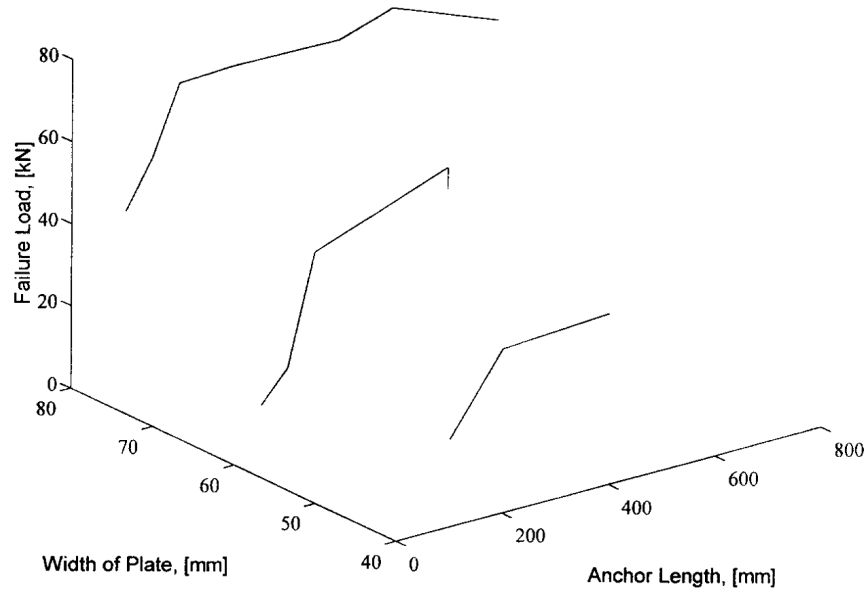
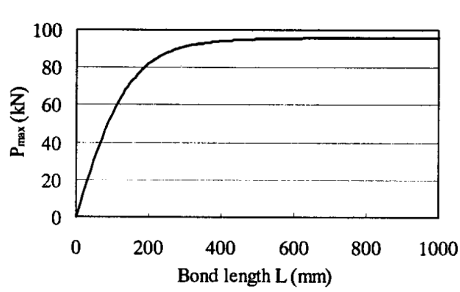
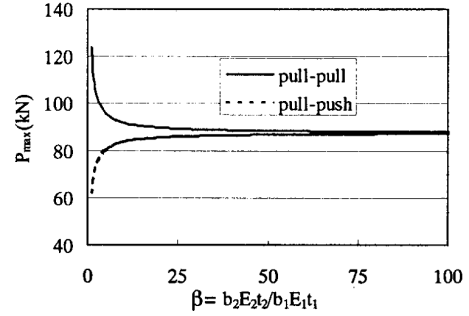


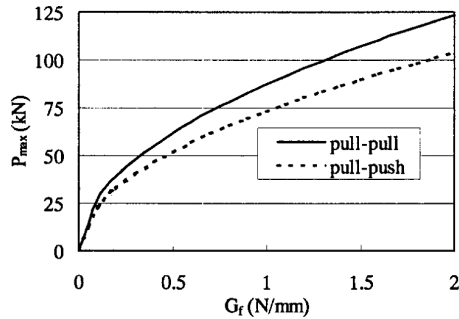
Figure 5.2: Classification of bond tests (Yao *et al.* 2005)


 Figure 5.3: Classification of bond tests (Yao *et al.* 2005)


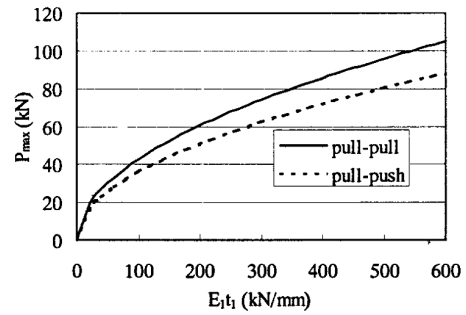
(a) Bonded length



(b) Support/reinforcement axial stiffness ratio



(c) Fracture energy



(d) Reinforcement axial stiffness per unit width

 Figure 5.4: Influence of the main parameters on the ultimate failure load (Wu *et al.* 2002)

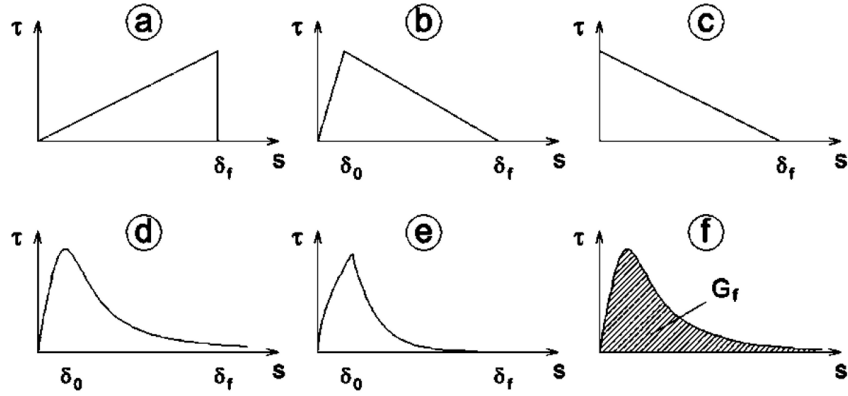


Figure 5.5: Different shapes for the bond-slip law

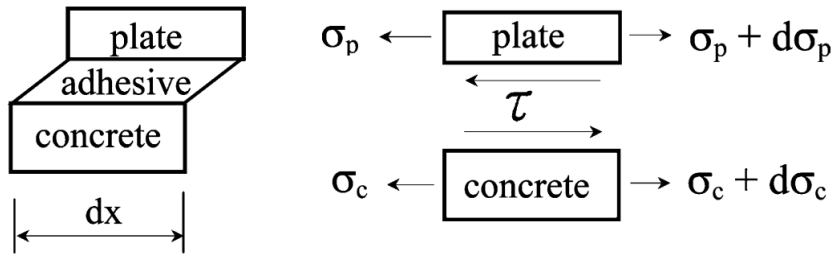
Figure 5.6: Simple model of reinforcement and concrete (Yuan *et al.* 2004)

Table 5.1: Parameters directly used by the predictive models

MODEL	b_f	L_b	t_f	E_f	t_a	E_a	G_a	b_c	t_{ref}	f'_c	f_t	E_c	G_c
Tanaka	*	*											
Hiroiyuki	*	*											
Maeda	*		*	*									
Khalifa	*		*	*						*			
Yang	*		*	*							*		
Sato	*		*	*						*			
Iso	*		*	*						*			
Izumo	*	*	*	*						*			
Neubauer & Rost�sy (1)	*		*	*				*			*		
Chen & Teng	*		*	*				*		*			
Monti	*		*	*	*	*		*			*	*	
Lu et al. bilinear	*		*	*				*			*		
Brosens & Van Gemert	*		*	*	*	*		*	*	*	*	*	
CNR	*		*	*	*		*	*	*	*	*		*
Nakaba	*		*	*						*			
Savoia	*		*	*						*			
Neubauer & Rost�sy (2)	*		*	*				*			*		
Dai & Ueda (1)	*		*	*	*		*			*			
Dai & Ueda (2)	*		*	*	*		*			*			
Lu et al. precise	*		*	*	*		*	*	*		*		*
Lu et al. simplified	*		*	*				*			*		

$$\tau_u = 5.88(0.1L_b)^{-0.669} \quad (5.2a)$$

$$P_u = b_f L_b \tau_u \quad (5.2b)$$

$$\tau_u = 110.6 \cdot 10^{-6} E_f t_f \quad (5.3a)$$

$$L_e = e^{6.13 - 0.580 \ln 10^{-3} E_f t_f} \quad (5.3b)$$

$$P_u = b_f L_e \tau_u \quad (5.3c)$$

$$\tau_u = 110.2 \cdot 10^{-6} \sqrt[3]{\left(\frac{f'_c}{42}\right)^2} E_f t_f \quad (5.4a)$$

$$L_e = e^{6.13 - 0.580 \ln 10^{-3} E_f t_f} \quad (5.4b)$$

$$P_u = b_f L_e \tau_u \quad (5.4c)$$

$$\tau_u = 0.5 f_t \quad (5.5a)$$

$$L_e = 100 \text{ mm} \quad (5.5b)$$

$$P_u = \left(0.5 + 0.8 \sqrt{\frac{E_f t_f}{100 f_t}}\right) b_f L_e \tau_u \quad (5.5c)$$

$$\tau_u = 2.68 f'_c{}^{0.2} E_f t_f \cdot 10^{-5} \quad (5.6a)$$

$$L_e = 1.89 (E_f t_f)^{0.4} \quad (5.6b)$$

$$P_u = (b_f + 2\Delta b_f) L_e \tau_u \quad \text{where } \Delta b_f = 3.7 \text{ mm} \quad (5.6c)$$

$$\tau_u = 0.93 f'_c{}^{0.44} \quad (5.7a)$$

$$L_e = 0.125 (E_f t_f)^{0.57} \quad (5.7b)$$

$$P_u = \begin{cases} b_f L_e \tau_u & \text{if } L_b \geq L_e \\ b_f L_b \tau_u & \text{otherwise} \end{cases} \quad (5.7c)$$

$$P_u = \begin{cases} \left(3.8 f'_c{}^{2/3} + 15.2\right) b_f L_b E_f t_f \cdot 10^{-6} & \text{(carbon fibres)} \\ \left(3.4 f'_c{}^{2/3} + 69\right) b_f L_b E_f t_f \cdot 10^{-6} & \text{(aramidic fibres)} \end{cases} \quad (5.8)$$

$$L_e = \sqrt{\frac{E_f t_f}{2 f_t}} \quad (5.9a)$$

$$P_u = \begin{cases} 0.64 \beta_w b_f \sqrt{E_f t_f f_t} & \text{if } L_b \geq L_e \\ 0.64 \beta_w b_f \sqrt{E_f t_f f_t} \frac{L_b}{L_e} \left(2 - \frac{L_b}{L_e}\right) & \text{otherwise} \end{cases} \quad (5.9b)$$

$$G_f = c_f f_t c_f = 0.204 \pm 0.053 \quad (5.9c)$$

$$\beta_w = 1.125 \sqrt{\frac{2 - b_f/b_c}{1 + b_f/400}} \quad (5.9d)$$

$$L_e = \sqrt{\frac{E_f t_f}{\sqrt{f'_c}}} \quad (5.10a)$$

$$P_u = 0.427 \beta_w \beta_l b_f L_e \sqrt{f'_c} \quad (5.10b)$$

$$\beta_w = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/b_c}} \quad (5.10c)$$

$$\beta_l = \begin{cases} 1 & \text{if } L_b > L_e \\ \sin \frac{\pi L_b}{2L_e} & \text{otherwise} \end{cases} \quad (5.10d)$$

$$P_u = b_f \sqrt{2E_f t_f G_f} \quad (5.11)$$

$$\text{Bilinear bond-slip curve: } \tau(s) = \begin{cases} \tau_{max} \left(\frac{s}{s_0} \right) & \text{if } s \leq s_0 \\ \tau_{max} \left(\frac{s_f - s}{s_f - s_0} \right) & \text{if } s_0 < s \leq s_f \\ 0 & \text{otherwise} \end{cases} \quad (5.12)$$

$$\tau_{max} = 1.8 \beta_w f_t \quad (5.13a)$$

$$s_0 = 2.5 \tau_{max} \left(\frac{t_a}{E_a} + \frac{50}{E_c} \right) \quad (5.13b)$$

$$s_f = 0.33 \beta_w \quad (5.13c)$$

$$\beta_w = \sqrt{1.5 \frac{2 - b_f/b_c}{1 + b_f/100}} \quad (5.13d)$$

$$\tau_{max} = 1.5 \beta_w f_t \quad (5.14a)$$

$$s_0 = 0.0195 \beta_w f_t \quad (5.14b)$$

$$s_f = \frac{2G_f}{\tau_{max}} \quad (5.14c)$$

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad (5.14d)$$

$$G_f = 0.308 \beta_w^2 \sqrt{f_c} f(K_a); \quad f(K_a) = 1 \text{ if not defined} \quad (5.14e)$$

$$\tau_{max} = \sqrt{\frac{1.47(2 - b_f/b_c)}{1 + b_f/85} \frac{f'_c f_t}{f'_c + f_t}} \quad (5.15a)$$

$$s_0 = \tau_{max} \left(2.4 \frac{t_{ref}}{E_c} + 2.5 \frac{t_m}{E_m} + n \frac{t_a}{E_a} \right) \quad (5.15b)$$

$$s_f = \frac{2G_f}{\tau_{max}} \quad (5.15c)$$

$$G_f = \frac{0.588(2 - b_f/b_c)}{1 + b_f/85} f_c \quad (5.15d)$$

$$L_e = \frac{2\lambda + a \tan(\tanh(2)/\lambda)}{\lambda\omega} \quad (5.15e)$$

$$\lambda = \sqrt{\frac{s_0}{s_f - s_0}} \quad (5.15f)$$

$$\omega = \sqrt{\frac{\tau_{max} \left(1 + \left(\frac{E_f A_f}{E_c A_c} \right) \right)}{s_0 E_f t_f}} \quad (5.15g)$$

$$a \text{ not defined in Karbhari } et al. (2006) \quad (5.15h)$$

$$\tau_{max} = 0.64\beta_w \sqrt{f_{ck} f_t} \quad (5.16a)$$

$$s_0 = \frac{\tau_{max}}{\frac{c_1}{t_a/G_a + t_c/G_c}} \quad (5.16b)$$

$$s_f = \frac{2G_f}{\tau_{max}} \quad (5.16c)$$

$$G_f = k_g \beta_w \sqrt{f_{ck} f_t} \quad (5.16d)$$

$$\beta_w = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/400}} \quad (5.16e)$$

$$k_g = 0.064 \quad \Rightarrow \quad s_f = 0.2 \quad (5.16f)$$

$$\text{Popovics-like bond-slip curve: } \tau(s) = \tau_{max} \frac{s}{s_0} \frac{n}{n - 1 + \left(\frac{s}{s_0} \right)^n} \quad (5.17)$$

$$\tau_{max} = 3.5 f'_c{}^0.19 s_0 = 0.065n = 3 \quad (5.18)$$

$$\tau_{max} = 3.5 f'_c{}^0.19 s_0 = 0.051n = 2.86 \quad (5.19)$$

$$\tau(s) = \begin{cases} \tau_{max} \left(\frac{s}{s_0} \right) & \text{if } s \leq s_0 \\ 0 & \text{otherwise} \end{cases} \quad (5.20a)$$

$$\tau_{max} = 1.8\beta_w f_t \quad (5.20b)$$

$$s_0 = 0.202\beta_w \quad (5.20c)$$

$$\beta_w = 1.125 \sqrt{\frac{2 - b_f/b_c}{1 + b_f/400}} \quad (5.20d)$$

$$\tau(s) = \begin{cases} \tau_{max} \left(\frac{s}{s_0}\right)^0 .575 & \text{if } s \leq s_0 \\ \tau_{max} e^{-\beta(s-s_0)} & \text{otherwise} \end{cases} \quad (5.21a)$$

$$\tau_{max} = \frac{-1.575\alpha K_a + \sqrt{2.481\alpha^2 K_a^2 + 6.3\alpha\beta^2 K_a G_f}}{2\beta} \quad (5.21b)$$

$$s_0 = \frac{\tau_{max}}{\alpha K_a} \quad (5.21c)$$

$$K_a = \frac{G_a}{t_a} \quad (5.21d)$$

$$G_f = 7.554 K_a^{-0.449} f_c'^{0.343} \quad (5.21e)$$

$$\alpha = 0.028 (10^{-3} E_f t_f)^{0.254} \quad (5.21f)$$

$$\beta = 0.0035 K_a (10^{-3} E_f t_f)^{0.34} \quad (5.21g)$$

$$\tau(s) = 2UG_f (e^{-Us} - e^{-2Us}) \quad (5.22a)$$

$$\tau_{max} = 0.5UG_f \quad (5.22b)$$

$$s_0 = \frac{\ln(2)}{U} \quad (5.22c)$$

$$U = 6.846 (10^{-3} E_f t_f)^{0.108} \left(10^{-3} \frac{G_a}{t_a}\right)^{0.833} \quad (5.22d)$$

$$G_f = 0.446 (10^{-3} E_f t_f)^{0.023} \left(10^{-3} \frac{G_a}{t_a}\right)^{-0.352} f_c'^{0.236} \quad (5.22e)$$

$$\tau(s) = \begin{cases} \tau_{max} \left(\sqrt{\frac{s}{s_0 A}} - B \right) & \text{if } s \leq s_0 \\ \tau_{max} e^{-\alpha \left(\frac{s}{s_0} - 1 \right)} & \text{otherwise} \end{cases} \quad (5.23a)$$

$$\tau_{max} = 1.5 \beta_w f_t \quad (5.23b)$$

$$s_0 = 0.0195 \beta_w f_t + s_e \quad (5.23c)$$

$$\alpha = \frac{\tau_{max} s_0}{G_f - G_f^a} \quad (5.23d)$$

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad (5.23e)$$

$$A = \frac{s_0 - s_e}{s_0} \quad (5.23f)$$

$$B = \frac{s_e}{2(s_0 - s_e)} \quad (5.23g)$$

$$s_e = \frac{\tau_{max}}{K_0} \quad (5.23h)$$

$$K_0 = \frac{K_a K_c}{K_a + K_c} \quad (5.23i)$$

$$K_a = \frac{G_a}{t_a} \quad (5.23j)$$

$$K_c = \frac{G_c}{t_c} \quad (5.23k)$$

$$G_f = 0.308 \beta_w^2 \sqrt{f_t} f((K_a)) \quad (5.23l)$$

$$G_f^a = \int_0^{s_0} \tau ds = \tau_{max} s_0 \left(\frac{2A}{3} \left(\frac{1 + B^2 A}{A} \right)^{\frac{2}{3}} - B - \frac{2}{3} B^3 A \right) \quad (5.23m)$$

$$\tau(s) = \begin{cases} \tau_{max} \left(\sqrt{\frac{s}{s_0}} \right) & \text{if } s \leq s_0 \\ \tau_{max} e^{-\alpha \left(\frac{s}{s_0} - 1 \right)} & \text{otherwise} \end{cases} \quad (5.24a)$$

$$\tau_{max} = 1.5 \beta_w f_t \quad (5.24b)$$

$$s_0 = 0.0195 \beta_w f_t \quad (5.24c)$$

$$\alpha = \frac{1}{\frac{G_f}{\tau_{max} s_0} - \frac{2}{3}} \quad (5.24d)$$

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad (5.24e)$$

$$G_f = 0.308 \beta_w^2 \sqrt{f_t} f((K_a)) \quad (5.24f)$$

$$P_u = b \sqrt{\frac{2G_f E_f t_f}{1 + \alpha_T}} \quad (5.25a)$$

$$\alpha_T = \frac{E_f t_f}{E_c t_c} \quad (5.25b)$$

$$P_u = \begin{cases} 0.78b_f\sqrt{2G_fE_ft_f} = 0.78\beta_w b_f\sqrt{2c_fE_ft_ff_{ctm}} & \text{if } L_b \geq L_e \\ 0.78b_f\sqrt{2G_fE_ft_f}\frac{L}{L_e} = 0.78\beta_w b_f\sqrt{2c_fE_ft_ff_{ctm}}\frac{L_b}{L_e} & \text{otherwise} \end{cases} \quad (5.26a)$$

$$L_e = \sqrt{\frac{E_ft_f}{2f_{ctm}}} \quad (5.26b)$$

$$G_f = c_f\beta_w^2 f_{ctm} \quad (5.26c)$$

$$\beta_w = 1.125\sqrt{\frac{2 - b_f/b_c}{1 + b/400}} \quad (5.26d)$$

$$P_u = \begin{cases} \frac{\tau_{max}b_f}{\lambda} & \text{if } L_b \geq L_e \\ \frac{\tau_{max}b_f}{\lambda} \sin(\lambda L_b) & \text{otherwise} \end{cases} \quad (5.27a)$$

$$L_e = \frac{\pi}{2\lambda} \quad (5.27b)$$

$$\lambda^2 = \frac{\tau_{max}}{s_f E_f t_f} (1 + \alpha_Y) \quad (5.27c)$$

$$\alpha_Y = \frac{b_f E_f t_f}{b_c E_c t_c} \quad (5.27d)$$

$$P_u = \frac{\tau_{max}b_f}{\lambda_2} \frac{s_f}{s_f - s_0} \sin(\lambda_2) \quad (5.28a)$$

$$L_e = a_0 + \frac{1}{2\lambda_1} \ln\left(\frac{\lambda_1 + \lambda_2 \tan(\lambda_2 a_0)}{\lambda_1 - \lambda_2 \tan(\lambda_2 a_0)}\right) \quad (5.28b)$$

$$\lambda_1^2 = \frac{\tau_{max}}{s_0 E_f t_f} (1 + \alpha_Y) \quad (5.28c)$$

$$\lambda_2^2 = \frac{\tau_{max}}{(s_f - s_0) E_f t_f} (1 + \alpha_Y) \quad (5.28d)$$

$$a_0 = \frac{1}{\lambda_2} \sin^{-1}\left(0.97\sqrt{\frac{s_f - s_0}{s_f}}\right) \quad (5.28e)$$

$$\tanh(\lambda_1 (L_b - a)) = \frac{\lambda_2}{\lambda_1} \tan(\lambda_2 a) \quad (5.28f)$$

$$G_f = \int_0^\infty \tau(s) ds \quad (5.29)$$

$$\frac{d\varepsilon(x)}{dx} = \frac{1}{E_f t_f} \tau(x) \quad (5.30a)$$

$$\varepsilon(x) = \frac{ds(x)}{dx} \quad \Rightarrow \quad s(x) = \int_0^x \varepsilon(x) dx \quad (5.30b)$$

$$\frac{d^2 s(x)}{dx} - \frac{1}{E_f t_f} \tau(x) = 0 \quad (5.30c)$$

5.2 Description of the experimental tests

In the following, the main details concerning the execution of ten Double-lap Push-pull Shear Tests are reported. This type of test consists in loading in tension two reinforcement strips, symmetrically connected to the support, in order to create shear stresses at the interface and to force the brittle support to be subjected to compressive stresses. In order to perform the related data analyses, the applied load was assumed, as usual *citepvarie*, to be equally distributed on the two strips.

Five samples for high-strength carbon reinforcement (CFRP) and five samples for alkali-resistant glass reinforcement (GFRP) were tested.

5.2.1 Materials characterization

Solid clay bricks (nominal dimension 250x120x55 mm), type 11010C produced by Fornaci Giuliane S.p.A. (Cormons-UD, Italy) were used as substrate, and the MBrace[®]Wet lay-up System as reinforcement. High strength carbon fibres (CFRP) on five specimens and alkali-resistant glass fibres (GFRP) on other five specimens were used. The main bricks properties (average values) are summarized in Table 5.2 (Cartolaro 2004), while the properties of the reinforcement system, obtained from producers datasheets, are reported in Table 5.3.

Table 5.2: Bricks properties

Mean cubic compressive strength	50.94	N/mm ²
Mean direct tensile strength	2.37	N/mm ²
Mean splitting tensile strength	3.99	N/mm ²
Mean flexural tensile strength	5.46	N/mm ²
Secant elastic modulus	16100	N/mm ²

Table 5.3: Reinforcement components properties

Adhesive MBrace[®]Saturant		
Characteristic compressive strength	>80	N/mm ²
Characteristic direct tensile strength	>50	N/mm ²
Maximum tensile strain	2.5	%
Tensile elastic modulus	>3000	N/mm ²
High-strength Carbon MBrace[®]C1-30		
Equivalent thickness	0.165	mm
Characteristic direct tensile strength	3430	N/mm ²
Maximum tensile strain	1.5	%
Tensile elastic modulus	230000	N/mm ²
Alkali-resistant Glass MBrace[®]G60-AR		
Equivalent thickness	0.230	mm
Characteristic direct tensile strength	1700	N/mm ²
Maximum tensile strain	2.8	%
Tensile elastic modulus	65000	N/mm ²

5.2.2 Experimental test set-up

Each sample was made by a single clay brick (Fig. 5.7) with two strips of externally bonded reinforcement (wet lay-up system), symmetrically applied on the opposite wider surfaces. Each strip was 50 mm wide and was bonded to the brick for 200 mm. An unbonded length of 30 mm, from the limit of the brick next to the loaded end, was imposed in order to minimize edge effects. The geometry of the specimen, reported in Table 5.4, is shown in Figure 5.8.

An universal mechanical press, Galdabini SUN60 (maximum load 600 kN), was used as test machine. Each strip of reinforcement, made by a single layer of fibres and two layers of epoxy resin, was bonded at the loaded end to a steel support connected to the test machine. The brick was fixed to the machine through a steel frame, made by two plates linked by bolts (Fig. 5.9 on the facing page).

The load was applied axially and tests were controlled by a displacement rate of 0.2 mm/minute.

On the outer side of one of the two reinforcement strip for each specimen, seven strain gauges were applied, distributed as follows: one on the unbonded zone, next to the loaded end of the reinforcement, and six on the bonded one. To optimize the number of instruments and to monitor the whole bonded region, the strain-gauges were not equally distributed, but less spaced near the loaded end (Fig. 5.10 on the next page).

Table 5.4: Dimensions of the specimen

Brick nominal dimensions			
Length	L_{brick}	250	mm
Width	b_{brick}	120	mm
Thickness	t_{brick}	55	mm
Reinforcement dimensions (single strip)			
Bonded length	L_b	200	mm
Strip width	b'_f	50	mm
Unbonded length	L_{ub}	30	mm



Figure 5.7: Solid clay brick used

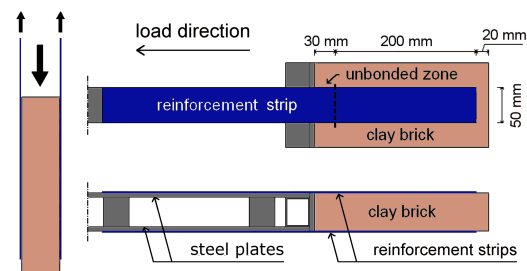


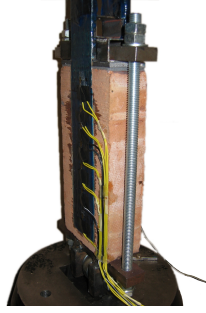
Figure 5.8: Specimen geometry (Panizza *et al.* 2008a)

5.2.3 Test results

After test, all specimens revealed the complete detachment of the reinforcement from the support. The failure involved the brick surface, where curved cracks and ripping of clay pieces were observed (Fig. 5.11 on page 52).



(a)



(b)

Figure 5.9: Test machine (left) and a sample ready for testing (Panizza *et al.* 2008a)

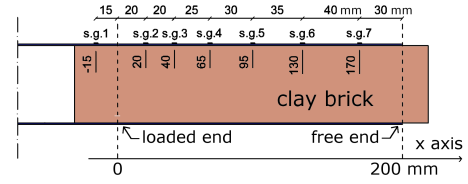


Figure 5.10: Strain-gauges distribution (Panizza *et al.* 2008a)

It was assumed that strain is uniform on the composite cross section, and that is possible to refer mechanical properties to the dry woven: this approach is accepted by many authors (Chen and Teng 2001, Lu *et al.* 2005) and by some guidelines, such as Italian CNR DT-200 (2004). Another possible approach, which can be found in older papers (e.g. Karbhari and Engineer (1996)), starts from the estimation of equivalent properties, such as tensile strength and elastic modulus, related to the actual area of the reinforcement that includes fibres and adhesive; however, on the basis of the common hypotheses, these two approaches could be considered equivalent (CNR DT-200 2004), on the condition that it is possible to measure the actual thickness of the composite.

Therefore, it was possible to calculate the nominal tensile stress on the textile, defined as the load and cross section area ratio. By coupling the strain measures (obtained from the strain-gauge glued the unbonded region) with the load values, and by assuming the linear elastic behaviour of the composite, it was also possible to evaluate Young's modulus of the reinforcement for each sample (Tab. 5.5 on page 53) through a best fitting involving the linear branch of the load-strain curve, as shown in Equation (5.31).

The failure loads for both CFRP and GFRP, P_u , are given in Table 5.5 on page 53, along with the failure load per unit width ($P_u/2b'_f$) and the nominal tensile stress on the cross section at failure (σ_u), and plotted in Figure 5.12 on the next page. It is possible to note as specimens strengthened with CFRP showed better performances than GFRP one: the mean failure load in the first case was around 35% higher than the latter. On the other hand, the nominal tensile stresses, evaluated on the reinforcement at failure, were closer to the strength f_t in the case of GFRP (95% in average) than CFRP (64% in average). Concerning the elastic moduli, they showed a quite large scattering, resulting in a mean value higher than the ones reported in the producer's datasheet (22% for CFRP and 25% for GFRP).

$$E_f = \frac{\sigma}{\varepsilon} = \left(\frac{P}{2b'_f t_f} \right) \quad (5.31)$$

where

E_f = longitudinal elastic modulus of the reinforcement

σ = nominal tensile stress on the reinforcement cross section

ε = longitudinal strain of the reinforcement, measured on the unbonded area

P = applied load

b'_f = width of a single strip of reinforcement

t_f = equivalent thickness of the fibres

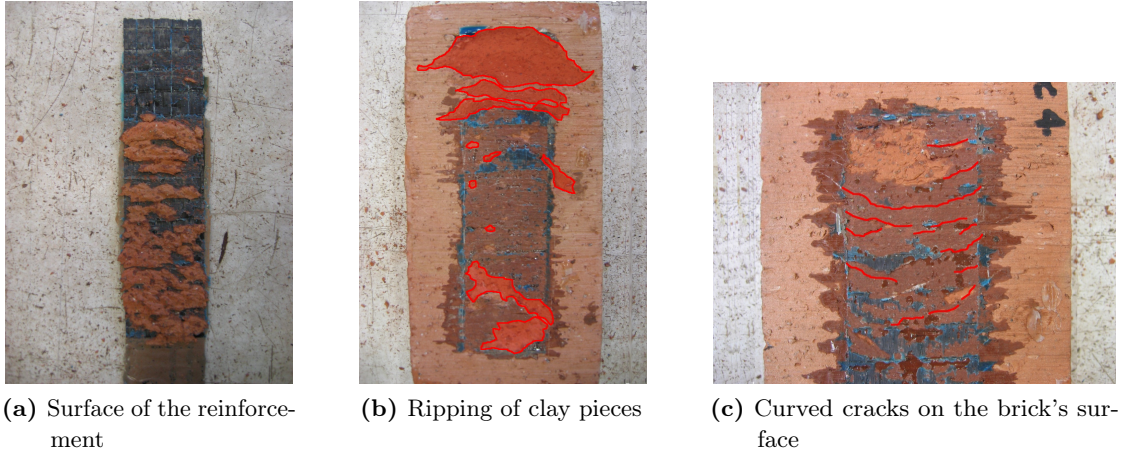


Figure 5.11: Reinforcement and support after the test

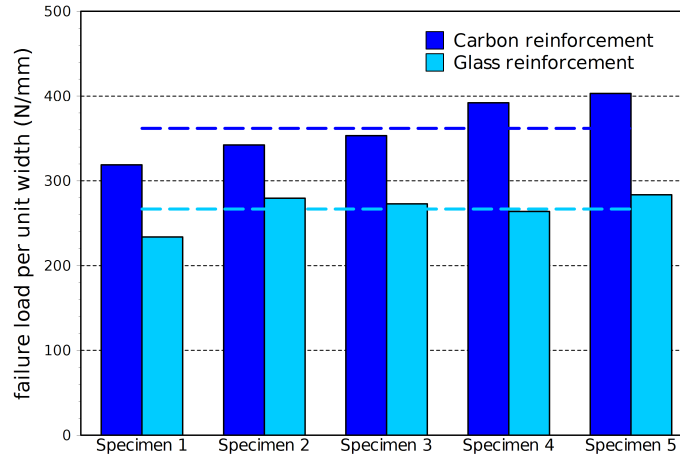


Figure 5.12: Failure loads per unit width

5.3 Tests analysis

5.3.1 Application of predictive models

Twenty one of the aforementioned models, which provides a prediction of the ultimate failure load P_{fu} , were applied in this study, in order to make a comparison with the

Table 5.5: Experimental results for carbon reinforcement

SPECIMEN	P_u N	P_u/b_f N/mm	E_f N/mm ²	σ_u N/mm ²	σ_u/f_t
High-strength Carbon MBrace[®]C1-30					
ShC1	31884	318.8	164419	1932	56%
ShC2	34233	342.3	336439	2075	60%
ShC3	35325	353.3	284991	2141	62%
ShC4	39210	392.1	277511	2376	69%
ShC5	40301	403.0	338456	2442	61%
Average	36191	361.9	280363	2193	64%
Standard deviation	3505	35.1	70696	212	
Coefficient of variation	9.7%	9.7%	25.2%	9.7%	
Alkali-resistant Glass MBrace[®]G60-AR					
ShG1	23380	233.8	50934	1417	80%
ShG2	27940	279.4	87014	1693	100%
ShG3	27300	273.0	80545	1655	97%
ShG4	26400	264.0	102598	1600	94%
ShG5	28360	283.6	84082	1719	101%
Average	26676	266.8	81035	1617	95%
Standard deviation	1985	19.9	18817	120	
Coefficient of variation	7.4%	7.4%	23.2%	7.4%	

experimental results of the tests on clay substrate. The results, given in Tables 5.6 on the following page and 5.7 on the next page, are shown in Figure 5.13 on page 55.

It can be seen that all the predictions, except for the models of Sato and Izumo in the case of carbon reinforcement, underestimate the mean experimental failure load. Moreover, all formulations provide an estimation closer to test results in the case of CFRP than GFRP, except for the Tanaka and Hiroyuki models.

However, results show large differences from model to model: they vary between 44% and 154% of experimental mean failure load, for carbon reinforcement, and between 43% and 85%, for glass one.

5.3.2 Fracture energy calibration

Although the aforementioned relations have been developed concerning the bond between FRP and concrete, they are considered significant also for the clay substrate adopted in the present work, since it behaves as a quasi-brittle material as well.

As previously remarked, in most cases the constant value α_T , or α_Y , which is involved in some models that relates the ultimate load to the fracture energy (has a slight influence on the calculation, since many authors (Savoia *et al.* 2003a, Dai *et al.* 2005, see) report the relation without any constant.

By applying these formulas to the experimental data of this work, it emerges that taking or not into account the parameters α_T or α_Y leads to a difference lower than 2%.

Accordingly with Equation (5.11), it was possible to calibrate the fracture energy G_f through the mean values of failure load and elastic modulus. The estimated quantity, for glass reinforcement, is around 35% higher than carbon one. Moreover, the fracture energy was also calibrated using a regression based on the square root of the axial stiffness (Fig. 5.14 on page 56); as the Equation (5.11) has been adopted, the regression coefficient is the square

Table 5.6: Predictions provided by the applied models (CFRP)

MODEL	$P_{u,pred}$ N	P_u/b_f N/mm	L_e mm	$P_{u,pred}/P_{u,exp}$ %
Tanaka	16634	166		46.0%
Hiroyuki & Wu	15850	159		43.8%
Maeda	25432	254	50	70.3%
Khalifa	24836	248	50	68.6%
Yang	19170	192	100	53.0%
Sato	41490	415	139	114.6%
Iso	27106	271	57	74.9%
Izumo	55704	557		153.9%
Neubauer & Rostásy (1)	28284	283	99	78.2%
Chen & Teng	24532	245	85	67.8%
Monti et al.	32114	321		88.7%
Lu et al. bilinear	21968	220		60.7%
Brosens & van Gemert	35854	359		99.1%
CNR DT-200/2004	26276	263		72.6%
Nakaba	35014	350		96.7%
Savoia	32784	328		90.6%
Neubauer & Rostásy (2)	26648	266		73.6%
Dai & Ueda (1)	32594	326		90.1%
Dai & Ueda (2)	32194	322		89.0%
Lu et al. precise	21968	220		60.7%
Lu et al. simplified	21968	220		60.7%

Table 5.7: Predictions provided by the applied models (GFRP)

MODEL	$P_{u,pred}$ N	P_u/b_f N/mm	L_e mm	$P_{u,pred}/P_{u,exp}$ %
Tanaka	16634	166		62.4%
Hiroyuki & wu	15850	159		59.4%
Maeda	17356	174	84.2	65.1%
Khalifa	17012	170	84.2	63.8%
Yang	14330	143	100.0	53.7%
Sato	11610	116	96.5	43.5%
Iso	16140	161	34.0	60.5%
Izumo	22430	224		84.1%
Neubauer & Rostásy (1)	17948	179	62.7	67.3%
Chen & Teng	15568	156	54.0	58.4%
Monti et al.	20378	204		76.4%
Lu et al. bilinear	13940	139		52.3%
Brosens & van Gemert	22752	228		85.3%
CNR DT-200/2004	16674	167		62.5%
Nakaba	22218	222		83.3%
Savoia	20804	208		78.0%
Neubauer & Rostásy (2)	16910	169		63.4%
Dai & Ueda (1)	20684	207		77.5%
Dai & Ueda (2)	20216	202		75.8%
Lu et al. precise	13940	139		52.3%
Lu et al. simplified	13940	139		52.3%

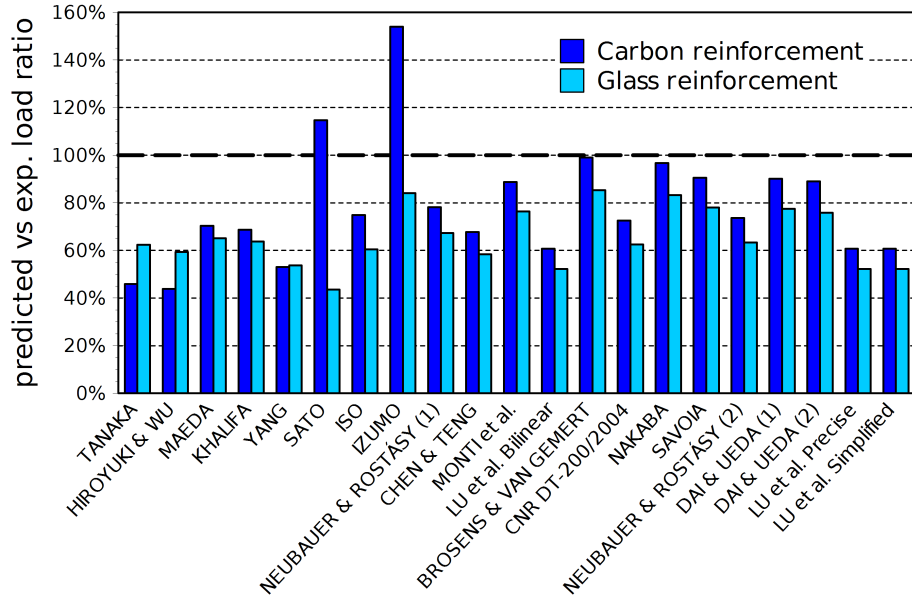


Figure 5.13: Predicted versus experimental load ratio

root of the doubled fracture energy. The results, reported in Table 5.8, show no significant difference from those directly obtained by mean values.

Table 5.8: Evaluation of fracture energy

REINFORCEMENT	G_f from mean values N/mm	G_f from s.r. fitting N/mm
Carbon	1.42	1.41
Glass	1.91	1.91

5.3.3 Calibration of the bond-slip law

To calibrate the bond-slip law on the experimental results, this combined approach was adopted: tangential stress and interface slip points (τ - s) were obtained from strain-gauges monitoring, while the fracture energy value, G_f , was calculated from failure loads through Equation 5.11 on page 44. Fracture energy represents a restraint for the bond-slip function (Eq 5.29 on page 48) and allows to reduce the number of free parameters involved in the calibration process.

Assuming to disregard the slip component of the substrate, sufficiently stiffer than composite, the discrete formulas given in Equation (5.32), used before in Valluzzi *et al.* (2003), Panizza *et al.* (2008a) were used; they allow to manipulate data from devices not uniformly spaced. Notation i-nth indicates the strain-gauge position; the orientation of the x axis makes i increasing from loaded end ($x = 0$) to free end ($x = 200\text{mm}$).

Hence, it was possible to couple slip values with the corresponding tangential stresses. As above explained, it is assumed that the bond-slip law should show an ascending segment and a softening behaviour. For the sake of simplicity, instead of using two different mathematical expressions for the ascending and the descending branch, a single function was chosen. Although there could be a slight loss of adherence to experimental data, it reduces the required parameters making easier the fitting process.

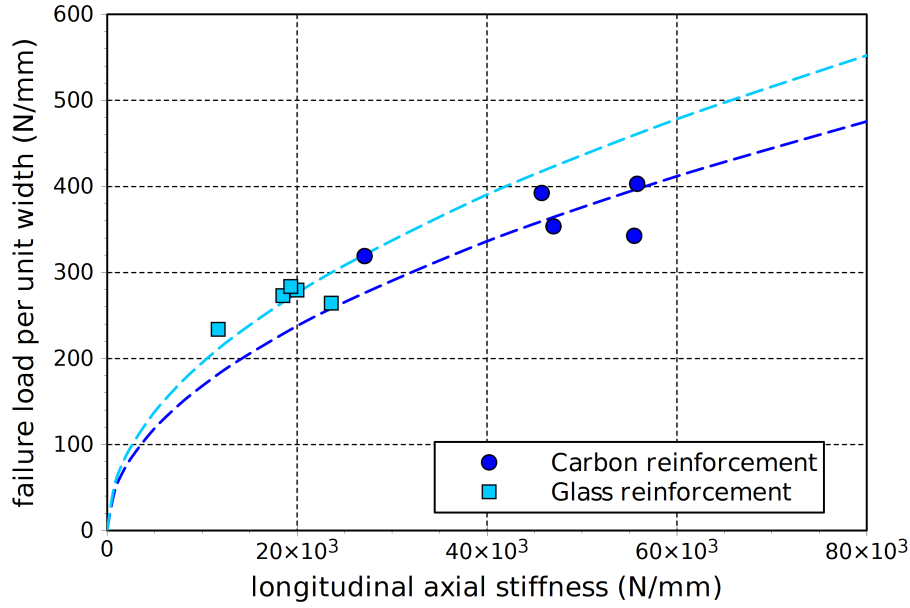


Figure 5.14: Fracture energy regressions based on the square root of the axial stiffness

The proposed law, easy to integrate and derive, is given in Equation (5.33), where A and B are regression constants, τ is the interface tangential stress and s the composite slip. Applying the calibrated fracture energy value, it was possible to have a fitting function that depends on just one parameter, as shown in Equation (5.34). It is useful to rewrite the law, herein labeled UniPd curve (Panizza *et al.* 2008a), in a normalized form (Eq. 5.35 on the next page), where $s_0 = 1/B$ and $\tau_{\max} = \tau(s_0)$ are the coordinates of the point of maximum tangential stress.

After the optimization of the UniPD curves, in the case of carbon reinforcement and glass one, it was possible to calibrate a bilinear law, whose analytical form is reported in Equation (5.12), where s_f is the ultimate strain, related to null stress. This form is commonly proposed by some guidelines (fib Bulletin 14 2001, CNR DT-200 2004).

Since the bilinear function depends on more parameters, to reduce them the peak tangential stress value, obtained from the fitted Unipd curve, and the calibrated fracture energy were imposed. Figures 5.15 on the facing page and 5.16 on page 58 show the optimized curves and the experimental stress-slip data. It can be noticed that carbon reinforcement interface seems to be slight stiffer than glass one.

Tables 5.9 on page 58 and 5.10 on page 59 report the significant values (fracture energy, peak tangential stress with related slip, and ultimate slip) calculated by the fitting process, in the case of CFRP and GFRP, respectively; they have been compared with the values estimated through the eleven models based on the fracture energy prediction previously mentioned.

Estimated values vary into a quite wide range. It has to be noticed that not all models provide significant differences from carbon to glass reinforcement; in particular, the difference between the fracture energy values experimentally calibrated, in the case of CFRP and GFRP, is not in agreement with most predictions.

$$\tau_i = \tau(x_i) = \frac{1}{2} E_f t_f \left(\frac{\varepsilon_i - \varepsilon_{i+1}}{x_{i+1} - x_i} + \frac{\varepsilon_{i-1} - \varepsilon_i}{x_i - x_{i-1}} \right) \quad (5.32a)$$

$$s_i = s(x_i) = s_{i+1} + \frac{1}{2} (\varepsilon_i + \varepsilon_{i+1}) (x_{i+1} - x_i) \quad (5.32b)$$

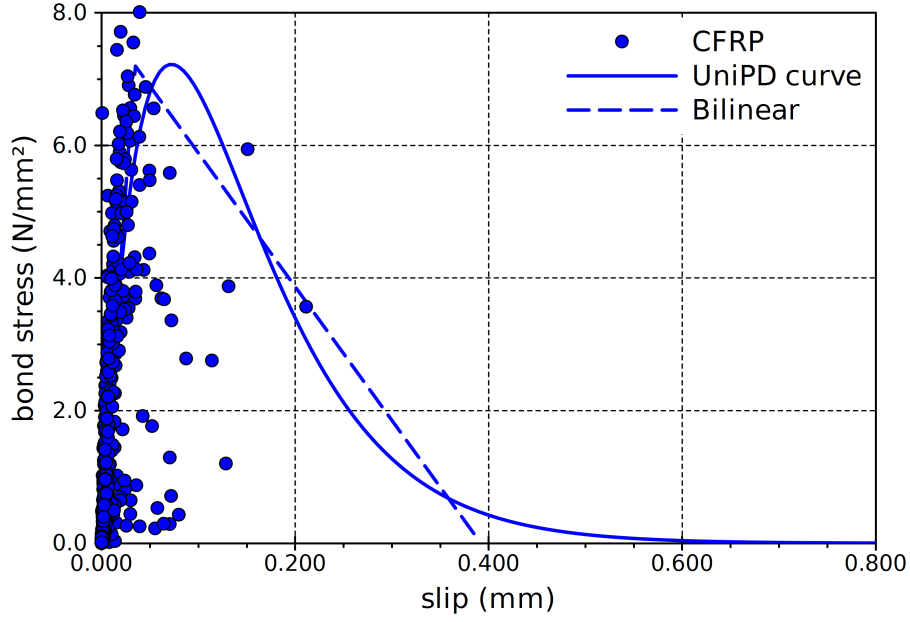


Figure 5.15: Calibrated bond-slip laws for CFRP

$$\tau(s) = Ase^{-Bs} \quad (5.33)$$

$$\int_0^\infty \tau(s) ds = \frac{A}{B^2} \quad \Rightarrow \quad A = B^2 G_f \quad (5.34a)$$

$$\tau(s) = B^2 G_f s e^{-Bs} \quad (5.34b)$$

$$(5.34c)$$

$$\tau(s) = \tau_{\max} \left(\frac{s}{s_0} \right) e^{1 - \frac{s}{s_0}} \quad (5.35)$$

5.4 FE analyses of bond

The bond behaviour of CFRP and GFRP laminates on brick masonry concerning the test specimens was studied by means of the finite element method. Simulations were performed using a refined commercially available non-linear finite element code, DIANA 9.3 (TNO Diana BV. 2008).

In order to reduce the computational burden, the 3D FRP-to-brick specimen was modelled as a 2D plane stress problem by using eight-node quadrilateral isoparametric element for both FRP and brick materials. Due to symmetry, only half of the specimen was modelled. Therefore, suitable boundary conditions were assigned to the brick and the FRP (Fig. 5.17 on page 60).

The Wet lay-up System can be modelled according with three options (Lu *et al.* 2005): (a) a plate with its thickness being equal or similar to the actual thickness of the laminate including all adhesives, but with the fibres concentrated in a thickness equal to the nominal thickness of the fibre sheet laying in the middle of the plate, (b) a plate with its thickness

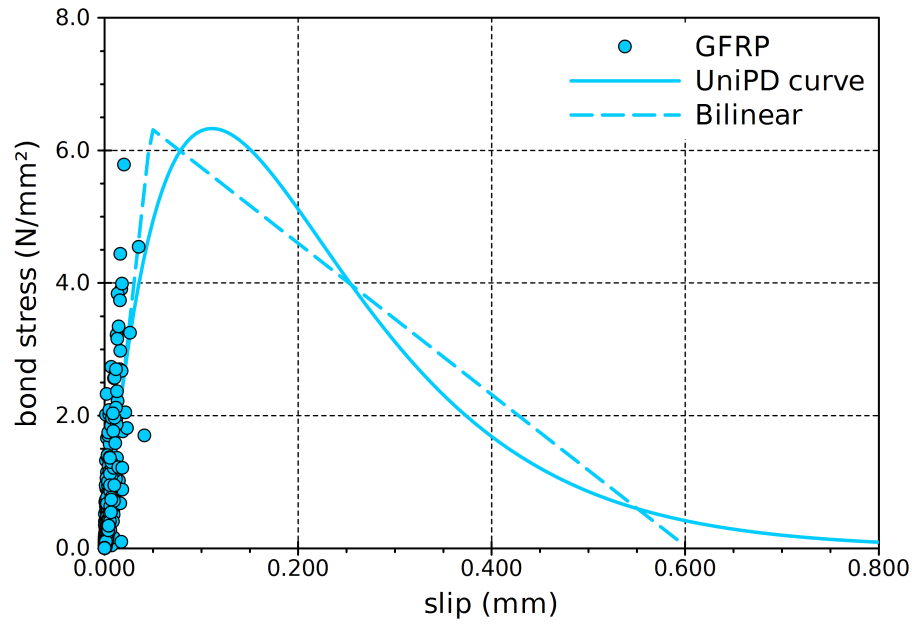


Figure 5.16: Calibrated bond-slip laws for GFRP

Table 5.9: Significant values for local bond of CFRP

CURVE	G_f N/mm	$\tau_{\max}$ N/mm ²	s_0 mm	s_f mm
UniPd fitting	1.42	7.22	0.072	–
Bilinear fitting	1.42	7.22	0.034	0.392
Monti et al.	1.11	5.37	0.046	0.415
Lu et al. Bilinear	0.52	3.73	0.048	0.280
Brosens & V. G.	1.39	2.71	0.012	1.025
CNR	0.75	7.46	0.056	0.200
Nakaba et al.	1.32	7.08	0.065	–
Savoia et al.	1.16	7.08	0.051	–
Neubauer and R.	0.77	5.69	0.270	–
Dai and Ueda (1)	1.15	8.58	0.103	–
Dai and Ueda (2)	1.12	6.41	0.061	–
Lu et al. Precise	0.52	3.73	0.054	–
Lu et al. Simplif.	0.52	3.73	0.048	–

Table 5.10: Significant values for local bond of GFRP

CURVE	G_f N/mm	$\tau_{\max}$ N/mm ²	s_0 mm	s_f mm
UniPd fitting	1.91	6.33	0.111	–
Bilinear fitting	1.91	6.33	0.048	0.603
Monti et al.	1.11	5.37	0.046	0.415
Lu et al. Bilinear	0.52	3.73	0.048	0.280
Brosens and V. G.	1.39	2.71	0.012	1.025
CNR	0.75	7.46	0.056	0.200
Nakaba et al.	1.32	7.08	0.065	–
Savoia et al.	1.16	7.08	0.051	–
Neubauer and R.	0.77	5.69	0.270	–
Dai and Ueda (1)	1.15	7.10	0.107	–
Dai and Ueda (2)	1.10	5.69	0.067	–
Lu et al. Precise	0.52	3.73	0.054	–
Lu et al. Simplif.	0.52	3.73	0.048	–

being equal or similar to the actual thickness of the laminate including all adhesives, with the fibres assumed to be evenly distributed across the plate thickness; and (c) a plate with a nominal thickness (generally the thickness of the fibre sheet) without considering the adhesive. In general, option (a) and (b) are more suitable for modelling FRP laminates since both material properties of fibre and matrix are explicitly considered. In order to examine the differences between options (a) and (b), a comparison of the finite element results was carried out. In the case of option (a), the fibres were modelled by using a full-bonded reinforcement element available in the finite element program, that was inserted in the centreline of the composite material. In the case of option (b), suitable homogenised mechanical properties for the FRP laminates were calculated from the experimental data.

The numerical models are completed by the introduction of six-node finite element interfaces between the FRP laminate and the brick. Therefore, the non-linear behaviour was condensed and modelled by the interface element. The calibrated exponential and bilinear bond stress-slip laws were assigned to the interface element according with the data obtained from Tables 5.9 on the preceding page and 5.10 and Figures 5.15 on page 57 and 5.16 on the facing page. In the finite element model, both FRP and brick are considered as isotropic elastic material.

Summaries of the material properties of brick and FRP used in the finite element analysis are reported in Tables 5.2 on page 49 and 5.3 on page 49, whereas Figure 5.18 on the next page shows the mesh used for finite element models.

Non-linear analyses were performed by using an energy based convergence criterion together with arc-length control method. Monotonic incremental load steps were assigned to the loading end of the FRP laminate. The equilibrium of the system was calculated and the corresponding reactions, internal stresses/strains and deformations of the specimen were obtained.

5.4.1 Concentrated versus distributed fibres

The numerical results, obtained using the exponential bond stress – slip law herein labelled UniPD curve, of the maximum loaded end displacement of FRP (s) and the corresponding load (P) of models with concentrated fibre and distributed fibre are shown in Figures 5.19 on page 61 and 5.20 on page 61.

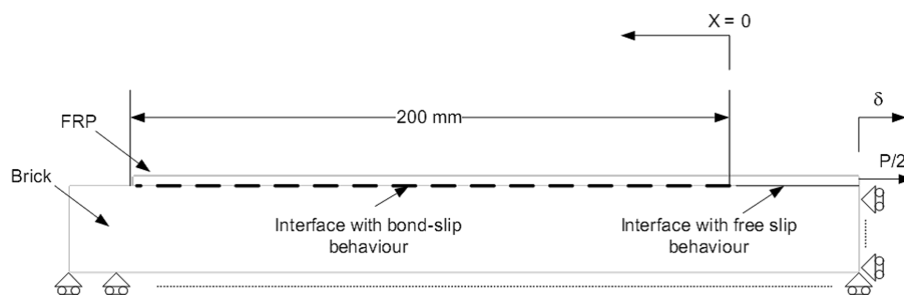


Figure 5.17: Boundary conditions of FE model

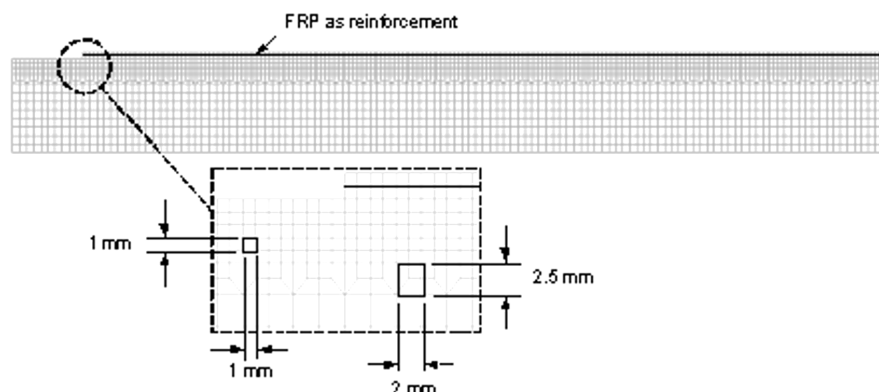


Figure 5.18: Typical fine mesh used in FE models

For the CFRP specimens, the maximum loaded end displacement and the maximum load for both models are almost the same. For the GFRP specimen, it is observed that the model with distributed fibre showed a slightly increase in maximum displacement and maximum load. Nevertheless, the results obtained in both models presented absolutely negligible differences in term of both maximum loaded end displacement and maximum load.

The finite element strain results for both CFRP and GFRP specimens, derived from the models with concentrated and distributed fibres, were compared to the experimental data at three different load levels, namely 30%, 90% and 100% of maximum loading level (Fig. 5.21 on page 62). The numerical results showed a very good agreement with the experimental ones in both cases of concentrated and distributed reinforcement fibres. Therefore, it can be concluded that the two numerical approaches lead almost to the same results, that match well with the experimental data, in the case of both CFRP and GFRP specimens (Lam 2009). Consequently, the distributed fibres approach might be preferred due to its slightly lower computational burden.

5.4.2 Exponential versus bilinear bond stress - slip law

The finite element results of the maximum loaded end displacement of FRP (s) and the corresponding load (P) obtained with the exponential-based (UniPD curve) and bilinear bond stress - slip laws are showed in Figures 5.22 on page 63 and 5.23 on page 63.

The maximum loaded end displacement and maximum load of GFRP and CFRP laminates are practically coincident for both exponential and bilinear bond stress - slip law. However, the model with the bilinear bond law showed a slightly increase in maximum displacement and maximum load. For CFRP laminates the average ultimate loads ($P_{u,ave}$) predicted by the models with exponential and bilinear bond law were the 95% and 96% of the experimental

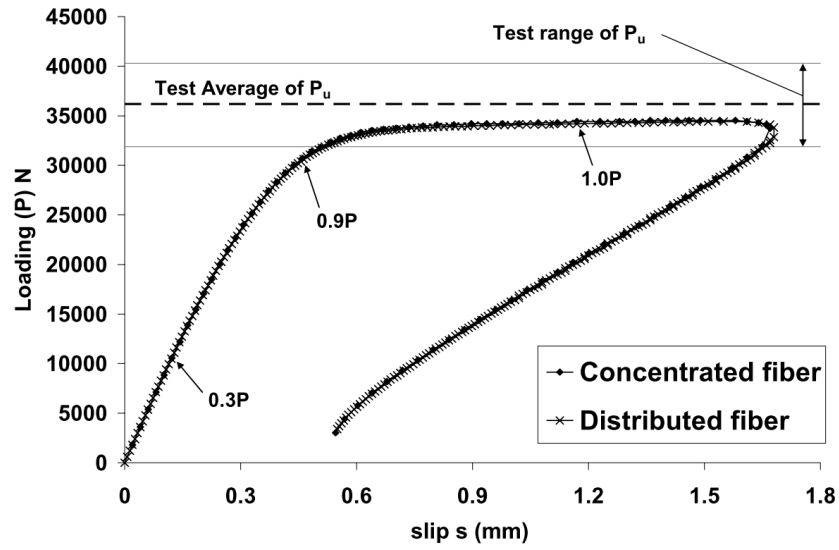


Figure 5.19: Comparison of the load versus loaded end displacement of CFRP specimens with exponential bond stress-slip law (concentrated versus distributed fibres)

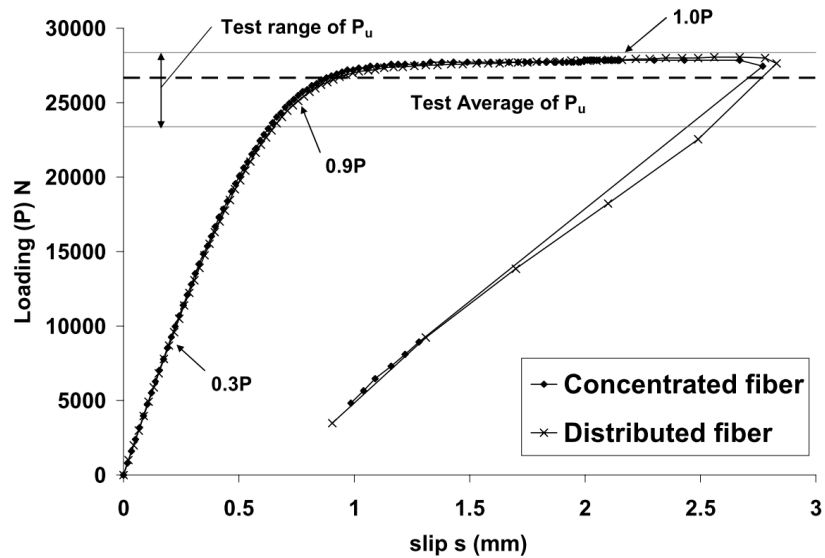


Figure 5.20: Comparison of the load versus loaded end displacement of GFRP specimens with exponential bond stress-slip law (concentrated versus distributed fibres)

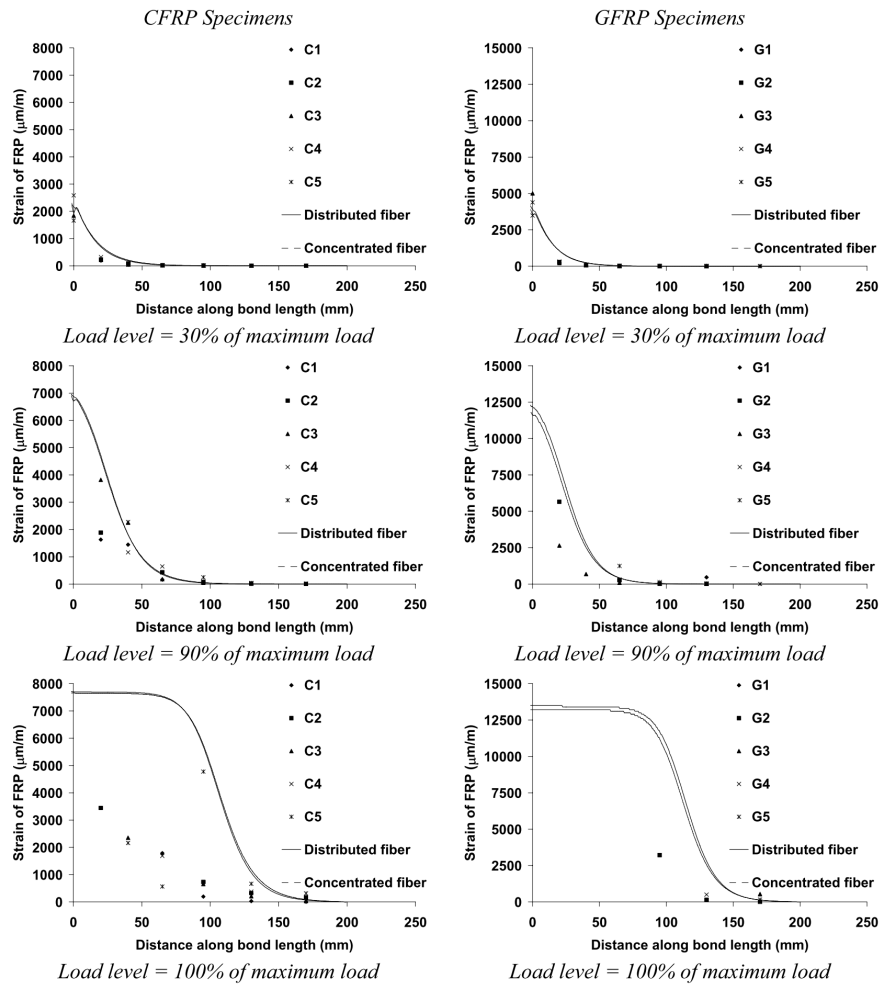


Figure 5.21: Comparison of numerical and experimental strain results for CFRP (left) and GFRP specimens (right) obtained with the exponential bond-slip law (concentrated versus distributed fibres)

mean failure load, respectively, whereas for GFRP laminates the predicted average ultimate loads ($P_{u,ave}$) with exponential and bilinear bond law were respectively the 104% and 106% of experimental one. Therefore, it can be concluded that both the bond laws lead to totally equivalent results in terms of maximum loaded end displacement and maximum load.

The finite element strain results of the models with exponential and bilinear bond law are showed in Figure 5.24 on the next page, compared to the experimental data, for both CFRP and GFRP specimens, at three different load levels (30Therefore, even a simple bilinear bond stress – slip law may be sufficient to properly describe the bond phenomenon between clay bricks (and likely any other quasi brittle material) and EB-FRP laminates, without resorting to more sophisticated local laws.

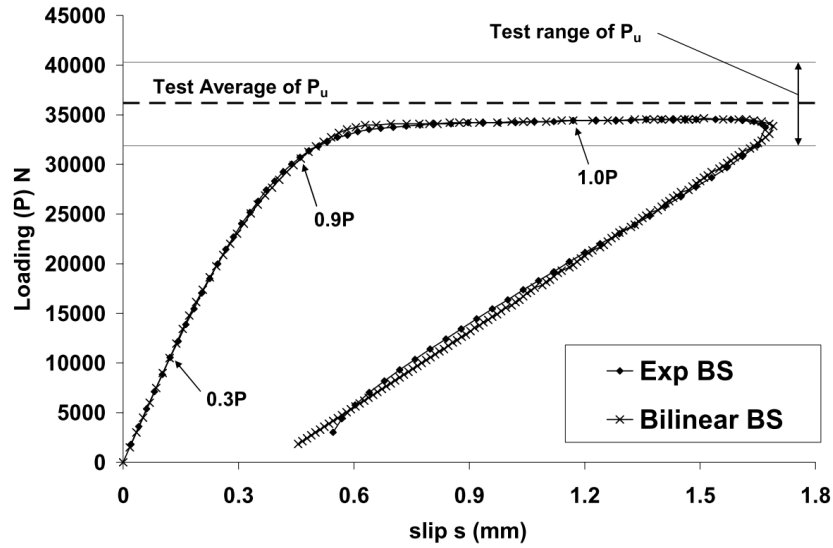


Figure 5.22: Comparison of load versus loaded end displacement of CFRP specimens (exponential versus bilinear bond-slip behaviour)

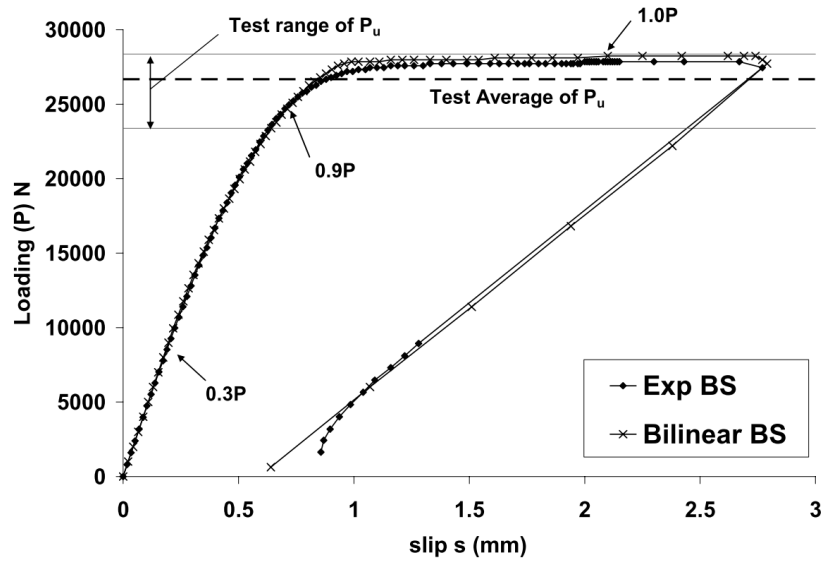


Figure 5.23: Comparison of load versus loaded end displacement of GFRP specimens (exponential versus bilinear bond-slip behaviour)

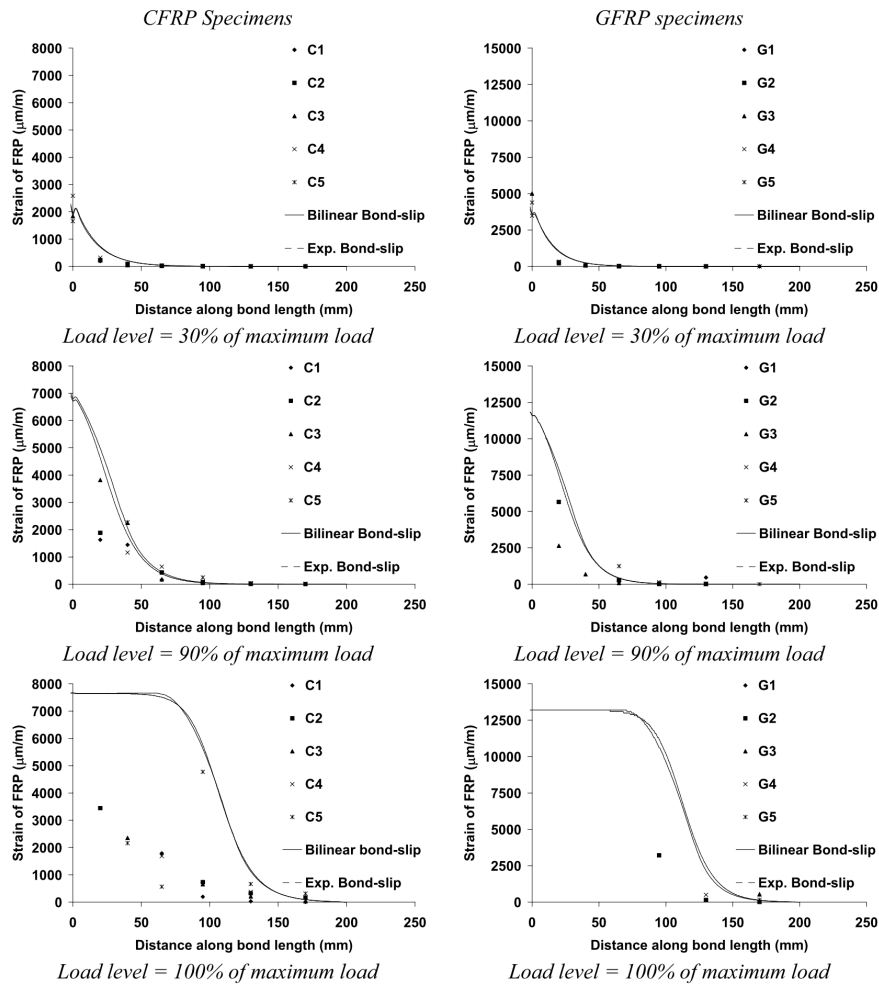


Figure 5.24: Comparison of numerical and experimental strain results for CFRP (left) and GFRP specimens (right) obtained (exponential versus bilinear bond-slip law)

5.4.3 Comparison with the analytical solution

The finite element results were compared with the prediction obtained by the full-range analytical solution proposed by Yuan *et al.* (2001, 2004), that makes use of a bilinear bond-slip law which features a linear ascending branch followed by a linear descending one. The finite element results of load versus slip curves were compared with the analytical solution, in which the FRP is considered as a homogenous material with a homogenised elastic modulus. Thus, the results given by the FE model with distributed fibres were used for comparison. Moreover, in the analytical solution, the actual width of the FRP laminate and brick can be assigned; therefore, two results, one with the width ratio equals to 50/120 and the other with the width ratio equals to 1, are included for comparison.

Figures 5.25 on the facing page and 5.25 on the next page show the results obtained from the finite element analysis and the analytical solution: it can be noticed as the analytical solution matches very well with the FE results, thus validating the use of the analytical model.

In general, for the maximum load, the results derived from the analytical solution are slightly lower than those obtained through the finite element analysis (about 3% lower for both specimens). For the maximum displacement, the results obtained from the analytical

solution are equivalent to those given by the finite element analysis. Furthermore, it can be observed that the analytical maximum loads evaluated using the width ratio of 50/120 are slightly larger than those obtained with the width ratio of 1 (about 3% higher for CFRP specimens and 1% higher for GFRP specimens). The corresponding maximum displacement obtained from the analytical solutions that considered the width ratio of 50/120 are slightly lower than that obtained adopting the width ratio of 1 (about 2% lower for CFRP specimens and 1% lower for GFRP specimens).

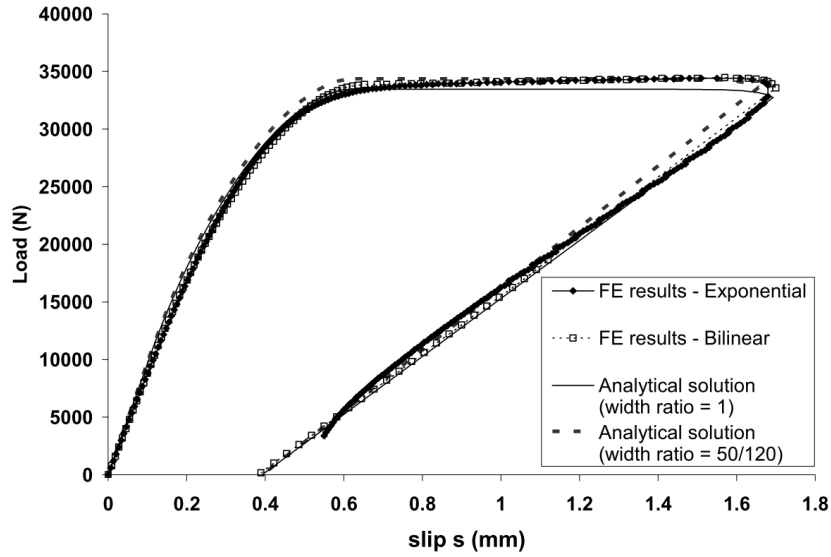


Figure 5.25: Comparison of FE results and analytical solution for CFRP specimens

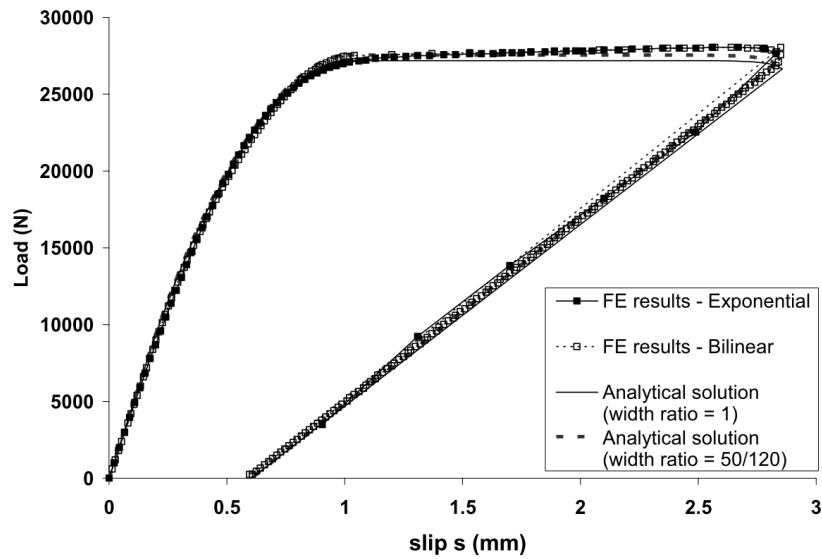


Figure 5.26: Comparison of FE results and analytical solution for GFRP specimens

5.4.4 FEA and analytical results: effect of the dimension of the bonded length

In this part of the work, several finite element models based on the distributed fibres concept were implemented with four different bond lengths, namely 80 mm, 120 mm, 160 mm and 200 mm. The bilinear bond-slip curve with the corresponding interfacial fracture energy was used, for both the FE model and the analytical solution proposed by Yuan *et al.* (2004).

The numerical and analytical results for CFRP and GFRP laminates are presented and compared in Figures 5.27 and 5.28 on the next page, respectively.

The results obtained from the analytical solution were stiffer than those obtained from finite element analyses, as the predicted maximum loaded end displacements were slightly smaller in the first case than in the latter. For the CFRP models with short bond length a higher numerical load than the analytical one was obtained. As the bond length increases, the difference between the finite element and analytical solutions becomes negligible. Nevertheless, the results obtained from the analytical solution matched well with the finite element results, in terms of both load and loaded end displacement, particularly in the case of the GFRP models that showed a very close agreement.

As it can be observed (Fig. 5.27 and 5.28 on the next page), bond lengths longer than 80 mm do not increase the maximum load, due to the fact that 80 mm was already longer than the minimum bond length requirement for both specimens, that can be calculated according to the model proposed by Yuan *et al.* (2001, 2004): the evaluation gives 70.13 mm and 59.41 mm for CFRP and GFRP specimens, respectively. When the bond length is smaller than the minimum required, the maximum load capacity of the joint can not be achieved. However, increasing the bond length has a significant effect only on the maximum loaded end displacement of the joint.

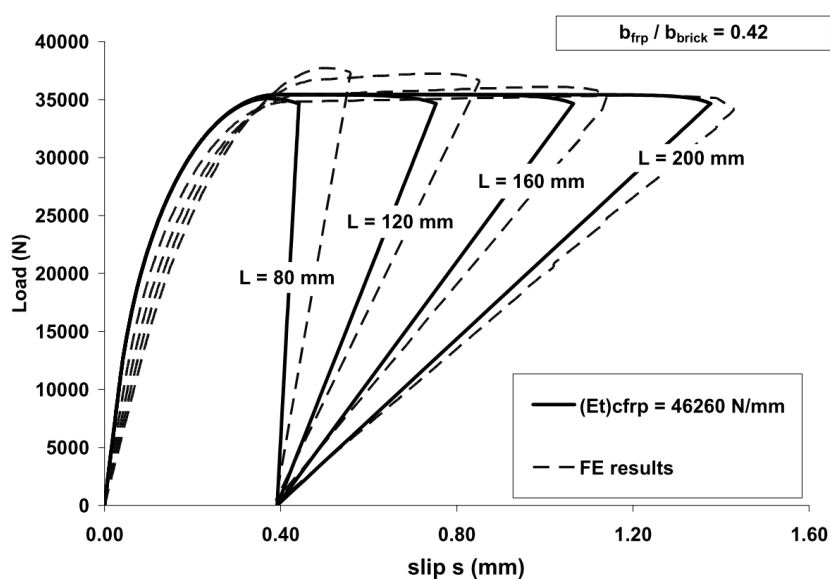


Figure 5.27: Load versus displacement curve of CFRP specimens with different bond lengths (Analytical versus finite element results)

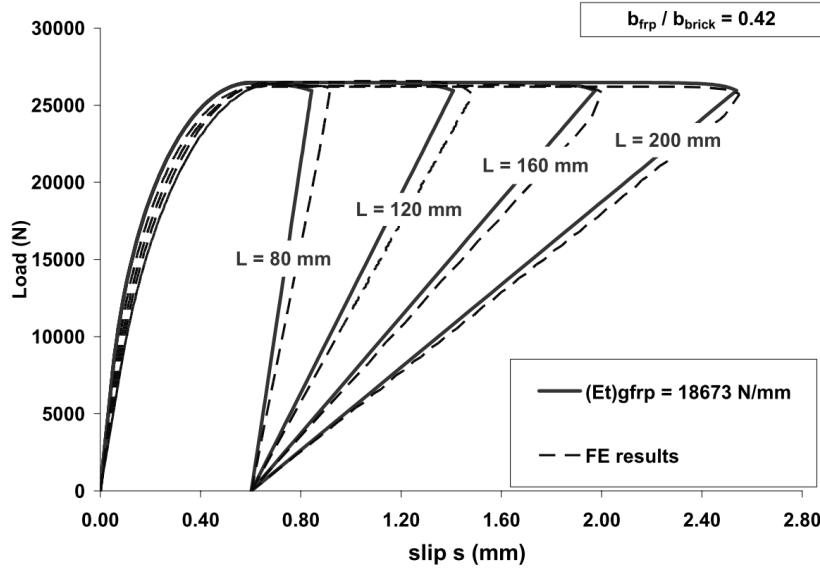


Figure 5.28: Load versus displacement curve of GFRP specimens with different bond lengths (Analytical versus finite element results)

5.5 Calibration of the CNR DT-200 provision for estimating the fracture energy

At present, the CNR DT-200 (2004) are the unique guidelines that explicitly deal with the FRP reinforcement of masonry structures.

Concerning the problem of the plate-end debonding, they propose an approach based on the current concrete-based formulations. An expression (Eq 5.36 on the following page) to evaluate the effective length L_e is provided, which is the same included in fib Bulletin 14 (2001) for CFRP reinforcement and derived by Neubauer and Rostásy modifying a proposal of Holzenkämpfer; E_f and t_f stand for elastic modulus and equivalent thickness of the fibres. The characteristic value of fracture energy, $\Gamma_{F,k}$, is also proposed as proportional to the square root of the product of characteristic compressive, f_{ck} , and tensile, f_{ctm} , strength of concrete (Eq. 5.37 on the next page), similarly to what fib Bulletin 14 (2001) proposes. The k_b factor takes into account the possible size effects, and it has been reported by many authors (see Eq. 5.39 on the following page, where b_f and b are the reinforcement and substrate width, respectively), whereas the k_G factor has been experimentally evaluated as 0.064 (mean value) ± 0.023 (standard deviation), with 0.026 as characteristic value and, therefore, 0.03 as design proposal (CNR DT-200 2004). Finally, the design parameter is given as a limit value of tensile stress for the reinforcement, substantially deduced by dividing the ultimate failure load evaluated through Eq. (5.11) by the equivalent fibres area (Eq. 5.38 on the next page, where $\gamma_{f,d}$ and γ_c are partial factors related to the failure mechanism and the concrete material).

The same approach is adopted in the case of masonry substrate, since the only difference, excluded the different partial factor γ_M related to masonry, is given by the expression of the characteristic fracture energy $\Gamma_{F,k}$ (Eq 5.37 on the following page). Instead of using the size-effect parameter k_b , only one coefficient, c_1 , is proposed. This moves from the fact that, in the case of application on masonry walls, the width ratio between reinforcement and substrate is not clearly defined, and probably less significant, as in the case of reinforced concrete beams. Another remark is related to the substitution of f_{ctm} , mean tensile strength

of concrete, with f_{mtm} , which is defined as the tensile strength of masonry, possibly equal to $0.1f_{\text{mk}}$, but, at the same time, as the mean tensile strength of the elements that compose the masonry, since they assure the development of the resisting mechanism.

$$L_e = \sqrt{\frac{E_f t_f}{2f_{\text{ctm}}}} \quad \text{concrete} \quad (5.36a)$$

$$L_e = \sqrt{\frac{E_f t_f}{2f_{\text{mtm}}}} \quad \text{masonry} \quad (5.36b)$$

$$\Gamma_{F,k} = k_G k_b \sqrt{f_{\text{ck}} f_{\text{ctm}}} \quad \text{concrete} \quad (5.37a)$$

$$\Gamma_{F,k} = c_1 \sqrt{f_{\text{mk}} f_{\text{mtm}}} \quad \text{masonry} \quad (5.37b)$$

$$f_{\text{fdd}} = \frac{1}{\gamma_{f,d} \sqrt{\gamma_c}} \sqrt{\frac{2E_f \Gamma_{F,k}}{t_f}} \quad \text{concrete} \quad (5.38a)$$

$$f_{\text{fdd}} = \frac{1}{\gamma_{f,d} \sqrt{\gamma_M}} \sqrt{\frac{2E_f \Gamma_{F,k}}{t_f}} \quad \text{masonry} \quad (5.38b)$$

$$k_b = 1.125 \sqrt{\frac{2 - b_f/b}{1 + b_f/400}} \quad \text{Neubauer \& Rostásy (Chen and Teng 2001)} \quad (5.39a)$$

$$k_b = 1.06 \sqrt{\frac{2 - b_f/b}{1 + b_f/400}} \quad \text{fib Bulletin 14 (2001)} \quad (5.39b)$$

$$k_b = \sqrt{\frac{2 - b_f/b}{1 + b_f/b}} \quad \text{Chen and Teng (2001)} \quad (5.39c)$$

$$k_b = \sqrt{1.5 \frac{2 - b_f/b}{1 + b_f/100}} \quad \text{Monti (Chen and Teng 2001)} \quad (5.39d)$$

$$k_b = \sqrt{\frac{2.25 - b_f/b}{1.25 + b_f/b}} \quad \text{Lu *et al.* (2004)} \quad (5.39e)$$

$$k_b = \sqrt{\frac{2 - b_f/b}{1 + b_f/400}} \quad \text{CNR DT-200 (2004)} \quad (5.39f)$$

Besides the ten samples tested at the University of Padova, which results have been above presented, data of other shear tests available in literature were collected to the aim of calibrating the fracture energy coefficient for masonry substrates.

Overall 78 tests were considered, 68 of which were taken into account for the calculations, since the unwanted failure modes, such as fibres breaking or support rupture, were excluded: 29 tests on solid clay bricks, but Briccoli Bati (2007, 2009)) reports only average results, 20 reinforced with carbon and 9 with glass fibres (Briccoli Bati *et al.* 2007, Briccoli Bati and Rovero 2009, Capozucca 2010, Faella *et al.* 2009, Grande *et al.* 2008, Panizza *et al.* 2008a); 4 tests on hollow clay blocks (Camli and Binici 2007), reinforced with carbon fibres; 18 tests on natural stones (calcarenite, Naples tuff, leccese stone), 15 reinforced with carbon and 3 with glass fibres (Aiello and Sciolti 2006, Faella *et al.* 2009); finally, 17 tests on masonry

prisms, 6 reinforced with aramidic, 5 carbon and 6 glass fibres (Basilio 2007, Casareto *et al.* 2003). Data are reported in details within the Appendix.

Three procedures were adopted to calibrate the c_1 coefficient: the first and the second ones are based on a regression of the experimental points depending on a single parameter (Eq. 5.40, obtained from 5.11 on page 44 modified by adopting a linear behaviour for the reinforcement, in order to express the ultimate load as a function of the strain at failure $\varepsilon_{\max}$), the third consists in the calculation of the average value, as previously done by other authors (Ferracuti *et al.* 2007a) in the case of concrete.

Where possible, mean experimental values for the mechanical properties of substrate and reinforcement were used for the calibrations.

Figure 5.29 on the next page gives the experimental data in terms of strain at failure $\varepsilon_{\max}$ and abscissa ξ (see Eq. 5.40). It can be noticed that, rather unexpectedly, points related to masonry prisms show a slightly better behaviour.

The results of the calibration of the c_1 factor are reported in Table 5.11 on the next page, while Figures 5.30 on the following page and 5.31 on page 71 show the regression curves compared to the experimental points. The knowledge of the actual elastic modulus of the reinforcement, usually not reported by papers that generally provides only the manufacturer's value, could lead to a more refined calibration, as the trend of UniPadova samples clearly show.

As a matter of fact, it cannot be defined an unique method to calibrate the c_1 coefficient, as the results (Tab. 5.11 on the following page) are quite different: this could be imputed to the still limited database, compared to the wide variability of the materials properties, and to the lack of some experimental data that were replaced by average values taken from literature or datasheets or evaluated by means of common design formulas. However, it can be observed that tests on stone and clay elements show a similar behaviour, while masonry prisms seem to offer slightly higher performances, evaluated on the basis of the previously presented procedure.

For comparison, data related to the shear tests on concrete specimens, reported by Lu *et al.* (2005), were treated using the same procedures previously applied, and superimposed to the masonry data (Fig. 5.32 on page 71).

Finally, the same methods have been applied to calibrate the k_G coefficient, adopting the formulations proposed for concrete substrate by CNR DT-200 (2004). Results are given in Table 5.12 on page 71 and shown in Figures 5.33 on page 72 and 5.34 on page 72.

$$\varepsilon_{\max} = \sqrt{c_1} \frac{1}{\xi} \quad \text{where} \quad \xi = \sqrt{\frac{E_t t_f}{2\sqrt{f_c f_t}}} \quad (5.40a)$$

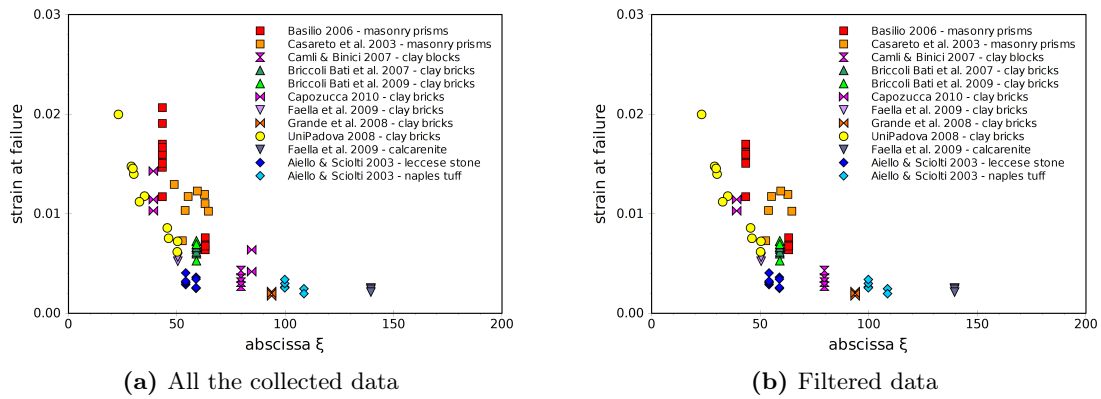
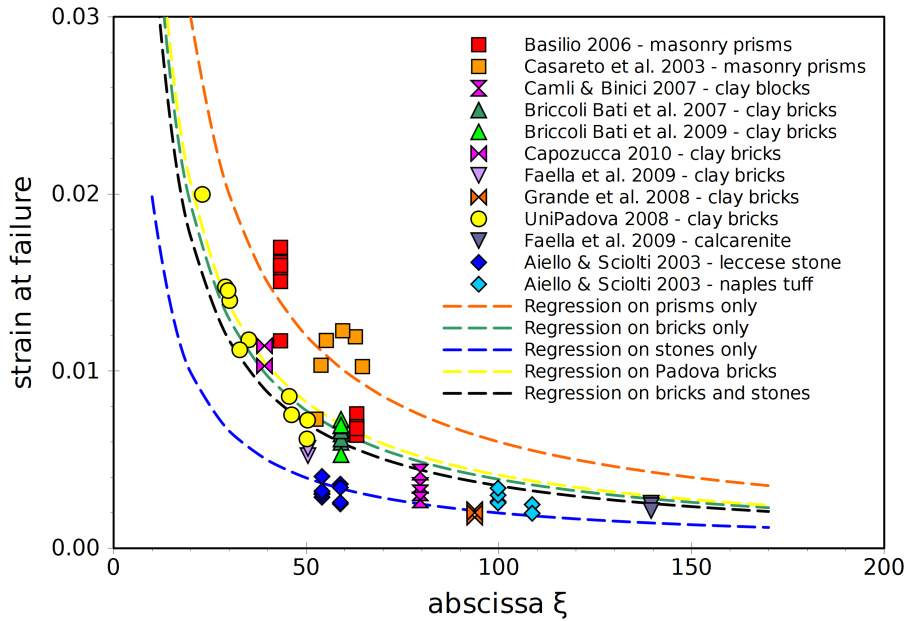
$$\varepsilon_{\max}^2 = \sqrt{c_1} \frac{1}{\xi'} \quad \text{where} \quad \xi' = \xi^2 \quad (5.40b)$$

5.6 Conclusive remarks

The experimental strength has been compared with twenty-one predictive models developed for concrete substrate. All predictions, except two in the case of CFRP, underestimate the results of the tests; the models, except two in the case of GFRP, seem to work better for the carbon reinforcement. However, the strength predictions vary into a wide range (between 44% and 154% of experimental mean failure load for CFRP, 43% and 85% for GFRP).

Table 5.11: Calibrated values for the c_1 coefficient

Data set	First regr. $\varepsilon_{\max} - \xi$	Second regr. $\varepsilon_{\max}^2 - \xi'$	Average values
UniPadova:	0.170	0.184	0.157 ± 0.036
Bricks + stones:	0.124	0.166	0.103 ± 0.057
Bricks only:	0.151	0.177	0.127 ± 0.054
Stones only:	0.039	0.035	0.059 ± 0.034
Prisms only:	0.359	0.395	0.353 ± 0.152
All data (prisms + elements):	0.171	0.204	0.165 ± 0.141

**Figure 5.29:** Shear tests on masonry: experimental results**Figure 5.30:** c_1 regressions (first type) on the experimental data

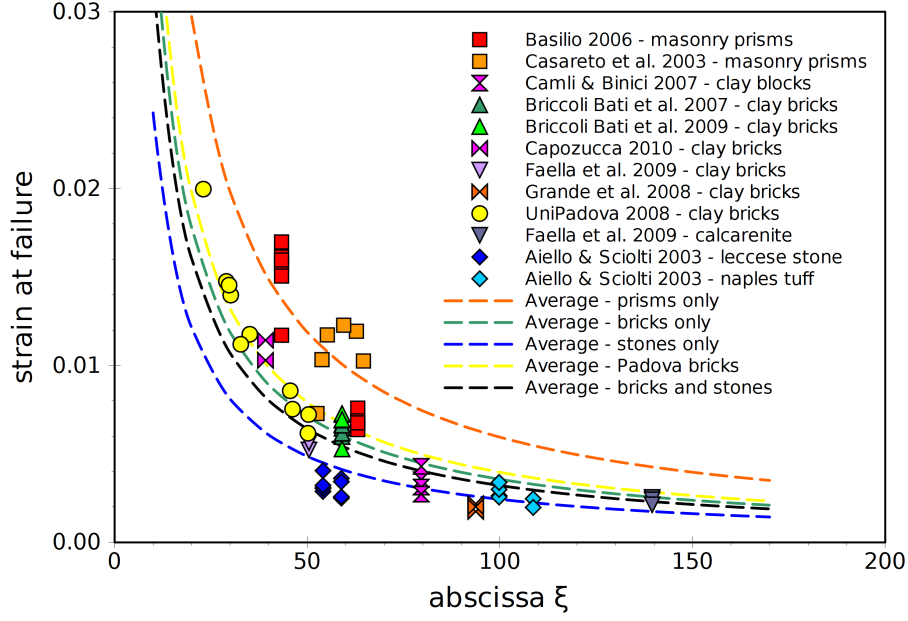
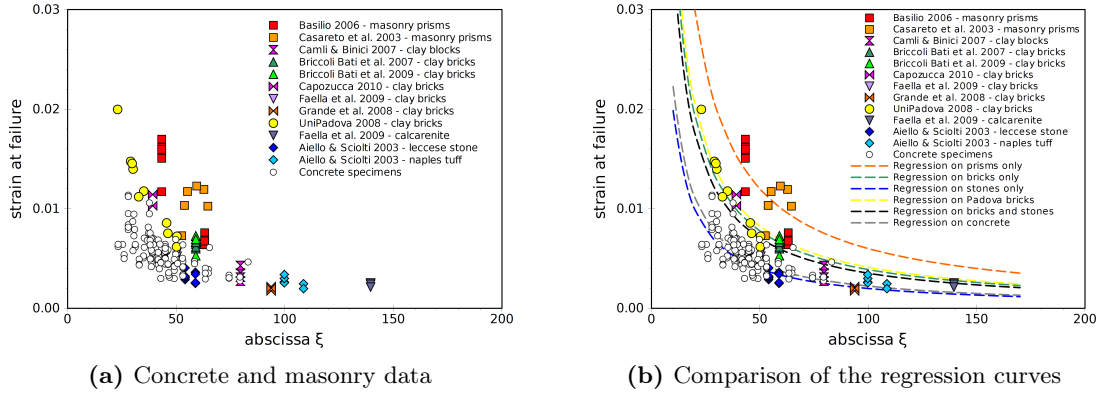
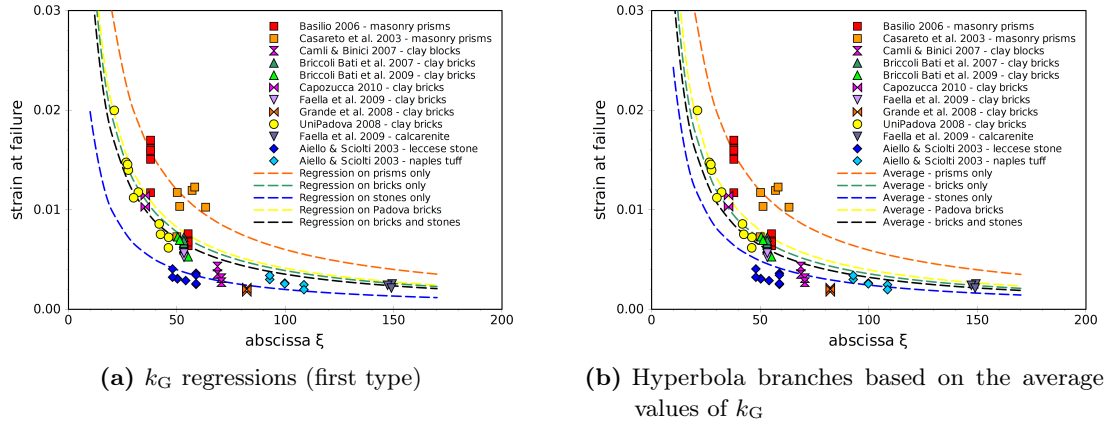
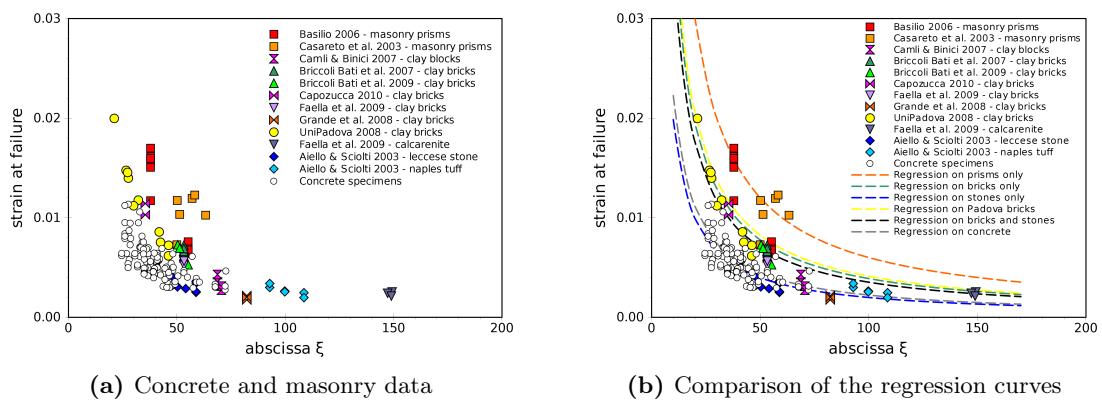

 Figure 5.31: Hyperbola branches based on the average values of c_1

 Figure 5.32: Shear tests on masonry compared to concrete: calibration of c_1

 Table 5.12: Calibrated values for the k_G coefficient

Data set	First regr. $\varepsilon_{\max} - \xi$	Second regr. $\varepsilon_{\max}^2 - \xi'$	Average values
UniPadova:	0.14333	0.15514	$0.132 + 0.030$
Bricks + stones:	0.10637	0.1413	$0.088 + 0.047$
Bricks only:	0.12637	0.14916	$0.105 + 0.044$
Stones only:	0.03612	0.03145	$0.059 + 0.037$
Prisms only:	0.28694	0.31196	$0.289 + 0.131$
All data (prisms + elements):	0.14539	0.173	$0.139 + 0.116$


 Figure 5.33: Shear tests on masonry: calibration of k_G

 Figure 5.34: Shear tests on masonry compared to concrete: calibration of k_G

From the measured failure loads, different fracture energy values have been derived, around 35% higher in the case of glass reinforcement than carbon one.

To analyze stress and slip from strain-gauges measurement, when instruments are not uniformly spaced, discrete equations have been used, consistent with central finite difference methods.

Moreover, a mathematical function easy to integrate and derive is proposed as bond-slip law. This function has been fitted in the case of both carbon and glass reinforcement; beside these fittings, two bilinear functions have been also calibrated. The optimized functions seem to show an interface local behaviour of CFRP slightly stiffer than GFRP.

The reliability of the experimental set-up needs to be verified; despite of its simplicity, the actual distribution of the load should be more clarified, as it could cause undesired bending moments inside the test specimen.

Finite element analyses of the test specimens were carried out implementing the calibrated bond stress – slip laws. Several comparison of numerical results obtained from different models were performed: (1) FRP modelled with the concentrated fibres approach versus FRP modelled with the distributed fibres, (2) exponential-based versus bilinear bond stress – slip law, (3) influence of the bond length. The finite element results were also compared with the results obtained from the analytical solution proposed by Yuan *et al.* (2001, 2004). As it has been shown, the analytical solution provides reasonable prediction of the maximum load and loaded end displacement. The results pointed out that the concentrated fibres approach is equivalent to the distributed one, although the last one might entail a slightly lower computational burden. Moreover, the use of a simple bilinear bond stress – slip law can be sufficient to properly describe the bond phenomenon, without resorting to more sophisticated local laws. The combination of numerical modelling and analytical solution provided the minimum required bond length both for CFRP and GFRP applied on clay brick, which are of 70.13 mm and 59.41 mm, respectively. The finite element analyses highlighted that the analytical solution proposed by Yuan *et al.* (2001, 2004) can be profitably used as design procedure for the studied EB-FRP system.

Concerning the calibration of the c_1 coefficient, related to the evaluation of the fracture energy proposed in CNR DT-200 (2004), the obtained results show a similar behaviour of stones and bricks, although a quite large scattering depending on the calibration method has been observed, whereas masonry prisms seem to offer higher performances. The reliability of these results, however, is strongly affected by the limited base of data currently available and by the lacking of some crucial data that are rarely given by the common sources.

Chapter 6

Interface behaviour in the case of mixed actions

This chapter reports the main results of a series of V-shape Peel Tests (herein somewhere labelled as V-peel for the sake of brevity), which experimental program is given in Table 6.1, carried out on solid clay bricks (coming from the same sets used for performing the pull-off tests) reinforced with high-strength carbon fibers (CFRP) bonded with epoxy to their surface. The aim of this type of test was to investigate the behaviour of the FRP-clay interface when subdue to mixed forces acting normally and on a parallel plane with respect to the reinforcement, since this condition should be verified in the case of incipient sliding on a mortar joint near the springer of a masonry arch reinforced at its intrados.

Table 6.1: Experimental program of the V-Shape Peel Tests

SPECIMEN	brick type	loading path
Vp01 (pilot test)	S4 - facing	monotonic (A)
Vp02 (pilot test)	S3 - extruded	cyclic (A)
Vp03	S3 - extruded	cyclic (A)
Vp04	S4 - facing	cyclic (A)
Vp05	S4 - facing	monotonic (A)
Vp06	S4 - facing	cyclic (A)
Vp07	S3 - extruded	cyclic (B)
Vp08	S3 - extruded	cyclic (B)
Vp09	S3 - extruded	monotonic (B)
Vp10	S3 - extruded	cyclic (B)
Vp11	S4 - facing	cyclic (B)
Vp12	S5 - extruded	monotonic (B)
Vp13	S2 - facing	monotonic (C)
Vp14	S2 - facing	cyclic (C)

6.1 Literature review

Peeling usually means the detachment between two materials connected by an adhesive layer; peel angle indicates the angle between the external force (peel force, generally a concentrated load positioned at the end) and the materials' plane. It is frequently supposed that the stiffness of one of the two elements is sufficiently higher than the other to let it be considered as a rigid material.

To analyze the peeling phenomenon, Bikerman (1957) moved from a peel angle of 90 degrees and identified the failure criterion as the maximum elongation of the adhesive, evaluated using the theory of beams on elastic soil (Fig. 6.1 on the facing page), reported by Briccoli Bati and Rovero (2008) as a possible evaluation of the FRP contribution to the shear strength of masonry arches with extrados reinforcement.

Gent and Hamed (1975) compared the result of an energy-based approach to the one related to the evaluation of the tensile stress on the adhesive, and imputed the differences to the energy released by local plastic deformations.

Nicholson (1977) developed an analysis based on great deformations hypotheses and demonstrated the coincidence of this approach to the energy-based ones, in terms of peel force and peel angle relation.

Thouless and Jensen (1992) performed an analysis base on linear elastic fracture mechanics on the peel test, recently reported by De Lorenzis and Zavarise (2008).

Yuan *et al.* (2003) proposed an analytical solution for normal and tangential stresses that affect the interface between adhesive and FRP laminates.

Sun *et al.* (2004) treated analytically and numerically the peel tests, defined by them as pull-off test (Fig. 6.2 on the next page).

Karbhari and Engineer (1996) performed various peel tests on concrete substrate, changing the imposed peel angle (Fig. 6.3 on the facing page). Dai and Ueda (2003) carried out experiments to measure mode I, mode II and mixed-mode fracture energies of FRP bonded on concrete (Fig. 6.4 on page 78), reporting that the fracture energy mode II is not considerably influenced by fibres stiffness, depending first on the adhesive properties and, then, on the concrete strength, whereas the fracture energy mode I depends mainly on the substrate properties; concerning the mixed-mode fracture energy, the involved effective length was rather short and the strength against the peel load, defined as dowel force, was rather low.

Other studies (Wu *et al.* 2005b,a) on the mixed-mode load conditions (Fig. 6.5 on page 78) were performed in Japan in relation to the repair technique consisting on applying FRP fibres on the intrados of tunnels and bridges in order to prevent the spalling of deteriorated concrete (Ueda and Dai 2005).

Concerning the experiments on concrete substrate, it was pointed out that (Wu *et al.* 2005a, Ueda and Dai 2005) the peel force per unit width and the peel angle, during the detachment, remain almost constant; it was also observed that the peel strength is quite low, and that the presence of a peel force (e.g. beams subject to shear and flexure) may affect the global resistance causing a premature detachment of the FRP laminate (Ueda and Dai 2005). Figure 6.6 on page 78 compares experimental and idealized load-displacement curves (Dai *et al.* 2007).

The formulas afterwards adopted to analyze the fracture energy are reported in Equations (6.1) and (6.2), following Thouless and Jensen (1992), Karbhari and Engineer (1996), De Lorenzis and Zavarise (2008), and (6.3), following Wu *et al.* (2005a), whereas the reference schemes are shown in Figure 6.7 on page 79.

$$F_0 = F \cos(\alpha) \tag{6.1a}$$

$$M_0 = \sqrt{\frac{E_f t_f^3}{6} \left(\frac{F^2 \sin^2 \alpha}{2 E_f t_f} \right) + F(1 - \cos \alpha)} \tag{6.1b}$$

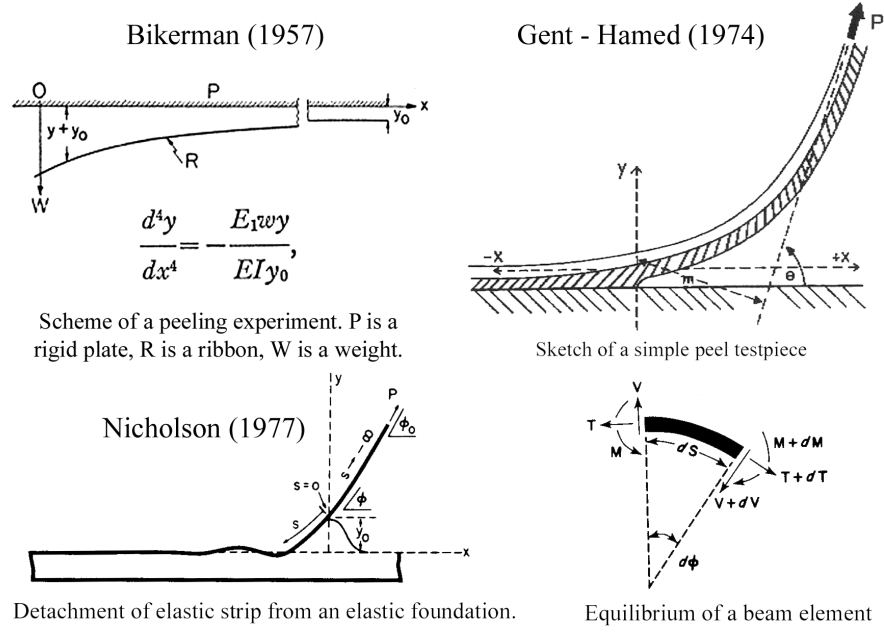


Figure 6.1: Schemes adopted for analysis (Bikerman 1957, Gent and Hamed 1975, Nicholson 1977)

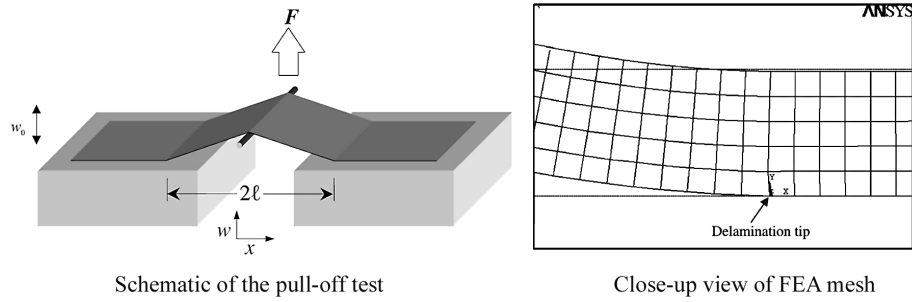


Figure 6.2: Peel analyses performed by Sun *et al.* (2004)

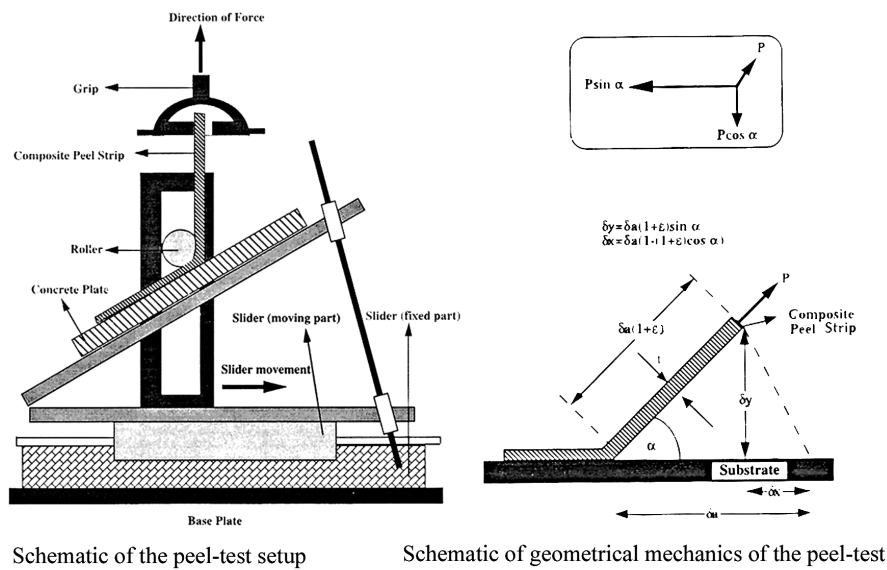


Figure 6.3: Peel test performed by Karbhari and Engineer (1996)

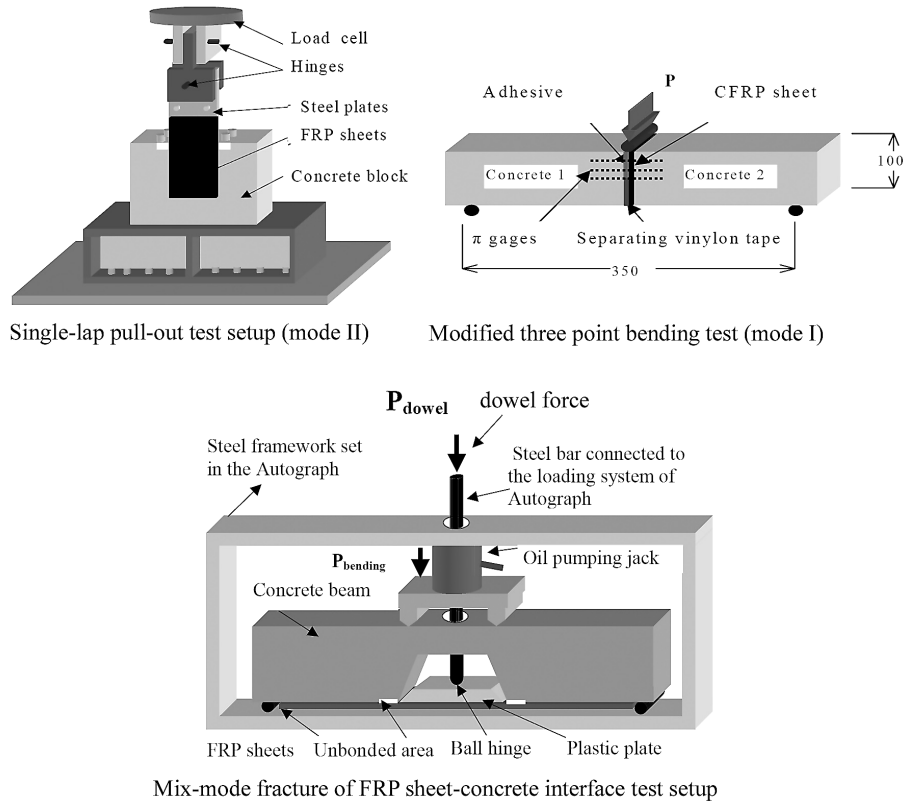


Figure 6.4: Peel test performed by Karbhari and Engineer (1996)

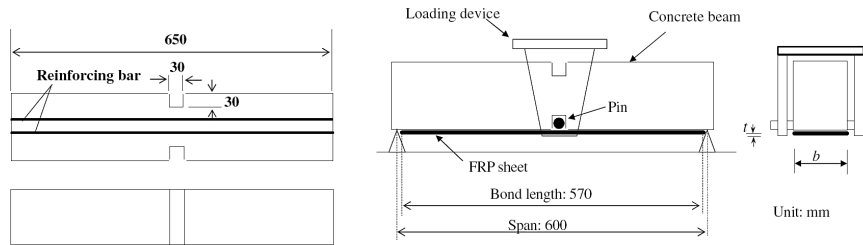


Figure 6.5: Peel test performed by Wu *et al.* (2005a)

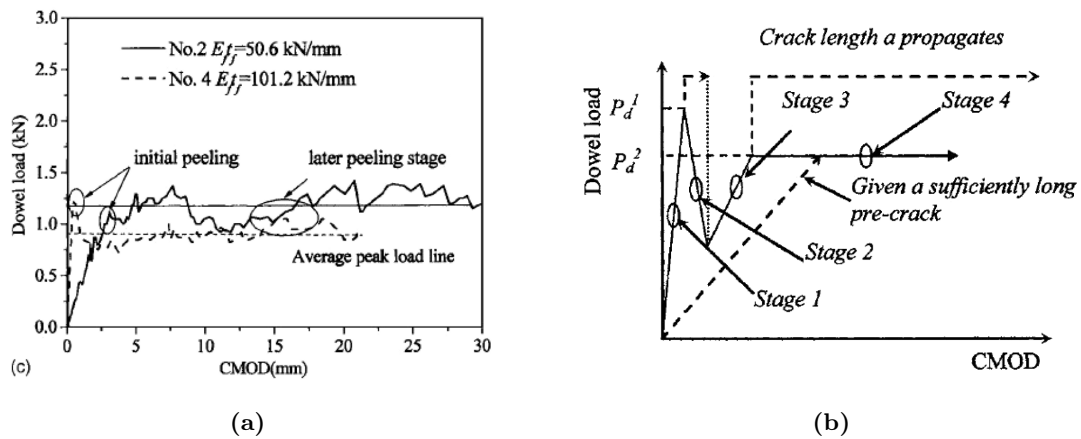


Figure 6.6: Experimental (left) and idealized load-displacement curve (Dai *et al.* 2007).

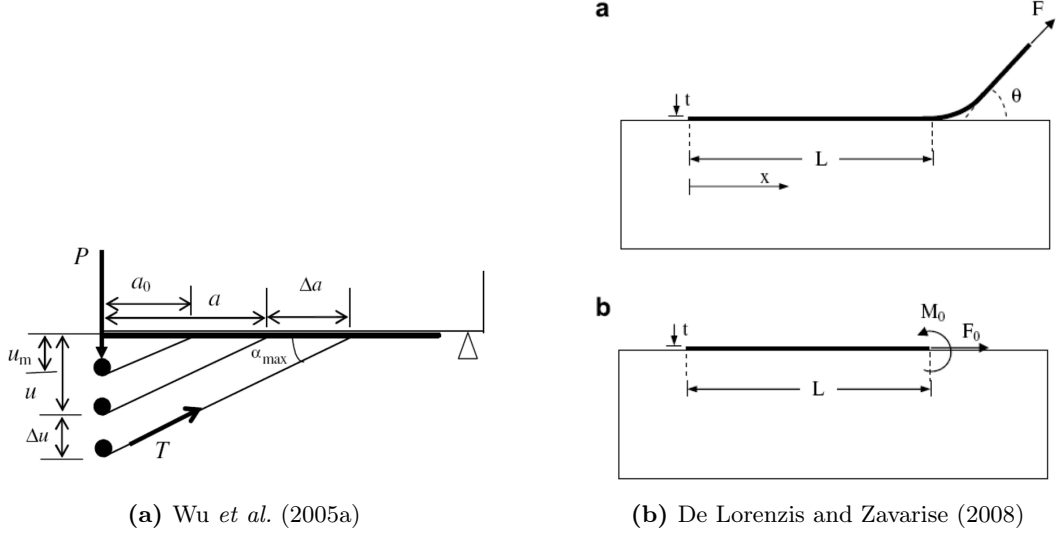


Figure 6.7: Reference schemes for the adopted fracture energy analyses

$$G_{II} = \frac{F_0^2}{2E_f t_f} \quad (6.2a)$$

$$G_I = \frac{6M_0^2}{E_f t_f^3} \quad (6.2b)$$

$$G = G_I + G_{II} \quad (6.2c)$$

$$\psi = \text{atan} \frac{t_f F_0}{\sqrt{12} M_0} \quad (6.2d)$$

$$P_{\max} = 2.087 b_f G^{0.75} (E_f t_f)^{0.25} \quad (6.3)$$

6.2 Description of the experimental tests

6.2.1 Basic materials

As previously mentioned, solid clay bricks belonging to the same sets used to perform the pull-off tests, namely S3 and S5 (extruded elements) and S2 and S4 (facing ones), were used as substrate; therefore, the mechanical characterization is given in 4 on page 15. One type of fibres was used as reinforcement, namely high-strength carbon MBrace[®] C1-30, which was the same used for Shear Tests and pull-off tests, hence its properties can be found in 5 on page 37 and 4 on page 15.

6.2.2 Experimental test set-up

The test set-up, aimed at reproducing the detachment mechanism related to the sliding on a mortar joint of a masonry arch reinforced at the extrados, is shown in Figure 6.8 on the following page. Two solid clay bricks were disposed along their axis and bonded to a top steel beam; the beam was sufficiently longer to be provided of two lateral steel supports sustained by the roller support of an apposite device generally used to perform flexural tests (see 4 on page 15). Therefore, the bricks were connected only to the top beam, being spaced

of 30 mm one from the other, in order to accommodate the steel pin used to apply the peel force on the reinforcement, and sufficiently spaced from the lateral supports to avoid unwanted contact during the test. On the bottom surface of the bricks, a reinforcement strip 50 mm wide was applied, glued to the end of the bricks imposing an unbonded distance of 30 mm from each central edge of the brick. A detailed scheme is reported in Figure 6.9 on the facing page.

Tests were performed on the universal test machine Galdabini SUN2500 (maximum load 2.5 kN), equipped with an additional load cell connected to the external acquisition system. The vertical load was transmitted by a solid steel ring surrounding the top beam and connected to a steel pin acting directly on the FRP strip. Each specimen was monitored by six coupled displacement devices (potentiometers) measuring the lowering in three positions: at the central point, corresponding to the pin, and the beginning of the bonded areas (points A, B, C and D). Three strain-gauges were applied to the FRP bottom surface of part of the samples. Figure 6.10 on the next page shows the positions of instruments.

The loading path, controlled by displacement rate, was monotonic for five specimens and cyclic for nine, and a pre-loading force equal to 10 N was always applied in order to accommodate part of the deformations related to the test system. Being exploratory tests, some parameters were adjusted during the campaign. Table 6.2 on page 82 reports the main properties of the adopted test procedures.

Figure 6.11 on page 82 shows images taken during the preparation of samples.

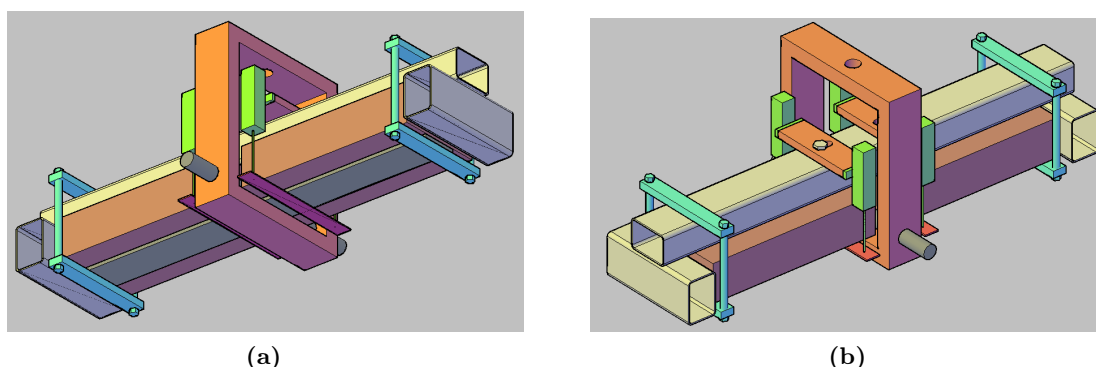


Figure 6.8: Model of the V-shape Peel test (Giomo 2008)

6.2.3 Test results

Images taken during the tests are reported in Figure 6.12 on page 83, while typical load-displacement curves for monotonic and cyclic tests are shown in Figures 6.14 on page 84 and 6.15 on page 84. Failure generally involved the detachment of a thin layer of brick, in the case of facing elements, whereas it was localized at the interface of extruded bricks, without ripping any clay portion (Fig. 6.13 on page 83). Results (Tab. 6.3 on page 85), in terms of maximum vertical load P_u , were rather variable (Tab. 6.4 on page 85), particularly for extruded bricks that showed the higher dispersion. Facing bricks generally offered larger values of strength, compared to extruded bricks. This fact could be correlated to the different failure mode observed: facing elements, whose surface is more scabrous and irregular, could have involved a stronger adhesion of the reinforcement, differently from the smoother and more compact extruded bricks, that affected the clay substrate.

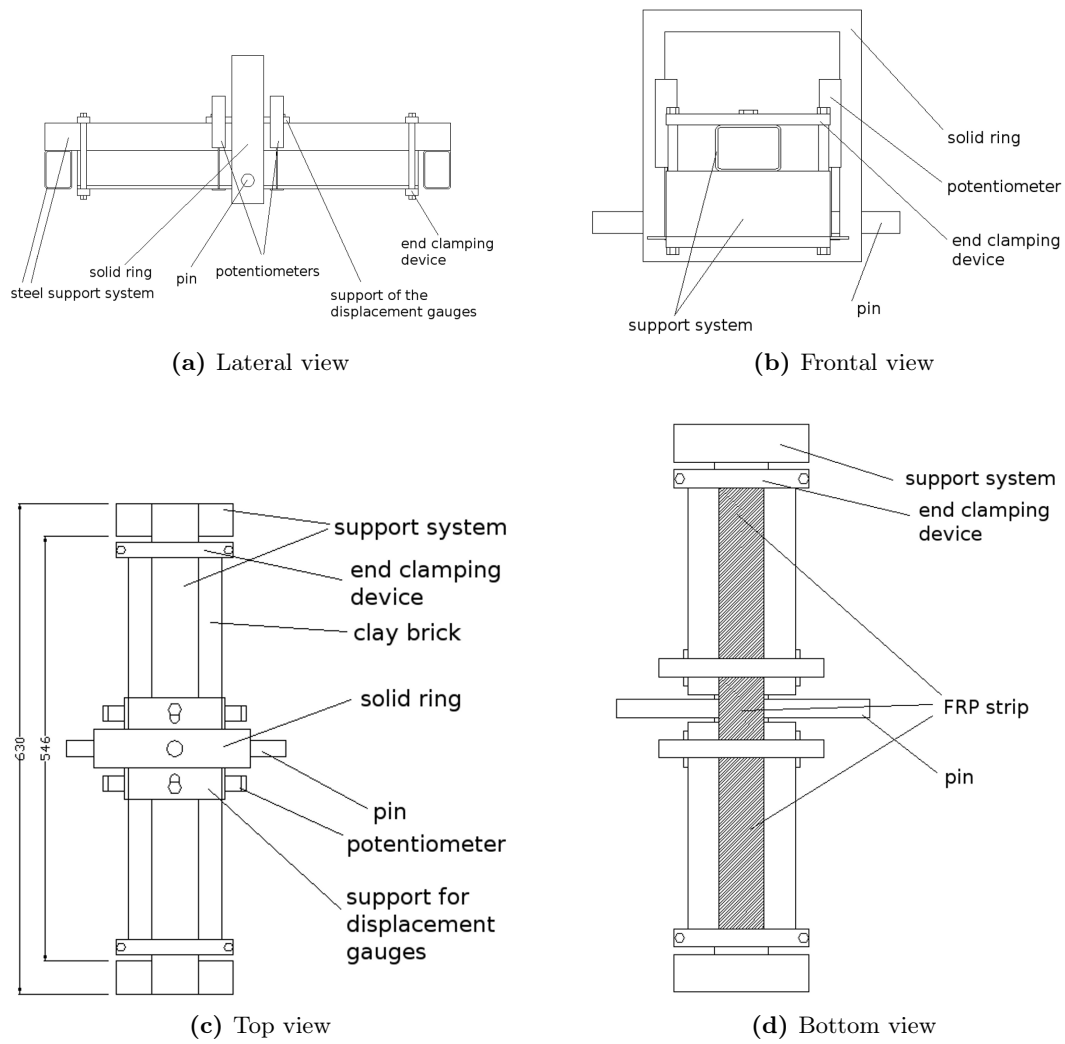


Figure 6.9: Scheme of the V-shape Peel test

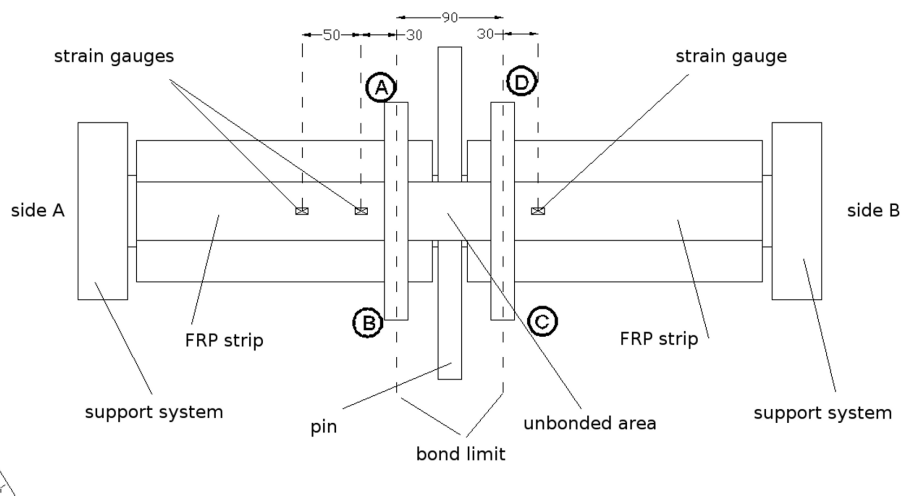


Figure 6.10: Positions of the applied measurement devices



Figure 6.11: Images taken during the specimens preparation: device for flexural tests (A), two samples before the FRP application (B), bonding of fibres (C) and specimens ready for testing (D)

Table 6.2: Predicted failure loads – extrados reinforcement

Phase	Displ. rate	pin direct.	duration	Displ. rate	pin direct.	duration
CYCLIC (A)				MONOTONIC (A)		
1	2 mm/min	down	180 s	2 mm/min	down	up to failure
2	2 mm/min	up	150 s			
3	2 mm/min	down	600 s			
4	back to phase (2)					
CYCLIC (B)				MONOTONIC (B)		
1	1 mm/min	down	360 s	1 mm/min	down	up to failure
2	2 mm/min	up	150 s			
3	2 mm/min	down	600 s			
4	back to phase (2)					
CYCLIC (C)				MONOTONIC (C)		
1	0.6 mm/min	down	600 s	0.6 mm/min	down	up to failure
2	1.2 mm/min	up	250 s			
3	1.2 mm/min	down	500 s			
4	back to phase 2					

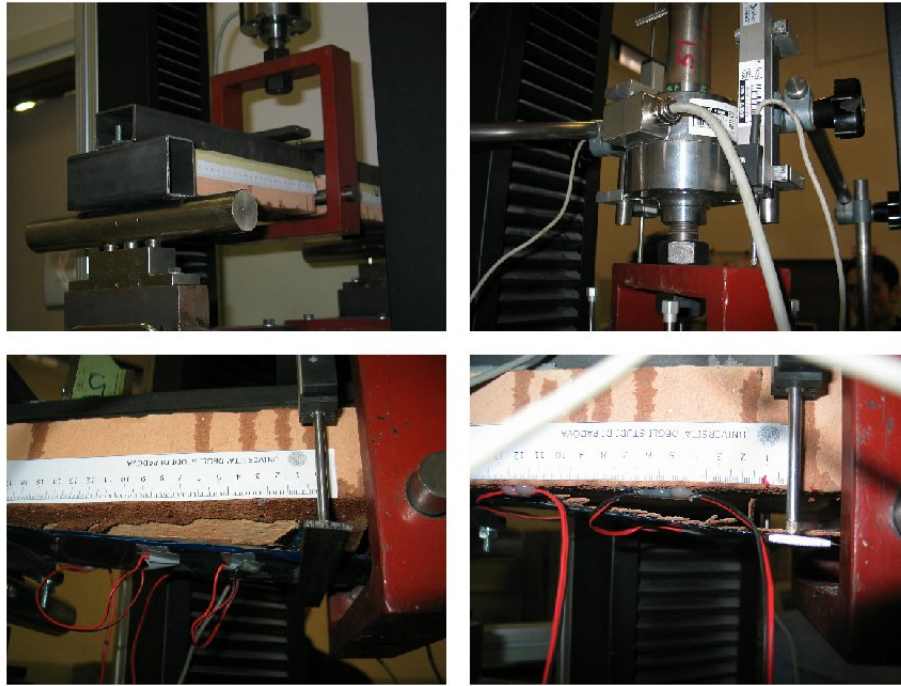


Figure 6.12: Images taken during the V-shape Peel Tests

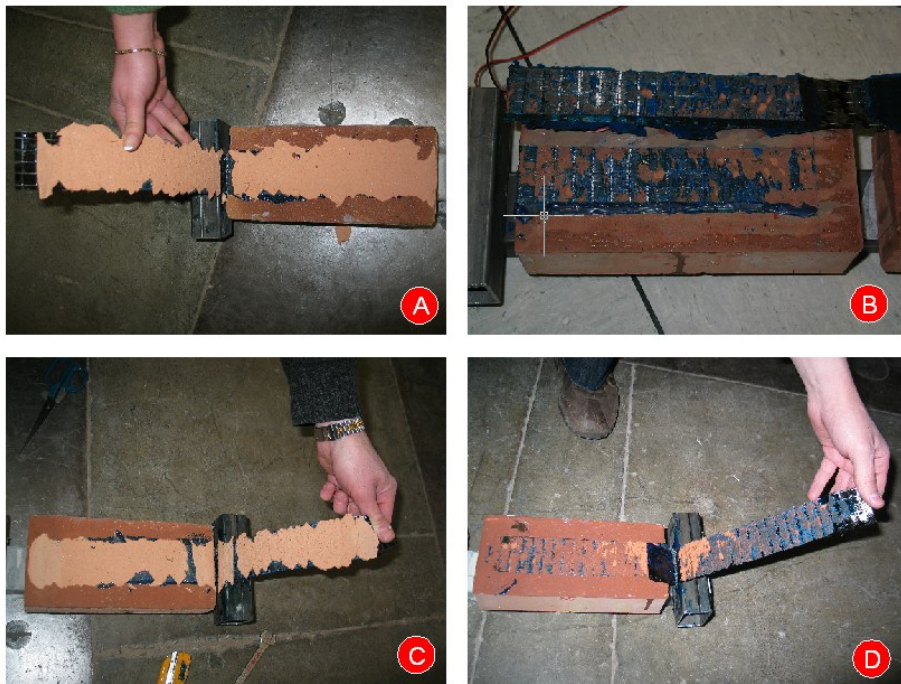


Figure 6.13: Typical failures: facing bricks S2 (A) and S4 (C), extruded bricks S3 (B) and S5 (D)

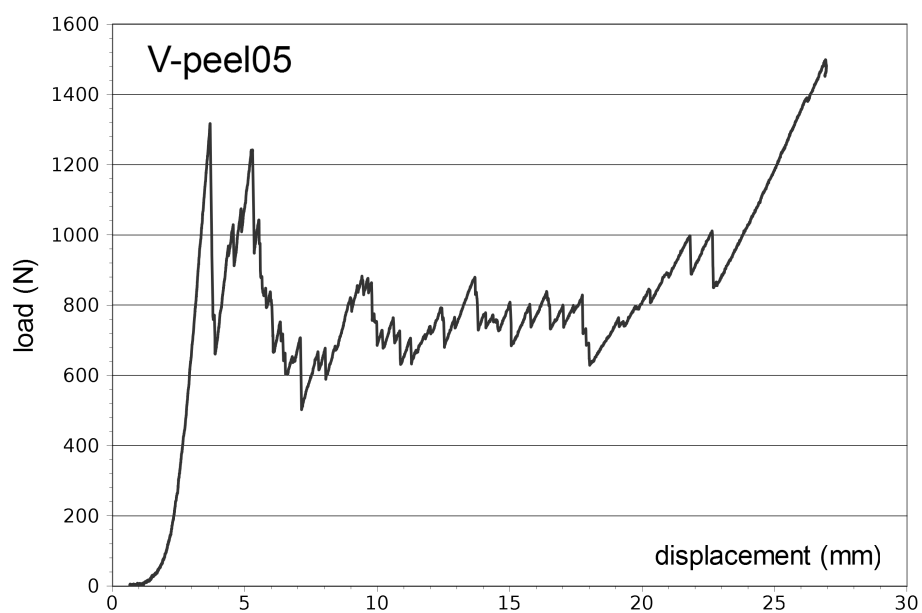


Figure 6.14: Typical monotonic load - vertical deflection of the central point functions

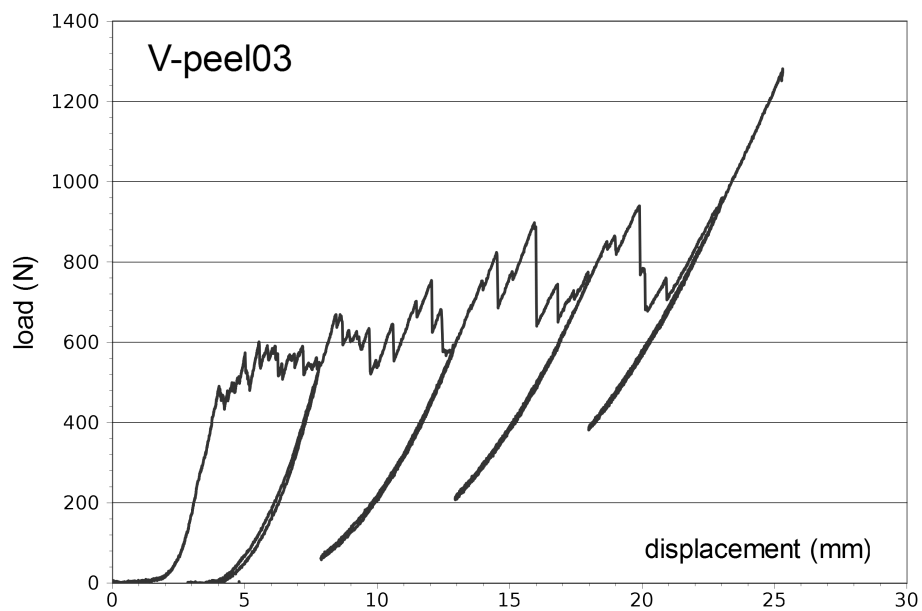


Figure 6.15: Typical cyclic load - vertical deflection of the central point functions

Table 6.3: Experimental results of the V-Shape Peel Tests

SPECIMEN	brick type	P_u N	P_u/b_f N/mm	failure cycle	failure localization
Vp01	S4 - facing	1555	31.1	-	substrate
Vp02	S3 - extruded	1302	26.0	third	interface
Vp03	S3 - extruded	963	19.3	fourth	interf. and substrate
Vp04	S4 - facing	1280	25.6	second	substrate
Vp05	S4 - facing	1317	26.3	-	substrate
Vp06	S4 - facing	1795	35.9	first	interf. and substrate
Vp07	S3 - extruded	935	18.7	fourth	interface
Vp08	S3 - extruded	853	17.1	first	interface
Vp09	S3 - extruded	717	14.3	-	interface
Vp10	S3 - extruded	830	16.6	fourth	interface
Vp11	S4 - facing	1243	24.9	first	interf. and substrate
Vp12	S5 - extruded	937	18.7	-	interface
Vp13	S2 - facing	912	18.2	-	substrate
Vp14	S2 - facing	912	18.2	first	substrate

Table 6.4: Experimental maximum loads sorted by brick type

BRICK TYPE	SPECIMEN	P_u N	P_u/b_f N/mm
S2 - facing	Vp13	912	18.2
S2 - facing	Vp14	912	18.2
average		912	18.2
S3 - extruded	Vp02	1302	26.0
S3 - extruded	Vp03	963	19.3
S3 - extruded	Vp07	935	18.7
S3 - extruded	Vp08	853	17.1
S3 - extruded	Vp09	717	14.3
S3 - extruded	Vp10	830	16.6
average		933	18.7
standard deviation		200	
coeff. of variation		21.5%	
S4 - facing	Vp01	1555	31.1
S4 - facing	Vp04	1280	25.6
S4 - facing	Vp05	1317	26.3
S4 - facing	Vp06	1795	35.9
S4 - facing	Vp11	1243	24.9
average		1438	28.8
standard deviation		234	
coeff. of variation		16.3%	
S5 - extruded	Vp12	937	18.7

6.3 Test analyses

6.3.1 Load-displacement

The maximum load value was recorded, for all the monotonic tests, when the reinforcement started to detach from the support; after that peak, the vertical load values oscillated within inferior limits (Tab. 6.5 and Fig. 6.16, which compare the first peak load to the average subsequent peaks), however showing a generally high scatter around an average, constant or slightly variable. This trend was lightly different to those observed during the cyclic tests, where the trend among the various peak loads were less recurrent.

Concerning the cyclic tests, as expected each cycle showed a lower stiffness on the loading/unloading phases, due to the progressive detachment of the reinforcement that implied an increasing unbonded length (Fig. 6.17 on the facing page).

However, it could be concluded, since the load values do not manifest any evident decrease related to the progressive shortening of the bonded area, that the effective length involved by this mechanism is quite small, if compared to the effective length related to the longitudinal debonding: possibly around the 20-30 %, as previously observed by Panizza (2006), Panizza *et al.* (2008b), where a different test setup was used to reproduce the same shear mechanism.

Table 6.5: Comparison among the peak load values – monotonic tests

SPECIMEN	BRICK TYPE	P_u N	P_{avg} N
Vp01	S4 - facing	1555	1047
Vp05	S4 - facing	1317	792
Vp09	S3 - extruded	718	537
Vp12	S5 - extruded	938	669
Vp13	S2 - facing	912	551

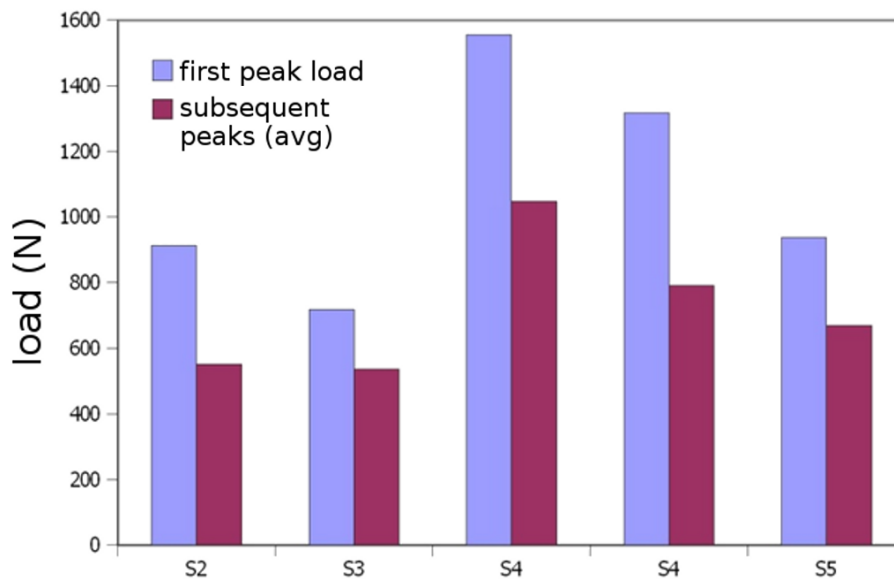


Figure 6.16: Comparison among the peak load values – monotonic tests

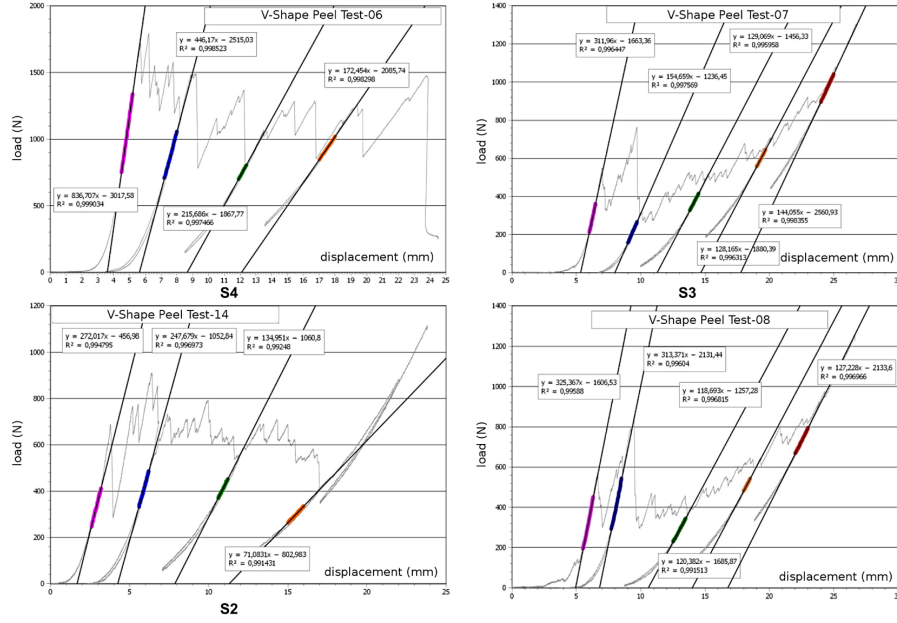


Figure 6.17: Comparison among load functions – cyclic tests

6.3.2 Peel angle

As shown in Figure 6.18 on the next page, the monitored displacements allowed to analytically evaluate (Eq. 6.4, where α is the peel angle, l_{ub} is the distance between the bonded edge, δy the appropriate vertical displacement) the slope of the reinforcement during the detachment; although this estimation was slightly rough, it permitted to point out some characteristics of the phenomenon.

It can be observed (Fig. 6.19 on the next page) that facing bricks showed a more scattering evolution of the peel angle during the progressive detachment, if compared to extruded bricks; this fact could be justified by the different observed failure mode: it involved the brittle clay substrate of facing elements, whereas it evolved within the interface or the adhesive in the case of extruded ones.

The average values of peel angle α , which range from 4.5 to 7 degrees (lower than the 10 degrees observed by Wu *et al.* in the case of concrete substrate) are given in Table 6.6 on page 89, for each specimen and each side, AB and CD, of the reinforcement strip.

$$\alpha = \text{atan} \frac{\delta y_{1,2} - \delta y_{AB \vee CD}}{0.5l_{ub}} \quad (6.4)$$

6.3.3 Fracture energy evaluation

On the basis of the evaluated peel angles, and being known the corresponding vertical load, the mixed-mode fracture energy components mode I, G_I , and mode II, G_{II} , were evaluated following the analysis proposed by Thouless and Jensen (1992), reported also in De Lorenzis and Zavarise (2008), and the total mixed-mode fracture energy G was assumed to be the sum of its components (Karbhari and Engineer 1996, De Lorenzis and Zavarise 2008). Moreover, the simplified analysis proposed by Wu *et al.* (2005a), which allows to estimate the mixed-mode fracture energy, indicated by G_W , as a function of the vertical load P and the reinforcement axial stiffness per unit width, $E_f t_f$, was applied as well, and the results compared to the ones calculated through the first method.

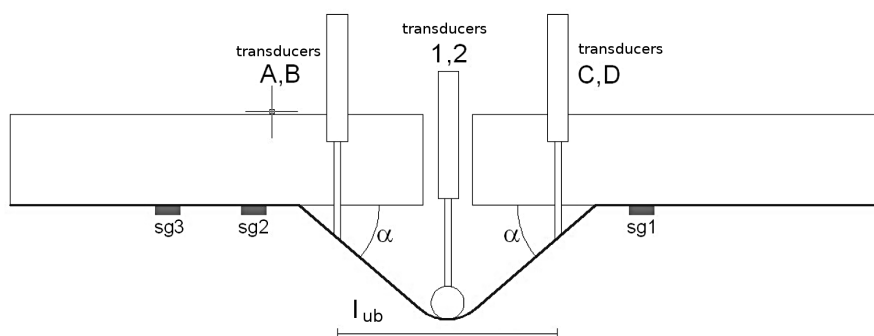


Figure 6.18: Evaluation of the peel angle during the test

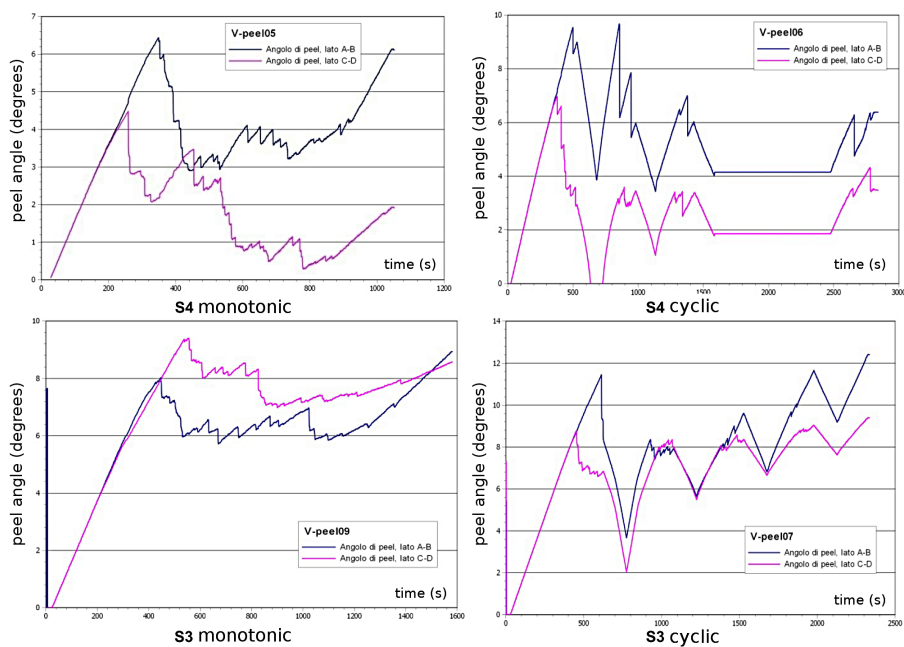


Figure 6.19: Peel angle evolution during monotonic and cyclic tests

Table 6.6: Average peel angles during the various tests phases

SPECIMEN	BRICK TYPE	side AB			side CD		
		$\alpha_{AB,1}$	$\alpha_{AB,2}$ degrees	$\alpha_{AB,3}$	$\alpha_{CD,1}$	$\alpha_{CD,2}$ degrees	$\alpha_{CD,3}$
Vp13 (monot.)	S2 - facing	4.45	—	—	3.00	—	—
Vp14 (cyclic)	S2 - facing	5.62	5.16	3.76	5.03	4.53	5.21
	average	5.04	5.16	3.76	4.02	4.53	5.21
Vp02 (cyclic)	S3 - extruded	8.54	6.25	—	8.51	8.88	8.54
Vp03 (cyclic)	S3 - extruded	6.57	5.95	—	3.02	2.57	2.65
Vp07 (cyclic)	S3 - extruded	10.73	7.8	8.68	7.03	8.10	8.18
Vp08 (cyclic)	S3 - extruded	6.78	6.18	6.75	7.74	7.71	7.94
Vp09 (monot.)	S3 - extruded	6.42	—	—	7.76	—	—
Vp10 (cyclic)	S3 - extruded	2.79	3.67	5.49	3.90	2.4	1.94
	average	6.97	5.97	6.97	6.33	5.93	5.85
	st. dev.	2.63	1.48	1.61	2.29	3.18	3.26
	coeff. of var.	37.72%	24.79%	23.04%	36.14%	53.53%	55.76%
Vp01 (monot.)	S4 - facing	10.08	—	—	11.25	—	—
Vp04 (cyclic)	S4 - facing	7.25	6.32	5.74	7.30	—	—
Vp05 (monot.)	S4 - facing	3.74	—	—	1.69	—	—
Vp06 (cyclic)	S4 - facing	8.88	6.74	6.26	4.44	3.13	3.07
Vp11 (cyclic)	S4 - facing	5.06	4.35	3.41	7.78	6.38	6.39
	average	7	5.8	5.14	6.49	4.76	4.73
	st. dev.	2.62	1.28	1.52	3.61	2.3	2.35
	coeff. of var.	37.43%	21.99%	29.55%	55.66%	48.33%	49.63%
Vp12 (monot.)	S5 - extruded	5.56	—	—	4.51	—	—

Results are given in Tables 6.7, 6.8 on the facing page, 6.9 on page 92 and 6.10 on page 92, for the S2, S3, S4 and S5 sets of bricks, respectively, where α stands for the calculated peel angle and P for the corresponding vertical load, ψ is the phase angle and ΔG the percentage difference of the two fracture energy evaluations. Figure 6.20 on page 93 compares the fracture energy composition for the tested bricks, whereas the phase angle ψ is plotted versus the peel angle α in Figure 6.21 on page 93.

It can be noticed that facing bricks belonging to S4 set shown fracture energy values very high, around twice the other elements, while S3 and S5 sets gave similar results, being different sets of the same industrial product. Moreover, the total mixed-mode fracture energy values seem to be quite reasonable if compared to those adopted by De Lorenzis and Zavarise (2008) for their numerical models concerning FRP sheets externally bonded to masonry substrates, 0.5 N/mm, and measured by Wu *et al.* (2005a) in the case of concrete substrate, from 0.3 to 1.5 N/mm, being 0.2 N/mm the lower bound for practical design proposed by Dai *et al.* (2007).

Table 6.7: Phase angle and fracture energy for S2 bricks – facing

Side	peak	P N	α deg	G_I N/mm	G_{II} N/mm	ψ deg	G N/mm	G_W N/mm	ΔG %
Vp13 – monotonic									
AB	1	552	4.45	0.066	0.215	29.04	0.281	0.274	-2.5%
CD	1	552	3.00	0.146	0.145	45.11	0.291	0.274	-5.8%
Vp14 – cyclic									
AB	1	639	5.62	0.056	0.314	22.81	0.370	0.333	-9.8%
	2	720	5.16	0.084	0.325	26.91	0.409	0.391	-4.4%
	3	660	3.76	0.133	0.217	38.00	0.350	0.348	-0.6%
CD	1	632	5.03	0.068	0.278	26.30	0.346	0.329	-5.1%
	2	715	4.53	0.107	0.284	31.59	0.391	0.387	-0.9%
	3	590	5.21	0.055	0.269	24.34	0.324	0.300	-7.5%
average		633	4.60	0.090	0.260	30.51	0.350	0.330	
st. dev.		66	0.86	0.040	0.060	7.57	0.050	0.050	
coeff. of .var.		10.4%	18.8%	39.5%	23.4%	24.8%	13.1%	13.8%	

6.4 Conclusive remarks

Fourteen V-shape Peel Tests, whose test set-up was adapted by similar tests carried out on concrete substrate (Wu *et al.* 2005a, Dai *et al.* 2007) were performed using CFRP reinforcement applied on solid clay bricks. Although these should be considered as pilot tests, the main characteristics of the investigated phenomenon were substantially identified, and the experimental observations do not differ much from what Wu *et al.* (2005a), Dai *et al.* (2007) reported:

- the peel load, during the detachment, oscillated within a limited range, though the scattering was in some cases very large;
- maximum loads of around 18 N/mm were observed, except for the S4 series that gave 28.8 N/mm;
- the first peak load was generally higher than the others, for monotonic tests;

Table 6.8: Phase angle and fracture energy for S3 bricks – extruded

Side	peak	P N	α deg	G_I N/mm	G_{II} N/mm	ψ deg	G N/mm	G_W N/mm	ΔG %
Vp02 – cyclic									
AB	1	940	8.54	0.052	0.703	15.16	0.755	0.558	-26.1%
	2	684	6.25	0.051	0.374	20.34	0.425	0.365	-14.2%
CD	1	910	8.51	0.049	0.678	15.01	0.727	0.534	-26.5%
	2	690	8.88	0.026	0.536	12.35	0.562	0.369	-34.3%
	3	1038	8.54	0.063	0.776	15.89	0.839	0.637	-24.2%
Vp03 – cyclic									
AB	1	550	6.57	0.030	0.316	17.12	0.346	0.273	-21.2%
	2	489	5.95	0.029	0.254	18.66	0.283	0.233	-17.7%
CD	1	498	3.02	0.117	0.132	43.30	0.249	0.239	-3.9%
	2	668	2.57	0.293	0.150	54.37	0.443	0.354	-20.1%
	3	864	2.65	0.459	0.201	56.54	0.660	0.499	-24.5%
Vp07 – cyclic									
AB	1	614	10.73	0.014	0.577	8.80	0.591	0.316	-46.5%
	2	527	7.80	0.020	0.360	13.11	0.379	0.258	-32.0%
	3	588	8.68	0.020	0.447	11.81	0.466	0.298	-36.0%
CD	1	369	7.03	0.012	0.227	12.85	0.239	0.160	-32.8%
	2	460	8.10	0.014	0.326	11.61	0.340	0.215	-36.7%
	3	589	8.18	0.022	0.422	12.90	0.444	0.299	-32.6%
Vp08 – cyclic									
AB	1	351	6.78	0.011	0.208	13.22	0.220	0.150	-31.7%
	2	387	6.18	0.017	0.209	15.84	0.226	0.171	-24.4%
	3	542	6.75	0.028	0.320	16.37	0.348	0.268	-23.0%
CD	1	341	7.74	0.008	0.231	10.73	0.239	0.144	-39.6%
	2	529	7.71	0.020	0.357	13.36	0.377	0.259	-31.3%
	3	870	7.94	0.051	0.605	16.23	0.656	0.503	-23.3%
Vp09 – monotonic									
AB	1	538	6.42	0.030	0.302	17.51	0.332	0.265	-20.3%
CD	1	538	7.76	0.021	0.365	13.33	0.386	0.265	-31.3%
Vp10 – cyclic									
AB	1	459	2.79	0.117	0.112	45.54	0.229	0.215	-6.2%
	2	460	3.67	0.068	0.148	34.11	0.215	0.215	-0.1%
	3	600	5.49	0.051	0.288	22.91	0.339	0.306	-9.7%
	4	760	3.43	0.212	0.228	43.98	0.440	0.420	-4.6%
CD	1	285	3.90	0.023	0.097	25.98	0.120	0.114	-5.4%
	2	305	2.40	0.070	0.064	46.23	0.134	0.124	-7.0%
	3	502	1.94	0.291	0.085	61.59	0.376	0.242	-35.7%
average		579	6.22	0.070	0.330	23.77	0.400	0.300	
st. dev.		192	2.39	0.100	0.190	15.63	0.180	0.130	
coeff. of .var.		33.2%	38.5%	139.6%	58.5%	65.8%	46.0%	44.5%	

Table 6.9: Phase angle and fracture energy for S4 bricks – facing

Side	peak	P N	α deg	G_I N/mm	G_{II} N/mm	ψ deg	G N/mm	G_W N/mm	ΔG %
Vp01 – monotonic									
AB	1	1047	10.80	0.040	0.991	11.32	1.031	0.644	-37.5%
CD	1	1047	11.25	0.037	1.033	10.65	1.069	0.644	-39.8%
Vp04 – cyclic									
AB	1	1160	7.25	0.110	0.737	21.09	0.846	0.738	-12.8%
	2	879	6.32	0.083	0.486	22.45	0.569	0.510	-10.4%
	3	872	5.74	0.099	0.438	25.44	0.537	0.505	-6.1%
	4	1131	5.82	0.162	0.577	27.94	0.739	0.714	-3.4%
CD	1	965	7.30	0.075	0.617	19.20	0.692	0.578	-16.5%
Vp05 – monotonic									
AB	1	792	3.74	0.194	0.259	40.82	0.453	0.444	-2.0%
CD	1	792	1.69	0.949	0.118	70.59	1.066	0.444	-58.4%
Vp06 – cyclic									
AB	1	1155	8.88	0.072	0.899	15.81	0.971	0.734	-24.4%
	2	1087	6.74	0.111	0.642	22.61	0.753	0.677	-10.1%
	3	1177	6.26	0.152	0.645	25.86	0.797	0.753	-5.6%
CD	1	1354	4.44	0.401	0.527	41.08	0.928	0.907	-2.2%
	2	784	3.13	0.271	0.215	48.34	0.486	0.438	-9.9%
	3	966	3.07	0.427	0.260	52.04	0.688	0.578	-15.9%
Vp11 – cyclic									
AB	1	1143	5.06	0.220	0.507	33.37	0.726	0.724	-0.3%
	2	812	4.35	0.150	0.310	34.83	0.459	0.459	-0.1%
	3	765	3.41	0.217	0.228	44.28	0.446	0.424	-4.9%
CD	1	887	7.78	0.055	0.604	16.86	0.660	0.516	-21.8%
	2	740	6.38	0.058	0.413	20.47	0.471	0.405	-13.9%
	3	884	6.89	0.071	0.533	19.99	0.604	0.514	-14.9%
	4	946	6.77	0.084	0.561	21.12	0.644	0.563	-12.7%
average		972	6.05	0.180	0.530	29.37	0.710	0.590	
st. dev.		168	2.40	0.200	0.250	14.89	0.200	0.140	
coeff. of .var.		17.3%	39.7%	109.9%	46.6%	50.7%	28.8%	23.1%	

Table 6.10: Phase angle and fracture energy for S5 bricks – extruded

Side	peak	P N	α deg	G_I N/mm	G_{II} N/mm	ψ deg	G N/mm	G_W N/mm	ΔG %
Vp12 – monotonic									
AB	1	669	5.56	0.062	0.326	23.59	0.388	0.354	-2.5%
CD	1	669	4.51	0.095	0.264	30.94	0.359	0.354	-5.8%
average		669	5.04	0.080	0.300	27.27	0.370	0.354	
st. dev.			0.74	0.020	0.040	5.20	0.020		
coeff. of .var.			14.8%	29.7%	14.9%	19.1%	5.5%		

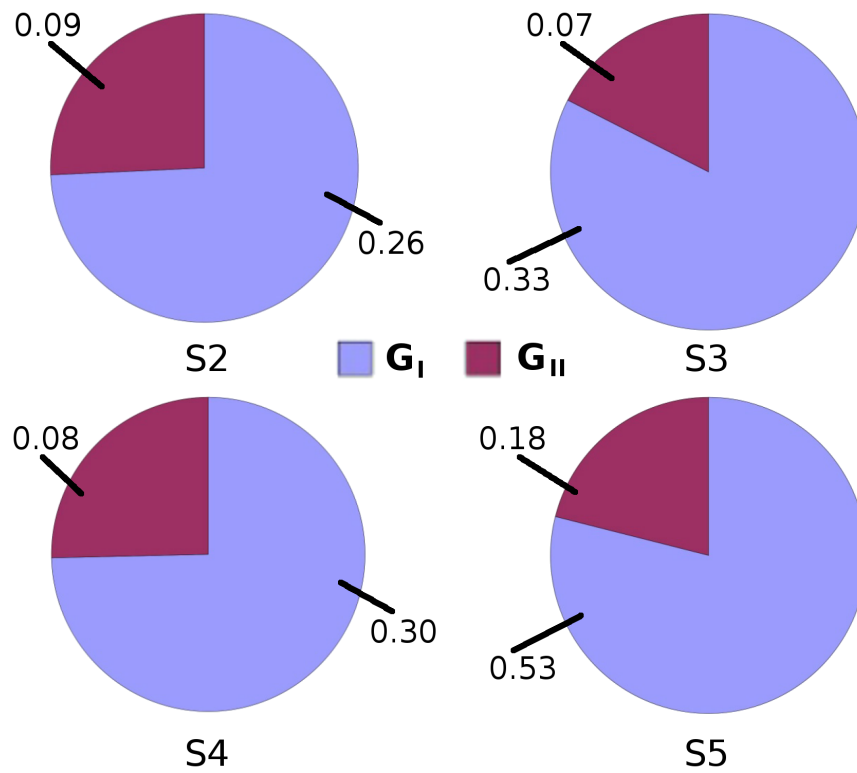


Figure 6.20: Evaluated components (N/mm) of the fracture energy for the tested bricks

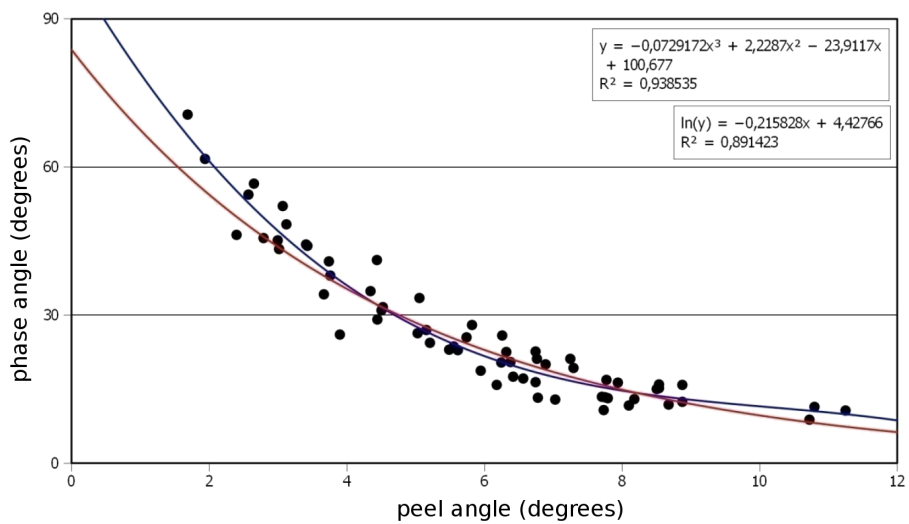


Figure 6.21: Experimentally evaluated phase plotted versus peel angles

- the peel angle, similarly to the peel load, oscillated around quasi-constant values, being high the scattering also in this case; the estimated average values ranged from 4.5 to 7 degrees;
- the calculated mixed-mode fracture energies ranged from 0.3 to 0.7 N/mm, consistently with values proposed in literature for quasi-brittle substrates (Wu *et al.* 2005a, Dai *et al.* 2007, De Lorenzis and Zavarise 2008).

Chapter 7

Analysis of the global structural behaviour

This chapter presents the results of the application of the available interpretative models to several experimental cases studies, masonry arches and barrel vaults strengthened to their extrados or intrados by means of FRP textiles, collected from literature.

First, a brief review of the experimental main results is reported, followed by a comparison of the experimental ultimate loads with the predicted failure loads, evaluated with regard to bending failure (crushing of masonry or fiber rupture), fibres debonding, shear failure at the springer (extrados only) and fiber detachment (intrados only).

7.1 Experimental case studies

The present collection of masonry arches experimentally tested was based on data reported by Valluzzi *et al.* (2001), who tested six arches having three different configurations that compared type of fibres, glass and carbon, and application at the extrados or the intrados; Briccoli Bati and Rovero (2008), who summarized the results of three reinforcement configurations at the extrados and three at the intrados, varying the width of carbon fibres; Basilio (2007), who dealt with four GFRP configuration, two for the extrados and two for the intrados; Castori (2006), whose work was mainly based on steel reinforced polymers and grouts (SRP/SRG), but tested one arch reinforced at the extrados and another at its intrados by carbon fibers; Badalá *et al.* (2008), who reported the results of one tested arch reinforced with carbon at the extrados.

Overall ten arch models (one or more specimens) for extrados reinforcement and six models for intrados reinforcement were gathered. In Tables 7.1 on the next page and 7.2 on the following page the main characteristics of the models, with extrados and intrados reinforcement, are summarized; “circular” or “catenary” refers to the axis shape, t and b are the thickness and the width of the masonry section, respectively, span and rise refer to the intrados surface, α , measured in degrees, is the slope of the starting section, W is the self-weight of the structure and, finally, FRP and b_f are fibres type and width.

In Figure 7.1 on page 98 a scaled comparison of the adopted shapes and dimensions shows differences among the analyzed models, while Figure 7.2 on page 98 reports some images from the experimental campaigns.

The main results are reported in Tables 7.3 on page 97 and 7.4 on page 97, concerning extrados and intrados reinforcement, respectively. For both extrados and intrados reinforcement the average failure loads per unit width of the arch, shown in Figure 7.3 on page 99 and 7.4 on page 99, were plotted versus the main geometric and mechanical parameters,

namely the reinforcement percentage μ , given by the FRP and masonry section area ratio, the FRP normalized area fraction ω (Triantafillou 1998a, Valluzzi *et al.* 2001, see), the rise/span ratio, the thickness/span and the span/thickness ratios (Fig 7.3 on page 99 and 7.4 on page 99). The corresponding data are reported in Table 7.5 on the next page and 7.6 on the facing page. As it can be noticed, no significative trend emerges from these data, except in the case of thickness/span ratio and its inverse, span/thickness ratio, whose trends resemble a linear or hyperbola branch progress, respectively.

Concerning the intrados reinforcement, no significant trend seems to emerge, considering that less data are available.

All the arches reinforced at their extrados failed due to shear sliding near the springer, except the Ricamato's one, which collapse mechanism is not clearly described and was probably due to crushing, and the arches reinforced at the intrados exhibited fibres detachment from the internal curved surface.

Table 7.1: Main properties of the experimental case studies – extrados reinforcement

TEST	shape	t mm	b mm	span mm	rise mm	α deg	W N	FRP	b_f mm
Badalà	circular	70	550	1497	398	34°	1271	carbon	200
Basilio 2x50mm	circular	50	450	1413	562	13°	816	glass	100
Basilio 2x80mm	circular	50	450	1413	562	13°	816	glass	160
Briccoli Bati 12.5mm	circular	100	100	1502	434	30°	346	carbon	12.5
Briccoli Bati 25mm	circular	100	100	1502	434	30°	346	carbon	25
Briccoli Bati 50mm	circular	100	100	1502	434	30°	346	carbon	50
Castori	catenary	100	200	1980	480	–	1172	carbon	150
Padova 2x200mm	catenary	55	1000	2000	730	–	2334	glass	400
Padova 2x70mm	catenary	55	1000	2000	730	–	2334	carbon	140
Ricamato	circular	120	250	903	393	8°	798	carbon	100

Table 7.2: Main properties of the experimental case studies – intrados reinforcement

TEST	shape	t mm	b mm	span mm	rise mm	α deg	W N	FRP	b_f mm
Basilio 2x50mm	circular	50	450	1413	562	13°	816	glass	100
Briccoli Bati 12.5mm	circular	100	100	1502	434	30°	346	carbon	12.5
Briccoli Bati 25mm	circular	100	100	1502	434	30°	346	carbon	25
Briccoli Bati 50mm	circular	100	100	1502	434	30°	346	carbon	50
Castori	catenary	100	200	1980	480	–	1172	carbon	150
Padova 2x70mm	catenary	55	1000	2000	730	–	2334	carbon	140

Table of arch plotted results - intrados

7.2 Analysis method

As well known, it is assumed that the combination of masonry and FRP works similarly to a reinforced masonry section, considering the FRP reinforcement corresponding to the steel one; moreover, the experimental testing on reinforced masonry arches demonstrated that, as reasonable, the presence of the fibres avoid the formation of an hinge placed on the opposite surface of the structure but not on the same face; after a certain load value is reached, the mortar on the joints star to cracks, and the hinges not inhibited by the

Table 7.3: Results of the experimental case studies – extrados reinforcement

TEST	load configuration	failure mechanism	Q_u kN	Q_u/b N/mm
Badalà	asymmetric	sliding	12.00	21.8
Basilio 2x50mm	asymmetric	sliding	3.17	7.0
Basilio 2x80mm	asymmetric	sliding	3.44	7.6
Briccoli Bati 12.5mm	symmetric	sliding	5.92	59.2
Briccoli Bati 25mm	symmetric	sliding	7.13	71.3
Briccoli Bati 50mm	symmetric	sliding	8.03	80.3
Castori	asymmetric	sliding	11.50	57.5
Padova 2x200mm	asymmetric	sliding	19.98	20.0
Padova 2x70mm	asymmetric	sliding	15.35	15.4
Ricamato	asymmetric	crushing	50.00	200.0

Table 7.4: Results of the experimental case studies – intrados reinforcement

TEST	load configuration	failure mechanism	Q_u kN	Q_u/b N/mm
Basilio 2x50mm	asymmetric	detachment	4.45	9.9
Briccoli Bati 12.5mm	symmetric	detachment	3.56	35.6
Briccoli Bati 25mm	symmetric	detachment	5.18	51.8
Briccoli Bati 50mm	symmetric	detachment	6.58	65.8
Castori	asymmetric	detachment	12.30	61.5
Padova 2x70mm	asymmetric	detachment	14.55	14.6

Table 7.5: Results of the experimental case studies – extrados reinforcement

TEST	Q_{exp}/b N/mm	μ	ω	rise/span ratio	thickn./span ratio	span/thickn. ratio
Badalà	21.8	0.00086	0.3235	0.266	0.0468	21.39
Basilio 2x50mm	7.0	0.00066	0.0331	0.398	0.0354	28.26
Basilio 2x80mm	7.6	0.00106	0.0530	0.398	0.0354	28.26
Briccoli Bati 12.5mm	59.2	0.00021	0.0359	0.289	0.0666	15.02
Briccoli Bati 25mm	71.3	0.00041	0.0717	0.289	0.0666	15.02
Briccoli Bati 50mm	80.3	0.00083	0.1434	0.289	0.0666	15.02
Castori	57.5	0.00125	0.1447	0.242	0.0505	19.80
Padova 2x200mm	20.0	0.00117	0.0888	0.365	0.0275	36.36
Padova 2x70mm	15.4	0.00042	0.1127	0.365	0.0275	36.36
Ricamato	200.0	0.00057	0.0810	0.435	0.1329	7.53

Table 7.6: Results of the experimental case studies – intrados reinforcement

TEST	Q_{exp}/b N/mm	μ	ω	rise/span ratio	thickn./span ratio	span/thickn. ratio
Basilio 2x50mm	9.89	0.00066	0.0331	0.398	0.0354	28.26
Briccoli Bati 12.5mm	35.60	0.00021	0.0359	0.289	0.0666	15.02
Briccoli Bati 25mm	51.80	0.00041	0.0717	0.289	0.0666	15.02
Briccoli Bati 50mm	65.80	0.00083	0.1434	0.289	0.0666	15.02
Castori	61.50	0.00125	0.1447	0.242	0.0505	19.80
Padova 2x70mm	14.55	0.00042	0.1127	0.365	0.0275	36.36

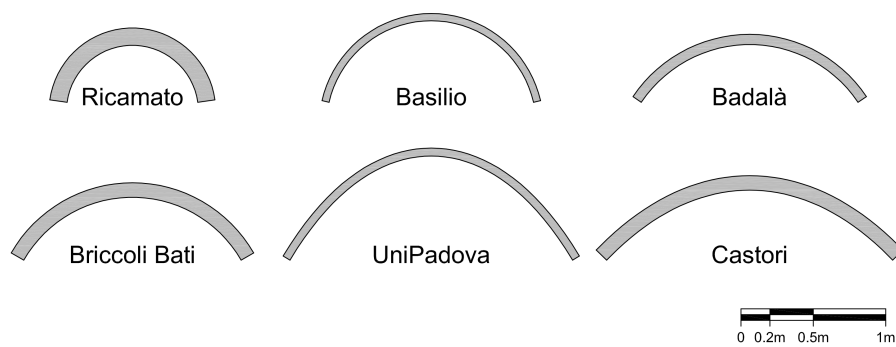
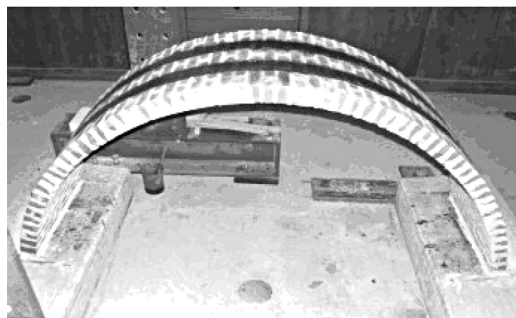


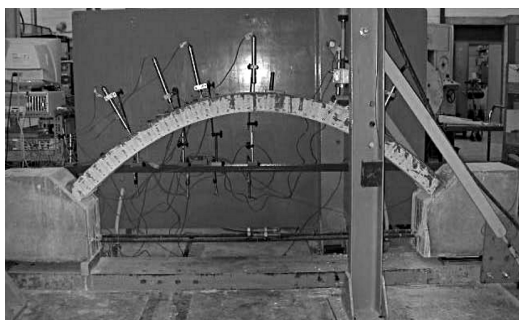
Figure 7.1: Comparison among the various shapes of the collected tests



(a) Padova (Valdemarca 1998)



(b) Basilio (2007)



(c) Castori (2006)



(d) Badalá *et al.* (2008)

Figure 7.2: Images of some tested arches

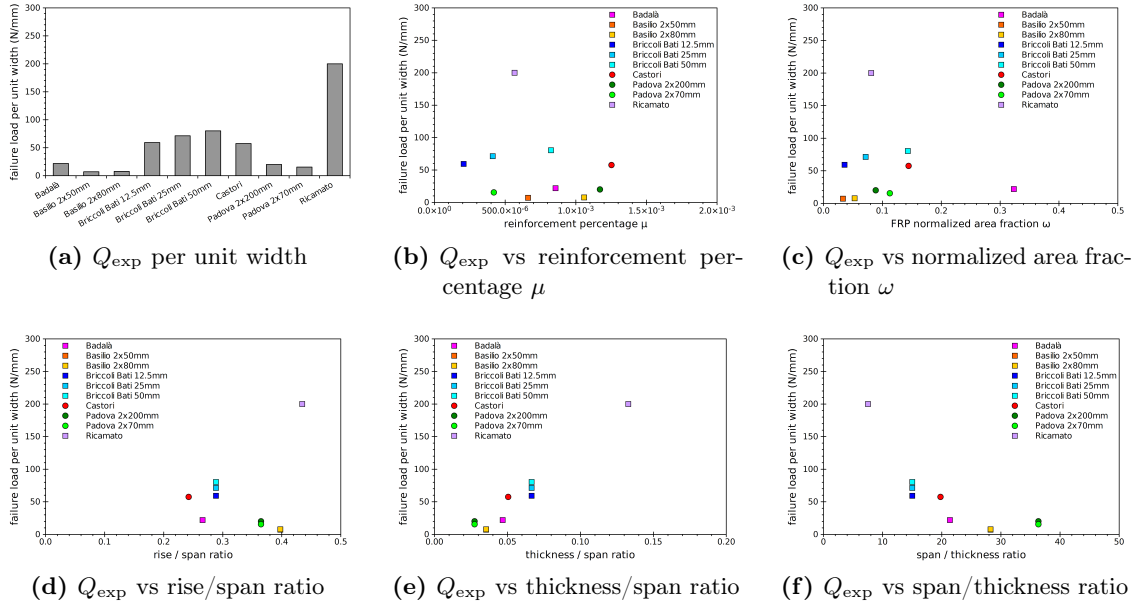


Figure 7.3: Experimental failure loads Q_{exp} plotted versus the main mechanical and geometrical parameters – extrados reinforcement

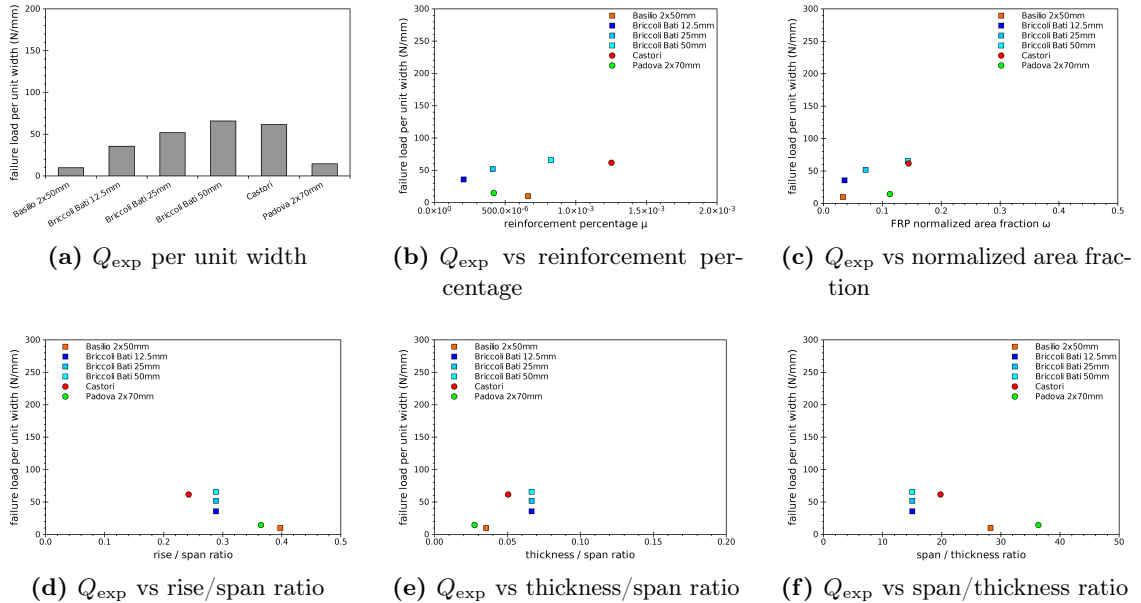


Figure 7.4: Experimental failure loads Q_{exp} plotted versus the main mechanical and geometrical parameters – intrados reinforcement

reinforcement begin to open (Triantafyllou 1998a, Valluzzi *et al.* 2001, Foraboschi 2004, Basilio 2007, Castori 2006, Briccoli Bati and Rovero 2008, Focacci 2008).

Therefore, it is supposed that the arched element, after the formation of the hinges on the same position pertinent to the unreinforced arch, where not prevented, could be described by an isostatic scheme, namely a three-hinge arch (Valluzzi *et al.* 2001, Briccoli Bati and Rovero 2008, Focacci 2008), in order to evaluate the reactions at the springer and the internal forces, compressive and shear one, and the bending moment (Fig. 7.5 and 7.6 on the facing page, respectively related to loading at a quarter of span and at the keystone). To place the second hinge for intrados reinforcement, the approximated method proposed by Briccoli Bati and Rovero (2008) was adopted.

To carry out the analyses, each structure was divided into discrete elements, adopting as incrementing parameter the horizontal coordinate, for catenary arches, and the cross section angle, for semicircular arches.

Therefore, once all the relevant quantities were calculated as function of the external load Q , it was possible to modify iteratively the load value until the appropriate acting parameter equalized the resisting one, with reference to each predictive model, obtaining in this manner the proper failure load Q_u .

Some examples of the analysis method are reported in Figure 7.7 on the next page, which shows thrust line, bending moment, normal force and shear force, for three different load levels (being Q_{exp} the experimental mean failure load), related to the model tested by Basilio (2007) and strengthened at its extrados with two stripes of glass fibres 50 mm wide. The same parameters are given in Figure 7.8 on the facing page, normalized with respect to the vertical load Q (normal and shear forces) and to the product Qt (bending moment), being t the thickness of the arch. It can be observed, as expected, that the normalized values show a slight variation with respect to the load level, since the adopted static scheme was isostatic.

Figure 7.9 on page 102 and 7.10 on page 102 show the above mentioned plots in the case of the model tested by Basilio (2007) and strengthened at its intrados with two stripes of glass fibres 50 mm wide. It can be observed that the thrust line shows a discontinuity under the concentrated load: as a matter of fact, under the load point the normal force decreases to low values and the jump on the shear profile provokes an inversion on the bending moment trend; therefore, to justify similar values of bending moment, which is given by the product Nu (being N the normal force and u its eccentricity), the eccentricity has to rise to higher values. If the load is applied on the keystone, as in the models tested by Briccoli Bati and Rovero (2008), the thrust line shows a cusp discontinuity, differently from the former one.

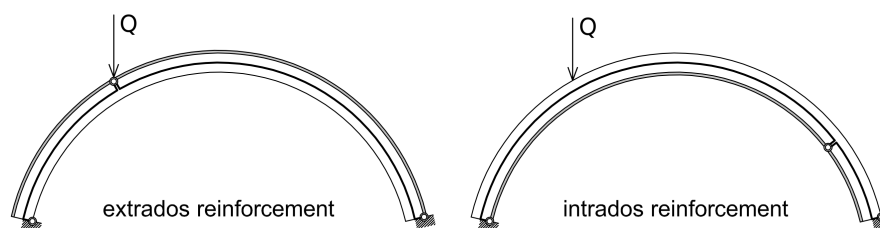


Figure 7.5: Static scheme to evaluate the internal forces - asymmetric concentrated load

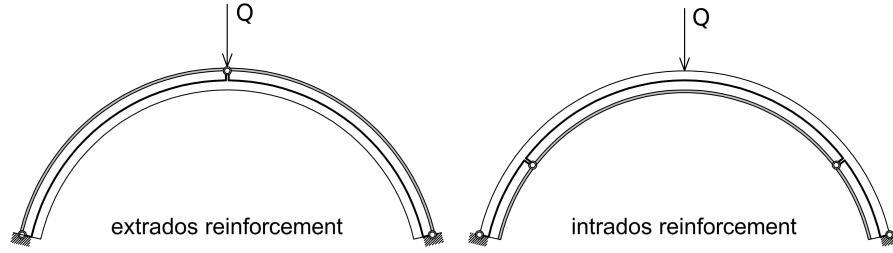


Figure 7.6: Static scheme to evaluate the internal forces - symmetric concentrated load

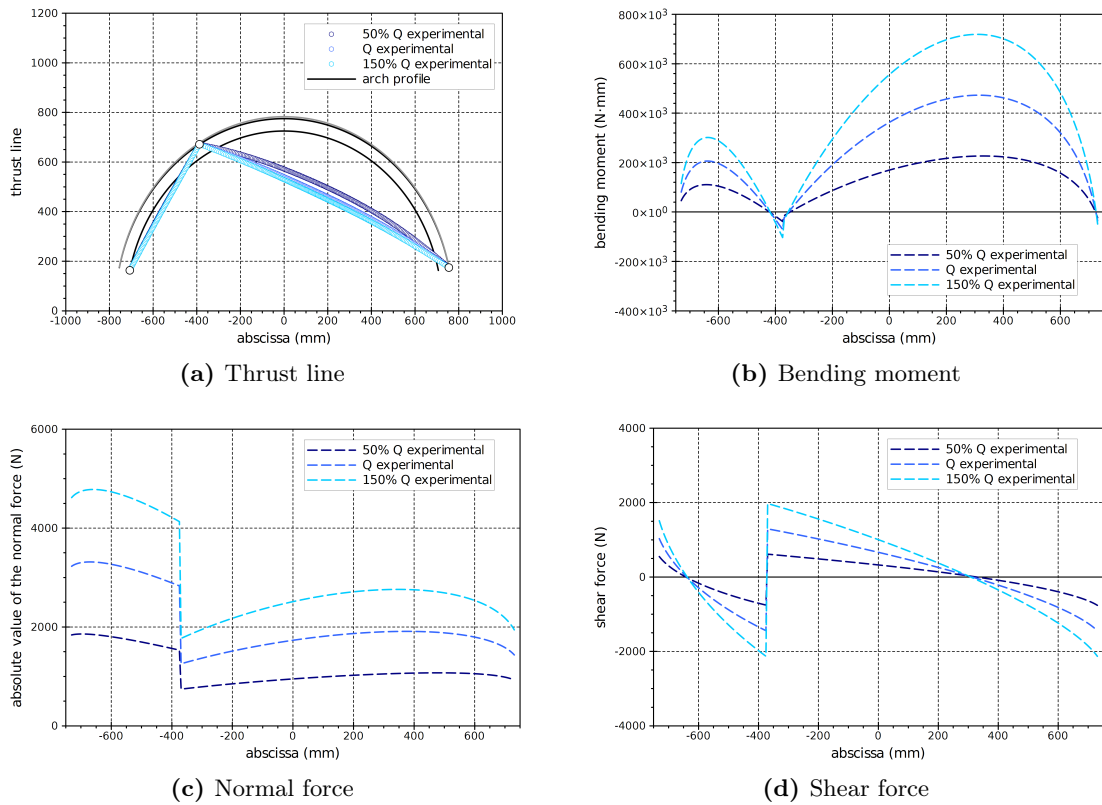


Figure 7.7: Thrust line and internal forces for extrados reinforcement at various load levels (Basilio 2007)

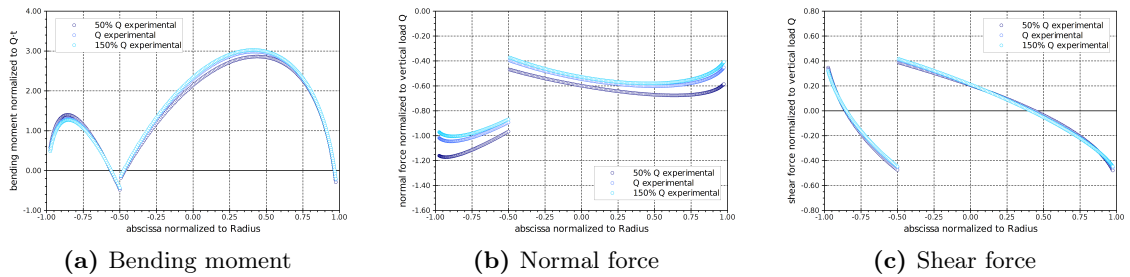


Figure 7.8: Normalized internal forces for extrados reinforcement at various load levels (Basilio 2007)

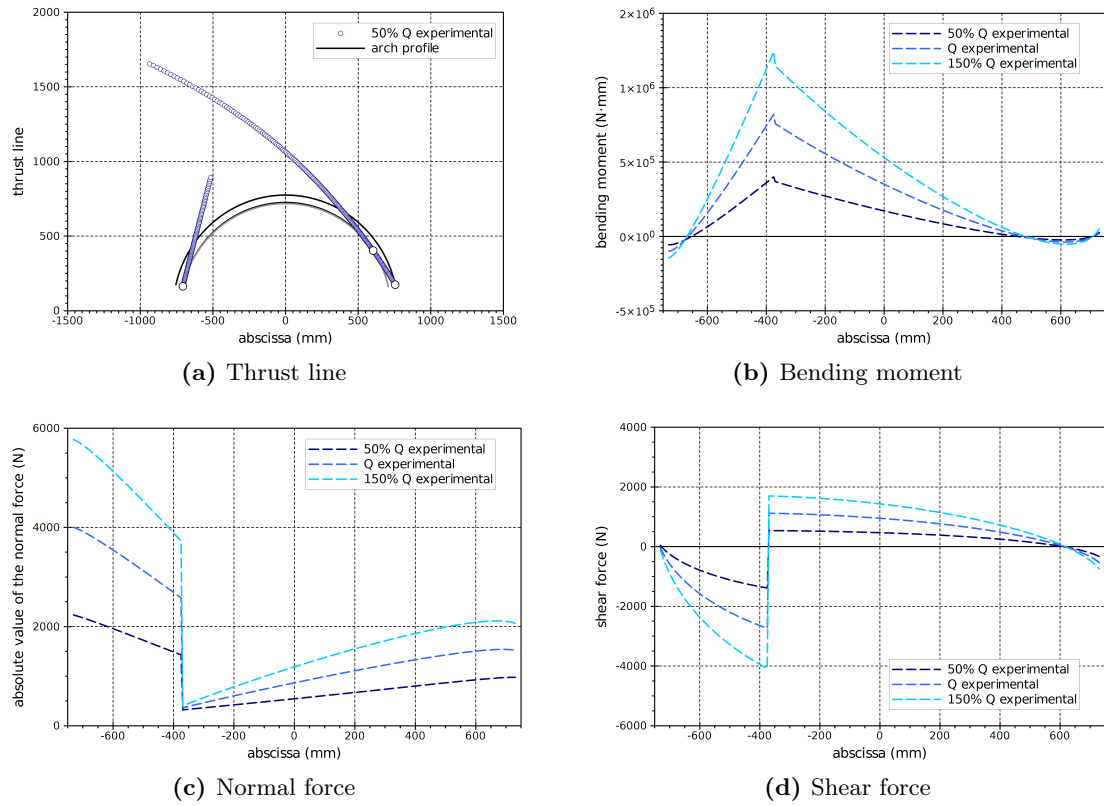


Figure 7.9: Thrust line and internal forces for intrados reinforcement at various load levels (Basilio 2007)

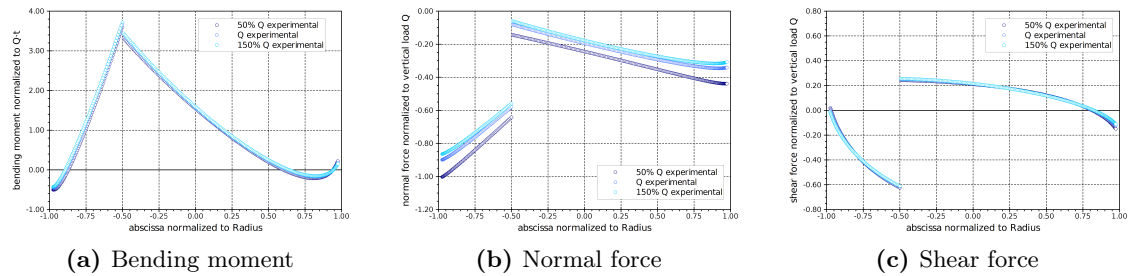


Figure 7.10: Normalized internal forces for intrados reinforcement at various load levels (Basilio 2007)

7.3 Application of predictive models

The application of the available predictive models was based on the previously shown static schemes, used to evaluate the reaction at the springers and the internal forces, normal, N , and shear, V , ones, the bending moment M and, consequently, the eccentricity u that is given by the M/N ratio, related to an arbitrary value of external concentrated load Q . Where needed, the simplified approach proposed by Foraboschi (2004), which assumes a fixed stress block of one third of the masonry section for the compressive force, was adopted to estimate C and T , respectively the compressive force on the masonry and the tensile force on the reinforcement. Mechanical and geometric data reported by the appropriate publications were used, assuming calculated values in place of missing experimentally measured ones, when needed.

7.3.1 Extrados reinforcements

Concerning the extrados reinforcement, the following mechanisms were considered:

- flexural failure: the corresponding failure load was evaluated applying the sectional analysis developed by Triantafillou (1998a) and the other proposed by Foraboschi (2004); for the latter model, a further check was performed, related to the fibres rupture;
- intermediate debonding: the simplified approach, which consists in assuming a limit stress proportional to the one related to the plate-end debonding, exposed by CNR DT-200 (2004) was adopted, using the c_1 coefficient corresponding to the concrete substrate (0.03) instead of the halved one proposed for masonry, and the c_1 coefficient regressed by the above mentioned data analyses (0.124); the simplified hypotheses formalized by Foraboschi (2004) were assumed to estimate the compressive and tensile forces on the cross section;
- shear failure: the usual model based on a Coulomb-like law was adopted, without assigning any strength contribution to the reinforcement, conjectured by Valluzzi *et al.* (2001), Briccoli Bati and Rovero (2008), since the analyzed sections were placed at the final part of the arch and that resistance, if present, could not be activated.

Results in terms of absolute values and predicted versus experimental failure load are given in Tables 7.7 on the next page and 7.8 on the following page, where TR stands for Triantafillou's model, FOR and FOR2 indicate Foraboschi's models, the latter calculated taking into account the fibres rupture (the maximum tensile force on the reinforcement was imposed), DB and DB2 stand for the intermediate debonding models evaluated assuming the c_1 values proposed by CNR DT-200 (2004) and calibrated in the present work, respectively, SF for shear failure, and shown in Figure 7.11 on page 105.

As it can be observed, Foraboschi's model for masonry crushing does not take into account the reinforcement amount, differently from Triantafillou's one. The intermediate debonding mechanism, although never observed, gives predictions generally lower than the experimental failure loads. Finally, the model for the shear resistance seems not to lead to reliable results (sometimes totally irrelevant, as for the models of Basilio, Briccoli Bati and Ricamato where the predictions are lower than a reasonable load value that provokes the hinges formation); this fact could be justified by taking into account the static scheme assumed to evaluate the internal forces: where the eccentricity of the internal normal force stands outside the masonry section, the reinforcement is assumed to work and the compressive force, given by

the sum of the normal force and the tensile force on the reinforcement, assures high shear resistance, while the shear strength is proportional to the normal force only where it stands on the masonry section, situation that is supposed to be verified near the extrados hinge at the springer where the shear failure is generally observed and causes a localized decrease of resistance (Fig. 7.12 on the facing page), rather independent from the applied load and strongly dependent on the assumed friction coefficient, which literature values could vary from 0.5 (Heyman 1982), 0.64 (Foraboschi 2004), 0.73 (Valluzzi *et al.* 2001).

Table 7.7: Predicted failure loads – extrados reinforcement

TEST	EXP N	TR N	FOR N	FOR2 N	DB N	DB2 N	SF N
Badalà	12000	21900	18650	73300	15600	31150	3000
Basilio 2x50mm	3170	8900	16900	10100	3700	7250	750
Basilio 2x80mm	3440	10700	16900	16000	5750	11500	750
Briccoli Bati 12.5mm	5920	14400	18400	13100	5550	11050	350
Briccoli Bati 25mm	7130	17600	18400	25900	10850	21800	350
Briccoli Bati 50mm	8030	20900	18400	51500	21500	43500	350
Castori	11500	16600	17200	63200	17300	34400	3450
Padova 2x200mm	19980	15900	20750	25150	9300	18250	21000
Padova 2x70mm	15350	17200	20750	17300	6400	21000	21000
Ricamato	50000	82300	84500	51500	54700	11500	750

Table 7.8: Predicted versus experimental failure loads – extrados reinforcement

TEST	TR	FOR	FOR2	DB	DB2	SF
Badalà	1.83	1.55	6.11	1.30	2.60	0.25
Basilio 2x50mm	2.81	5.33	3.19	1.17	2.29	0.24
Basilio 2x80mm	3.11	4.91	4.65	1.67	3.34	0.22
Briccoli Bati 12.5mm	2.43	3.11	2.21	0.94	1.87	0.06
Briccoli Bati 25mm	2.47	2.58	3.63	1.52	3.06	0.05
Briccoli Bati 50mm	2.60	2.29	6.41	2.68	5.42	0.04
Castori	1.44	1.50	5.50	1.50	2.99	0.30
Padova 2x200mm	0.80	1.04	1.26	0.47	0.91	1.05
Padova 2x70mm	1.12	1.35	1.13	0.42	1.37	1.37
Ricamato	1.65	1.69	1.03	1.09	0.23	0.01

7.3.2 Intrados reinforcement

Concerning the intrados reinforcement, the following mechanisms were considered:

- flexural failure: the corresponding failure load was evaluated applying the sectional analysis developed by Triantafyllou (1998a), since the Foraboschi's model has been discussed in the previous section;
- intermediate debonding: the simplified approach, which consists in assuming a limit stress proportional to the one related to the plate-end debonding, exposed by CNR DT-200 (2004) was adopted, using the c_1 coefficient corresponding to the concrete substrate (0.03) instead of the halved one proposed for masonry, and the c_1 coefficient regressed by the above mentioned data analyses (0.124); the simplified hypotheses formalized by

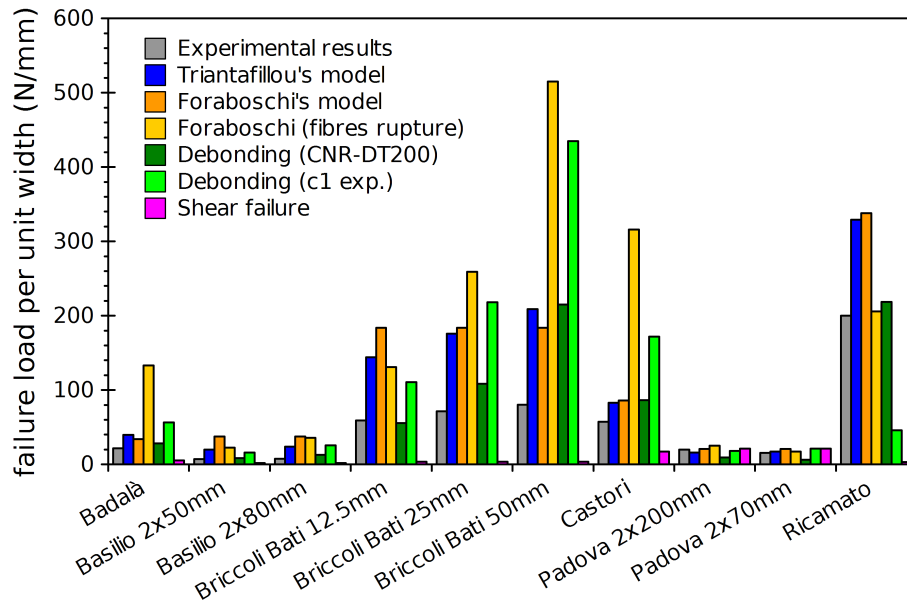
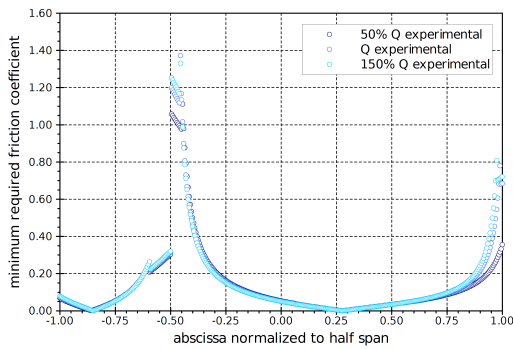
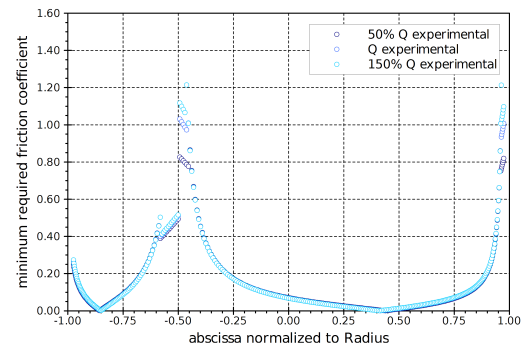


Figure 7.11: Comparison among experimental and predicted failure loads – extrados reinforcement



(a) Padova 2x70mm (Valluzzi *et al.* 2001)



(b) Basilio (2007) - 2x50mm

Figure 7.12: Minimum friction coefficient required to assure the shear resistance

Foraboschi (2004) were assumed to estimate the compressive and tensile forces on the cross section;

- fibres detachment from the surface: the model usually reported in literature (Valluzzi *et al.* 2001, Briccoli Bati and Rovero 2008), which relates the normal tension on the masonry-FRP interface to the pull-off strength experimentally measured, was applied.

Results in terms of absolute values and predicted versus experimental failure load are given in Tables 7.9 and 7.10 on the facing page, where TR stands for Triantafillou's model, DB and DB2 stand for the intermediate debonding models evaluated assuming the c_1 values proposed by CNR DT-200 (2004) and calibrated in the present work, respectively, FD for fibres detachment, and shown in Figures 7.13 on the next page and 7.13 on the facing page.

Regarding the models related to flexural failure and intermediate debonding, the considerations previously presented can apply also to the intrados reinforcement, being the same mechanisms; it can be observed as shear failure does not affect arches reinforced at their intrados, since the shear forces are generally lower (Focacci 2008, see).

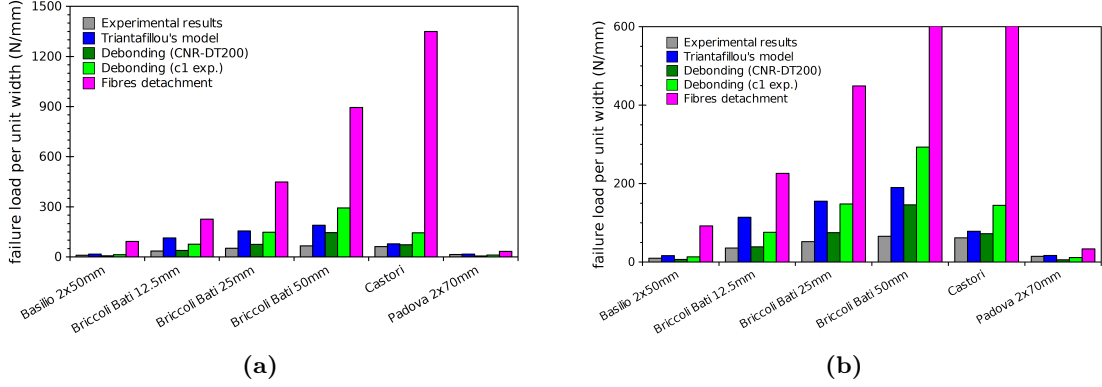
Concerning the fiber detachment model, the pull-off strength was evaluated by applying the third experimental relation (4.10), related to all data, which gives a strength of 1.1 N/mm² for a brick compressive strength of 9 N/mm², that is the case of the models of UniPadova and Basilio; pull-off tests on masonry prisms (with mortar joint within the cut area), carried out at the University of Minho on the same materials used for the arches (Basilio 2007), measured an average strength of 1.2 N/mm²; tests made at the University of Perugia (Castori 2006) gave a strength of 1.6 N/mm² for concrete brick compressive strength of 43.3, whereas the formula provides a pull-off strength of 3.1 N/mm². The minimum intrados curvature radii of the collected models, constant in the case of semicircular shape and variable for catenary arches (in these cases they were evaluated following Eq. 7.1 on the next page, where $y(x)$ is the algebraic expression of the arch profile on a Cartesian plane, κ and R stand for curvature and curvature radius, respectively), vary from 0.73 m (Basilio) to 1.08 m (Castori). As reported by Valluzzi *et al.* (2001), Briccoli Bati and Rovero (2008), the normal stress σ on the interface is proportional to the tensile force T on the reinforcement and inversely proportional to the product of FRP width b_f and curvature radius R ; therefore, imposing the limit tensile strength of the masonry, it should be possible to calculate the maximum tensile force to be withstood by the reinforcement (Eq. 7.2 on the facing page. Imposing the minimum value of curvature radius, 0.73 m, and a tensile strength of 0.5 N/mm², a value of 730 N/mm arises, which is really high compared to usual strength per unit width of externally bonded reinforcement, as the MBrace[®] datasheets provide a range from 270 N/mm for very high modulus CFRP to 565 N/mm for high strength CFRP, being glass and aramid included between these two limits.

Table 7.9: Predicted failure loads – intrados reinforcement

TEST	EXP N	TR N	DB N	DB2 N	FD N
Basilio 2x50mm	4450	7300	2950	5900	41500
Briccoli Bati 12.5mm	3560	11400	3900	7600	22600
Briccoli Bati 25mm	5180	15500	7500	14800	44900
Briccoli Bati 50mm	6580	19000	14600	29300	89500
Castori	12300	15700	14500	28900	270000
Padova 2x70mm	14550	16700	5900	11500	33300

Table 7.10: Predicted versus experimental failure loads – intrados reinforcement

TEST	TR	DB	DB2	FD
Basilio 2x50mm	1.64	0.66	1.33	9.33
Briccoli Bati 12.5mm	3.20	1.10	2.13	6.35
Briccoli Bati 25mm	2.99	1.45	2.86	8.67
Briccoli Bati 50mm	2.89	2.22	4.45	13.60
Castori	1.28	1.18	2.35	21.95
Padova 2x70mm	1.15	0.41	0.79	2.29

**Figure 7.13:** Comparison among experimental and predicted failure loads – intrados reinforcement

$$\kappa = \frac{y''(x)}{\sqrt{(1 + (y'(x))^2)^3}}; \quad R = \frac{1}{\kappa} \quad (7.1)$$

$$\sigma = \frac{T}{b_f R} \quad \Rightarrow \quad \frac{T}{b_f} = \sigma R \quad (7.2)$$

7.4 Conclusive remarks

Concerning the extrados reinforcement, the models related to the flexural crisis (masonry crushing or fibres rupture) lead to failure loads generally higher (from 150% to 300%) than the experimental ones, since the bending strength could not be reached before the shear crisis occurs at lower load levels. Only in the case of the models tested at the University of Padova these predictions were very close (80% and 112%) to the experimental failure loads, despite the different mechanisms, and it could be related to the peculiar thickness/span ratio of these arches, that could had emphasized the flexural behaviour.

The models related to the intermediate FRP debonding, never observed during the reported campaigns, give to predictions that vary from 40% to 270% ($c_1 = 0.03$) or from 90% to 540% ($c_1 = 0.124$) of the experimental failure loads, strongly dependent on the chosen values of the experimental coefficient c_1 . It has also to be noticed that the bond behaviour of FRP applied on masonry prisms with mortar joints, at present, is far from being investigated, as most tests have been carried out on single resisting elements: therefore, it is difficult to suppose if the presence of the joint has a negative or positive influence, since the irregular surface could generate a sort of dissipative mechanism similar to friction that could justified some experimental results reported by Basilio (2007), Casareto *et al.* (2003).

Moreover, the shear failure criterion does not seem to be sufficiently adequate to provide a reliable estimation of the failure load, since it is strongly dependent on the assumed friction coefficient, on the geometry of the structure and the adopted static schemes, including the position of the hinges. Where the cross section is bended and the reinforcement is activated, the compressive force assures high strength against shear, as observed by Foraboschi (2004), but near the extrados hinge at the springer the eccentricity is lower than half the thickness of the arch, therefore the reinforcement is not active (and it could not be for the collected tests, since in that area the bonded length is lower than the effective one and there were no anchoring devices): hence, the minimum required friction coefficient, to assure the resistance of the joint, quickly increases along a very small area.

Concerning the intrados reinforcement, the models related to flexure and intermediate debonding give to considerations similar to the extrados reinforcement. The model describing the fibres detachment due to normal stresses generated by the curved morphology of the intrados, while correctly interprets the phenomenon, seems to overestimate the failure load (more than 230% the experimental one); this could be justified, for the collected cases, since assuming the minimum curvature radius pertinent to the collected specimens, 0.76 m, the model gives a very high value (730 N/mm) of tensile force that activates the ultimate strength for 1 N/mm² of tensile strength of the substrate, approximately varying the commonly used FRP textiles tensile strength per ply from 270 N/mm to 565 N/mm (MBrace® datasheets).

Finally, it has to be considered that the above presented inferences are related to the aim of predicting the experimental behaviour of structures that are significantly different to real structures, for absence of filling, the selected geometric proportion, the adopted materials.

Chapter 8

Conclusions

The results of the investigation on the behaviour of the FRP-masonry interface in the case of normal actions have led to the following conclusions:

- if the reinforcement is correctly applied, the type of fibers should not have any influence on the strength measured by the pull-off test; only the failure mode could be influenced, as glass fibers caused some unwanted failures that should however be imputed to contingent factors related to that available portion of reinforcement, such as an incorrect conservation or cleaning. Moreover, the absence of primer could increase the amount of unwanted failures.
- The pull-off tensile strength could be correlated to the flexural, splitting and compressive strength of the bricks by means of power-based regressions; except the relation with the compressive strength, of which a single function seems to correctly depict the tendency, in the other cases extruded and facing bricks showed a different behaviour, although the trends were similar.
- A different behaviour of extruded and facing elements was observed also in the correlation between flexural and compressive strength, whereas a single power-based regression could be sufficient to describe the trend that relates flexural and splitting tensile strength.
- Finally, the correlations among average values of the bricks properties, where sufficiently evident, substantially confirmed the previously calculated trends.

Concerning the investigation on bond behaviour, it has been remarked that:

- the experimental strength has been compared with twenty-one predictive models developed for concrete substrate. All predictions, except two in the case of CFRP, underestimate the results of the tests; the models, except two in the case of GFRP, seem to work better for the carbon reinforcement. However, the strength predictions vary into a wide range (between 44% and 154% of experimental mean failure load for CFRP, 43% and 85% for GFRP).
- From the measured failure loads, different fracture energy values have been derived, around 35% higher in the case of glass reinforcement than carbon one. To analyze stress and slip from strain-gauges measurement, when instruments are not uniformly spaced, discrete equations have been used, consistent with central finite difference methods. Moreover, a mathematical function easy to integrate and derive is proposed as bond-slip law. This function has been fitted in the case of both carbon and glass

reinforcement; beside these fittings, two bilinear functions have been also calibrated. The optimized functions seem to show an interface local behaviour of CFRP slightly stiffer than GFRP. The reliability of the experimental set-up needs to be verified; despite of its simplicity, the actual distribution of the load should be more clarified, as it could cause undesired bending moments inside the test specimen.

- Finite element analyses of the test specimens were carried out implementing the calibrated bond stress – slip laws. Several comparison of numerical results obtained from different models were performed: (1) FRP modelled with the concentrated fibres approach versus FRP modelled with the distributed fibres, (2) exponential-based versus bilinear bond stress – slip law, (3) influence of the bond length. The finite element results were also compared with the results obtained from the analytical solution proposed by Yuan *et al.* (2001, 2004). As it has been shown, the analytical solution provides reasonable prediction of the maximum load and loaded end displacement. The results pointed out that the concentrated fibres approach is equivalent to the distributed one, although the last one might entail a slightly lower computational burden. Moreover, the use of a simple bilinear bond stress – slip law can be sufficient to properly describe the bond phenomenon, without resorting to more sophisticated local laws. The combination of numerical modelling and analytical solution provided the minimum required bond length both for CFRP and GFRP applied on clay brick, which are of 70.13 mm and 59.41 mm, respectively. The finite element analyses highlighted that the analytical solution proposed by Yuan *et al.* (2001, 2004) can be profitably used as design procedure for the studied EB-FRP system.
- Concerning the calibration of the c_1 coefficient, related to the evaluation of the fracture energy proposed in CNR DT-200 (2004), the obtained results show a similar behaviour of stones and bricks, although a quite large scattering depending on the calibration method has been observed, whereas masonry prisms seem to offer higher performances. The reliability of these results, however, is strongly affected by the limited base of data currently available and by the lacking of some crucial data that are rarely given by the common sources.

Fourteen V-shape Peel Tests, whose test set-up was adapted by similar tests carried out on concrete substrate (Wu *et al.* 2005a, Dai *et al.* 2007) were performed using CFRP reinforcement applied on solid clay bricks. Although these should be considered as pilot tests, the main characteristics of the investigated phenomenon were substantially identified, and the experimental observations do not differ much from what Wu *et al.* (2005a), Dai *et al.* (2007) reported:

- the peel load, during the detachment, oscillated within a limited range, though the scattering was in some cases very large;
- maximum loads of around 18 N/mm were observed, except for the S4 series that gave 28.8 N/mm;
- the first peak load was generally higher than the others, for monotonic tests;
- the peel angle, similarly to the peel load, oscillated around quasi-constant values, being high the scattering also in this case; the estimated average values ranged from 4.5 to 7 degrees;

- the calculated mixed-mode fracture energies ranged from 0.3 to 0.7 N/mm, consistently with values proposed in literature for quasi-brittle substrates (Wu *et al.* 2005a, Dai *et al.* 2007, De Lorenzis and Zavarise 2008).

The application of the available interpretative models to the global behaviour of the collected experimental case studies, has led to the following observations:

- concerning the extrados reinforcement, the models related to the flexural crisis (masonry crushing or fibres rupture) lead to failure loads generally higher (from 150% to 300%) than the experimental ones, since the bending strength could not be reached before the shear crisis occurs at lower load levels. Only in the case of the models tested at the University of Padova these predictions were very close (80% and 112%) to the experimental failure loads, despite the different mechanisms, and it could be related to the peculiar thickness/span ratio of these arches, that could have emphasized the flexural behaviour.
- The models related to the intermediate FRP debonding, never observed during the reported campaigns, give to predictions that vary from 40% to 270% ($c_1 = 0.03$) or from 90% to 540% ($c_1 = 0.124$) of the experimental failure loads, strongly dependent on the chosen values of the experimental coefficient c_1 . It has also to be noticed that the bond behaviour of FRP applied on masonry prisms with mortar joints, at present, is far from being investigated, as most tests have been carried out on single resisting elements: therefore, it is difficult to suppose if the presence of the joint has a negative or positive influence, since the irregular surface could generate a sort of dissipative mechanism similar to friction that could justify some experimental results reported by Basilio (2007), Casareto *et al.* (2003).
- The shear failure criterion does not seem to be sufficiently adequate to provide a reliable estimation of the failure load, since it is strongly dependent on the assumed friction coefficient, on the geometry of the structure and the adopted static schemes, including the position of the hinges. Where the cross section is bended and the reinforcement is activated, the compressive force assures high strength against shear, as observed by Foraboschi (2004), but near the extrados hinge at the springer the eccentricity is lower than half the thickness of the arch, therefore the reinforcement is not active (and it could not be for the collected tests, since in that area the bonded length is lower than the effective one and there were no anchoring devices): hence, the minimum required friction coefficient, to assure the resistance of the joint, quickly increases along a very small area. Concerning the intrados reinforcement, the models related to flexure and intermediate debonding give to considerations similar to the extrados reinforcement.
- The model describing the fibres detachment due to normal stresses generated by the curved morphology of the intrados, while correctly interprets the phenomenon, seems to overestimate the failure load (more than 230% the experimental one); this could be justified, for the collected cases, since assuming the minimum curvature radius pertinent to the collected specimens, 0.76 m, the model gives a very high value (730 N/mm) of tensile force that activates the ultimate strength for 1 N/mm² of tensile strength of the substrate, approximately varying the commonly used FRP textiles tensile strength per ply from 270 N/mm to 565 N/mm (MBrace[®] datasheets).

References

- ACI 318-99 (2004). Building code requirements for structural concrete. American Concrete Institute.
- Aiello, M. A. and Sciolti, M. S. (2006). Bond analysis of masonry structures strengthened with cfrp sheets. *Construction and Building Materials*, **20**, 90–100.
- ASTM C1583 (2004). Standard test method for tensile strength of concrete surfaces and the bond strength or tensile strength of concrete repair and overlay materials by direct tension (pull-off method). American...
- Badalá, A., Cuomo, M., and D’Agata, G. (2008). Analisi limite di volte a botte rinforzate con CFRP. In *XVII Convegno Italiano di Meccanica Computazionale – GIMC2008*. In Italian.
- Barbieri, A., Borri, A., Corradi, M., and Di Tommaso, A. (2002). Dynamic behaviour of masonry vaults repaired with FRP: Experimental analysis. In *6th Int. Masonry Conf. of the British Masonry Society*.
- Basilio, I. (2007). *Strengthening of arched masonry structures with composite materials*. Ph.D. thesis, University of Minho (PT).
- Bikerman, J. J. (1957). Theory of peeling through a hookean solid. *Journal of Applied Physics*, **28**(12), 1484–1485.
- Bizindavyi, L. and Neale, K. (1999). Transfer lengths and bond strengths for composites bonded to concrete. *ASCE Journal of Composites for Construction*, **3**(4), 153–160.
- Briccoli Bati, S. and Rovero, L. (2000). Consolidation of masonry arches with carbon-fiber reinforced plastics. In *12th Int. Brick and Block Masonry Conference*, Madrid, (ES).
- Briccoli Bati, S. and Rovero, L. (2008). Towards a methodology for estimating strength and collapse mechanism in masonry arches strengthened with fibre reinforced polymer applied on external surfaces. *RILEM Materials and Structures*, **41**(7), 1291–1306.
- Briccoli Bati, S. and Rovero, L. (2009). Bond strength between brick and CFRP strips. In *III Nat. Conf. on Mechanics of masonry structures strengthened with composite materials: modeling, testing, design, control – MuRiCo3*, pages 146–152, Venice, (IT).
- Briccoli Bati, S., Rovero, L., and Tonietti, U. (2007). Adesione fra blocchi in laterizio e rinforzo in CFRP (in italian). In *Materiali ed Approcci Innovativi per il Progetto in Zona Sismica e la Mitigazione della Vulnerabilità delle Strutture – Nat. W. ReLUIS*, pages 213–220, Salerno, (IT). in Italian.
- Camli, U. and Binici, B. (2007). Strength of carbon fiber reinforced polymers bonded to concrete and masonry. *Construction and Building Materials*, **21**, 1431–1446.

- Capozucca, R. (2010). Experimental frp/srp-historic masonry delamination. *Composite Structures*, **92**, 891–903.
- Carbone, I., Fiore, A., and Pistone, G. (2001). *Le costruzioni in muratura. Interpretazione del comportamento statico e tecniche di intervento (in Italian)*. Hoepli, Milano (IT).
- Cartolaro, A. (2004). *Volte in muratura di mattoni rinforzate con FRP. Studio sperimentale dei meccanismi resistenti alle interfacce (in Italian)*. Master's thesis, University of Padova (Italy).
- Casareto, M., Olivieri, A., Romelli, A., and Lagomarsino, S. (2003). Bond behaviour of FRP laminates adherent to masonry. In *Int. Conf. Advancing with Composites 2003*, Milan, (IT).
- Castori, G. (2006). *Strengthening of masonry elements with innovative composite laminates*. Ph.D. thesis, University of Perugia (IT).
- Chajes, M., Finch, W. J., Januszka, T., and Thomson, T. (1996). Bond and force transfer of composite material plates bonded to concrete. *ACI Structural Journal*, **93**(2), 295–303.
- Chen, J. F. and Teng, J. G. (2001). Anchorage strength models for FRP and steel plates bonded to concrete. *Journal of Structural Engineering*, **127**(7), 784–791.
- CNR DT-200 (2004). Guide for the design and construction of externally bonded frp systems for strengthening existing structures. Italian National Research Council.
- Dai, J. and Ueda, T. (2003). Local bond stress slip relations for FRP Sheets-Concrete interfaces. In *6th Int. Symp. on Fibre-Reinforced Polymer Reinforcement for Concrete Structures – FRPRCS-6*, page 143–152, Singapore, (SGP).
- Dai, J., Ueda, T., and Sato, Y. (2005). Development of the nonlinear bond Stress-Slip model of fiber reinforced plastics Sheet-Concrete interfaces with a simple method. *ASCE Journal of Composites for Construction*, **9**(1), 52–62.
- Dai, J., Ueda, T., and Sato, Y. (2007). Bonding characteristics of fiber-reinforced polymer sheet-concrete interfaces under dowel load. *ASCE Journal of Composites for Construction*, **11**(2), 138–148.
- De Lorenzis, L. and Zavarise, G. (2008). Modeling of mixed-mode debonding in the peel test applied to superficial reinforcements. *Int. Journal of Solids and Structures*, **45**, 5419–5436.
- De Lorenzis, L., Miller, B., and Nanni, A. (2001). Bond of frp laminates to concrete. *ACI Materials Journal*, **98**(3), 256–264.
- DM 09-01-96 (1996). Norme tecniche per il calcolo, l'esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresso, e per le strutture metalliche. Decreto del Ministero dei Lavori Pubblici.
- Faccio, P. and Foraboschi, P. (2000). Analisi agli stati limite ultimi di volte in muratura con rinforzi in FRP (in italian). *L'edilizia*, **5**(6), 48–56.
- Faella, C., Martinelli, E., Paciello, S., and Perri, F. (2009). Composite materials for masonry structures: the adhesion issue. In *III Nat. Conf. on Mechanics of masonry structures strengthened with composite materials: modeling, testing, design, control – MuRiCo3*, pages 266–273, Venice, (IT).

- Ferracuti, B., Martinelli, E., Nigro, E., and Savoia, M. (2007a). Fracture energy and design rules against FRP - concrete debonding. In *Fibre-Reinforced Polymer Reinforcement for Concrete Structures – FRPRCS-6*, Patras, (?).
- Ferracuti, B., Savoia, M., and Mazzotti, C. (2007b). Interface law for frp–concrete delamination. *Composite structures*, **80**(4), 523–531.
- fib Bulletin 14 (2001). Externally bonded frp reinforcement for rc structures - technical report on the design and use of externally bonded fibre reinforced polymer reinforcement for reinforced concrete structures. Fédération Internationale du Béton.
- Focacci, F. (2008). *Rinforzo delle murature con materiali compositi (in Italian)*. Dario Flaccovio Ed., Palermo (IT).
- Foraboschi, P. (2004). Strengthening of masonry arches with fiber-reinforced polymer strips. *ASCE Int. Journal of Composites for Construction*, **8**(3), 191–202.
- Gent, A. N. and Hamed, G. R. (1975). Peel mechanics. *Journal of Adhesion*, **7**, 91–95.
- Giomo, F. (2008). *Rinforzo di strutture murarie con materiali compositi FRP: analisi sperimentale e modellazione del comportamento dell’interfaccia soggetta ad azioni composte nel caso di applicazione ad archi e volte (in Italian)*. Master’s thesis, University of Padova (Italy).
- Grande, E., Imbimbo, M., and Sacco, E. (2008). FRP-strengthening of masonry structures: effect of debonding phenomenon. In *VI Int. Conf. on Structural Analysis of Historic Constructions*, pages 1017–1023, Bath, (UK). ISBN 978-0-415-46872-5.
- Heyman, J. (1982). *The masonry arch*. Ellis Horwood Limited, Chichester (UK).
- Karbhari, V. M. and Engineer, M. (1996). Investigation of bond between concrete and composites: Use of a peel test. *Journal of Reinforced Plastics and Composites*, **15**, 208–227.
- Karbhari, V. M., Niu, H., and Sikorsky, C. (2006). Review and comparison of fracture mechanics-based bond strength models for FRP-strengthened structures. *Journal of Reinforced Plastics and Composites*, **25**, 1757–1794. DOI: 10.1177/0731684406074475.
- Lam, C. C. (2009). *Finite element study of bond-slip behaviour of CFRP and GFRP laminates on brick masonry*. Master’s thesis, University of Padova (Italy).
- Lee, Y., Boothby, T., Bakis, C., and Nanni, A. (1999). Slip modulus of frp sheets bonded to concrete. *ASCE J. of Composites for Construction*, **3**(4), 161–167.
- Lu, X. Z., Teng, J. G., Ye, L. P., and Jang, J. J. (2004). Bond-slip models for FRP sheet/plate-to-concrete interfaces. In *2nd Int. Conf. on Advanced Polymer Composites for Structural Applications in Construction – ACIC-2004*, pages 152–161, Guildford, (UK).
- Lu, X. Z., Teng, J. G., Ye, L. P., and Jang, J. J. (2005). Bond-slip models for FRP sheets/plates bonded to concrete. *Engineering Structures*, **27**, 920–937.
- Mazzotti, C., Savoia, M., and Ferracuti, B. (2009). A new single-shear set-up for stable debonding of frp–concrete joints. *Construction and Building Materials*, **23**, 1529–1537.
- Nakaba, K., Kanakubo, T., Furuta, T., and Yoshizawa, H. (2001). Bond behavior between Fiber-Reinforced polymer laminates ad concrete. *ACI Structural Journal*, **98**(3), 359–367.

- Nicholson, D. W. (1977). Peel mechanics with large bending. *Int. Journal of Fracture*, **13**(3), 279–287.
- Panizza, M. (2006). *Analisi sperimentale di meccanismi di interazione locale di volte in muratura rinforzate con tessuti FRP (in Italian)*. Master's thesis, University of Padova (Italy).
- Panizza, M., Garbin, E., Valluzzi, M. R., and Modena, C. (2008a). Bond behaviour of CFRP and GFRP laminates on brick masonry. In *VI Int. Conf. on Structural Analysis of Historic Constructions*, pages 763–770, Bath, (UK). ISBN 978-0-415-46872-5.
- Panizza, M., Garbin, E., Valluzzi, M. R., and Modena, C. (2008b). Shear mechanism of brick masonry vaults. In *Structural Faults + Repair – SF&R2008*, pages 763–770, Edimburgh, (UK). On CD-Rom.
- Panizza, M., Garbin, E., Valluzzi, M. R., and Modena, C. (2009). Experimental study of the FRP-clay interface subject to normal stresses. In *III Nat. Conf. on Mechanics of masonry structures strengthened with composite materials: modeling, testing, design, control – MuRiCo3*, pages 391–398, Venice, (IT).
- Savoia, M., Ferracuti, B., and Mazzotti, C. (2003a). Non linear bond-slip law for FRP-concrete interface. In *6th Int. Symp. on Fibre-Reinforced Polymer Reinforcement for Concrete Structures – FRPRCS-6*, pages 1–10, Singapore (SGP).
- Savoia, M., Ferracuti, B., and Mazzotti, C. (2003b). Una legge di interfaccia non lineare per placcaggi con lamine in FRP. In *16th AIMETA Congress of Theoretical and Applied Mechanics*, Ferrara (IT). In Italian.
- Subramanian, K., Focacci, F., and Carloni, C. (2009). An investigation on the interface fracture propagation between frp and masonry. In *III Nat. Conf. on Mechanics of masonry structures strengthened with composite materials: modeling, testing, design, control – MuRiCo3*, pages 423–430, Venice, (IT).
- Sun, Z., Wan, K. T., and Dillard, D. A. (2004). A theoretical and numerical study of thin film delamination using the pull-off test. *Int. Journal of Solids and Structures*, **41**, 717–730.
- Täljsten, B. (1996). Strengthening of concrete prisms using the plate-bonding technique. *International Journal of Fracture*, **82**, 253–266.
- Täljsten, B. (1997). Defining anchor lengths of steel and CFRP plates bonded to concrete. *Int. J. of Adhesion and Adhesives*, **19**, 319–327.
- Thouless, M. D. and Jensen, H. M. (1992). Elastic fracture mechanics of the peel-test geometry. *Journal of Adhesion*, **38**, 185–197.
- TNO Diana BV. (2008). Diana finite element analysis release 9.3.
- Triantafillou, T. (1998a). Strengthening of masonry structures using epoxy-bonded FRP laminates. *ASCE Int. Journal of Composites for Construction*, **2**(2), 96–104.
- Triantafillou, T. (1998b). Strengthening of masonry structures using epoxy-bonded FRP laminates. errata. *ASCE Int. Journal of Composites for Construction*, **2**(4), 203.

- Ueda, T. and Dai, J. (2005). Interface bond between FRP sheets and concrete substrates: properties, numerical modelling and roles in member behaviour. *Progress in Structural Engineering Materials*, **7**, 27–43.
- UNI-EN 1015-12 (199?). Determinazione dell’aderenza al supporto di malte da intonaco esterno ed interno. Ente Nazionale Italiano per l’Unificazione.
- Valdemarca, M. (1998). *Analisi sperimentale del comportamento di volte murarie rinforzate con FRP (in Italian)*. Master’s thesis, University of Padova (Italy).
- Valluzzi, M. R., Valdemarca, M., and Modena, C. (2001). Behaviour of brick masonry vaults strengthened by frp laminates. *ASCE Int. Journal of Composites for Construction*, **5**(3), 163–169.
- Valluzzi, M. R., Tinazzi, D., Garbin, E., and Modena, C. (2003). Fem modelling of cfrp strips bond behaviour for bed joints reinforcement techniques. In *6th Int. Conf. on Computer Methods in Structural Masonry*, Rome, (IT).
- Wu, Z., Yuan, H., and Niu, H. (2002). Stress transfer and fracture propagation in different kinds of adhesive joints. *ASCE Journal of Engineering Mechanics*, **128**(5), 562–573.
- Wu, Z., Yuan, H., Kojima, Y., and Ahmed, E. (2005a). Experimental and analytical studies on peeling and spalling resistance of unidirectional FRP sheets binded to concrete. *Composites Science and Technology*, **65**, 1088–1097.
- Wu, Z., Yuan, H., Asakura, T., Yoshizawa, H., Kobayashi, A., Kojima, Y., and Ahmed, E. (2005b). Peeling behavior and spalling resistance of bonded bidirectional fiber reinforced polymer sheets. *Int. Journal of Composites for Construction*, **9**(3), 214–226.
- Yao, J., Teng, J., and Chen, J. (2005). Experimental study on FRP-to-concrete bonded joints. *Composites: Part B*, **36**, 99–113.
- Yuan, H., Wu, Z., and Yoshizawa, H. (2001). Theoretical solutions on interfacial stress transfer of externally bonded steel/composite plates. *Journal of Structural Mechanics and Earthquake Engineering*, **18**(1), 27–39.
- Yuan, H., Chen, J. F., and Teng, J. G. (2003). Interfacial stresses between frp plate and concrete in a peel test: an analytical solution. In *10th Int. Conf. on Structural Faults + Repair*, London, (UK).
- Yuan, H., Teng, J., Seracino, R., Wu, Z., and Yao, J. (2004). Full-range behavior of FRP-to-concrete bonded joints. *Engineering Structures*, **26**, 553–565.