

Universidade do Minho
Escola de Engenharia

Christoph Fernandes de Sousa

Development of a multifunctional composite sandwich panel
for the rehabilitation of building façades

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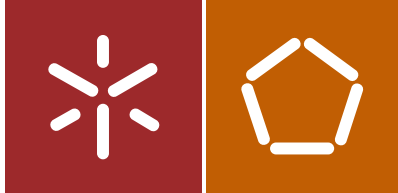
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Christoph Fernandes de Sousa

Development of a multifunctional composite
sandwich panel for the rehabilitation of
building façades

Philosophy Doctorate Thesis
Civil Engineering

Work conducted under supervision of:
Professor Joaquim A. O. Barros
Professor João Ramôa Correia

July, 2021

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ABSTRACT

This PhD thesis presents a research work aiming at the development of a sustainable and multifunctional composite sandwich panel for the rehabilitation of reinforced concrete (RC) buildings from the 1960s to the mid-1980s. The sandwich panel retrofit solution developed in the thesis comprises four main components: (i) thin outer layers of Recycled Steel Fibre Reinforced micro Concrete (RSFRC); (ii) a lightweight core made of polystyrene; (iii) internally distributed glass fibre reinforced polymer (GFRP) connectors; and (iv) steel anchors for fixing the panel to the existing structure.

The first part of this work's experimental program encompassed pushout and pullout tests, carried out on reduced-scale specimens representative of the sandwich panel solution; these tests aimed at assessing the overall composite behaviour of the sandwich panel and analysing the influence of the type of core insulation layer, and of the anchoring conditions and diameter of the GFRP connectors. The tests showed that the adopted structural GFRP connectors are able to adequately ensure shear load transfer between RSFRC layers.

The second part of the experimental program involved testing intermediate-scale RC frame specimens, representative of the target building typology, under in-plane cyclic loading conditions. The cyclic tests were performed on different variations of the referred RC frames: (i) a bare RC frame; (ii) an RC frame with a masonry infill wall; and (iii) an RC frame with the incorporation of the sandwich panel prototype developed in the scope of this research work. The results of the tests show that, in comparison with the traditional masonry infill wall solution, the proposed rehabilitation solution enabled a significant improvement of the RC frame's cyclic performance, providing higher levels of load carrying capacity and energy dissipation.

The numerical part of this study included numerical simulations conducted to assist the sandwich panel design process and, more specifically, the modelling of the failure mechanisms observed at the interface between the RSFRC layers and the polystyrene core; good agreement was obtained between experimental and numerical results, with important conclusions being drawn regarding the cohesion and friction angle between these components of the sandwich panel.

Keywords: Rehabilitation; RC frame buildings; building façades; sandwich panel; precast concrete; recycled steel fibres; GFRP connectors; polystyrene; pushout tests; pullout tests; cyclic tests; finite element method.

RESUMO

Esta tese apresenta um trabalho de investigação relativo ao desenvolvimento de um painel sanduíche compósito sustentável e multifuncional concebido para a reabilitação de edifícios porticados de betão armado (BA) construídos entre 1960 e meados da década de 1980. Este painel é composto essencialmente por quatro componentes principais: (i) finas camadas exteriores em microbetão reforçado com fibras de aço recicladas (BRFR); (ii) núcleo em poliestireno; (iii) conectores poliméricos reforçados com fibras de vidro (GFRP); e (iv) ancoragens metálicas para ligação do painel à estrutura a reabilitar.

A primeira parte do programa experimental do presente trabalho envolveu a realização de ensaios de corte e ensaios de arrancamento em provetes de escala reduzida, representativos do painel em desenvolvimento; com estes ensaios, pretendeu-se avaliar o funcionamento compósito do painel através da análise da influência do tipo de camada de isolamento e do comprimento de ancoragem e diâmetro dos conectores em GFRP. Os resultados obtidos revelaram que os conectores em GFRP de cariz estrutural são eficazes na transmissão de esforços de corte entre as camadas de BRFR.

A segunda parte do programa experimental consistiu na realização de ensaios cíclicos em pórticos de BA de escala intermédia, representativos da tipologia de edifícios a reabilitar. Os ensaios foram realizados utilizando três variantes dos referidos pórticos: (i) pórtico de BA isolado; (ii) pórtico de BA com parede de enchimento em alvenaria; e (iii) pórtico de BA reforçado com um protótipo do painel sanduíche desenvolvido no âmbito deste trabalho. Os resultados mostraram que, em comparação com a solução tradicional com enchimento em alvenaria, a solução de reabilitação proposta produziu uma melhoria significativa no comportamento cíclico do pórtico em termos de capacidade de carga e energia dissipada.

A parte numérica do presente estudo incluiu a realização de simulações de apoio ao processo de dimensionamento do painel e, mais especificamente, no auxílio à interpretação do mecanismo de rotura observado na interface entre as lâminas de BRFR e o núcleo em poliestireno. Registou-se uma boa concordância entre as respostas numérica e experimental, o que permitiu obter conclusões importantes relativamente aos valores de coesão e ângulo de atrito que caracterizam a interface entre estes componentes do painel sanduíche.

Palavras-chave: Reabilitação; edifícios porticados; fachadas de edifícios; painéis sanduíche; betão pré-fabricado; fibras recicladas; conectores GFRP; poliestireno; ensaios de corte; ensaios de arranque; método dos elementos finitos.

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LIST OF ABBREVIATIONS

ADRS	–	Acceleration-displacement response spectra
BRB	–	Buckling-restrained brace
CMOD	–	Crack mouth opening displacement
CoV	–	Coefficient of variation
EN	–	European norm
EPS	–	Expanded polystyrene
ETICS	–	External thermal insulation composite system
EU	–	European Union
FEM	–	Finite element method
FEMA	–	American Federal Emergency Management Agency
F-GFRP	–	Functional GFRP connector
FRCC	–	Fibre reinforced cementitious composites
FRP	–	Fibre reinforced polymer
GFRP	–	Glass fibre reinforced polymer
GL-IS	–	Gauss Legendre integration scheme
HIT	–	Adhesive anchoring system
HST	–	Mechanical anchoring system
HVAC	–	Heating, ventilation and air conditioning
IFE	–	Interface finite element
ISF	–	Industrial steel fibres
ISO	–	International Organization for Standardization
LVDT	–	Linear variable differential transducer
MEP	–	Mechanical, electrical and plumbing
NP	–	Portuguese norm
PCM	–	Phase change materials
RC	–	Reinforced concrete
REBA	–	Portuguese code for reinforced concrete structures
RGR	–	Portuguese general regulation on noise
RSA	–	Portuguese code for safety and actions for building and bridge structures
RRAE	–	Portuguese regulation on acoustic requirements for buildings
RSEP	–	Portuguese code for building and bridge loads

RSF	–	Recycled steel fibres
RSFRC	–	Recycled steel fibre reinforced micro-concrete
RSCCS	–	Portuguese code for building safety against earthquakes
SDOF	–	Single degree of freedom
S-GFRP	–	Structural GFRP connector
SHS	–	Steel hollow section
SLS	–	Service limit states
TRM	–	Textile reinforced mortars
ULS	–	Ultimate limit states
U-value	–	Heat transfer coefficient
XPS	–	Extruded polystyrene

LIST OF SYMBOLS

Roman upper-case letters

A_i	–	Areas enclosed between experimental and bilinear curves
A_{ce}	–	Effective area of contact in the concrete/polystyrene interface
$D_{2m,n,w}$	–	Airborne sound insulation index of the façade
$D_{n,w}$	–	Airborne sound insulation index for partitions
E	–	Elastic modulus
F_{max}	–	Maximum load
F_{max1}	–	Maximum load obtained for the first loading cycle
F_{max2}	–	Maximum load obtained for the second loading cycle
$G_{F,I}$	–	Fracture energy in mode I
$G_{F,II}$	–	Fracture energy in mode II
$G_{F,c}$	–	Compressive fracture energy
$L'_{n,w}$	–	Impact sound insulation index
N_{Ed}	–	Design acting axial force
N_{Rd}	–	Design value for tensile resistance
V_{Ed}	–	Design acting shear force
V_{Rd}	–	Design value for shear resistance
W^{cr}	–	Work produced during the fracture process

Roman lower-case letters

c	–	Cohesion
c_0	–	Initial cohesion
d_c	–	Concrete cone depth
d_f	–	Fibre diameter
d_{Fmax}	–	Lateral displacement corresponding to maximum load

$d_{F_{\max 1}}$	–	Lateral displacement corresponding to maximum load at first loading cycle
$d_{F_{\max 2}}$	–	Lateral displacement corresponding to maximum load at second loading cycle
d_u	–	Ultimate lateral displacement
d_y	–	Displacement at yield point
f_c	–	Compressive strength
$f_{s,h}$	–	Hardening stress of steel
$f_{s,u}$	–	Ultimate stress of steel
$f_{s,y}$	–	Yield stress of steel
f_t	–	Tensile strength
$f_{t,0}$	–	Initial tensile strength
f_{yk}	–	Characteristic yield strength
h	–	Height of the horizontal load application
h_c	–	Embedment depth
k_{con}	–	Elastic stiffness of the F-GFRP connector
k_{el}	–	Elastic stiffness
k_n	–	Elastic stiffness in orthogonal direction
k_s	–	Lateral secant stiffness
k_{t_1}	–	Elastic stiffness in tangential direction t_1
k_{t_2}	–	Elastic stiffness in tangential direction t_2
l_f	–	Fibre length
p	–	Third branch exponent for steel (stress-strain constitutive law)
p_1	–	Parameter defining the mode I fracture energy available to the new crack
s_0	–	Slip at the end of the linear bond stress-slip relationship
s_m	–	Slip at peak bond stress
$\underline{u}$	–	Displacement vector
u_1	–	Lateral displacement
u_n	–	Displacement in normal direction
u_{t_1}	–	Displacement in tangential direction t_1

u_{t_2} – Displacement in tangential direction t_2

Greek letters

α_0 – Parameter defining the beginning of the non-linear behaviour in compression

α_i – Normalized stress parameter ($i = 1, 2$) in tri-linear stress-strain diagram

α_c – Concrete cone angle

α_{th} – Threshold angle

β – Load reduction factor

δ – Lateral drift

$\delta_{\max}$ – Maximum relative displacement

$\delta_{F_{\max}}$ – Relative displacement at peak load

ε_y – Yield strain of reinforcing steel bars

ε_{cp1} – Strain at compression peak stress

$\varepsilon_{s,h}$ – Hardening strain of steel

$\varepsilon_{s,u}$ – Ultimate strain of steel

$\varepsilon_{s,y}$ – Yield strain of steel

λ – Softening parameter

λ_c – Softening parameter for the cohesion

λ_{f_t} – Softening parameter for the tensile strength

λ_f – Fibre aspect ratio

$\lambda_{\tan \phi}$ – Softening parameter for the friction angle

μ_{Δ} – Displacement ductility factor

ν – Poisson's coefficient

ξ_i – Normalised strain parameter ($i = 1, 2$) in tri-linear stress-strain diagram

$\underline{\sigma}$ – Stress vector

σ_n – Normal stress

σ_{VM} – Von Mises stress

- σ_X – Axial stress according to local axis X
- τ – Shear stress
- τ_{t_1} – Shear stress in tangential direction t_1
- τ_{t_2} – Shear stress in tangential direction t_2
- τ_Y – Shear stress according to local axis Y
- τ_Z – Shear stress according to local axis Z
- ϕ – Friction angle
- ϕ_0 – Initial friction angle
- ϕ_r – Residual friction angle

1 INTRODUCTION

1.1 Scope and motivation

The activities of rehabilitation and conservation constitute key factors for the sustainability of the built heritage. Taking into account that buildings are designed for a service life of several decades, it is clear that there is a need for keeping them under adequate conditions, which implies maintenance operations that must be conducted during the buildings' service life. This is, therefore, a very relevant issue for modern societies, not only because civil engineering structures naturally age and deteriorate, but also because modern design codes involve more stringent requirements that a significant portion of existing buildings cannot cope with. Regarding the retrofitting of the existing housing stock, the interest of the research community is nowadays focusing on two different research areas: (i) structural rehabilitation, namely on buildings that exhibit structural vulnerabilities under seismic actions; and (ii) energetic refurbishment, due to the targets defined by the European Union (EU) regarding energy efficiency, reduction of carbon emissions and use of on-site renewable energy sources.

In the case of Portugal, the 2011 Census Survey (<http://censos.ine.pt>) revealed a country with about 400'000 buildings that need significant repair works; this represents more than 10% of the housing stock (4.4% of which are very degraded or in need of major repairs). In 2001, rehabilitated dwellings represented 1.8% of the total number of building units. This percentage rose to 15.6% in 2011, as depicted in Figure 1.1, but this increase was strongly influenced by the 75.2% reduction in new construction of housing units completed during this period. In fact, the weight of rehabilitation in the construction sector with regard to production volume was merely 8% in 2011, which is a rather small percentage when compared to the average of the 28 EU member states, which was 26% (DGEG, 2014; IHRU, 2015).

During the last decades, devastating earthquakes worldwide confirmed the deficiencies of numerous building structures. The experience gained from field observations and back-analysis over the years led to the improvement of the level of knowledge and the evolution of seismic codes. However, a large inventory of building structures in seismic regions throughout the world still exhibits structural vulnerabilities as it comprises buildings that do not comply with modern seismic design codes. In order to mitigate the seismic and other natural hazards, existing inadequate or deteriorating structures should

be rehabilitated (Ghobarah, 2000; Frosch, 2006; Thermou and Elnashai, 2006). This issue is particularly concerning for reinforced concrete (RC) buildings, which in Portugal represent about 50% of the total building stock, hosting approximately 60% of the national population according to the 2011 Census Survey (INE, 2012; Silva *et al.*, 2015a). The economic and social impact resulting from a loss of these structures during a seismic event is enormous, encouraging governments and public institutions to establish policies and support systems that promote structural rehabilitation and the improvement of the seismic resistance of buildings subjected to rehabilitation works (IHRU, 2015). In fact, this was recently emphasized in Portugal, through the issuance of Decree-Law 95/2019 (DRE, 2019a) and Ordinance 302/2019 (DRE, 2019b), which consider seismic vulnerability assessment reports to be mandatory in most large-scale rehabilitation interventions. This new legal framework provides clarification regarding: (i) the terms on which the seismic vulnerability of buildings must be assessed and reported; and (ii) the situations in which the elaboration of a seismic strengthening project is required (Maio *et al.*, 2020).

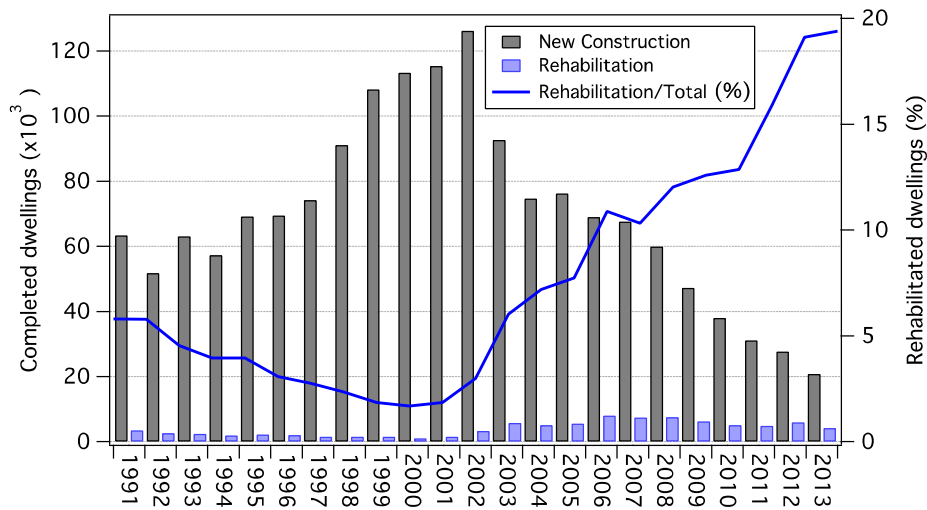


Figure 1.1: Evolution of the number of completed dwellings and the weight of dwellings completed through renovation works (1991-2013) (IHRU, 2015).

Concomitantly with the safety of these buildings and protection of their inhabitants, the overall indoor comfort condition of buildings and their energy efficiency is also a key factor in the overall economy and in the future of the building sector. Taking into account that, in 2011, buildings accounted for about 40% of the EU's energy consumption (European Commission, 2012), the control of the energy consumption in this sector is mandatory in order to reduce the EU energy dependency and decrease the greenhouse gas emissions: EU has set itself the long-term goal of reducing greenhouse gas emissions by 80-95% until 2050, when compared to 1990 levels. In fact, one of the EU's main goals is moving towards new and retrofitted nearly-zero energy buildings: in accordance with Directives 2010/31/EU and 2012/27/EU

(European Commission, 2010, 2012), all new buildings should be nearly-zero energy buildings by 2020 and energy efficient renovations should be applied to at least 3% of the existing buildings owned and occupied by central governments. Moreover, according to the above-mentioned Directives, it is mandatory for all EU member states to draw-up long-term national building renovation strategies, directed to both public and private buildings.

1.2 Research objectives

Bearing in mind the scope of this work, the main objective for this PhD thesis consists on the development of a multifunctional and sustainable composite sandwich panel, specifically designed for structural and thermal retrofitting of existing RC frame buildings, built in Portugal until the mid-1980s (target buildings for the present thesis), through intervention on their envelope (refurbishment of building façades – see schematic illustration in Figure 1.2). In short, the main goal is to be achieved by taking advantage of the usual characteristics associated to sandwich panel technology: high stiffness-to-weight and strength-to-weight ratios, and improved thermal insulation performance, obtained by enclosing (i) two thin skins of a stiff material (e.g. fibre reinforced cement composites), which essentially provides stiffness, with (ii) a lightweight softer core in between (e.g. polymeric foam), which fulfils thermal insulation requirements. The final contribution resulting from the development and application of the referred multifunctional sandwich panel solution is the safety and indoor comfort of inhabitants leading to an increase on the real estate value of the referred buildings, with consequent social and economic benefits.



Figure 1.2: Illustration of proposed retrofit approach.

Taking into account the complexity involved in the development of such composite constructive system, the partial objectives and scientific challenges of the present doctoral thesis are listed below:

- The first objective of this research work consists of studying and characterizing the target building typology. The characterization of the target housing stock focused on the appraisal of characteristics of existing RC frame buildings built in Portugal until the mid-1980s (buildings that were designed without adequate earthquake provisions) for identification of their limitations in terms of structural behaviour and thermal comfort of their inhabitants.
- The sustainability of the developed rehabilitation solution should be taken into account (at least, from a qualitative point of view) when selecting materials and components for the sandwich panel. In view of this, one of the goals that was pointed out for this thesis is the use of steel fibres resulting from the tyre recycling industry as discrete reinforcement of the cementitious sandwich panel layers (instead of industrial steel fibres or longitudinal steel reinforcing bars). This decision entailed a necessary adjustment in terms of concrete mix composition, which had to accommodate steel fibres with irregular lengths/shapes while maintaining proper rheological characteristics that allow for the production of thin concrete wythes (between 20 and 30 mm of thickness) and, at the same time, ensuring high levels of both compressive and tensile strengths with ductile post-cracking behaviour.
- Taking firstly and separately into account the need for structural rehabilitation of the identified target buildings, the adequate fulfilment of the sandwich panel's structural requirements was one of the main objectives set for this research work. One expected that the retrofit of buildings through the use of this façade sandwich panels would result in an increase of the building's load carrying capacity for lateral loads associated to high levels of drift (namely seismic actions).
- One of the main objectives was to apply the developed façade sandwich panel to an RC frame specimen that is representative of the target building typology, and afterwards to subject the strengthened RC frame specimen to cyclic tests. For comparison, additional RC frame specimens, with and without traditional masonry infill walls, were subjected to the same loading conditions.
- On the other hand, in view of the energetic refurbishment of the referred target buildings, which are commonly characterized by exterior masonry walls with inadequate/absent thermal insulation, an important aim of this work relied on the improvement of the thermal insulation performance of the façade panel. The thermal insulation requirements were accomplished by adopting an insulation core configuration for the sandwich panel that allows for a continuous

insulation layer throughout the building envelope in order to avoid thermal bridges and reduce energy consumption by optimizing its thermal efficiency.

- Considering the type of intervention to be conducted through the use of this rehabilitation strategy, which basically consists of applying the façade panels to the envelope of existing buildings, the aesthetical appearance of the panel is of utmost importance, since it directly affects the final/new appearance of the retrofitted building structure. In fact, there are documented cases where aesthetic and psychological issues dictate the rehabilitation strategies, hence the importance of the aesthetical compatibility of the intervention scheme with the existing building and with the general opinion of their inhabitants. This issue is directly related to the aesthetical appeal of the panel's external RSFRC layers, which need to have a smooth surface finishing.
- In general, there is still a propensity for the use of traditional materials for rehabilitation purposes, such as steel profiles or thick plain concrete elements, which introduce significant dead loads in existing constructions that can lead to an increase in their seismic vulnerability and pose constructive problems associated to transport, elevation and placement operations, especially in old districts with narrow accesses. This study however aims for the development of a lightweight prefabricated rehabilitation solution. The use/production of RSFRC layers, which allows for the adoption of very thin layers, opens the possibility of fitting more lightweight insulation into the sandwich panel configuration, therefore contributing to a minimal added weight to the original structure and requiring less powerful lifting equipment.
- The prefabricated sandwich panel should be easy to assemble, handle, install and, if necessary, dismantle. The objective here is to simplify all the workmanship in the construction site in order to develop, as much as possible, an attractive, competitive, sustainable and straightforward prefabricated construction solution for the rehabilitation sector. This particular objective is related to the assembly of the different components of the panel during its fabrication/casting process and, afterwards, to the use of auxiliary anchors/connectors: the type of panel-building connectivity to be adopted plays an extremely important role in this respect.

1.3 Research methodology

The research methodology adopted for this doctoral thesis follows a work strategy that is organized in three classes of interdependent research methods, namely: (i) bibliographical research (chapter 2); (ii) experimental/laboratory research (chapters 3 and 4); and (iii) numerical research (chapter 5). Figure 1.3 presents a flowchart of the research strategy that was adopted for this thesis, with illustration of the overall workflow structure and the interconnection between chapters/sections, which is addressed in the next paragraphs:

- The study of the details that characterize the target buildings, addressed in section 2.2, served as basis for the development of representative RC frame specimens to be tested experimentally, which were designed accordingly (following older design codes – see subsection 4.2.2).
- The literature review that was conducted to study the main trends in structural and thermal rehabilitation solutions, addressed in sections 2.3 and 2.4, influenced the final design of the façade sandwich panel prototype: it can be viewed as an integrated structural/thermal rehabilitation approach, which is a combination of construction technology based on the concepts of infill wall, sandwich panel and façade panel.
- The laboratory work initiated with the material characterization (section 3.2) of the main materials that were considered for the development of the sandwich panel prototype, namely the recycled steel fibre reinforced micro-concrete (RSFRC), the glass fibre reinforced polymer (GFRP) connectors and the expanded (EPS) and extruded (XPS) polystyrene insulation layers. The material characterization can be seen as a primary step towards the development of the panel (later described in subsection 4.2.3), with inclusion of an initial iterative process that led to the concrete mix composition of the developed RSFRC, which was specifically designed for the production of the main structural components of the panel (thin layers of RSFRC). The material characterization provided also relevant information for the numerical studies addressed in chapter 5.
- The second part of chapter 3 includes pushout (subsections 3.3.2 and 3.3.3) and pullout tests (subsection 3.3.4) that were conducted on sandwich panel assemblies in order to explore the synergetic action between the adopted materials/components. The experimental findings that were derived from these tests were important for the final design of the sandwich panel prototype (subsection 4.2.3) by providing decisive information for the selection of the insulation core layer

and for the definition of shear and tensile strength values that characterize the connection between RSFRC wythes and GFRP connectors.

- Chapter 4 contains the main part of the laboratory work conducted in the scope of this thesis: in-plane cyclic tests conducted on intermediate-scale specimens (section 4.3). It plays a central role in this research by being interconnected with the work that is reported in all the remaining chapters (see Figure 1.3). The experimental program presented in this chapter was conducted to assess the structural performance of the proposed strengthening solution, and to compare it with specimens that are representative of existing/target buildings. Over 30 sensors were employed during the cyclic loading tests, which enabled the analysis of different performance indicators, such as: load-displacement curves, damage evolution, strength and stiffness degradation, displacement ductility factor, energy dissipation, column deformation and tensile strains in the reinforcing bars.
- The numerical part of this research work, reported in chapter 5, is essentially divided in three parts. The first part (section 5.2) contains finite element method (FEM) analysis of the bond behaviour between layers of RSFRC and XPS, which aims to numerically capture the behaviour of sandwich panel specimens when subjected to shear loading (pushout tests of subsection 3.3.2), and to provide additional insights on the underlying failure mechanisms. The second and third parts (sections 5.3 and 5.4, respectively) are interconnected with the work conducted in chapter 4. Section 5.3 is solely based on the information derived from section 4.3: it comprises an attempt to numerically fit the experimental structural behaviour of the RC frame specimen when subjected to in-plane loading. The numerical models presented in section 5.4, which are based on the work performed in section 5.3, were then used to assist the design of the connection between the sandwich panel and the RC frame, which is detailed in subsection 4.2.3.

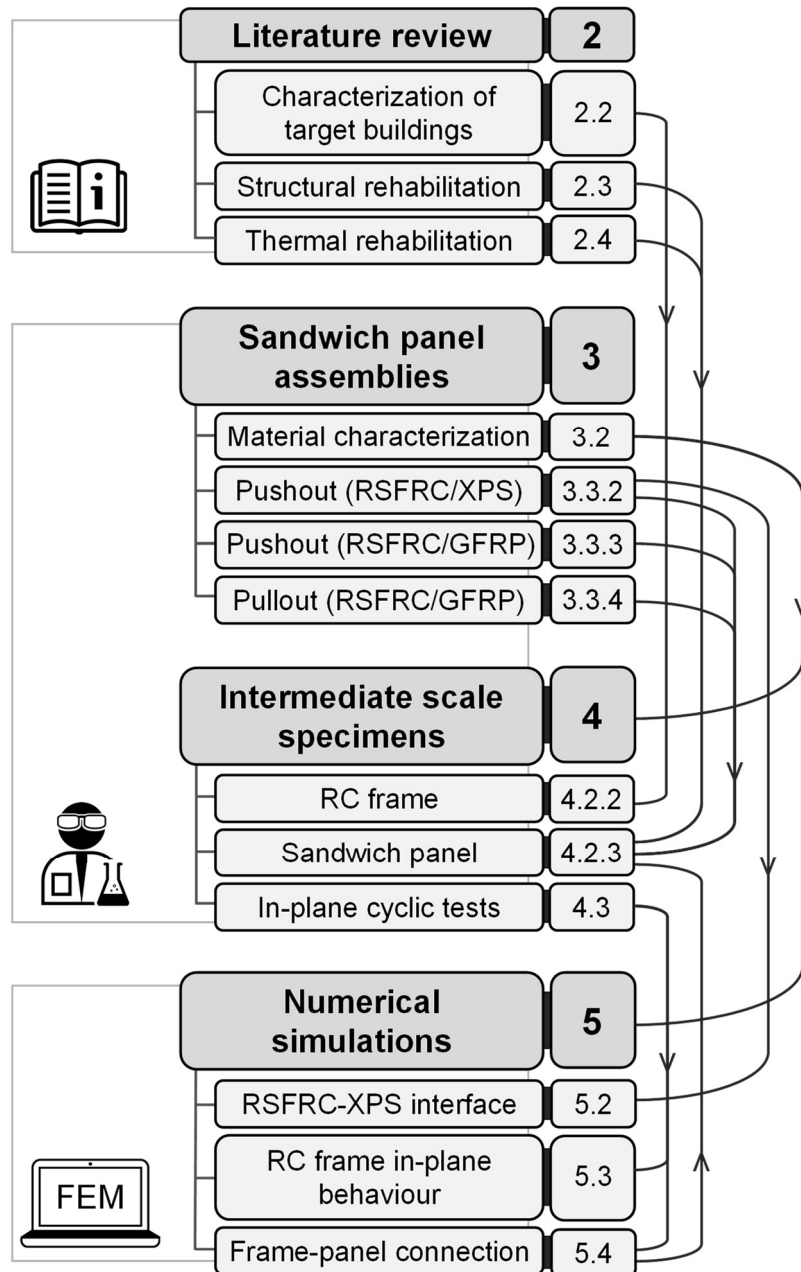


Figure 1.3: Research methodology: workflow structure and interconnection.

1.4 Thesis outline

The presentation of the scientific research carried out in this thesis is organized in six chapters. Chapter 1 is an introductory chapter that essentially describes the scope and motivation behind this research work for a better understanding of the goals set for this study. In addition, it lays out the research objectives and methodology, and the outline of this document.

Chapter 2 presents a literature review focused on the characterization of the target building typology. The main structural and thermal characteristics of these buildings are analysed, as well as their actual degradation condition. This chapter also includes the research work that was conducted to assess the state of the art concerning both structural and thermal rehabilitation solutions.

Chapter 3 presents the first part of the experimental programme that was conducted in the scope of this thesis. The tests that are described in this chapter include the material characterization of the individual components of the developed sandwich panel, and also the pushout and pullout tests that were performed on sandwich panel specimens to assist the selection of the insulation material and the structural GFRP connectors to be incorporated in the final sandwich panel configuration.

The second part of the experimental study is presented in chapter 4, which presents and discusses the results obtained by testing intermediate-scale RC frame specimens under cyclic loading conditions. A total of three RC frame specimens were tested: one bare RC frame, one RC frame with a masonry infill wall and one RC frame strengthened through the application of the developed sandwich panel prototype.

Chapter 5 describes the numerical study that was carried out alongside the design process of the rehabilitation solution and the conducted experimental program. The numerical study includes the modelling of: (i) the failure mechanisms that were observed experimentally, during the pushout tests between RSFRC and polystyrene layers; (ii) the experimental behaviour of an RC frame specimen (bare RC frame) subjected to lateral in-plane loading; and (iii) different design scenarios for the connection between the sandwich panel and the RC frame.

Finally, chapter 6 summarizes the main conclusions of this thesis, and presents prospects for future developments related to the performed research.

2 LITERATURE REVIEW

2.1 Introductory remarks

This chapter describes the literature review that was conducted in the scope of this research work. The first part of the review (section 2.2.2) focuses on the characterization of the built heritage, with specific analysis towards the degradation condition of the building typology that was targeted for this study, including the geometric characterization of the target buildings, as well as their limitations in terms of structural and thermal/acoustic performance. The second part and third part of the review address some of the main trends that are commonly adopted in the construction industry (rehabilitation sector) for the structural (section 2.2.3) and thermal rehabilitation (section 2.2.4) of buildings, respectively.

2.2 Characterization and degradation condition of multi-storey RC frame buildings (1960-80s)

2.2.1 Overview

As mentioned, concrete construction accounts for large portion of the Portuguese building stock. Moreover, in comparison with other construction typologies, RC buildings contain, on average, more dwellings (taller buildings are typically built with RC frame structures, as shown in Figure 2.1a) and host therefore approximately 60% of the national population. Among the different attributes considered in the Building Census survey of 2011 (INE, 2012), the year of construction plays a central role on any kind of retrofit attempt, not only because it is directly related to the level of deterioration of building structures, whose constituent materials naturally age as time goes by, but also because it dictates the level of design code requirements that were in use during the design process.

In Portugal, specifically regarding RC buildings, the nowadays commonly adopted structural configuration based on RC moment frames emerged during the 1940-60s (see schematic representation of the built environment evolution in Portugal in Figure 2.2). Later on, between the 1960 and 1990 decades, a significant rise on the construction of RC buildings followed, as shown in Figure 2.1.

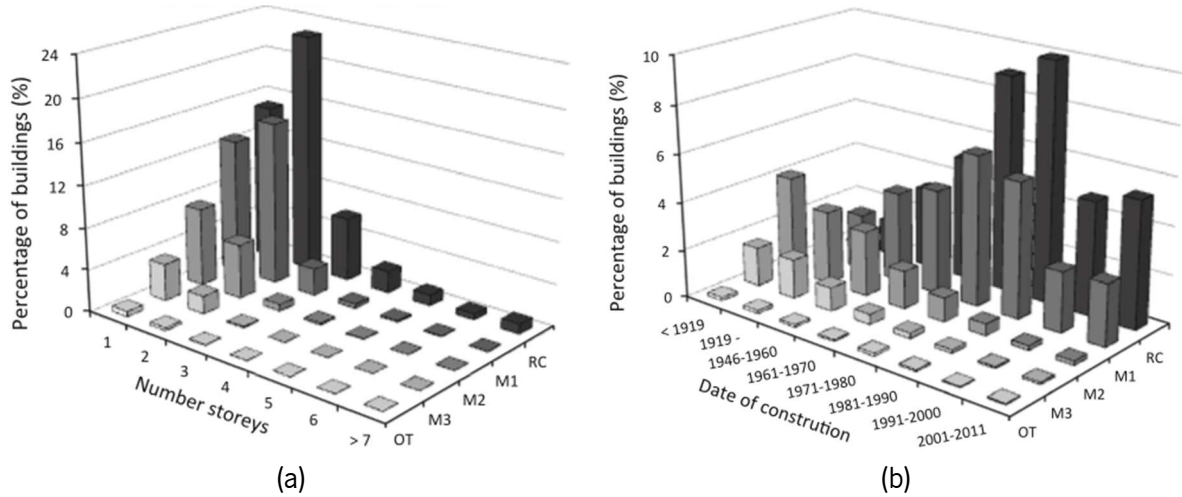


Figure 2.1: Distribution of buildings according to the type of construction [reinforced concrete (RC), masonry with concrete (M1) and timber floors (M2), weak masonry (M3) and other types (OT)]: (a) *vs.* number of storeys; (b) *vs.* period of construction (Silva *et al.*, 2015b).

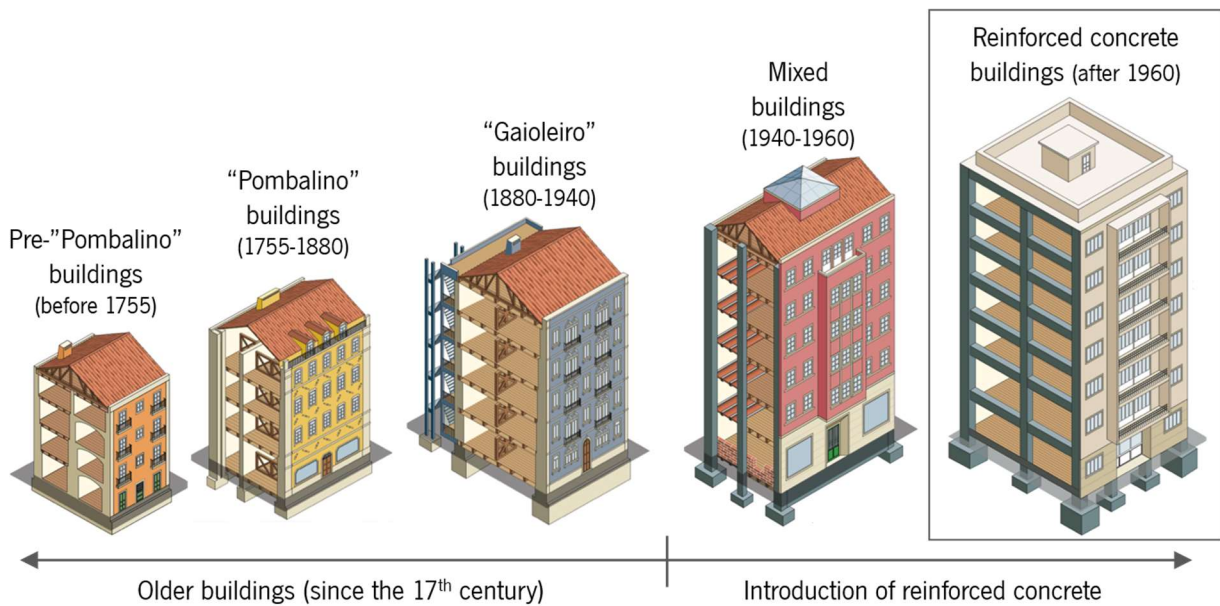


Figure 2.2: Adopted solutions for multi-storey building construction: evolution in Portugal (Expresso, 2009).

In fact, the first design codes that contained provisions regarding the consideration of seismic action date back to 1958 (RSCCS), with recommendations on how to consider in a very simplified manner the lateral loads caused by a seismic event, through the use of a seismic coefficient method (equivalent static analysis – with inclusion of initial zoning of the territory). Three years later, a revised version of the first design code was released (RSEP, 1961), with guidelines that included provisions regarding site effects (exacerbation of seismic coefficients on unfavourable ground for foundations). These recommendations were simplified and could not effectively guarantee an adequate seismic performance. Hence, in RC

buildings from this period, structural members were mostly designed for gravity loads, through simplified individual plane frame analysis.

Modern seismic design codes were only introduced in 1983, when a new and much more demanding design code was introduced (RSA, 1983), which already included dynamic analyses and provided a more objective introduction of the concept of ductility (with specific provisions and differentiation in detailing of RC structures). The RSA design code brought important improvements, in such a way that it is still being used nowadays by many design engineers, along with the modern regulatory framework that is being used today: the Eurocode 8 (CEN, 2010), which is applied together with Eurocode 2 (CEN, 2004) for designing RC structures. The Eurocode 8 brought a new set of seismic hazard zone maps for each type of seismic scenario (intraplate or interplate event), among other adjustments (Proença and Gago, 2011; Lamego, 2014; Silva *et al.*, 2015b).

It has been estimated that only 51% of the RC building stock, as of 2011 (INE, 2012), were designed after the introduction of the 1983 RSA design code, which implies that roughly 3 million of people live in structures that might not adequately withstand the effects of an eventual seismic event (Silva *et al.*, 2015b). In addition, these buildings are usually also associated to thermal deficiencies, which are discussed in further depth ahead in this chapter, as the use of single-layer or double-layer masonry walls with no addition of thermal insulation was a widespread solution adopted for the envelope walls: the introduction of thermal insulation systems in between the two layers of brick walls has only began in the 1980s, and this practice only became typical in Portuguese building construction during the 1990s. Hence, a significant percentage of this type of buildings also require thermal retrofit.

This alarming data, which is detailed here for the particular case of Portugal, but that can be extended to other seismic-prone south Mediterranean countries, such as Italy, south of France, Spain, Greece and Turkey, is the main reason that led to the definition of the target buildings defined for the present thesis: multi-storey RC moment frame buildings, designed before the introduction of the aforementioned 1983 RSA design code.

2.2.2 Main characteristics of target buildings

The building typology that was targeted as a pertinent case for the application of the rehabilitation solution to be developed in the scope of this thesis began to be used in Portugal during the 1950 decade. The late-1950s and early-1960s marked the fast transition to multi-storey structures made entirely of reinforced concrete (RC): these buildings are characterized by a structural configuration based on

foundations (footings), columns, beams and slabs, entirely made of RC, which are connected to each other forming RC moment frame structures.

RC shear walls were introduced later to the structural configuration of the buildings, usually by adopting central core walls, which often encase elevator shafts and/or staircases. These walls, which are typically coupled with the slabs and perimeter gravity columns that carry the gravity load, are regularly used as the main lateral load resisting systems in highly seismic zones, as they are very effective in providing lateral strength and stiffness to limit the structural drift, especially for tall buildings, by adopting centrally located RC core wall systems (Değer *et al.*, 2015).

During the 1960-80 decades, the columns and beams were essentially designed through simplified individual plane frame analyses. In these buildings, the moment resisting frames are typically arranged in one specific direction (see schematic representation in Figure 2.3a) and linked to each other through edge beams or lintels that act as bracing members (as depicted in Figure 2.3b, with the bracing elements illustrated in blue) and slabs, forming the final structural configuration presented in Figure 2.3c. An example of this building typology, with inclusion of shear walls, is illustrated in Figure 2.4a: the frames are all laid out in parallel, with the bracing elements being the beams (L1 and L2) and slabs (S1).

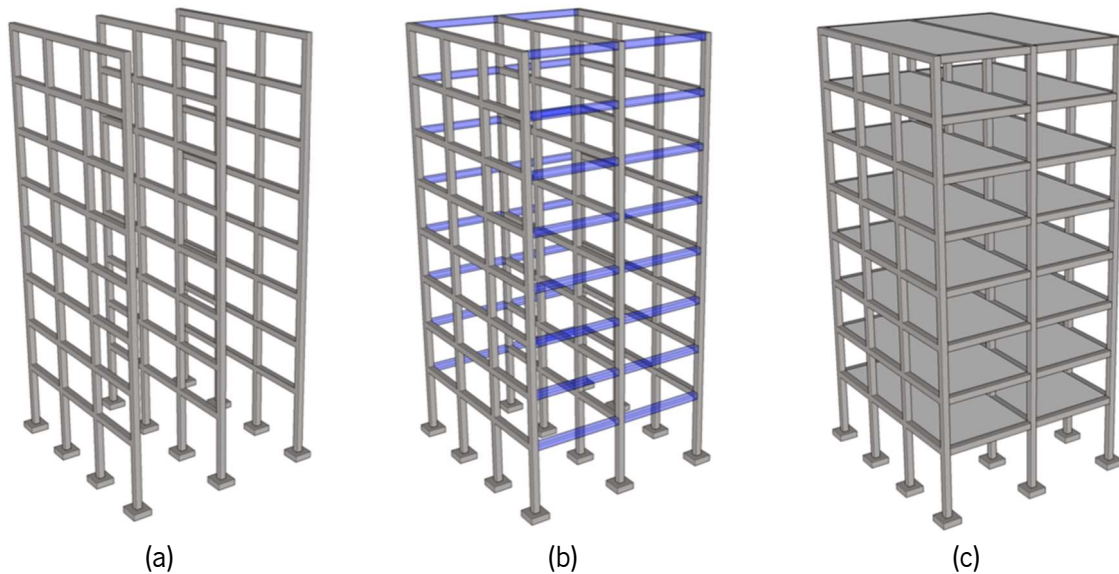


Figure 2.3: The target buildings' typical structural configuration (schematic representation):

- (a) RC moment frames arranged in parallel;
- (b) RC frames with the bracing members;
- (c) final RC structure.

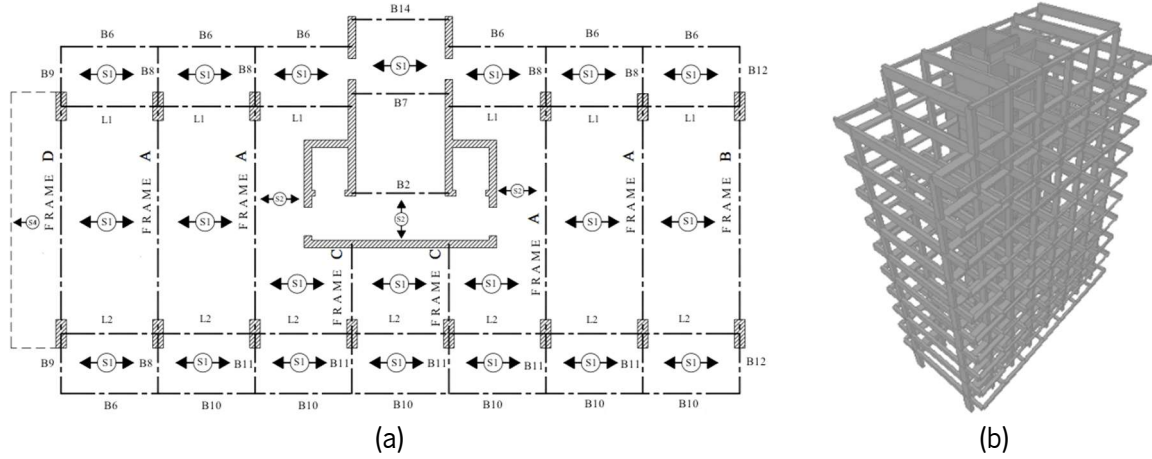


Figure 2.4: Example of a target building: (a) typical footprint for the storeys; (b) 3D representation of the RC structure (Lamego, 2014).

Regarding the material properties of the structural RC elements, these were usually built using concrete “B180”, “B225” or “B300” (REBA, 1967), which exhibit characteristic values of compressive strength of cube specimens at 28 days equal to 18.0, 22.5 and 30.0 MPa, respectively [corresponding roughly to the classes C16/20, C20/25 and C25/30 defined in the European standard EN 206-1 (CEN, 2007), respectively]. For the reinforcing bars (rebars), classes “A24” and “A40”, which were described in the REBA design code by a characteristic value of yield strength equal to 240 and 400 MPa, were the ones most commonly used. It is also worth mentioning that the vast majority of rebars had smooth finishing, as opposed to the ribbed rebars that are commonly used nowadays.

In terms of geometric properties, a minimum clear height between the floor and the ceiling of 2.7 m for private areas and a minimum clear height of 3.0 m for public areas, commercial spaces and offices was usually guaranteed, mostly due to the Portuguese regulation for urban buildings (RGEU, 2008), which was first established in 1951 (Silva *et al.*, 2015a). In fact, these buildings present, in some cases, differences in height between the ground storey and the remaining upper floors, essentially due to the need for wider spaces at the ground floor level for garages and/or commercial purposes. The difference between the height of the ground storey and the subsequent regular storey height, associated to the absence of claddings at the first floor, which is frequently only existent at the upper storeys (presence of masonry infill walls – see representative frame structure in Figure 2.5a), are defining characteristics of the so called “Pilotis” buildings (some examples are depicted in Figures 2.5b and 2.5c). Within the target buildings, the Pilotis configuration has been subject of extensive studies by the research community, namely in terms of structural/seismic retrofit, due to their seismic vulnerability (the structural behaviour of target buildings is subject of analysis in Section 2.2.3).



Figure 2.5: “Pilotis” buildings: (a) the corresponding RC frame structure (Bhatt, 2007); (b) example 1 (Shin *et al.*, 2014); (c) example 2 (Mezzi, 2006).

In specific regard to the columns, the cross section of these structural elements typically varies along the height of the building (for almost every storey), especially the column depth, for which there is in fact a high dependency of the building height: higher axial loads lead to larger sections, thus much thinner sections were adopted towards the top of the building. As for the column width, no relevant discrepancies are found between buildings with different number of storeys and between storeys of the same buildings, since it was assumed that these elements would not have to withstand any significant bending moment in the frame’s out-of-plane direction (due to individual plane frame analysis – the column width was established beforehand, within a range between 0.20 m and 0.40 m, and kept constant). The longitudinal reinforcement usually complies with the recommended minimum reinforcement area, with deficient confinement (Lamego, 2014; Silva *et al.*, 2015a) – see additional details in subsection 2.2.3.

For the beams, a general simplification was adopted by structural designers regarding the cross-sections’ configuration: there is a trend for only slight variations on the beams geometry used for the whole building, with repetition of beam dimensions and reinforcing ratios along the building, among different storeys and even for beams with different lengths. The longitudinal and transverse steel reinforcement are normally adequate for supporting vertical loads (designed according to worst-case scenario) and the shear or transverse reinforcement may, in many cases, exhibit an inclined configuration, at 45°, to resist shear forces (Lamego, 2014). Through the analysis of a vast collection of footprints (for comparison of RC moment-frame buildings constructed before and after 1983), Silva *et al.* (2015a) concluded that the beams’ width is usually more influenced by architectural requirements – determined by the thickness of the walls that are positioned in the same alignment – rather than code guidelines. Silva *et al.* (2015a) also concluded that there is a clear increase in the beam depth between the target buildings (built before 1983) and the post-code buildings (designed according to modern earthquake provisions): the adequate

consideration of the lateral loads due to seismic action originated higher bending moments in the beams and therefore a greater beam depth was needed to withstand such demands.

In the North and Centre of Portugal, the slabs used in this type of buildings are most often composed of pre-cast pre-stressed RC beams with clay hollow blocks and a cast-in-place concrete topping layer, which cannot be considered as solid diaphragms, especially taking into account the unidirectional disposition of the pre-cast pre-stressed RC beams; in those regions, this is still a rather common solution for RC buildings in general (see Figure 2.6). In the South of the country, the most commonly adopted slab configuration is the purely cast-in-place RC slab, which can also be found occasionally in floors with long spans in the previously referred North and Centre regions. In fact, in older buildings built before the 1970s, cast-in-place RC slabs were adopted for the whole structure, in most of the Portuguese continental territory. In general, the slabs are supported directly on the existing beams and present a total thickness that varies between 15 cm and 25 cm (Lamego, 2014; Silva *et al.*, 2015a).

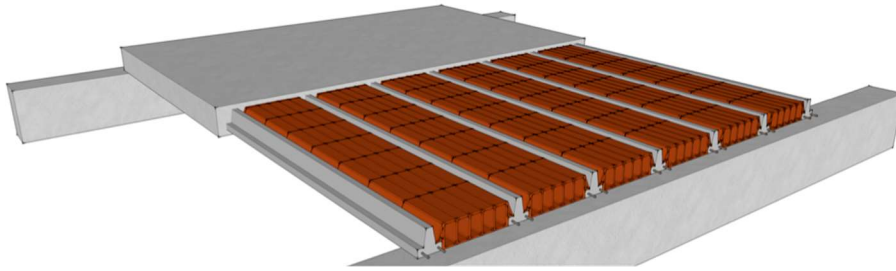


Figure 2.6: Representation of a pre-cast pre-stressed floor system typically adopted in Portugal.

In addition to the RC structure, non-structural elements are used for the interior and exterior walls of the buildings. The vast majority of target buildings include masonry walls (typically clay hollow bricks) to close the envelope and separate interior spaces as partition walls. For the envelope walls, which constitute the infill walls of the RC frames (see Figure 2.7a), the most conventional solution comprises double masonry walls (illustrated in Figure 2.7b) that consist of two layers of hollow clay bricks (in some cases, concrete masonry was also used instead of clay masonry) with a cavity (air layer) in between them. Typical dimensions for the two layers of masonry blocks are 15 cm and 11 cm of thickness for the exterior and interior masonry layers, respectively. Besides this 15+11 (cm) double layer configuration, other dimensions such as 15+15, 11+11, 20+7, 15+7 (cm) or even single skin masonry walls with 15, 22 or 25 cm can also be found in the exterior walls of this type of buildings. The cavity between the masonry brick layers serves mainly as drainage to protect the indoor areas from rainwater or moisture, which can be potentially absorbed by the exterior brick layer. For the interior walls of the buildings, a single leaf hollow brick wall is usually adopted, normally with a thickness of 11 cm. The final coating for the walls,

both in the interior and exterior faces, is usually composed by a thin layer of painted plaster (only in the interior) or render (the finishing layer has usually between 1 and 2 cm of thickness).

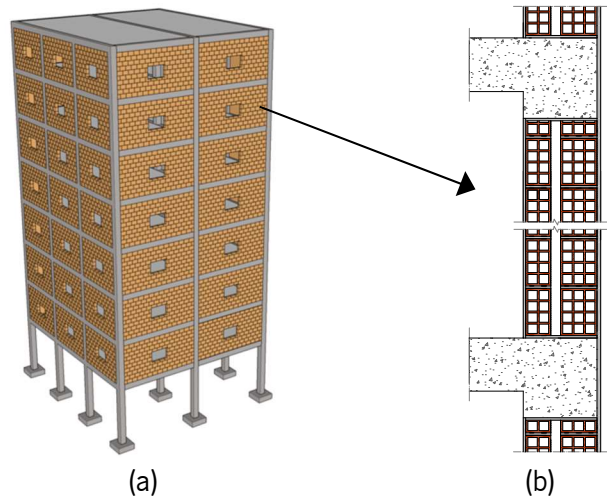


Figure 2.7: Traditional building envelope: (a) schematic representation of the RC frames with inclusion of the masonry infill walls; (b) cross section of a conventional solution (double brick cavity wall).

As noted before, the widespread introduction of thermal insulation into the cavity of double brick walls only began in the 1980s, becoming compulsory in building regulation only during the 1990s: the first Portuguese regulation on the thermal behaviour of buildings was implemented in 1991 (RCCTE, 1990). In addition, the implementation of this building regulation also induced the widespread use of double glazing in fenestration systems. Due to this circumstance, the great majority of target buildings were built with very deficient or absent façade and roof insulation and with fenestration glazing systems composed of only one layer of glass. As for the adoption of shading devices for protection against heating due to intense solar radiation, the windows are generally protected with exterior roller shutters, commonly made of plastic, as shown in Figure 2.8a (usual solution adopted since the 1970s). The use of roller shutters is, in some cases, complemented by the adoption of balconies at each storey level (see Figure 2.8b), which actually contributes as passive shading projection features: in fact, balconies or cantilever slabs provide a great deal of shading to the windows existent in the floor below (Bragança *et al.*, 2007).



Figure 2.8: Common shading devices: (a) plastic roller shutters; (b) shading provided by balconies.

2.2.3 Structural behaviour

2.2.3.1 Non-ductile reinforcing steel detailing

Research on earthquake engineering has clearly demonstrated that existing RC frame buildings designed before the introduction of modern seismic codes are the most vulnerable ones to severe damage or collapse when subjected to strong ground motion. Many of these structures are still in use, well beyond their design service life: many buildings constructed in high seismic risk regions have either collapsed or suffered considerable levels of damage during the past earthquake events (Ghobarah, 2000; ACI Committee 369, 2011; Görgülü *et al.*, 2012). During seismic events such as, for instance, the 1995 Hanshin-Awaji (Kobe), the 1999 Izmit (Turkey), the 2001 Gujarat (India), the 2010 Concepción (Chile), the 2011 Sendai (Japan) and the 2015 Kathmandu (Nepal) earthquakes, the behaviour of old RC buildings exhibited severe structural pathologies due to inadequate lateral load resistance capacity and limited ductility.

The vulnerability of older RC skeleton frame buildings is in part attributed to non-ductile detailing, especially common in structures built in Europe before the 1970s. The weaknesses in the structural system and deficiencies in reinforcing steel detailing have a strong influence on the potential damage and on the failure modes of the structural components during a seismic event. Common non-ductile reinforcement detailing used in these buildings is schematically illustrated in Figure 2.9 and can be summed up by the following characteristics and associated failure modes (Ghobarah, 2000; Bob, 2004; Görgülü *et al.*, 2012):

- Inadequate column flexural and shear capacity, which is due to scarce transverse reinforcement crossties in columns to provide shear resistance and confinement (see Figure 2.9, detail 1): shear reinforcement is not sufficient, with adoption of exaggerated spacing dimensions and simple configuration of stirrups (column ties often consisting of a single loop with a 90° hook and without any supplementary crossties or overlapping hoops). This promotes the occurrence of brittle shear failure mode (see Figure 2.10a). A relatively small percentage of flexural reinforcement at the end-part of the columns promotes the formation of plastic hinges with concrete crushing failure mode.
- Insufficient lap splice length in columns (lap splices of the column's main reinforcing bars normally located just above the floor level of each storey) and inadequate confinement of lap splice regions (splices not confined with closely spaced ties) – see detail 2 in Figure 2.9. These are locations of maximum bending moments and high stresses in the columns under lateral

loading, which may lead to anchorage failure of the spliced bars, causing concrete splitting and bond failure of the column lap splice (see Figure 2.10b).

- Insufficient joint shear resistance, due to scarce or absent transverse reinforcement in the beam-column joint regions (see Figure 2.9, detail 3), which can lead to premature joint shear failure (see Figure 2.10c - exterior joints are less confined and hence more susceptible to fail).
- Inadequate anchorage of the beams' positive reinforcement at the beam-column joints, due to termination of the beams' bottom bars at the face of the supporting column or short embedment of bars into the beam-column joint (see Figure 2.9, detail 4). Such lack of anchorage effect may lead to bond slip and consequent anchorage failure of the beams' positive (bottom) reinforcement at the beam-column joints (the pullout or bond slip effect can be perceived in Figure 2.10d).
- Insufficient beam shear resistance and deficient confinement at the beams' ends due to excessively spaced stirrups that do not avoid the occurrence of brittle shear failure in these regions (see Figure 2.9, detail 5).

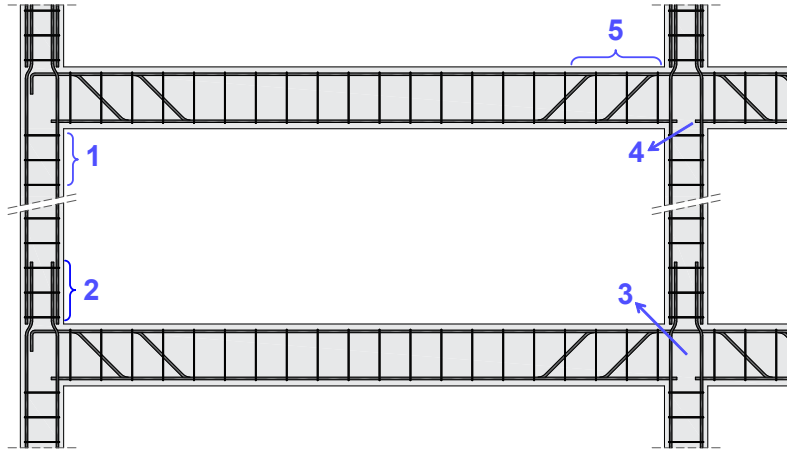


Figure 2.9: Typical reinforcement details in existing RC frame structures (REBA, 1967; Ghobarah, 2000).

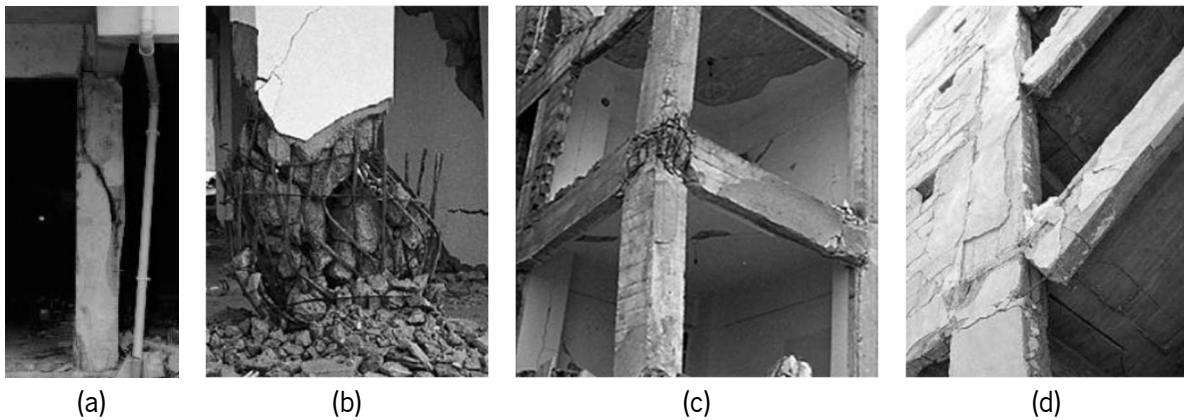


Figure 2.10: Failure modes due to non-ductile reinforcing steel detailing (Saatcioglu *et al.*, 2001; Sezen *et al.*, 2003): (a) column shear failure; (b) inadequate lap splices; (c) failure of beam-column joints; (d) anchorage failure of the beam's bottom reinforcement.

2.2.3.2 Weak-column/strong-beam configuration

Another typical shortcoming of older pre-code RC frame buildings is the weak-column/strong-beam structural configuration, which was the usual design practice at that time. Column failures are more frequent in these cases since the strength of beams in such constructions was kept higher than that of the columns – this is easily perceptible by the common use of thick beams in comparison to the concurrent thinner columns' sections. The strong-beam *vs.* weak-column configuration, associated to the gradual weakening of the column sections along the height, negatively affects the seismic behaviour: it induces heavy deformation demands on the columns and increasing storey drifts, forcing plastic hinging or yielding to occur in the vertical elements, especially at the ground storey level, as shown in Figure 2.11a. This configuration is opposed to the strong-column/weak-beam design philosophy that is inherent to modern design codes, which follows the principle of capacity design, where the objective is to maximize the number of plastic hinges (at the beam ends, rather than at the columns), as depicted in Figure 2.11b. Column damage resulting from strong-beam/weak-column connection can be seen in Figure 2.11c. The strong demand induced in the columns at the first storey level might gradually increase due to second-order effects, causing the formation of plastic hinges that lead to the collapse of the structure in a soft-storey failure mechanism that is particularly common in Pilotis buildings (also called “soft-storey” buildings, due to this effect), as illustrated in Figure 2.11d (Mezzi, 2006; Thermou and Elnashai, 2006; Antonopoulos and Anagnostopoulos, 2010, 2013).

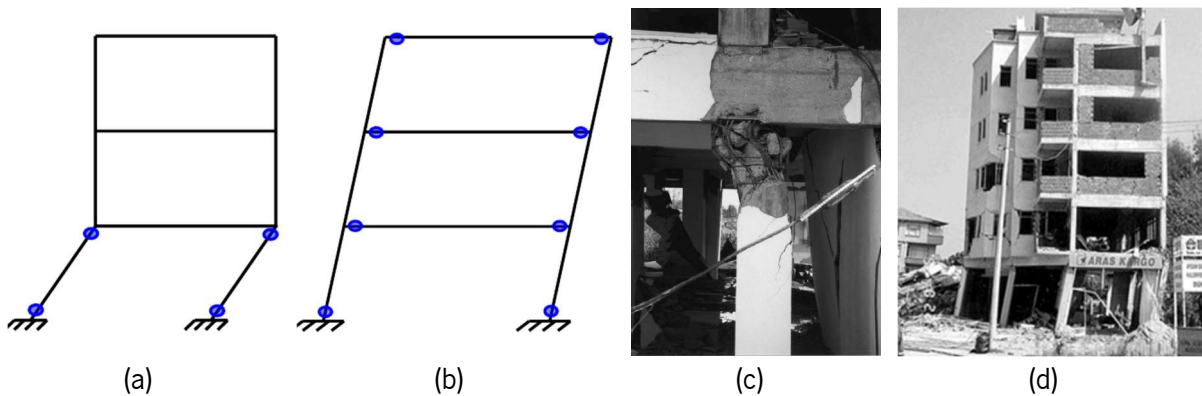


Figure 2.11: Typical weaknesses of target buildings: (a) strong-beam/weak-column concept; (b) strong-column/weak-beam concept (Marques, 2012); (c) column damage resulting from the concept illustrated in Figure 2.11a; (d) soft-storey failure mechanism (Saatcioglu *et al.*, 2001).

One can see that the gravity load designed structures have a tendency for behaving in a non-ductile manner with undesirable modes of failure. Usually, the existence of large shear walls attenuates this

problem, since they are the main lateral force resisting elements of the structure. However, taking into account that, in some cases, these elements are only lightly reinforced, the shear walls are also expected to be inadequate according to current requirements (Saatcioglu *et al.*, 2001; Baros *et al.*, 2009). An example of damage in beams and columns, caused during the 1999 Kocaeli earthquake, is shown in Figure 2.12a. Figure 2.12b illustrates a damaged shear wall constructed near elevators and stairwells: these walls were detailed similarly to regular moment-frame columns with light transverse reinforcement, adoption of 90° hooks and absence of complementary crossties (Sezen *et al.*, 2003).

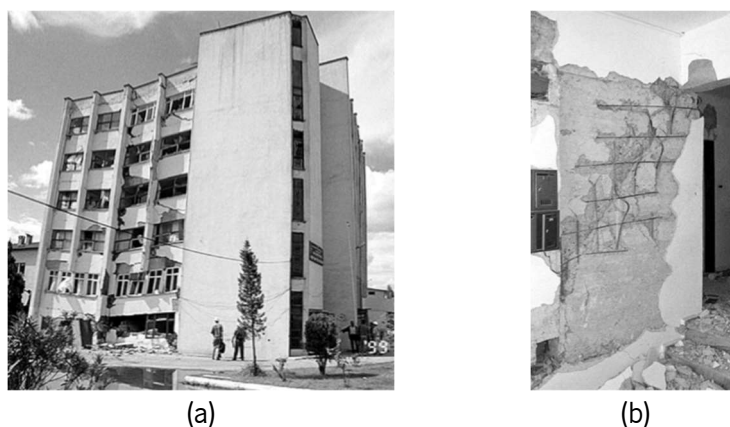


Figure 2.12: RC frame buildings with shear walls included in their structural system: (a) damaged wall frame building due to ground failure and wall rotation; (b) shear failure of an RC shear wall (Sezen *et al.*, 2003).

2.2.3.3 Assessment of seismic performance

The seismic performance of existing RC frame buildings can be determined using (and combining) different approaches, associated to different degrees of complexity and degrees of accuracy and reliability, such as: (i) structural inspection, which usually includes empirical and statistical methods; (ii) static pushover analysis, which is a simple but efficient approach based on the assumption that it is possible to relate the real structural response to that of an equivalent single-degree-of-freedom (SDOF) system; (iii) dynamic approach, which can also consider the material nonlinear behaviour and other time-dependent phenomena. In fact, the nonlinear static pushover analysis is probably the most commonly used tool among engineers and researchers for the evaluation of the seismic performance of a structure in terms of forces and displacements imposed by a given seismic event, as it can account for the redistribution of the internal forces developed within the nonlinear range of the structural behaviour. It is conducted through the use of 3D or simplified 2D finite element method (FEM) based models of the studied structural system, and consists of applying a monotonically increasing lateral load to the structure (namely in the

form of imposed displacement, as depicted in Figure 2.13a). The outcome resulting from this type of analysis is a convenient representation called “capacity curve”, which represent the relationship between the base shear reaction and the displacement at the roof storey level (see Figure 2.13b). The capacity curves are then converted to corresponding capacity spectra using an Acceleration-Displacement Response Spectra (ADRS) format and then overlapped with design code recommended demand spectra, as illustrated in Figure 2.13c (Bruno *et al.*, 2000; Ghobarah, 2000; Raju *et al.*, 2012): the performance point of a building under a given earthquake scenario is found from the intersection of an acceleration-displacement spectrum, which represents ground motion, and the building’s capacity curve (pushover curve converted to plots of spectral acceleration *vs.* displacement).

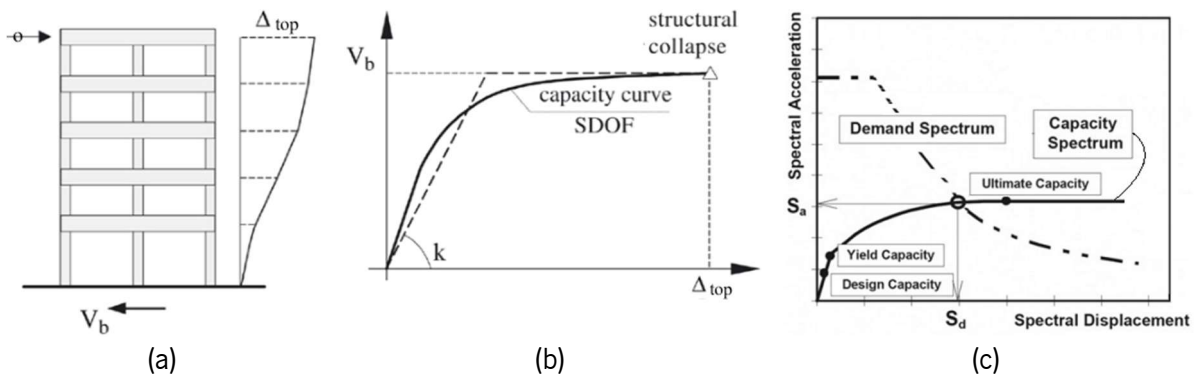


Figure 2.13: Static pushover analysis: a) boundary conditions; b) resulting capacity curve [adapted from Iervolino *et al.* (2007)]; c) capacity and demand spectra curves [adapted from Borzi *et al.* (2008)].

Pushover analyses on existing RC frame buildings have been performed in various research works during the last decade (Bruno *et al.*, 2000; Iervolino *et al.*, 2007; Rozman and Fajfar, 2009; Baros *et al.*, 2009; Raju *et al.*, 2012; Lamego, 2014); results obtained provide clear evidence of the seismic vulnerability of the target buildings. Rozman and Fajfar (2009), for instance, studied and compared the seismic response of different variants of 3-storey RC frame structures: a basic reference structure that represents design practice adopted before the development of seismic codes and four modified variants of structures designed completely or partially according to Eurocode 8. The obtained results were demonstrative of the low seismic resistance of buildings that were designed for merely gravity load (the capacity of the buildings representing pre-code or deficient seismic design practice was less than 1/3 of the capacity of buildings designed according to the Eurocode family of standards). Lamego (2014) performed nonlinear pushover analysis for the assessment of the seismic vulnerability of two typical cases of target buildings with soft-storey or Pilotis configuration (with 9 and 11 storeys) and inclusion of RC core shear walls around the elevator shafts and stairwells, for both case studies. The obtained estimate for the ultimate seismic

drift capacity of the buildings was lower than the seismic deformational demands considered according to modern design recommendations (CEN, 2010), namely for Type 1 spectra defined in Eurocode 8 (moderate to large magnitude earthquakes, representing highly seismic regions).

2.2.4 Thermal and acoustic behaviour

2.2.4.1 Thermal insulation

Taking into account that the transfer of energy (in this case, in the form of heat) between the building and its surrounding environment strongly affects the thermal performance of its indoor areas, much energy is wasted when a building is thermally inefficient due to the constant need for operating auxiliary heating, ventilation and air conditioning (HVAC) installations. This is typically the case of the building typology discussed herein, especially due to the absence of thermal insulation and the existence of thermal bridges in the building envelope: during extreme temperature conditions, the HVAC equipment is usually in permanent activity (when users can afford it), hence increasing the underlying energy costs (Carneiro *et al.*, 2014).

Figure 2.14 presents the evolution of the average values of the nominal space heating needs in Portugal (considering construction typologies with 2, 3, 4 and 5 bedrooms) plotted against the construction year of the building stock, which corresponds to one of the indicators used within the modern Portuguese thermal regulation codes (RCCTE, 2006) for the certification of the energy performance of buildings. By analysing the data evolution, one can see that the construction period corresponding to target buildings is associated to a continuous decrease of the nominal values. This continuous improvement of the thermal performance was first associated to the introduction of hollow bricks to complement or replace the traditional use of rubble stone masonry in building walls, followed, years later, by the adoption of air layers (cavities) between the brick leafs, which was at the time a very common option, especially for the RC frame building typology (Magalhães and Leal, 2014). However, despite the positive evolution, it is clear that the target buildings are far from complying with the thermal insulation performance specified in modern regulations. In fact, Figure 2.14 shows that an even more pronounced decrease of the heating needs of buildings takes place after the time period associated to target buildings, initially influenced by the implementation of the first legal instrument to impose minimum standards on the thermal quality of the building envelope (RCCTE, 1990) and afterwards strongly triggered by the implementation of the revised version of the Portuguese regulation (RCCTE, 2006).

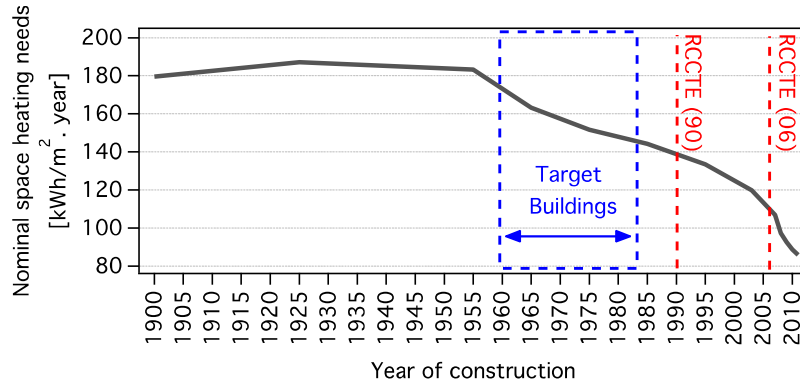


Figure 2.14: Evolution of the average nominal heating energy needs with construction period in Portugal [adapted from Magalhães and Leal (2014)].

Compared to the former version of the RCCTE regulation (1990), the upgraded version of RCCTE (2006) practically doubled the thermal comfort requirements for both new and renovated buildings (envisaging the duplication of envelope insulation thickness, in general); and the present Portuguese regulation for thermal behaviour (REH, 2013) tightened the mentioned requirements even more. Thermal comfort requirements demand that the exterior walls contribute towards ensuring comfortable indoor thermal conditions. The adjustments introduced for improvement of the buildings' behaviour during winter are reflected mainly on the reduction of the allowable maximum values and recommended reference values for the overall heat transfer coefficient (U-value) of walls and roofs, depending on the location of the construction (Sousa *et al.*, 2010).

Table 2.1 presents the U-values for different solutions adopted among target buildings and the maximum and reference U-values defined in modern regulation (REH, 2013) for areas of Portugal Mainland with most severe conditions during winter. By comparing those values, it is possible to verify that the U-values representing different masonry wall solutions are closer to the maximum admissible value than to the specified reference value, lying within the range of 0.86 and 1.90 W/m²°C. For example, in the case of the double masonry wall solution with 11+11 cm bricks (1.5 cm of render + 11 cm hollow brick + 5 cm air cavity + 11 cm hollow brick + 1.5 cm of plaster), which is undoubtedly a representative case, the corresponding U-value of 1.10 W/m²°C, although below the maximum level, is quite far from the recommended reference levels.

Table 2.1: U-values [$\text{W}/\text{m}^2\text{°C}$] for different types of envelope walls adopted in target buildings.

Typical masonry wall solutions	U	U _{max}	U _{ref}
Single clay brick wall (22 cm)	1.30		
Single concrete brick wall (22 cm)	1.90		
Double clay brick wall (11+11 cm)	1.10		
Double clay brick wall (11+15 cm)	0.96		
Double clay brick wall (15+15 cm)	0.86	1.40	0.40
Double concrete brick wall (11+11 cm)	1.40		
Double concrete brick wall (11+15 cm)	1.30		
Double concrete brick wall (15+15 cm)	1.30		
Double brick wall (11+11 cm) with thermal insulation (EPS, 3 cm)	0.68		

2.2.4.2 Thermal bridges

Another important shortcoming of target buildings in terms of thermal performance is the presence of thermal bridges in the building envelope, which have a strong impact on the energy consumption of buildings, namely by increasing winter heat losses and summer heat gains, leading also to building pathologies associated to condensation phenomena (Araújo *et al.*, 2013). Target buildings are particularly susceptible to the occurrence of thermal bridges, namely due to the existence of material heterogeneities in the building envelope, which is composed both by RC frame structural elements (columns, beams, and slabs, with lower thermal resistance) and infill walls (masonry walls, with higher thermal resistance). Although the total area of the building envelope is mainly composed by the masonry infill walls, the discontinuities caused by RC structural elements lead to a localized increase of thermal exchanges with the environment (heat flow lines tend to concentrate on these elements, as shown in Figures 2.15a and 2.15b), due to their higher thermal conductivity. Figures 2.15c and 2.15d present thermographic photographs of representative RC frame building envelopes, where the identification of areas corresponding to the thermal bridges is clearly evident, as they match the localization of RC structural elements within the building envelope: Figure 2.15c illustrates the heat losses during winter and Figure 2.15d is representative of the heat gains during summer. Previous studies (Valério, 2007; Theodosiou and Papadopoulos, 2008; Evola *et al.*, 2011; Araújo *et al.*, 2013) have shown that buildings designed according to modern thermal behaviour recommendations – namely after the release of the Energy Performance of Buildings Directive (European Commission, 2002), whose recommendations were transposed to the EU member states' legislation – present less thermal exchanges with the environment through thermal bridges, when compared to older buildings, mainly due to the fact that these thermal

regulations have a more accurate and strict approach towards envelope insulation configurations, as they take into consideration the undesirable thermal bridge effect.

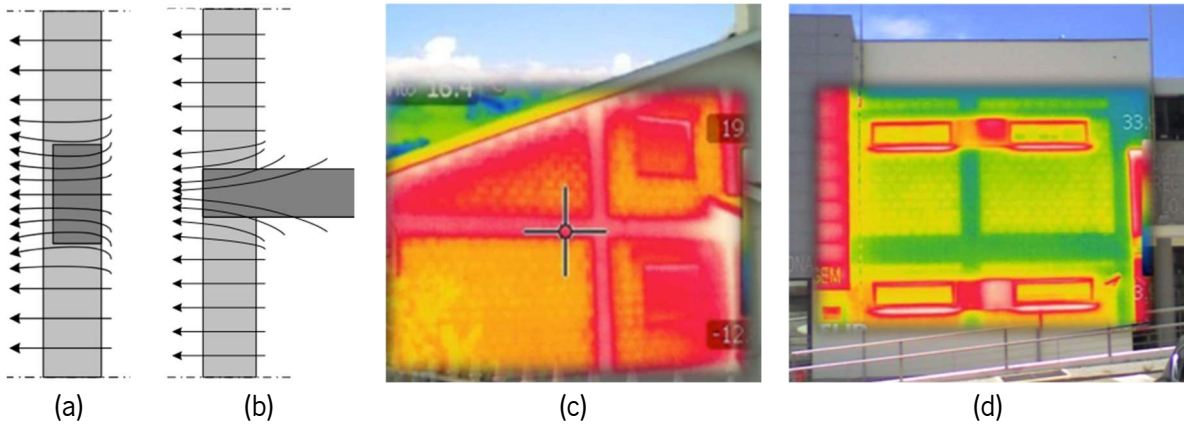


Figure 2.15: Thermal bridges in target buildings: heat flow direction in the presence of RC columns (a) and slabs (b) in the envelope's proximity (Valério, 2007); thermographic photographs during winter - heat losses (c) - and during summer - heat gains (d) - (Simões, 2014).

2.2.4.3 Sound insulation

In terms of sound insulation, acoustic comfort is a very persistent problem in Portugal, especially in older constructions, which are subject of frequent complaints among their inhabitants. Taking into account that buildings built before 1987 were not designed according to any kind of sound insulation regulation codes (the first version of the Portuguese noise code was implemented in that year), the building typology that is subject of investigation in the scope of this research is naturally also affected by this problem. Moreover, in the case of target buildings, this issue is more evident for the partition elements that are disposed horizontally (slabs). In fact, the majority of complaints are related to the noise (both airborne and impact sounds) originated in areas of the building that are above or below the complainant's dwelling, with more emphasis in the case of separation between commercial areas and residential dwellings. This issue is mainly due to an insufficient sound insulation level at the slabs (Bragança *et al.*, 2007). Complaints also arise for vertical partition elements, especially when the building is located in areas with high urban traffic noise, but these are considerably less frequent and dependent on the kind of outdoor noise source.

Figure 2.16 depicts the mandatory sound insulation levels for partition elements, defined by the Portuguese regulations on acoustic and noise requirements for new construction and rehabilitation works (RGR, 2006; RRAE, 2008; DRE, 2019a, 2019c), where $D_{2m,n,w}$ is the airborne sound insulation index of the façade, $D_{n,w}$ is the airborne sound insulation index for partitions and $L'_{n,w}$ is the impact sound

insulation index (particularly important for horizontal elements). It is possible to verify that, for façade elements (building elements relevant for this research work) the legislation defines a minimum airborne sound insulation index of 28 dB and 33 dB for sensitive zones (residential dwellings) and mixed zones (commercial areas), respectively. Additionally, taking into account that the majority of multi-storey RC frame buildings are located in consolidated urban areas (sensitive or mixed zone with steady urban occupation), the façade sound insulation should be reinforced by 3 dB. Hence, a sound insulation index of 36 dB should be guaranteed for façade elements in a worst-case scenario. The traditionally adopted single/double leaf hollow brick walls (solutions presented in Table 2.1) are characterized by a weighted sound reduction index that usually varies between approximately 45 dB (single 15 cm hollow brick wall) and 55 dB (double 15+15 cm hollow brick wall) (Ferreira, 2007; Mateus, 2008; Garg *et al.*, 2013), therefore leading to airborne sound insulation values that normally satisfy sound insulation requirements.

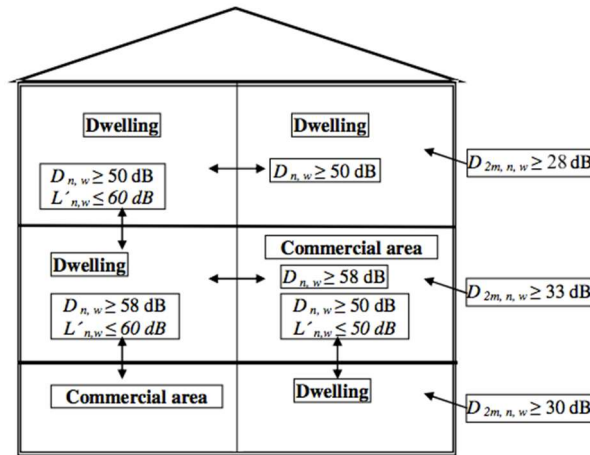


Figure 2.16: Acoustic requirements defined in (RRAE, 2008) [adapted from Bragança *et al.* (2007)].

The above-mentioned values corroborate that there is no critical necessity for an acoustic retrofit of the typical envelope wall solutions existent in the studied building typology. There is hence no need for special acoustic properties regarding the multifunctional façade panels to be developed. However, it should be noted that, depending on the rehabilitation/intervention approach, the acoustic properties of the panels might be more or less relevant: the panels can be added to the existing construction, without removal of the existing walls (in this case, sound insulation properties of the panel are normally not imperative) or integrated in the existing buildings by partial/total substitution of the existing infill walls (in this scenario, the minimum sound insulation requirements for the building façade should be guaranteed considering the sandwich panel solution alone). These aspects dealing with the detailing and execution of the strengthening system proposed herein are discussed in further depth ahead in section 4.2.3.

2.3 Structural rehabilitation of buildings

2.3.1 Overview

Seismic rehabilitation of existing buildings where new components are added and/or existing components are modified or retrofitted with new materials is a big contribution to the mitigation of risks regarding future earthquakes. In fact, seismic rehabilitation or seismic strengthening strategies are expected not only to reduce the damage level due to seismic occurrence, but also to extend the service life of existing buildings by using the minimum and proper materials and techniques towards safe and sustainability interventions (ACI Committee 369, 2011). Seismic retrofit techniques can be divided in two different approaches, depending on the type of seismic strengthening, which can be either a local or global intervention to the original structure.

2.3.2 Local intervention techniques

Local intervention techniques consist on the modification of elements that suffer from structural deficiencies in order to increase their load carrying capacity, so that the deficient elements will not reach their limit state conditions as the building responds at a required level when subjected to design loads (Jirsa, 2006; Thermou and Elnashai, 2006; Kaplan *et al.*, 2011). A great variety of element-based strengthening or repair techniques can be found in the literature, such as: (i) the injection of cracks for repairing damaged RC elements (see Figure 2.17a) with common use of resins for thinner cracks with up to 2-3 mm of width and cement grout for wider cracks, with up to 20 mm of width (Marthong *et al.*, 2012; Ahmad *et al.*, 2013; Duarte *et al.*, 2014); (ii) the use of externally bonded fibre reinforced polymers (FRP) through the application of fabrics (*in situ* cured systems) or laminates (pre-cured systems) on the surface of the elements to be strengthened (as shown in Figure 2.17b) through the use of epoxy adhesives (Barros *et al.*, 2007; Duarte *et al.*, 2014), and/or mechanical fixation/fastening (Coelho *et al.*, 2012; Correia *et al.*, 2015), and/or anchorage/pre-stressing systems (Sena-Cruz *et al.*, 2014; Correia, 2018); (iii) the insertion of FRP laminates into pre-cut slits opened on the concrete cover, which are bonded to the concrete elements through the use of epoxy adhesives (Barros *et al.*, 2008; Dias and Barros, 2012; Mofidi *et al.*, 2015); (iv) RC jacketing, which is performed by adding longitudinal and transverse reinforcement or a welded wire mesh that surrounds the original section (see Figure 2.17c), covering it with new cast-in-place concrete or (Thermou *et al.*, 2014) fibre reinforced concrete (Ruano *et al.*, 2014); (v) steel jackets, consisting of steel plates with L- or U-section configuration that are welded together in order to form hoops around the strengthened element, as depicted in Figure 2.17d, being bonded to the

RC elements through the use of cement or resin grouts (Sasmal *et al.*, 2011), and (vi) FRP wrapping or jacketing (see Figure 2.17e), through the application of confinement systems made of fibre-reinforced cement composite layers (Karayannis and Sirkelis, 2008; Sasmal *et al.*, 2011; Garcia *et al.*, 2013; Ribeiro *et al.*, 2018).

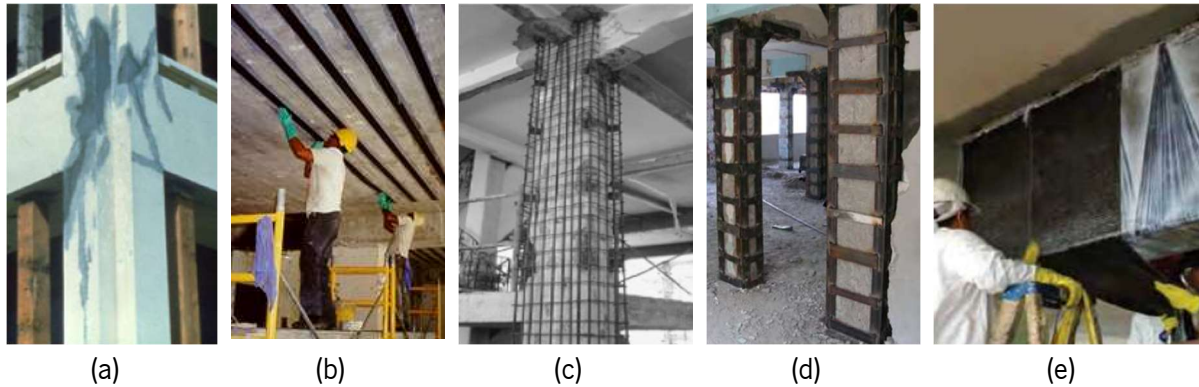


Figure 2.17: Local strengthening techniques: (a) injection of cracks (Thermou and Elnashai, 2006); (b) externally bonded FRP laminates (Buccola, 2011); (c) RC jacketing (Thermou and Elnashai, 2006); (d) steel jacketing (Kaplan and Yilmaz, 2012); (e) FRP jacketing (Kaplan and Yilmaz, 2012).

2.3.3 Global intervention techniques

In cases with several deficient structural elements, local intervention strategies can turn out as a too expensive and time consuming rehabilitation option, being a global intervention a possible efficient alternative. In the case of global strengthening schemes, the whole structure is usually strengthened for limiting lateral displacements (older RC structures are usually characterized as being highly flexible) in order to compensate its low ductility and hence, in these schemes, the entire behaviour of the structural system is transformed. Therefore, for the rehabilitation of an entire building structure, global strengthening schemes can be regarded as a more adequate (and economic) rehabilitation approach (Kaplan *et al.*, 2011).

One of the most commonly used global strengthening methods is based on the addition of new RC walls to the existing structure, usually by infilling the RC frames with RC shear walls (Frosch, 2006) or by adding external RC shear wall elements (Kaltakci *et al.*, 2008; Kaplan *et al.*, 2011), as shown in the illustrative representations depicted in Figures 2.18a and 2.18c, respectively. The adopted reinforcing systems can be cast in place (see Figure 2.18b) (Kaplan *et al.*, 2011) or be based on prefabricated RC elements (Frosch, 2006; Baran *et al.*, 2011; Baran and Aktas, 2013). The structural walls are linked to the existing structure through the use of different types of connectors for force transfer, among which it can be pointed

out the common use of steel connectors (for example steel rebar strips or bolts) that are epoxy grouted into holes drilled into the existing structure, therefore working as interface dowels. Due to the increase in lateral strength and stiffness caused by the addition of structural walls, this method yields satisfactory results by efficiently controlling the global lateral drift of the intervened building, which consequently reduces the damage in the structural elements, as demonstrated experimentally in various studies (Frosch, 2006; Kaltakci *et al.*, 2008; Kaplan *et al.*, 2011). Some shortcomings of this strengthening methodology are: (i) the increasing overturning moment caused by the additional weight that is added to the existing structure in case of thick RC shear walls, which may lead to the need for strengthening the foundation system (Thermou and Elnashai, 2006); (ii) and the increase of eccentricity produced by possible shifts in the centres of mass and stiffness of the building, which may induce undesirable torsional effects.

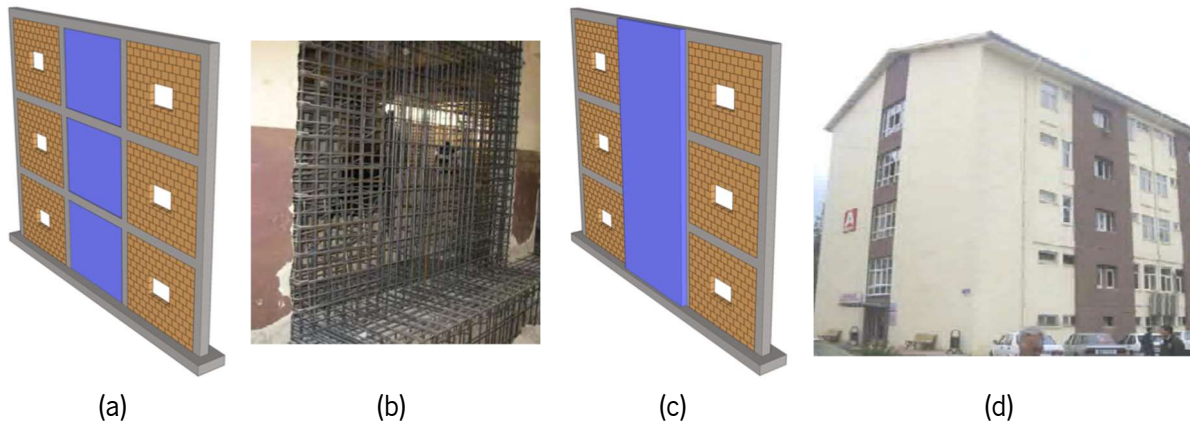


Figure 2.18: Addition of structural RC walls: infill walls (a,b) and external RC shear wall elements(c, d) (Kaplan and Yilmaz, 2012).

An alternative approach consists on the strengthening of RC structures through the use of steel braced shear walls or steel bracing techniques that, similarly to the RC wall case, can either be implemented inside the frame or applied externally. This type of strengthening approach presents the advantage of adding a minimal weight to the existing structure, when compared to the strengthening techniques based on RC elements, and the possibility of minimizing the disruption to the normal functioning of the building and its occupants during intervention. Research of various styles of bracing configurations (mainly in diagonal, “X” and “reverse V” form, as shown in Figures 2.19a, 2.19b and 2.20a, respectively) is available in the literature (Yu *et al.*, 2015), focusing on the experimental (Youssef *et al.*, 2007; Görgülü *et al.*, 2012) and numerical (Antonopoulos and Anagnostopoulos, 2010, 2012, 2013; Safarizki *et al.*, 2013) investigation of the seismic performance improvement of existing RC structures through the adoption of steel shear walls. Due to possible buckling effect on steel members, much research has also

been dedicated to the development of buckling-restrained brace (BRB) systems (Moghaddasi *et al.*, 2012; Shin *et al.*, 2012; Sutcu *et al.*, 2014), which are typically composed by a steel core and a restrainer (which prevents buckling of the steel elements, usually designed to resist tensile loading) in order to achieve stable behaviour under both tensile and compressive loads.

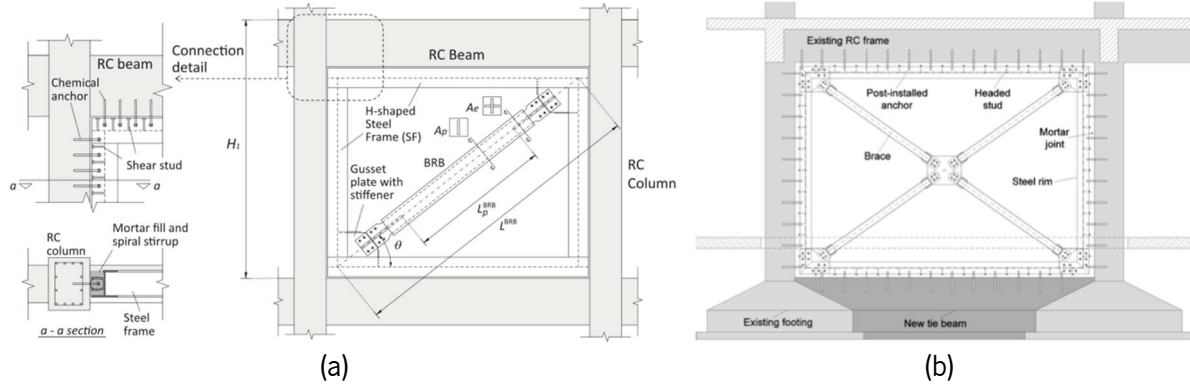


Figure 2.19: Steel braced shear infill walls: examples of (a) diagonal (Sutcu *et al.*, 2014) and (b) X-bracing system [adapted from (Antonopoulos and Anagnostopoulos, 2012)].

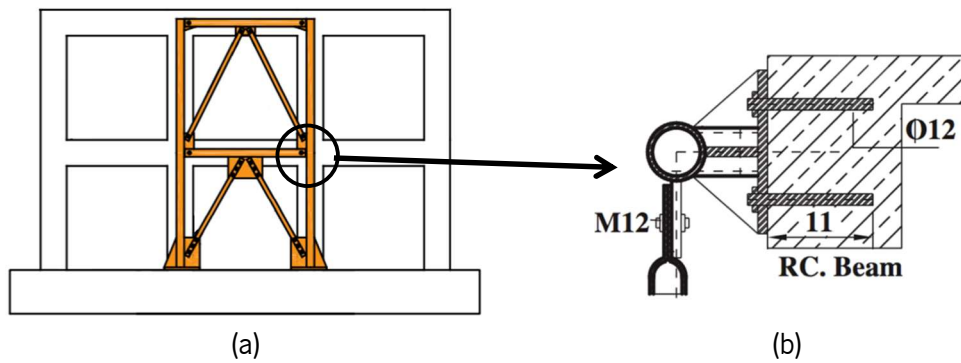


Figure 2.20: External steel shear wall proposed by Görgülü *et al.* (2012): (a) overall structural configuration; (b) connection detail between the bracing system and the existing structure.

Additionally, in some cases, bracing systems are integrated with base isolation systems, inserted at the bottom or at the top of the first storey columns, and/or equipped with energy dissipation devices, such as friction dampers or hydraulic/viscous dampers, for additional seismic protection (Di Sarno and Elnashai, 2005; Mezzi, 2006; Ribakov, 2010; Fakhouri and Igarashi, 2013). The basic objective is to concentrate/absorb as much energy as possible into specially designed devices, rather than to allow the total transfer of energy to the primary structural elements, therefore reducing the overall stress level and consequently the possible damage to the existing structure. For the generality of steel bracing systems, researchers report the achievement of an improved seismic performance (enhanced behaviour in terms of ductility and energy dissipation), while highlighting the importance of the connectors or dowels (usually

bolts and anchors, as shown in Figure 2.20b) that transfer the load to the existing RC frames, which directly influence the performance of the strengthening technique.

The distribution of both steel and RC shear walls in plan and elevation can differ from case to case, depending on the structural deficiencies of each building. Alternative configurations of bracing systems may be used in selected bays of RC frames to provide increase in lateral capacity of the structure. For example, in the specific case of the Pilotis-type building configuration, the strengthening is in many situations applied only to the ground level in order to keep costs to a minimum. A regular distribution of walls in height is however considered to be more appropriate since strengthening restricted to the ground storey will typically transfer the structural limitations to the storeys above (Thermou and Elnashai, 2006; Antonopoulos and Anagnostopoulos, 2010, 2012, 2013). Table 2.2 summarizes the local and global structural effects that result from the application of the global intervention techniques discussed above, together with some additional considerations related to the construction technology and the rehabilitation process.

Table 2.2: Effect of global intervention techniques on the rehabilitated building structure.

Retrofit scheme	Local effect	Global effect	Considerations
RC shear walls	Deformation demand is decreased; structural demand may increase in case of partial rehabilitation (irregular distribution of walls); high demand at connection between existing structure and RC walls.	Reduction of global lateral drift of the structure; increase of lateral strength and stiffness of the existing structure; resulting structural system has different dynamic response (may require full reassessment).	Importance of ensuring full interaction and load transfer between existing structure and new elements (through design/optimization of connectors at interface level); foundations may need additional strengthening or a change in foundation system ("L" shaped in the perimeter of the building for additional connection between walls and foundation elements).
Steel bracing systems	High stress levels may be generated in connections between bracing members and existing structure (at brace ends).	Increase in lateral strength and stiffness of the existing structure; achievement of enhanced behaviour in terms of ductility and energy dissipation.	Structural elements may need additional strengthening due to the high level of axial forces induced by the steel bracing system; foundation system may need additional strengthening to withstand increased strength and stiffness; special attention should be given to the connectors/dowels between braces and existing structure.
Energy dissipation devices	Reduction of seismic impact on both structural and non-structural elements.	The energy involved in a seismic event is dissipated by damping devices inserted at the bottom or top of the first storey columns.	Need for specialized engineering expertise; intervention at first storey level is sufficient (there is usually no need for structural rehabilitation of the upper part of the building's structure).

2.4 Thermal rehabilitation of buildings

2.4.1 Overview

In Portugal, the thermal renovation of existing buildings, regardless of the building typology and pathology, is traditionally addressed with partial solutions to solve the more immediate or apparent problems, such as air leaks, condensation (moisture problems) or thermal bridges. However, when facing major thermal problems (i.e. thermally inefficient envelope walls), this traditional approach is evidently not suited. In situations like these, a large-scale thermal rehabilitation project is needed with a comprehensive solution in order to reach an efficient and sustainable upgrade of the building's envelope (Costa, 2015). To tackle this problem, nowadays the thermal retrofit of buildings recurrently consists on the application of an additional layer of internal or external insulation, to the internal or external surfaces of the existing envelope walls.

2.4.2 Interior thermal insulation systems

The thermal rehabilitation using interior thermal insulation systems is a frequent rehabilitation technique in cases where the preservation of the original architectural layout of the building is a priority, or when the exterior façades of a building present complex geometrical configurations, or simply when there are geometric incompatibilities between the thickness of the insulation needed and that of protruding elements, such as window frames or sills. Different solutions can be found in building practice for the application of interior thermal insulation, such as prefabricated insulation panels (see Figure 2.21a) or interior façade systems applied on site. In Portugal, the latter solution is commonly executed by using gypsum plasterboard panels (detail 1 in Figure 2.21b), an insulation layer (detail 2 in Figure 2.21b) and a metallic support structure (detail 3 in Figure 2.21b), which is fixed to the existing wall, represented in Figure 2.21b with the number 4 (Guimarães, 2011). The main drawbacks associated to this type of rehabilitation approach are the impossibility of fully eliminating thermal bridges (due to inevitable discontinuities at wall-floor intersection, this solution is unable to provide continuous thermal insulation for the building envelope as a whole), the higher risk of water vapour condensation phenomenon in the thermal insulation layer, and the associated reduction of interior space (Pavlik and Černý, 2009). In addition, it has also been proven that, in comparison with exterior thermal insulation solutions, interior insulation systems are less effective on the damping of temperature fluctuations throughout the day (Sousa *et al.*, 2015).

2.4.3 External thermal insulation systems

Among the most commonly adopted thermal renovation solutions in Europe, with a widely increasing use in Portugal during the last decade (Amaro *et al.*, 2014), are External Thermal Insulation Composite Systems (ETICS). This rehabilitation solution is composed by a set of superimposed layers, which are directly applied to the existing envelope walls (detail 1 in Figure 2.21c) of intervened buildings: a thermal insulation layer – usually Expanded (EPS) or Extruded (XPS) Polystyrene (detail 2 in Figure 2.21c) – is fixed to the external building walls through an adhesive layer (detail 3 in Figure 2.21c) and/or by mechanical anchors (steel or plastic bolts and brackets – detail 4 in Figure 2.21c), over which a thin cementitious coating layer is applied (detail 5 in Figure 2.21c) incorporating a reinforcement (usually a glass fibre grid – detail 6 in Figure 2.21c) to provide additional strength, and a finishing decorative coating (plastic paint – detail 7 in Figure 2.21c) for aesthetical reasons. This thermal rehabilitation technique can be an effective solution, with successful energy efficiency improvement due to the elimination of thermal bridges and the addition of thermal insulation to the whole building envelope. However, its efficiency depends highly on the workmanship on site, which can be considered an important disadvantage when compared to prefabricated solutions. In fact, the incorrect interpretation of all the specifications and detailing contained in the rehabilitation project might lead to deficient or incorrect applications of this technique, and consequently to the occurrence of different anomalies (surface irregularities, visible joints between plates, oriented cracking, etc.), as reported by Amaro *et al.* (2013, 2014).

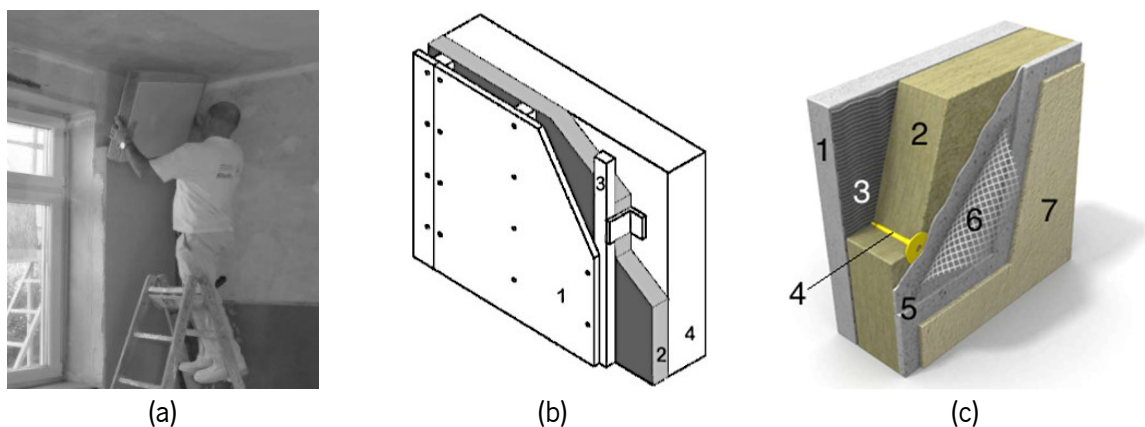


Figure 2.21: Interior thermal insulation systems: (a) prefabricated insulation panels (Guimarães, 2011); (b) insulation system applied on site, based on the application of gypsum plasterboard panels [adapted from Paiva *et al.* (2006)]; (c) ETICS [adapted from *PAROC Group* (2015)].

Prefabricated modular façades, usually associated to new construction of prefabricated modular housing systems, are an increasingly more common external thermal insulation system used for the

thermal/energetic refurbishment of the built environment, especially in northern and central European countries (Mjörnell, 2015). These systems are usually designed to fulfil high thermal requirements while using the advantages of prefabricated construction technologies – they are easier to install, provide better quality-control and usually already integrate all the necessary MEP (mechanical, electrical and plumbing) installations – for reduction of construction time and minimization of disturbances to the inhabitants on site, hence contributing to a more efficient rehabilitation process. Numerous solutions can be found in the market and in the literature, with integration of thermal insulation in façade systems based on different materials, such as wood (Lattke *et al.*, 2010; Lylykangas *et al.*, 2011), fibre reinforced cementitious materials (Pereira, 2013; Costa, 2015) and fibre reinforced polymers (Paiho *et al.*, 2015).

Within prefabricated façades, very relevant solutions for thermal rehabilitation of buildings have been developed that are able to transform incident sunlight into thermal energy for heating/cooling purposes, hot water supply and power generation (Quesada *et al.*, 2012; Lai and Hokoi, 2015; Zimmermann, 2012; Azpeitia *et al.*, 2015; Du *et al.*, 2019). It is also noteworthy to mention the increasing number of adaptive façades solutions (Azpeitia *et al.*, 2015; Arambatzis, 2020), for which the thermal properties of the panels will adapt to outdoor climate conditions through the use of regulation systems that integrate sensors/monitoring or phase change materials (PCM), which have the latent ability of storing solar energy during day time (PCM absorb heat from the surroundings and start to melt) and releasing gained heat during the night period (PCM start to solidify, releasing the stored heat to surroundings). These added heat storage capacities can lead to a reduction of thermal amplitudes, winter heat losses and summer heat gains (Kuznik *et al.*, 2011; Li *et al.*, 2019). PCM can be included, for instance, within lightweight aggregates of cementitious materials by modified vacuum impregnation technology (cement based binder is adjusted for optimal heat exchange to and from the lightweight aggregates) (Kastiukas *et al.*, 2016), encapsulated as additives in cementitious materials (Snoeck *et al.*, 2016) or embedded in concrete walls in the form of integrated PCM layers (Marani and Madhkhani, 2021).

2.4.4 Integrated structural/thermal rehabilitation

As presented so far in this section, the majority of retrofit techniques that can be found in the literature are either designed to increase the structural/seismic (subsection 2.3.1) or thermal performance (subsection 2.4) of the buildings at hand. Although, in many cases, there is separate consideration of the referred issues, integrated rehabilitation approaches for both structural/seismic and thermal rehabilitation of the existing housing stock are also starting to gain attraction in the scientific committee,

encouraged by the need for a more sustainable policy for building rehabilitation, taking into account that a large quantity of buildings present both structural and thermal deficiencies. In order to avoid the relocation of inhabitants and to minimize building downtime, some studies propose the use of double skin rehabilitation concepts (Marini *et al.*, 2017; Manfredi and Masi, 2018; Parsaeimaram *et al.*, 2018; D'Angola *et al.*, 2019). These involucres skins (see Figure 2.22a), composed by exterior RC frames infilled with properly insulated masonry walls can be an effective solution regarding improvement of thermal insulation performance, however, the improvement in structural performance is highly dependent on the connection system adopted to transfer loads between the old and the new structure. This connection can be guaranteed by using mechanical or adhesive metallic shear connectors/anchors.

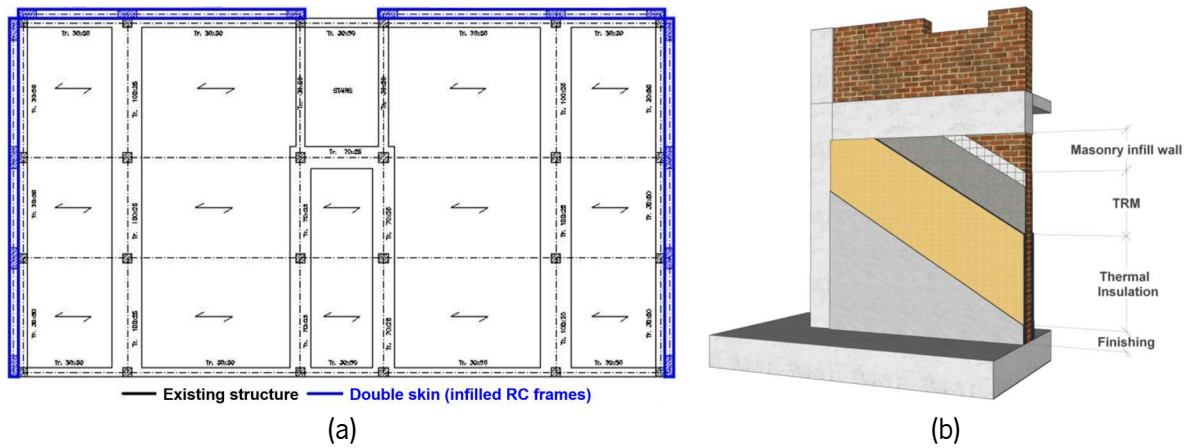


Figure 2.22: Integrated structural/thermal rehabilitation solutions: (a) double skin approach (Manfredi and Masi, 2018); (b) TRM jacketing approach (Pohoryles *et al.*, 2020).

Another approach considering rehabilitation of existing RC frame buildings from an holistic point of view is the application of jacketing techniques to the building envelopes, based on textile reinforced mortars (TRM) (Triantafillou *et al.*, 2017; Triantafillou *et al.*, 2018; Mistretta *et al.*, 2019; Gkournelos Panagiotis *et al.*, 2019; Pohoryles *et al.*, 2020) combined with thermal insulation. A schematic example of TRM jacketing is shown in Figure 2.22b. The seismic strengthening provided by this rehabilitation approach is usually provided by TRM composites made of high strength bi-directional textile structures (comprised of glass, carbon or basalt fibres), which are adhesively bonded to the building envelope by cement-based mortars. In order to improve the energy performance of buildings, the seismic reinforcement is integrated with a thermal insulation layer (e.g. polystyrene or wool). This system may present different variations: (i) jacketing can be placed either on the exterior side of the existing envelope walls or on both sides; and (ii) the TRM layer can be placed in the outer or inner side of the insulation layer.

2.5 Concluding remarks

The bibliographical review provided a deep dive into the target building typology, with specific mention of their main physical and geometric characteristics, pointing out their limitations regarding structural and thermal behaviour. It is possible to conclude that these buildings are characterized by a pattern of common structural and thermal rehabilitation needs, mainly related to their vulnerability under lateral loading conditions (e.g. seismic action) and insufficient thermal insulation leading to high energy consumption. The awareness of the underlying issues that led to the referred rehabilitation needs, and the identification of common trends of retrofit concepts that have been employed in the rehabilitation sector, point out the importance of adopting integrated structural/energetic rehabilitation strategies to effectively tackle building pathologies, which inspired the rehabilitation solution proposed herein, described in chapter 4.

3 MATERIAL CHARACTERIZATION AND SHEAR INTERACTION OF PANEL ASSEMBLIES

3.1 Introduction

Sandwich panels are often designed as multifunctional composite components due to their capacity to address various requirements, namely structural, thermal and acoustic. Such capacity stems partly from the insulation layer that separates the outer load-bearing layers. For insulated precast concrete sandwich panels, rigid foam insulation plates are frequently used, such as polystyrene plates, due to their lightweight and low thermal conductivity. More specifically, expanded polystyrene (EPS) and extruded polystyrene (XPS) are amongst the most common insulation materials. In terms of structural design, sandwich panels are usually categorized as fully, partially or non-composite, according to the degree of composite action between concrete wythes, which can be assessed by performing pushout tests designed to study the shear interaction between the referred wythes (Choi *et al.*, 2015; Choi *et al.*, 2019; Cox *et al.*, 2019; Gombeda *et al.*, 2020; Hodicky *et al.*, 2015; Huang *et al.*, 2020; Huang and Dai, 2019; Pahn, 2013; Tomlinson *et al.*, 2016). This shear interaction is known to be highly dependent on the connection between wythes, typically provided by shear connectors or solid concrete regions (Lameiras *et al.*, 2013; Tomlinson *et al.*, 2016). The available literature has shown that the insulation layer can also contribute to the panels' composite action depending on its mechanical and surface properties (Choi *et al.*, 2015; Frankl *et al.*, 2011). The relevance of this contribution depends mainly on the adhesion between the insulation and the surrounding concrete layers, which is addressed in the present research work.

Besides the thermal insulation enhancement provided to the façade and the consequent reduction of energy consumption, another relevant feature of the proposed rehabilitation solution, from an eco-efficiency point of view, is the use of recycled steel fibres as discrete reinforcement of the outer concrete layers of the sandwich panel. Nowadays, the waste management of used pneumatic tyres is still a major environmental challenge worldwide; both the reuse of tyres and the recovery of constituent materials are sustainable options for disposing of post-consumed tyres (Pilakoutas *et al.*, 2004). At the same time, in the field of civil engineering, due to a growing interest regarding innovative and recycled materials for sustainable building construction, research efforts have been made towards the incorporation in concrete of different by-products obtained from the recycling of waste tyres, namely granulated rubber (Aslani *et*

al., 2018; Marques *et al.*, 2013) and steel fibres (Lourenço *et al.*, 2018; Pilakoutas *et al.*, 2004). The addition of small fractions of recycled steel fibres (RSF) to the concrete during mixing is one example of such strategy, with research studies on this topic reporting favourable effects on concrete structures in terms of structural efficiency. In fact, it has been shown that such incorporation can decrease the brittleness of the concrete matrix in tension, enhancing its toughness and post-cracking resistance (Aiello *et al.*, 2009; Caggiano *et al.*, 2017; Centonze *et al.*, 2012; Zamanzadeh *et al.*, 2015). By using steel fibres resulting from the tyre recycling industry (Figure 3.1a), instead of industrial steel fibres (Figure 3.1b), significant environmental benefits can be retrieved, as was demonstrated by Frazão (Frazão, 2019) in a life cycle assessment.



Figure 3.1: Recycled (a) and industrial (b) steel fibres (Frazão *et al.*, 2020).

This chapter presents the first phase of the experiments that were conducted towards the development of the sandwich panel described in chapter 4 (see subsection 4.2.3 for additional information). For this stage of the development process, the experiments included: (i) material characterization tests of the different individual components of the sandwich panel - external RSFRC layers, GFRP connectors and polystyrene core layer; and (ii) pushout and pullout tests on sandwich panel specimens subjected to shear loads. The former tests, described in section 3.2, aimed at collecting relevant information about the properties of the constituent materials. The latter experiments, described in section 3.3, aimed at defining (i) the insulation material to be adopted in the sandwich panel, taking into account the influence that different insulation typologies (in terms of material and surface finishing) have on the overall composite behaviour of the panel, and (ii) the structural GFRP connectors (type and spacing) distributed along the sandwich panel.

3.2 Material characterization

3.2.1 Recycled steel fibre reinforced micro-concrete (RSFRC)

The final composition of the RSFRC, which was developed specifically for the production of thin concrete layers to be adopted as wythes of precast sandwich panels, is presented in Table 3.1. Considering the adopted thickness for the concrete layers of the studied sandwich panel, defined as 25 mm, a nominal maximum aggregate size of 9 mm was adopted, therefore justifying the “micro-concrete” designation that was attributed to the concrete composition.

Table 3.1: RSFRC mixture composition.

Component	Content [kg/m ³]
Cement (CEM I 42.5R)	400.0
Fly ash	200.0
Water	173.0
Crushed granite (9 mm)	597.2
River sand (4 mm)	734.7
Fine sand (0.6 mm)	147.5
Superplasticiser (BASF SKY 617)	7.2
Recycled steel fibres	70.0

The RSF used for this research were obtained through a commonly used tyre recycling technique that is applied to post-consumed truck tires, which combines mechanical shredding and granulation. Further processing of the RSF through an electromagnetic extraction system ensures the separation of the steel fibres from the rubber matrix. The fibres obtained from this process have irregular diameters, lengths and shapes. The average values that characterize the fibres used in this research, obtained from a thorough geometrical and mechanical characterization study involving a representative sample of approximately 2000 fibres (Frazão *et al.*, 2020), are as follows: length (l_f) of 20 mm (CoV = 41.6%), diameter (d_f) of 0.25 mm (CoV = 30.3%), aspect ratio ($\lambda_f = l_f / d_f$) of 110 (CoV = 39.9%). In terms of mechanical properties, the RSF present elastic modulus of 200 GPa and tensile strength of 2570 MPa (Hu *et al.*, 2018). An RSF volume content of 0.89% (70 kg/m³ of concrete) was added to the concrete mixture composition, constituting the only steel reinforcement of the concrete layers (*i.e.*, no conventional mesh or rebar steel reinforcement was adopted).

The modulus of elasticity and compressive strength of the RSFRC were assessed at the age of 28 days by testing cylinders with 150 mm of diameter and 300 mm of height, according to standards EN 12390-13 (CEN, 2013) and EN 12390-3 (CEN, 2009a), respectively. The average values obtained for the

modulus of elasticity and compressive strength were, respectively, 27.5 GPa (coefficient of variation, CoV, equal to 2.5%) and 50.3 MPa (CoV = 7.8%).

For the type and content of fibres, and taking into account the compressive strength of the matrix, it is expected that the RSFRC has a tensile strain softening nature (Frazão, 2019), whose tensile behaviour should be assessed by using notched specimens (Fédération Internationale du Béton, 2011). Therefore, the post-cracking behaviour of the developed RSFRC was assessed by direct tensile tests on notched dog-bone shaped specimens, also at the age of 28 days. The test setup, depicted in Figure 3.2a, includes a servo-controlled direct tensile testing machine with an actuator and load cell of 50 kN capacity, and two ribbed grips that secured the specimen on both ends. Additionally, four inductive proximity sensors (Schneider Electronics, 2018) were installed at the lateral, front and back faces of the specimens, near the notch tip, in order to measure the crack mouth opening displacement (CMOD), which was determined by the average value of the displacements registered by the referred sensors. The geometric configuration of the specimens, which had 30 mm of thickness, is presented in Figure 3.3Figure 3.2b.

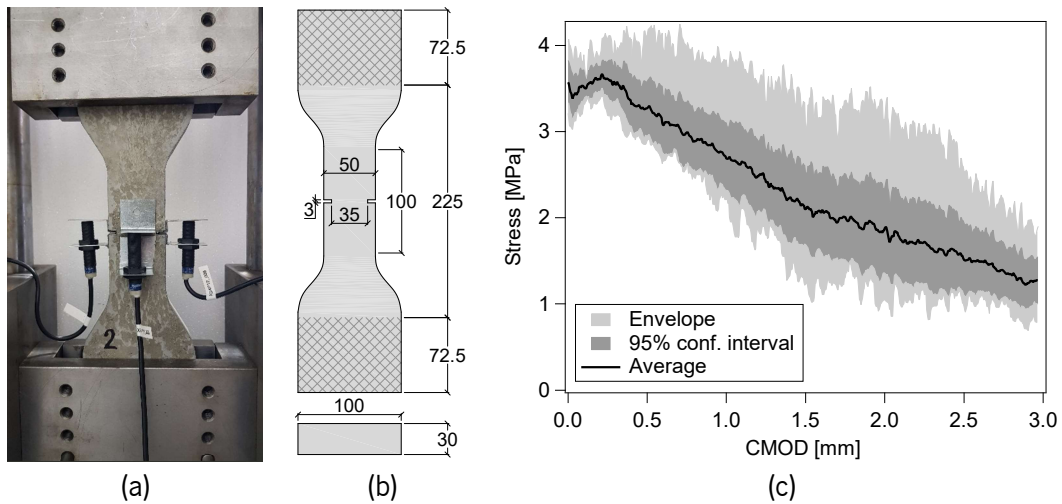


Figure 3.2: Evaluation of tensile properties of RSFRC: (a) Test setup; (b) Specimen geometry [dimensions in mm]; (c) Load *vs.* CMOD curves.

Since only one crack is expected to be formed in the notched plane, the results from this test can provide the fracture mode I parameters for the developed RSFRC. The tests were performed by imposing a displacement rate of 0.1 mm/min to the top grip of the equipment. A slow displacement rate was adopted to enable observing and capturing as accurately as possible the evolution of the CMOD, which was recorded continuously at a rate of 4 Hz. Figure 3.2c presents the envelope (with representation of confidence interval, corresponding to a confidence level equal to 95%) and corresponding average tensile stress *vs.* CMOD relationships obtained in these tests (8 specimens were tested, in total).

At crack initiation and at peak load, average tensile stresses of 3.7 MPa (CoV = 8.5%) and 3.9 MPa (CoV = 6.5%) were obtained, respectively. The post-cracking behaviour is characterized by residual tensile strengths of 3.3 MPa (CoV = 10.3%) and 1.6 MPa (CoV = 30.3%) at CMOD = 0.5 mm and CMOD = 2.5 mm, respectively, being these last two residual strength values used to establish the stress-crack width constitutive relationship for the design analysis at service and ultimate limit states according to the Model Code 2010 (Fédération Internationale du Béton, 2011). The obtained post-cracking ductility assumes particular importance for this material in light of its incorporation in seismic applications, as it can have a relevant contribution for increasing energy dissipation during a seismic event.

3.2.2 GFRP connectors

Two types of GFRP connectors were used during the experimental program. Both GFRP connectors were specifically designed for the production of sandwich panels in the precast concrete industry, namely to provide connection between outer concrete wythes and an inner insulation layer (for typical concrete/polystyrene/concrete sandwich panel configurations). Taking into account the thickness of the sandwich panel under development in the scope of this research project (*cf.* subsection 4.2.3), the adopted connectors had total length of 100 mm, compatible with a sandwich panel solution encompassing external concrete wythes of 25 mm of thickness each, and a 50 mm thick core insulation layer (see cross-sectional dimensions in Figure 3.3a). The first connector, hereby designated as “F-GFRP”, is a connector of functional nature that allows for higher quality control of the sandwich panels during the casting process (see Figure 3.3a, left-hand side). In fact, the connector has a protruding thicker section in the upper concrete/insulation layers’ transition, which acts as a gauge for an easier control of the position of the insulation layer and, consequently, of the thickness of the RSFRC layers. The second connector, designated as “S-GFRP”, is a structural connector, which consists of ribbed glass fibre reinforcing bars with the ends inclined at 30° (*cf.* Figure 3.3a, right-hand side).

Uniaxial tensile tests on GFRP connectors were performed on two universal test machines (UTM), with load capacities of 1000 kN (F-GFRP, Figure 3.3b) and 250 kN (S-GFRP). The axial stress *vs.* strain curves obtained are shown in Figure 3.3c. According to the data provided by the suppliers, the F-GFRP and S-GFRP connectors have tensile modulus of 10.1 GPa and 60.0 GPa, respectively, and tensile strength of 91 MPa and 1000 MPa, respectively. The average tensile modulus and tensile strength measured in the tests (six specimens for each type) were 10.4 GPa (CoV = 12.2%) and 96 MPa (CoV = 1.3%) for the F-GFRP connectors, and 58.4 GPa (CoV = 2.7%) and 1320 MPa (CoV = 12.5%) for the S-GFRP connectors

(see Figure 3.3c), respectively. These results are in agreement with (or exceed) the data provided by the suppliers of the F-GFRP (RIA-Polymers GmbH, 2012) and S-GFRP connectors (Schöck Bauteile GmbH, 2015). The properties obtained show that the S-GFRP connectors present (much) higher tensile strength than steel rebars and can therefore be regarded as an adequate solution in light of the multidisciplinary nature of the objectives set for this study, focused on both structural and thermal rehabilitation of the built patrimony: capacity of load transfer between layers of the sandwich panel without inclusion of metallic elements to avoid thermal bridges. The F-GFRP connectors have sufficient stiffness and strength for their required functionalities in the panel.

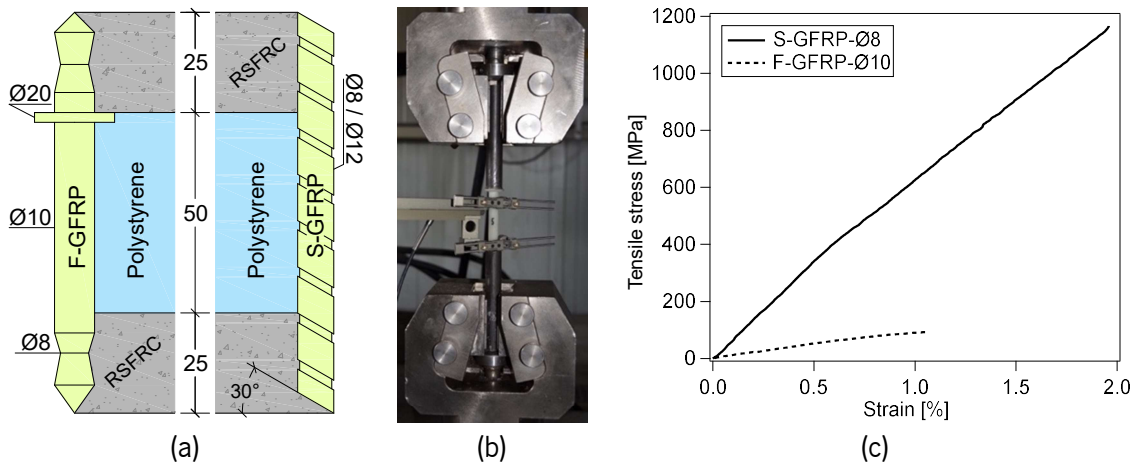


Figure 3.3: Characterization of GFRP connectors: (a) Representation of F-GFRP and S-GFRP connectors on sandwich cross-section [dimensions in mm]; (b) Test setup; (c) Axial stress vs. strain (average curves).

3.2.3 Polystyrene

To assess the influence of the interface between the RSFRC wythes and the core layer when subjected to shear forces, different types of polystyrene panels were considered for this experimental program. The main objective was to understand the dependence of the adhesion between RSFRC and polystyrene on the type of polystyrene material and its surface finishing. In view of this, the tests included five different types of commercially available polystyrene plates: (i) expanded polystyrene (EPS-1, Figure 3.4a); (ii) extruded polystyrene with smooth surface, with parallel notches cut on both sides, with spacing of 100 mm, depth of 12.5 mm and thickness of 3 mm (XPS-2, Figure 3.4b); (iii) extruded polystyrene with irregular/rough surface (XPS-3, Figure 3.4c); (iv) extruded polystyrene with diagonally ribbed wafer-like surface (XPS-4, Figure 3.4d); and (v) extruded polystyrene with smooth surface (XPS-5, Figure 3.4e). The material properties of the aforementioned polystyrene typologies, according to data provided by the

suppliers (Danosa, 2018; EPS - Empresa de Poliestireno Expandido, 2017; fibran, 2009), are detailed in Table 3.2.

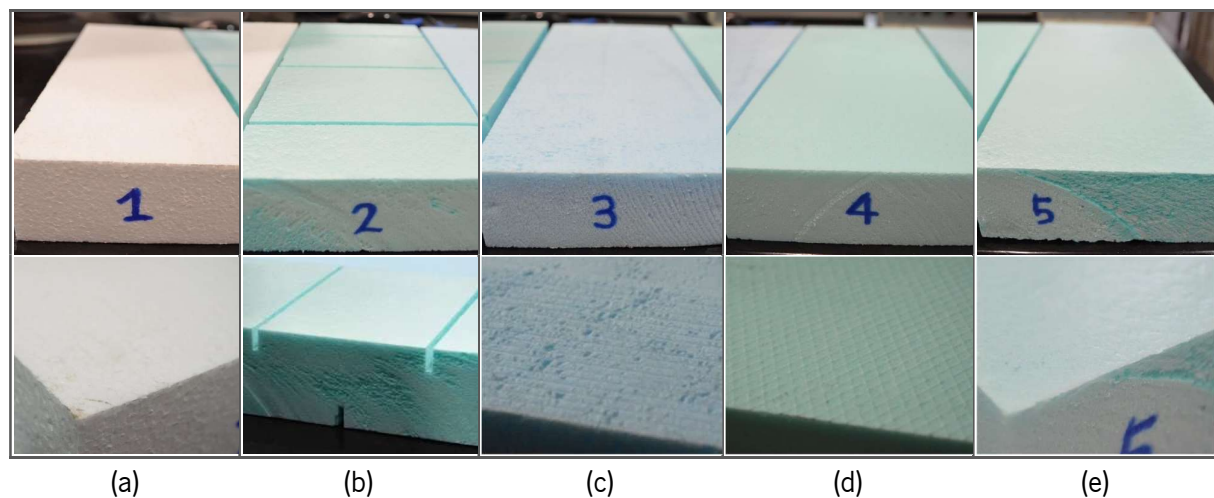


Figure 3.4: Studied polystyrene typologies: (a) EPS-1; (b) XPS-2; (c) XPS-3; (d) XPS-4; (e) XPS-5.

Table 3.2: Material properties of the tested polystyrene typologies (Danosa, 2018; EPS - Empresa de Poliestireno Expandido, 2017; fibran, 2009).

Material	Density [kg/m ³]	Compressive strength at 10% deformation [kPa]	Thermal conductivity [W/mK]
EPS-1	20	100	0.036
XPS-2	[30 – 33]	[200 – 300]	0.035
XPS-3	32	> 200	0.036
XPS-4	[30 – 33]	[200 – 300]	0.035
XPS-5	[30 – 33]	[200 – 300]	0.035

3.3 Pushout and pullout tests on sandwich panel assemblies

3.3.1 Overview

This Section describes the experimental program conducted to evaluate the structural behaviour of sandwich panel assemblies, when subjected to shear loading. The experiments comprised pushout and pullout tests, which were divided in two stages: (i) pushout tests for the selection of the insulation layer of the sandwich panel under development (Section 3.3.2), where different types of insulation layers were assessed; and (ii) pushout and pullout tests to assist the selection of structural GFRP connectors (Sections 3.3.3 and 3.3.4, respectively), where S-GFRP connectors with different diameters (8 and 12 mm) and embedment depths (25 and 35 mm) were evaluated. The experimental program included a total of 37 specimens divided in 14 series, whose specifications are summarized in Table 3.3.

Table 3.3: Specifications of the specimens subjected to pushout and pullout tests.

Stage of experiments	Specimen designation	N° of samples	Type of GFRP connector	Diameter [mm]	Embedment depth [mm]	Type of polystyrene
Selection of insulation layer (pushout)	FCØ10-H25	1	F-GFRP	10	25	-
	FCØ10-H25-EPS1	2				EPS-1
	FCØ10-H25-XPS2	2				XPS-2
	FCØ10-H25-XPS3	3				XPS-3
	FCØ10-H25-XPS4	3				XPS-4
	FCØ10-H25-XPS5	3				XPS-5
Selection of S-GFRP (pushout)	SCØ8-H25-XPS3	2	S-GFRP	8	25	XPS-3
	SCØ8-H35-XPS3	3			35	
	SCØ12-H25-XPS3	3		12	25	
	SCØ12-H35-XPS3	3			35	
Selection of S-GFRP (pullout)	SCØ8-H25	3		8	25	-
	SCØ8-H35	3			35	
	SCØ12-H25	3		12	25	
	SCØ12-H35	3			35	

3.3.2 Pushout tests for selection of core material

3.3.2.1 Experimental setup and procedure

For the pushout tests, $300 \times 300 \text{ mm}^2$ specimens representative of the investigated sandwich panel configuration were adopted. The geometry of the specimens is presented in Figure 3.5. It can be seen that the polystyrene layer does not cover the whole extent of the specimen plan area: it is placed only in a 150 mm wide central portion of the specimen. This is due to the need for free space in the peripheral areas of the specimen, in order to ensure proper support/anchoring of the bottom RSFRC layer to the test fixture. The test setup is shown in Figures 3.6a (front view) and 3.6b (top view), where the aforementioned anchoring system is illustrated: the bottom RSFRC layer was firmly fixed to a metallic frame with four steel plates (one near each corner), whereas the upper RSFRC layer was fixed to an actuator with 50 kN capacity, which was used to impose shear loading to the specimen (load was measured through a load cell with 25 kN capacity in tension). The tests were performed under displacement control at a rate of $5 \mu\text{m/s}$ (assessed by an LVDT with $\pm 0.25 \mu\text{m}$ precision, which measured the relative displacement between RSFRC wythes), as adopted elsewhere for this type of tests (Saari *et al.*, 1999; Shim *et al.*, 2010; Wang *et al.*, 2017).

In order to study the influence of the insulation layer on the composite behaviour of sandwich panel specimens, pushout tests were performed on a total of thirteen specimens containing different types of

polystyrene insulation layers, with two specimens being considered for the tests with EPS-1 and XPS-2, and three specimens being tested for each one of the remaining XPS typologies that were mentioned in Section 3.2.2.

In order to understand the local mechanisms that govern the interaction between RSFRC, polystyrene and F-GFRP connector, the pushout tests were performed using only one connector to study its individual contribution (as an alternative to using a test setup with more connectors, potentially more stable, but of more complexity to precisely interpret their individual contribution). Hence, for all specimens, an F-GFRP connector with 10 mm of diameter was placed vertically in the geometric centre of the specimen (see Figures 3.5 and 3.6a), bridging its constituent layers (*cf.* first part of Table 3.3 for additional information). An additional test was performed on a specimen without polystyrene layer (only RSFRC layers and S-GFRP connector). Due to the limited specimens available and the inherent instability of this specimen geometry (mitigated through the use of two small pieces of polystyrene, with no adherence to RSFRC), the results concerning this additional test are presented merely for the sake of comparison, as less information was retrieved, and to assist on the numerical simulations regarding the behaviour of the connector up to its failure.

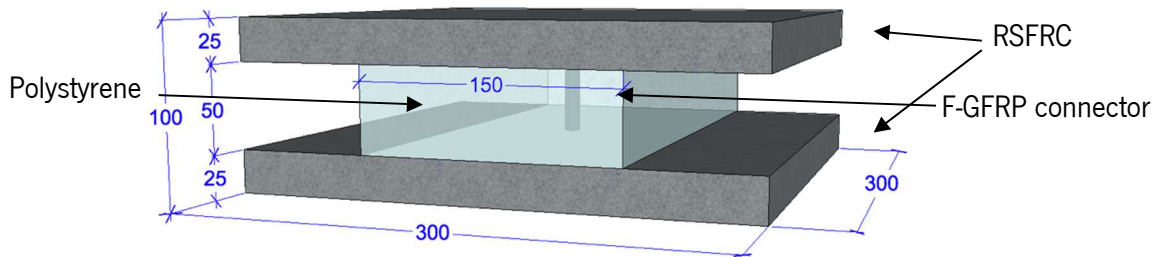


Figure 3.5: Specimen geometry adopted for pushout tests [dimensions in mm].

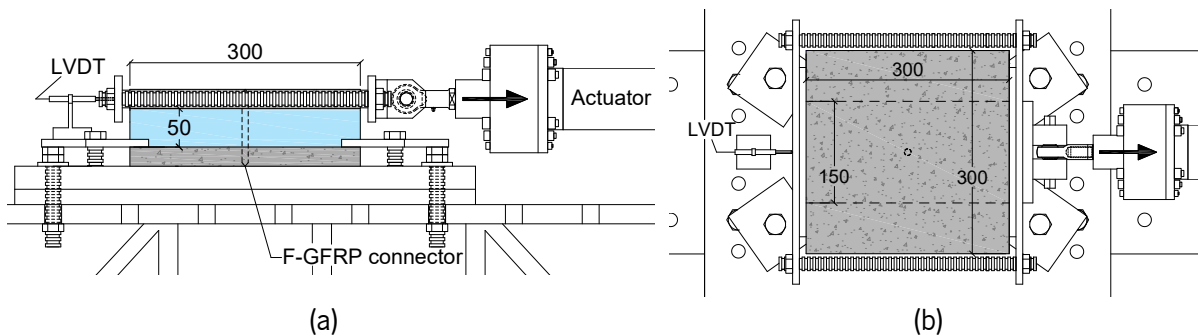


Figure 3.6: Experimental setup for pushout tests with F-GFRP connectors [dimensions in mm]:

(a) Front view; (b) Top view.

3.3.2.2 Results and failure modes

Figures 3.7a, 3.7b, 3.7c, 3.7d and 3.7e present the load *vs.* horizontal relative displacement curves obtained for specimens with polystyrene typologies EPS-1, XPS-2, XPS-3, XPS-4 and XPS-5, respectively. The relative displacement is the result of the transversal deformation of the F-GFRP connector and polystyrene layer, and of the interfacial slip (debonding) between polystyrene and RSFRC layers. The comparison between all the tests series is shown in Figure 3.7f, which depicts the average curves for each type of investigated sandwich panel configuration, including the result obtained from the specimen that did not contain a polystyrene core layer (FCØ10-H25). Figure 3.8 presents a collection of photos taken before and after testing all specimen typologies, complementing the data presented in Figure 3.7. Relevant results obtained for each test series, such as elastic stiffness (k_{el}), maximum load (F_{max}), relative displacement at peak load ($\delta_{F_{max}}$) and maximum relative displacement (δ_{max}), are also presented in Table 3.4. The maximum relative displacement, in this case, corresponds to the relative displacement for which full debonding between polystyrene and RSFRC is reached. It can be seen that specimens with core layer of type XPS-3 ensured the highest load carrying capacity until approximately 3 mm of lateral deformation, with an average peak load of 3.9 kN. The lowest structural performance was, as expected, provided by the specimen that did not contain a polystyrene layer (only 0.8 kN of maximum load), followed by specimens with insulation type XPS-5 (XPS with smooth surface finishing, for which a peak load of 1.3 kN was attained).

In general, the structural behaviour of the various sandwich panel assemblies when subjected to shear loading can be described by the following four main phases (identified for specimens XPS-4, as an example - see Figure 3.7d): (i) linear behaviour until approximately [0.5-1.5] mm of relative displacement between wythes; (ii) beginning of non-linearity triggered by stiffness reduction, then followed by a load decrease after initiation of the debonding process between the bottom RSFRC and its contacting polystyrene (the load decrease is steeper for polystyrene typologies that provide better adhesion to the RSFRC wythes, since this also ensured higher peak load); (iii) hardening phase in the post-peak stage, characterized by a gradual load increase due to friction at the RSFRC-polystyrene interfaces that takes place during the continuing debonding process; and (iv) softening phase, initiated after failure of the F-GFRP connector at the bottom RSFRC-polystyrene interface (see Figures 3.8c, 3.8f, 3.8i, 3.8l and 3.8o) that occurs between approximately 10 and 12 mm of relative displacement, until full debonding of the bottom RSFRC-polystyrene interface (see Figures 3.8b, 3.8e, 3.8h, 3.8k and 3.8n).

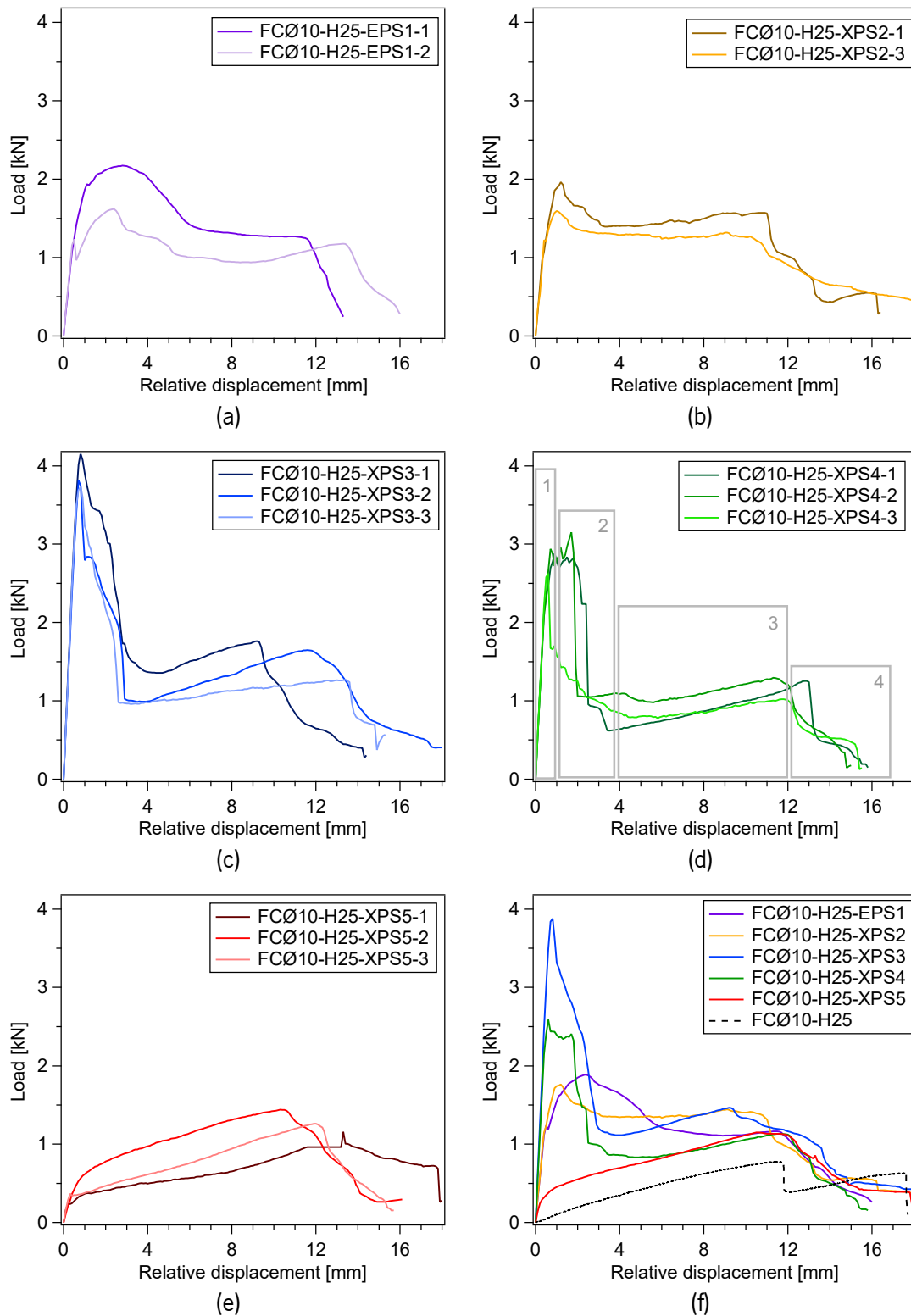


Figure 3.7: Pushout tests on specimens with different polystyrene typologies – load vs. relative displacement responses for: (a) EPS-1; (b) XPS-2; (c) XPS-3; (d) XPS-4; (e) XPS-5; (f) average curves.

Table 3.4: Pushout tests: Relevant results obtained for all specimen typologies (average values).

Specimen designation	k_{el} [kN/m]	F_{max} [kN]	δ_{Fmax} [mm]	δ_{max} [mm]
FCØ10-H25	99	0.8	11.7	17.7
FCØ10-H25-EPS1	2768	1.9	2.6	14.4
FCØ10-H25-XPS2	3413	1.8	1.3	15.6
FCØ10-H25-XPS3	6305	3.9	0.8	16.0
FCØ10-H25-XPS4	5623	2.9	1.1	15.4
FCØ10-H25-XPS5	1437	1.3	11.9	16.6
SCØ8-H25-XPS3	7265	6.7	14.8	37.2
SCØ8-H35-XPS3	8767	8.4	18.2	19.2
SCØ12-H25-XPS3	8870	7.7	3.4	37.9
SCØ12-H35-XPS3	13205	9.9	6.2	33.7

When analysing the differences obtained for the different specimen typologies, it can be seen that, for specimens with EPS only partial debonding occurred (in approximately half of the RSFRC-polystyrene interface area), with actual shear failure taking place in the polystyrene layer itself, near the interface, as illustrated in Figure 3.8b (a small portion of EPS remained attached to the bottom RSFRC layer). Through the analysis of all tested specimens it was verified that XPS-5 specimens presented the “cleanest” bottom polystyrene surface (perfect separation between RSFRC and XPS – see Figure 3.8n), which indicates that this polystyrene typology provided the smallest adhesion to the RSFRC. In fact, particular disparities were observed in case of XPS-5 when compared to the remaining typologies, for which the linear/non-linear transition loading state was reached at a significantly lower load (approximately 0.3 kN) and was followed by a hardening response (the load has increased with the relative displacement) in the above-mentioned second phase, rather than a softening response, observed in the tests of the other polystyrene systems. Also, for XPS-5, the maximum load was reached at the point of F-GFRP connector failure (already described in this section), corresponding to approximately 12 mm of relative displacement. This behaviour has similarities with the specimen that was tested without polystyrene layer (see Figure 3.7f and Table 3.4).

It is worth highlighting that among the two most commonly used polystyrene solutions, namely specimens EPS-1 and XPS-5, EPS provided significantly higher shear strength to the composite solution (+ 50%), which may be justified by the higher adhesion between EPS (*vs.* XPS with smooth surface finishing) and RSFRC. Overall, based on the obtained results (best performance until the end of phase 2 was presented by specimens XPS-3, XPS-4 and EPS-1), irregular surface finishing proved to be a relevant parameter to be taken into consideration for the selection of the polystyrene typology. Further investigations comprising the detailed characterization of surface roughness should focus on optimizing the surface pattern of polystyrene plates. Parametric studies with the model described in section 5.2 will allow to optimize the bond conditions between

core materials and RSFRC layers in terms of target performing indexes.

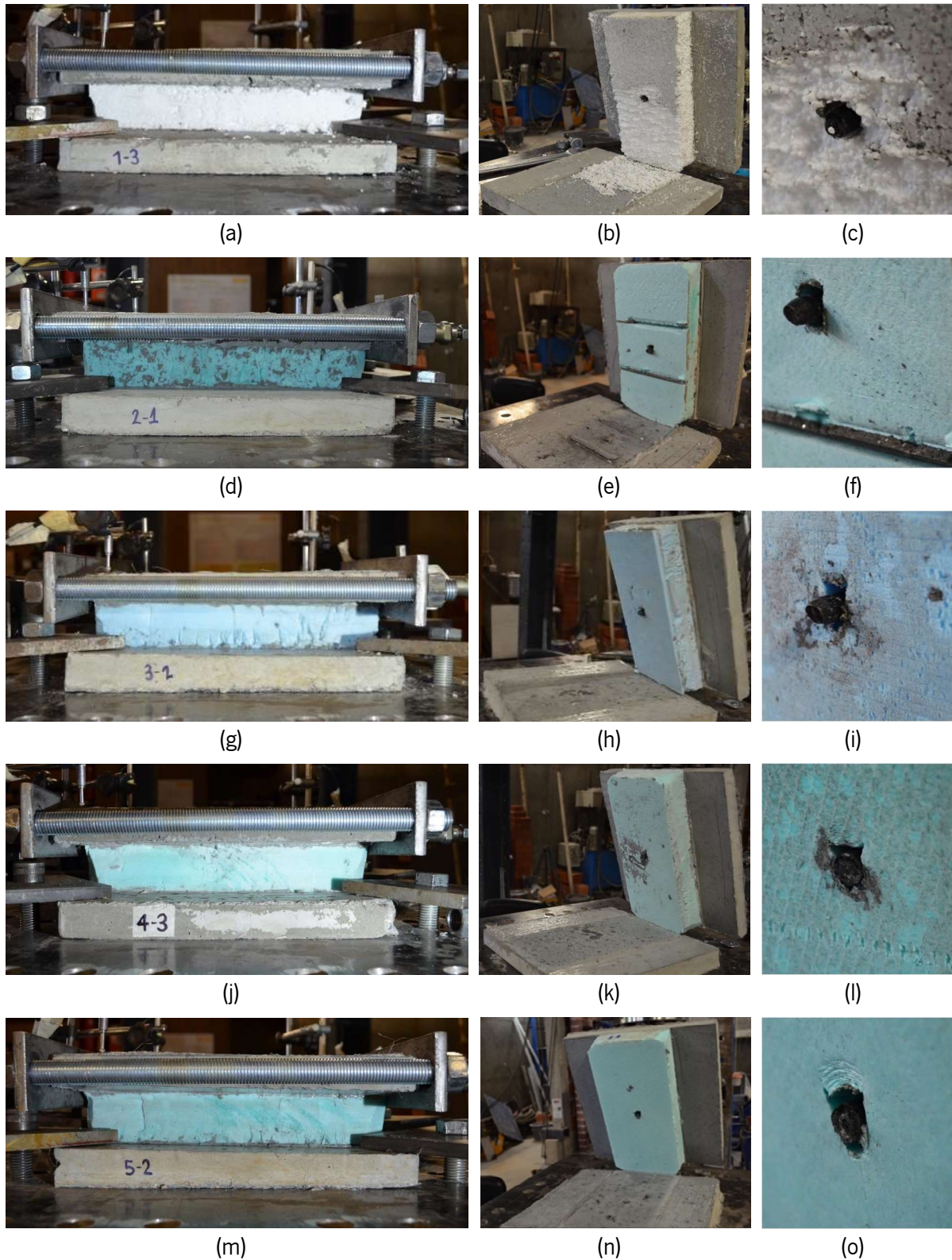


Figure 3.8: Pushout tests for selection of insulation layer. Specimens before (left) and after testing (centre and right): (a, b, c) EPS-1; (d, e, f) XPS-2; (g, h, i) XPS-3; (j, k, l) XPS-4; (m, n, o) XPS-5.

3.3.3 Pushout tests for selection of structural GFRP connectors

3.3.3.1 Experimental setup and procedure

The overall geometry of the specimens (see Figure 3.5) and experimental setup adopted for the second stage of pushout tests were equivalent to those used in the first one (*cf.* Section 3.3.2.1), with just a slight difference regarding the test setup: the anchoring of the bottom RSFRC layer was achieved using two 560 mm long steel plates (which cover the two longitudinal borders of the specimens – see Figure 3.9), rather than one steel plate at each corner. The tests were also conducted under displacement control, at a rate of 5 $\mu\text{m/s}$, up to 20 mm; thereafter, the speed was increased to 10 $\mu\text{m/s}$ until specimen failure.

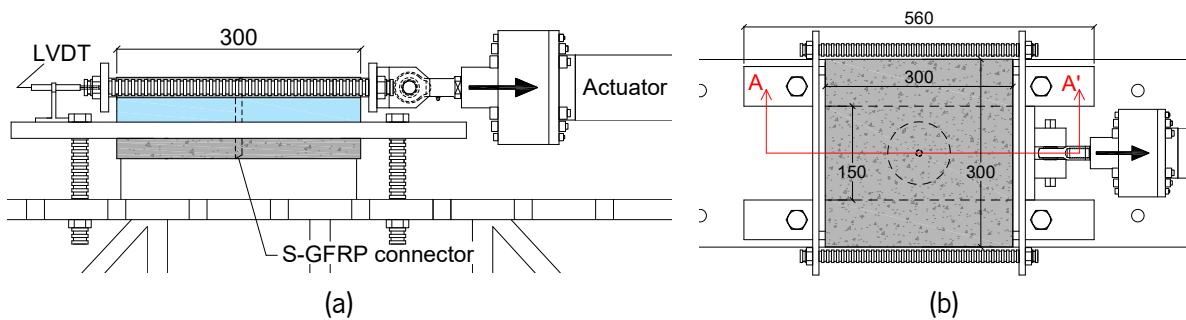


Figure 3.9: Experimental setup for pushout tests with different S-GFRP connectors [dimensions in mm]:
(a) Front view; (b) Top view.

Compared to the first series of tests, the second series of pushout tests were performed with focus on the enhancement of structural performance provided by the use of S-GFRP connectors. Hence, and as shown in the second part of Table 3.3, the same type of polystyrene typology was adopted for all specimens, namely XPS-3 (the one that provided the highest load carrying capacity – see Figure 3.7f), while varying the diameter (8 mm and 12 mm) and embedment depth (25 mm and 35 mm) of the S-GFRP connectors. The four specimen typologies considered for this study are depicted in Figure 3.10, which provides, for each specimen typology, a 2D representation of Section A-A' (indicated in Figure 3.9b) and a 3D illustration (front view of the specimen). Note that to assure an embedment length of 35 mm to the connectors, the RSFRC layers are locally thicker in the vicinity of the connector (visible in Figures 3.1b and 3.1d). This geometrical detail can potentially increase the local frictional resistance provided by the XPS layer.

The shear interaction performance of sandwich panels using structural GFRP connectors with different geometrical configurations and arrangements has been widely studied in the literature by means of pushout tests (Choi *et al.*, 2015; Choi *et al.*, 2019; Cox *et al.*, 2019; Flansbjerg *et al.*, 2016; Huang *et al.*,

2020; Pahn, 2013). However, the majority of studies report concrete wythes of plain concrete reinforced with steel rebars or meshes. To the author's best knowledge, no studies have been performed where the sandwich panel concept involves GFRP connectors in combination with thin concrete wythes reinforced with recycled steel fibres.

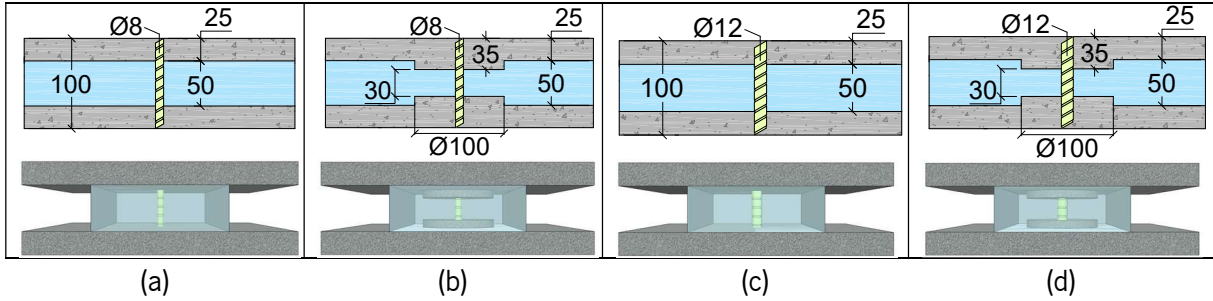


Figure 3.10: Specimen typologies for pushout tests with different S-GFRP connectors [dimensions in mm]: (a) SC-Ø8-H25-XPS3; (b) SC-Ø8-H35-XPS3; (c) SC-Ø12-H25-XPS3; (d) SC-Ø12-H35-XPS3.

3.3.3.2 Results and failure modes

Figures 3.11a, 3.11b, 3.11c and 3.11d present the load *vs.* horizontal relative displacement curves for series SC-Ø8-H25-XPS3, SC-Ø8-H35-XPS3, SC-Ø12-H25-XPS3 and SC-Ø12-H35-XPS3, respectively. The comparison between all test series is shown in Figure 3.11e, where the average curves obtained from all pushout tests (including pushout test with XPS-3 reported in Section 3.3.2) are presented. The summary of results obtained for each test series is also included in Table 3.4, where, for this series of pushout tests, the maximum relative displacement ($\delta_{\max}$) corresponds to the relative deformation for which specimen failure occurs or, in case of gradual load decrease, when the applied load becomes lower than 1 kN. Photos of a representative test specimen before and during the experimental procedure are shown in Figures 3.12a and 3.12b, respectively. Since only one connector was used, placed at the centre of the specimens, a slight (not measured) rotation between wythes was observed (visible in Figure 3.12b). However, the setup was stable enough to provide reliable results.

The structural behaviour of the various sandwich panel specimens containing S-GFRP connectors can be described by the following four main phases: (i) linear-elastic behaviour until approximately 0.65 mm of relative displacement between wythes; (ii) beginning of non-linearity with slight stiffness reduction (due to initial damage in the RSFRC matrix, near the anchorage zone) until a first peak load is reached; (iii) a phase in which the load almost tended to a plateau, up to approximately 16 mm of relative displacement between wythes in the series SC-Ø8-H25-XPS3, SC-Ø8-H35-XPS3 and SC-Ø12-H35-XPS3 (see deformed specimen in Figure 3.12b); and (iv) a softening phase until specimen failure, which was reached earlier

for specimens with higher embedment length. Test series SC-Ø12-H25-XPS3 (see Figure 3.11c) had a very small load plateau phase (ended at about 5 mm of relative displacement) due to the relatively high stiffness of this connector (12 mm diameter), together with the relatively small embedment length (25 mm), resulting in larger damage in the RSFRC anchorage area.

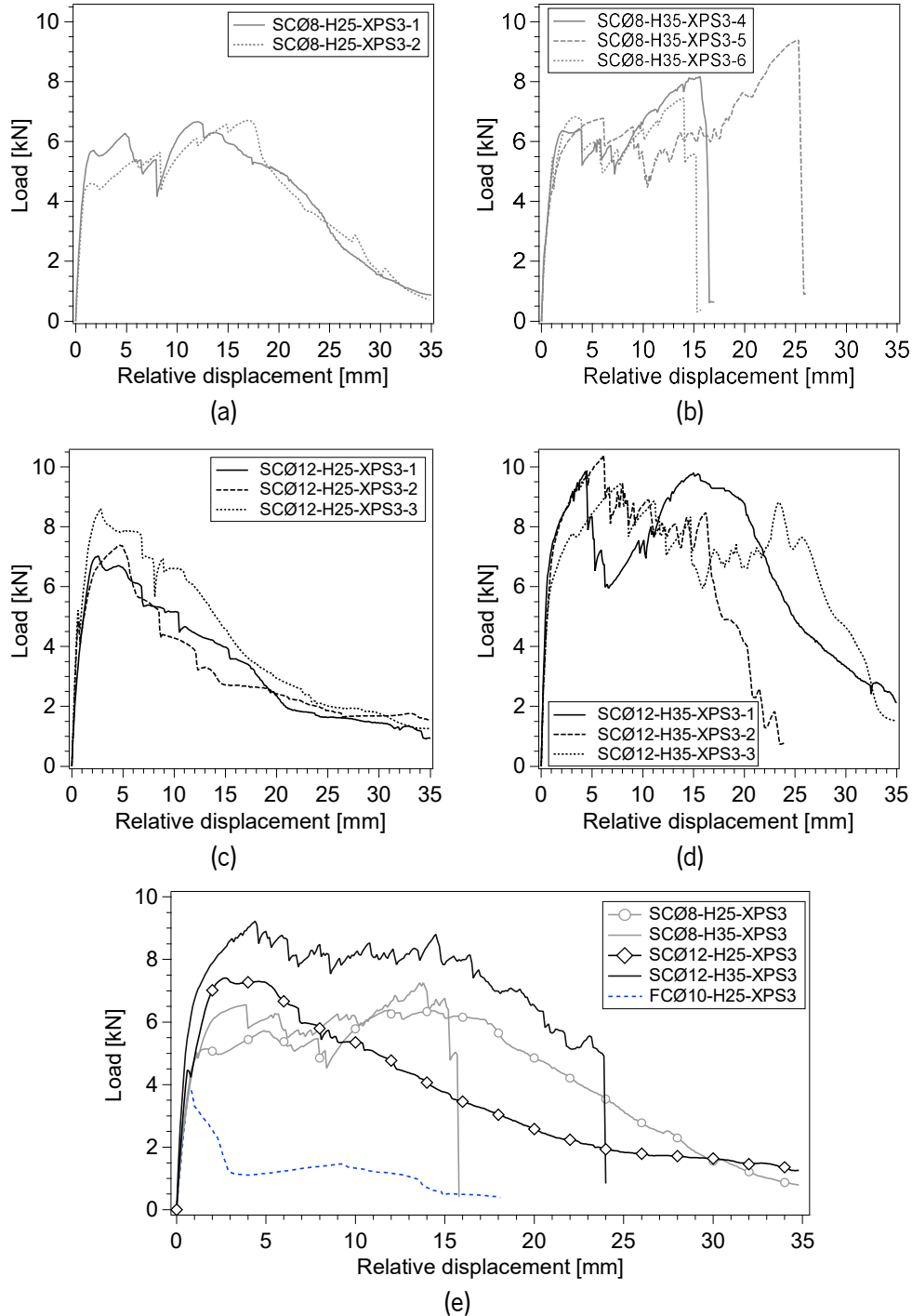


Figure 3.11: Pushout tests on specimens with different S-GFRP connectors – load *vs.* relative displacement responses for: (a) SC-Ø8-H25-XPS3; (b) SC-Ø8-H35-XPS3; (c) SC-Ø12-H25-XPS3; (d) SC-Ø12-H35-PS3; (e) average curves.

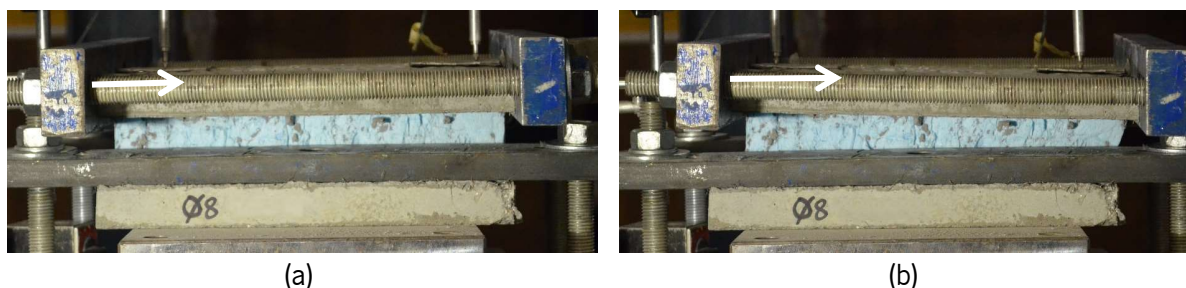


Figure 3.12: Pushout tests on specimens with different S-GFRP connectors: specimen before (a) and during (b) testing procedure (for an imposed displacement equal to 16 mm).

Figure 3.13 shows the damage patterns for each specimen typology after testing (the polystyrene layer was removed to enable a clearer observation), with emphasis on both the RSFRC layers (left-hand side of Figure 3.13) and the S-GFRP connectors (right-hand side of Figure 3.13). The figure prompts the following remarks: (i) specimens of typology SC-Ø8-H25-XPS3 exhibit minor RSFRC spalling near the anchorage areas (Figure 3.13a) and visible damage in the S-GFRP connectors (splitting of the connector in its longitudinal direction – Figure 3.13b); (ii) shear failure of the S-GFRP connectors occurred for specimens of typology SC-Ø8-H35-XPS3 (rupture of the connector is visible in Figures 3.13c and 3.13d), in contrast with the remaining specimen typologies for which higher ductility was obtained and failure was due to gradual debonding of the upper GFRP-RSFRC connection; (iii) specimens of typology SC-Ø12-H25-XPS3 present visible cracks and spalling of the RSFRC layer (see Figure 3.13e) and the lowest damage level on the S-GFRP connectors (see Figure 3.13f), which explains the more pronounced softening phase for this specimen configuration (the shear capacity of the S-GFRP connector was not mobilized, being the spalling and crushing of the RSFRC the failure mechanisms of this connection system); and (iv) SC-Ø12-H35-XPS3 specimens afforded the highest load carrying capacity up to approximately 24 mm of relative displacement between RSFRC wythes, reaching an average peak load of 9.9 kN and presenting damage both in the RSFRC layer (see in Figure 3.13g the radial cracks that formed in the augmented anchorage depth zone) and in the S-GFRP connector (splitting of the connector due to formation of multiple longitudinal cracks during the plateau phase – Figure 3.13h).

This last connection system provided the highest shear load transfer, pseudo-ductility and energy absorption, due to the full exhaustion of the S-GFRP connector and the surrounding RSFRC, being therefore the most appropriate arrangement for structural purposes. However, it should be noted that materializing the thicker part of the RSFRC for ensuring a thickness of 35 mm has detrimental consequences on execution time and costs of the type of panel in development. In view of this, SC-Ø8-H25-XPS3 is regarded as a more rationally balanced system, as long as the shear load per connector does not exceed 5 kN.

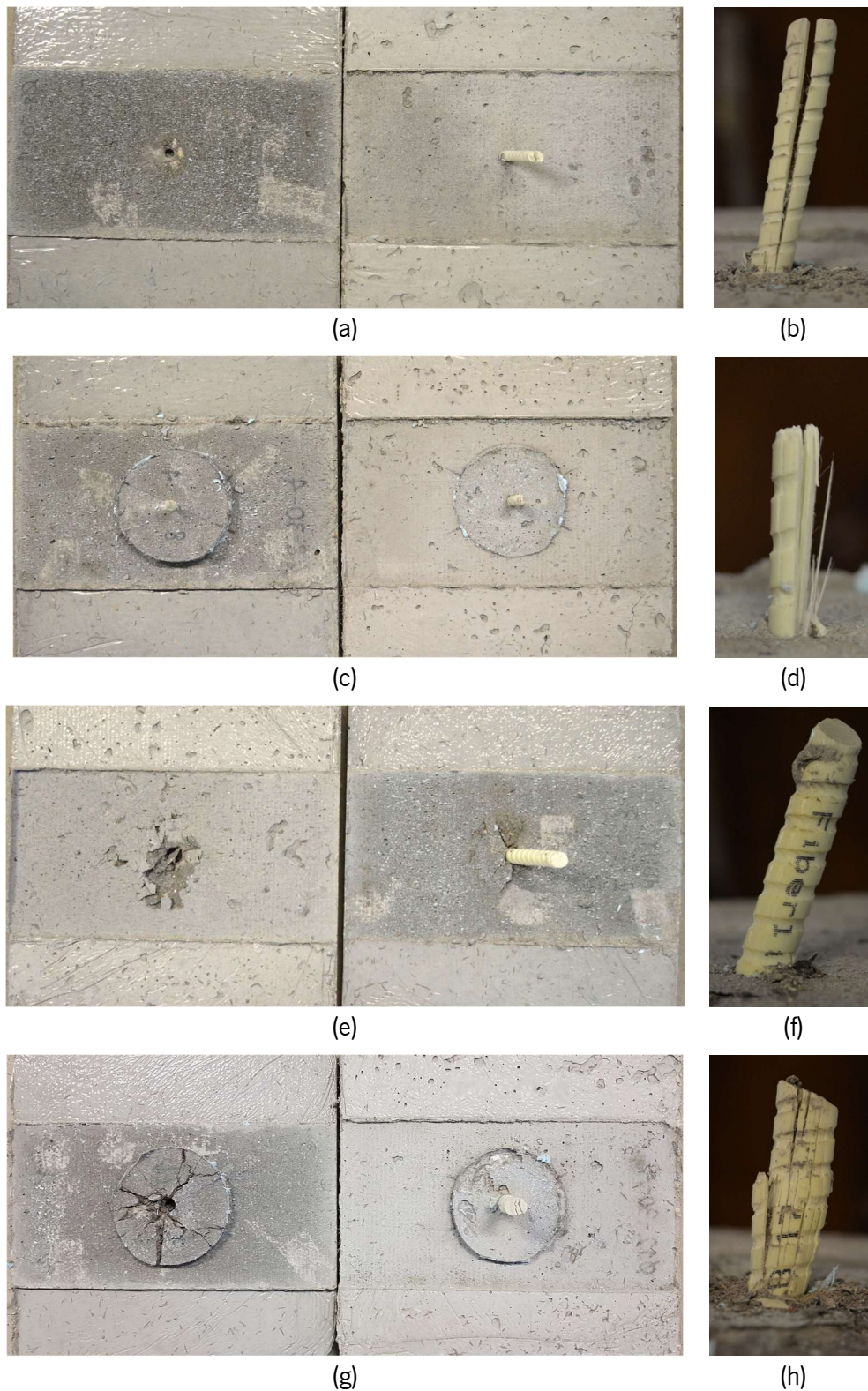


Figure 3.13: Pushout tests using different S-GFRP connectors (photos showing damage state of tested specimens): (a, b) SC-Ø8-H25-XPS3; (c, d) SC-Ø8-H35-XPS3; (e, f) SC-Ø12-H25-XPS3; (g, h) SC-Ø12-H35-XPS3.

3.3.4 Pullout tests for selection of structural GFRP connectors

3.3.4.1 Experimental setup and procedure

In order to obtain comprehensive information to make a well-supported decision on the type of S-GFRP connectors for the sandwich panel in development, pullout tests were conducted to assess the bond performance of this type of connectors to RSFRC. Figures 3.14a and 3.14b illustrate the setup of these tests. The parameters considered in the pushout test program, namely the diameter and embedment depth (see Table 3.3), were also adopted in the pullout test program, with 3 specimens for each parameter value, resulting in a total of 12 tests. It should be noted that, due to the reduced length of the GFRP connectors (100 mm), the end part of the connectors had to be surrounded by a Steel Hollow Section (SHS) profile, bonded to the GFRP connector with epoxy adhesive (S&P Clever Reinforcement, 2018) (see Figure 3.14c). This was done to provide higher contact area between the grip of the testing equipment and the GFRP connector (the grip is clamped around the SHS profile, as shown in Figure 3.14a). Due to space restrictions, it was impossible to use additional LVDTs for measuring exclusively the slip between the S-GFRP and the surrounding RSFRC during the pullout test. Therefore, the presented results only include information regarding load application (measured by a load cell of 50 kN capacity) and additional measurements conducted after performing the pullout tests. The pullout tests were performed under displacement control, at a rate of $10 \mu\text{m/s}$.

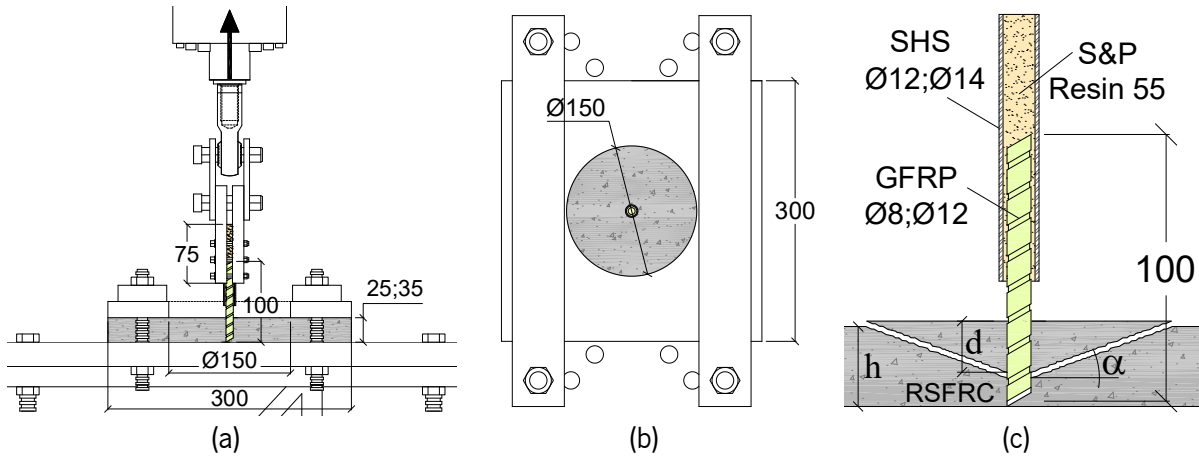


Figure 3.14: Experimental setup for pullout tests with different S-GFRP connectors [dimensions in mm]:

(a) Front view; (b) Top view; (c) Detailed view of GFRP connector.

3.3.4.2 Results and failure modes

Figure 3.15 presents the maximum pullout force that was obtained during the pullout tests for all specimens. Average values for each specimen typology are presented in Table 3.5. All specimens failed by debonding between the S-GFRP connector and the surrounding RSFRC, followed by the formation of a *quasi*-concrete cone that disintegrated by micro-spalling (see Figure 3.16). Additional information concerning the registered failure mode is also provided in Table 3.5 (equivalent concrete cone diameter, cone depth and cone angle, as depicted in Figure 3.14c). The equivalent concrete cone diameter was determined by the average value resultant from four measurements that were taken on each of the tested specimens, as shown in Figure 3.16d.

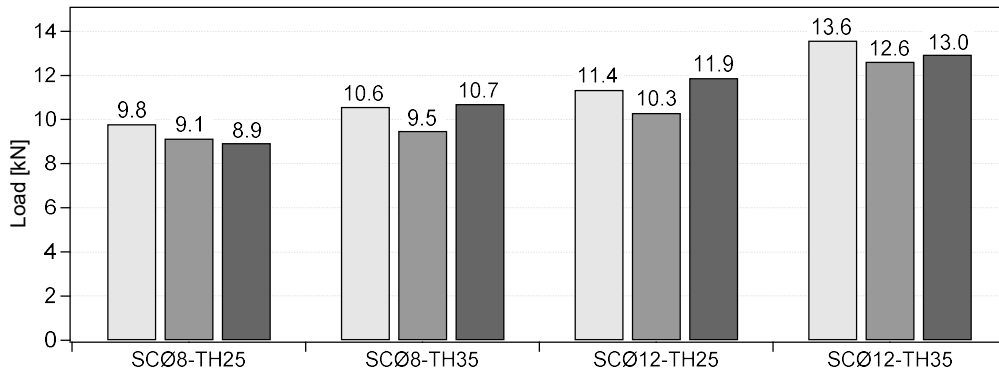


Figure 3.15: Maximum pullout force obtained during pullout tests (for all specimens).

Table 3.5: Pullout tests: relevant results for tested specimen typologies (average values).

Specimen designation	$F_{\max}$ [kN]	Embedment depth (h_c) [mm]	Bond strength [MPa]	Cone diameter [mm]	Cone depth (d_c) [mm]	Cone angle (α_c)
SCØ8-H25	9.3	27.4	13.5	62.0	18.5	17.1°
SCØ8-H35	10.3	34.4	11.9	62.2	20.4	18.8°
SCØ12-H25	11.2	26.9	11.0	81.1	22.2	15.6°
SCØ12-H35	13.1	34.2	10.2	90.0	25.7	16.1°

As expected, the minimum pullout force was obtained for test series SC-Ø8-H25, which registered an average load equal to 9.3 kN. For the remaining test series SC-Ø8-H35, SC-Ø12-H25 and SC-Ø12-H35, average pullout forces of respectively 10.3, 11.2 and 13.1 kN were registered. It can also be seen that the increase of embedment depth (RSFRC thickness) had a more pronounced effect for the case of connectors with 12 mm of diameter, resulting in an increase of pullout force of approximately 17% (that was only 10% in the case of SCØ8). Assuming that at peak load no spalling has occurred in the RSFRC,

and considering the embedment depth for the S-GFRP registered in the post-inspection tests, an average bond strength varying between 10.2 to 13.5 MPa was obtained, with a decreasing trend with the increase of the S-GFRP diameter and embedment depth. It would have been possible to derive a local bond constitutive law by an inverse analysis technique, such as the one described in (Rosa *et al.*, 2019); however, loaded end slip would have to be measured, together with the applied force – future experiments should be conducted in this respect.

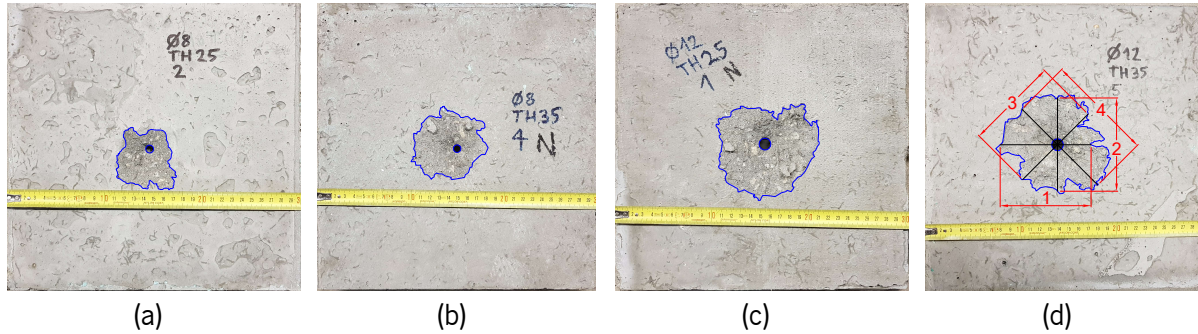


Figure 3.16: Pullout tests using different S-GFRP connectors (photos showing damage state on RSFRC layer): (a) Ø8-H25; (b) Ø8-H35; (c) Ø12-H25; (d) Ø12-H35.

3.4 Concluding remarks

The experiments presented in this chapter focused mainly (i) on the characterization of the behaviour of the panels' individual components, and also (ii) on the shear connection behaviour of the panels, namely on the influence of the type of insulation layer and the type of GFRP connector. Based on the experimental results obtained, the following main conclusions can be drawn:

1. The RSFRC exhibited satisfactory behaviour in terms of compressive strength and stress at crack initiation, and, more importantly, relatively high post-cracking tensile capacity. It has been shown that RSFRC is a viable solution for the production of façade panels to be applied in the structural (*e.g.*, seismic) rehabilitation of RC-frame buildings, as it allows for the production of thin laminar structural concrete elements, while contributing to a more sustainable construction sector.
2. The pushout tests performed for the selection of the type of insulation layer to be used in the sandwich panels showed that, although this component is regarded as being responsible almost exclusively for providing thermal insulation, it can afford non-negligible contribution to the structural performance of the panel. The experimental program revealed clear differences between specimens with different polystyrene typologies, therefore highlighting the influence of

the adhesion between RSFRC and polystyrene on the overall structural performance of the sandwich panel solution under shear loading conditions. Specimens with XPS-3 core presented the highest in-plane shear capacity up to a relatively large lateral deformation (3 mm). EPS specimens, although having the lowest mechanical properties among all tested polystyrene typologies, performed better under shear loading when compared to the ones containing XPS with smooth surface finishing (XPS-5) – this result also stems from the different adhesion between the sandwich panel layers due to the polystyrene surface treatment.

3. The pushout tests where the diameter and anchorage depth of structural GFRP connectors were analysed showed that, for connectors with 8 mm of diameter, 35 mm of anchorage depth provided higher shear capacity due to the full mobilization of the connector's shear capacity. However, for the referred diameter, the specimens with 25 mm of anchorage depth presented higher ductility, indicating a better composite behaviour in terms of maximum lateral deformation (37 mm, + 48% compared to 35 mm of anchorage depth), while still ensuring a quite significant shear capacity. Regarding the connectors with 12 mm of diameter, better composite behaviour was obtained for specimens with the higher anchorage depth of 35 mm, which provided a ductile behaviour and the highest load carrying capacity among all tested specimen typologies up to a lateral relative displacement of 24 mm. Overall, considering the proposed sandwich panel solution, the results obtained show that structural GFRP connectors with diameters of 8 and 12 mm are able to ensure the load transfer between RSFRC layers subjected to shear loading, especially when combined with anchorage depths of 25 and 35 mm, respectively; 12 mm diameter connectors are particularly well suited for applications with more demanding structural requirements.
4. The pullout tests showed a good bond behaviour between RSFRC and GFRP connectors, with the failure mode being governed by damage introduced in the surrounding RSFRC matrix, with average pullout loads higher than 9 kN for all tested configurations. The results of these tests also showed that the increase in anchorage capacity due to increasing RSFRC layer thickness seems to be more relevant for GFRP connectors with 12 mm of diameter.

4 CYCLIC BEHAVIOUR OF RC FRAMES STRENGTHENED WITH COMPOSITE SANDWICH PANEL

4.1 Introduction

To avoid the addition of overly heavy structural elements and aiming to improve structural and energy performance, various types of lightweight and insulated precast concrete panel solutions have been proposed and experimentally tested under seismic/cyclic loading, designed for both new/modular construction (Guo *et al.*, 2019; Refaei *et al.*, 2015; Lameiras, 2015; Hamid and Fudzee, 2013; O'Hegarty and Kinnane, 2020) and for rehabilitation (Hashemi *et al.*, 2018; Bournas, 2018; Palermo and Trombetti, 2016).

Also, taking into account the need to reduce the recourse to new raw materials and to promote sustainability (ACI Committee 369, 2011), several research works, mostly focusing on thermal/energetic refurbishment (Bagarić *et al.*, 2020; Moussa *et al.*, 2018; Maalouf *et al.*, 2018; Asdrubali *et al.*, 2015; Banjad Pečur *et al.*, 2014), have developed rehabilitation solutions for building envelopes that use recycled materials.

The use of recycled materials for structural purposes, such as the addition of recycled steel fibres (RSF) for discrete reinforcement of RC elements, has also been increasingly addressed in the literature (Aslani *et al.*, 2018; Caggiano *et al.*, 2017; Frazão, 2019; Lourenço *et al.*, 2018; Zamanzadeh *et al.*, 2015). However, the use of RSF for the production of applications designed for enhancing seismic performance is still scarce, especially regarding the production of façade/sandwich panels.

Considerable efforts have also been made on using FRP shear connectors (Choi *et al.*, 2015; Choi *et al.*, 2019; Cox *et al.*, 2019; Flansbjer *et al.*, 2016; Frankl *et al.*, 2011; Huanzhi *et al.*, 2017; Lameiras *et al.*, 2018; Pahn, 2013; Tomlinson *et al.*, 2016; Yossef, 2017) instead of steel connectors to avoid thermal bridges between sandwich panel wythes. However, most sandwich panel solutions proposed for integrated seismic and thermal retrofit of building envelopes did not assure a continuous insulation or avoided thermal bridges in building envelopes, namely in the discontinuities caused by localized areas corresponding to RC structural members (beams, columns and slabs).

This chapter is related to the second phase of the experimental program associated to the research work of the thesis. It presents the results obtained by testing RC frame specimens, designed according to old codes that do not fulfil present seismic design requirements. The experiments reported herein are a continuation of the body of work laid out in the previous chapter, comprising two RC frames with and without the addition of masonry infill walls (representative of the target building typology), and one RC frame including a sandwich panel prototype - these specimens are tested under displacement reversals until failure. Results obtained are analysed in terms of hysteretic load-displacement response, damage evolution, strength and stiffness degradation, ductility, energy dissipation, column deformation pattern and strain-drift evolution (rebars).

4.2 Experimental program

4.2.1 Overview

The experimental program presented in this chapter aimed at evaluating the cyclic behaviour of intermediate-scale RC frame specimens, namely by adopting three different specimen configurations: (i) a bare RC frame (reference specimen “R”), to assess the individual behaviour of an RC frame that is representative of the structural skeleton of the referred target buildings (Figure 4.1a); (ii) an RC frame with masonry infill wall (specimen “M”), representative of the target building envelopes (structural skeleton and envelope masonry walls – see Figure 4.1b); and (iii) a strengthened RC frame, through the attachment of a sandwich panel prototype (specimen “S” - see Figures 4.1c, 4.2b and 4.2c), in order to evaluate the feasibility of the rehabilitation solution proposed in this research work (see Figures 1.2 and 4.2a).

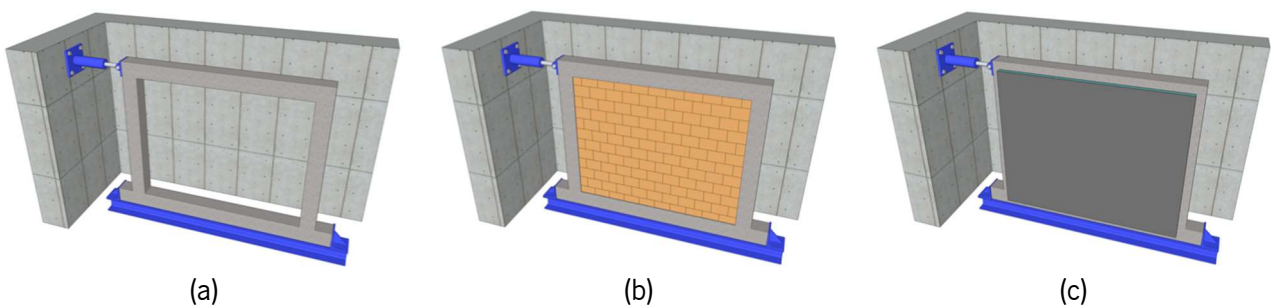


Figure 4.1: Evaluation of cyclic behaviour for: (a) bare RC frame (specimen “R”); (b) RC frame with masonry infill wall (specimen “M”); (c) RC frame with sandwich panel (specimen “S”).

This section presents specific details regarding: (i) the design approach and final dimensions of the RC frame specimens (subsections 4.2.2 and 4.2.3); (ii) material characterization (subsection 4.2.4); (iii) test setup and instrumentation (subsection 4.2.5); and (iv) loading protocol (subsection 4.2.6).

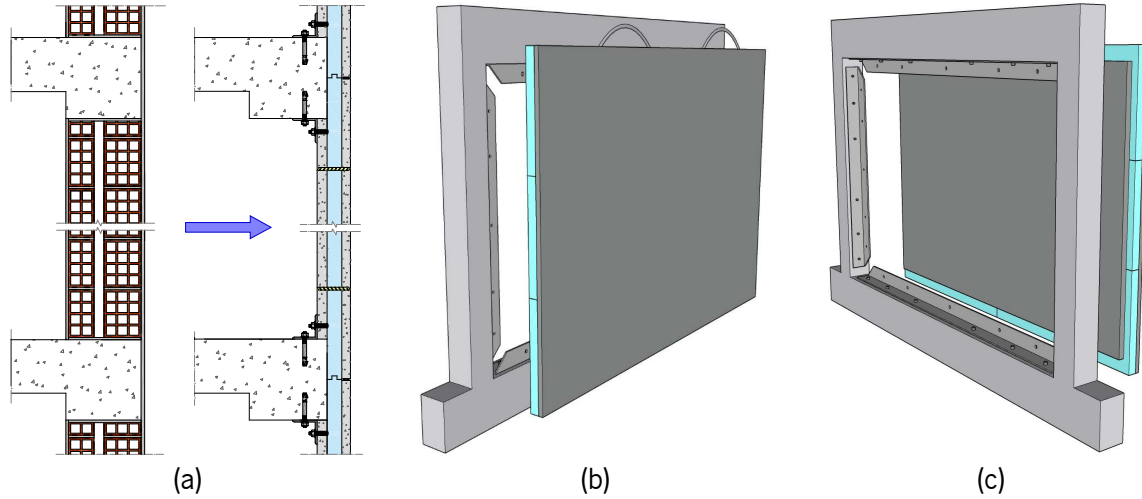


Figure 4.2: Rehabilitation concept proposal: (a) before/after retrofit; (b) 3D, front view; (c) 3D, back view.

4.2.2 Reduced-scale RC frame specimens

The RC frame specimens were designed to be representative of the target buildings mentioned in section 0. Accordingly, the steel reinforcement of the frame elements was designed and detailed according to the recommendations of first-generation seismic design codes (REBA, 1967) and based on the study performed by Furtado *et al.* (2016) that focused on the geometric characterization of the Portuguese RC building stock since 1960. This study entailed an extensive collection of data derived from about 80 structural and architectural design projects for the development of statistical models and probabilistic distributions, focusing on parameters such as column and beam dimensions and reinforcement ratios, gathered from a database of approximately 500 beams and columns.

Due to laboratory geometry restrictions, intermediate-scale specimens had to be considered, namely by adopting a scale factor of 1:1.5 with respect to typical real-scale dimensions (for both the overall geometry of the structural members and the reinforcing bars). Figure 4.3 presents the dimensions of the RC frame specimens, including the details of the adopted reinforcement scheme (photographs taken during the casting process of the RC frames are illustrated in Figure 4.4).

As expected, the adopted design recommendations led to a reinforcement scheme with the non-ductile detailing characteristics that were mentioned in subsection 2.2.3 (see Figure 2.9). Table 4.1 presents the correlation between the adopted RC frame specimen characteristics (dimensions and longitudinal reinforcement ratio for columns and beams) and the values that match the normally distributed statistical analysis presented in the referred geometric characterisation study (Furtado *et al.*, 2016): in general, the corresponding real-scale dimensions compare well with the mean values of the statistical distribution.

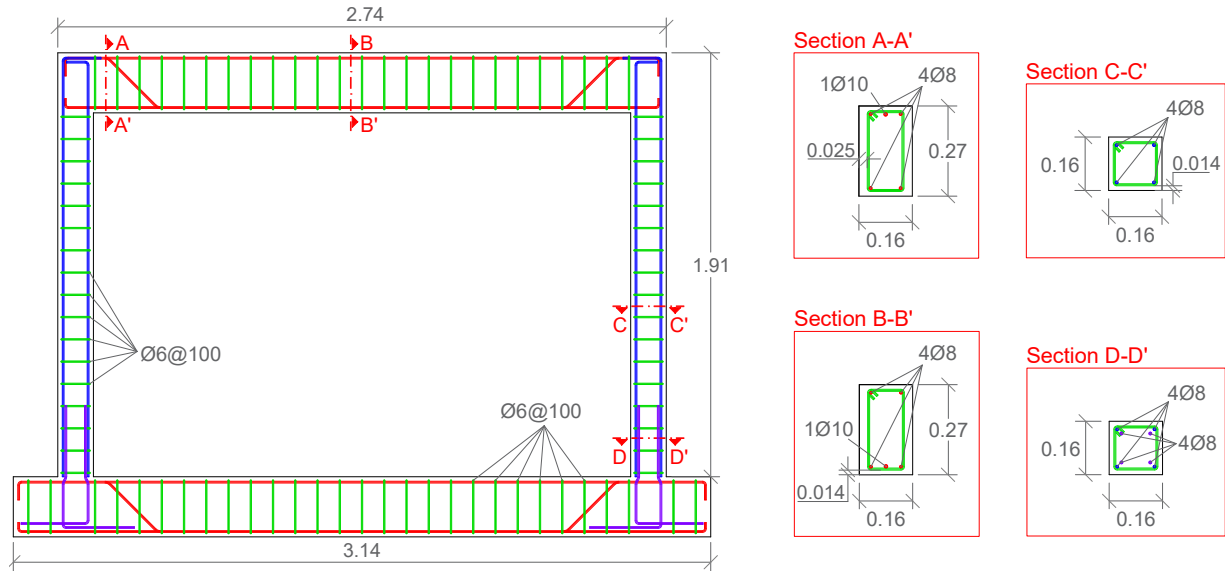


Figure 4.3: RC frame specimens [dimensions in m]: geometry and reinforcement scheme.

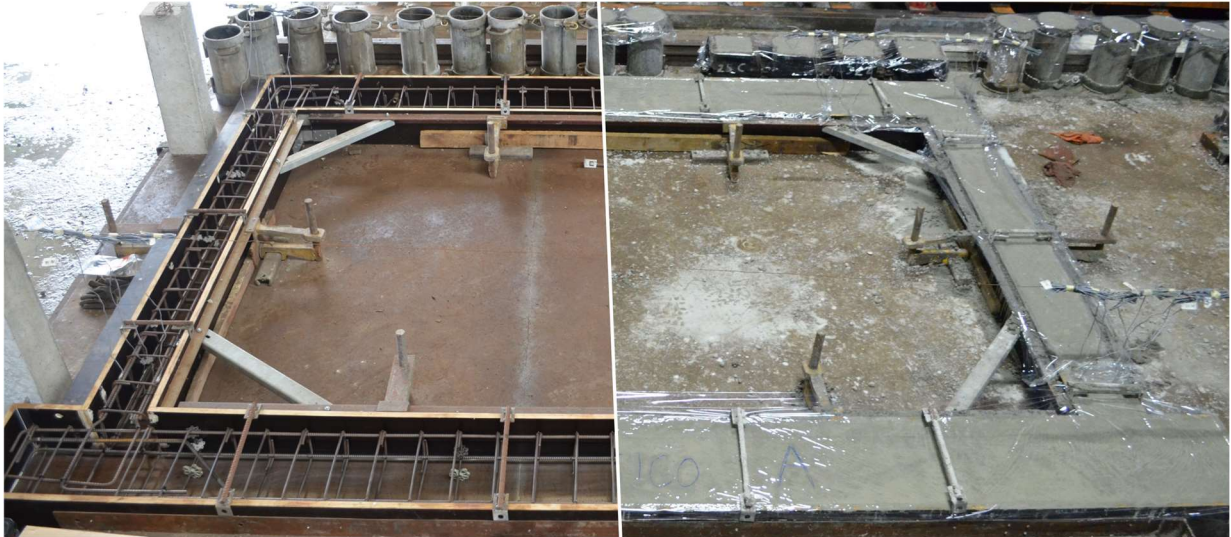


Figure 4.4: RC frame specimens before (left hand side) and after (right hand side) casting.

Table 4.1: RC frame details and correlation with the work conducted by Furtado *et al.* (2016).

RC element characteristics	Specimen	Real-scale	Mean value (Furtado <i>et al.</i> , 2016)
Beam length [m]	2.58	3.86	4.42
Beam height [m]	0.27	0.41	0.43
Beam width [m]	0.16	0.24	0.29
Beam reinforcement ratio [%]	0.65		0.63
Column height [m]	1.90	2.85	2.77
Column width [m]	0.16	0.24	0.26
Column depth [m]	0.16	0.24	0.38
Column stirrup spacing [m]	0.10	0.15	NA
Column reinforcement ratio [%]	0.79		0.61

4.2.3 Sandwich panel prototype

4.2.3.1 Sandwich panel components

The sandwich panel solution that is being proposed in this research work is composed essentially by three main components, namely (see Figure 4.5): (i) thin outer layers made of RSFRC; (ii) an insulation core layer made of XPS; (iii) GFRP connectors that join the aforementioned outer and inner layers of the panel (see subsection 3.2.2 for additional information). Figure 4.6 presents a more detailed view of the panel and its dimensions. The components of this panel were selected and characterized in the first part of this extended experimental program, described in chapter 3.

The strengthening solution proposed herein involves the application of the developed sandwich panel to the existing structure after removal/demolition of existing masonry infill walls (see Figure 4.2a). Therefore, the sandwich panel acts both as structural infill wall and as building skin with thermal insulation properties: (i) the inner RSFRC layer is thicker and has slightly smaller dimensions in comparison to the outer RSFRC layer (see dimensions and cross-section detail 1 in Figure 4.6 and detail 2 in Figure 4.9), acting as structural infill wall to provide structural strengthening to the RC frames against lateral loads; (ii) the XPS insulation layer covers the building envelope area, preventing common thermal bridges through the RC structural members (beams and columns) of the building envelope in direct contact with the natural environmental conditions; and (iii) the outer RSFRC layer, joined to the inner RSFRC wythe through the aforementioned GFRP connectors, acts as external shield to the building envelope, providing additional structural strengthening through the connection to adjacent façade panels (see Figure 4.2a) and an extra level of noise insulation.

Photographs taken during the sandwich panel's casting process can be seen in Figure 4.7. This figure illustrates the steps that were taken during this process: (i) casting of the first layer of RSFRC (outer wythe

of the sandwich panel) – Figure 4.7a; (ii) preparation of the insulation layer (assembly of XPS plates) and placement of GFRP connectors – Figure 4.7b; (iii) mould adjustment for casting of the second RSFRC layer (inner layer of the sandwich panel) by disposing strips of XPS in contour to adjust the mould to the dimensions of the second RSFRC layer (inner layer of the sandwich panel) – Figure 4.7c; and (iv) casting of the second RSFRC layer (inner/infill wythe of the sandwich panel) – Figure 4.7d.

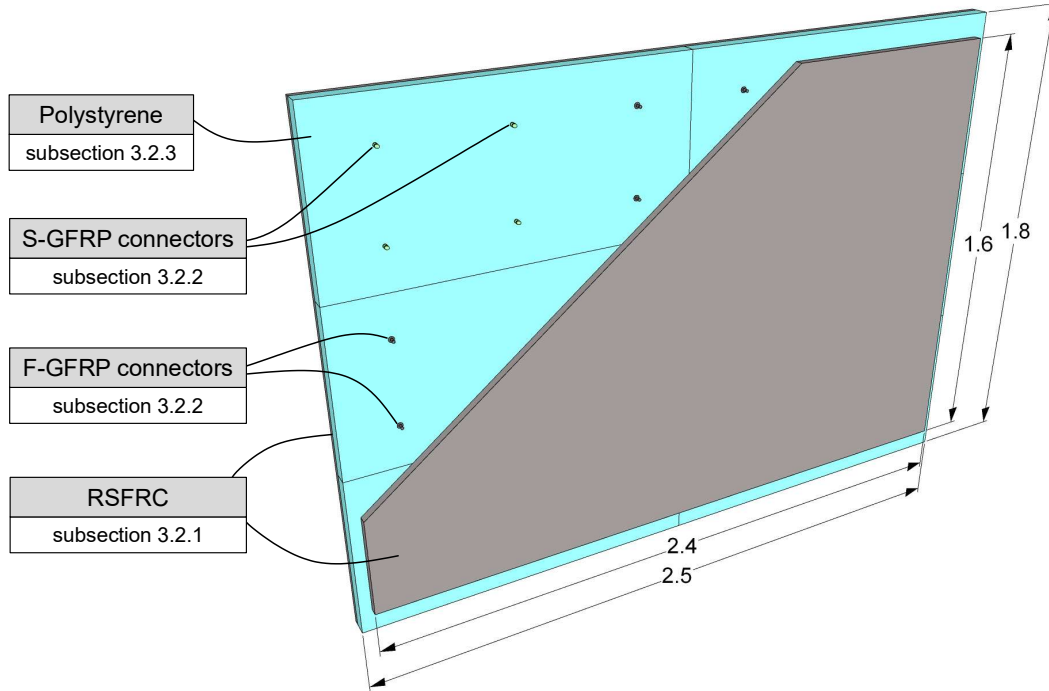


Figure 4.5: 3D representation of sandwich panel prototype with indication of sections in which each component is addressed [dimensions in m].

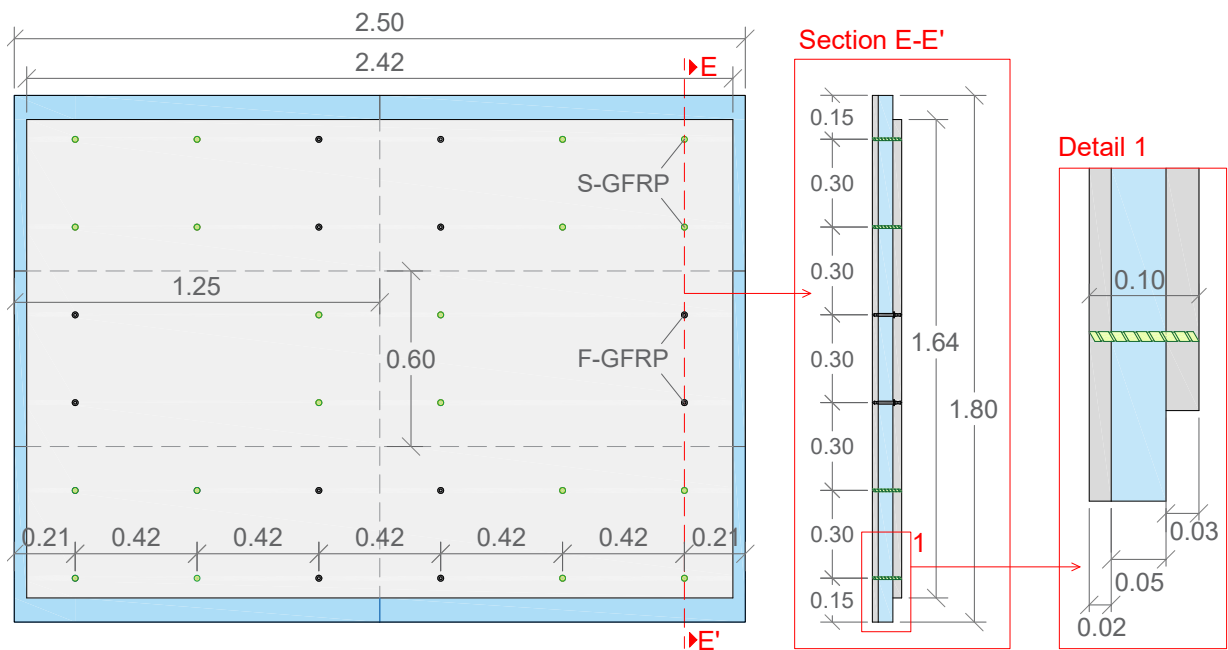


Figure 4.6: 2D representations of sandwich panel prototype [dimensions in m].

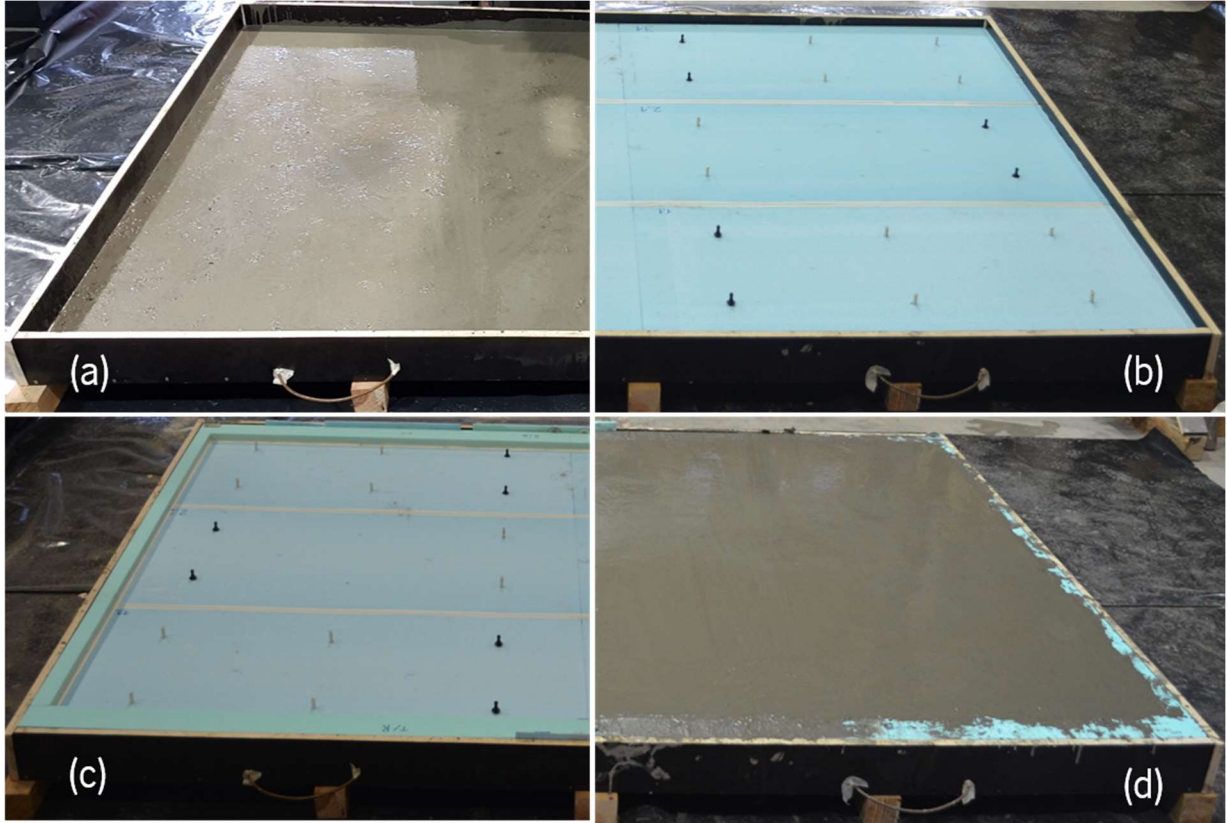


Figure 4.7: Casting process of sandwich panel prototype: (a) casting of first/outer RSFRC layer; (b) placement of XPS plates and GFRP connectors; (c) mould adaptation for casting of second/inner RSFRC layer; (d) casting of inner RSFRC layer.

4.2.3.2 Internal GFRP connectors

The number of adopted S-GFRP connectors, whose distribution is illustrated in Figure 4.6, was defined with basis on numerical simulations using version 4.0 of *FEMIX* (Azevedo *et al.*, 2003; Sena-Cruz *et al.*, 2007; Ventura-Gouveia, 2011), a finite element method (FEM) based computer program, together with the pre- and post-processing software *GiD* (version 14). The models were calibrated based on the experimental work presented in chapter 3. Detailed information regarding the type of finite elements and integration scheme adopted are presented in Section 5.4. The number of required connectors was evaluated for the following scenarios: (i) lifting operations associated with transportation and post-production activities (see Figure 4.8a); (ii) on site installation process, where the weight of adjacent panels can cause shear action between internal and external wythes of the panel (see Figure 4.8b – linear distributed load equal to 2.0 kN/m); and (iii) wind action (out of plane pressure loads, as represented in Figure 4.8c – pressure load up to 2.0 kN/m²). The constituent materials of the panel were modelled by assuming linear elastic behaviour and partial safety factors of 1.5 for live load effects and material properties (to derive design values from characteristic values) were

considered. Taking into account the strength levels obtained experimentally in chapter 3 for the RSFRC-GFRP connection when subjected to shear (pushout tests – see subsection 3.3.3) and tensile loading (pullout tests – see subsection 3.3.4), the results obtained numerically showed that at least 2.7 S-GFRP connectors per square meter of sandwich panel are required to withstand shear and tensile loads induced by the referred design scenarios. In view of this, a minimum of 12 S-GFRP connectors are required for this particular sandwich panel prototype. The final adopted distribution of GFRP connectors, depicted in Figure 4.6, presented minimum distances of 300 mm between connectors and 150 mm between connectors and sandwich panel edges, thus allowing to fulfil safety verifications, while ensuring an adequate load distribution in the RSFRC wythes.

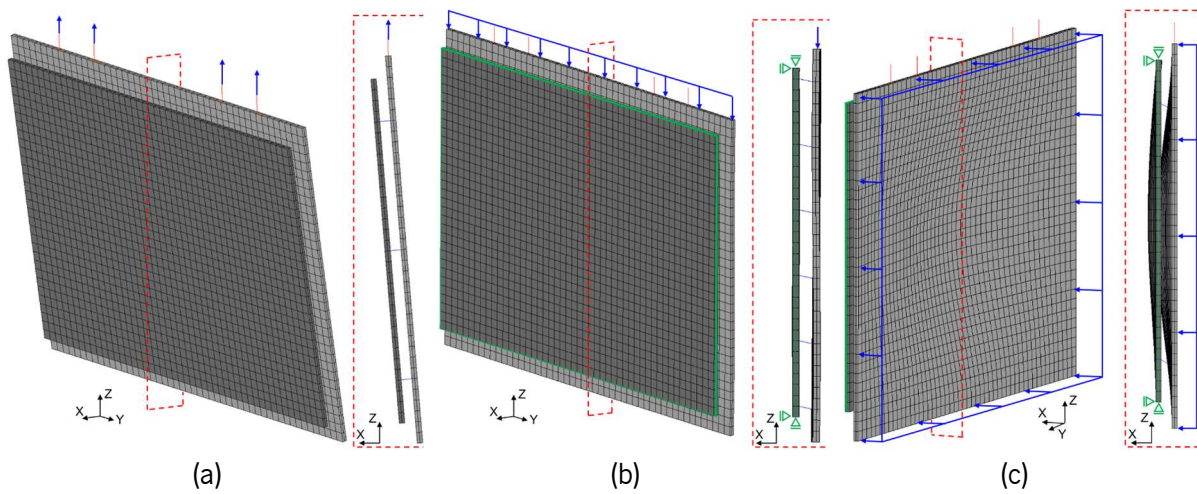


Figure 4.8: Deformed finite element mesh for the considered design scenarios: (a) lifting operations; (b) installation process; (c) wind action.

4.2.3.3 Connection between panel and RC frames

In addition to the main three components, this rehabilitation solution is complemented by steel components that ensure the connection between the façade sandwich panel and the existing structure (in this study, materialized by the RC frame specimens). The connections between the panel and the existing RC frames is provided by the following components: (i) metallic equal-leg angle profiles in the contour of the panel-frame connection (see Figures 4.2b and 4.2c); (ii) mechanical anchors (type HST 8.8 M12 – see detail 1 in Figure 4.9) that ensure the connection between the angle profiles and the RC frames prior to the application of the panel (as shown in Figure 4.2c); and (iii) adhesively bonded anchor bolts (type HIT-V 5.8 M10 – see detail 2 in Figure 4.9) that ensure the connection between the angle profiles and the infill layer of the panel, being fastened to the angle profiles to place the panel in its final position. Figure

4.9 presents the final dimensions adopted for the angle profiles and anchors. This final configuration was determined with basis on FEM-based numerical simulations, which are addressed in Section 5.4.

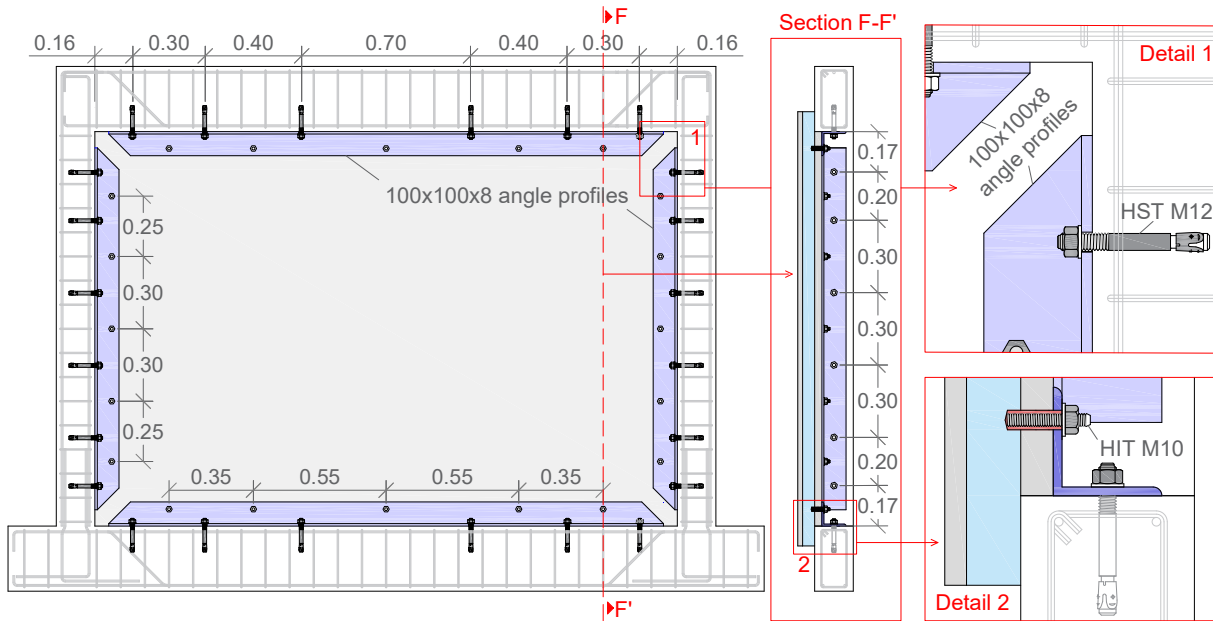


Figure 4.9: Connection between sandwich panel and RC frame [dimensions in m].

4.2.4 Materials

This subsection describes the characterization tests performed to assess the mechanical properties of the various materials involved in this chapter, namely: (i) plain concrete and steel rebars adopted for production of the RC frames (subsection 4.2.4.1); (ii) RSFRC that was used for production of the sandwich panel; and (iii) masonry bricks and mortars adopted for production of the masonry infill walls. The material properties of GFRP and XPS are detailed in the previous chapter, namely in subsections 3.2.2 and 3.2.2, respectively.

4.2.4.1 Reinforced concrete

The target multi-storey building's RC skeleton was commonly built using concrete class formerly designated as "B300" (REBA, 1967), with a characteristic compressive strength of cube specimens at 28 days equal to 30.0 MPa – corresponding roughly to the current class C25/30 (CEN, 2007). A concrete mixture composition aiming to replicate the referred concrete strength class was hence adopted for the production of the RC frames, which is presented in Table 4.2 (plain concrete).

Compressive strength, tensile strength and elastic modulus were assessed at the age of 28 days and at three additional occasions, corresponding to the age at which the RC frames were subjected to in-plane

cyclic tests (202, 296 and 770 days), by testing cylinders with 150 mm of diameter and 300 mm of height, according to standards EN 12390-13 (CEN, 2013), EN 12390-6 (CEN, 2009b) and EN 12390-3 (CEN, 2009a), respectively.

The obtained results are shown in Table 4.3. The tensile properties obtained for the steel reinforcing bars (rebars) are also presented in Table 4.3. These properties were assessed by testing 500 mm long rebar samples for each of the adopted rebar diameters, according to standard EN ISO 6892-1 (CEN, 2009c): stirrups ($\varnothing 6$) and longitudinal rebars ($\varnothing 8$ and $\varnothing 10$), as mentioned in Section 4.2.2. The results obtained, presented in Table 4.3 (average values and coefficients of variation), stem from characterization tests conducted on a minimum of 3 specimens for each material property.

Table 4.2: Adopted concrete mixture composition.

Component	Plain concrete [kg/m ³]
Cement (CEM I 42.5R)	260.0
Fly ash	-
Water	122.0
Crushed granite (14 mm)	718.0
Crushed granite (9 mm)	-
River sand (4 mm)	1256.0
Fine sand (0.6 mm)	-
Limestone filler	80.0
Superplasticiser (BASF SKY 617)	-
Superplasticiser (SikaPlast 912)	1.0
Recycled steel fibres	-

Table 4.3: Properties of concrete, RSFRC, steel rebars and GFRP connectors, with indication of corresponding coefficients of variation (CoV).

Material	Elastic modulus [GPa]	Compressive strength [MPa]	Tensile strength [MPa]	Yield strength [MPa]	Ultimate strength [MPa]
Concrete (28d)	26.9 (3.9%)	27.9 (5.4%)	2.2 (6.4%)	-	-
Concrete (208d)	28.6 (4.4%)	35.7 (2.8%)	2.7 (6.5%)	-	-
Concrete (296d)	29.6 (2.8%)	41.0 (3.5%)	2.8 (7.6%)	-	-
Concrete (770d)	28.9 (0.9%)	40.8 (3.9%)	2.8 (4.8%)	-	-
Steel ($\varnothing 6$ bar)	225.3 (1.3%)	-	-	583.1 (1.8%)	699.9 (0.9%)
Steel ($\varnothing 8$ bar)	203.4 (1.4%)	-	-	639.4 (3.5%)	765.7 (2.7%)
Steel ($\varnothing 10$ bar)	199.6 (1.8%)	-	-	585.8 (1.6%)	731.4 (2.1%)
RSFRC (28d)	27.3 (1.1%)	48.2 (1.9%)	-	-	-
RSFRC (196d)	28.3 (1.7%)	61.7 (2.9%)	3.4 (14.1%)	-	-
S-GFRP ($\varnothing 8$)	58.4 (2.7%)	-	1320.0 (12.5%)	-	-
F-GFRP ($\varnothing 10$)	10.4 (12.2%)	-	96.0 (1.3%)	-	-

4.2.4.2 Recycled steel fibre reinforced micro-concrete (RSFRC)

The mixture composition of the RSFRC is shown in Table 3.1. More information regarding the adopted RSFRC can be found in subsection 3.2.1. Although the mixture composition is equivalent to the one adopted for casting the specimens addressed in chapter 3, an additional material characterization was conducted for characterizing the relevant properties of the RSFRC of the sandwich panel prototype used in the cyclic test with the strengthened RC frame, at the age of this test.

Concomitantly, to the characterization of the plain concrete mentioned in the previous subsection, the compressive strength and modulus of elasticity of the RSFRC were assessed using the same testing procedures. The tests were performed at the age of 28 days and also at 196 days, which corresponds to the age at which the sandwich panel was subjected to in-plane cyclic tests. The results obtained are shown in Table 4.3.

The characterization of the tensile behaviour of the RSFRC including its post-cracking behaviour was assessed by performing direct tensile tests on notched dog-bone shaped specimens, as reported in subsection 3.2.1 (specimen geometry and test setup shown in Figure 3.2a and 3.2b, respectively), at the age of 196 days. The results obtained are shown in Figure 4.10. At crack initiation and at peak load, average tensile stresses of 3.4 MPa (CoV = 14.1%) and 4.0 MPa (CoV = 10.0%) were obtained, respectively. The post-cracking behaviour is characterized by residual tensile strengths of 3.7 MPa (CoV = 11.7%) at CMOD = 0.5 mm and 2.2 MPa (CoV = 17.3%) for CMOD = 2.5 mm (crack width values recommended by Model Code 2010 (Fédération Internationale du Béton, 2011) for determining the tensile stresses for serviceability and ultimate limit state conditions, respectively).

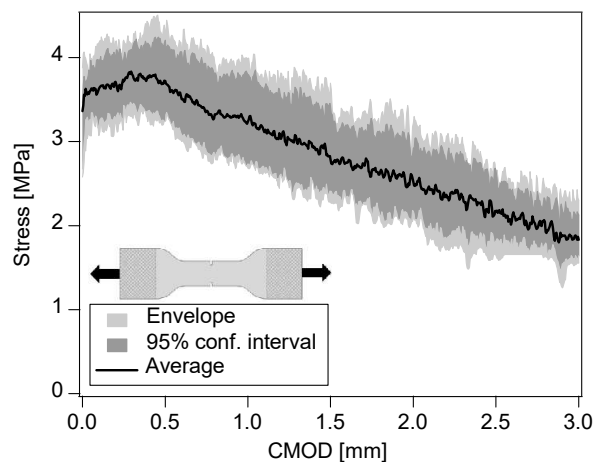


Figure 4.10: Post-cracking behaviour of RSFRC: load *vs.* CMOD curves.

4.2.4.3 Masonry and mortars

The masonry infill wall that was enclosed in one of the RC frames was built using multi-cell hollowed ceramic brick units (Cerâmica Vale da Gândara, 2012), with dimensions indicated in Figure 4.11a. An extensive experimental program was conducted by Martins (2018) for the assessment of the mechanical properties of this type of brick. The reported compressive strength values (with regards to the gross area of brick units), obtained according to EN 772-1 (CEN, 2000) for directions X, Y and Z (see Figure 4.11a), were equal to 4.15 MPa, 1.45 MPa and 1.18 MPa, respectively; these results reflect the anisotropic nature of the masonry bricks.

In addition to the masonry elements, two types of class M5 (CEN, 2016) mortars were adopted for the production of the masonry infill wall: (i) general purpose laying mortar, which was used to bond the brick units (mortar joints with approximately 20 mm of thickness); and (ii) pre-mixed plastering mortar, which was used to plaster coat the infill wall on both sides (5 mm of thickness, approximately).

The compressive and flexural strengths of the mortar were determined according to EN 1015-11 (CEN, 2017), at 28 days of age. For each type of mortar, a total of six specimens were prepared for the flexural tests (see Figure 4.11b) and 12 specimens (obtained from the specimens used in the flexural tests – see Figure 4.11c) were used for the compression tests. The obtained average compressive and flexural strengths were 8.3 MPa (CoV = 15.7%) and 1.4 MPa (CoV = 12.4%) for the laying mortar and 9.4 MPa (CoV = 12.0%) and 1.9 MPa (CoV = 8.4%) for the plastering mortar, respectively.

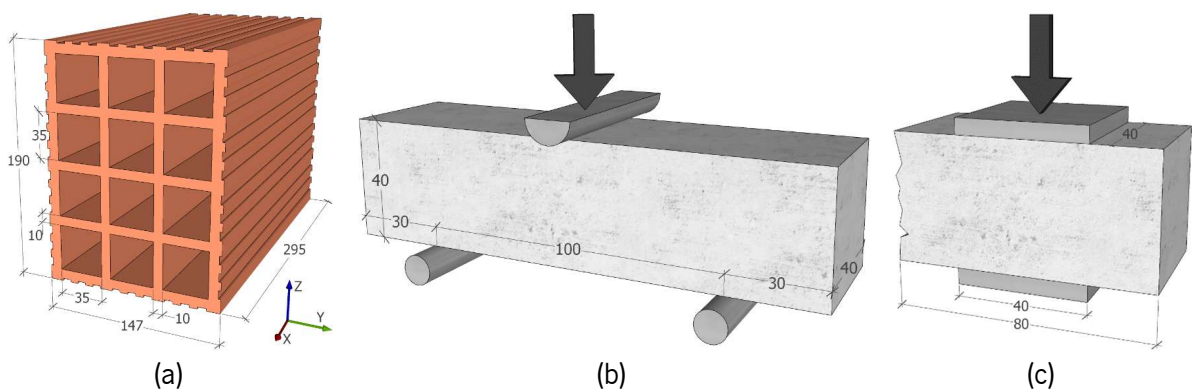


Figure 4.11: 3D representations of masonry and mortar elements [dimensions in mm]: brick unit (a) and mortar specimens subjected to flexural (b) and compression (c) tests.

4.2.5 Test setup and instrumentation

The test setup used in the cyclic tests of the RC frame is schematically represented in Figure 4.12 (numbered details mentioned in this paragraph refer to this Figure), being similar to the configuration adopted by Akhoundi (2016) and Silva *et al.* (2016). The RC frame was placed on top of two steel beams (HEA300, detail 1) that were attached to the laboratory's strong floor (detail 2) to avoid sliding/uplifting. Sliding and uplifting of the RC frame itself were prevented by two L-shaped steel profiles (L200, detail 3) and two tubular steel profiles (2UPN140, detail 4), respectively, both bolted to the referred steel beam. Out-of-plane movement of the upper concrete beam was restrained by L-shaped steel profiles (L100, detail 5) equipped with rollers (to avoid friction between RC frames and steel profiles), which are connected to a group of top steel frames (HEB240, detail 6) – this ensured that the hydraulic actuator (capacity of 300 kN, detail 7) that was attached to the reaction wall (detail 8) acted on the RC frame by imposing a strictly lateral (in-plane) displacement. The lateral displacement was imposed to the upper concrete beam through two steel plates with dimensions of $400 \times 400 \times 30 \text{ mm}^3$, which were connected by two $\varnothing 50 \text{ mm}$ *Dywidag* tie rods (detail 9). The experimental test setup was complemented by two vertical hydraulic jacks (detail 10) pinned to the lower steel beam through four vertical M16 steel rods (detail 11), which were placed on the top of the RC frame's columns, applying a vertical load of 160 kN. This load, which corresponds to approximately 21% and 15% of the concrete column's compressive strength at 28 and 296 days of age, respectively, aims to simulate the effect of an upper building storey (continuation of columns throughout the height of the building), which is a typical axial compressive stress level to which columns are subjected to when an earthquake occurs.

Measurements were conducted by using the following instrumentation scheme: (i) two displacement transducers (LVDTs) from *RDP Electronics* with amplitude of $\pm 75 \text{ mm}$ that measured the horizontal movement of the upper RC beam, one at each end (u_{1-2} , Figure 4.12); (ii) three *RDP* LVDTs with amplitudes of $\pm 75 \text{ mm}$, $\pm 50 \text{ mm}$ and $\pm 25 \text{ mm}$ that measured one of the column's deformation throughout its height (u_3 , u_4 and u_5 , respectively – see Figure 4.12); (iii) an LVDT with amplitude of $\pm 5 \text{ mm}$, which measured the relative horizontal displacement between the bottom RC beam and the steel beams that are connected to the strong floor (u_6 , Figure 4.12), to ensure that the supporting conditions during the test were kept as planned; (iv) a total of twenty four electrical *TML* strain gauges (type FLK-6-11-3L) to measure the strains of the RC frame's steel rebars, namely in the areas where higher damage was expected to occur (near the joints, as represented in Figure 4.13); and (v) a load cell

(*AEP Transducers*) with capacity of 300 kN to measure the applied lateral load. It is worth mentioning that the support conditions of the RC frames proved to be quite effective throughout the testing procedure, since in all cyclic tests the maximum relative displacements between the bottom RC beam and the steel beams connected to the laboratory's strong floor (u_6 , Figure 4.12) were lower than ± 0.05 mm.

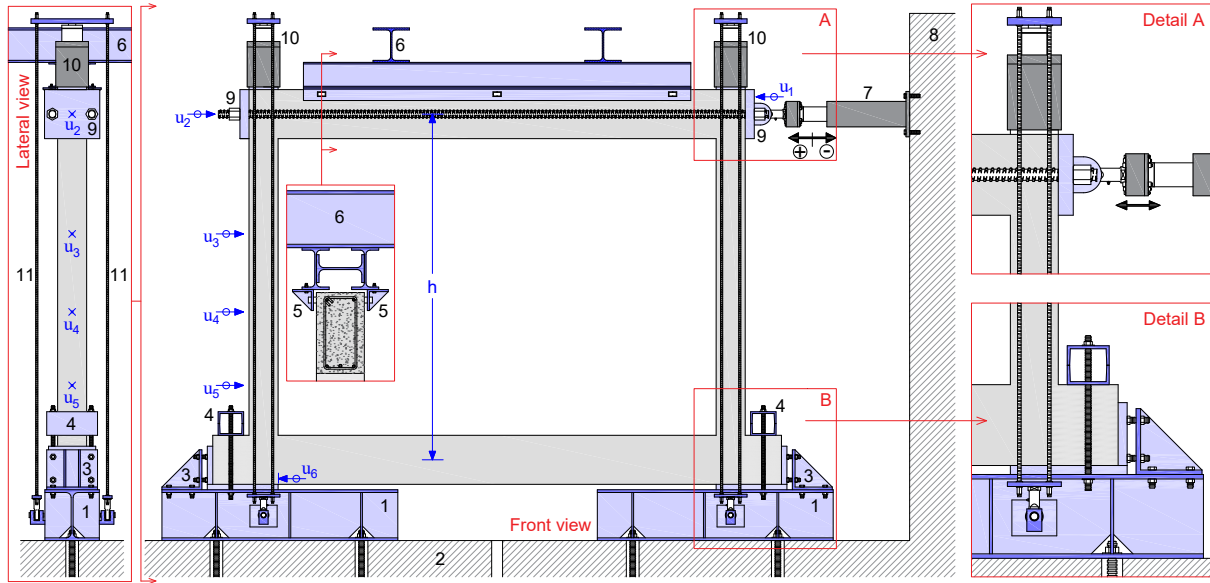


Figure 4.12: Test setup for in-plane cyclic loading of RC frames (1 – HEA300 beams; 2 – laboratory's strong floor; 3 – L200 profiles; 4 – 2 UPN140 profiles; 5 – L100 profiles; 6 – HEB240 profiles; 7 – hydraulic actuator; 8 – reaction wall; 9 – steel plates and Dywidag ties; 10 – hydraulic jacks).

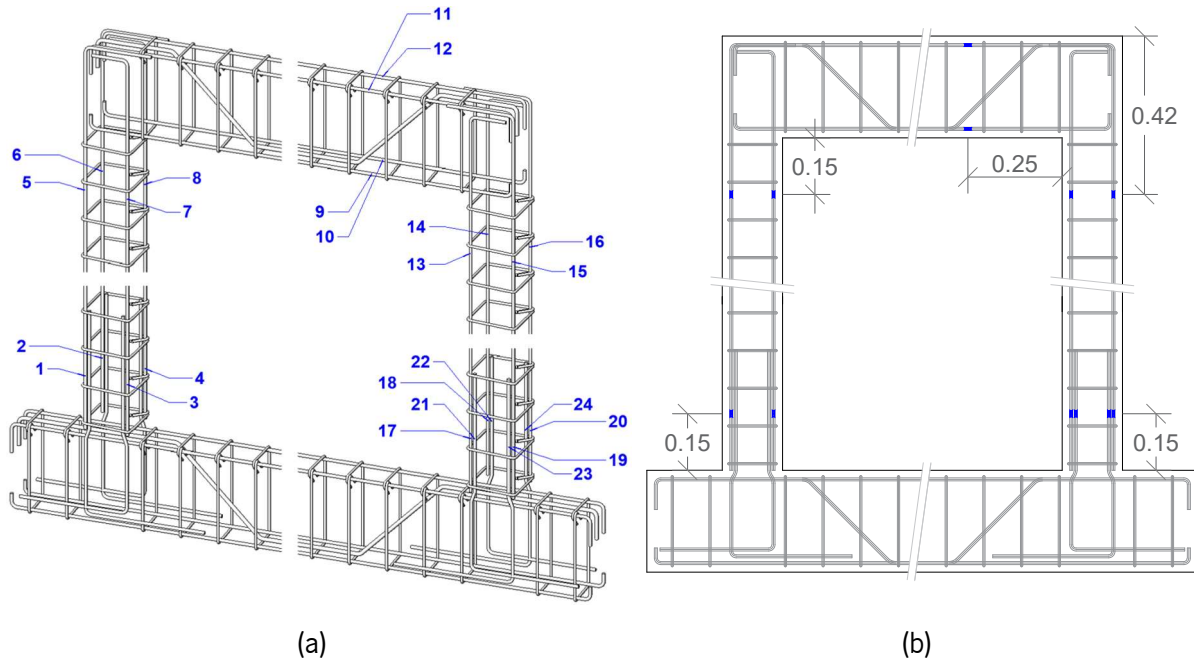


Figure 4.13: Location of strain gauges: (a) 3D representation of rebar; (b) 2D representation with distance to RC frame joints [dimensions in mm].

4.2.6 Load protocol

The in-plane quasi-static cyclic tests were performed under displacement control, imposed by an actuator (see detail 7 in Figure 4.12) and controlled by an LVDT that was connected to it (u_1 , Figure 4.12). The adopted displacement protocol, presented in Figure 4.14, is defined by increasing levels of displacement amplitude, which are pre-established according to the recommendations provided by FEMA 461 (2007).

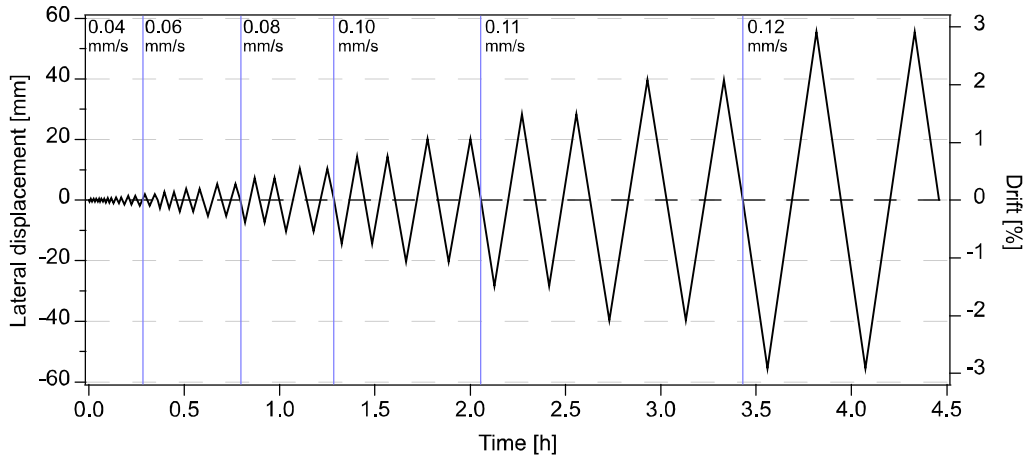


Figure 4.14: Displacement history adopted for in-place quasi-static cyclic tests.

In total, the loading protocol includes fifteen different steps, starting with a displacement amplitude of 0.5 mm. The displacement amplitude of each subsequent step is defined as 1.4 times the amplitude of its preceding step, up to a maximum displacement equal to 55.6 mm (see displacement history in Figure 4.14), which represents 2.9% of lateral drift, defined as $\delta = u_1 / h$ (ratio between the lateral displacement at the top of the RC frame, u_1 , and the height of the horizontal load application relative to the base of the RC frame, h - see Figure 4.12). Each step is composed by two cycles, with exception of the first step, which comprises six cycles. In terms of displacement rate, the tests start with a velocity of 0.04 mm/s, which gradually increases along the steps, reaching 0.12 mm/s at the end of the cyclic tests.

As mentioned in Section 4.2.1, the cyclic tests were performed on a total of three RC frame specimens, following the load protocol described above. However, it should be noted that, for specimens M and S, the tests were interrupted for a maximum lateral displacement of 39.69 mm (2.1% of lateral drift) due to the extension and severity of the damage presented by those specimens (at this lateral displacement, the lateral load was much higher in these specimens than in the bare RC frame – see subsection 4.3.2).

4.3 Experimental results and discussion

4.3.1 Overview

This section presents and discusses the results that were obtained from the in-plane cyclic tests. The in-plane performance of the tested specimens is systematically compared by analysing different performance indicators that reflect their seismic behaviour, namely: (i) hysteretic load-displacement curves (subsection 4.3.2); (ii) crack patterns and failure modes (subsection 4.3.3); (iii) strength degradation through the analysis of the load *vs.* lateral drift envelopes for first and second cycle repetitions (subsection 4.3.4); (iv) degradation of secant stiffness (subsection 4.3.5); (v) displacement ductility factor (subsection 4.3.6); (vi) energy dissipation capacity (subsection 4.3.7); (vii) column deformation throughout the cyclic tests (subsection 4.3.8); and (viii) strain of steel rebars in critical sections of the RC frames (subsection 4.3.9).

4.3.2 Hysteretic load-displacement curves

The individual lateral load *vs.* displacement (u_l , Figure 4.12) curves obtained for specimens R, M and S are presented in Figures 4.15a, 4.15b and 4.15c, respectively. Figure 4.15d shows the overlapping of curves from the different series for direct comparison of the aforementioned results. The positive and negative directions are defined by the movement of the hydraulic actuator, corresponding respectively to the pushing and pulling of the specimen by the actuator.

For an easier comparison of the tested specimens' in-plane behaviour, and in order to complement the analysis of the hysteretic load-displacement curves, Table 4.4 presents some performance indicators that were computed based on them for the identified loading directions: (i) the maximum in-plane loads obtained for the first and second cycles, respectively, $F_{\max 1}$ and $F_{\max 2}$; (ii) the lateral displacement corresponding to the maximum load levels, $d_{F_{\max}}$ ($d_{F_{\max 1}} \approx d_{F_{\max 2}}$); and (iii) the initial elastic stiffness that characterizes the linear-elastic phase of the obtained cyclic behaviours, k_{el} .

Overall, the lateral load-displacement diagrams present an initial linear/elastic behaviour, followed by an increasingly nonlinear response due to the onset of cracking and continuous development of damage in the tested specimens. An immediate observation is the significant increase in terms of stiffness and lateral load carrying capacity provided by both the masonry infill wall and the proposed sandwich panel. The highest elastic stiffness was obtained for specimen M, which reflects the relevance of the structural

contribution of masonry infill walls regarding the lateral behaviour of RC frame buildings, already reported in several works (Akhoundi *et al.*, 2018; Dautaj *et al.*, 2018; Milheiro *et al.*, 2016). In fact, specimen M registered the highest lateral strength until about +0.4% (7.4 mm) and -0.5% (-10.2 mm) of lateral drift. The higher continuity of the contact between the masonry wall and the RC members, and its much higher thickness compared to the interior wythe of the sandwich panel justify these results, namely up to the initiation of cracking in the masonry wall (0.2% drift). For lateral displacements above this damage condition, specimen S presented the best performance in terms of lateral strength, with a more pronounced strength increase than specimen M (approximately 20%) for the positive loading direction.

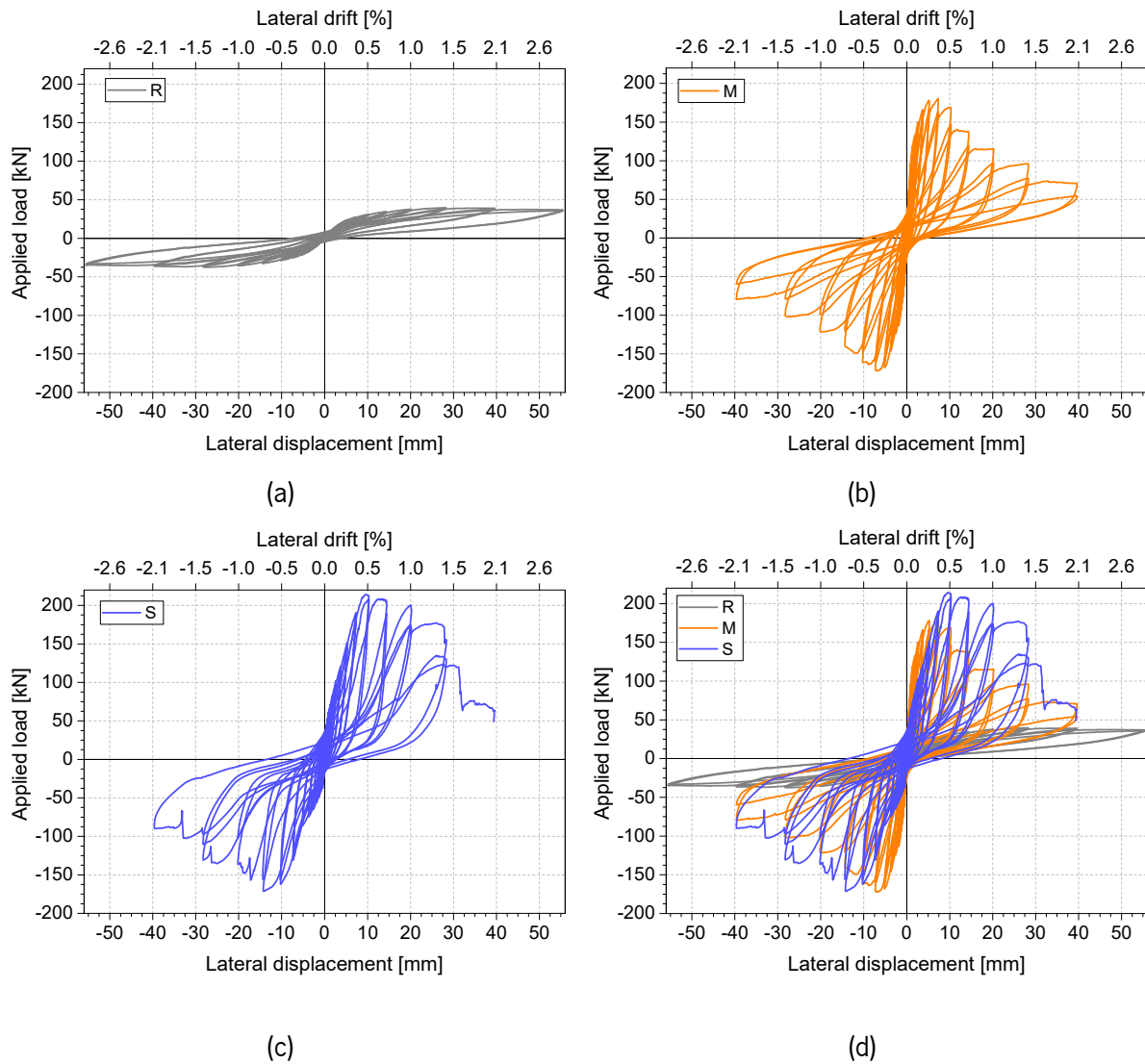


Figure 4.15: Lateral load-displacement curves: (a) specimen R; (b) specimen M; (c) specimen S; and (d) overlapped results.

Table 4.4: Specimens' in-plane performance: relevant results for negative and positive directions.

Specimen	Direction: positive				Direction: negative			
	k_{el}	F_{mx1}	F_{mx2}	d_{Fmax}	k_{el}	F_{mx1}	F_{mx2}	d_{Fmax}
	[kN/m]	[kN]	[kN]	[mm]	[kN/mm]	[kN]	[kN]	[mm]
R	6012.2	39.7	39.7	28.2	5266.7	-37.5	-36.1	-28.3
M	111355.3	178.1	161.5	7.4	95375.3	-168.9	-159.14	-7.3
S	89355.5	212.6	207.2	10.3	71800.2	-169.7	-154.8	-14.3

The hysteretic load-displacement diagrams and the values presented Table 4.4 also show that all specimens presented a somewhat asymmetric behaviour. This asymmetry in the curves is mostly related to the onset of damage for each cycle, reflecting the different damage state for each loading direction (asymmetric crack patterns and failure modes – see subsection 4.3.3). This uneven behaviour is more pronounced for specimen S and this is believed to be related to the inherent characteristics of the proposed strengthening approach, as the façade panel is applied on the front side of the RC frame (Figure 4.2), triggering an eccentric in-plane loading configuration and an even more asymmetric crack pattern and failure mode. It is noteworthy to mention that such issues should be mitigated in real applications, since the out-of-plane movement of the RC frames is restricted at each storey level by bracing elements, such as transverse beams and slabs.

4.3.3 Damage evolution and failure modes

The cracking pattern and damage evolution registered for specimens R, M and S, for different levels of lateral drift (namely 0.5%, 1.1% and 2.1%), are shown in Figures 4.16, 4.18 and 4.19, respectively. Due to the asymmetric nature of the sandwich panel and the associated strengthening process (it is idealized as a façade panel, hence the application to the front side of the RC frame), the crack pattern figures are presented for both sides of the specimen (front and back) in the case of specimen S.

For specimen R, cracking formation started at around 0.1% of drift, namely at the bottom end of one of the columns (right hand side in Figures 4.16 and 4.17), near the joint. The cracking pattern obtained for this specimen shows that all the cracks (flexural) are located in the columns (no relevant cracks formed in the beams), with higher damage concentration near the joints (see Figures 4.16c and 4.17a) that resulted in the development of plastic hinges at the bottom (more pronounced) and top ends of the columns. The obtained cracking pattern for the bare RC frame is a natural consequence of a design procedure conducted according to code provisions from the 1960s (*cf.* subsection 4.2.2), which promotes this type of behaviour and failure mode. In fact, the cracking configuration reflects a weak-column/strong-

beam structural behaviour, confirming that the RC frames designed for this study are indeed representative of the target building typology.

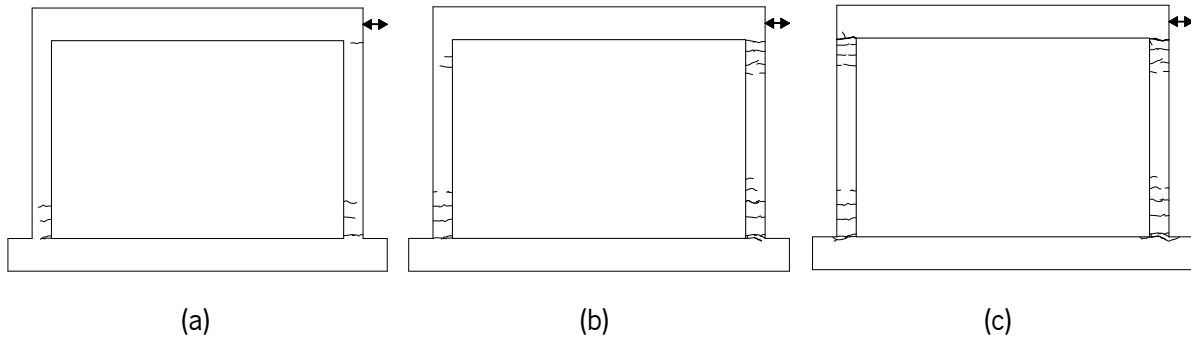


Figure 4.16: Damage evolution on specimen R: (a) 0.5% drift; (b) 1.1% drift; (c) 2.1% drift.

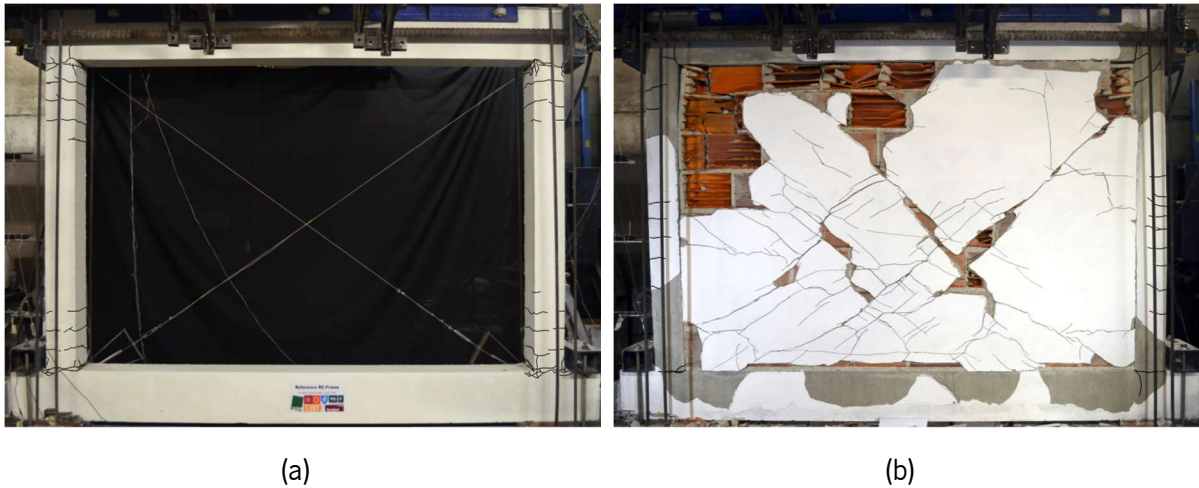


Figure 4.17: Damage state of the specimens for 2.1% of drift: (a) specimen R; (b) specimen M.

For specimen M, the cracking pattern can be described as follows: (i) cracking formation started at around 0.1% of drift, namely at the interface between the RC members and the masonry bricks, due to the separation between the RC frame and the infill wall; (ii) diagonal cracking in the masonry wall, which initiated at approximately 0.2% of drift and propagated steadily with increasing cycles; (iii) delamination of the plaster coating at 0.5% drift, for which a significant number of diagonal cracks were observed in the wall (see Figure 4.18a), and diagonal cracks started to form in the upper joints of the RC frame, namely in the junction between the upper beam and the columns; (iv) additional damage in the infill walls at 1.1% of drift (approximately 20 mm of imposed displacement), namely crushing of the brick units located near the upper joints of the RC frame (see darkened areas in Figure 4.18b), and formation of cracks along the columns; and (v) at 2.1% of drift, damage involved a vaster area of crushed brick units and the propagation of flexural cracks in the RC frame columns, (see Figure 4.18c and 4.17b) – these flexural cracks were located only in the outer part of the RC frame columns – the inner part registered

less damage due to the direct connection to the infill wall. Besides the flexural cracks in the columns, additional cracks could be observed, namely diagonal cracks near the joints of the RC frame.

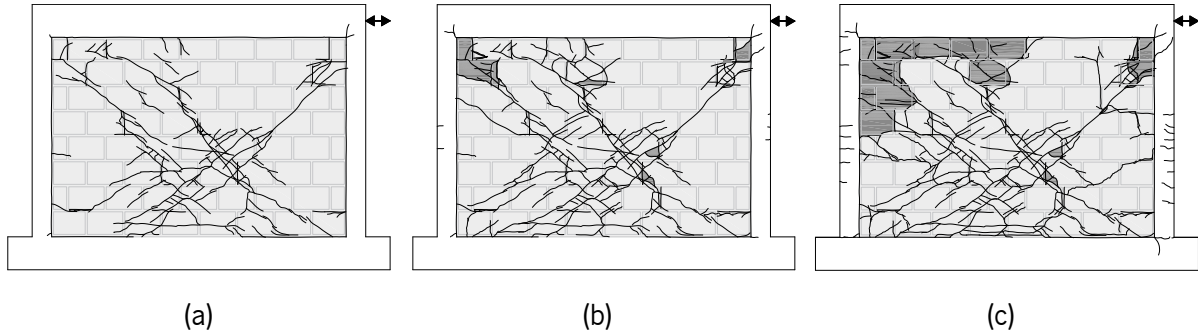


Figure 4.18: Damage evolution of specimen M (dark grey areas represent crushing of brick units): (a) 0.5% drift; (b) 1.1% drift; (c) 2.1% drift.

Finally, regarding specimen S, the sequence of the registered cracking pattern and damage modes were as follows (Figure 4.19 – note that the right hand side of Figures 4.19a, 4.19c and 4.19e corresponds to the left hand side of Figures 4.19b, 4.19d and 4.19f, respectively): (i) cracking initiation at approximately 0.2% of drift, namely through the appearance of two diagonal cracks, one at each bottom corner of the sandwich panel (between HIT anchors); (ii) cracking formation in the upper-left corner of the sandwich panel (visible in Figure 4.19b) for circa 0.3% of drift (also between HIT anchors located near the edge of the angle profiles), as well as initiation of cracking in the RC frame, namely at the bottom of the columns; (iii) development of diagonal cracking at the prominent part of the foundation beam of the RC frame when reaching 0.4% of drift (visible in the bottom left hand side of Figure 4.19a); (iv) for 0.5% of drift, additional cracks formed both in the sandwich panel, with new diagonal cracks originating from (and towards) other HIT anchor connection locations (see Figure 4.19b), and in the RC frame, namely in the bottom beam, in the top of the left column and in the bottom of the right column (as depicted in Figure 4.19a); (v) when the lateral drift of 1.1% is reached, the further propagation of the cracks mentioned for 0.5% of drift was clear, a macro-crack was formed in the top-left corner of the panel (see Figures 4.19d and 4.21a), additional diagonal cracks arose with origin on the HIT anchor connections located next to the mid-column-level, the corners of the panel's infill wythe started crushing (Figure 4.21b); new cracks were also formed in the top end of the RC frame columns (see Figure 4.19c), together with diagonal cracks in one of the RC frame upper joints (see Figure 4.19d); (vi) the damage state of the panel for 2.1% of drift involved further propagation of the aforementioned diagonal cracks in the sandwich panel, leading to the formation of even more crack patterns between HIT connectors, closer to the centre of the panel (see Figures 4.19f and 4.20b); and (vii) between 1.1% and 2.1% of drift, helicoidal cracks (due to the eccentric

disposition of the panel into the RC frame, generating torsional effects) occurred in the mid-level of the RC frame columns (visible in Figures 4.19e, 4.19f and 4.21c), as well as shear cracks and concrete cover spalling at the top-end of the columns (see top end of the columns in Figures 4.19e, 4.19f and 4.20b, on the left). It is noteworthy to mention that all the sandwich panel related damage described herein is strictly related to its interior/infill wythe, as no visible damage was detected on the exterior wythe (see Figure 4.20a).

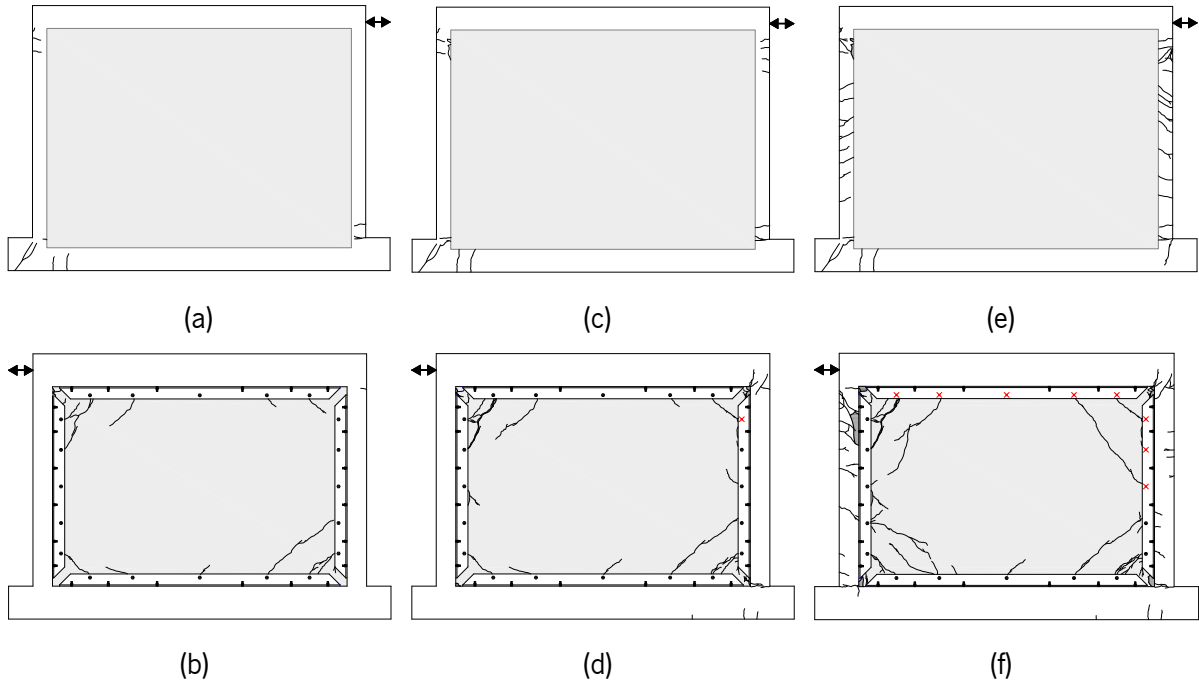


Figure 4.19: Damage evolution on specimen S (dark grey areas represent concrete crushing and spalling and red markers signalize failure of HIT anchors): (a) 0.5% drift (front); (b) 0.5% drift (back); (c) 1.1% drift (front); (d) 1.1% drift (back); (e) 2.1% drift (front); (f) 2.1% drift (back).

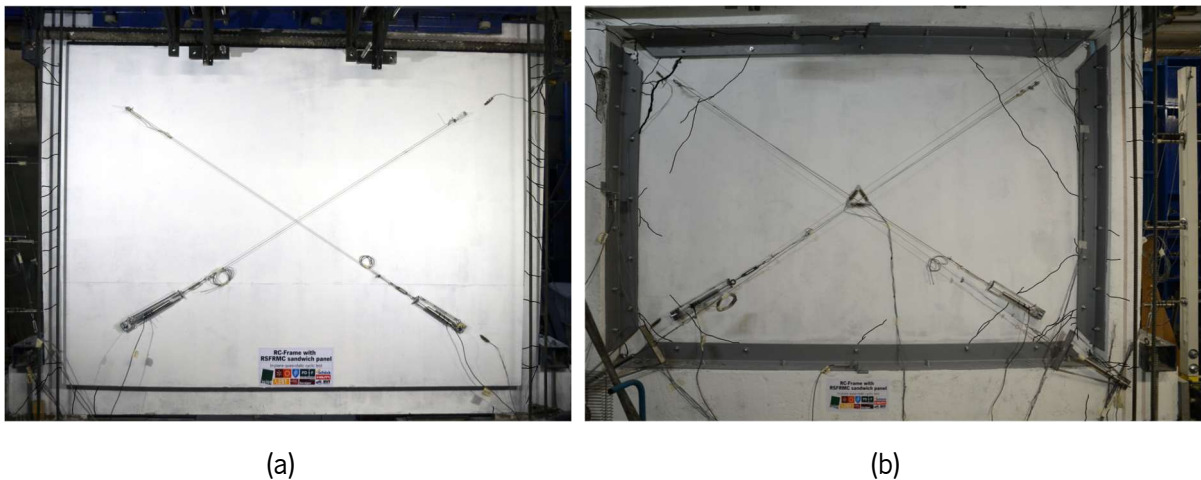


Figure 4.20: Damage state of specimen S for 2.1% of drift: front (a) and back (b) of the specimen.

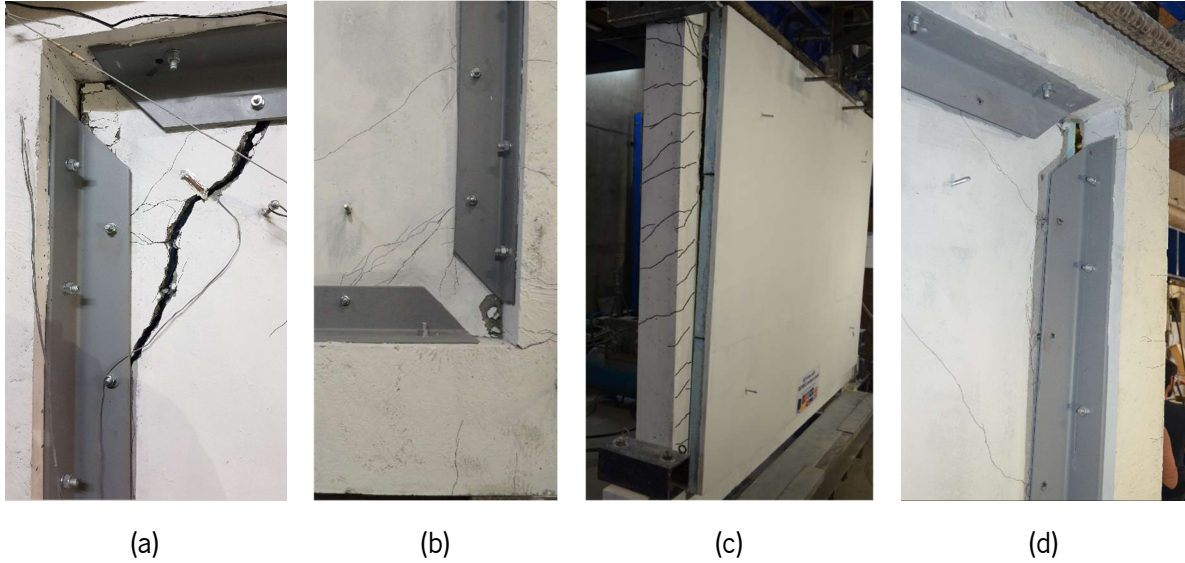


Figure 4.21: Details of the failure mode of the frame-panel connection in specimen S: (a) macro-crack; (b) crushing of panel corners; (c) torsional cracks and out-of-plane panel deformation; (d) failure of HIT anchors.

It is also important to highlight the failure mode registered for the sandwich panel connection, which was characterized by a gradual shear failure of the anchors that provided connection between the angle profiles and the frame frame/panel. The following remarks can be made: (i) first shear failure of a HIT anchor (located next to the upper-right corner of the panel – see red marker in Figure 4.19d) occurred before reaching 1.1% of lateral drift, namely for approximately -17 mm of imposed lateral displacement (which was reflected in a slight dip/reduction of lateral load, visible in the hysteretic load-displacement curve – see Figure 4.15c); (ii) shear failure of additional HIT anchors located in the top part of the specimen occurred between 1.1% and 2.1% of drift, namely for +20 mm, -27 mm, +32 mm and -33 mm of lateral imposed displacement (see corresponding load reductions in Figure 4.15c); (iii) the rupture of HIT anchors caused increasing out-of-plane deformation of the upper part of the sandwich panel, which is visible in Figures 4.21c and 4.21d). The referred out-of-plane movement is believed to be a result of the aforementioned eccentricity in the in-plane load application. The eccentric load justifies also the formation of torsional cracks in the RC columns, as mentioned previously (see Figure 4.21c).

The explicit comparison between the damage state evolution of the RC frames reveals different cracking evolution for specimens M and S, with formation of flexural and torsional cracks in the columns, respectively. In all cases, damage evolution was controlled and progressive, but the sandwich panel connection contributed to higher damage accumulation, leading to wider cracks and concrete cover spalling phenomena in the columns. Nonetheless, specimen S presented a more distributed crack

pattern, with cracking propagation not being restricted to the column members (various cracks also developed in the foundation beam).

4.3.4 Strength degradation

Figure 4.22 presents the monotonic envelopes of the load-displacement diagrams for successive cycles of the various tested specimens. It is possible to observe that the strength degradation from the first to second cycles is negligible approximately up to the peak load of the obtained responses (up to 0.25% and 0.50% of lateral drift for specimens M and S, respectively). The post-peak response was associated to higher levels of strength degradation, particularly for specimens M and S, which naturally increased with the drift due to damage accumulation. Strength loss was similar for specimens M and S, with average strength degradation after the peak load of 13% and 12%, respectively.

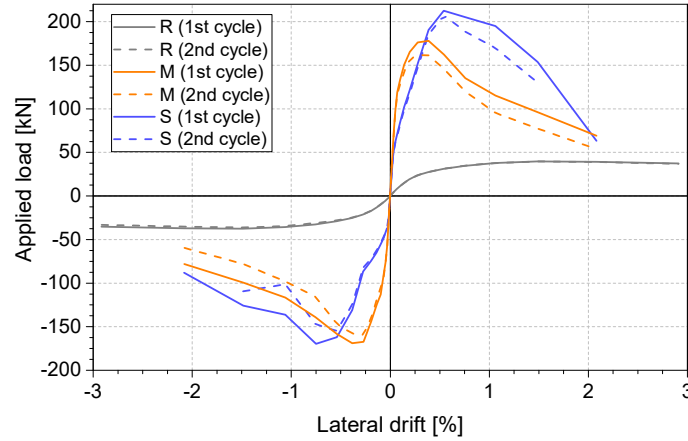


Figure 4.22: Strength degradation *vs.* lateral drift, in successive (1st and 2nd) cycles.

4.3.5 Stiffness degradation

Figure 4.23b presents the degradation of the lateral secant stiffness, k_s , with the evolution of the lateral drift. The secant stiffness is determined by the slope of the straight line defined by the two points that represent the peak loads for the positive and negative lateral load *vs.* displacement envelopes outlined for each cycle of the hysteretic responses of the tested specimens, as represented in Figure 4.23a.

The obtained stiffness degradation evolution shows that all specimens present a continuous stiffness decrease with the increase of lateral drift, this decrease being particularly steep in the case of specimens M and S, mainly due to the higher damage accumulation in the masonry infill wall and in the sandwich panel, addressed in subsection 4.3.3. Specimen M presents a well-defined exponential reduction trend,

which is in agreement with the results obtained in other research works (Akhoundi *et al.*, 2018; Martins, 2018). As expected, the presence of infill walls (masonry or sandwich) entails a significant enhancement of the in-plane stiffness when compared to the bare RC frame response, especially for the initial cycles leading to peak load – circa 15 and 11 times higher in the case of masonry and sandwich panel infill, respectively. In the lateral drift interval of 0.25% to 0.50%, the rate of secant stiffness degradation for specimen S was significantly less pronounced than for specimen M – this stems from the much higher post-cracking tensile strength and energy absorption capacity of RSFRC compared to masonry brick units. Above a drift of approximately 0.42%, the secant stiffness of specimen S became higher in relation to specimen M. However, at the final drift all tested prototypes had virtually similar secant stiffness.

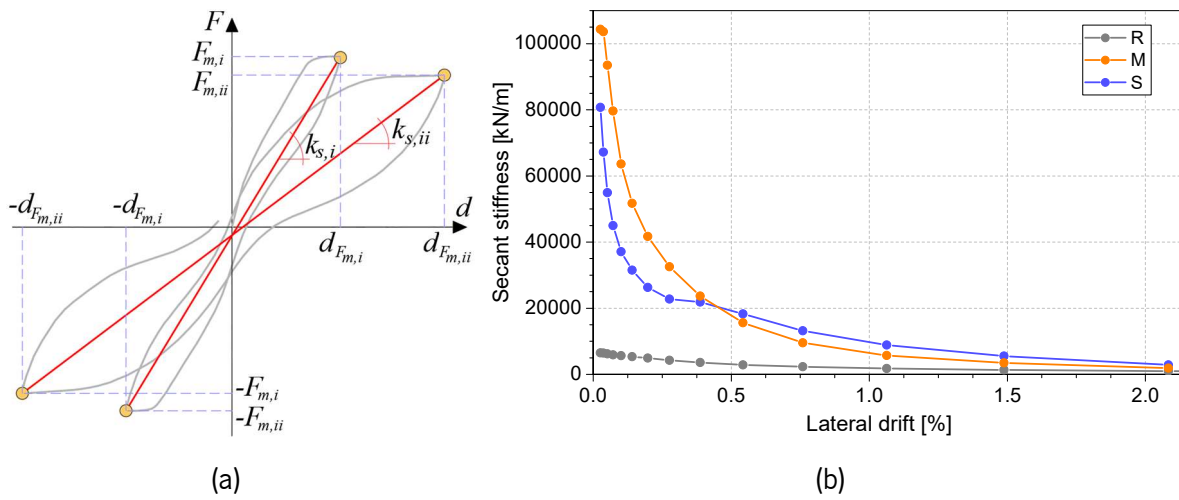


Figure 4.23: Stiffness degradation vs. lateral drift: (a) schematic illustration; (b) experimental results.

4.3.6 Displacement ductility

To assess the capacity of a structure to undergo large deformations during the post-peak phase while maintaining a high percentage of its ultimate lateral load capacity, it is common to determine the displacement ductility factor and the associated parameters, which can be derived from the structure's cyclic response (Esmaeeli *et al.*, 2015). The displacement ductility factor, μ_{Δ} , is determined by the ratio between the ultimate lateral displacement, d_u , and the displacement at yield point, d_y . Following the schematic illustration presented in Figure 4.24a, d_u is defined by a point in the post-peak response of the specimens that corresponds to a slight reduction, typically between 10% and 25% (Esmaeeli *et al.*, 2015; CEN, 2005; Park, 1988) (20% in the case of this study, $\beta = 0.8$) of the maximum lateral load, $F_{\max}$; d_y is determined by an equivalent bilinear curve defined through the fulfilment of the following conditions: (i) the sum of the differences of the areas enclosed between the experimental and the

equivalent bilinear curve should be equal to zero, $\sum A_i = 0$; and (ii) the deviation between the experimental and equivalent curves is minimized, $\min \{ \sum |A_i| \}$.

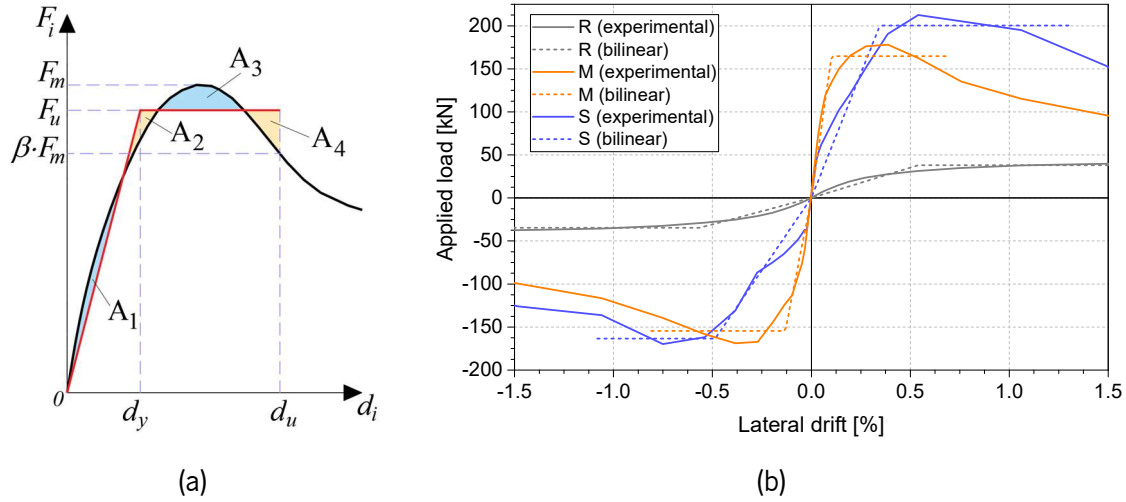


Figure 4.24: Displacement ductility: (a) schematic illustration; (b) experimental load.

Table 4.5: Displacement ductility factor and parameters.

Specimen	Direction: positive		Direction: negative		μ_{Δ}
	d_y [mm]	d_u [mm]	d_y [mm]	d_u [mm]	
R	10.3 (0.54%)	55.5 (2.92%)	-10.7 (-0.56%)	-55.4 (-2.91%)	5.3
M	1.9 (0.10%)	13.4 (0.70%)	-2.5 (-0.13%)	-15.4 (-0.81%)	6.5
S	6.6 (0.35%)	25.1 (1.32%)	-9.2 (-0.49%)	-20.6 (-1.08%)	3.0

Figure 4.24b presents the obtained bilinear elastic-perfectly plastic curve plotted together with the load-displacement envelopes. The computed values of d_y and d_u for positive and negative directions and the resultant μ_{Δ} (average value based on positive and negative directions) are presented in Table 4.5. When comparing the unreinforced specimen R to specimens M and S, it can be seen that the addition of masonry infill wall caused an increase in ductility of 23%, while the sandwich panel caused a reduction in ductility of 43%. The high ductility factor obtained for specimens M can be explained by the higher initial stiffness that characterizes its behaviour (see subsection 4.3.5), which yields considerably lower values of d_y (hence affecting μ_{Δ} positively). Although the lowest value of μ_{Δ} was registered for specimen S, it is noteworthy to mention that, in comparison to specimen M, much higher values of ultimate lateral displacement were obtained for both loading directions (89% and 34% higher for positive and negative directions, respectively).

4.3.7 Energy dissipation

Figure 4.25 shows the relationship between the dissipated energy and the accumulated lateral deflection, which was determined by adding the absolute values of the lateral displacements measured in all cycles using the LVDT that monitored the horizontal movement of the upper RC beam at its loaded end (u_1 , Figure 4.12). The energy dissipation recorded by a lateral load resisting system provides an indication of its capacity to sustain loading demands resembling seismic events. The amount of dissipated energy was computed from the enclosed area corresponding to each loading cycle of the lateral applied load *vs.* displacement hysteretic curves presented in Figure 4.15.

It can be seen that both masonry and sandwich panel solutions provided a much higher energy dissipation capacity in comparison to the bare RC frame specimen, which is in agreement with the obtained hysteretic curves. In fact, quite similar results were obtained in this respect for specimens M and S up to around 0.5 m of accumulated lateral deflection, for which the dissipated energy was approximately 10 times higher than for specimen R. For increasing accumulated lateral displacements, specimen S provided the highest amount of amount of dissipated energy: for 0.9 m of accumulated lateral deflection, the maximum dissipated energy of specimen S was 26.4 kN.m, which is 28% and 700% higher compared to specimens M and R, respectively.

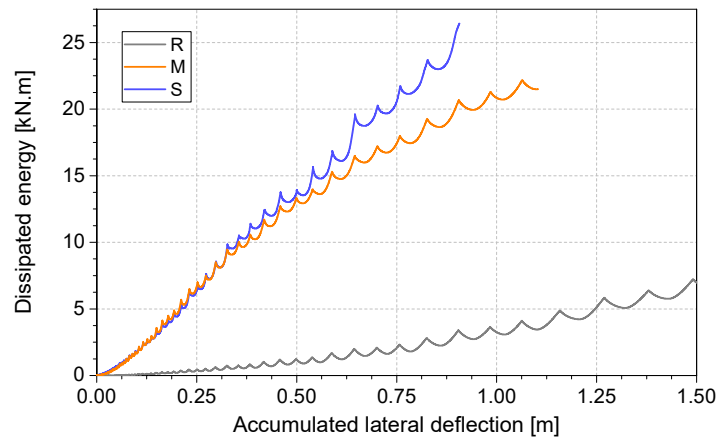


Figure 4.25: Dissipated energy *vs.* accumulated lateral deflection.

4.3.8 Column deformation *vs.* drift level

Figures 4.26a, 4.26b and 4.26c present the column deformation of the tested RC frames for different levels of lateral drift, obtained from the four measuring points distributed along the column height (see u_2 , u_3 , u_4 and u_5 , in Figure 4.12), for specimens R, M and S, respectively. For an easier comparison,

Figure 4.26d presents the results for these three specimens, namely for two specific phases of the cyclic tests, 0.5% and 2.1% of lateral drift, which correspond roughly to lateral deformations at peak load and to maximum lateral deformation in the case of specimens M and S, respectively.

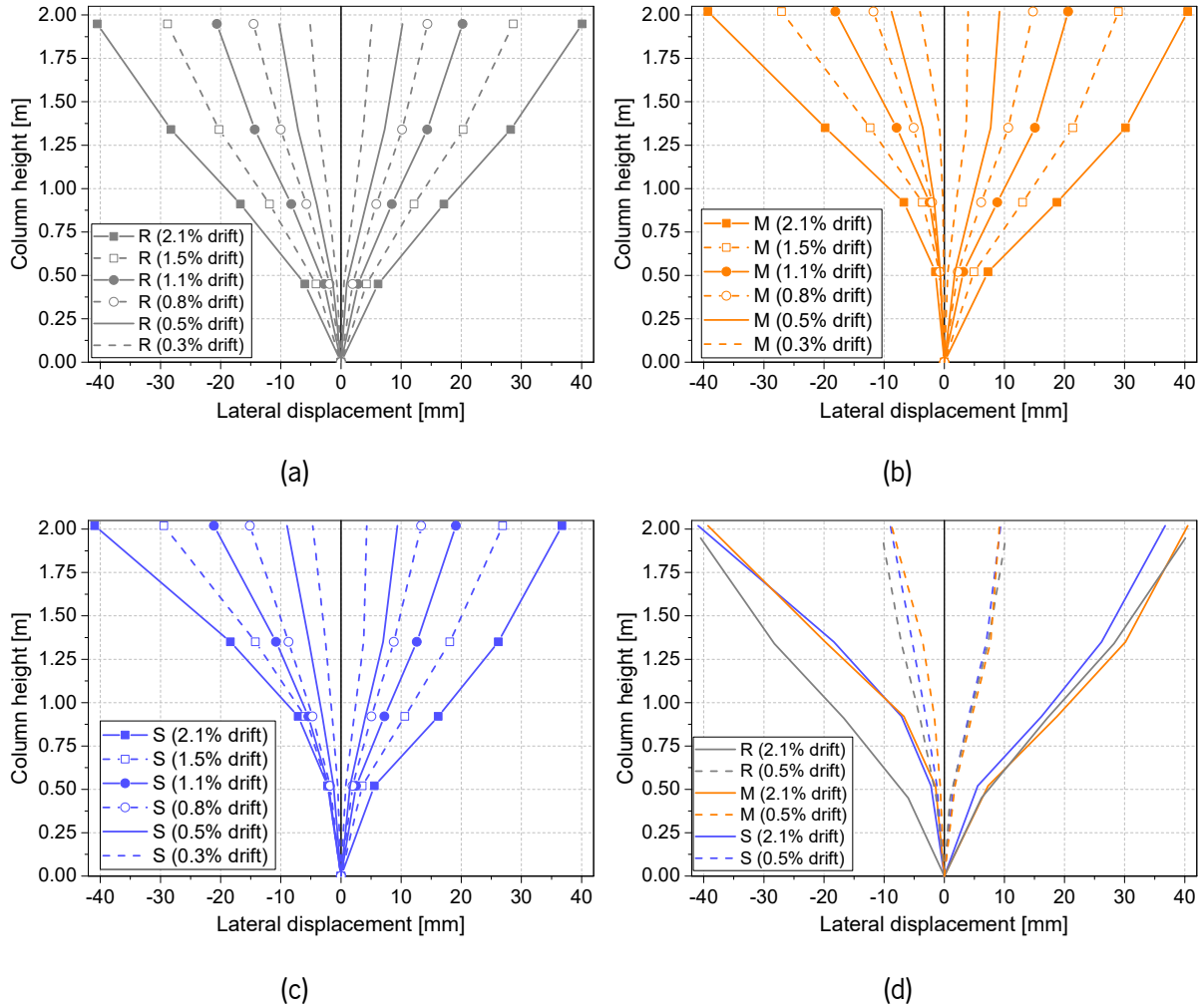


Figure 4.26: Column deformation along its height for different lateral drift levels: (a) specimen R; (b) specimen M; (c) specimen S; and (d) overlapped results.

It is possible to observe that the columns of specimen R registered a practically symmetric deformed shape for positive and negative directions, which was not the case for the two other specimens. In fact, a similar asymmetric deformed shape was registered for the columns of specimens M and S, with lower lateral displacements at the bottom half of the columns for the negative loading direction. This is clearly visible in Figure 4.26d, especially for high levels of lateral displacement: as an example, for 2.1% of lateral drift, the specimen S reached -2.2 mm of displacement at 0.5 m of column height, which is about 36% of the value measured in the case of specimen R (-6.0 mm). The reasons for these differences in the behaviour of specimens M and R (asymmetry and lower lateral displacements in bottom part of the

columns) are three-fold: (i) the damage underwent by the infill wall and sandwich panel in the first incursions for a given drift in the negative direction, which explains the asymmetry in the column's response; (ii) the contribution of the masonry wall and sandwich panel to resist lateral loads and prevent lateral deformations of the columns (namely in their bottom part), as well as the damage they underwent under lateral loads – note that higher concentration of damage was identified in the top half of specimens M (crushing of brick units – see Figures Figure 4.17b and 4.18c) and S (shear failure of HIT anchors in the upper part of the specimen – see Figure 4.19f); and (iii) the formation of compressive diagonal struts in the masonry infill wall and in the sandwich panel, during the cyclic tests, which for the positive loading direction introduce a lateral pressure in the bottom part of the left column (leading to higher lateral deformation), as represented in Figure 4.27a (this lateral pressure is not as intense when the lateral load is applied in the negative direction – see Figure 4.27b). These differences are not so evident for lateral drifts below 0.5%, which also supports the aforementioned hypothesis (less overall damage at this stage of the experiments). Furthermore, the measured in-plane column deformation also explains the crack patterns observed for these columns, with (i) more uniform damage distribution for specimen R (see Figure 4.16c), and (ii) higher damage concentration in the upper part of the columns in the remaining specimens (see diagonal cracks in Figures 4.18c and 4.19e).

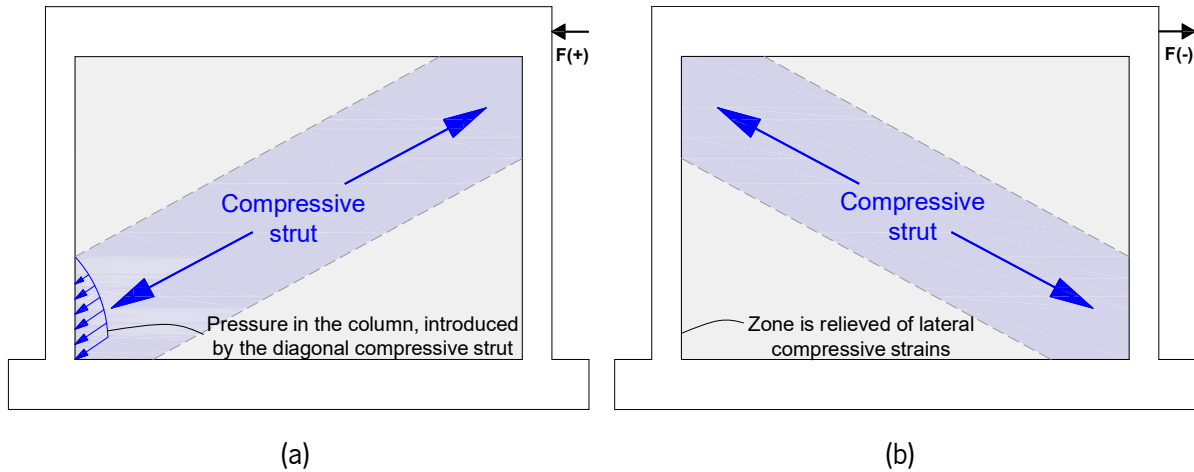


Figure 4.27: Formation of compressive diagonal struts in specimens M and S for the positive (a) and negative (b) loading directions.

4.3.9 Strain vs. drift in reinforcing bars

This subsection presents the results obtained from the strain gauges that were installed in the rebars of the tested RC frame specimens (see Figure 4.13). The results are shown in different figures, divided

according to clusters of sensors that are located in the same section of the column/beam members, near the joints, namely: (i) bottom of the columns (see Figures 4.28d and 4.28e); (ii) top of the columns (see Figures 4.28a and 4.28c); and (iii) upper beam (see Figure 4.28b). The curves are defined by points that represent the average tensile strain (average value for the sensors that are illustrated in those figures) registered at the maximum imposed lateral drift that characterizes each loading cycle. For comparison purposes, it should be mentioned that an average yield strain, ε_y , of 3.16‰ (see red line in Figures 4.28a-4.28e) was obtained during the characterization of the steel rebars mentioned in subsection 4.2.4.1.

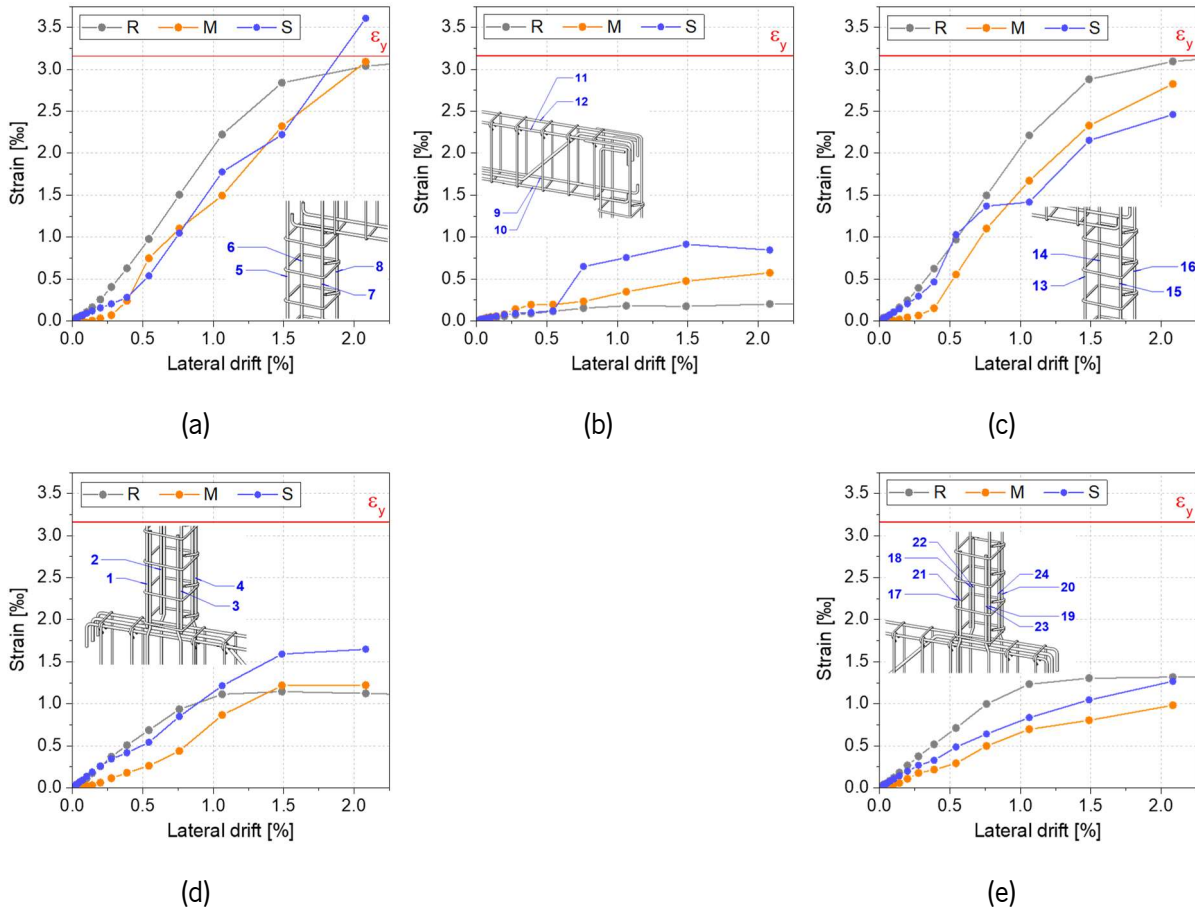


Figure 4.28: Elongation of steel rebars throughout cyclic tests: (a) sensors 6-7; (b) sensors 9-12; (c) sensors 13-16; (d) sensors 1-4; (e) sensors 17-24.

The results obtained show that, overall, the maximum tensile strains throughout the cyclic tests are generally higher for specimen R (top of the columns), showing the beneficial effect of infill walls and sandwich panel strengthening in reducing the tensile strains that develop in these RC structural members. A few exceptions can be detected for later stages of the cyclic tests (for lateral drift higher than 1.5% - see Figures 4.28a and 4.28d), which are associated to high damage concentration both in the masonry walls

(large areas with crushed bricks) and in the sandwich panel (failure of anchors that ensure panel-frame connection). In fact, the maximum tensile strain was measured in specimen S, namely in the top of one of the columns (see Figure 4.28a), reaching 3.6%, which is higher than the yield strain. This can be related to the concrete cover spalling that occurred in these parts of the columns (cf. subsection 4.3.3 and Figures 4.19f and 4.20b).

For all specimens, the maximum strains in the bottom of the columns (Figures 4.28d and 4.28e) were approximately half of those measured in the top of the columns (Figures 4.28a and 4.28c), remaining well below the yield strain. This can be explained essentially by the lap splice region located at the bottom end of the columns – in other words, the tensile load was shared by more rebars.

The results measured by strain gauges located in the upper beam of the RC frames, depicted in Figure 4.28b, show that in all specimens the maximum strains remained well below the yield strain. In addition, for lateral drifts higher than 0.5%, higher tensile strains were measured in specimen S. This is in agreement with the cracking pattern observed in this specimen (see subsection 4.3.3), with a more distributed damage between beams and columns.

4.4 Concluding remarks

The work presented in this chapter is essentially centred on the in-plane cyclic tests that were performed on RC frame specimens to evaluate the seismic performance of the proposed rehabilitation solution. From the work that was carried out, the following main conclusions can be pointed out:

1. The RC frame with masonry infill wall presented the highest elastic stiffness, which emphasizes the structural contribution of masonry infill walls for the lateral behaviour of RC frame buildings (commonly disregarded during the design process of the building structure).
2. The RC frame strengthened with the sandwich panel prototype presented the highest lateral strength, namely for imposed lateral drifts higher than $\pm 0.5\%$ (± 10.3 mm). For the initial phase of the cyclic tests (up to $\pm 0.5\%$ of lateral drift), the specimen with masonry infill wall registered higher levels of lateral strength.
3. Overall, the proposed sandwich panel strengthening system proved to be an effective solution, providing the best performance in comparison to the bare RC frame and the traditional masonry infill wall solution, in terms of both load bearing capacity (395% and 10% increase, respectively) and energy dissipation (700% and 28% increase, respectively).

4. The in-plane loading applied to the sandwich panel specimen involved some eccentricity, as a consequence of the panel application to the “front” face of the RC frame. Such asymmetric strengthening configuration caused torsional cracking in the RC frame columns and out-of-plane deformation of the sandwich panel; in real applications, such torsional deformations should be prevented by transverse beams and slabs.
5. The damage evolution in the RC frames revealed the formation of flexural and torsional cracks in the columns of the RC frames with masonry infill and sandwich panel strengthening, respectively. Although the sandwich panel connection led to higher damage (wider cracks and concrete cover spalling), it also led to a more distributed crack pattern (cracking propagation in beam and column members).
6. The axial strains recorded in the steel rebars show that, in general, higher tensile strain values were obtained in the rebars of the columns of the bare RC frame, which shows the beneficial effect of infill walls and/or panel strengthening in reducing tensile stresses in the columns’ longitudinal reinforcement.

5 NUMERICAL SIMULATIONS

5.1 Introduction

This chapter presents the numerical study that was conducted in the scope of this research work. The chapter is divided into three parts, associated to different stages of this work's experimental development: (i) section 5.2, which is focused on the study of the bond behaviour between RSFRC and polystyrene through numerical simulations that aim at modelling the experiments described in subsection 3.3.2; (ii) section 5.3 aims at numerically simulating the experimental behaviour observed in the cyclic tests performed on the bare RC frame, addressed in subsection 4.3.2; and (iii) section 5.4, which presents design-oriented numerical simulations that were performed to assist the sandwich panel design process. All the numerical FEM-based models were deployed using *FEMIX 4.0* (Azevedo *et al.*, 2003; Sena-Cruz *et al.*, 2007; Ventura-Gouveia, 2011), a FEM-based computer program, together with the pre- and post-processing software *GiD* (version 14).

5.2 Bond behaviour between RSFRC and polystyrene

5.2.1 Overview

To assess the relevant properties of the bond conditions between RSFRC layers and the core material, material nonlinear FE analyses were carried out with the FEMIX computer program (Barros, 2016). The information derived from the analyses can be of paramount relevance in future numerical simulations, namely to assess the strengthening effectiveness of the sandwich panel that has been developed regarding the seismic retrofitting of the target buildings referred in Section 0. Taking into account the pushout tests that were performed for selection of the core material, the numerical simulation reported herein attempts to capture the experimental behaviour of specimen typology FCØ10-H25-XPS3 (the best performing one in terms of bond between RSFRC and polystyrene). As mentioned in subsection 3.3.2.2, the failure mechanisms of this specimen were governed by a coupled effect of sliding-opening at the interface between the bottom layer of the RSFRC and XPS-3, together with the rupture of the connector at relatively large sliding of the top RSFRC layer.

To illustrate the importance of modelling the interface between RSFRC and XPS layers, Figure 5.1 presents the experimental findings that were obtained in the pushout tests for the specimens with (FCØ10-H25-XPS3) and without (FCØ10-H25) polystyrene layer. It can be seen that the increment in shear resistance, represented by the red area enclosed between the two curves, can be attributed, as discussed in chapter 3, to the interface between the referred materials/layers. Therefore, to model these complex failure mechanisms, a constitutive model for a 3D interface finite element (IFE) was selected, capable of coupling the opening-sliding effects by considering the tensile strength, cohesion and internal friction angle of the interface, as well as the evolution of those parameters during the opening-sliding process.

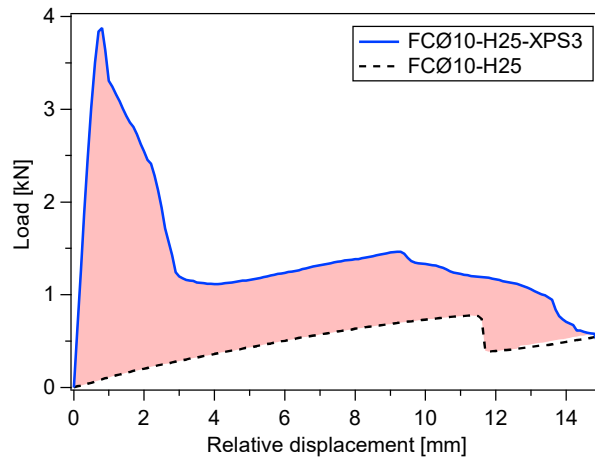


Figure 5.1: Contribution of the polystyrene core layer for the shear resistance on specimen FCØ10-H25-XPS3 (red area).

The failure modes discussed in subsection 3.3.2.2 (illustrated in Figure 3.8) stress the importance of considering an interface model that captures opening-sliding effects, since the occurrence of gradual debonding between the bottom RSFRC and the contacting XPS layers was reported. In fact, the contact area in the RSFRC/XPS interface can be taken into account through a simplified analytical approach, expressed by the following equation:

$$F_{\text{exp}} = A_{ce} \cdot c + k_{con} \cdot u \quad (1)$$

where F_{exp} is the shear load measured experimentally (from FCØ10-H25-XPS3 curve in Figure 5.1), A_{ce} is the effective area of contact between the referred RSFRC and XPS layers, c is the material cohesion and $k_{con} \cdot u$ is the load contribution provided by the F-GFRP connector, with k_{con} and u being the stiffness of the connector and the relative displacement between RSFRC layers, respectively. The effective

contact area decreases with the increase of relative displacement due to shear loading, which can be estimated through Equation (2), as presented in Figure 5.2. It can be seen that the estimated contact area is approximately 80% of the initial one at peak load and only 20% for a relative displacement equal to 2.5 mm.

$$A_{ce} = \frac{F_{\text{exp}} - k_{\text{con}} \cdot u}{c} \quad (2)$$

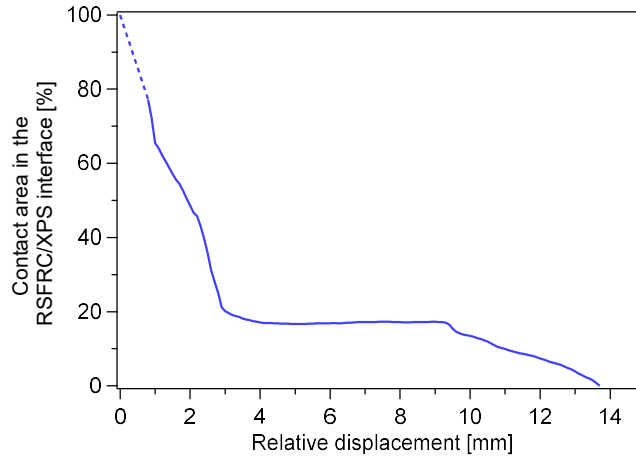


Figure 5.2: Estimation of contact area reduction with the increase of relative displacement during pushout tests (specimen FCØ10-H25-XPS3).

5.2.2 Constitutive model for surface interface finite element

In this section a brief overview of the main features of the adopted constitutive model is provided. Further details can be consulted elsewhere (Caballero *et al.*, 2008; Coelho, 2016). The constitutive model is prepared for simulating the contact between bodies discretized by solid finite elements, therefore three stress components ($\underline{\sigma} = [\sigma_n \quad \tau_{t_1} \quad \tau_{t_2}]^T$) and corresponding displacements ($\underline{u} = [u_n \quad u_{t_1} \quad u_{t_2}]^T$) can be installed in the corresponding IFE (see Figure 5.3).

Before the criterion of movement separation of the faces of this IFE has been violated, this “zero-thickness” IFE is modelled by the following linear-elastic stiffness matrix:

$$K_{el} = \begin{bmatrix} k_n & 0 & 0 \\ 0 & k_{t_1} & 0 \\ 0 & 0 & k_{t_2} \end{bmatrix} \quad (3)$$

where k_n , k_{t_1} and k_{t_2} represent the elastic stiffness in the orthogonal, and in the t_1 and t_2 tangential directions, respectively. Adequate values for these elastic stiffness components should be provided to avoid interpenetration of the bodies in contact (too low values) or numerical instabilities (too high values).

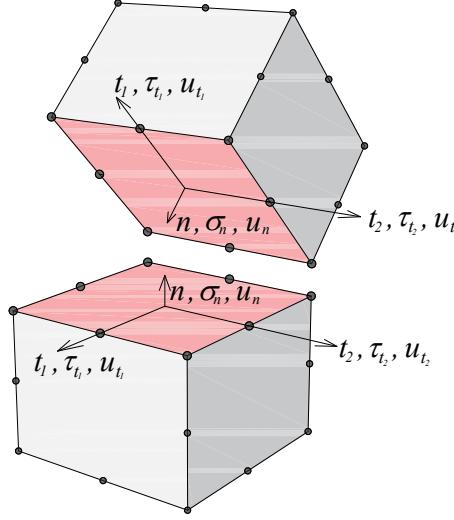


Figure 5.3: Stress and displacement components in the surface interface finite element.

To model the movement of the two faces in contact of the IFE, the following hyperbolic cracking failure surface is adopted (Caballero *et al.*, 2008):

$$F(\underline{\sigma}, \underline{r}_F) = -(c_F - \sigma_n \tan \phi_F) + \sqrt{(\tau_{t_1})^2 + (\tau_{t_2})^2 + (c_F - f_{t,F} \tan \phi_F)^2} \quad (4)$$

where $f_{t,F}$ is the tensile strength, c_F is the asymptotic cohesion, and $\tan \phi_F$ is the tangent of the asymptotic friction angle, being collected in the vector $\underline{r}_F = [f_{t,F}, c_F, \tan \phi_F]^T$.

For modelling the dilatancy effect due to the roughness of the faces in contact, which leads to the separation of these faces (opening) with the sliding, a process that is also influenced by the applied orthogonal stress, σ_n , a non-associated formulation is adopted by considering the following plastic potential function:

$$P(\underline{\sigma}, \underline{r}_P) = -(c_P - \sigma_n \tan \phi_P) + \sqrt{(\tau_{t_1})^2 + (\tau_{t_2})^2 + (c_P - f_{t,P} \tan \phi_P)^2} \quad (5)$$

that is formally equal to the cracking failure surface (Eq. (4)), but now defined from the parameters $\underline{r}_P = [f_{t,P}, c_P, \tan \phi_P]^T$. In the present formulation it is assumed that $f_{t,F} = f_{t,P} = f_t$.

The evolution of the cracking surface, $F = 0$, and plastic potential, $Q = 0$, is governed by the hardening parameters, $\underline{r}_\alpha = [f_t, c_\alpha, \tan \phi_\alpha]^T$ with $\alpha = F$ or P , according to the following equations:

$$\xi_I = \begin{cases} \frac{1}{2} - \frac{1}{2} \cos\left(\frac{\pi W^{cr}}{G_{F,I}}\right) & 0 \leq W^{cr} \leq G_{F,I} \\ 1 & W^{cr} > G_{F,I} \end{cases} \quad (6)$$

and

$$\xi_{II} = \begin{cases} \frac{1}{2} - \frac{1}{2} \cos\left(\frac{\pi W^{cr}}{G_{F,II}}\right) & 0 \leq W^{cr} \leq G_{F,II} \\ 1 & W^{cr} > G_{F,II} \end{cases} \quad (7)$$

where $G_{F,I}$ and $G_{F,II}$ are the fracture energy in mode I and mode II, respectively, being assumed that $G_{F,II}$ is the same in the t_1 and t_2 tangential directions to the plane of the interface. In Eqs. (6) and (7), W^{cr} is the work produced during the fracture process, being the unique internal history variable controlling the evolution of cracking surface and potential plastic functions, determined from:

$$dW^{cr} = \begin{cases} \sigma_n du_n^{cr} & \sigma_n \geq 0 \text{ (tension)} \\ (\|\tau\| + \sigma_n \tan \phi) \|du_t^{cr}\| & \sigma_n < 0 \text{ (compression)} \end{cases} \quad (8)$$

where

$$\|\tau\| = \sqrt{(\tau_{t_1})^2 + (\tau_{t_2})^2} \quad (9a)$$

$$\|du_t^{cr}\| = \sqrt{(u_{t_1}^{cr})^2 + (u_{t_2}^{cr})^2} \quad (9b)$$

being $\underline{u}^{cr} = \underline{u} - \underline{u}_{el}$ the post-cracking displacement components.

The evolution of the materials parameters with the work produced during the fracture process is simulated by the same type of function (except for the friction angle of the plastic potential):

$$\chi = \chi_0 [1 - f(\xi_i)] \quad (10)$$

where χ can represent f_t , c_α or $\tan \phi_F$, and, accordingly, χ_0 will represent the initial tensile strength ($f_{t,0}$), the initial cohesion ($c_{0,\alpha}$), and the tangent of the initial friction angle for the cracking surface ($\tan \phi_{0,F}$). When Eq. (10) is used for modelling the evolution of the tensile strength, $\xi_i = \xi_I$, while when adopted for simulating the evolution of both the cohesion and the friction angle, $\xi_i = \xi_{II}$.

Finally, the evolution of the friction angle in the plastic potential function is simulated by the following equation:

$$\tan \phi_P = \tan \phi_{0,P} - (\tan \phi_{0,P} - \tan \phi_r) f(\xi_{II}) \quad (11)$$

where $\tan \phi_{0,P}$ and $\tan \phi_r$ are the tangent of the initial and residual friction angle, respectively. The scaling function $f(\xi_i)$ in Eq. (12) is obtained from:

$$f(\xi_i) = \frac{e^{-\lambda} \xi_i}{1 + (e^{-\lambda} - 1) \xi_i} \quad (12)$$

where λ is the softening parameter that define the damage evolution of the $\underline{r} = [f_t, c, \tan \phi]^T$ variables. The damage evolution for the tensile strength, cohesion and friction angle can be different by adopting distinct values for the corresponding λ parameter (λ_{f_t} , λ_c , $\lambda_{\tan \phi}$).

5.2.3 Finite element mesh, simulations and analysis

Due to the symmetry of the experimental testing conditions, only half of the specimen was simulated, by using the finite element mesh represented in Figure 5.4, where the type of finite elements, supports and loading conditions are indicated. Both the core material (XPS-3) and the two outer RSFRC layers were modelled by solid serendipity finite elements of 20-nodes with $2 \times 2 \times 2$ Gauss Legendre integration scheme (GL-IS), while surface interface serendipity finite elements of 16 nodes were adopted in the contact between the bottom RSFRC and the XPS-3 layers, by using 2×2 GL-IS. The experimental tests showed that no movement occurred between the top RSFRC layer and the XPS-3, therefore perfect bond conditions were assumed for this contact zone.

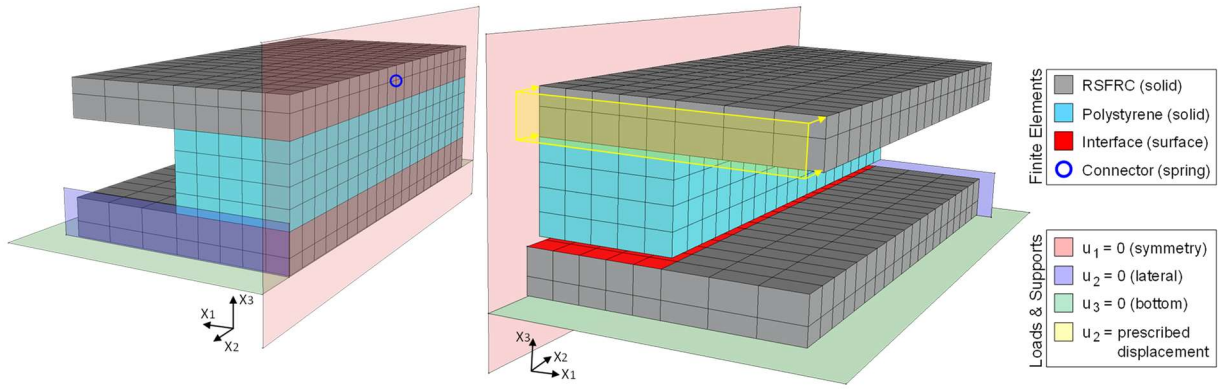


Figure 5.4: Finite element mesh and boundary conditions (u_i , $i = [1 - 3]$, displacement in X_i direction).

Taking into account the results obtained in the pushout test with a specimen where the two outer RSFRC layers were exclusively connected by the S-GFRP connector (FCØ10-H25 in Figure 3.7f), the axial and shear stiffness provided by this connector were considered in the numerical model by replacing the connector with equivalent springs, since this strategy can reproduce effectively the contribution of the connector up to its rupture, with benefits in terms of computing time (see Figure 5.5).

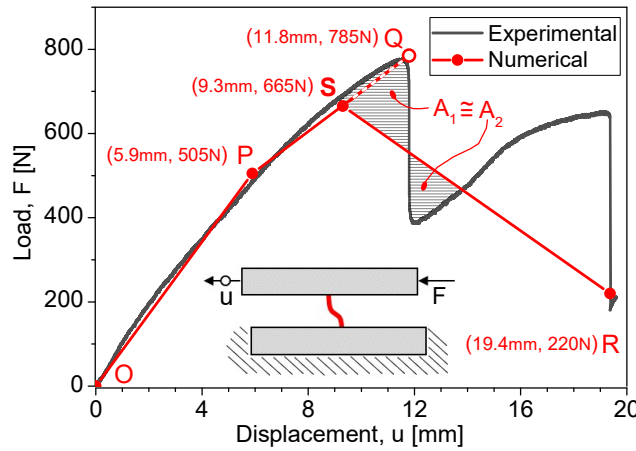


Figure 5.5: Pushout test for assessing the stiffness contribution of the GFRP connector and the derived shear stiffness for its numerical simulation.

The complex behaviour of the connector was simulated by a trilinear diagram defined by the branches OP, PS and SR. The point S is located in the branch PQ and was determined in order that the areas A_1 and A_2 would be approximately equal, by pivoting in the point R, corresponding to the full rupture of the connector. This strategy aims at simulating the last complex stage of the behaviour of the connector by an equivalent softening modulus as the final branch of its constitutive law correspondent to its substituting spring. The values of displacement and corresponding force for the points P, S and R define the trilinear

load-displacement diagram for the spring simulating the shear contribution of the connector. The axial stiffness of this connector was also simulated by a spring (in X_3 direction) connected to the point (blue circle) indicated in Figure 5.4, by assuming linear-elastic behaviour in tension up to its rupture (at an elongation of 0.9mm). Displacements in the X_2 direction were imposed in the face of the top RSFRC layer in contact with the actuator (see Figure 5.4).

Table 5.1 indicates the values adopted for the IFE model parameters. Values such as the initial tensile strength and initial cohesion were determined based on information published by other researchers (Papadopoulos, 2005; Müller, 2015; Figueira *et al.*, 2019). The remaining values were inferred through an inverse analysis approach by fitting, as much as possible, the load *vs.* relative displacement registered experimentally for specimen FCØ10-H25-XPS3, since no experimental information is available for the evaluation of this data, namely the damage evolution of the cohesion and friction angle during the fracture process.

Figures 5.6a and 5.6b present the evolution of the normalized cohesion ($C_F(\xi_{II})/C_{0,F}$) and friction angle ($\tan\phi_F(\xi_{II})/\tan\phi_{0,F}$) of the cracking failure surface, respectively, with $W_{II}^{cr}/G_{F,II}$, where W_{II}^{cr} is the work produced during the sliding component of the cracking process (second expression of Eq. (8)). It is verified that the decrease of the cohesion with the work produced during the fracture process (W^{cr}) was less abrupt than in the case of the friction angle., respectively. It is verified that the decrease of the cohesion with the work produced during the fracture process (W^{cr}) was less abrupt than in the case of the friction angle. Other materials, namely RSFRC and XPS, were modelled assuming linear elastic behaviour. The adopted values for elastic stiffness of RSFRC (27500 MPa) and XPS (20 MPa) were obtained from the material characterization reported in section 3.2 and from the information that was provided by the manufacturer (Danosa, 2018), respectively.

Figure 5.7 compares the load *vs.* relative lateral displacement recorded experimentally and obtained numerically, where it can be seen that the adopted model with the considered parameter values is capable of reproducing the main behavioural features registered in the experimental tests with overall good precision. The influence of the connector is only mandatory for a deflection level above the one corresponding to point C. The three branches between the peak point and point C registered experimentally were not captured with high precision due to the continuous nature of the functions of the adopted formulation.

Table 5.1: Values adopted in the numerical simulation for the parameters of the IFE model.

Parameter	Value
Elastic stiffness in the orthogonal direction (k_n) [N/mm]	1.00
Elastic stiffness in the tangential directions ($k_t = k_{t_2}$) [N/mm]	1.00
Initial tensile strength ($f_{t,0}$) [MPa]	0.10
Initial cohesion for the cracking failure surface ($c_{0,F}$) [MPa]	0.11
Initial cohesion for the plastic potential function ($c_{0,P}$) [MPa]	0.025
Tangent of initial friction angle for the cracking failure surface ($\tan \phi_{0,F}$) [-]	0.37
Tangent of the initial friction angle for the potential function ($\tan \phi_{0,P}$) [-]	0.01
Tangent of the residual friction angle for the potential function ($\tan \phi_r$) [-]	0.001
Mode I fracture energy ($G_{F,I}$) [N/mm]	0.15
Mode II fracture energy ($G_{F,II}$) [N/mm]	0.20
Softening parameter for the tensile strength (λ_f) [-]	0.00
Softening parameter for the cohesion (λ_c) [-]	2.00
Softening parameter for the tangent of the friction angle ($\lambda_{\tan \phi}$) [-]	-1.00

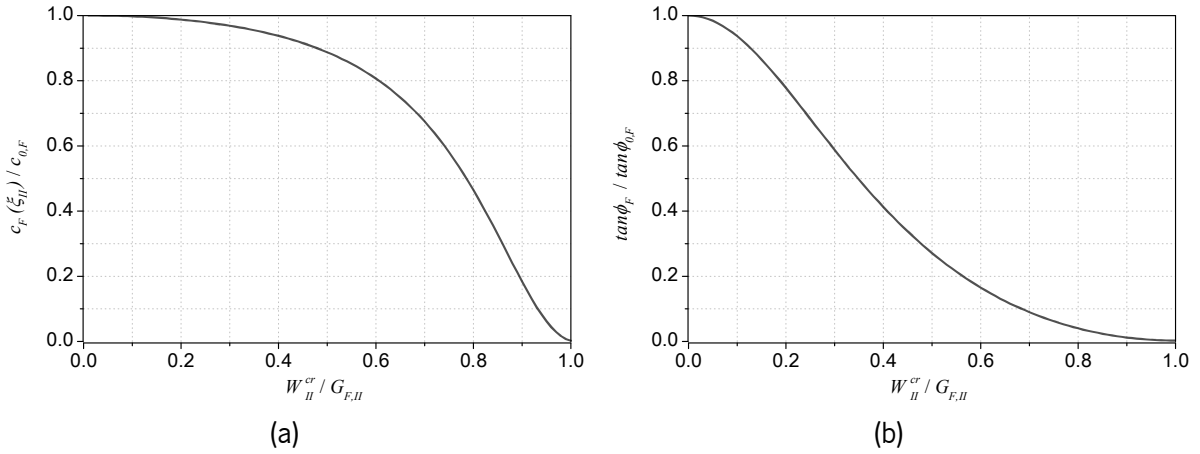


Figure 5.6: Evolution of the parameters adopted in the numerical simulation (Table 5.1):

(a) $c_F(\xi_{II})/c_{0,F}$ vs. $W^{cr}/G_{F,II}$; (b) $\tan \phi_F(\xi_{II})/\tan \phi_{0,F}$ vs. $W^{cr}/G_{F,II}$.

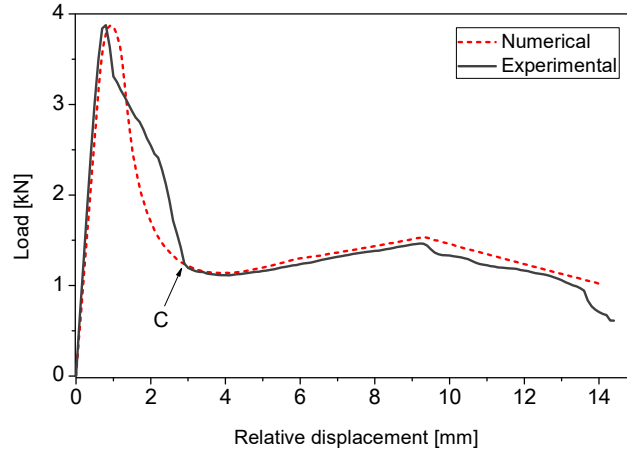


Figure 5.7: Comparison between experimental and numerical results (for specimen typology FCØ10-H25-XPS3).

5.3 Cyclic behaviour of bare RC frame

5.3.1 Overview

This Section deals with the numerical studies conducted to numerically simulate the experimental behaviour of the one-bay RC frame specimen subjected to cyclic in-plane loading. The corresponding experimental work is detailed in chapter 4 of this thesis. The fitting of the numerical results to the experimental data is conducted to achieve a calibrated numerical model that is representative of the experimental findings.

5.3.2 Finite element mesh

The RC frame, with dimensions described in Section 4.2.2, was modelled using 20-node Serendipity solid elements ($2 \times 2 \times 2$ Gauss-Legendre integration scheme) for the concrete structure and embedded 3-node line elements of axial stiffness exclusively (one degree of freedom, 2 Gauss-Legendre integration scheme) for the reinforcing steel bars. The definition of the final mesh configuration was consequence of an initial mesh refinement study, which considered two different levels of mesh refinement. The main difference between the analysed mesh configurations is the density of elements in the longitudinal direction of the structural members (columns and beams), as shown in Figure 5.8. The resultant mesh aspect ratios, $\lambda_{FEM} = X_1 : X_2 : X_3$, for the analysed mesh arrangements are displayed in Figure 5.8, with indication of the mesh scale proportions in global directions X_1 , X_2 and X_3 (according to the axes presented in Figure 5.8). Taking into account that both mesh configurations yielded very similar results

for the load-deflection results (practically superimposed results, with relative differences of less than $\pm 1\%$), the mesh with less element density (Figure 5.8, right hand side) was selected for this numerical analysis as it requires lower computational effort.

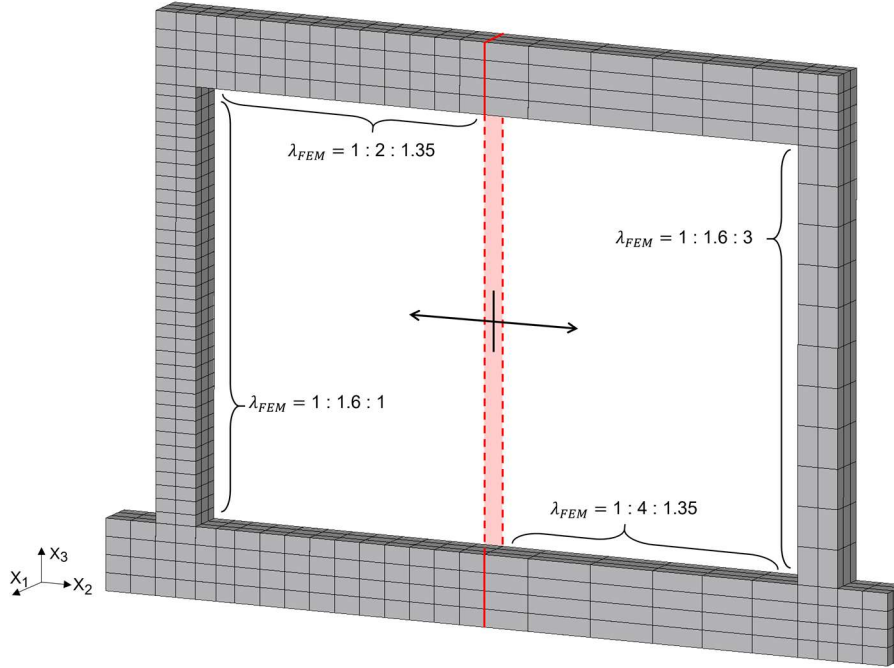


Figure 5.8: Considered levels of mesh refinement, with indication of mesh aspect ratio,

$$\lambda_{FEM} = X_1 : X_2 : X_3.$$

The final adopted mesh is depicted in Figure 5.9a, where the corresponding concrete and steel elements can be noticed (translucent solid concrete elements to enable visibility of steel rebar elements). In order to adequately simulate the boundary conditions to which the RC frames were subjected to, described in Section 4.2.5, additional steel elements were also included in the numerical model, also visible in Figure 5.9a, namely: (i) two 30 mm thick steel plates (20-node solid elements) connected by two Ø50mm *Dywidag* tie rods (3-node truss 3D line elements), through which the lateral load is applied to the RC frame; (ii) two M20 threaded steel rods (3-node 3D truss elements) connected to a tubular 2UPN140 steel profile (3-node 3D frame elements), one for each side of the prominent foundation beam. Additional information regarding boundary conditions is presented in Figure 5.9b. The loading conditions were simulated by a monotonic increase of lateral displacement and a constant pressure applied at the top of the columns to account for the vertical loads that were applied in these areas (detail 10 in Figure 4.12).

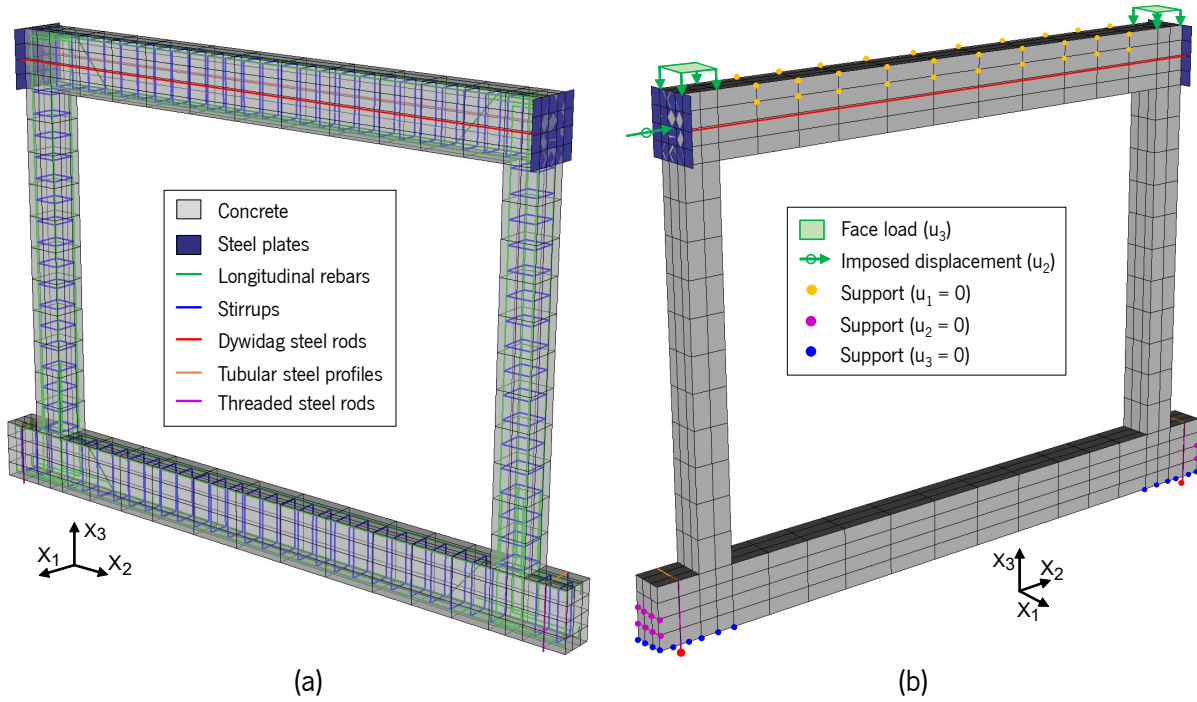


Figure 5.9: Simulation of RC frame specimens: components of finite element mesh (a) and boundary conditions (b) adopted to simulate RC-frame specimens.

5.3.3 Constitutive material models

For the calibration of the constitutive material models, both concrete and embedded steel rebars were simulated considering their material non-linear behaviour, while all the additional steel elements were assumed to have linear elastic material behaviour. Concrete was modelled by adopting a plastic-damage smeared crack model developed by Behbahani *et al.* (2015). This constitutive model is an extended version of the multi-directional fixed smeared crack model (Barros, 1996; Sena-Cruz, 2005; Ventura-Gouveia *et al.*, 2008; Ventura-Gouveia, 2011), with the extra functionality of being capable of simulating the nonlinear behaviour of cement based materials in compression by using a plastic-damage approach. The values adopted for the parameters that define the plastic-damage smeared crack model are presented in Table 5.2. The values of the elastic modulus, compressive strength and tensile strength were determined based on the material characterization reported in subsection 4.2.4.1. The strain at peak compressive strength, the parameter defining the initial yield surface and the fracture energy were obtained considering the recommendations of Model Code 2010 (Fédération Internationale du Béton, 2011) for the used concrete strength class, while the shape of the tensile strain softening diagram is typical of plain concrete of this type of strength class. The values of the remaining parameters are those

suggested in publications where FEMIX was used for the simulation of the material nonlinear behaviour of RC structures (Barros, 1996; Sena-Cruz, 2005; Ventura-Gouveia, 2011; Behbahani, 2017).

Figures 5.10a and 5.10b present a schematic illustration of the stress-strain diagrams simulating the behaviour of the concrete in the post-cracking tensile and in the post-elastic compressive stages, respectively. The post-cracking tensile behaviour is simulated by a tri-linear tension-softening diagram (Figure 5.10a), which is defined by the tensile strength, f_{ct} , α_i and ξ_i parameters, the mode I fracture energy, $G_{f,I}$, and the crack bandwidth, l_b . The stress-plastic strain relationship simulating the nonlinear behaviour of concrete in compression (Figure 5.10b) is defined by the parameter α_o (initiation of the elasto-plastic behaviour), the compressive strength, f_c , and its corresponding plastic strain, ε_{cp1} , the energy dissipated in the softening stage, $G_{f,c}$, and the bandwidth where $G_{f,c}$ is consumed, assumed equal to the crack bandwidth by simplification. The crack bandwidth (l_b) for plane stress and solid finite elements is assumed equal to the $\sqrt{A_{IP}}$ and $\sqrt[3]{V_{IP}}$, respectively, where A_{IP} and V_{IP} are the area and volume of the integration point (IP).

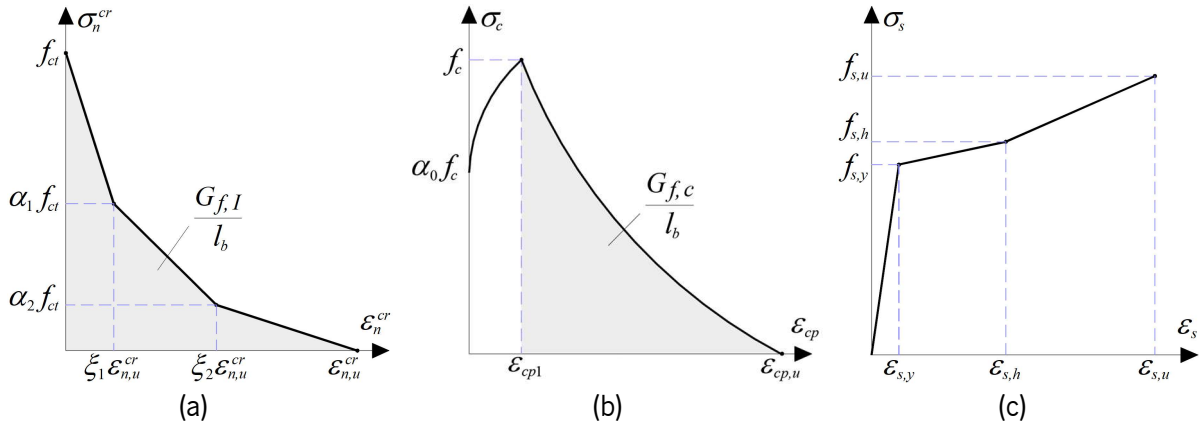


Figure 5.10: Constitutive material models: diagrams for modelling the behaviour of concrete in its post-cracking stage (a) and post-linear stage in compression (b), and the behaviour of steel in both tension and compression (c).

The non-linear behaviour of the steel rebars was modelled following the stress-strain constitutive law presented in Figure 5.10c. The curve is defined by the three points that characterize the yielding, hardening and ultimate strain/stress stage of the steel rebars, and by a parameter that varies between 1.0 and 4.0 for the shape of the last branch of the stress-strain curve. The values adopted for these parameters are presented in Table 5.3. More information regarding this constitutive model can be found in (Sena-Cruz, 2005).

Table 5.2: Values adopted in the numerical simulation for the parameters defining the concrete constitutive model.

Parameter	Value
Poisson's coefficient (ν) [-]	0.15
Elastic modulus (E) [MPa]	2.86×10^4
Compressive strength (f_c) [MPa]	41.00
Strain at compression peak stress (ε_{cp1}) [-]	2.10×10^{-3}
Parameter defining the initial yield surface (α_o) [-]	0.20
Compressive fracture energy ($G_{F,c}$) [N/mm]	20.00
Tensile strength (f_{ct}) [MPa]	2.83
Multiple of the ultimate tensile strain for defining the first point (ξ_1) [-]	0.01
Multiple of the ultimate tensile strength for defining the first point (α_1) [-]	0.50
Multiple of the ultimate tensile strain for defining the second point (ξ_2) [-]	0.20
Multiple of the ultimate tensile strength for defining the second point (α_2) [-]	0.10
Mode I fracture energy ($G_{F,I}$) [N/mm]	0.10
Parameter defining the mode I fracture energy available for a new crack (p_1)	1.00
Threshold angle (α_{th}) [°]	30.00
Maximum number of cracks per integration point [-]	2.00

Table 5.3: Values adopted in the numerical simulation for the parameters defining the steel constitutive model.

Parameter	Value
Strain at the end of the first branch ($\varepsilon_{s,y}$) [-]	2.67×10^{-3}
Stress at the end of the first branch ($f_{s,y}$) [MPa]	400.00
Strain at the end of the second branch ($\varepsilon_{s,h}$) [-]	5.00×10^{-3}
Stress at the end of the second branch ($f_{s,h}$) [MPa]	430.00
Strain at the end of the third branch ($\varepsilon_{s,u}$) [-]	1.40×10^{-2}
Stress at the end of the third branch ($f_{s,u}$) [MPa]	460.00
Third branch exponent (p) [-]	1.00

5.3.4 Experimental vs. numerical response

Figure 5.11 presents the numerical results obtained, plotted together with the experimental ones, the latter referring to the cyclic test on RC frame specimen “R”. It can be seen that the calibrated numerical model showed good agreement with the envelope of the hysteretic load-displacement curve.

The deformed mesh is depicted in Figure 5.12, where the high deformation of the columns, in comparison to the beams, is clearly visible. Figure 5.13 presents the normal stress field for direction X_3 for 0.1% of lateral drift (approximately 2 mm of lateral displacement), which corresponds to the lateral drift when cracking has initiated, as mentioned in section 4.3.3. For this loading stage, a maximum tensile stress of 2.8 MPa and a maximum compressive stress of -10.7 MPa are installed in these regions of the columns, near the joints.

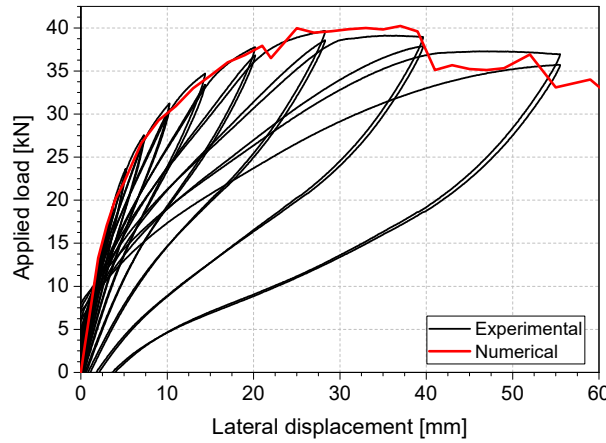


Figure 5.11: Experimental (cyclic test) vs. numerical (monotonic test) results for specimen R.

Figures 5.14 and 5.15 present the numerical crack pattern obtained for lateral displacements corresponding to, respectively, 1.1% and 2.1% of drift. The illustrations present two types of crack status, depending on the tensile strain in the crack (ε_n^{cr}): opening cracks (red cracks), which are those at $\varepsilon_n^{cr} < \varepsilon_{n,u}^{cr}$ (see Figure 5.1a), and fully open cracks (vivid-red cracks) where the mode I fracture energy was fully exhausted ($\varepsilon_n^{cr} \geq \varepsilon_{n,u}^{cr}$).

A direct comparison can be made between experimentally and numerically obtained crack patterns for 1.1% and 2.1% of drift by analysing Figures 4.16b and 5.14 and Figures 4.16c and 5.15, respectively.

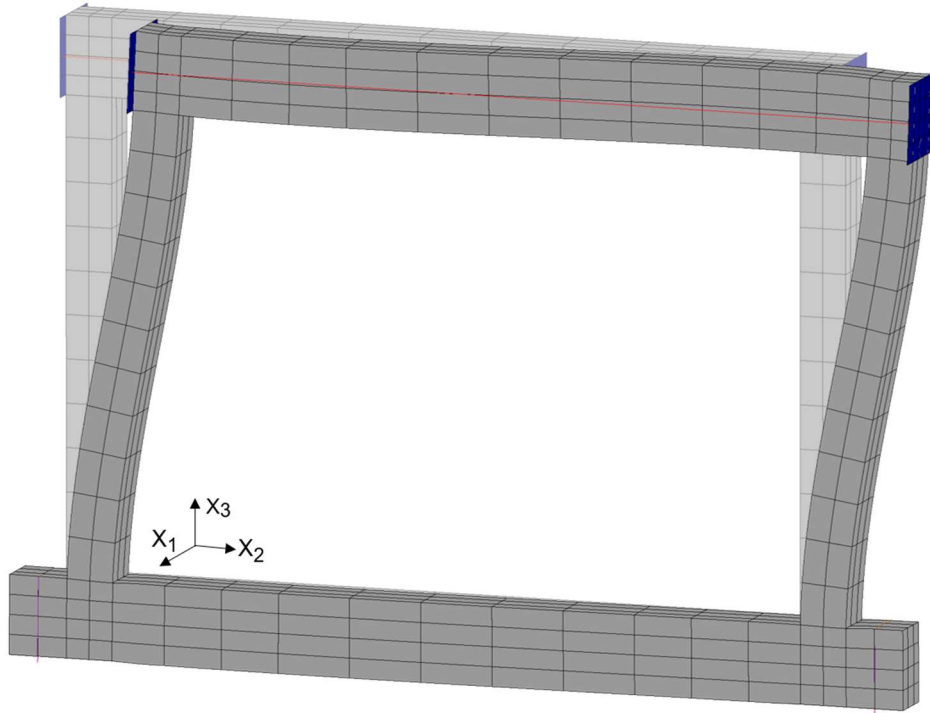


Figure 5.12: Undeformed *vs.* deformed finite element mesh (for 1.1% of lateral drift, with 15× magnification factor).

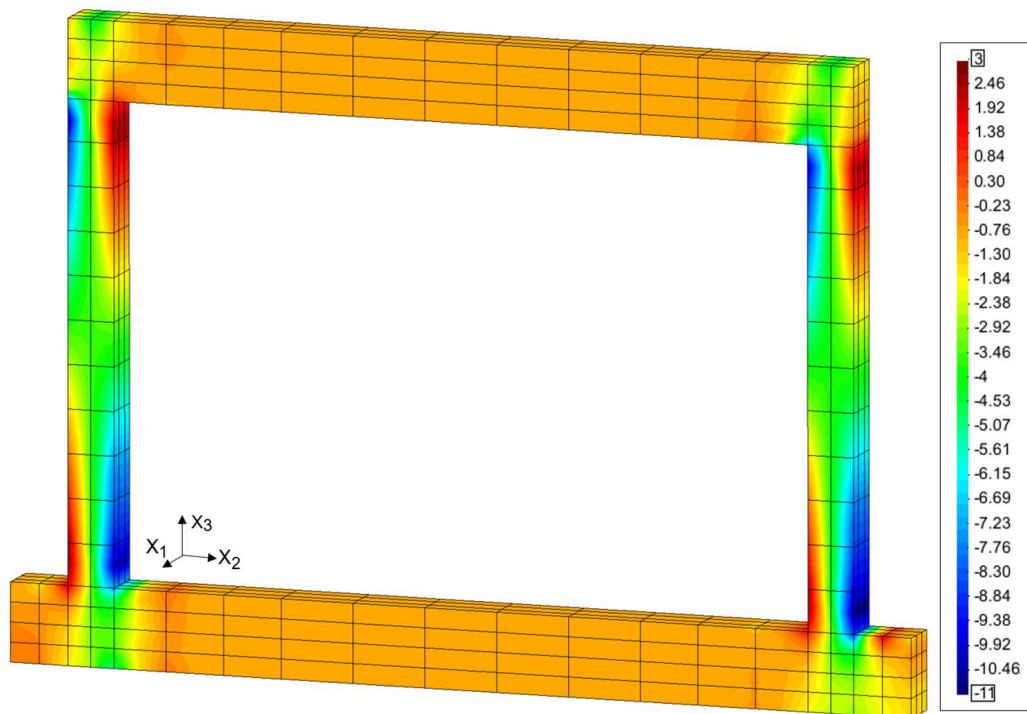


Figure 5.13: Normal stress field in direction X_3 for 0.1% of lateral drift [stress field scale in MPa].

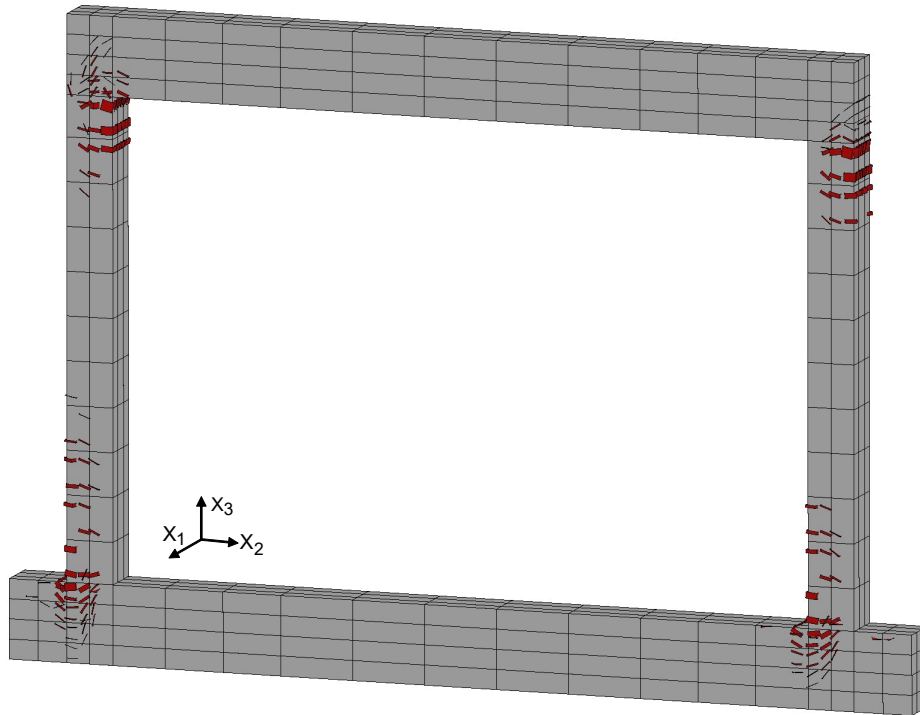


Figure 5.14: Numerical crack pattern for 1.1% of lateral drift, showing opening (red) cracks.

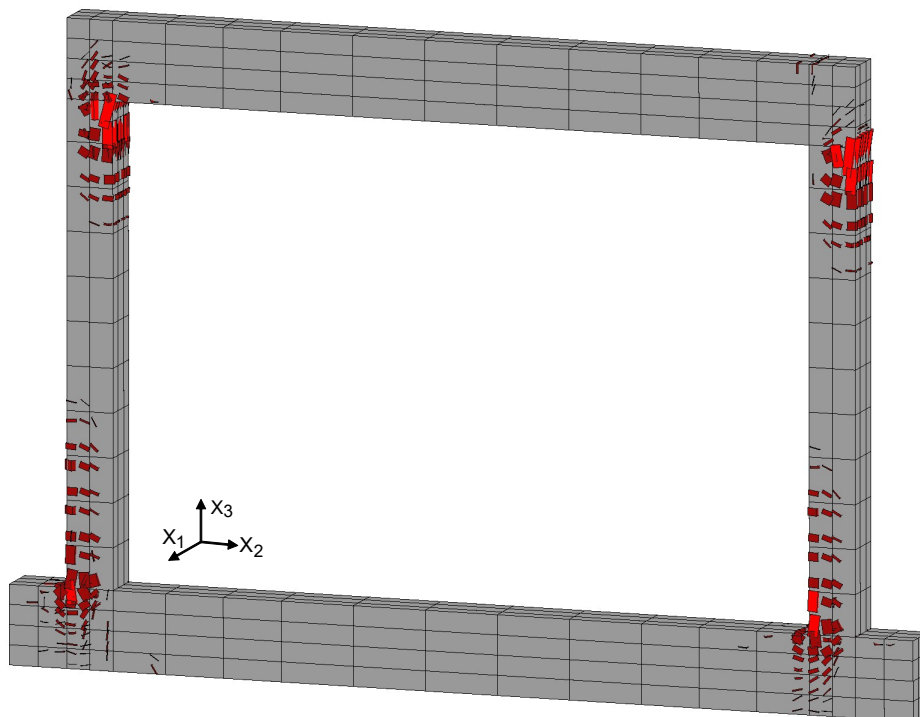


Figure 5.15: Numerical crack pattern for 2.1% of lateral drift, showing opening (red) and fully open (vivid-red) cracks.

It can be seen that many similarities can be found, with cracks located in the columns, primarily near the joints. Also, for 2.1% of drift, fully opened cracks are visible near the joints (vivid-red cracks in Figure 5.15) and further propagation of cracks can be observed in the joint areas, which can be correlated to the formation of plastic hinges that were observed experimentally in the same location (see Figures 4.16c and 4.17a). The main difference between experimental and numerical crack patterns is related to the load application method (cyclic *vs.* monotonic), which results in cracks being located mainly at the inner or outer surface of the columns for the numerical models, rather than the configuration that is visible experimentally, where cracks develop across the entire cross-section.

5.4 Connection between sandwich panel and RC frame

5.4.1 Overview

This section presents the numerical study conducted to assist the design process related to the steel elements that provide connection between the sandwich panel and the RC frame (angle profiles and mechanical/adhesive anchors). The conducted analysis encompasses design scenarios that consider variations on the length and configuration of the angle profiles and also on the number of anchors that establish structural connection to the RC frame and to the sandwich panel.

5.4.2 Finite element mesh

The numerical models, deployed using the software tools already mentioned in this chapter, aim to simulate the connection between RC frames and the developed sandwich panel, according to the connection approach presented in Figure 4.9, when subjected to in-plane lateral loading conditions equivalent to the test setup illustrated in Figure 4.12. The finite element mesh adopted for one of the analysed design scenarios is illustrated in Figure 5.16.

In what concerns the sandwich panel, all layers (RSFRC and XPS) were modelled using 20-node solid elements. The GFRP connectors, as well as all *Hilti* anchors were modelled as 2-node 3D Timoshenko beam elements, while 8-node surface elements were employed for the angle profiles. 3D surface interface finite elements of 16-nodes were introduced in the contact regions between the RC frame and the sandwich panel and also between the angle brackets and the frame/panel. Except these last finite elements, the remaining ones were modelled assuming linear elastic behaviour for their constituent materials.

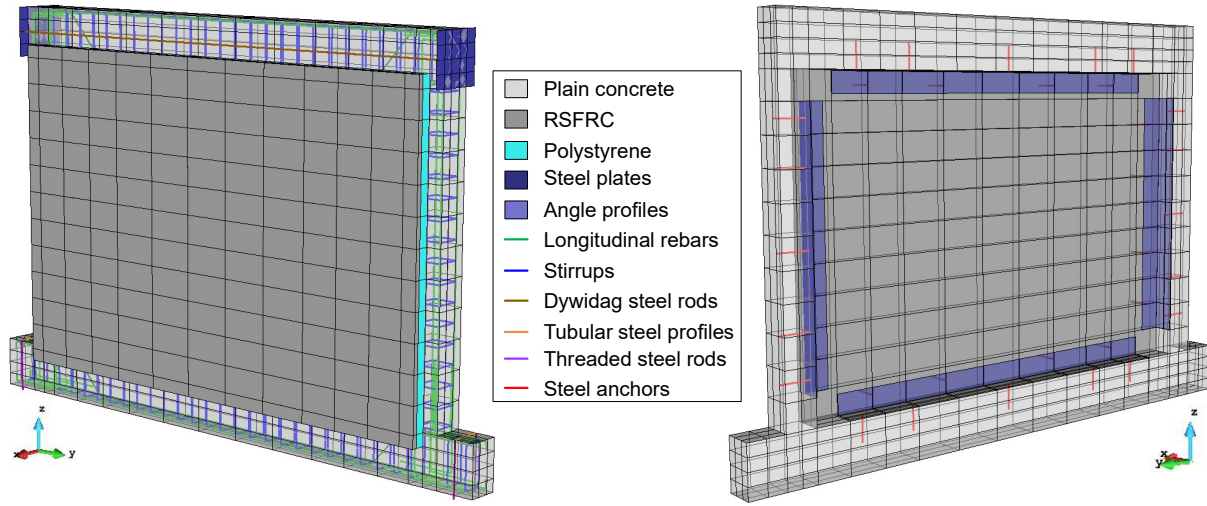


Figure 5.16: Finite element mesh for the simulations to design the metallic angle brackets and mechanical/adhesive anchors.

5.4.3 Constitutive material models

The interface elements (with 2×2 Gauss-Legendre integration scheme) aim to simulate the contour contact surfaces between the sandwich panel and the RC frame and the contact surfaces between the angle brackets and the panel/frame (see Figure 5.17). Since the referred surfaces are unconnected (no bond between the panel, RC frame and angle brackets – only mechanical contact), it was assumed that no tensile forces/stresses can be transferred through these interfaces. However, a linear elastic behaviour occurs when the interface is subjected to compression. The sliding behaviour is modelled according to the shear-sliding response defined by Equation (13), where s_0 and s_m correspond to the slip/slide displacements at the end of the linear-elastic branch and at the peak shear strength of the interface (see Figure 5.18). The peak shear stress, τ_m , is computed through Mohr-Coulomb failure criterion, as presented in Equation (14), where μ is the friction coefficient, which corresponds to the tangent of the internal friction angle ($\mu = \tan \phi$) and c is the interface material's cohesion.

Table 5.4 presents the adopted values for the different parameters. These values were defined with basis on a comprehensive study where the influence of such parameters on the opening-sliding behaviour of the interface was evaluated (Barros and di Prisco, 2009) and on another study where a similar interface model was applied for the simulation of a concrete slab supported on ground, and submitted to a push-off test (Valente, 2019). The same values were adopted for the three interface regions represented in Figure 5.18 due to lack of material characterization to accurately estimate the values of cohesion and

friction angle that characterize these contact surfaces. The full formulation of this interface finite element and its constitutive model are described elsewhere (Barros and di Prisco, 2009; Valente, 2019).

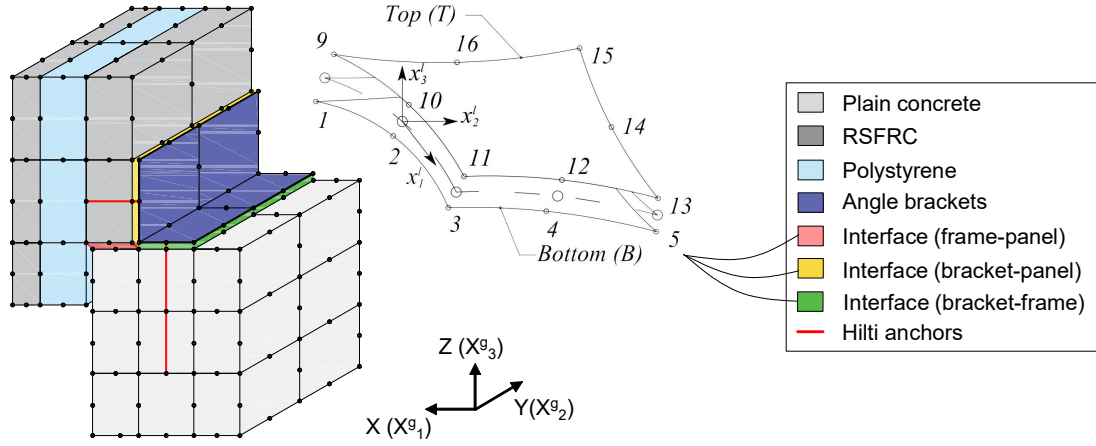


Figure 5.17: Serendipity 16-node surface interface elements.

$$\tau_i = \begin{cases} \frac{\tau_0}{s_0} s_i & 0 \leq |s_i| \leq s_0 \\ \tau_m \left(\frac{s_i}{s_m} \right)^{\alpha_1} & s_0 < |s_i| < s_m \\ \tau_m \left(\frac{s_i}{s_m} \right)^{-\alpha_2} & |s_i| > s_m \end{cases} \quad (13)$$

$$\tau_m = \sigma_n \mu + c \quad (14)$$

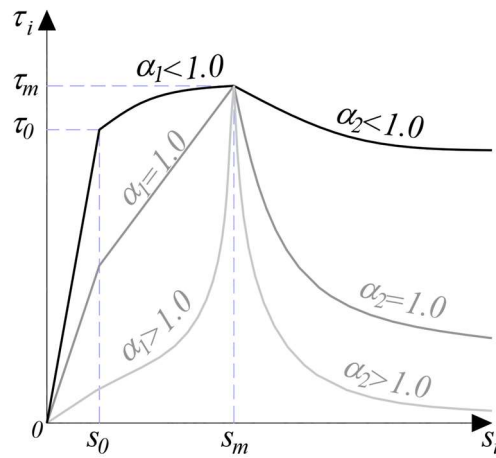


Figure 5.18: Schematic illustration of the shear-sliding relationship [adapted from Valente (2019)].

Table 5.4: Values adopted for the interface finite element model.

Parameter	Value
Slip at the end of the linear bond stress-slip relationship (s_0) [mm]	0.5
Slip at peak bond stress (s_m) [mm]	1.0
Material cohesion (c) [MPa]	0.1
Friction angle between materials in contact (ϕ) [°]	26.0
Parameter defining pre-peak bond stress-slip relationship (α_1) [-]	0.2
Parameter defining post-peak bond stress-slip relationship (α_2) [-]	0.3
Normal stiffness (k_n) [N/mm]	1×10^6

5.4.4 Design scenarios

Different scenarios were studied for the design of the connection between the sandwich panel and the RC frame, by considering different configurations for the angle profiles and the number of anchors, namely: (i) scenario A, using corner angle brackets in the RC frame joint areas, with a total of 24 units of both HST and HIT anchors (see Figure 5.19a – red lines and dots represent HST and HIT anchors, respectively); (ii) scenario B, using bigger corner angle and additional anchor connections, namely 32 HST and 32 HIT anchors (see Figure 5.19b); (iii) scenario C, using angle brackets connected to each beam/column through a total of 20 HST anchors and 16 units of HIT anchors for providing connection to the panel (see Figure 5.19c); and (iv) scenario D, similar to design scenario C, but with additional connection points, resulting in 24 units of both HST and HIT anchors (see Figure 5.19d).

It should be noted that, since the dimensions of the sandwich panel and RC frame were pre-determined in this design process, diameters of 12 mm and 10 mm were adopted for the HST and HIT anchors, respectively, based on the design recommendations provided by the manufacturer (Hilti, 2014) regarding minimum edge distances to be respected in order to prevent splitting failure.

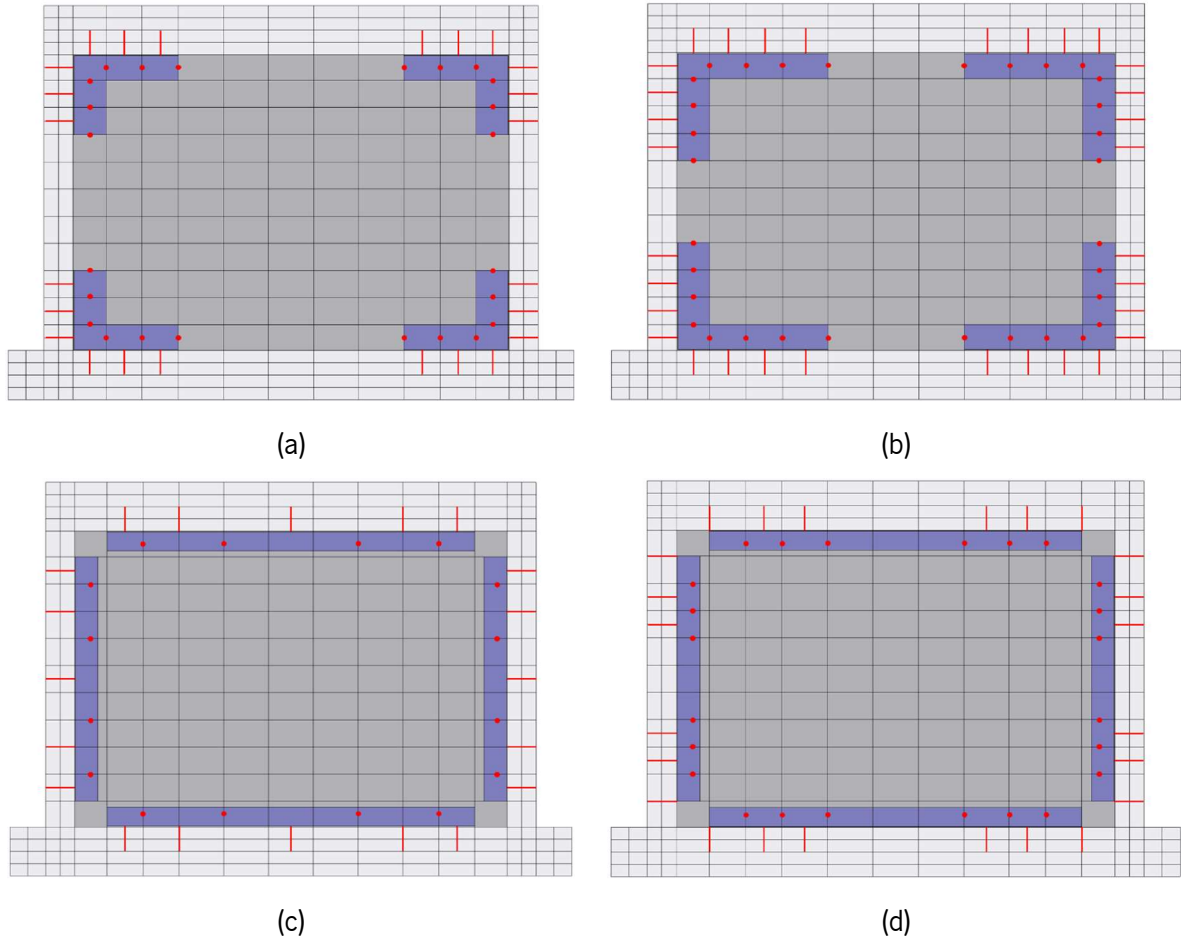


Figure 5.19: Design scenarios (red lines and dots represent HST and HIT anchors, respectively):

(a) A-24HST-24HIT; (b) B-32HST-32HIT; (c) C-20HST-16HIT; (d) D-24HST-24HIT.

5.4.5 Results and discussion

The design approach focused on guarantying that safety verifications were met for all anchors when subjecting the RC frame to a lateral design load value equal to 1.5×180 kN (monotonic load). This load level is equivalent to the peak load that was obtained for the in-plane cyclic tests that were performed on the RC frame specimen containing masonry infill walls (see Figure 4.15b), aggravated by the partial safety factor of 1.5. Table 5.5 summarizes the design-base governing forces installed in the anchors, their characteristic yield stress and the safety design verifications. In this table, N_{Rd} and V_{Rd} are the design values for tensile and shear resistance, respectively, recommended by the manufacturer based on validated test results under controlled laboratory conditions (Hilti, 2014; Deutsches Institut für Bautechnik, 2011, 2012); N_{Ed} and V_{Ed} are design values of the maximum tensile and shear forces, respectively, registered in the finite elements that simulate the Hilti anchors in the numerical model (based on the three principal stresses of each line element). The principal shear force for each line element is

obtained from $\sqrt{V_Y^2 + V_Z^2}$, where V_Y and V_Z are the two shear components in the element. The Von Mises stress criterion (σ_{VM}) was taken into account in order to combine the effect of axial and shear loading, according to Equation (15), where σ_X is the axial stress and τ_{XY} and τ_{XZ} are the shear stresses in the Y and Z axes of the element coordinate system. σ_{VM} is compared to the characteristic yield strength of the anchors (f_{yk}), according to the mechanical properties indicated by the manufacturer.

$$\sigma_{VM} = \sqrt{\sigma_X^2 + 3(\tau_{XY}^2 + \tau_{XZ}^2)} \quad (15)$$

Table 5.5: Main indicators for design of HST-M12 and HIT-M10 anchors.

Type of anchor	Design scenario	N_{Rd} [kN]	N_{Ed} [kN]	$\frac{N_{Ed}}{N_{Rd}}$	V_{Rd} [kN]	V_{Ed} [kN]	$\frac{V_{Ed}}{V_{Rd}}$	f_{yk} [MPa]	σ_{VM} [MPa]	$\frac{\sigma_{VM}}{f_{yk}}$
HST M12	A-24HST-24HIT	8.00	1.97	0.25	28.00	5.50	0.20	560.0	108.46	0.19
	B-32HST-32HIT		1.47	0.18		5.38	0.19		106.70	0.19
	C-20HST-16HIT		1.18	0.15		9.41	0.34		240.65	0.43
	D-24HST-24HIT		1.90	0.24		7.12	0.25		182.06	0.33
HIT M10	A-24HST-24HIT	12.60	1.68	0.13	12.00	27.86	1.86	400	546.20	1.37
	B-32HST-32HIT		1.03	0.08		24.21	1.61		474.54	1.19
	C-20HST-16HIT		0.66	0.05		15.75	1.05		402.74	1.01
	D-24HST-24HIT		0.61	0.05		14.03	0.94		358.66	0.90

Based on the information presented in Table 5.5, it was possible to conclude that, overall, the shear action has a more significant effect, especially for the HIT anchors, with the V_{Ed} / V_{Rd} ratio being significantly higher than 1.0 for the design scenarios A and B, which contemplate corner angle brackets, in comparison to scenarios C and D. The Von Mises stress analysis also led to the conclusion that safer results were obtained for scenarios C and D regarding the HIT M10 anchors and a better mobilization of the strength capacity of HST M12. This is also shown in Figures 5.20 and 5.21, which present the σ_{VM} installed in all connectors of scenarios B and C, respectively. A red dashed line is also indicated for identifying the f_{yk} of HIT anchors (representation of f_{yk} for HST is not visible – the corresponding value is beyond the adopted graph scaling interval and f_{yk} vs. σ_{VM} comparison for these anchors is less relevant). HST anchors have a stronger contribution in scenarios C and D, which facilitates the mitigation of higher stress levels registered in HIT anchors. In view of this, the final configuration for the connection between the sandwich panel and RC frames, presented in Figure 4.9, is a middle ground solution between configurations C-20HST-16HIT and D-24HST-24HST, comprising a total of 24 units of HST and 20 units of HIT anchors. It can also be seen that the models were able to adequately predict the higher tendency for shear failure on the anchors located at the upper half of the sandwich panel specimen. This can be confirmed by comparing the stress distributions obtained numerically (e.g. Figure 5.21) to the damage evolution and failure modes observed in the cyclic tests conducted on specimen S, described in section 4.3.3 (e.g. Figure 4.19f).

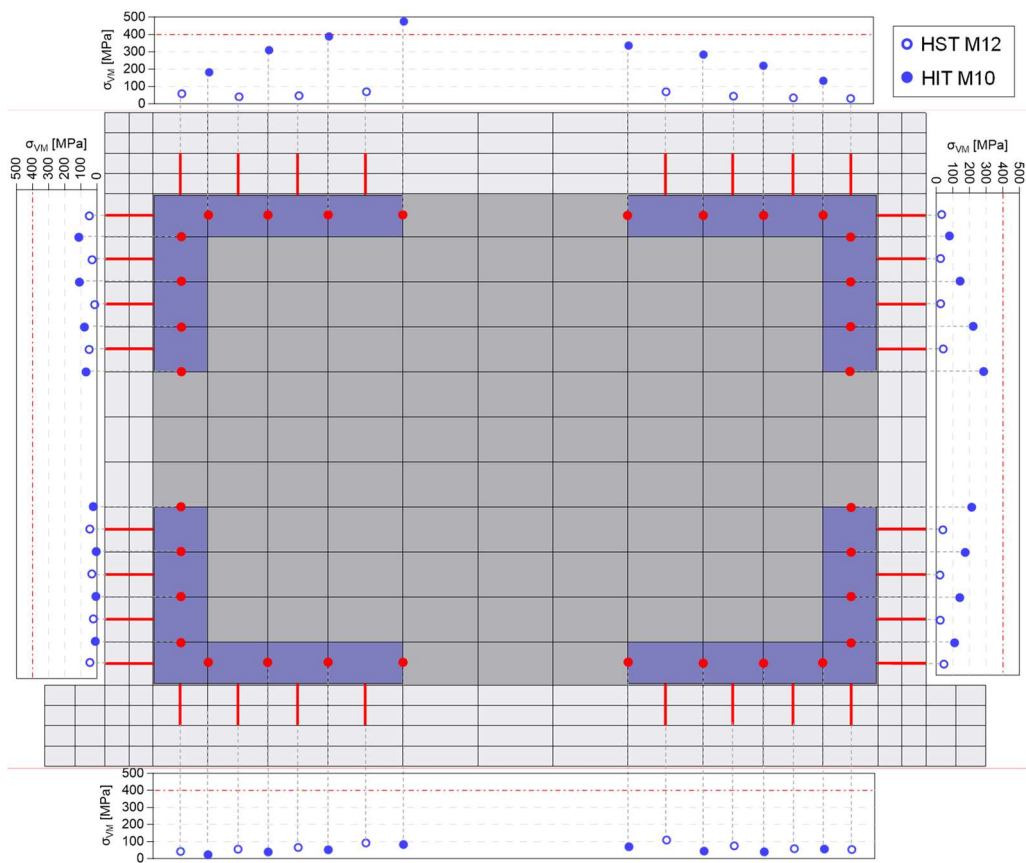


Figure 5.20: Stress distribution throughout all anchors for design scenario B-32HST-32HIT.

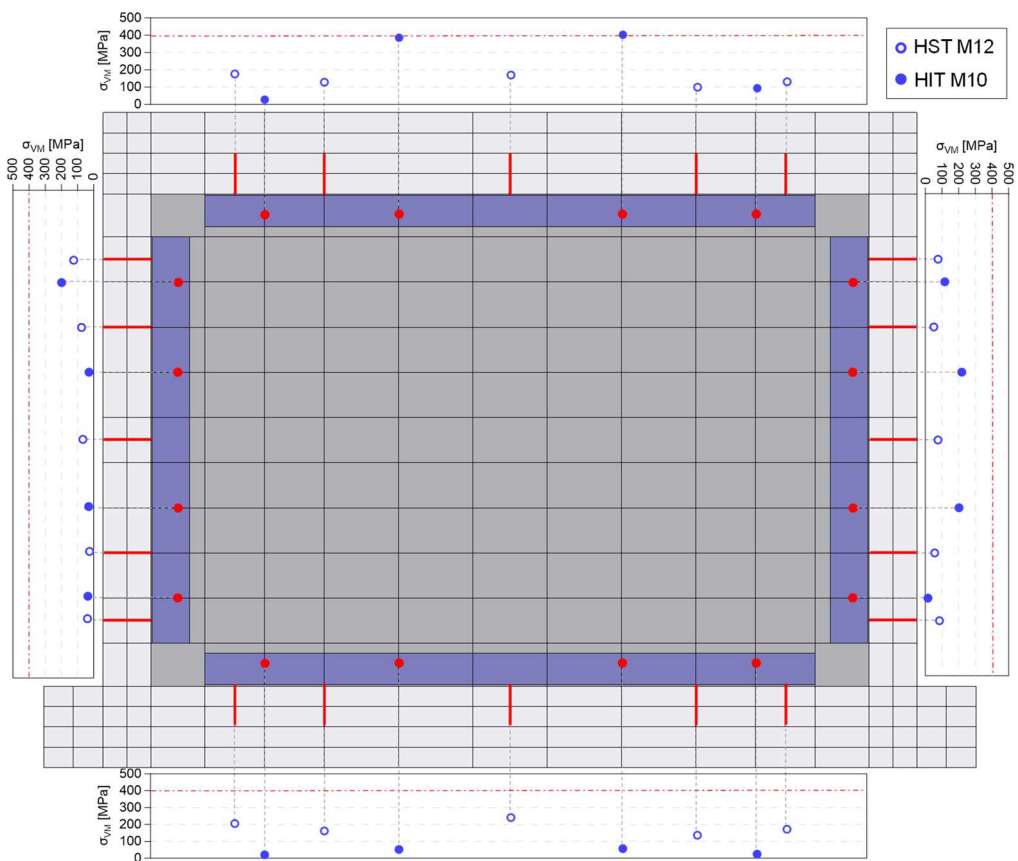


Figure 5.21: Stress distribution throughout all anchors for design scenario C-20HST-16HIT.

5.5 Concluding remarks

The studies presented in this chapter focused on the numerical modelling that was conducted in the scope of this doctoral thesis, namely to assist the design process of the developed sandwich panel solution and to model complex failure mechanisms that were observed at the interface between components of the sandwich panel. Based on the obtained results, the following main conclusions can be drawn:

1. The performed numerical simulations conducted to model the failure mechanisms observed at the interface between RSFRC and XPS-3 in the pushout tests showed good agreement between experimental and numerical results. Based on the obtained results, the initial cohesion and friction angle between these materials was estimated as 110 kPa and 20° , respectively, with a softening evolution during the shear loading process, more pronounced in the friction angle.
2. The numerical models of the bare RC frame, which were used to conduct mesh refinement studies and to calibrate the material properties and loading conditions, showed good agreement with the experimental behaviour obtained for that frame specimen when subjected to in-plane cyclic loading. This showed that the type of models adopted herein can be used in the context of design with the developed strengthening solution, allowing to derive properly the values of the parameters of the constitutive laws of the intervenient materials.
3. The models that were used for the design of the connection between the sandwich panel and the RC frame structure were able to predict the higher tendency for shear failure on the anchors located at the upper half of the sandwich panel specimen.

6 CONCLUSIONS AND FUTURE DEVELOPMENTS

6.1 Main conclusions

This thesis presents the research work that was conducted towards the development of a multifunctional composite sandwich panel for the rehabilitation of RC buildings built in Portugal between 1960 and the mid-1980s. A retrofit solution was presented, based on the application of a façade panel that comprises the following components: (i) thin RSFRC wythes; (ii) insulation core layer made of XPS; (iii) internal GFRP connectors; and (iv) additional steel members (anchors and angle brackets) that provide connection between the panel and the existing building's RC structure. Specific conclusions regarding each stage of the developed studies are presented at the end sections of each chapter. This section in particular presents a more general overview of the main conclusions that were drawn in this doctoral thesis.

Based on the performed literature review, the main innovative contributions provided by this research are linked to: (i) the incorporation of a sustainable structural reinforcement element in the concrete wythes of the sandwich panel, namely recycled steel fibres, avoiding the use of conventional steel rebars, which allows to decrease significantly the thickness of the wythes with the consequent economic and technical benefits regarding seismic strengthening; and (ii) the development of an integrated seismic and thermal retrofit solution that ensures continuous insulation of the building envelope, avoiding issues related to thermal bridges.

The concept of the referred rehabilitation technique was heavily based on the experimental findings that were gathered during the first phase of the laboratory work carried out for this thesis, which focused mainly on the mechanical characterization of the materials involved – RSFRC, GFRP and polystyrene – and on the execution of pushout and pullout tests using reduced-scale sandwich panel specimens. For the studies performed at the material level, emphasis was given to the development and characterization of the RSFRC, specifically targeted for the production of laminar/thin concrete layers, which showed high levels of compressive strength and very satisfactory levels of post-cracking tensile capacity. The pushout and pullout tests, on the other hand, provided insights on the impact of the adhesion between RSFRC and polystyrene (influenced essentially by polystyrene surface finishing) on the overall structural performance of the sandwich panel under shear loading conditions. This information has supported the sandwich panel design process by providing relevant data related to the diameter and anchorage depth

to be adopted for the structural GFRP connectors in order to assure proper connection between the sandwich panel layers. In fact, the construction, movement and installation of the sandwich panel in a RC frame to be strengthened, as well as the cyclic test with this strengthened RC frame, have demonstrated that these connectors are able to adequately ensure shear load transfer between RSFRC wythes in all relevant phases.

The second phase of experimental testing focused on the execution of in-plane cyclic tests using RC frame specimens. These cyclic tests were performed on three RC frame specimens, two of them being representative of the target building typology (RC frame tested with and without a traditional masonry infill wall), and a third RC frame that was strengthened with a façade panel prototype. The experimental results showed that the proposed sandwich panel strengthening technique is an effective solution since the RC frame strengthened with the façade/sandwich panel presented the overall highest lateral strength and energy dissipation capacity. Another important conclusion could be drawn through the observation and comparison of the damage evolution on the RC frame specimens, where it was possible to perceive that the inclusion of the sandwich panel led to higher levels of damage in the RC frame specimen in comparison to the more traditional masonry infill wall solution (wider cracks and concrete cover spalling). However, it is also noteworthy to mention that a more distributed crack pattern was obtained for the specimen strengthened with the proposed sandwich panel prototype (cracking propagation in both beams and column members). Furthermore, this higher level of damage is also a consequence of the eccentric disposition of the panel into the RC frame, generating detrimental torsional effects in the experiments whose level should be higher than the one expected to occur in a real building, where the axial/membrane stiffness provided by transversal RC beams and slabs are expected to significantly reduce these out-of-plane effects.

Finally, the numerical work associated to this thesis was conducted mainly to assist the design process of the developed façade panel prototype. For this purpose, a numerical model was developed to capture the cyclic behaviour of the RC frame specimen subjected to in-plane loading, showing good agreement between experimental and numerical results. The referred RC frame model served as basis for additional numerical simulations that focused on the study of design scenarios for the connection between the sandwich panel and the RC frame. Besides the models that were focused on the intermediate-scale RC frame specimens, this thesis also included numerical simulations conducted to model some of the complex failure mechanisms that were observed during the experiments performed on reduced-scale sandwich panel specimens subjected to shear loading (pushout tests). These models were able to estimate the cohesion and

friction angle of the interfaces between the materials forming the panel (RSFRC and polystyrene, in this case), which are mobilized in the shear loading process.

6.2 Known limitations and prospects for future developments

The research that was carried out in the scope of this doctoral thesis provides new contributions to the literature related to the application of pre-fabricated structural façade panels, namely composite sandwich panels for the rehabilitation sector, with special emphasis on the innovative use of recycled steel fibres as discrete reinforcement of the sandwich panel's concrete layers. However, this investigation also lays the ground for further research developments. Taking into account the interdisciplinary nature of a multifunctional rehabilitation approach, which should cover structural, thermal and also acoustic aspects, numerous limitations can be found in the conducted research and many further developments are prompted. This section presents recommendations for future developments that could lead to a more holistic and comprehensive rehabilitation solution based on the research developed so far:

- In order to fully assess the multifunctional performance of the developed sandwich panel, additional experiments need to be conducted in order to evaluate the effectiveness of the proposed rehabilitation solution in terms of building physics criteria, namely by evaluating its thermal and acoustic performance. In terms of thermal performance, experimental tests could be conducted in order to determine the thermal insulation properties of the panel, with focus on its thermal resistance and heat transfer characteristics (the thermal conductivity for each material independently, as well as of the assembled panel). In addition, the sound insulation performance of the panel should also be assessed, especially taking into account that this feature is particularly relevant in the case of the sandwich panel solution that was developed in the scope of this work, which implies the total demolition of existing infill walls (in this case, the panel has to ensure the acoustic airborne sound insulation requirements defined for building envelope walls – see Figure 2.16).
- The fire endurance of the sandwich panels should also be taken into consideration. In this case, since the sandwich panel concept is structurally designed for the seismic action, in a fire scenario they mainly need to fulfil integrity and insulation functions, as defined in EN 1991-1-2 (CEN, 2002). Although specific fire tests were not made, the 50 mm of concrete thickness (two layers of 30+20 mm) together with the thermal insulation layer (50 mm of thickness) should be able to fulfil those requirements, namely for the height of the target buildings the panels were designed

for. However, this is an important topic that does require further specific investigation, namely taking into consideration the following two aspects/limitations: (i) due to the combustibility of the insulation layer, particular attention needs to be given to the joints and edges, which need to be carefully detailed to prevent direct exposure of the insulation layer to fire; and (ii) the reduction of composite action and the risk of delamination of the exterior RSFRC wythe, due to the softening of the GFRP connectors when exposed to elevated temperature, need to be duly taken into account – one needs to guarantee that the tensile (pull-out) strength provided by the GFRP connectors (as well as their anchoring capacity) is sufficient to prevent the delamination of the exterior wythe (in this respect, a redundant metallic anchoring system, connecting the exterior wythe to the RC structure, may be necessary).

- By looking at the objectives that were defined for this research work, it is clear that the development of this sandwich panel solution, to be used for rehabilitation of building façades, involves the fulfilment of multiple functional demands: structural performance, thermal insulation, fire resistance, lightweight, aesthetical appearance, etc. Hence, from a broader point of view, a multi-criteria optimization of the sandwich panel could be conducted aiming at the simultaneous fulfilment of the structural and functional requirements. This could be achieved by the direct application of Building Information Modelling (BIM) methodologies, namely through optimized interoperability frameworks between BIM platforms and simulation software.
- The pushout tests that were conducted for the selection of the insulation layer showed that the adhesion at the interface between concrete and polystyrene layers can have a non-negligible contribution to the structural performance of the panel. The conducted investigation, which considered mainly commercially available polystyrene plates with different surface treatment, could however be extended and deepened by considering a more detailed characterization of the interface between concrete and polystyrene elements, in order to determine specifically which type of surface treatment would yield the best results in terms of adhesion between these materials (considering variables such as the depth, shape and concentration of surface incisions/irregularities).
- The experimental findings that were obtained by performing cyclic tests, complemented by the extensive material characterization that was conducted in the scope of this research, can be used to carry out additional FEM-based numerical simulations, starting first with the numerical modelling of the experimental behaviour obtained for specimens M and S (RC frames that include masonry infill walls and the application of the proposed strengthening solution, respectively),

which could subsequently evolve towards a modelling a complete large-scale RC-frame building structure, aiming to provide additional insights regarding the effectiveness of the proposed rehabilitation solution (numerical simulations comparing original and retrofitted building structures).

- In this research, the sustainability of the sandwich panel was addressed in a qualitative way, by incorporating recycled steel fibres as sole reinforcement of the panel's concrete wythes. However, a detailed energy balance and life cycle analysis needs to be conducted in order to fully weigh the environmental benefits of the panel.
- This research could be regarded as the foundation work for the development of additional/alternative sandwich panel solutions, using the same test setups for direct comparison of different sandwich panel solutions subjected to the same in-plane cyclic loading conditions (possibly out-of-plane loading as well). New iterations would be explored by addressing the deficiencies that were observed experimentally for the developed sandwich panel (aiming to solve eccentricity issues and reduce the damage that was caused to the RC frame specimen). Different materials/components (e.g. pultruded GFRP profiles instead of steel profiles) and rehabilitation approaches (e.g. sandwich panel to be applied to building façade without demolition of masonry infill wall, as represented in Figure 6.1a) could also be explored without losing the basic characteristics that were achieved in this work. 3D illustrations of a possible alternative sandwich panel prototype following this type of retrofit approach are shown in Figures 6.1b and 6.1c.
- The incorporation of smart sensing technology into future iterations of the developed sandwich panel could be an important improvement regarding the sustainability of this rehabilitation solution. Different types of sensors (e.g. optical fibre sensors, piezoelectric sensors or wireless network sensors) for different physical parameters (e.g. temperature, humidity, light, strain or pressure) can be investigated for a possible upgrade into a smart sensing structure to be used in the context of rehabilitation. The sensing capabilities could be used for optimizing the building's energy consumption and also for damage detection purposes.
- Similarly to the previous recommendation, the integration of photovoltaic systems could also be considered. The integration of photovoltaic cells, placed on the visible surface of the sandwich panels (throughout the building envelope), would transform the passive energy saving façade panels (passive due to their insulation layer) into active energy absorbing systems through harvesting solar energy.

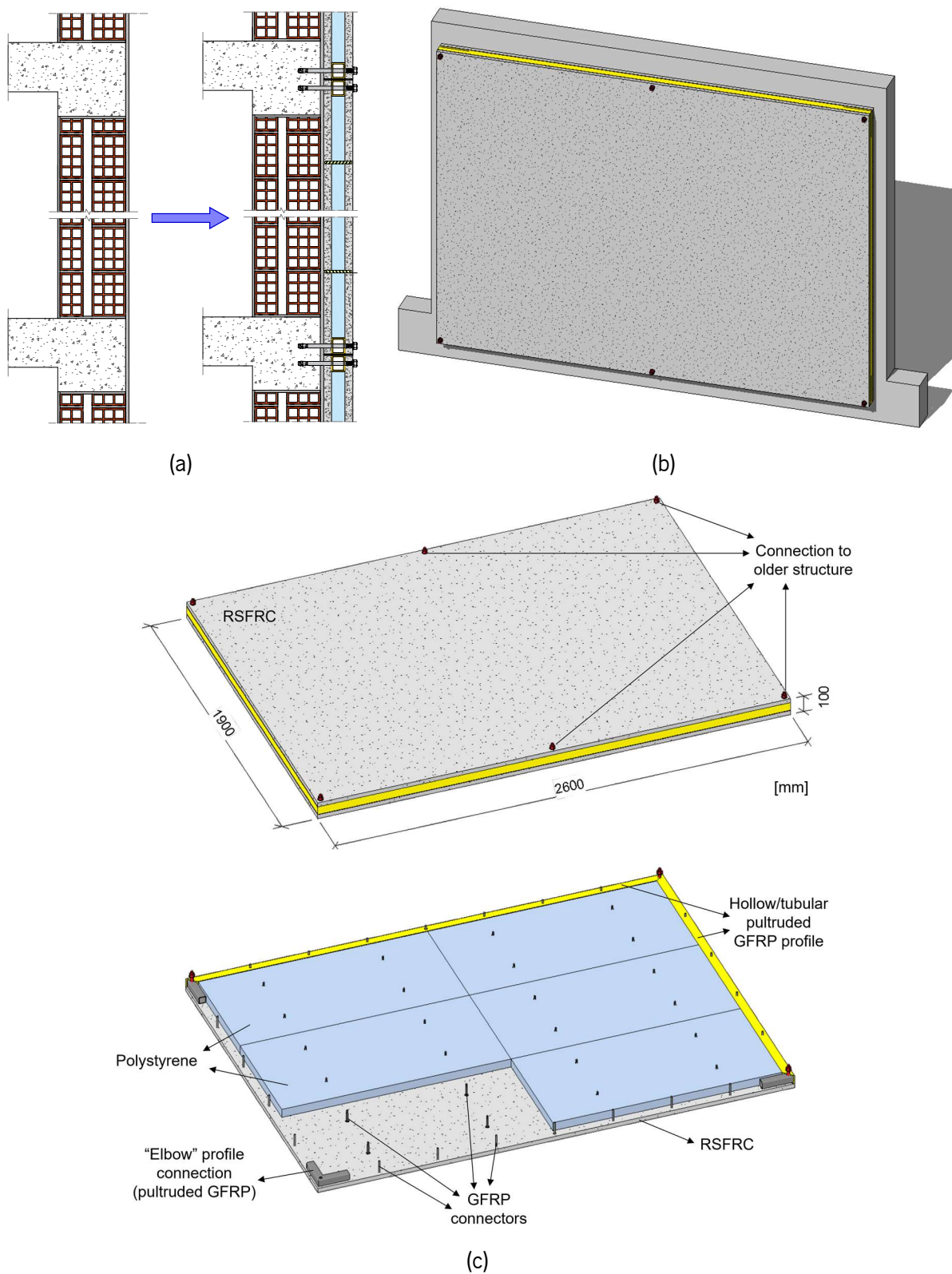


Figure 6.1: Alternative sandwich panel prototype: (a) rehabilitation concept; (b) façade panel applied to RC frame specimen (c) details/components.

REFERENCES

- ACI Committee 369 (2011). Guide for seismic rehabilitation of existing concrete frame buildings and commentary (ACI 369R-11). American Concrete Institute. 35
- Ahmad, S., Elahi, A., Barbhuiya, S. & Farooqi, Y. (2013). Repair of cracks in simply supported beams using epoxy injection technique. *Materials and Structures*, 46, 1547-1559.
- Aiello, M. A., Leuzzi, F., Centonze, G. & Maffezzoli, A. (2009). Use of steel fibres recovered from waste tyres as reinforcement in concrete: Pull-out behaviour, compressive and flexural strength. *Waste Management*, 29, 1960-1970.
- Akhoundi, F. (2016). *Strategies for seismic strengthening of masonry infilled reinforced concrete frames*. PhD Thesis, University of Minho, School of Engineering.
- Akhoundi, F., Vasconcelos, G., Lourenço, P., Silva, L. M., Cunha, F. & Figueiro, R. (2018). In-plane behavior of cavity masonry infills and strengthening with textile reinforced mortar. *Engineering Structures*, 156, 145-160.
- Amaro, B., Saraiva, D., de Brito, J. & Flores-Colen, I. (2013). Inspection and diagnosis system of ETICS on walls. *Construction and Building Materials*, 47, 1257-1267.
- Amaro, B., Saraiva, D., de Brito, J. & Flores-Colen, I. (2014). Statistical survey of the pathology, diagnosis and rehabilitation of ETICS in walls. *Journal of Civil Engineering and Management*, 20, 511-526.
- Antonopoulos, T. A. & Anagnostopoulos, S. A. (2010). Optimum partial strengthening for improved seismic performance of old reinforced concrete buildings with open ground story. In: Fardis, M. N. (ed.) *Advances in performance-based earthquake engineering*. Springer Netherlands.
- Antonopoulos, T. A. & Anagnostopoulos, S. A. (2012). Seismic evaluation and upgrading of RC buildings with weak open ground stories. *Earthquakes and Structures*, 3, 611-628.
- Antonopoulos, T. A. & Anagnostopoulos, S. A. (2013). Improving the seismic performance of existing old pilotis type buildings by strengthening only the ground story. In: Papadrakis, M., Papadopoulos, V. & Plevris, V. (eds.) *4th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*. Kos Island, Greece.
- Arambatzis, S. R. (2020). *Office-user oriented façade design: an interactive/adaptive design approach*. MSc Thesis, Delft University of Technology.
- Araújo, C., Almeida, M. & Bragança, L. (2013). Analysis of some Portuguese thermal regulation parameters. *Energy and Buildings*, 58, 141-150.
- Asdrubali, F., Pisello, A. L., Alessandro, F. D., Bianchi, F., Cornicchia, M. & Fabiani, C. (2015). Innovative Cardboard Based Panels with Recycled Materials from the Packaging Industry: Thermal and Acoustic Performance Analysis. *Energy Procedia*, 78, 321-326.
- Aslani, F., Ma, G., Yim Wan, D. L. & Muselin, G. (2018). Development of high-performance self-compacting concrete using waste recycled concrete aggregates and rubber granules. *Journal of Cleaner Production*, 182, 553-566.

- Azevedo, A. F. M., Barros, J. A. O., Sena-Cruz, J. M. & Gouveia, A. V. (2003). Software in structural engineering education and design. *III Portuguese-Mozambican Conference of Engineering*.
- Azpeitia, I. L., Pando, A. R., Donkervoort, R. & Dijkman, T. (2015). How to design an adaptive wall panel for retrofitting multiple innovative technologies. *VII International Congress on Architectural Envelopes*. San Sebastian - Donostia, Spain.
- Bagarić, M., Banjad Pečur, I. & Milovanović, B. (2020). Hygrothermal performance of ventilated prefabricated sandwich wall panel from recycled construction and demolition waste – A case study. *Energy and Buildings*, 206, 109573.
- Banjad Pečur, I., Štirmer, N., Milovanović, B., Carević, I. & Alagušić, M. (2014). Energy Efficiency Aspects of Recycled Aggregate Concrete. *Proceedings of the International Symposium on Eco-Crete*. Reykjavik, Island.
- Baran, M. & Aktas, M. (2013). Occupant friendly seismic retrofit by concrete plates. *Journal of Zhejiang University*, 14, 789-804.
- Baran, M., Okuyucu, D., Susoy, M. & Tankut, T. (2011). Seismic strengthening of reinforced concrete frames by precast concrete panels. *Magazine of Concrete Research*, 63, 321-332.
- Baros, D., Kyrkos, M., Maravas, A. & Anagnostopoulos, S. (2009). Assessment of Seismic Capacity of Existing Buildings – Effects of Uncertainties. *In: Mucciarelli, M., Herak, M. & Cassidy, J. (eds.) Increasing Seismic Safety by Combining Engineering Technologies and Seismological Data*. Springer Netherlands.
- Barros, J. A. O. (1996). *Behaviour of fibre reinforced concrete. Experimental analysis and numerical simulation (in Portuguese)*. PhD Thesis, Faculty of Engineering, University of Porto.
- Barros, J. A. O. (2016). Debilities and strengths of FEM-based constitutive models for the material nonlinear analysis of steel fiber reinforced concrete structures. *In: Saouma, V., Bolander, J. & Landis, E. (eds.) 9th International Conference on Fracture Mechanics of Concrete and Concrete Structures - FraMCoS-9*. California.
- Barros, J. A. O. & di Prisco, M. (2009). Assessing the possibilities of fibre reinforced concrete for underground prefabricated structures (Report 09-DEC/E-12). University of Minho / Polytechnic of Milan. 70
- Barros, J. A. O., Dias, S. J. E. & Lima, J. L. T. (2007). Efficacy of CFRP-based techniques for the flexural and shear strengthening of concrete beams. *Cement and Concrete Composites*, 29, 203-217.
- Barros, J. A. O., Varma, R. K., Sena-Cruz, J. M. & Azevedo, A. F. M. (2008). Near surface mounted CFRP strips for the flexural strengthening of RC columns: Experimental and numerical research. *Engineering Structures*, 30, 3412-3425.
- Behbahani, A. E. (2017). *Constitutive models to simulate failure of structures made by cement based materials*. PhD Thesis, University of Minho - School of Engineering.
- Bhatt, C. A. F. (2007). *Seismic analysis of reinforced concrete buildings according to Eurocode 8 (in Portuguese)*. MSc Thesis, Instituto Superior Técnico.

- Bob, C. (2004). Evaluation and rehabilitation of a building affected by a gas explosion. *Progress in Structural Engineering and Materials*, 6, 137-146.
- Borzi, B., Pinho, R. & Crowley, H. (2008). Simplified pushover-based vulnerability analysis for large-scale assessment of RC buildings. *Engineering Structures*, 30, 804-820.
- Bournas, D. A. (2018). Concurrent seismic and energy retrofitting of RC and masonry building envelopes using inorganic textile-based composites combined with insulation materials: A new concept. *Composites Part B: Engineering*, 148, 166-179.
- Bragança, L., Almeida, M. G. & Mateus, R. (2007). State of the Art: Portugal. In: Andeweg, M.-T., Brunoro, S. & Verhoef, L. G. W. (eds.) *COST C16 - Improving the quality of existing urban building envelopes: state of the art*. Amsterdam, Netherlands: IOS Press.
- Bruno, S., Decanini, L. D. & Mollaioli, A. F. (2000). Seismic performance of pre-code reinforced concrete buildings. *12th World Conference on Earthquake Engineering (12-WCEE-2000)*. Auckland, New Zealand.
- Buccola, G. 2011. FRP Concrete Strengthening 101. *Luckett&Farley* [Online]. Available from: <http://www.luckett-farley.com/frp-strengthening/>.
- Caballero, A., Willam, K. J. & Carol, I. (2008). Consistent tangent formulation for 3D interface modeling of cracking/fracture in quasi-brittle materials. *Computer Methods in Applied Mechanics and Engineering*, 197, 2804-2822.
- Caggiano, A., Folino, P., Lima, C., Martinelli, E. & Pepe, M. (2017). On the mechanical response of Hybrid Fiber Reinforced Concrete with Recycled and Industrial Steel Fibers. *Construction and Building Materials*, 147, 286-295.
- Carneiro, J. O., Vasconcelos, G., Azevedo, S., Jesus, C., Palha, C., Gomes, N. & Teixeira, V. (2014). The evaluation of the thermal behaviour of a mortar based brick masonry wall coated with TiO₂ nanoparticles: An experimental assessment towards energy efficient buildings. *Energy and Buildings*, 81, 1-8.
- CEN (2000). EN 772-1. Methods of test for masonry units - Part 1: Determination of compressive strength. European Committee for Standardization.
- CEN (2002). Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire.
- CEN (2004). EN 1992-1-1. European Standard Eurocode 2: Design of concrete structures - Part 1-1: general rules and rules for buildings.
- CEN (2005). EN 1998-3. Eurocode 8 - Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings.
- CEN (2007). NP EN 206-1. Concrete - Part 1: Specification, performance, production and conformity. European Committee for Standardization.
- CEN (2009a). EN 12390-3. Testing hardened concrete - Part 3: Compressive strength of specimens.
- CEN (2009b). EN 12390-6. Testing hardened concrete - Part 6: Tensile splitting strength of test specimens.

- CEN (2009c). EN ISO 6892-1. Metallic materials. Tensile testing. Part 1: Method of test at room temperature (ISO 6892-1:2009). European Committee for Standardization.
- CEN (2010). EN 1998-1. European Standard Eurocode 8 - Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings.
- CEN (2013). EN 12390-13. Testing hardened concrete - Part 13: Determination of secant modulus of elasticity in compression.
- CEN (2016). EN 998-2. Specification for mortar for masonry - Part 2: Masonry mortar. European Committee for Standardization.
- CEN (2017). EN 1015-11. Methods of test for mortar for masonry - Part 11: Determination of flexural and compressive strength of hardened mortar. European Committee for Standardization.
- Centonze, G., Leone, M. & Aiello, M. A. (2012). Steel fibers from waste tires as reinforcement in concrete: A mechanical characterization. *Construction and Building Materials*, 36, 46-57.
- Cerâmica Vale da Gândara (2012). Manual for application of ceramic face bricks and pavers (in Portuguese). Viseu, Portugal.
- Choi, K.-B., Choi, W.-C., Feo, L., Jang, S.-J. & Yun, H.-D. (2015). In-plane shear behavior of insulated precast concrete sandwich panels reinforced with corrugated GFRP shear connectors. *Composites Part B: Engineering*, 79, 419-429.
- Choi, W., Jang, S.-J. & Yun, H.-D. (2019). Design properties of insulated precast concrete sandwich panels with composite shear connectors. *Composites Part B: Engineering*, 157, 36-42.
- Coelho, M. (2016). *Bond behaviour of NSM FRP systems in concrete*. PhD Thesis, University of Minho.
- Coelho, M. R. F., Fernandes, P. M. G., Melo, J., Sena-Cruz, J., Varum, H., Barros, J. A. O. & Costa, A. (2012). Seismic retrofit of RC beam-column joints using the MF-EBR strengthening technique. *Advanced Materials Research*, 452-453, 1110-1115.
- Correia, L. (2018). *Durability and long-term behaviour of RC slabs strengthened in flexure with prestressed CFRP laminate strips*. PhD Thesis, University of Minho - School of Engineering.
- Correia, L., Teixeira, T., Michels, J., Almeida, J. A. P. P. & Sena-Cruz, J. (2015). Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems. *Composites Part B: Engineering*, 81, 158-170.
- Costa, P. A. B. S. e. (2015). *Development of sandwich panels of customized functionalities for the rehabilitation of the built patrimony*. MSc Thesis, University of Minho, School of Engineering.
- Cox, B., Syndergaard, P., Al-Rubaye, S., Pozo-Lora, F. F., Tawadrous, R. & Maguire, M. (2019). Lumped GFRP star connector system for partial composite action in insulated precast concrete sandwich panels. *Composite Structures*, 229, 111465.
- D'Angola, A., Manfredi, G., Masi, A. & Mecca, M. (2019). Energy and seismic rehabilitation of RC buildings through an integrated approach: an application case study. *Green Energy Advances*.
- Danosa (2018). DANOPREN FS-P 50. Technical data sheet (in Portuguese). 4

- Dautaj, A. D., Kadiri, Q. & Kabashi, N. (2018). Experimental study on the contribution of masonry infill in the behavior of RC frame under seismic loading. *Engineering Structures*, 165, 27-37.
- Değer, Z. T., Yang, T. Y., Wallace, J. W. & Moehle, J. (2015). Seismic performance of reinforced concrete core wall buildings with and without moment resisting frames. *The Structural Design of Tall and Special Buildings*, 24, 477-490.
- Deutsches Institut für Bautechnik (2011). European Technical Approval ETA-98/0001 - Hilti stud anchor HST, HST-R and HST-HCR. 23
- Deutsches Institut für Bautechnik (2012). European Technical Approval ETA-07/0260 - Injection System Hilti HIT-RE 500-SD for cracked concrete. 31
- DGEG (2014). National strategy for the renovation of buildings (in Portuguese). Directorate-General for Energy and Geology. 33
- Di Sarno, L. & Elnashai, A. S. (2005). Innovative strategies for seismic retrofitting of steel and composite structures. *Progress in Structural Engineering and Materials*, 7, 115-135.
- Dias, S. J. E. & Barros, J. A. O. (2012). NSM shear strengthening technique with CFRP laminates applied in high-strength concrete beams with or without pre-cracking. *Composites Part B: Engineering*, 43, 290-301.
- DRE (2019a). Decree-law 95/2019. Scheme which regulates the rehabilitation of buildings. (in Portuguese). *In: Ministers, P. o. t. C. o. (ed.)*. Lisbon, Portugal: Republic Diary. 35-45
- DRE (2019b). Ordinance 302/2019. (in Portuguese). *In: Housing, M. o. I. a. (ed.)*. Lisbon, Portugal: Republic Diary. 134
- DRE (2019c). Ordinance 305/2019. Acoustic requirements for existent buildings (in Portuguese). *In: Housing, M. o. I. a. (ed.)*. Lisbon, Portugal: Republic Diary. 134
- Du, H., Huang, P. & Jones, P. (2019). Modular facade retrofit with renewable energy technologies: The definition and current status in Europe. *Energy and Buildings*, 205, 109543.
- Duarte, P., Correia, J. R., Ferreira, J. G., Nunes, F. & Arruda, M. R. T. (2014). Experimental and numerical study on the effect of repairing reinforced concrete cracked beams strengthened with carbon fibre reinforced polymer laminates. *Canadian Journal of Civil Engineering*, 41, 222-231.
- Edalat Behbahani, A., Barros, J. A. O. & Ventura-Gouveia, A. (2015). Plastic-damage smeared crack model to simulate the behaviour of structures made by cement based materials. *International Journal of Solids and Structures*, 73-74, 20-40.
- EPS - Empresa de Poliestireno Expandido, L. (2017). EPS 100. Technical data sheet (in Portuguese). 1
- Esmaeeli, E., Barros, J. A. O., Sena-Cruz, J., Varum, H. & Melo, J. (2015). Assessment of the efficiency of prefabricated hybrid composite plates (HCPs) for retrofitting of damaged interior RC beam-column joints. *Composite Structures*, 119, 24-37.
- European Comission (2002). Directive 2002/91/EC of the European Parliament and of the Council of 16 December 2002 on the energy performance of buildings *Official Journal of the European Union*.

- European Commission (2010). Directive 2010/31/EU of the European Parliament and of the Council of 19 May 2010 on the energy performance of buildings (recast). *Official Journal of the European Union*, 13-35.
- European Commission (2012). Directive 2012/27/EU of the European Parliament and of the Council of 25 October 2012 on energy efficiency, amending Directives 2009/125/EC and 2010/30/EU and repealing Directives 2004/8/EC and 2006/32/EC. *Official Journal of the European Union*, 1-56.
- Evola, G., Margani, G. & Marletta, L. (2011). Energy and cost evaluation of thermal bridge correction in Mediterranean climate. *Energy and Buildings*, 43, 2385-2393.
- Expresso (2009). Earthquakes in Portugal – Main construction typologies in the city of Lisbon (in Portuguese).
- Fakhouri, M. Y. & Igarashi, A. (2013). Multiple-slider surfaces bearing for seismic retrofitting of frame structures with soft first stories. *Earthquake Engineering & Structural Dynamics*, 42, 145-161.
- Fédération Internationale du Béton (2011). *fib Model Code 2010* (final draft). *fib*, prepared by fib Special Activity Group 5, based on the draft published as *fib Bulletins* 55 and 56. 653
- FEMA 461 (2007). Interim testing protocols for determining the seismic performance characteristics of structural and nonstructural components. Washington, DC, USA.: Federal Emergency Management Agency. 113
- Ferreira, A. R. P. C. (2007). *Technical solutions for sound insulation in dwellings*. MSc Thesis, Instituto Superior Técnico.
- fibran (2009). fibranXPS ETICS GF. Technical Data Sheet. 2
- Figueira, D., Sena-Cruz, J., Pereira, E., Valente, I., Barros, J., Castro, F. & Soares, D. (2019). Influence of service temperature on shear creep behaviour of a rigid low-density closed-cell PIR foam. *Construction and Building Materials*, 225, 1052-1063.
- Flansbjerg, M., Honfi, D., Vennetti, D., Mueller, U., Williams Portal, N. & Wlasak, L. (2016). Structural performance of GFRP connectors in composite sandwich facade elements. *Journal of Facade Design and Engineering*, 4, 35-52.
- Frankl, B. A., Lucier, G. W., Hassan, T. K. & Rizkalla, S. H. (2011). Behavior of precast, prestressed concrete sandwich wall panels reinforced with CFRP shear grid. *PCI Journal*, 56, 42-54.
- Frazão, C. (2019). *Recycled Steel Fiber Reinforced Concrete for Structural elements subjected to chloride attack: Mechanical and Durability performance*. PhD Thesis, School of Engineering of the University of Minho.
- Frazão, C., Barros, J., Bogas, A., Pilakoutas, K. & de Sousa, C. (2020). An experimental investigation on the post-cracking behaviour of recycled steel fibre reinforced concrete. *Fibre Reinforced Concrete: from Design to Structural Applications. Proceedings of the ACI-fib-RILEM International Workshop - FRC2018*, fib Bulletin 95.
- Frosch, R. (2006). Seismic rehabilitation using infill wall systems. In: Wasti, S. T. & Ozcebe, G. (eds.) *Advances in Earthquake Engineering for Urban Risk Reduction*. Springer Netherlands.

- Furtado, A., Costa, C., Arêde, A. & Rodrigues, H. (2016). Geometric characterisation of Portuguese RC buildings with masonry infill walls. *European Journal of Environmental and Civil Engineering*, 20, 396-411.
- Garcia, R., Jemaa, Y., Helal, Y., Guadagnini, M. & Pilakoutas, K. (2013). Seismic Strengthening of Severely Damaged Beam-Column RC Joints Using CFRP. *Journal of Composites for Construction*, 18, 04013048.
- Garg, N., Kumar, A. & Maji, S. (2013). Significance and implications of airborne sound insulation criteria in building elements for traffic noise abatement. *Applied Acoustics*, 74, 1429-1435.
- Ghobarah, A. (2000). Seismic assessment of existing RC structures. *Progress in Structural Engineering and Materials*, 2, 60-71.
- Gkournelos Panagiotis, D., Bournas Dionysios, A. & Triantafillou Thanasis, C. (2019). Combined seismic and energy upgrading of existing reinforced concrete buildings using TRM jacketing and thermal insulation. *Earthquakes and Structures*, 16, 625-639.
- Gombeda, M. J., Naito, C. J. & Quiel, S. E. (2020). Development and performance of a ductile shear tie for precast concrete insulated wall panels. *Journal of Building Engineering*, 28, 101084.
- Görgülü, T., Tama, Y. S., Yilmaz, S., Kaplan, H. & Ay, Z. (2012). Strengthening of reinforced concrete structures with external steel shear walls. *Journal of Constructional Steel Research*, 70, 226-235.
- Guimarães, M. J. P. H. M. (2011). *Thermal rehabilitation strategies for residential buildings (in Portuguese)*. MSc Thesis, Instituto Superior Técnico, Universidade Técnica de Lisboa.
- Guo, W., Zhai, Z., Cui, Y., Yu, Z. & Wu, X. (2019). Seismic performance assessment of low-rise precast wall panel structure with bolt connections. *Engineering Structures*, 181, 562-578.
- Hamid, N. H. A. & Fudzee, M. F. M. (2013). Seismic performance of insulated sandwich wall panel (ISWP) under in-plane lateral cyclic loading. *International Journal of Emerging Technology and Advanced Engineering*, 3 (3).
- Hashemi, S. J., Razzaghi, J., Moghadam, A. S. & Lourenço, P. B. (2018). Cyclic testing of steel frames infilled with concrete sandwich panels. *Archives of Civil and Mechanical Engineering*, 18, 557-572.
- Hilti (2014). Anchor fastening technology manual. 09/2014. Liechtenstein: Hilti Corporation. 989
- Hodicky, K., Sopal, G., Rizkalla, S., Hulin, T. & Stang, H. (2015). Experimental and Numerical Investigation of the FRP Shear Mechanism for Concrete Sandwich Panels. *Journal of Composites for Construction*, 19, 04014083.
- Hu, H., Papastergiou, P., Angelakopoulos, H., Guadagnini, M. & Pilakoutas, K. (2018). Mechanical properties of SFRC using blended manufactured and recycled tyre steel fibres. *Construction and Building Materials*, 163, 376-389.
- Huang, J.-Q. & Dai, J.-G. (2019). Direct shear tests of glass fiber reinforced polymer connectors for use in precast concrete sandwich panels. *Composite Structures*, 207, 136-147.

- Huang, J., Jiang, Q., Chong, X., Ye, X. & Wang, D. (2020). Experimental study on precast concrete sandwich panel with cross-shaped GFRP connectors. *Magazine of Concrete Research*, 72, 149-162.
- Huanzhi, J., Zhengxing, G. & Jiabin, L. (2017). Composite behavior of sandwich panels with W-shaped SGFRP connectors. *KSCE Journal of Civil Engineering*.
- Iervolino, I., Manfredi, G., Polese, M., Verderame, G. M. & Fabbrocino, G. (2007). Seismic risk of R.C. building classes. *Engineering Structures*, 29, 813-820.
- IHRU (2015). National strategy for housing: challenges and changes. Institute for Housing and Urban Rehabilitation.
- INE (2012). *Census 2011. 15th population census. 5th housing census. Final results - Portugal. (in Portuguese)*, Statistics Portugal.
- Jirsa, J. (2006). Learning from earthquakes to improve rehabilitation guidelines for reinforced concrete buildings. In: Wasti, S. T. & Ozcebe, G. (eds.) *Advances in Earthquake Engineering for Urban Risk Reduction*. Springer Netherlands.
- Kaltakci, M. Y., Arslan, M. H., Yilmaz, U. S. & Arslan, H. D. (2008). A new approach on the strengthening of primary school buildings in Turkey: An application of external shear wall. *Building and Environment*, 43, 983-990.
- Kaplan, H. & Yilmaz, S. (2012). Seismic Strengthening of Reinforced Concrete Buildings. In: Abbas, M. (ed.) *Earthquake-Resistant Structures - Design, Assessment and Rehabilitation*. InTech.
- Kaplan, H., Yilmaz, S., Cetinkaya, N. & Atimtay, E. (2011). Seismic strengthening of RC structures with exterior shear walls. *Sadhana*, 36, 17-34.
- Karayannis, C. G. & Sirkelis, G. M. (2008). Strengthening and rehabilitation of RC beam-column joints using carbon-FRP jacketing and epoxy resin injection. *Earthquake Engineering & Structural Dynamics*, 37, 769-790.
- Kastiukas, G., Zhou, X. & Castro-Gomes, J. (2016). Development and optimisation of phase change material-impregnated lightweight aggregates for geopolymer composites made from aluminosilicate rich mud and milled glass powder. *Construction and Building Materials*, 110, 201-210.
- Kuznik, F., David, D., Johannes, K. & Roux, J.-J. (2011). A review on phase change materials integrated in building walls. *Renewable and Sustainable Energy Reviews*, 15, 379-391.
- Lai, C.-M. & Hokoi, S. (2015). Solar façades: A review. *Building and Environment*, 91, 152-165.
- Lamego, P. R. C. (2014). *Seismic strengthening of residential buildings. Risk analysis and mitigation*. PhD Thesis, University of Minho, School of Engineering.
- Lameiras, R. (2015). *Sandwich structural panels comprising thin-walled SFRSCC and GFRP connectors: from material features to structural behaviour*. PhD Thesis, University of Minho - School of Engineering.

- Lameiras, R., Barros, J., Azenha, M. & Valente, I. B. (2013). Development of sandwich panels combining fibre reinforced concrete layers and fibre reinforced polymer connectors. Part II: Evaluation of mechanical behaviour. *Composite Structures*, 105, 460-470.
- Lameiras, R., Valente, I. B., Barros, J. A. O., Azenha, M. & Gonçalves, C. (2018). Pull-out behaviour of Glass-Fibre Reinforced Polymer perforated plate connectors embedded in concrete. Part I: Experimental program. *Construction and Building Materials*, 162, 155-169.
- Lattke, F., Larsen, K. E., Ott, S. & Cronhjort, Y. (2010). TES EnergyFaçade - prefabricated timber based building system for improving the energy efficiency of the building envelope. *In: Heikkinen, P., Kaufmann, H., Winter, S. & E., L. K. (eds.) Research project from 2008-2009.*
- Li, Y., Darkwa, J., Kokogiannakis, G. & Su, W. (2019). Phase change material blind system for double skin façade integration: System development and thermal performance evaluation. *Applied Energy*, 252, 113376.
- Lourenço, L., Zamanzadeh, Z., Barros, J. A. O. & Rezazadeh, M. (2018). Shear strengthening of RC beams with thin panels of mortar reinforced with recycled steel fibres. *Journal of Cleaner Production*, 194, 112-126.
- Lylykangas, K., Päätaalo, J., Sivunen, M. & Tulamo, T. (2011). *SmartTES: Innovation in timber construction for the modernisation of the building envelope. Project report 26.08.2011*, Finland, Aalto University - School of Arts, Design and Architecture.
- Maalouf, C., Ingrao, C., Scrucca, F., Moussa, T., Bourdot, A., Tricase, C., Presciutti, A. & Asdrubali, F. (2018). An energy and carbon footprint assessment upon the usage of hemp-lime concrete and recycled-PET façades for office facilities in France and Italy. *Journal of Cleaner Production*, 170, 1640-1653.
- Magalhães, S. M. C. & Leal, V. M. S. (2014). Characterization of thermal performance and nominal heating gap of the residential building stock using the EPBD-derived databases: The case of Portugal mainland. *Energy and Buildings*, 70, 167-179.
- Maio, R., Estêvão, J. M. C., Ferreira, T. M. & Vicente, R. (2020). Casting a new light on the seismic risk assessment of stone masonry buildings located within historic centres. *Structures*, 25, 578-592.
- Manfredi, V. & Masi, A. (2018). Seismic strengthening and energy efficiency: towards an integrated approach for the rehabilitation of existing RC buildings. *Buildings*, 8 (36).
- Marani, A. & Madhkhani, M. (2021). Thermal performance of concrete sandwich panels incorporating phase change materials: An experimental study. *Journal of Materials Research and Technology*, 12, 760-775.
- Marini, A., Passoni, C., Belleri, A., Feroldi, F., Preti, M., Metelli, G., Riva, P., Giuriani, E. & Plizzari, G. (2017). Combining seismic retrofit with energy refurbishment for the sustainable renovation of RC buildings: a proof of concept. *European Journal of Environmental and Civil Engineering*, 1-21.
- Marques, A. M., Correia, J. R. & de Brito, J. (2013). Post-fire residual mechanical properties of concrete made with recycled rubber aggregate. *Fire Safety Journal*, 58, 49-57.
- Marques, D. F. F. (2012). *Seismic behaviour of old reinforced concrete buildings (in Portuguese)*. MSc Thesis, Instituto Superior Técnico.

- Marthong, C., Dutta, A. & Deb, S. K. (2012). Seismic Rehabilitation of RC Exterior Beam-Column Connections Using Epoxy Resin Injection. *Journal of Earthquake Engineering*, 17, 378-398.
- Martins, A. P. G. (2018). *Seismic behaviour of masonry veneer walls*. PhD Thesis, University of Minho, School of Engineering.
- Mateus, D. (2008). Building acoustics and noise control (in Portuguese). Faculty of Sciences and Technology, University of Coimbra.
- Mezzi, M. (2006). Enhancing the seismic performance of existing "Pilotis" configurations. *Responding Tomorrow's Challenges in Structural Engineering (IABSE Symposium)* Budapest, Hungary.
- Milheiro, J., Rodrigues, H. & Arêde, A. (2016). Evaluation of the contribution of masonry infill panels on the seismic behaviour of two existing reinforced concrete buildings. *KSCE Journal of Civil Engineering*, 20, 1365-1374.
- Mistretta, F., Stochino, F. & Sassu, M. (2019). Structural and thermal retrofitting of masonry walls: An integrated cost-analysis approach for the Italian context. *Building and Environment*, 155, 127-136.
- Mjörnell, K. (2015). Experiences of using prefabricated elements for adding insulation and upgrading external façades. *1st International Symposium on Building Pathology (ISBP 2015)*. Porto, Portugal. 635-642
- Mofidi, A., Chaallal, O., Cheng, L. & Shao, Y. (2015). Investigation of Near Surface-Mounted Method for Shear Rehabilitation of Reinforced Concrete Beams Using Fiber Reinforced-Polymer Composites. *Journal of Composites for Construction*, 04015048.
- Moghaddasi, N., Zhang, Y. & Hu, X. (2012). Seismic retrofitting of reinforced concrete frame structures using GFRP-tube-confined-concrete composite braces. *Earthquake Engineering and Engineering Vibration*, 11, 91-105.
- Moussa, T., Maalouf, C., Ingrao, C., Scrucca, F., Costantine, G. & Asdrubali, F. (2018). Bio-based and recycled-waste materials in buildings: A study of energy performance of hemp-lime concrete and recycled-polyethylene terephthalate façades for office facilities in France and Italy. *Science and Technology for the Built Environment*, 24, 492-501.
- Müller, F. (2015). *Sandwichelemente mit Deckschichten aus Hochleistungsbeton und einem Kern aus extrudiertem Polystyrol*. PhD Thesis, Technischen Universität Kaiserslautern.
- O'Hegarty, R. & Kinnane, O. (2020). Review of precast concrete sandwich panels and their innovations. *Construction and Building Materials*, 233, 117145.
- Pahn, M. (2013). GFRP-reinforcement bars as fasteners for high heat-insulating multi-layered sandwich panels. *FRPRCS'11: 11th International Symposium on Fiber Reinforced Polymers for Reinforced Concrete Structures*. Guimarães.
- Paiho, S., Seppä, I. P. & Jimenez, C. (2015). An energetic analysis of a multifunctional façade system for energy efficient retrofitting of residential buildings in cold climates of Finland and Russia. *Sustainable Cities and Society*, 15, 75-85.

- Paiva, J. V., Aguiar, J. & Pinho, A. (2006). *Technical guide for housing rehabilitation (in Portuguese)*, Lisbon, Portugal, Instituto Nacional de Habitação - Laboratório Nacional de Engenharia Civil (LNEC).
- Palermo, M. & Trombetti, T. (2016). Experimentally-validated modelling of thin RC sandwich walls subjected to seismic loads. *Engineering Structures*, 119, 95-109.
- Papadopoulos, A. M. (2005). State of the art in thermal insulation materials and aims for future developments. *Energy and Buildings*, 37, 77-86.
- Park, R. (1988). Ductility evaluation from laboratory and analytical testing. *Proceedings of Ninth World Conference on Earthquake Engineering*. Tokyo-Kyoto, Japan.
- PAROC Group (2015). *Rendered façades: thin rendering* [Online]. Available: http://www.paroc.ru/solutions-and-products/solutions/walls/thin-rendering?sc_lang=en [Accessed December 1st 2015].
- Parsaeimaram, M., Fang, C., Luo, X. & Shakya, C. (2018). Seismic Performance Evaluation of Double-Skin Semi-Base-Isolated Building Using Incremental Dynamic Analysis. *Advances in Civil Engineering*, 2018, 2747836.
- Pavlik, Z. & Černý, R. (2009). Hygrothermal performance study of an innovative interior thermal insulation system. *Applied Thermal Engineering*, 29, 1941-1946.
- Pereira, A. C. C. (2013). *The use of GRC in the renovation of buildings (in Portuguese)*. MSc Thesis, Faculdade de Engenharia da Universidade do Porto.
- Pilakoutas, K., Neocleous, K. & Tlemat, H. (2004). Reuse of tyre steel fibres as concrete reinforcement. *Proceedings of the ICE - Engineering Sustainability*, 157, 131-138.
- Pohoryles, D. A., Maduta, C., Bournas, D. A. & Kouris, L. A. (2020). Energy performance of existing residential buildings in Europe: A novel approach combining energy with seismic retrofitting. *Energy and Buildings*, 223, 110024.
- Proença, J. M. & Gago, A. S. (2011). *Seismic strengthening of school buildings*, Parque Escolar, EPE.
- Quesada, G., Rousse, D., Dutil, Y., Badache, M. & Hallé, S. (2012). A comprehensive review of solar facades. Opaque solar facades. *Renewable and Sustainable Energy Reviews*, 16, 2820-2832.
- Raju, K., Cinitha, A. & Iyer, N. (2012). Seismic performance evaluation of existing RC buildings designed as per past codes of practice. *Sadhana*, 37, 281-297.
- RCCTE (1990). Regulation on the characteristics of the thermal performance of buildings (RCCTE) - Decree-Law 40/90 of February 6th (in Portuguese). Lisbon, Portugal.
- RCCTE (2006). Regulation on the characteristics of the thermal performance of buildings (RCCTE) - Decree-Law 80/2006 of April 4th (in Portuguese). Lisbon, Portugal.
- REBA (1967). Code for reinforced concrete structures (REBA) - Decree n° 47723 of May 20th (in Portuguese). Lisbon, Portugal.
- Refaei, F. A. I., El-Mihilmy, M. T. & Bahaa, T. M. (2015). Seismic behavior of sandwich panel walls. *World Applied Sciences Journal*, 33 (11), 1718-1731.

- REH (2013). Regulation of the energy performance of the residential buildings (REH) - Decree-Law 118/2013 of August 20th (in Portuguese). Lisbon, Portugal.
- RGEU (2008). General regulations for urban buildings (RGEU) - Decree-Law n° 38382 of August 7th (in Portuguese). Lisbon, Portugal.
- RGR (2006). General regulation on noise (RGR) - Decree-law 146/2006 of July 31st (in Portuguese). Lisbon, Portugal.
- RIA-Polymers GmbH (2012). RIALENE P 101 SGF50 ST natural. Technical Data Sheet.
- Ribakov, Y. (2010). Reduction of structural response to near fault earthquakes by seismic isolation columns and variable friction dampers. *Earthquake Engineering and Engineering Vibration*, 9, 113-122.
- Ribeiro, F., Sena-Cruz, J., Branco, F. G. & Júlio, E. (2018). Hybrid FRP jacketing for enhanced confinement of circular concrete columns in compression. *Construction and Building Materials*, 184, 681-704.
- Rosa, I. C., Firmo, J. P., Correia, J. R. & Barros, J. A. O. (2019). Bond behaviour of sand coated GFRP bars to concrete at elevated temperature – Definition of bond vs. slip relations. *Composites Part B: Engineering*, 160, 329-340.
- Rozman, M. & Fajfar, P. (2009). Seismic response of a RC frame building designed according to old and modern practices. *Bulletin of Earthquake Engineering*, 7, 779-799.
- RRAE (2008). Regulation on acoustic requirements for buildings (RRAE) - Decree-Law 96/2008 of June 9th (in Portuguese). Lisbon, Portugal.
- RSA (1983). Code for safety and actions for building and bridge structures (RSA) - Decree-Law n° 235/83 of May 31st (in Portuguese). Lisbon, Portugal.
- RSCCS (1958). Code for building safety against earthquakes (RSCCS) - Decree n° 41658 of May 31st (in Portuguese). Lisbon, Portugal.
- RSEP (1961). Code for building and bridge loads (RSEP) - Decree n° 44041 of November 18th (in Portuguese). Lisbon, Portugal.
- Ruano, G., Isla, F., Pedraza, R. I., Sfer, D. & Luccioni, B. (2014). Shear retrofitting of reinforced concrete beams with steel fiber reinforced concrete. *Construction and Building Materials*, 54, 646-658.
- S&P Clever Reinforcement (2018). S&P Resin 55 - Epoxy adhesive (laminating resin).
- Saari, W. K., Schultz, A. E., Hajjar, J. F. & Shield, C. K. (1999). Behavior of shear connectors in steel frames with reinforced concrete infill walls. *Structural Engineering Report No. ST-99-1*. Minneapolis, Minnesota: University of Minnesota - Institute of Technology. Department of Civil Engineering. 189
- Saatcioglu, M., Mitchell, D., Tinawi, N., Gardner, N. J., Gillies, A. G., Ghobarah, A., Anderson, D. L. & Lau, D. (2001). The August 17, 1999, Kocaeli (Turkey) earthquake - damage to structures. *Canadian Journal of Civil Engineering*, 28, 715-737.

- Safarizki, H. A., Kristiawan, S. A. & Basuki, A. (2013). Evaluation of the Use of Steel Bracing to Improve Seismic Performance of Reinforced Concrete Building. *Procedia Engineering*, 54, 447-456.
- Sasmal, S., Ramanjaneyulu, K., Novák, B., Srinivas, V., Saravana Kumar, K., Korkowski, C., Roehm, C., Lakshmanan, N. & Iyer, N. R. (2011). Seismic retrofitting of nonductile beam-column sub-assembly using FRP wrapping and steel plate jacketing. *Construction and Building Materials*, 25, 175-182.
- Schneider Electronics (2018). Inductive sensors XS4. Product data sheet XS4P12AB120. 7
- Schöck Bauteile GmbH (2015). Schöck Combar® - Technical Information. 44
- Sena-Cruz, J., Barros, J. A. O., Azevedo, Á. F. M. & Ventura-Gouveia, A. (2007). Numerical simulation of the nonlinear behavior of RC beams strengthened with NSM CFRP strips. 20.
- Sena-Cruz, J., Correia, L., Teixeira, T. & Michels, J. (2014). Flexural strengthening of RC slabs with prestressed CFRP strips : serviceability and ultimate load state behavior. *1º Congresso Brasileiro de Patologia das Construções*, 1-14.
- Sena-Cruz, J. M. (2005). *Strengthening of concrete structures with near-surface mounted CFRP laminate strips*. PhD Thesis, University of Minho.
- Sezen, H., Whittaker, A. S., Elwood, K. J. & Mosalam, K. M. (2003). Performance of reinforced concrete buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practise in Turkey. *Engineering Structures*, 25, 103-114.
- Shim, H. B., Chung, K. S., Jang, S. H., Park, S. J. & Lee, J. H. (2010). Push-out tests on shear studs in high strength concrete. *Fracture Mechanics of Concrete and Concrete Structures - Assessment, Durability, Monitoring and Retrofitting of Concrete Structures (FraMCos-7)*. Korea Concrete Institute, Seoul, South Korea.
- Shin, J., Kim, J. & Lee, K. (2014). Seismic assessment of damaged piloti-type RC building subjected to successive earthquakes. *Earthquake Engineering & Structural Dynamics*, 43, 1603-1619.
- Shin, J., Lee, K., Jeong, S.-H., Lee, H.-S. & Kim, J. (2012). Experimental and analytical studies on Buckling-Restrained Knee Bracing systems with channel sections. *International Journal of Steel Structures*, 12, 93-106.
- Silva, L., Vasconcelos, G., Lourenco, P. & Akhoundi, F. (2016) Published. Experimental evaluation of a constructive system for earthquake resisting masonry enclosure walls. *In: Modena, C., da Porto, F. & Valluzzi, M. R., eds. 16th International Brick and Block Masonry Conference (IBMAC 2016), 26-30 June 2016. Padova, Italy. 1333-1340.*
- Silva, V., Crowley, H., Varum, H. & Pinho, R. (2015b). Seismic risk assessment for mainland Portugal. *Bulletin of Earthquake Engineering*, 13, 429-457.
- Silva, V., Crowley, H., Varum, H., Pinho, R. & Sousa, L. (2015a). Investigation of the characteristics of Portuguese regular moment-frame RC buildings and development of a vulnerability model. *Bulletin of Earthquake Engineering*, 13, 1455-1490.

- Simões, I. (2014). Building performance inspection and evaluation techniques (in Portuguese). Coimbra, Portugal: ITeCons - Institute for Research and Technological Development in Construction Sciences on behalf of the Energy for Sustainability Initiative of the University of Coimbra.
- Snoeck, D., Priem, B., Dubruel, P. & De Belie, N. (2016). Encapsulated Phase-Change Materials as additives in cementitious materials to promote thermal comfort in concrete constructions. *Materials and Structures*, 49, 225-239.
- Sousa, H., Sousa, F. & Sousa, R. (2015). Thermal mass numerical study - analysis of some factors involved and their importance. *6th International Conference on Mechanics and Materials in Design (M2D'2015)*. Ponta Delgada, Azores, Portugal. 14
- Sousa, H., Sousa, L. C., Castro, C. F. & António, C. C. (2010). Portuguese recent development of high thermal performance masonry systems. *8th International Masonry Conference*. Dresden, Germany. 9
- Sutcu, F., Takeuchi, T. & Matsui, R. (2014). Seismic retrofit design method for RC buildings using buckling-restrained braces and steel frames. *Journal of Constructional Steel Research*, 101, 304-313.
- Theodosiou, T. G. & Papadopoulos, A. M. (2008). The impact of thermal bridges on the energy demand of buildings with double brick wall constructions. *Energy and Buildings*, 40, 2083-2089.
- Thermou, G. E. & Elnashai, A. S. (2006). Seismic retrofit schemes for RC structures and local-global consequences. *Progress in Structural Engineering and Materials*, 8, 1-15.
- Thermou, G. E., Papanikolaou, V. K. & Kappos, A. J. (2014). Flexural behaviour of reinforced concrete jacketed columns under reversed cyclic loading. *Engineering Structures*, 76, 270-282.
- Tomlinson, D. G., Teixeira, N. & Fam, A. (2016). New Shear Connector Design for Insulated Concrete Sandwich Panels Using Basalt Fiber-Reinforced Polymer Bars. *Journal of Composites for Construction*, 20, 04016003.
- Triantafyllou, T. C., Karlos, K., Kapsalis, P. & Georgiou, L. (2018). Innovative Structural and Energy Retrofitting System for Masonry Walls Using Textile Reinforced Mortars Combined with Thermal Insulation: In-Plane Mechanical Behavior. *Journal of Composites for Construction*, 22, 04018029.
- Triantafyllou, T. C., Karlos, K., Kefalou, K. & Argyropoulou, E. (2017). An innovative structural and energy retrofitting system for URM walls using textile reinforced mortars combined with thermal insulation: Mechanical and fire behavior. *Construction and Building Materials*, 133, 1-13.
- Valente, T. D. S. (2019). *Advanced tools for design and analysis of fiber reinforced concrete structures*. PhD Thesis, University of Minho - School of Engineering.
- Valério, J. G. M. A. P. (2007). *Impact assessment of thermal bridges on residential buildings performance (in Portuguese)*. MSc Thesis, Instituto Superior Técnico.
- Ventura-Gouveia, A. (2011). *Constitutive models for the material nonlinear analysis of concrete structures including time-dependent effects*. PhD Thesis, University of Minho, School of Engineering.

- Ventura-Gouveia, A., Barros, J. A. O., Azevedo, A. F. M. & Sena-Cruz, J. (2008). Multi-fixed smeared 3D crack model to simulate the behaviour of fiber reinforced concrete structures. *CCC 2008 - Challenges for Civil Construction*. FEUP, Porto, Portugal.
- Wang, W.-I., Chen, J. & Jin, W.-I. (2017). Experimental Investigation of Corroded Stud Shear Connectors Subjected to Fatigue Loading. *Journal of Materials in Civil Engineering*, 29, 04016175.
- Yossef, M. (2017). *Development of a multifunctional photovoltaic intergated insulated concrete sandwich panel*. PhD Thesis, Iowa State University.
- Youssef, M. A., Ghaffarzadeh, H. & Nehdi, M. (2007). Seismic performance of RC frames with concentric internal steel bracing. *Engineering Structures*, 29, 1561-1568.
- Yu, X., Ji, T. & Zheng, T. (2015). Relationships between internal forces, bracing patterns and lateral stiffnesses of a simple frame. *Engineering Structures*, 89, 147-161.
- Zamanzadeh, Z., Lourenço, L. & Barros, J. (2015). Recycled Steel Fibre Reinforced Concrete failing in bending and in shear. *Construction and Building Materials*, 85, 195-207.
- Zimmermann, M. (2012). ECBCS Annex 50: Prefabricated Systems for Low Energy Renovation of REsidential Buildings. *Project Summary Report*. Hertfordshire, United Kingdom: International Energy Agency - Energy Conservation in Buildings and Community Systems Programme.