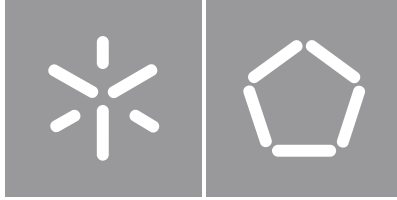




Pier Giovanni Benzo
**A lightweight floor system based on
sandwich panels: characterization and
development**

Universidade do Minho
Escola de Engenharia





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development**

Philosophy Doctorate Thesis

Civil Engineering

Work conducted under supervision of:

Professor Doctor José Manuel de Sena Cruz

Doctor João Miguel Pereira

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STATEMENT OF INTEGRITY

I hereby declare having conducted this academic work with integrity. I confirm that I have not used plagiarism or any form of undue use of information or falsification of results along the process leading to its elaboration.

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RESUMO

O setor da construção deve continuamente transformar-se de modo a mitigar os riscos associados às alterações climáticas. É crucial promover abordagens de projeto e construção que minimizem as emissões de CO₂. As construções modulares e a reabilitação de edifícios existentes podem reduzir o consumo de matérias-primas e de energia, bem como a produção de resíduos. Neste contexto, sistemas de pisos leves são essenciais para tais práticas.

Esta tese apresenta o desenvolvimento de um sistema de piso leve baseado em painéis sanduíche e seus sistemas de ligação que contribuíram para o resultado do projeto de investigação LightSlab. O sistema de pisos destina-se a construções modulares e reabilitação de pisos degradados em edifícios de alvenaria. A unidade modular pré-fabricada do sistema de piso é constituída por um painel sanduíche reforçado com nervuras de aço formado a frio e preenchido com espuma de poliuretano. Este trabalho de investigação visa proporcionar uma melhor compreensão do comportamento mecânico de tais membros estruturais compósitos.

O painel sanduíche foi projetado utilizando ferramentas analíticas para garantir sua segurança estrutural, bem como isolamentos térmicos e acústicos adequados. Um processo de otimização foi conduzido usando algoritmos genéticos para minimizar seu peso, custo e impacto ambiental, ao mesmo tempo em que atende aos padrões estruturais e de construção. A otimização foi validada e estudos foram realizados para examinar como os principais parâmetros de projeto afetam seu desempenho.

O estudo experimental analisa as propriedades mecânicas dos materiais constituintes dos painéis sanduíche e avalia o seu comportamento à flexão estático e dinâmico. Adicionalmente, os sistemas de conexão são avaliados experimentalmente. As juntas de painéis sanduíche usam chapas fixadas nas faces inferiores dos painéis adjacentes com parafusos autoperfurantes. A ligação com a estrutura vertical é materializada por cantoneiras de aço aparafusadas e/ou coladas adesivamente ao painel sanduíche.

Os resultados do programa experimental são usados para verificar ferramentas analíticas e calibrar modelos numéricos com o software ABAQUS. Simulações numéricas identificam os principais aspetos físicos que regem o comportamento do painel sanduíche. Com base nos modelos analíticos validados, são desenvolvidas ferramentas de projeto simplificadas para diversas configurações de suporte e carga para promover a utilização das novas soluções de piso.

Palavras-chave: aço formado a frio; algoritmo genético; espuma PUR; painel sanduíche; modelagem FE.

ABSTRACT

The building sector must continuously transform to mitigate risks associated with climate change. Modular construction and rehabilitating of existing buildings can reduce raw material consumption, energy usage, waste production and minimize CO₂ emissions. Lightweight floor systems are essential for such practices.

This thesis presents the development of a lightweight floor system based on sandwich panels and its connection systems which contributed to the outcome of LightSlab R&D Project. The floor system is intended for weight-sensitive applications like modular constructions and rehabilitation of degraded floors in masonry buildings. The prefabricated modular unit of the floor system is made of a cold-formed steel web core sandwich panel infilled with polyurethane foam. This work aims to provide a better understanding of the mechanical behaviour of such composite structural members.

The sandwich panel was designed using analytical tools to ensure its structural safety as well as adequate thermal and acoustic insulations. An optimization process was conducted using genetic algorithms to minimize its weight, cost, and environmental impact while meeting structural and building standards. The optimization was validated, and studies were performed to examine how key design parameters affect its performance.

The experimental study analyses the mechanical properties of the constituent materials of the sandwich panels and assesses their static and dynamic flexural behaviour. Additionally, an experimental program is carried out to evaluate the connection systems. To ensure interlocking between adjoining panels, a tongue and groove geometry was implemented on the webs of the panel. The adjacent panels are fastened together using cover plates that are attached to the bottom face sheets with self-drilling screws. As for the connection with vertical structures, the sandwich panels are bolted and/or adhesively bonded to steel angles that are connected to the walls.

Experimental program outcomes are used to verify analytical tools and calibrate numerical models with ABAQUS software. Numerical simulations identify key mechanical aspects governing sandwich panel behaviour. Based on the validated analytical models, simplified design tools are developed for various support and load configurations to promote the use of the new floor solutions.

Keywords: sandwich panel; cold-formed steel; polyurethane foam; design and optimization; analytical and FE simulations.

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SYMBOLS

D	Effective bending stiffness
$D_{nt,w}$	Weighted standardised sound pressure level difference
E	Young's modulus
F_{SLS}	Concentrated force corresponding to the characteristic load combination
F_u	Ultimate load
F_{ULS}	Concentrated force corresponding to the bending moment resistance of the cross-section
G	Sear modulus
K	Axial/flexural stiffness
K_{Mi}	Analytical prediction of the flexural stiffness according to design method i
L	Span length
$L_{nt,w}$	Weighted standardised impact sound pressure level
M_t	Matin pool at generation t
N	Size of the population
NFT	Near feasible threshold
P_t	Population at generation t
Q_t	Offspring population at generation t
$R_{Md,Q}$	Live load carrying capacity with respect to bending moment per square meter
$R_{Vd,Q}$	Live load carrying capacity with respect to shear per square meter
U	Shear stiffness
U_c	Corrected thermal transmittance
$\mathbf{X}$	Solution space
b_f	Width of the flange
cst	Cost of the sandwich panel per square meter
$f(\mathbf{x})$	Objective function
$f_p(\mathbf{x}, t)$	Penalized objective function
gwp	Global warming potential of the sandwich panel per square meter
n_w	Number of webs
t_s	Thickness of the cold-formed steel sheet
t_{tot}	Total thickness of the sandwich panel

wt	Weight of the sandwich panel per square meter
$\mathbf{x}$	Variable vector
$\delta_{\max}$	Total vertical deflection
δ_{Mi}	Analytical prediction of the total vertical deflection according to design method i
δ_{SLS}	Displacement at characteristic combination load level
δ_u	Displacement at ultimate load
$\varepsilon_{\text{bottom}}$	Strain in the bottom face sheet
ε_e	Engineering strain
ε_t	True strain
ε_{top}	Strain in the top face sheet
λ	Dynamic search parameter
ν	Poisson's ratio
ρ_{PUR}	Density of the polyurethane foam
σ_e	Engineering stress
σ_t	True stress
σ_u	Ultimate strength
σ_y	Yielding strength
τ_y	Shear yielding strength

ACRONYMS

1L	Panel-to-wall connection with bolts and bottom angle
2L	Panel-to-wall connection with bolts, top and bottom angles
2LA	Panel-to-wall connection with bolts, adhesive, top and bottom angles
AD	Analytical design prediction
Avg	Average value
CFS	Cold-formed steel
CoV	Coefficient of variation
DIC	Digital image correlation
EMW	Effective width method
FRP	Fibre-reinforced polymer
GA	Genetic algorithm
LVDT	Linear variable differential transformer
PUR	Polyurethane foam
SB	Panel-to-panel connection with self-drilling screws and bottom angles
SG	Strain gauge
SW	Panel-to-panel connection with self-drilling screws through the webs of the panels
SBT	Panel-to-panel connection with self-drilling screws, top and bottom cover plates
SLS	Serviceability limit state
S_1	Lightest sandwich panel
S_2	Most economical sandwich panel
S_3	Least environmentally impacting sandwich panel
TGA	Thermogravimetric analysis
ULS	Ultimate limit state

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CHAPTER 1

1. INTRODUCTION

Major environmental problems such as raw materials depletion and waste accumulation are linked to the construction sector [1]. Since the global climate is changing rapidly, there is an urgent need to foster more sustainable construction practices [2].

Structural optimization tools can assist in fully utilizing the potential of construction materials, thereby reducing natural resource consumption and embodied energy of materials. Currently, structural engineers tend to overdesign structural members to minimize design effort. As a result, some elements such as steel beams possess more than 50% of the actual mechanical resistance required by the standards [3].

Modular constructions comprise building modules prefabricated off-site which can lead to substantial resource wastage reduction [4]. On the other hand, rehabilitation of existing buildings consists of prolonging the service life of the existing structures readapting their original function [5]. In addition to preserving the built cultural heritage, the practice of reuse of existing buildings prevents the wasteful process of demolishing and reconstructing new buildings as well as limiting urban sprawl and the use of non-sealed soil [6].

Modular constructions and reuse of existing buildings are both weight sensitive applications requiring the development of low mass structural elements. The former practice imposes constraints on the structural module, which must be transported to the construction site. The latter requires lightweight floor system with a mass comparable to those of degraded timber floors in masonry buildings, which are often found in city centres of Europe. This gives an additional advantage in seismic-prone areas where a low dead weight diminishes the inertial forces generated by the earthquake [7].

Regarding lightweight floor systems, there have been several proposed solutions that mostly involve materials such as fibre-reinforced polymer (FRP) and cold-formed steel (CFS). FRP materials are increasingly used as face sheets for sandwich panels in bridge deck and roof systems. Sandwich panels consist of two thin and stiff face sheets that are separated by a low-density core material [5]. FRP sandwich panels combine structural and building physics functions due to their high strength-to-weight ratio and low thermal conductivity. Additionally, cold-formed steel profiles are also known for their structural efficiency. They can be combined with filling materials to create sandwich structures or dry board, which can serve as the subfloor of the floor system.

In this thesis, a lightweight floor system based on sandwich panels is developed and characterised within the scope of the R&D Project LightSlab – *Development of innovative solutions of slab sandwich panels*. The project is a joint effort of University of Minho's Department of Civil Engineering (DEC) and Department of Polymer Engineering (DEP), and Ferpainel, S.A. Each partner in the R&D project is responsible for specific tasks.

The development of the adhesive connection between the components of the sandwich panels falls under the responsibility of the DEP team. Ferpainel is a manufacturing company specialising in the production of steel insulating sandwich panels for wall and roof cladding systems, as well as cold storage. They are responsible for fabricating the prototypes and making the necessary modifications to their continuous production line to accommodate the new sandwich panel. The scope of this thesis concerns the work carried out by the DEC team, which includes selecting the materials, optimising, and characterising the structural layout of the sandwich panel, and designing the panel-to-panel and panel-to-wall connection systems of the modular floor.

The sandwich panel's design needed to meet the requirements of both building codes and the manufacturing process of Ferpainel the industrial partner of the R&D project. The sandwich panel will be a prefabricated modular unit manufactured in a continuous process.

When working with prefabricated components, structural efficiency plays a significant role in determining the cost and environmental impact of the final product. The aim of the sandwich panel's design is to create a structural member that is lightweight, cost-effective and has a low environmental impact. To achieve these goals, optimisation procedures based on genetic algorithms are used to explore optimal solutions based on different objectives. In addition, the modular floor system needs to be easily transported and assembled, especially in constrained spaces such as the rehabilitation of existing

degraded floors. The system should also be lightweight enough for easy transportation from the factory to the construction site in the context of modular construction.

1.1. RESEARCH OBJECTIVES AND METHODOLOGY

The research in this thesis is intended to demonstrate the integration of optimisation procedures in the design of civil engineering structures, as well as in-depth understanding of the main mechanical feature governing the behaviour of the proposed residential floor system. Furthermore, it intends to provide a database of test results that can be used to establish design tools for structural elements such as built-up and foam filled CFS profiles. These elements are not currently covered by existing building standards, which reduces the material efficiency of lightweight floor systems. The outcomes of this research provide also useful information for the inclusion of essential aspects in the numerical modelling of composite structures and the interaction between the different constituent materials.

The specific objectives of this thesis were set as follows:

1. Identification of the most suitable materials and architectural layout of the sandwich panel;
2. Design of the sandwich panel and optimisation with regards to weight, cost and environmental footprint by means of genetic algorithms;
3. Mechanical characterisation of the constituent materials of the sandwich panel;
4. Development of the connection systems between sandwich panels and between those and the vertical load-bearing structure;
5. Evaluation of the mechanical performance of the sandwich panel, jointed sandwich panels, and the connection systems with the vertical load-bearing structures;
6. Development of numerical simulations to identify the key aspects governing the sandwich panel behaviour, validate analytical models, and assist to the development of simplified design tools;
7. Development of simplified analytical tools and recommendations for the design of the modular floor system compliant with the requirements of the building codes.

The first objective consists of identifying the building code requirements for residential floors and selecting a range of materials for the sandwich panel's core and face sheet that can meet these requirements. The requirements pertain to the structural safety, thermal insulation, acoustic performance, and fire resistance of the floor system.

For the second objective, the available design standards in the context of sandwich panels are reviewed, and a suitable design methodology for the developed prototype is selected. After a preliminary trial-and-error optimization, an optimization procedure based on genetic algorithms is developed. The goal of the optimization is to minimize weight, cost and environmental footprint of the sandwich panel while ensuring that the minimum requirements concerning structural safety, thermal and acoustic performance are met. In the context of prefabricated structural elements, the optimization of the material efficiency of the system plays a crucial role. The algorithm is also intended to be made versatile and accessible to non-experts in order to promote structural optimization tools among practitioners.

The third objective of the project involves conducting an experimental campaign on the constituent materials of the sandwich panel. The aim is to obtain their relevant mechanical characteristics. Additionally, tests are carried out to assess the physical properties of the core materials, such as thermogravimetric analysis. Microscope examinations of the coating layer of the face sheet material are also conducted. These tests provide additional information that helps to understand the influence of the manufacturing process of the core material. Furthermore, the results provide the coating layer design parameter to be included in the development of the simplified analytical tools.

The fourth objective is divided into two main categories, namely, the development of panel-to-panel connection and panel-to-wall connection. A review of current practices is carried out to develop such connection systems, and the most suitable solution is selected based on the fulfilment of code requirements and the facilitation of assembly and maintenance operations.

The fifth objective of the study is to perform flexural tests on small-scale and full-scale sandwich panels in three- and four-point bending configurations. The purpose of testing the single sandwich panels is to evaluate their performance at the serviceability limit state (SLS) and ultimate limit state (ULS), as well as to identify the failure mechanisms. The test results also provided the opportunity to validate the design methodology used to achieve the second objective. In addition, the results of the flexural tests provided a database to calibrate numerical models to gain further insight into the mechanical behaviour of the sandwich panel. Small-scale and full-scale jointed sandwich panels allowed for the study of the connection behaviour along the transverse and longitudinal directions, respectively. The panel-to-wall connection involved tests on full-scale sandwich panels that simulated real-service life conditions. These tests provided information on the failure mechanisms and stress distribution among the sandwich panel and the connection components.

The sixth objective of this project is to develop finite element models that can replicate the behaviour of constituent materials, small-scale steel sandwich panels with PUR foam core, and the dynamic and static structural response of the sandwich floor panels. By comparing the experimental and analytical results with the models, insight into the failure progression and stress state within the different sandwich panel components is provided. Furthermore, this comparison assesses the reliability of simplifying assumptions made during the preliminary design stage.

The seventh objective is a summary of the knowledge obtained from the analytical design, experimental campaign, and numerical modelling, concerning the behaviour of a sandwich panel. Using the validated results of the design procedure, a simplified analytical model is proposed as a design guideline for the sandwich panel floor system.

1.2. OUTLINE OF THE THESIS

The thesis is organized in eight chapters. A brief summary of each chapter is presented in the next paragraphs.

In the current chapter, the motivation and context of the thesis are presented. As previously mentioned, the thesis has been developed within the scope of the LightSlab R&D project, which is briefly introduced. Additionally, the research objectives and the research methodology adopted for this thesis are discussed.

The literature review in Chapter 2 includes a discussion of lightweight structural systems based on sandwich panels as well as CFS structural members. The mechanical behaviour of sandwich panels is briefly introduced, and the challenges of improving the performance of the core while maintaining a reduced weight are discussed. Regarding CFS profiles, the current research trend is to enhance their resistance against instability phenomena. Furthermore, the accuracy of current analytical tools available for predicting CFS structural systems, considering their interaction with possible filling and subfloor components, is discussed. Finally, the application of a genetic algorithm procedure for optimizing the structural design of civil engineering structures is presented, with a focus on including constraints in the formulation of the optimization problem.

Three sandwich panel solutions based on the combination of different core, webs and face sheets materials are presented in Chapter 3. Among these solutions, the one that is based on CFS face sheets and webs and polyurethane (PUR) foam core is selected for the optimization process due to its low cost and Ferpainel's expertise in manufacturing steel insulating sandwich panels. The preliminary structural,

thermal, and acoustic designs of the selected solution are described in detail. Additionally, the effective width method for designing cold-formed steel members is discussed, and the proposed design methodology used in this thesis is presented along with its underlying assumptions. The thesis also illustrates the development and validation of the original optimization program based on genetic algorithms. The optimal solutions in terms of weight, cost, and environmental footprint are presented along with parametric studies on the influence of the most relevant design parameters. Finally, the most economical solution is adopted for the production of the sandwich panel prototypes. The problems faced during different stages of the manufacturing process of the prototypes and their respective solutions are discussed.

The mechanical characterisation of the constituent materials of the sandwich panel is presented in Chapter 4. Tensile tests are conducted on CFS coupon specimens extracted from coils and later from the face sheets and webs of sandwich panels. A literature review is presented on how the manufacturing process affects the microstructure and mechanical properties of PUR foam. The PUR foam core material is evaluated under different loading conditions, namely flatwise compressive and tensile tests, as well as shear tests. Moreover, thermogravimetric analyses are carried out to obtain information on the physical properties of the foam. Regarding the PUR-based adhesive developed by the DEP team for joining the components of the sandwich panels, its mechanical performance is characterised by means of tensile tests on dogbone specimens and single lap shear tests. Edgewise compressive tests are carried out on small-scale sandwich panels without webs to preliminarily assess the quality of the composite structure. The results of the edgewise compressive test are also compared to the analytical prediction of the axial stiffness and ultimate load of the sandwich panel.

In Chapter 5, the four-point bending test results conducted on a small-scale prototype of the final sandwich panel are presented. The design methodology employed in Chapter 3 is validated using these results. The full-scale sandwich panel's dynamic and static behaviour is investigated through flexural dynamic tests, three- and four-point bending tests. The analytical predictions are compared with the experimental results, considering different assumptions on the level of contribution of the core to the overall mechanical performance of the sandwich panel. Additionally, the influence of assuming the gross and effective properties of the CFS cross-section is also assessed.

The development and characterisation of the panel-to-panel and panel-to-wall connection systems are presented in Chapter 6. After reviewing the literature on the currently used panel-to-panel joints, the analytical design of the proposed connection system is described. Three different connection systems

were tested using three-point bending on joint small-scale sandwich panels in the transverse direction. Considering factors such as the ease of assembly of the joint and mechanical performance, the connection system based on a cover plate fastened by means of self-drilling screws to the bottom face sheet of the sandwich panel is adopted for full-scale testing. Four-point bending tests are carried out on joint full-scale sandwich panel to assess the mechanical behaviour of the connection in the longitudinal direction. In the second stage, the topic of the connection between floor systems and the existing vertical structure is reviewed. The proposed connection systems are illustrated, and their performance is investigated by means of four-point bending tests.

The numerical modelling of the constituent materials and dynamic and static flexural behaviour of the sandwich panel is presented in Chapter 7. The material models for the CFS face sheet and PUR foam core have been validated against the results obtained from the mechanical characterization campaign. The strategies used to replicate the mechanical behaviour of the full-scale sandwich panel are discussed, highlighting the key physical features that should be included to obtain reliable results. Finally, based on a comparison of the analytical and numerical models against experimental results, a simplified design tool is proposed. This tool is based on the most conservative prediction of the sandwich panels' performance.

The conclusions drawn from the work developed in this thesis are summarised in Chapter 8. Potential solutions for addressing the issues that arose during the development of sandwich floor panels are outlined, and suggestions are provided for future research work that involves both experimental and numerical analyses.

CHAPTER 2

2. LIGHTWEIGHT STRUCTURES AND MATERIAL EFFICIENCY

The slenderness of the structural composing elements and the lightness of the look have always been a concern for the design of structures. In the past, when technology was not yet advanced enough, perspective helped disguise squat and heavy elements necessary for the stability of the structure, such as in the Parthenon [8]. As the master constructors accumulated experience, they pushed the limit of natural materials in search of highly slender structures. This effort is still vivid in the architecture of Gothic cathedrals [9]. Iron, steel, and reinforced concrete emerged as new building materials capable of outperforming stone, wood, and bricks after the Industrial Revolution [10]. Steel bridges and skyscrapers are nowadays some of the most iconic structures of the human metropolis.

Currently, the search for efficient and lightweight building materials is led not only by appeasing the human eye. The sustainability challenge and rapidly progressing climate change must be addressed by the construction sector [11]. As technological advancements progress, new combinations of materials are made possible. New optimisation approaches based on machine learning allow us to improve the design of structural systems in terms of cost, mass, and environmental footprint [12], [13]. The advantages of lightweight structures include, among others, reduced raw material consumption, ease of transportation and installation, decreased manufacturing costs and construction time [14]. In the following paragraph, a brief description of structural systems recently developed for weight-sensitive applications in civil engineering is presented. Further detail on each system will be provided in the next sections.

Structural systems based on sandwich panels, cold-formed steel (CFS) profiles or a combination thereof have been employed and studied in weight-sensitive applications in civil engineering in recent years. Sandwich panels are made of two thin face sheets separated, usually, by a low-density core material (see

Figure 2.1a). The face sheets are generally composed of stiff and high-strength materials whereas the core is relatively light and provide adequate stiffness in the direction normal to the faces of the panel [15].

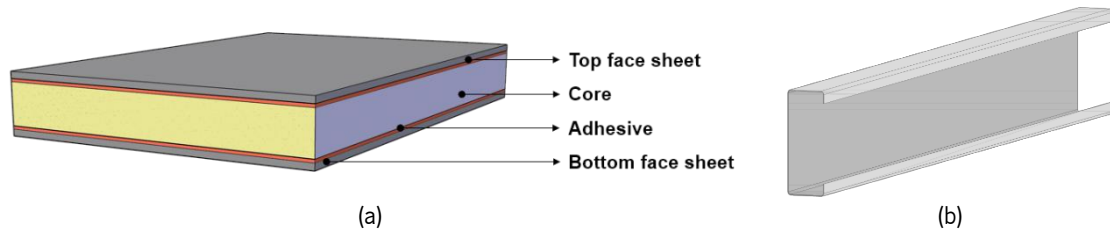


Figure 2.1. Lightweight structural members: (a) sandwich panel; (b) CFS-lipped channel section.

CFS profiles are produced by bending flat sheets of metal at room temperature into shapes that will provide a higher load-bearing capacity than the original flat sheets (see Figure 2.1b) [16]. Sandwich panels and CFS profiles have been used in weight-sensitive applications such as temporary shelters, bridge decks, façade cladding, roof systems, modular constructions, and the rehabilitation of degraded floors in existing buildings ([17], [18]). The corresponding benefits of these lightweight structural systems for each application are summarised hereafter.

In the context of sandwich panels, fibre-reinforced polymer (FRP) face sheets provide strength properties similar to those of metals, with a weight approximately four times lighter than metallic materials [15]. Lightness is a key factor when developing a temporary house for disaster-hit areas (see Figure 2.3a) since accessibility to roads is usually limited [19]. Thus, such composite solutions show potential to be employed as modular systems in temporary houses [18].



Figure 2.2. Current and potential applications of composite sandwich panels: (a) temporary shelter [18]; (b) bridge decks [20].

Bridge deck solutions based on composite sandwich panels provide longer lifespans eliminating structural integrity issues related to corrosion [21]. Furthermore, lightweight panels allow for rapid construction time and minimum traffic disruption when replacing old concrete deck slabs (see Figure 2.3b) [20]. Flat or

lightly profiled metal sheets with low-density polymeric foam cores possess a high strength-to-weight ratio, durable thermal insulation, and can be rapidly erected and installed even in extreme weather conditions, and economically mass-produced [22].

Metal sandwich panels are already a mature product in several industries, including the industrial partner of the LightSlab R&D Project, and are commonly used to clad low-rise industrial and commercial buildings (see Figure 2.3a). Nevertheless, their use has been restricted to non-structural or secondary structural elements [17].

CFS C-section profiles generally constitute the framing of the load-bearing unit of modular constructions (see Figure 2.3b) [23]. The CFS framing may be combined with sandwich wall panels, though further studies are required in view of their application in residential buildings [4]. The use of lightweight materials for modular construction is necessary due to the limit imposed by the unit transportation. Off-site manufacturing of the units offers several advantages such as reduced construction schedule, improved quality, and reduced resource wastage [24].



Figure 2.3. Current and potential applications of CFS sandwich panels and profiles: (a) Esso headquarters in Leatherhead (UK) [25]; (b) CFS framing of a modular unit [24].

Finally, sandwich panel- and CFS-based structural systems show potential to be exploited as lightweight floor systems for the rehabilitation of existing buildings. Degraded timber floors are commonly found in several types of masonry buildings across Europe (see Figure 2.3) [26].



Figure 2.4. Degraded timber floor in an existing building in Porto (PT).

Replacement with new timber floors does not appear a feasible solution due to the limited durability and the costly maintenance required during its service life [12]. Thus, the development of new solutions with comparable mass and improved mechanical properties is required. When such buildings are located in seismic-prone areas, lightweight floors are desirable to decrease the inertial forces generated by the earthquake [27]. Furthermore, the practice of reuse of existing buildings contributes positively to the reduction of construction waste, preservation of cultural heritage, and reduction of urban sprawl and the use of non-sealed soil [28].

The mitigation of the environmental impact of the construction sector may be addressed on two levels: i) the development of material-efficient structural systems; and ii) the implementation of such systems in construction methods which reduce the use of raw materials, waste production, and urban sprawl.

Regarding the former level, new optimisation methods have been developed to aid the design of material-efficient and low-environmental-impact structures. There is an increasing interest in applying metaheuristic search techniques in structural design due to the presence of conflicting interests [29]. Indeed, in the construction sector, the cost-to-performance ratio is often of primary importance [22]. This may conflict with the achievement of other features, such as low mass and low environmental impact. Furthermore, conflicts may arise from the wide set of requirements imposed on structural elements by the building codes, i.e. structural safety, and thermal and acoustic performances [30]. These optimisation procedures have been researched extensively in the literature, but they are not frequently applied in practice [3]. A more problem-oriented approach focused on the need of structural designers is required to introduce optimisation methods in daily engineering.

After providing a brief overview of lightweight structures, the optimisation tools used for such structures, and their current and potential applications, the specifics of the ongoing research in these various fields will be addressed in the next sections.

The literature review reflects the organised chronological progression of the research. The state-of-the-art discusses the essential topics that were required for the development of the new sandwich floor panel design. Additional areas that were extensively studied will be outlined in dedicated chapters of this thesis to maintain a smooth flow of the narrative.

An overview of the current standards for designing sandwich panels and CFS structural systems is provided in Chapter 3. A discussion on the characterisation of the PUR foam used as core material in sandwich panels is presented in Chapter 4. Particular attention is given to the connection systems used for the floor's structural modules as well as the vertical load-bearing structure (see Chapter 6). Moreover, the numerical techniques employed to replicate the mechanical performance of sandwich panels are reviewed in Chapter 7.

The literature review has been conducted with a focus on exploring potential lightweight structural systems that could be employed in weight-sensitive applications in civil engineering. The areas of interest include the rehabilitation of degraded timber floors in existing buildings, as well as the use of structural elements in modular buildings. The research on sandwich panels has primarily been concerned with improving the mechanical performance of the core while keeping the weight of the composite structure at a minimum. Studies have been conducted on strategies to enhance the resistance of CFS (cold-formed steel) structures against local and distortional instabilities. Analytical tools to consider the interaction between structural and non-structural components with the aim of maximising the material efficiency of the system have also been investigated. Additionally, metaheuristic optimisation techniques have been examined as tools for obtaining lightweight, cost-saving, and environmentally friendly solutions. Particular attention has been given to genetic algorithms (GA) and how various studies have implemented building code requirements within them.

2.1. SANDWICH PANELS

As mentioned before, sandwich panels are generally constituted of two thin face sheets joined together by a relatively thick and low-density core. A wide variety of combinations of face sheets and core materials

may be obtained to satisfy the requirements of each specific application. In the construction sector, common face sheet materials are divided into metallic and non-metallic face-sheet.

Metal-insulated sandwich panels with polyurethane (PUR) foam or mineral wool core are already widely employed as façade cladding and roofing systems. The widespread use of this type of panel is mostly attributable to their highly standardised and economic manufacturing process [22]. The liquid monomers of the PUR foam are injected on the bottom face sheet, where they polymerise and cross-links, providing an adhesive bond in a single-step process called sandwich foaming [31]. Both the thermal insulation and the bonding with the face sheets are provided by the foaming process.

The PUR foam and mineral wool possess good thermal insulations and stabilise the face sheets against buckling phenomena. The metal face sheets, in exchange, protect the core from moisture and provide the required strength and stiffness (for bending, membrane and shear actions). Nevertheless, their use has been restricted to non-structural or secondary structural applications designed to only support their self-weight and variable actions such as wind pressure and snow loads.

FRP face sheet sandwich panels have also registered an increase in civil engineering structural applications. The advantage of composite sandwich panels is combining structural and building physics functions due to their low thermal conductivity and the high strength-to-weight ratio [32]. The most common production method of FRP sandwich panels involves two phases: i) prefabricated foam blocks must be cut and then ii) adhered to both face sheets by manual operation. These foams are manufactured in bulk, and their properties are relatively well-controlled [33]. On the other hand, the dust resulting from the machining of the foam blocks may weaken the bond quality between the core and face sheets [34].

2.1.1. Mechanics of sandwich panels

The mechanical behaviour of a sandwich panel may be compared to that of an I-beam: most of the material is situated in the flanges that are further from the neutral axis of bending; there is only sufficient material remaining in the web that is responsible for enabling the flanges to function cohesively and withstand shear. In a sandwich panel, the face sheets replace the flanges, and the core replaces the web [35].

When subjected to a load in the direction normal to its plane, the face sheets are mostly responsible to carry the bending stresses, whereas the core carries the shear stresses. Furthermore, the core offers continuous support to the face sheets rather than the concentrated support provided by webs. The

adhesive bond resists shear and tensile stresses arising between the face sheets and the core ensuring that the two components work as one cross-section.

Despite the high strength- and stiffness-to-weight ratios, the structural use of sandwich panels for building floors and bridge decks is not yet feasible for standard spans and loads [36]. The main drawbacks are the debonding between the core and the face sheets and the insufficient strength or stiffness of the core material [22]. Regarding the latter issue, shear deformation is generally neglected for beams with homogeneous cross-sections unless a short span length is considered. However, it must be considered for beams with low shear stiffness, such as the case of sandwich panels [15].

Due to the significant permanent loads imposed on many civil engineering structures, creep deformations may also occur as a result of the viscoelastic nature of polymeric adhesives, resins and foams [32]. Nevertheless, the creep behaviour of those materials is out of the scope of this thesis, and it is acknowledged in the last chapter as a topic to address in the future. For the reasons mentioned above, particular attention is dedicated to the research involving the improvement of the flexural behaviour of sandwich panels with low-density core materials with low shear strength and stiffness.

2.1.2. Structural sandwich panels

As the performance of sandwich panels heavily relies on the core layer, extensive research has been conducted on core reinforcing techniques. The commonly used methodology involves evaluating the flexural performance via three and four-point bending tests [37], [38], [39] and the in-plane behaviour through edgewise compressive tests [36], [40]. These experimental assessments enable the identification of failure mechanisms and provide a reliable database for the validation of numerical models. Through numerical modelling it is possible to expand the investigation of the most influential parameters affecting the mechanical behaviour of sandwich panels [41], [42].

The addition of yarns and/or pins to connect the top and bottom face sheets is a common technique used to enhance debonding and delamination resistance of composite sandwich panels [43], [44], [45]. A study in [46] compares the performance of polymer pin-reinforced foam core composite sandwich panels to that of unreinforced high-density foam core sandwich panels (see Figure 2.5). The results of flexural tests showed that the pin-reinforced sandwich panels have a peak load and corresponding ultimate deformation that is 40% higher than the unreinforced panels. Furthermore, an increase in pin diameter resulted in a slight improvement of the above-mentioned parameters.

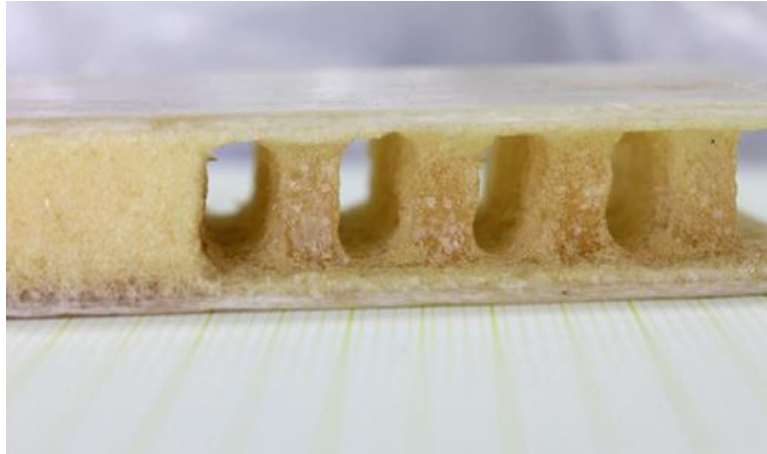


Figure 2.5. Composite sandwich panel with partially removed foam to show the pin reinforcement [46].

The stitching reinforcement of foam core composite sandwich panels is addressed in [47]. Flexural tests were performed on non-stitched, stitched, and stiffened sandwich panels. The stiffened sandwich panel consists of placing a longitudinal web at the mid-width of the sandwich panel. Both static and fatigue performance were assessed. Although there was a slight improvement between the non-stitched and stitched panels, the stiffened sandwich panel was found to outperform both by an order of magnitude in terms of peak load and bending stiffness.

The conventional process of pinning and stitching to improve the debonding resistance of composite sandwich panels is known to be a time-consuming and costly fabrication process [48], [49]. As an alternative, the insertion of semi-circular shear keys made of glass fibres and epoxy resin on the inner surface of the face sheets of sandwich panels has been proposed [50]. These shear keys are placed in foamed-core grooves before the resin infusion process to enhance the debonding performance of composite sandwich panels. The use of such shear keys has been observed to provide an approximately 25% improvement in debonding resistance, as demonstrated by the results of edgewise compressive tests. Additionally, in the case of high-density polyethylene face sheet sandwich panels with high-density PUR foam core, the use of shear keys in the form of studs placed at alternate spacing on the inner surface has also been addressed [51]. In-plane compressive tests have revealed that wrinkling occurs between two consecutive studs, which limits the progression of debonding.

Multilayered core sandwich panels have been studied through flexural tests to enhance their load carrying capacity. By gluing together stacks of the core material in either the edgewise or flatwise position, higher load carrying capacity can be achieved to meet the requirements of specific applications [52], [53]. Additionally, combining several layers of different types of core materials or varying densities of the same

core material in regions of the sandwich panels that require specific properties can significantly improve their structural performance [54], [55].

In [56], a structurally graded core is designed to reduce the weight of a GFRP sandwich panel with balsa wood core. High-density balsa wood is employed in the upper layer of the core and in the support regions to increase resistance to wrinkling and shear action, respectively. In the low-stressed zone of the core a low-density balsa is utilized to reduce the overall weight of the panel. An arch shaped GFRP laminate is used to separate the two types of balsa wood. The arch-type beam performance was assessed by means of flexural test and its resistance compared to those of homogeneous high density balsa core layer beams. The arch-type beam demonstrated higher stiffness and flexural load than the homogeneous beams with a reduction in weight of about 15%.

A steel sandwich panel with a hybrid core made of layers of two different materials is studied in [57]. Sandwich beams with a PUR foam layer and a mineral wool layer are manufactured with different proportions of the core materials (see Figure 2.6). The aim is to combine the benefits of both cores, the mineral wool ensures suitable fire resistance and the PUR foam layer provide adequate thermal insulation to obtain a relatively low-weight sandwich panel. The results of four-point bending test showed that employing thicker layer of mineral wool in the core corresponded to a decrease in the ultimate load of the sandwich panels.



Figure 2.6. Four-point bending test of the hybrid core steel sandwich panel [57].

One promising solution for reinforcing the core of sandwich panels is the use of webs [58]. This solution is cost-effective and allows for the use of lower-strength core materials that are already available while keeping the production process simple. The insertion of webs decreases the contribution of the core material to the overall mechanical behaviour of the sandwich panel [59]. In fact, flexural tests on web-reinforced sandwich panels with PUR foam and polypropylene honeycomb core have shown that the difference in strength and stiffness between the panels is negligible. The location of the webs also plays an important role in improving the flexural properties, but it's important to note that one-way bending tests cannot be used to assess the effectiveness of transverse web reinforcement [19].

In research, it has been found that the effectiveness of the web in reinforcing structures is similar to increasing the density of the core material. A study showed that a sandwich panel with a 3% core cross-section occupied by longitudinal webs had equivalent flexural properties to a similar panel with twice the core material density [59] (see Figure 2.7a). High-density core sandwich panels are considered a solution that underutilizes the composite face sheet, as its longitudinal strain is found to be far within the ultimate strain at the peak of the flexural test [60]. Similar conclusions were drawn in another study where a GFRP sandwich beam with inclined longitudinal webs, conferring it a trapezoidal cross-section shape, with low-density PUR foam core was compared to an unreinforced GFRP beam with high-density PUR foam core [61]. The flexural test showed that the web-reinforced beam was more than twice as stiff as the unreinforced beam (see Figure 2.7b). Additionally, in the web-reinforced beam, the shear deflection was estimated to be around 1%, demonstrating that the shear stresses are almost completely carried by the webs.



Figure 2.7. Structural sandwich panels: (a) web-reinforced sandwich panel subjected to distributed load test [59]; (b) inclined longitudinal web sandwich panel failure in the four-point bending test [61].

The use of low-density core materials is required to achieve the thermal insulation properties required by building codes, making the web-core system an advantageous option for metal sandwich panels. In a study of roofing system applications, a box beam composed of CFS face sheets and C-section profiles acting as webs was investigated [62]. Three and four-point bending tests were performed to assess the resistance of the metal webs to bearing failure and shear buckling. Analytical models estimate that the PUR foam contributes to almost 80% of the web bearing failure resistance at the supports. Another study focused on the flexural resistance of steel web-core sandwich panels. The web core system included trapezoidal sheeting welded to the face sheets. In flexural tests, the panel tended to fail due to instability phenomena such as wrinkling and local instability of the webs at midspan [63].

There are various methods being used to improve the mechanical behaviour of sandwich panels in order to make them suitable as primary structural elements. Among these methods, the web-core system has shown to be the most effective. However, it adds significant weight which can be a hindrance in replacing degraded timber floors in existing buildings.

For composite face sheet sandwich panels, it is possible to enhance the mechanical properties of the polymeric core material, as both materials have high thermal performance. However, sandwich panels with high-density core materials tend to underutilise the composite material and fail in a brittle manner due to debonding between the core and the face sheet. To address this, low-density foam core materials combined with longitudinal webs are an attractive solution to reinforced steel face sheet sandwich panels, particularly for thermal insulation performance. Steel web-core sandwich panels are also ductile, which is a desirable feature for primary structural elements.

2.2. CFS STRUCTURES

CFS members have been extensively researched in recent years, and their use as primary structural elements has consequently increased [64]. The main advantages with respect to hot-rolled steel elements are the following ones: i) wider possibilities of cross-section shapes which leads to more efficient material usage [65]; and ii) ease of transportation, installation, and connection [66]. Nevertheless, CFS profiles are sensitive to local and global instabilities due to the thinness of the plate elements. Therefore, different strategies have been developed to improve the resistance to instability phenomena which can be grouped into three categories: i) lightweight floor system consisting of CFS components, self-drilling screws, and a dry board topping layer; ii) CFS profiles infilled with PUR foam; iii) CFS built-up members.

In the following sections, a review of the various strategies used to reduce the slenderness of the plate elements which compose the cross-section of CFS profiles is presented. Despite the different methods employed, the literature highlights common challenges. There is a general lack of understanding of the physical features of the structural system, such as the behaviour of the interfaces between different materials or the degree of interaction provided by connection systems (be it adhesive or mechanical). Due to this lack of understanding, analytical tools have so far failed to incorporate these aspects into design standards. Simplified analytical models which exclude such features have been proposed with varying degrees of success.

2.2.1. Lightweight CFS floor system

Lightweight floor systems are based on the combination of CFS profiles and a topping layer which may consists of dry boards of different materials as well as hot-rolled steel plates [67], [68]. Nevertheless the current design codes on CFS members do not include any provision on the degree of composite action to be considered between the boards and the CFS profiles.

The assessment of the degree of composite action has been addressed by comparing numerical and simplified analytical models with the results of experimental tests [69]. Analytical model generally considers a transformed T-shape cross-section where the floorboard material is converted into an equivalent area of steel material [70]. The validated numerical models may then be used to explore the influence of different parameters on the degree of composite action reached by the composite structures such as thickness of the CFS profiles, spacing and type of the shear connectors, thickness and type of dry boards, and depth of the CFS profiles [64].

In [71] various shear transfer mechanisms between CFS C-lipped profiles and wooden floorboards are investigated, namely self-drilling screws, wood adhesive and epoxy resin. Four-point bending test are carried out on the composite structures and the results are compared to the performance of bare steel sections and analytical models (see Figure 2.8). Furthermore, the tests revealed that the highest level of composite action is achieved by bonding the CFS beams and floorboards with epoxy resin. The analytical approach considers a perfect bond between the floor components and a plastic distribution of stresses. The ultimate load ratio between experimental and analytical prediction is found to be between 0.4 and 0.9 depending on the shear transfer mechanism. Regarding the estimation of the flexural stiffness, the analytical predictions show a better agreement with the experimental results with the ratio included between 0.6 and 0.85.

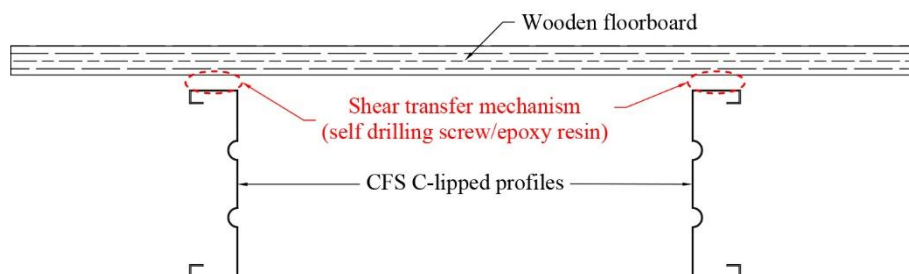


Figure 2.8. Four-point bending test of the CFS beams and wooden boards floor system [71].

2.2.2. PUR foam infilled CFS profiles

Foam-filled CFS profiles can provide significant benefits in terms of mechanical performance, particularly regarding the bending resistance [72]. The PUR foam filling provides a continuous support for the thin plate composing the CFS cross-section thus improving the buckling resistance. Furthermore, with a little increase in weight, major improvement in terms of shock absorption, acoustic and thermal insulations can be achieved [73]. Nevertheless, to optimise the design of such structures further experiments are required on the single constituents and the composite members as well as the development of adequate design methods [74].

The flexural performance of two types of CFS built-up members is compared in [75]. The first type is a bare CFS built-up member consisting of two main profiles laced together by L-profiles. The second type is the same member encased in CFS sheets and filled with PUR foam having a density of 40 kg/m³. Based on the three and four-point bending configuration, flexural tests were conducted to investigate the influence of foam filling on the bending and shear response of the beam. The results of the tests indicate that the load-carrying capacity of members filled with PUR foam is two to three times higher than the load-carrying capacity of bare CFS members in terms of flexural and shear resistance, respectively. The authors suggest that for the design of PUR foam infill CFS members, the design code for hot-rolled steel members should be used since no premature localised failure such as local or distortional buckling was observed during the experiments. Analytical predictions show that the hot-rolled steel guidelines provide a conservative estimation of the load-carrying capacity of infilled CFS members.

The effects of foam stiffening on the flexural resistance of CFS lipped C-sections filled with PUR foam is investigated in [74]. The foam is of various densities and the beam specimens have different width-to-thickness ratios. The performance of the infilled C-sections is compared to that of bare CFS C-sections. All the specimens show the same type of failure, which is local buckling of the compressed flange. However, the infilled profiles reach the ultimate load at higher levels of stress in the plate elements of the cross-section. Analytical predictions that take into account the stiffening effect of the foam by introducing a buckling coefficient are compared to the experimental results. The predictions are accurate up to a width-to-thickness ratio of about 150. Beyond 150, the predictions become nonconservative as the yielding stress and the buckling stress of the plates approach each other, making the elastic buckling stress less accurate.

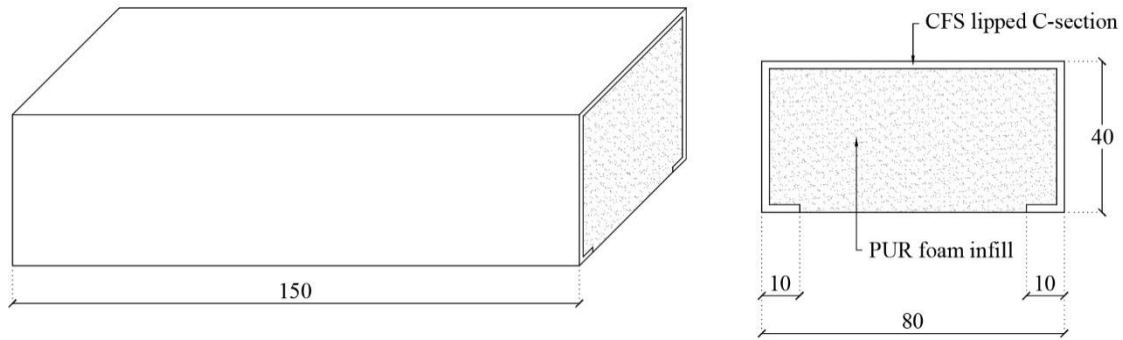


Figure 2.9. Dimensions of the infilled C-section [74]. All units in [mm].

Methods to improve the local and distortional buckling load of CFS (cold-formed steel) members is discussed in [76]. The slenderness of the plate, which affects the buckling loads, can be improved by using stiffeners or corrugations. However, these methods primarily improve local buckling rather than distortional buckling. In this research, PUR foam is suggested as a strategy to enhance the performance of lipped C-sections. Both numerical simulations and experimental testing are carried out to assess the axial load-carrying capacity of the infilled profiles. The numerical simulations showed an improvement of approximately 50% in terms of ultimate load compared to bare CFS profiles. However, the axial compression test showed a more limited improvement of approximately 25%. The authors suggest that the reason for not achieving the same level of ultimate load in the test is due to the poor adhesion between the PUR foam and the plates of the CFS cross-section (see Figure 2.10).



Figure 2.10. Cut-out piece of the PUR foam infilled CFS profile with delamination between the foam and the steel [76].

2.2.3. CFS built-up members

CFS profiles are also combined with each other to improve the resistance of common open C and Z-sections to lateral torsional buckling [77]. Common solutions involve joining together back-to-back CFS-

lipped C-sections to obtain a doubly symmetric open cross-section or assembling one CFS-lipped C-section with a plain C-section to obtain a box beam [78]. Self-drilling screws are used to connect the CFS members. Nevertheless, CFS design codes [79], [80] do not provide methods for the design of built-up CFS cross-sections, namely the degree to which different members work as a monolithic element [81].

Several researchers have proposed different design methods. According to [82], the flexural capacity of a built-up box section subjected to concentric loading may be obtained by application of the superposition method. The sum of the capacity of the individual members yields the strength of the built-up member. A numerical model was created to replicate the results of flexural tests on a box beam composed of a CFS-lipped C-section and a track section fastened by self-drilling screws. The numerical model was validated and used to conduct a parametric study, investigating the impact of various factors such as steel yield strength, width-to-thickness ratio, screw spacing, and location of load application. The study concluded that the superposition method [79] is only reliable for concentric loading, and design guidelines are more accurate for height-to-thickness ratios greater than approximately 220 and screw spacings less than 150 mm.

Different analytical methods for predicting the flexural resistance of regular and lipped CFS C-sections and built-up box beams composed of the latter profiles are compared against experimental and numerical results in [83]. The EWM described in [79] accurately predicts the flexural strength of lipped C-sections but not plain C-sections. The superposition method for predicting the flexural strength of built-up box beams leads to slight overestimations and requires a correction factor of 0.9. The Chinese Specification for thin-walled steel buildings (2011) [84] proposes an equivalent box beam method which considers the built-up section as an equivalent thin-walled rectangular hollow section. The overlapping flanges are converted into a single stiffened element through the equivalent built-up box beam, with the thickness of a single flange. This assumption is deemed reliable for built-up box sections with a width-to-thickness (w/t) ratio not greater than 100 (see Figure 2.11).

In reference to predicting deflection at the serviceability limit state (SLS), [66] has compared the results of flexural tests on back-to-back built-up C-sections with the analytical predictions based on the EWM as described in [80]. Different methods have been proposed, taking varying assumptions into account: i) variation of the modulus of elasticity along the beam; ii) variation of the effective properties of the beam based on the stress level along the beam; iii) variation of the previous two along the beam. The comparison with experimental results shows that Young's modulus has a negligible influence on deflection prediction. However, the effective properties of the cross-section have a significant impact on analytical

estimations. Assuming that the gross-cross-section properties remain constant throughout the beam's length leads to an underestimation of deflection by approximately 8%. To account for the variation of effective properties along the beam, the interpolation method described in [80] is a simplified yet reliable method. It slightly overestimates the vertical deflection by about 4%.

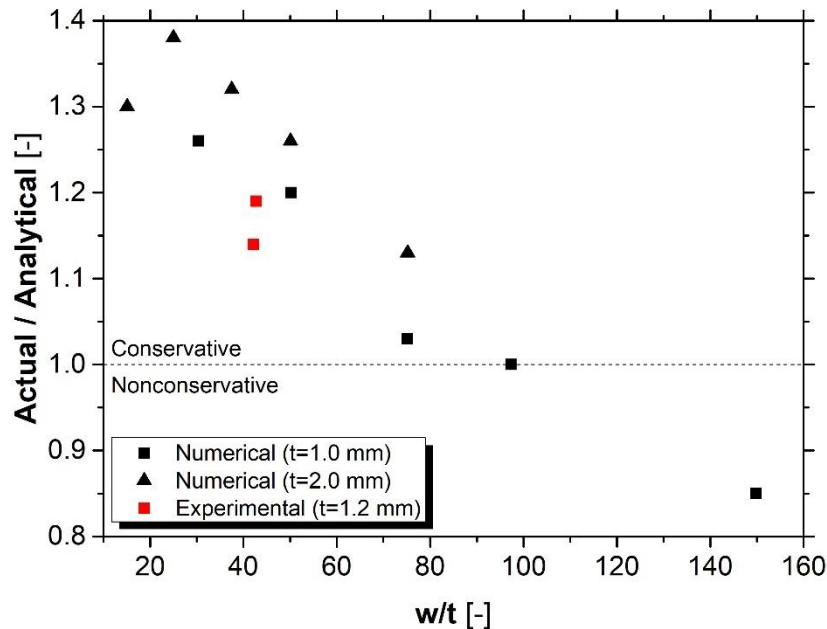


Figure 2.11. Comparison of actual and predicted strength of built-up box beam [83].

Based on the literature review of lightweight structural systems based on CFS profiles the following conclusions may be drawn: i) PUR foams with densities not greater than 40 kg/m^3 are commonly used as an infill material to improve the resistance of single CFS profiles and built-up CFS cross-section; ii) analytical models for considering the mechanical contribution of the PUR foam on the overall behaviour of the CFS infilled profiles lack experimental and numerical validation; iii) building codes design guidelines appeared to be nonconservative for built-up box beam and even for single CFS profiles; iv) assembly processes of CFS profiles-based structural system are not yet automated; v) connection methods mostly involve self-drilling screws and laser welding; vi) adhesive connections between CFS elements have not yet been explored.

2.3. OPTIMIZATION SEARCH METHODS

The mathematical definition of an optimisation problem for the objective function $f(\mathbf{x})$ is the following: given a variable vector $\mathbf{x}$ in the solution space $\mathbf{X}$, find the vector $\mathbf{x}^*$ that minimises (or maximises) the objective function so that $\min f(\mathbf{x}) = f(\mathbf{x}^*)$. Furthermore, the solution space $\mathbf{X}$ is generally restricted

by one or multiple constraints. The constraints may be expressed as inequalities in the form of $g_i(\mathbf{x}) \leq 0$ with $i = 1, \dots, m$ and/or as equalities such as $h_j(\mathbf{x}) = 0$ with $j = m, \dots, p$ [85]. In civil engineering, the challenge often involves finding the optimal combination of design variables from all feasible designs for a structural element. This solution should minimize construction costs while also meeting strength and serviceability requirements [86].

Traditional search methods for optimisation problems, such as calculus-based, enumerative, and random, lack robustness [87]. When searching for the optimal solution, calculus-based methods explore the surrounding area of the initial search point. However, they rely on continuity and derivative existence in equations, which are not always present in real-world problems with a vast multimodal (multi-peaks) search space full of discontinuities. On the other hand, enumerative methods involve assessing the objective function value at each point in the space, but this approach can be inefficient for extensive search spaces. Metaheuristic search methods have received increasing popularity among researchers in the solution of optimisation problems [12]. Such procedures sample a subset of the solution space otherwise too large to be fully investigated. Random choices are used by these procedures to guide a highly exploitative search of the solution space [87]. Genetic algorithms are an example of such a search procedure. When compared to other search methods, the main advantages of these methods can be summarised as follows: (i) they can handle various types of variables such as continuous, discrete, integer, and/or categorical; (ii) they do not require any extra information apart from the value of the objective function itself; and (iii) being a population-based approach, they are less likely to be trapped in local optima [88]. In the next section, an overview of genetic algorithms is presented, as well as a description of the main genetic operators.

2.3.1. Genetic algorithms

A genetic algorithm (GA) is an optimisation and search procedure based on the Darwinian principle of the survival of the fittest. An initial population P_1 of N individuals (chromosomes), which represents the potential solutions to the optimisation problem, is randomly initialised. The individuals are evaluated using a predefined fitness (objective) function, and the best solutions are given higher chances to reproduce, whereas the weakest may not reproduce at all. New individuals (offspring) are produced by recombination (crossover) of the parent's chromosomes. Sometimes random new features may be introduced through mutation in the offspring population Q_t . The solutions in Q_t are evaluated and, according to different selection procedure, they are merged with solutions from P_t to form the population P_{t+1} . Improvement

in the population results from the repeated selection of the best-performing individuals and the consequent elimination of low-performers [89]. A generic GA procedure is illustrated in Figure 2.12.

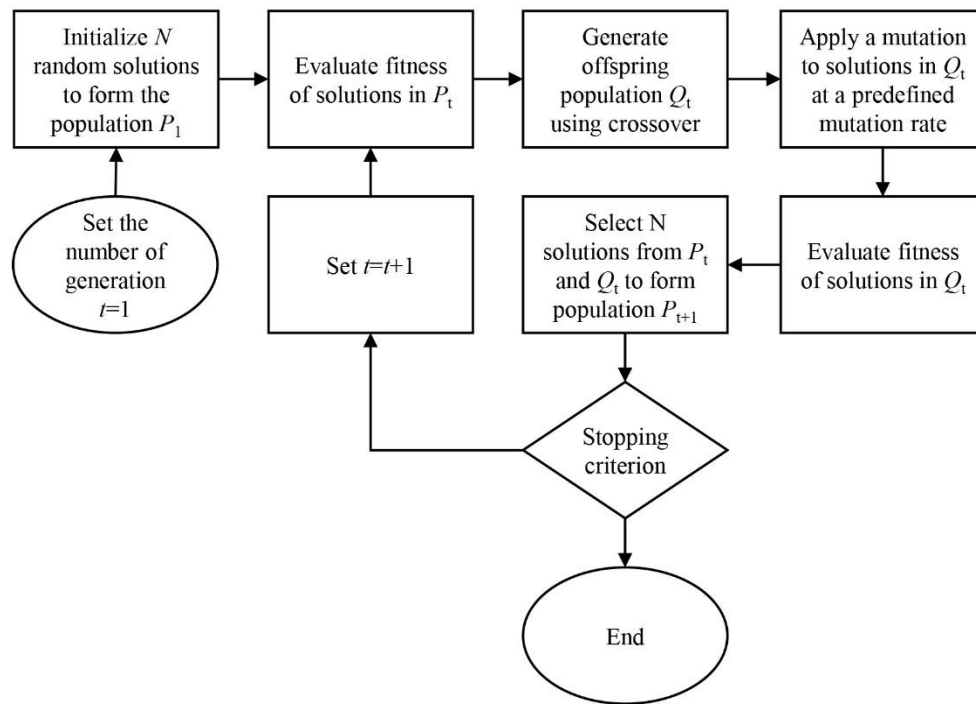


Figure 2.12. The procedure of a generic GA.

Although all GAs follow the steps shown in Figure 2.12, various techniques have been employed to customise their application for specific purposes. When dealing with multi-objective optimisation problems, GAs can be categorised into two primary groups. In the first category, the objective functions are merged into a single composite function. This is typically done by assigning weights to each objective and creating a linear combination that incorporates all of them. Another approach consists in retrieving a Pareto solution set. The Pareto solution set includes solutions that are not dominated by any other solutions, meaning that no other solutions are superior in all attributes [90]. Implementing the former approach is advantageous as it is a simple extension of the single objective GA. On the other hand, Pareto-based approaches offer a set of solutions equally distributed in the performance space, which allows a possible decision-maker to perform trade-off studies and select the most suitable solution.

Another issue that has been addressed concerns the transformation of the objective function, a measurement of the performance of the solution, into the probability of reproduction of that solution [91]. The direct use of a solution's performance level to determine its mating probability can result in premature convergence at the start of the run, as it may favour “super individuals” [92]. For the same reason, the procedure is predicted to slow down at the end of the search when the difference between the individuals’

performance is negligible as the algorithm approaches convergence. To overcome these issues, two types of fitness assignment have been developed: i) scaling, which consists in calculating the fitness by means of a non-linear function of the objective functions; and ii) ranking which orders the population according to the objective function and then assigns a fitness value based on the absolute position of the individual in the population [89].

Finally, it is necessary to devise a way to incorporate constraints in GAs, as they are unconstrained search methods [29]. One of the most common strategies is the introduction of a penalty function [93]. The penalty function involves adding a specific value to the objective function, depending on the number of constraint violations in a given solution. Penalty functions may be divided into three main categories: i) static; ii) dynamic; and iii) adaptive. Static penalty functions add a constant penalty to infeasible solutions, i.e. solutions that do not satisfy the constraint of the optimisation problem. Dynamic penalty functions generally increase the severity of the penalty with an increasing number of generations in order to force the final solution to be feasible. Finally, adaptive penalty functions will increase the severity of the penalty based on the ongoing success of the search for the global optimum. The increase in the penalty depends on the feasibility of the best solutions found so far in the evolutionary process. The choice of the penalty functions is particularly important since the global optimum often lies at the boundary of the feasible solution space [94].

2.3.2. Applications in structural design

Over the last few decades, numerous studies have been conducted on optimization techniques [63]. However, the utilization of such methods has been limited in current engineering practices, often relying on trial-and-error procedures [26]. This may be due to various factors, as highlighted in [64], which recommends that researchers collaborate with engineering firms to develop practical design optimization software. Additionally, much of the literature on optimization algorithms focuses on their mathematical aspects and their success in solving rather trivial examples [95]. Therefore, the following paragraphs will focus on studies related to the development of civil engineering problem-oriented optimisation methods, to support the creation of a benchmark library of existing and new heuristic search procedures, as suggested in [96]. The creation of such a repository may contribute to fostering the use of optimisation methods among practitioners.

In [30] the objective is to find the most economical design for a sandwich panel consisting of steel face sheets and a PUR foam core. The optimal sandwich panel should also be able to cover a wide range of

span lengths. In order to meet the conflicting demands of the market, a distributed parallel evolutionary algorithm is employed. The cross-section dimensions, shear modulus of the core and span length constitute the design variables of the sandwich panels. The span length is simultaneously defined as the variable and the objective function along with the cost. The structural model considered is a two-span beam subjected to uniformly distributed load and thermal actions. A static penalty function account for the constraints imposed by the structural requirements at the ultimate (ULS) and serviceability limit states (SLS). A significant increase in the cost of the sandwich panel is observed for a span length greater than 3.5 m (see Figure 2.13a). The plot of the penalty function against the span length shows that panels above 5.9 m fail to comply with the structural safety requirements (see Figure 2.13b).

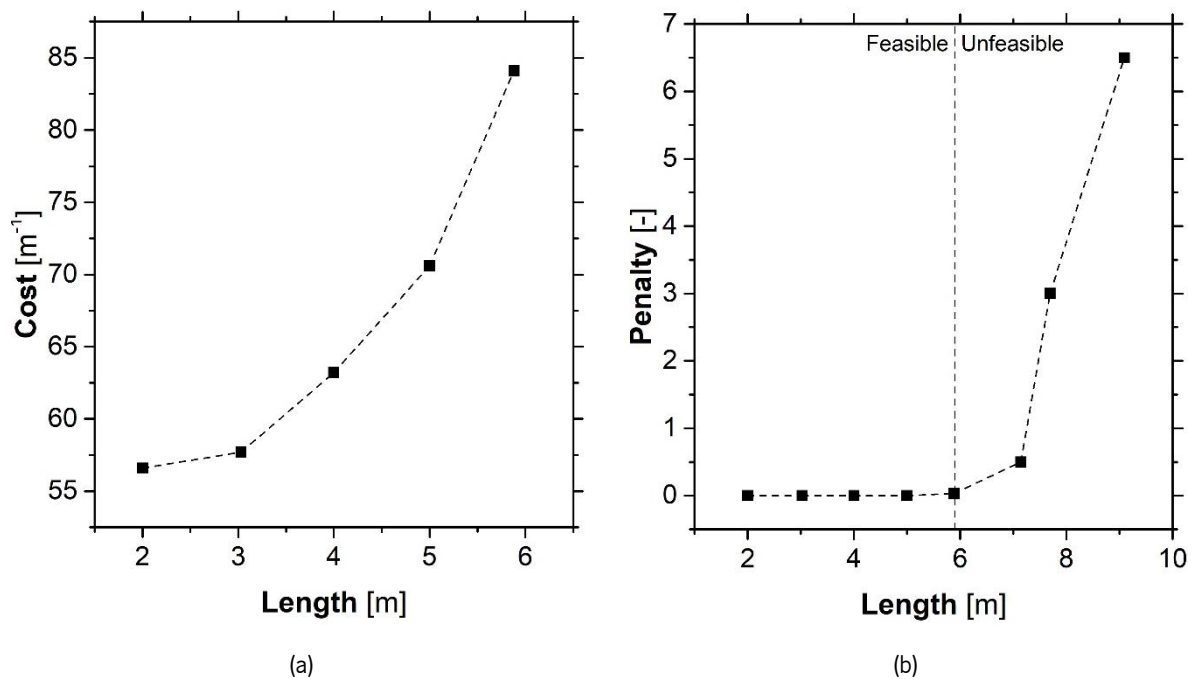


Figure 2.13. Optimal solutions for different span lengths: (a) cost of the solutions; (b) plot of the penalty function [30].

The target of the study in [86] is to minimise the cost of a prestressed concrete slab of dimensions 5 m $\times$ 5 m simply supported at the corners by columns. The study shows that the direct search method cannot retrieve optimal solutions due to the presence of multiple optima, the complexity and nonlinearity of the constraint functions, and the discreteness of some of the variables. Thus, a genetic algorithm optimisation procedure is employed for the solution of the same problem. The constraint imposed by the Canadian Code (2004) [97] at the ULS and SLS are incorporated in the GA by means of a static penalty function. The amount added to the objective function is linearly proportional to the level of constraint violation. Through subsequent trials and improvement of the original algorithm, a final optimisation method is proposed. The GA simultaneously minimise the cost function and the penalty function generating a Pareto front solution set. Furthermore, it is recommended to take an additional step to repair

one of the unfeasible solutions close to the boundary of the feasible region. This is accomplished through a two-step process, where the location of the steel reinforcement is first optimised, followed by the optimisation of the geometrical characteristics of the cross-section.

A multi-objective optimisation algorithm is presented in [98] in order to minimise both the initial cost and the life cycle cost of large steel structures. The design variables of the problem are the cross-section dimensions of beams and columns for multi-storey steel frame structures. The initial cost is proportional to the amount of building materials included in the initial design. On the other hand, the life cycle cost depends on the repair cost established by inter storey drift ratio obtained by means of pushover analysis. Due to the expensive computation cost of the pushover analysis, a radial basis function (RBF) neural network is trained based on the accurate natural frequencies of specific designs. The RBF network then returns the value of the objective function and the amount of constraint violation. The constraints are divided into two groups. The first group of constraints, i.e. the beam-to-column strength, column cross-section dimensions, and gravity loads verification, is enforced by modifying the initial unfeasible solution. The second group, i.e. the inter-storey drift ratio and member force limit, is enforced by means of tournament selection in which feasible solutions are preferred over infeasible solutions. A Pareto front is obtained by running the algorithms five times for 500 generations employing an initial population of 100 individuals. The selected solutions are the extreme of the Pareto front, i.e. the steel frame with the minimum initial cost and the minimum life cycle cost.

The minimisation of weight, cost, and environmental footprint of sandwich panels to be used in flooring applications by means of GA is studied in [13]. Four different architectures are considered for the sandwich panel. All the designs include a PUR foam core and a fibre-reinforced mortar top layer. The bottom laminate stack architecture may comprise GFRP or a combination of GFRP and carbon-fibre reinforced polymer (HYBRID). The presence (R) or absence (U) of longitudinal webs is also considered. The design variables include the overall geometric characteristics of the panel as well as the stacking sequence of the composite laminates. A wide range of constraints is also considered ranging from structural safety to thermal and acoustic requirements. Optimal solutions are retrieved for the four different architectures using a starting population of 100 designs. The GFRP design without web reinforcement was found to be the most economical and environmentally friendly, however it did not fulfil the constraint regarding the total thickness of the sandwich panel. On the other hand, the design with the hybrid GFRP-CFRP bottom face sheet and no longitudinal web present the best fitness while simultaneously complying with all the requirements (see Figure 2.14).

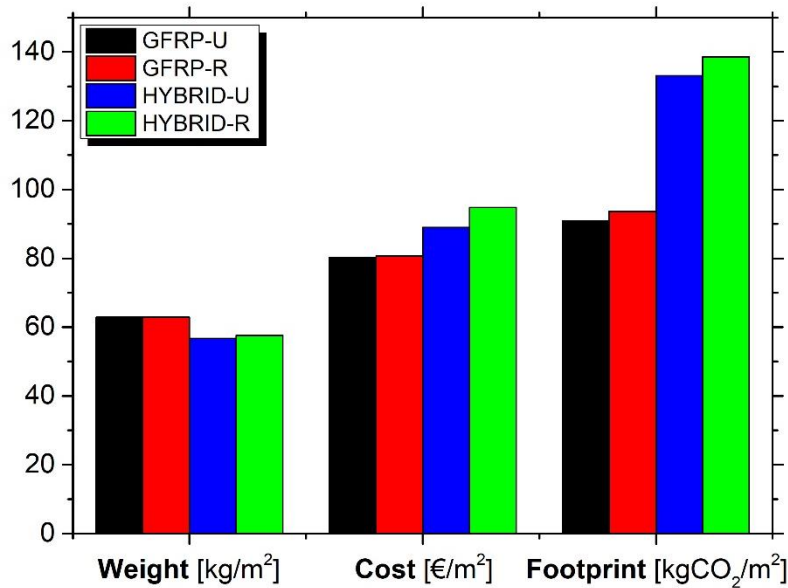


Figure 2.14. Objective function values for each sandwich panel architecture [13].

A comprehensive study on the relationship between design variables, objective functions, and requirements of sandwich floor panels is presented in [12]. This study is extensive and covers a wide range of building requirements, as well as desired mechanical and physical properties for the sandwich panel. The generic cross-section of the studied sandwich panel is made of GFRP face sheets and webs and a PUR foam core. The procedure employed is the Direct MultiSearch (DMS) multi-objective optimisation which is inspired by direct-search calculus-based method [99]. However, the DMS method incorporates the Pareto concept of non-dominated solutions from which local searches for improved solutions are carried out. Relationships between cost and weight, thermal insulation, acoustic insulation, and axial longitudinal and transverse stiffness are established through the projection of the Pareto front on the corresponding planes. Furthermore, recurrent values of the design variables, such as the thickness and composition of the face sheets laminates, the core density, the panel width, and the number of webs, are identified by plotting their frequency in the non-dominated set of solutions. Finally, the best-performing solutions for each objective function are presented, and their architecture and performance are analysed.

Based on the literature review presented in this chapter, some conclusions may be drawn about the current state of implementing optimisation methods that include the requirements of structural design practice. The most common way of including constraints is through static penalty functions. However, they may not be sufficient to guarantee the feasibility of the optimal designs. Furthermore, only a few examples so far have included the full range of requirements imposed on residential building elements. The set of Pareto front solutions can be challenging to analyse due to its vastness and complexity. As a result, the optimal designs are often presented based on the extremes of the Pareto front, which are the

best designs for each objective function included in the optimisation problem. As mentioned in [99], once the Pareto front is obtained, additional data processing is necessary to identify the trade-off design points within the non-dominated Pareto optimal points.

2.4. CONCLUDING REMARKS

The literature review indicates that efforts have been made to develop GFRP face sheet sandwich panels with polymeric foam cores for primary structural applications in civil engineering. Reinforcing longitudinal elements in the core are necessary due to the significant permanent load imposed on civil engineering structures. Additionally, the manufacturing process plays a vital role in determining the panel's failure mode, with the bond between the face sheets and core being a critical aspect. On the other hand, steel face sheet sandwich panels with PUR foam core have a highly industrialised manufacturing process that ensures high-quality production. However, research efforts have focused on secondary structural applications such as roofing systems. Moreover, despite the widespread use of CFS structural members, analytical tools for built-up cross-section and infilled members have shown nonconservative predictions in terms of strength and stiffness.

Using optimisation methods to assist in designing civil engineering structures has also been addressed in the literature. Research has demonstrated that GAs are the most common and efficient optimisation techniques. Nevertheless, it is important to approach their practical use in a way that is accessible to practitioners. Additionally, the incorporation of building code requirements remains an unresolved issue, as the feasibility of the optimised design depends on this.

CFS structural elements filled with PUR foam show potential to be employed as lightweight floor systems. GA optimisation methods also have proven to be reliable tools for optimizing civil engineering structures. However, more investigation is needed to ensure that such optimized structural systems fulfil the design requirements of residential building floors.

CHAPTER 3

3. DESIGN, OPTIMISATION, AND MANUFACTURING

In the preliminary research stage, requirements for building floor structural elements were identified. Materials were selected for the sandwich floor panel architecture based on literature review information and target application requirements. Furthermore, the material selection was also determined based on the requirements of the manufacturing process described in the subsequent paragraphs. The driving factors for developing the floor system were cost-competitiveness, reduced weight, and low environmental impact. Different panel architectures were considered in view of the optimisation of the above-mentioned targets.

The development of the lightweight floor system falls within the scope of the R&D Project Lightslab - *Development of innovative solutions of slab sandwich panels*. The Department of Civil Engineering (DEC) and the Department of Polymer Engineering (DEP) of the University of Minho, along with the private company Ferpainel, developed the R&D proposal aimed at creating a floor system based on sandwich panels. This system is ideal for weight-sensitive applications such as rehabilitating degraded floors in existing buildings and modular constructions. The sandwich panel design had to meet the constraints outlined in the research project. Ferpainel is a company that specializes in producing steel face sheet sandwich panels for roofing and cladding systems, as well as cold store construction. It's worth noting that the current production process is only for non-structural panels. The Lightslab project aims to produce a panel continuously, similar to the current production lines at Ferpainel.

The manufacturing process of steel insulating sandwich panels consists of the following stages (see Figure 3.1): i) CFS sheets are unrolled from the coil and fed into the conveyor belt of the continuous production line; ii) the CFS face sheets pass through roll former to obtain the required profiling; iii) the liquid monomer of the PUR foam is injected on the bottom face sheet of the sandwich panel; iv) the top face sheet is

pressed on top of the expanding PUR foam; v) the sandwich panel is heated up to 40 °C to speed up the curing of the PUR foam; vi) the continuous panel is cut to the required lengths by a flying saw. Based on the current manufacturing process of Ferpainel, in the R&D proposal, it was established that all the different panel architectures had to include metal face sheets. This is due to the fact that changing the face sheet material would imply modifications to the production line of Ferpainel not achievable during the timeline of the project.

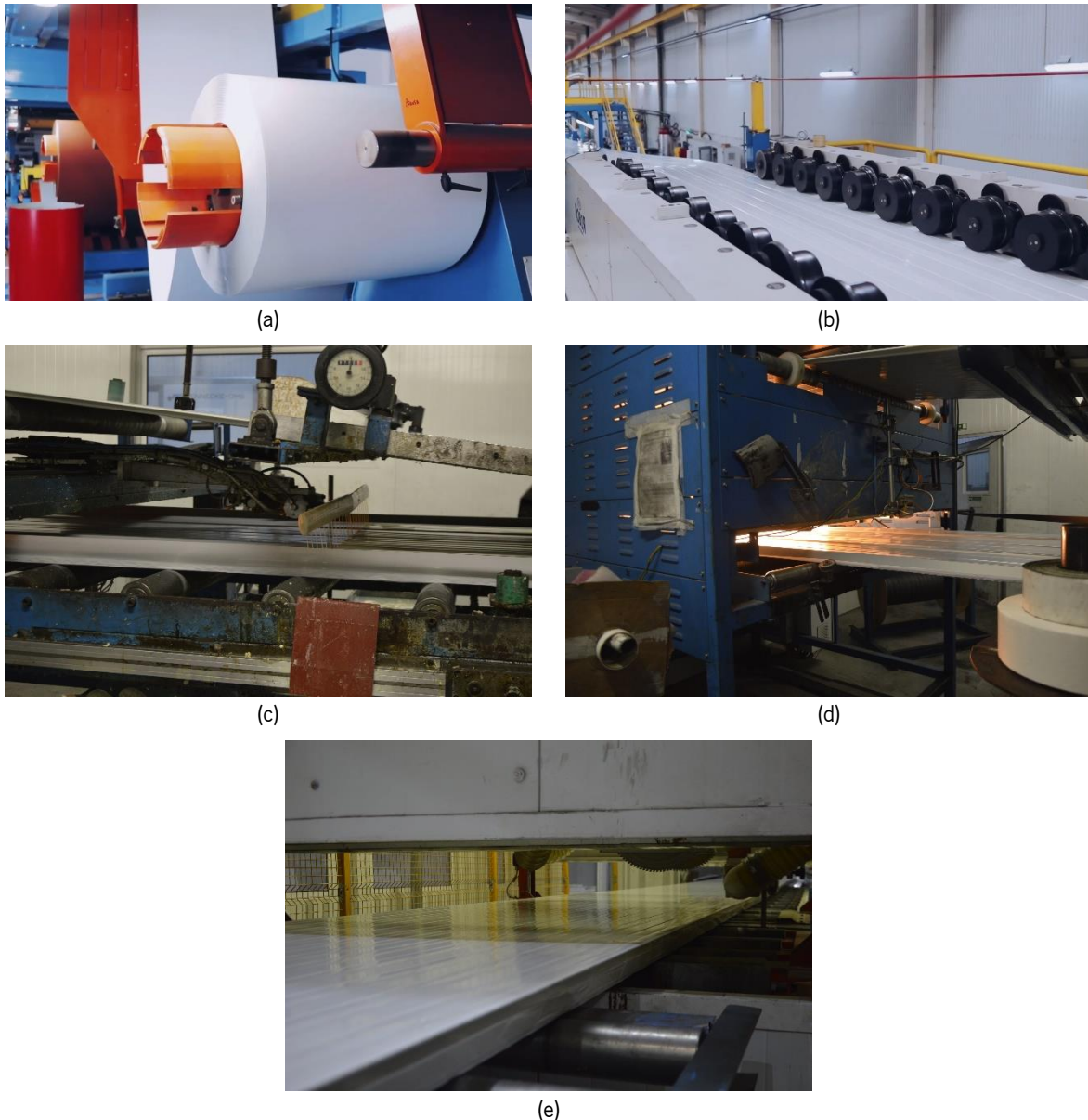


Figure 3.1. Manufacturing process of steel insulating sandwich panels: (a) uncoiling CFS sheet; (b) roll forming CFS sheet; (c) PUR foam's monomers injection; (d) PUR foam curing in heating chamber; (e) cutting of the sandwich panel.

As discussed in the literature review in Chapter 2, sandwich panels with low-density cores do not currently meet the necessary requirements for primary structural applications in civil engineering. Thus, the sandwich panel developed in this research implements a web-core system to enhance shear stiffness

while maintaining a reduced weight. For that purpose, core reinforcing elements made of steel and GFRP, and core materials made of PUR foam and balsa wood are the chosen options based on information gathered in the literature review. Low-density PUR foams have low thermal conductivity, which can lead to significant weight reduction. On the other hand, balsa wood has higher mechanical properties that can provide improved shear stiffness, reducing the number of webs (core reinforcing elements) necessary to achieve the required structural performance.

A schematic of various panel architectures and their constituent materials which are considered as possible modules of the sandwich panel floor system is illustrated in Figure 3.2. Detailed information about the dimensions of the cross-sections and the characteristics of the sandwich panels obtained from the trial-and-error optimization procedure are described in Section 3.5. Solution SP1 uses CFS face sheets, PUR foam core, and longitudinal webs in the shape of C- and Z-section profiles. SP2 involves CFS face sheets, GFRP webs, and PUR foam core. Finally, SP3 is made up of a balsa wood core enclosed by CFS face sheets and C-section outer webs.

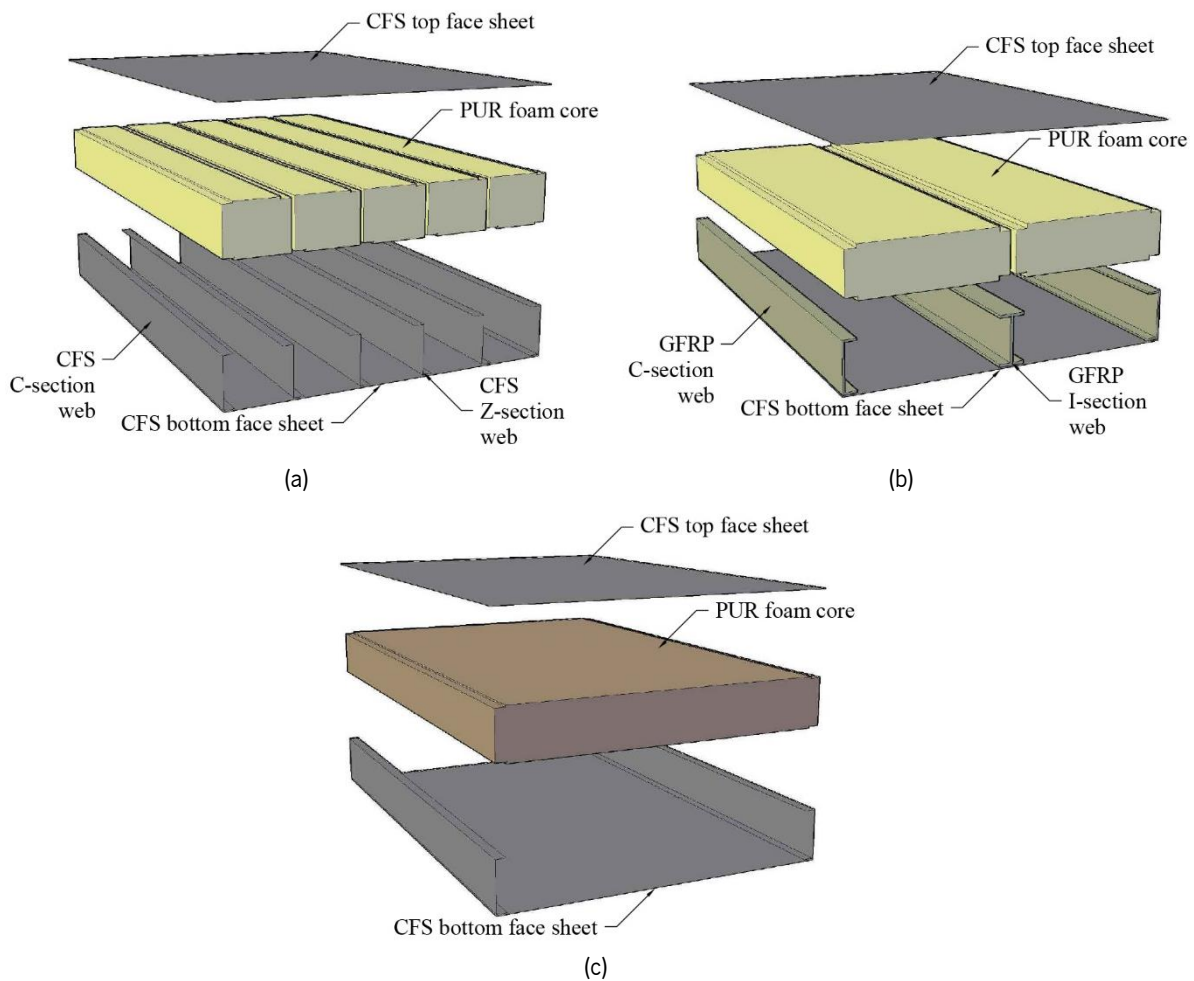


Figure 3.2. Layout of the sandwich panels: (a) SP1; (b) SP2; (c) SP3.

3.1. STRUCTURAL AND TECHNICAL REQUIREMENTS

The sandwich floor panels shall meet the standards for building floors, which include structural, thermal, acoustic, and fire performance requirements. However, this thesis only focuses on the structural design of the sandwich panel and does not cover its thermal and acoustic behaviour, as well as fire performance. These aspects are important for the overall performance of the sandwich panels and should be addressed in future studies. Nonetheless, a preliminary study of the thermal, acoustic, and fire resistance performances was also conducted.

The structural analysis and design process adhered to the standards set by Eurocodes, the relevant transposed Portuguese norm, and the European Recommendations for Sandwich Panels that were applicable at the time of execution of this task of the research project. The following guidelines and standards were taken into consideration during the process:

- Eurocode 0 – Basis of structural design (2002) [100];
- Eurocode 1 – Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings (2002) [101];
- Eurocode 3 – Design of steel structures – Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting (2006) [80];
- Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements (2006) [102];
- “Eurocódigo 3 – Projeto de estruturas de aço – Parte 1-1: Regras gerais e regras para edifícios” (2010) [103];
- CNR – DT 205/2007, Guide for the Design and Construction of Structures made of FRP Pultruded Elements (2007) [104];
- CEN WG 4, Fibre Reinforced Polymer Structures – Scientific and Technical Report (2014) [105].

In the preliminary thermal design of the sandwich panel, the thermal transmittance limit value was taken into consideration as per the Portuguese code for building energy efficiency [106]. Such limits are set to ensure good thermal performance of the building or its autonomous part with regard to energy consumption, technical system efficiency, and condensation reduction. The design was carried out for the most rigorous climatic zone, which has a maximum thermal transmittance limit of $0.30 \text{ W/m}^2\text{K}$.

The primary application of the sandwich floor panel is for the rehabilitation of existing buildings, which does not envisage any specific acoustic requirement. However, to ensure a minimal level of sound insulation for its use in new constructions, it is desirable to consider the acoustic performance. The

Portuguese code on the acoustic requirements for buildings [107] specifies limit values for airborne and impact sound insulation. Lightweight building elements have inherent difficulties in achieving the required sound insulation, as sound insulation is directly proportional to the mass of the building element. Therefore, reduced values of acoustic requirements were considered, which are 60% of those established by the Portuguese standards. For the sandwich panel design, a minimum value of 50 dB was considered for airborne sound insulation ($D_{nT,w}$). The airborne sound insulation value is a measure of the amount of sound pressure reduction offered by the panel separating two rooms. For impact sound insulation ($L_{nT,w}$), a maximum value of 60 dB was considered as the threshold sound pressure level in the room. Impact sound insulation is expressed as the sound pressure level registered in a room when one of the enveloping elements is excited by impact.

The ability of a building component to resist fire is measured in terms of its resistance to collapse or excessive deflection (R), resistance to flame and hot gases penetration while maintaining structural integrity (E), and insulation offered to the unexposed face to prevent ignition of material in contact with it (I). These values are represented by a number which indicates the time duration for which the criteria must be satisfied. The Portuguese fire safety code [108] specifies the values that are applicable based on the risk category of the building. For floors and roofs of residential buildings belonging to the lowest risk category, a minimum resistance of 30 minutes (REI 30) should be provided.

3.2. STRUCTURAL DESIGN

All the sandwich panels were designed according to the requirements of the Eurocode 0 (2002) [100] and Eurocode 1 (2002) [101]. A detailed description of all the design procedures, including design equations and safety factor coefficient, may be found in [109] and [110]. In this section, the overall design approach is presented, and specific details are provided only for the design of the CFS components of the sandwich panel since they are present in all the proposed solutions. Furthermore, the influence on the sandwich panel final cross-section related to challenges faced during the prototype manufacturing process and the connection system between sandwich panels are also discussed.

The adopted structural model is a simply supported beam subjected to a uniformly distributed load (see Figure 3.3). This represents the worst-case scenario for design as it leads to high stresses and mid-span deflections. Furthermore, this assumption fails to consider the impact of the connections between panels and walls, which could prove advantageous in terms of decreasing the vertical deflection. The span length

is 5.0 m, which is considerable in comparison to the average span length found in old masonry buildings across Europe, approximately 4.0 m, according to the literature on sandwich panels for civil engineering applications [111] [36]. The uniformly distributed load (q) considers the self-weight of the sandwich panel, an additional permanent load of 1.5 kN/m² to account for the presence of floor finishing, partition walls and suspended ceiling, and the load due to variable action (imposed load) on residential buildings, which is set equal to 2.0 kN/m² according to Eurocode 1 (2002) [101].

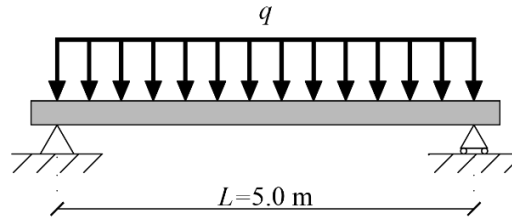


Figure 3.3. Structural model adopted for the design of the sandwich panel.

The self-weight of the non-structural members envisages the possibility of using additional systems such as insulation layers, floating floors, and drop-down ceilings. A thick layer of PUR foam in sandwich panels generally ensures good thermal insulation. Nevertheless, the presence of metal webs inside the core represents thermal bridges which may make necessary the addition of adequate insulation layers. Floating floors and drop-down ceilings will probably be required due to the poor acoustic insulation of sandwich panels. Indeed, acoustic insulation properties rely on the mass of the structural element, which in the case of sandwich panels, is generally reduced. The plenum space between the sandwich panels and the drop-down ceiling may accommodate sprinklers and detectors, which will improve the fire resistance of the sandwich floor panel. Nevertheless, the thermal, acoustic and fire resistance performances of the sandwich floor panel will not be discussed in detail as they are out of the scope of this thesis.

The general design approach at the ultimate limit state (ULS) consists in estimating the design cross-section resistance to bending moment (M_{Rd}) and shear (V_{Rd}). Substituting M_{Rd} and V_{Rd} in the equations of the bending moment and shear diagrams of the simply supported beam and solving for the load carrying capacity of the sandwich panel yields:

$$R_{Md} = \frac{8 \cdot M_{Rd}}{L^2} \quad (3.1)$$

$$R_{Vd} = \frac{2 \cdot V_{Rd}}{L} \quad (3.2)$$

where L is the span length, R_{Md} is the load carrying capacity of the panel with respect to bending moment, and R_{Vd} is the load carrying capacity of the panel with respect to shear. The structural safety

verifications at the ULS are considered satisfied if the load carrying capacity of the panel is equal or greater than the design load calculated according to the fundamental load combination of Eurocode 0 (2002) [100] as stated in Equations (3.3) and (3.4):

$$R_{Md} \geq \gamma_G \cdot G_k + \gamma_Q \cdot Q_k \quad (3.3)$$

$$R_{Vd} \geq \gamma_G \cdot G_k + \gamma_Q \cdot Q_k \quad (3.4)$$

where G_k represent the sum of the self-weight of the sandwich panel and the additional 1.5 kN/m² of permanent loads, Q_k represent the imposed load for residential buildings, γ_G and γ_Q are the partial safety factors for permanent and live loads and they are set equal to 1.35 and 1.5, respectively [100].

For what concerns the serviceability limit state (SLS), the sandwich panel's stiffness is designed to fulfil the requirements imposed on the vertical deflection of generic floors in the national annex of the transposed Portuguese version of the Eurocode 3 Part 1-1 [103]. The total vertical deflection (δ_{max}) is the sum of the deflections due to the permanent actions (δ_1) and due to the variable action for residential buildings (δ_2). The limits on the above-mentioned vertical deflections are given by the following equations:

$$\delta_{max} \leq \frac{L}{250} \quad (3.5)$$

$$\delta_2 \leq \frac{L}{300} \quad (3.6)$$

Regarding the vibrations, the Portuguese National Annex [103] considers the requirements fulfilled if the natural frequency of the element is higher than 3 Hz. The estimation of the natural frequency may be exempted if the total vertical deflection estimated for the characteristic load combination, given in Equation (3.7), is less than 28 mm. Such a requirement is already fulfilled if Equation (3.5) is satisfied, given that $L/500$ for a span length of 5.0 m result in a limit of 20 mm. It should be noted that for the assessment of the SLS, the loads should be combined according to the following equation [100]:

$$G_k + \psi_0 \cdot Q_k \quad (3.7)$$

where ψ_0 is the factor for the characteristic value of the variable action for residential areas set equal to 0.7 [100].

3.2.1. Effective width method

As previously stated, CFS members may be a preferable option compared to the common hot-rolled steel members. A wide range of shapes may be obtained for CFS members' cross-section by cold forming operations, and therefore favourable strength-to-weight ratio can be achieved economically. Furthermore, CFS members are ideal for prefabrication and mass production, they are fast and easy to erect and install, economical to transport and handle, and they are recyclable. All these advantages can provide significant cost saving in construction [112]. Usual shapes for CFS members are C-section, Z-section, hat sections, as well as profiled sheets for roof and wall panels as illustrated in Figure 3.4.

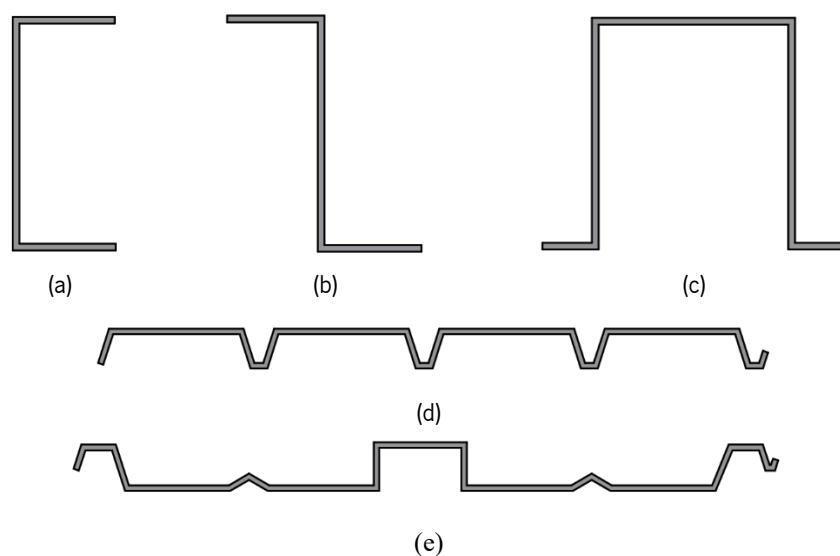


Figure 3.4. Typical CFS cross-sections: (a) C-section; (b) Z-section; (c) hat section; (d) roof panel; (e) curtain wall panel.

The verification process for these profiles is similar to class 4 hot-rolled steel profiles as outlined in Eurocode 3 Part 1-1 (2005) [113]. The effective properties of the profiles are determined using methods described in Eurocode 3 Part 1-3 (2006) [80] and 1-5 (2006) [102]. Eurocode 3 Part 1-3 (2006) [80] specifically addresses the verification of cold-formed steel profiles, while Eurocode 3 Part 1-5 (2006) [102] covers both hot-rolled and cold-formed steel profiles of class 4 and provides guidance on determining their effective properties.

The approach proposed in the Eurocode assumes that each element of the cross-section of a cold-formed steel member (flanges, webs, edge stiffeners) may be considered as a thin flat plate supported on its lateral edges by their contiguous part. These plates may buckle at a stress level generally lower than the yielding stress of the material, or the stress required to trigger global buckling. The calculation of the elastic critical buckling load of a plate represents a first assessment of the behaviour of thin plates. Solving the differential equation of the vertical displacement of a thin plate simply supported on all four sides and

subjected to a uniform compressive state of stress in one direction yields the elastic critical buckling of the plate (σ_{cr}) according to Equation (3.8):

$$\sigma_{cr} = k_{\sigma} \cdot \frac{\pi^2 \cdot E}{12(1 - \nu^2)} \cdot \left(\frac{t}{b}\right)^2 \quad (3.8)$$

where b is the width of the plate, E is the Young's modulus of the plate material, k_{σ} is the plate buckling coefficient depending on the stress ratio and the boundary conditions, t is the thickness of the plate, and ν is the Poisson's ratio of the material. However, plates, unlike columns, may sustain greater loads than the predicted buckling load. This is due to the redistribution of the compressive stresses once the plate buckles as a result of the membrane tensile and shear stresses developed in the direction perpendicular to loading. The method to include the postbuckling reserve in the analytical estimation of the ultimate strength of thin plates was first proposed by von Karmán [114]. The method, known as EWM, assumes that when the plate reaches collapse a non-uniform distribution of stresses arises as illustrated in Figure 3.5a. The stresses are maximum at the edges of the plate and may be approximated to a uniform distribution of stresses (σ_{av}).

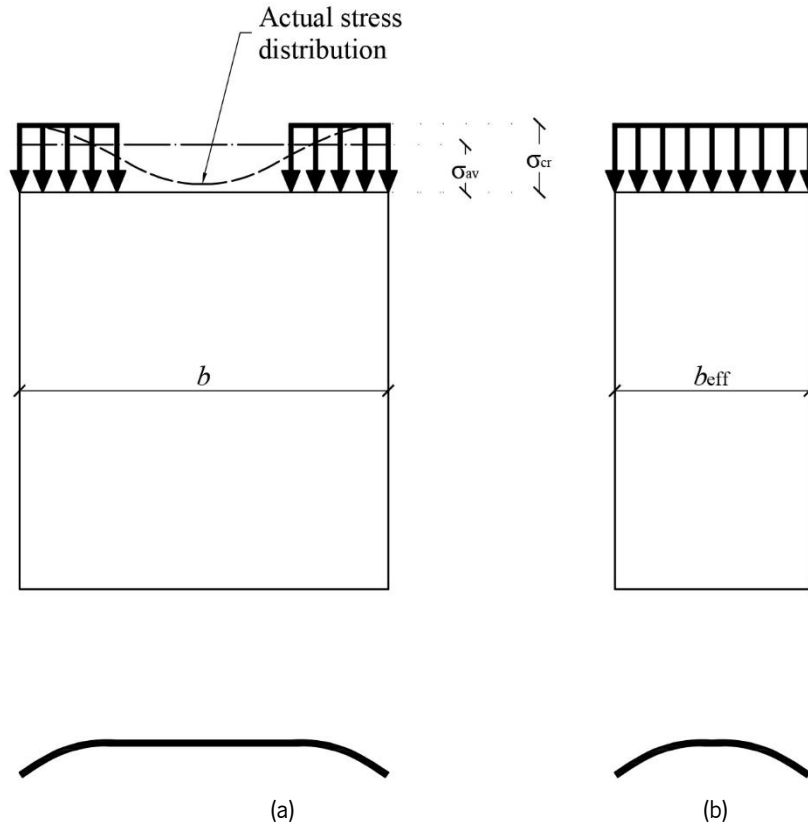


Figure 3.5. Compressive stress distribution and buckled cross section according to the EWM: (a) simply supported plate; (b) Von Karmán plate.

The effective width is the width of an equivalent plate which fails when the compressive stresses reach the yielding strength of the material (σ_y). Thus, the effective width (b_{eff}) may be expressed as:

$$\sigma_{\text{av}} \cdot b = \sigma_y \cdot b_{\text{eff}} \quad (3.9)$$

Imposing the equivalence between the critical load of the plate reaching collapse, given by Equation (3.8), and the critical load of the effective width, and solving for b_{eff} yields:

$$b_{\text{eff}} = b \cdot \sqrt{\frac{\sigma_{\text{cr}}}{\sigma_y}} \quad (3.10)$$

Substituting Equation (3.9) into Equation (3.10) yields the ultimate stress of the plate considering the postbuckling reserve:

$$\sigma_{\text{av}} = \sqrt{\sigma_{\text{cr}} \cdot \sigma_y} \quad (3.11)$$

The Eurocode approach involves the estimation of the slenderness of the plate (λ_p) to establish the effective width of the plate element by combining Equations (3.8) and (3.10):

$$\lambda_p = \frac{b}{b_{\text{eff}}} = \sqrt{\frac{\sigma_y}{\sigma_{\text{cr}}}} = \frac{b}{t} \cdot \sqrt{\frac{12(1 - \nu^2)}{\pi^2 \cdot E \cdot k_\sigma}} \cdot \sigma_y \quad (3.12)$$

A further condition on λ_p is prescribed by the Eurocode to determine the reduction factor of the cross-section due to local instabilities (ρ). The correction coefficient is based on studies of postbuckling strength of plates conducted by Winter [115]. They include the effect of out-of-plane imperfection, and residual stresses on real cold-formed steel members.

When a CFS members is subjected to a compressive state of stress due to axial loading, bending moments or shear, in order to evaluate the ultimate strength, it is necessary to estimate the effective widths of all the plate elements composing the cross-section. The iterative procedure proposed in Eurocode 3 Part 1-5 (2006) [102] to account for the effect of local buckling is illustrated in Figure 3.6.

First, the stress distribution (ψ) in the plate elements of the cross-section is calculated according to the gross cross-section characteristics. The reduction factor for the area of each plate element is determined according to the slenderness of the plate which in turn depends on the buckling coefficient of the plate. The effective characteristics of the cross-section are estimated. The stress distribution is calculated again according to the effective cross-section characteristics until convergence is reached. In the scope of this

thesis, the location of the neutral axis (y_G) was considered the convergence criterion. Convergence was assumed when the difference between the neutral axis location between two consecutive iterations was less than 10^{-3} mm.

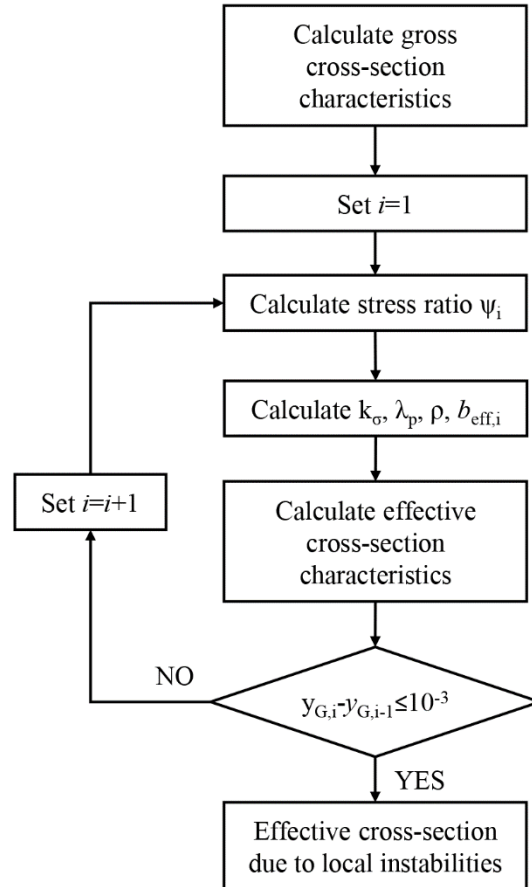


Figure 3.6. Flow chart of the calculation of the effective properties of CFS cross-sections considering the effect of local instabilities.

3.2.2. Effect of distortional instabilities

CFS members are often augmented with stiffeners which further complicate the estimation of the ultimate strength [116]. Stiffener may consist of folds or bends located at the extremities or in the middle of the plate element (see Figure 3.7).

The instability mode associated with the stiffener is defined as distortional buckling. It generally involves the movement of the line junction between the flange and the stiffener without a rigid body rotation or translation of the whole cross-section [16]. This is opposed to local buckling modes where typically out-of-plane deformation of the web or flange elements occur without movement of the line junction between those elements and those elements and the eventual stiffener.

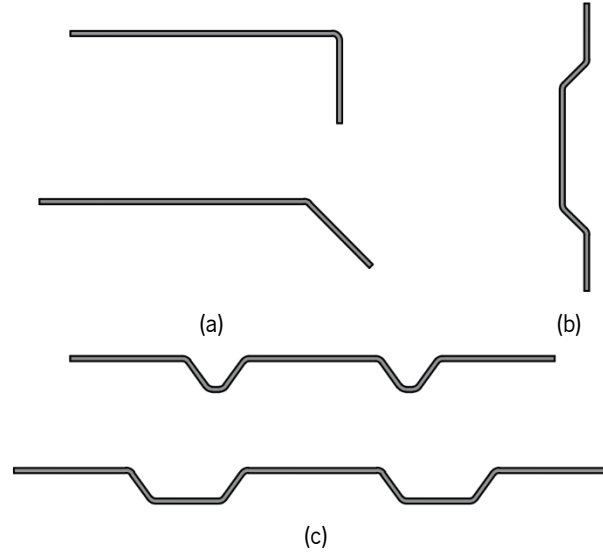


Figure 3.7. Typical forms of stiffeners for CFS members and sheets: (a) edge stiffeners; (b) web intermediate stiffener; (c) flange intermediate stiffeners.

The Eurocode method assumes that the stiffener behaves as a compressed strut on an elastic foundation [117]. The equivalent strut is continuously supported by elastic springs of stiffness (K_s) acting along its centroidal axis (see Figure 3.8). The stiffness of the spring accounts for the bending stiffness of the plate elements contiguous to the stiffener as well as rotation restraint offered by the other elements of the cross-section [118]. The stiffness of an intermediate flange stiffener is given by:

$$K_s = \frac{3(b_1 + b_2)}{b_1^2 \cdot b_2^2} \cdot \frac{E \cdot t^3}{12(1 - \nu^2)} \quad (3.13)$$

where b_1 and b_2 are the effective width of the plate elements adjacent to the stiffener.

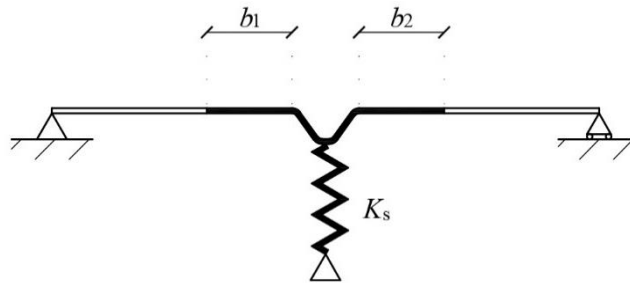


Figure 3.8. Equivalent structural system of an intermediate stiffener in CFS members.

The elastic critical buckling of the stiffener ($\sigma_{cr,s}$) and the slenderness of the stiffener (λ_d) are given by:

$$\sigma_{cr,s} = \frac{2\sqrt{K_s \cdot E \cdot I_s}}{A_s} \quad (3.14)$$

$$\lambda_d = \sqrt{f_y / \sigma_{cr,s}} \quad (3.15)$$

where I_s is the moment of inertia of the stiffener about its centroidal axis, A_s is the area of the stiffener considering the effective width of the adjacent plate elements. As previously established for the local instability, the area reduction factor (χ_d) of the stiffener due to distortional instabilities is determined according to λ_d .

The iterative procedure to account for the effect of distortional buckling proposed in Eurocode 3 Part 1-3 (2006) [80] is illustrated in Figure 3.9. The first step consists in determining the effective width of the plate elements based on the effect of the local buckling considering an infinite rigidity of the elastic spring supporting the stiffeners. Subsequently, $\sigma_{cr,s}$ is estimated, which in turn provides λ_d and the reduction coefficient (χ_d) of the area of the stiffener and its adjacent elements. In the estimation of the effective properties, based on χ_d , a reduced thickness (t_{red}) of the stiffener and the adjacent plate elements shall be considered. The stress distribution calculation is repeated according to the effective cross-section characteristics until convergence is reached. As previously stated, the convergence criterion considered in this work is the location of the neutral axis.

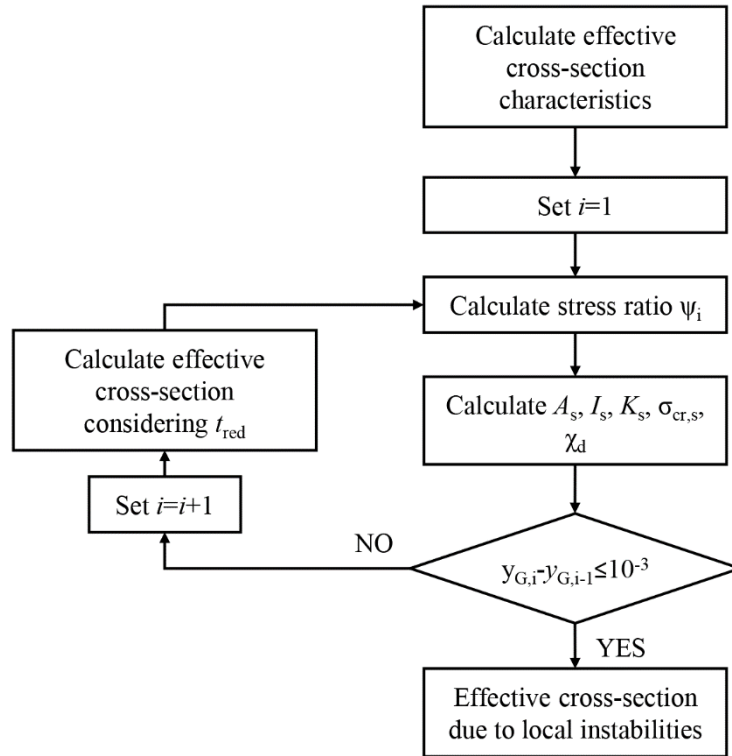


Figure 3.9. Flow chart of the calculation of the effective properties of CFS cross-sections considering the effect of distortional instabilities.

3.2.3. Built-up members

Eurocode 3 procedure covers only single-shape profiles and does not provide any information on the design of built-up members [119]. Furthermore, studies on the structural behaviours of CFS built-up members only involved mechanical connections ([65], [77], [78], [81], [82], [83]). As previously mentioned in Section 3.6, the final prototype manufactured and tested in the scope of this thesis involved an adhesive connection between the CFS components of the built-up cross-section. Therefore, it was necessary to envisage a new method to account for the presence of the adhesive bond and, in general, the design of CFS built-up members. The method developed in this thesis follows the approach proposed by [83] for a built-up box section bending about its weak axis, and it is illustrated in Figure 3.10.

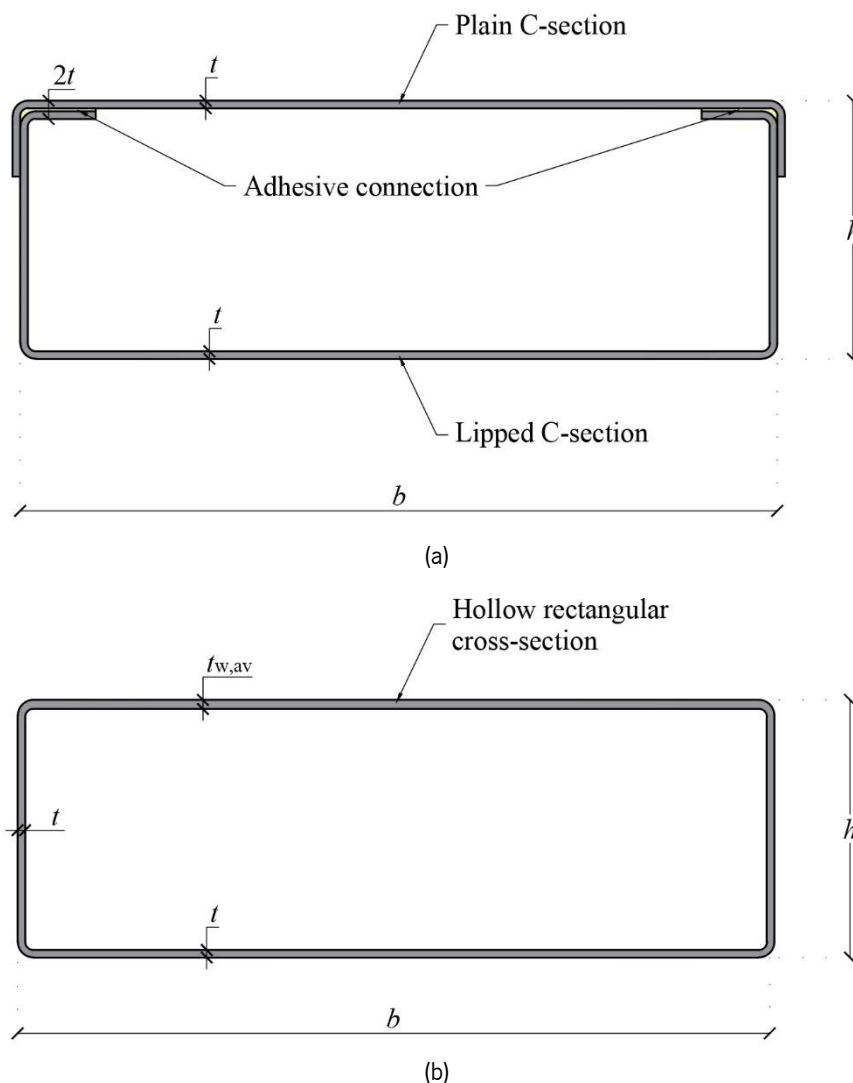


Figure 3.10. Simplified cross-section of the sandwich panel: (a) built-up box section; (b) equivalent hollow rectangular section.

For the sake of simplicity, a simplified built-up box beam cross-section is considered to describe the proposed method. The PUR foam is neglected, and the sandwich panel cross-section may be considered a built-up CFS cross-section composed of a plain C-section and a lipped (with edge stiffeners) C-section. The built-up box section is simplified into an equivalent thin-walled hollow rectangular cross-section. The flanges of the plain C-section are disregarded in the analytical calculation as a conservative assumption. The top plate element thickness of the equivalent cross-section is the weighted average thickness ($t_{w,av}$) of the overlapping and non-overlapping parts of the built-up cross-section. The simplified cross-section may then be treated as a single profile, and its effective characteristics are established according to the EMW described in the Eurocode 3 Part 1-3 (2006) [80] and 1-5 (2006) [102].

3.3. THERMAL DESIGN

The presence of CFS webs within the PUR foam core of the sandwich floor panel may represent an issue in view of its compliance with the thermal requirements of building floors imposed by the Portuguese code [106]. Therefore, a preliminary assessment of the thermal transmittance (U) of the sandwich panel was carried out according to the analytical method described in ISO 6946 (2017) [120]. This estimation considers the presence of a floor insulation layer made of polyisocyanurate foam and a drop-down ceiling insulation made of glass mineral wool bags (see Figure 3.11) and does not consider specific areas such as connections between panels or between panels and structures. It is worth noticing that the presence of the insulation layer and the drop-down ceiling are included in the additional 1.5 kN/m² permanent load.

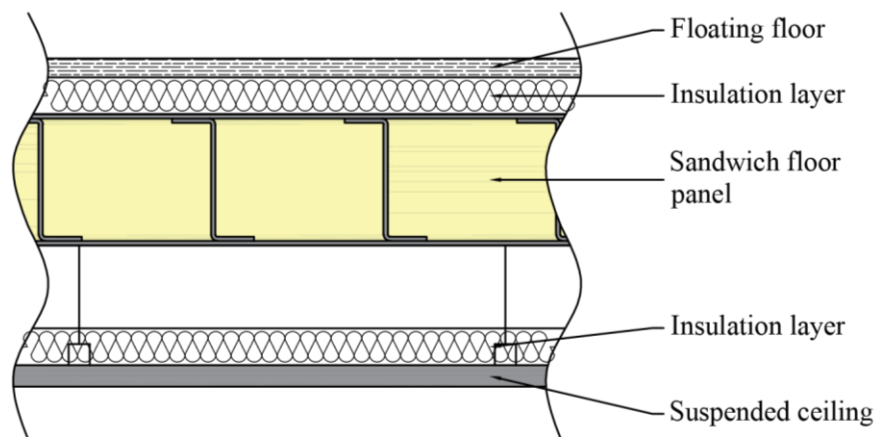


Figure 3.11. Cross-section of the residential building floor considered for the thermal transmittance estimation.

The total thermal resistance of the floor system (R_T) considers heat transfer by conduction through the thickness of the floor elements and convection and radiation on the surfaces of the floor. It is calculated by the following expression:

$$R_T = R_{si} + R_1 + R_2 + \dots + R_i + R_{se} \quad (3.16)$$

where R_{si} and R_{se} are the internal and external surface resistance, and R_i is the thermal resistance of each thermally homogeneous layer of the floor system. U is then given by:

$$U = 1/R_T \quad (3.17)$$

The value given by Equation (3.17) must be corrected to allow for the effects of the metal webs. To the best of the author knowledge, no current code includes analytical formulation to consider the presence of metal webs in the core of sandwich panels for the estimation of their thermal transmittance. Therefore, as a preliminary approximation, the CFS webs are treated as mechanical fasteners penetrating through the PUR foam core and the correction factor to the thermal transmittance (ΔU_f) is given by:

$$\Delta U_f = \alpha \cdot \frac{\lambda_s \cdot A_w \cdot n_w}{h_w} \cdot \left(\frac{R_1}{R_T} \right)^2 \quad (3.18)$$

where α is a coefficient related to the ratio between the thickness of the insulation layer and the height of the webs, λ_s is the conductivity of steel, A_w is the cross-sectional area of the web, R_1 is the thermal resistance of the PUR foam core, and R_T is the thermal resistance of the whole floor system disregarding the presence of the webs. Finally, the corrected thermal transmittance (U_c) accounting for the presence of the CFS webs is determined as follows:

$$U_c = U + \Delta U_f \quad (3.19)$$

3.4. ACOUSTIC DESIGN

Lightweight floor systems generally do not provide a high level of acoustic insulation. The lightness of the system is actually the main reason for the poor acoustic performance [121]. A preliminary assessment of the airborne and impact sound insulation of the sandwich panel was carried out according to the methods described in EN 12354-1(2017) [122] and EN 12354-2 (2017) [123], respectively. The analytical procedures described in these standards apply to heavyweight construction elements behaving like homogeneous plate. Nevertheless, they may be used to roughly assess the acoustic behaviour of the sandwich panel, although significant error may still occur.

Since the cross-section of the sandwich panel is made of two materials, a homogenised cross-section is firstly obtained made only of PUR foam by widening the area of the CFS face sheets and webs of an amount proportional to the ratio of the Young's moduli of steel and PUR foam. The bending stiffness of the homogenised cross-section is then set equal to the bending stiffness of an equivalent cross-section with a unitary width and the thickness of the original cross-section. Solving the equation for the equivalent Young's modulus (E_{eq}) yields:

$$E_{eq} = \frac{12 \cdot I_h \cdot E_c}{I_{eq}} \quad (3.20)$$

where I_h is the moment of inertia of the homogenised cross-section, E_c is the Young's modulus of the PUR foam core, and I_{eq} is the moment of inertia of the equivalent cross-section. The equivalent cross-section may now be used to estimate the laboratory sound reduction index R and the normalized impact sound pressure level L_n according to the methods proposed in the EN ISO 12354-1 (2017) [122] and EN ISO 12354-2 (2017) [123], assuming the sandwich panel behaves like a monolithic element. The critical frequency of the panel (f_c) is given by:

$$f_c = \frac{c_0^2}{1.8 \cdot c_L \cdot t_{tot}} \quad (3.21)$$

where c_0 is the speed of sound in air, c_L is the speed of the longitudinal wave through the equivalent material, and t_{tot} is the total thickness of the sandwich panel. The sound transmission coefficient (τ), which is the ratio between the sound power in the receiving room and the sound power in the source room, may then be estimated according to the following equations:

$$\tau = \left(\frac{2 \cdot \rho_0 \cdot c_0}{2 \cdot \pi \cdot f \cdot m'} \right)^2 \cdot \frac{\pi \cdot f_c \cdot \sigma^2}{2 \cdot f \cdot \eta_{tot}} \text{ if } f \geq f_c \quad (3.22)$$

$$\tau = \left(\frac{2 \cdot \rho_0 \cdot c_0}{2 \cdot \pi \cdot f \cdot m'} \right)^2 \cdot \left(2\sigma_f + \frac{(l_1 + l_2)^2}{l_1^2 + l_2^2} \cdot \sqrt{\frac{f_c}{f}} \cdot \frac{\sigma^2}{\eta_{tot}} \right) \text{ if } f < f_c \quad (3.23)$$

where f is the frequency, m' is the mass per unit area of the floor system, l_1 and l_2 are the size of the laboratory room, η_{tot} is the total loss factor, ρ_0 is the density of air, and σ and σ_f are the radiation factor for free bending waves and for forced transmission, respectively. Based on the sound transmission coefficient, R and L_n are estimated according to the following equations:

$$R = -10 \log(\tau) \quad (3.24)$$

$$L_n = 155 - 30 \log(m') + 10 \log(T_s) + 10 \log(\sigma) + \log\left(\frac{f}{f_{\text{ref}}}\right) \quad (3.25)$$

$$L_n = 43 + 30 \log(f) - R \quad (3.26)$$

where T_s is the structural reverberation time. In EN ISO 12354-2 (2017) two alternative methods are proposed for the estimation of L_n as reported in Equations (3.25) and (3.26). Therefore, the average value obtained from the two expressions is considered. Both the values of R and L_n are plotted over an interval of frequencies ranging from 100 Hz to 3500 Hz. To simplify the design, a single number quantity of the sound insulation values can be estimated according to the methods proposed in the EN ISO 717-1 (2013) [124] and ISO 717-2 (2013) [125]. The method consists in translating a reference curve until the difference with the estimated curve is less than 32 dB (see Figure 3.12).

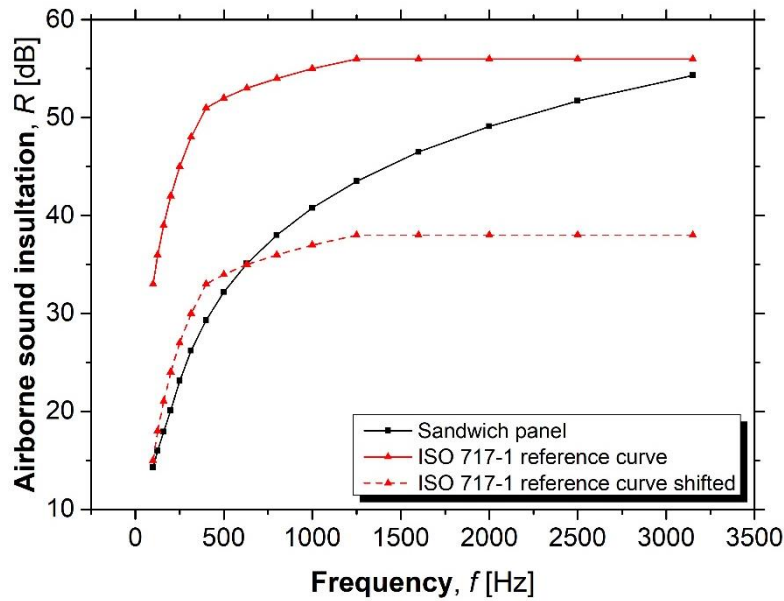


Figure 3.12. Airborne sound insulation (R) of the sandwich panel, the ISO 717-1 (2013) [124] reference curve, and the translated reference curve.

The value of the shifted reference curves at 500 Hz represents the single number quantity sought, namely the weighted sound reduction index (R_w) and the weighted normalized impact sound pressure level ($L_{n,w}$). R_w and $L_{n,w}$ are then converted into the weighted standardized level difference ($D_{nT,w}$) and the weighted standardized impact sound pressure level ($L_{nT,w}$) according to the following Equations:

$$D_{nT,w} = R_w + 10 \log\left(0.32 \cdot \frac{V}{S_s}\right) \quad (3.27)$$

$$L_{nT,w} = L_{n,w} - 10 \log\left(0.32 \cdot \frac{V}{S_s}\right) \quad (3.28)$$

where V is the volume of the laboratory receiving room and S_s is the surface of the sandwich floor panels separating the receiving room from the source room.

3.5. FIRE RESISTANCE ASSESSMENT

Modelling the thermo-mechanical behaviour of sandwich panels in fire is a challenging task because many thermal, physical, and mechanical processes occur simultaneously, and they all depend on various factors, such as temperature, heat flux, and the duration of the fire [126]. Therefore, research studies involve conducting fire tests to quantify the fire resistance of the sandwich panels and to gather the necessary input material thermal properties for developing numerical and analytical models [127], [128].

Although the analysis of the sandwich panel's fire resistance behaviour is out of the scope of this thesis, the results of the fire resistance test (see Figure 3.13) carried out at the laboratory of the Spanish Association for the Promotion of Research and Fire Safety Technology (AFITI) in collaboration with Kolumbarium Consulting SL are presented below. The experimental campaign's outcome will serve as a reference database for future studies on the sandwich panel's fire resistance performance.

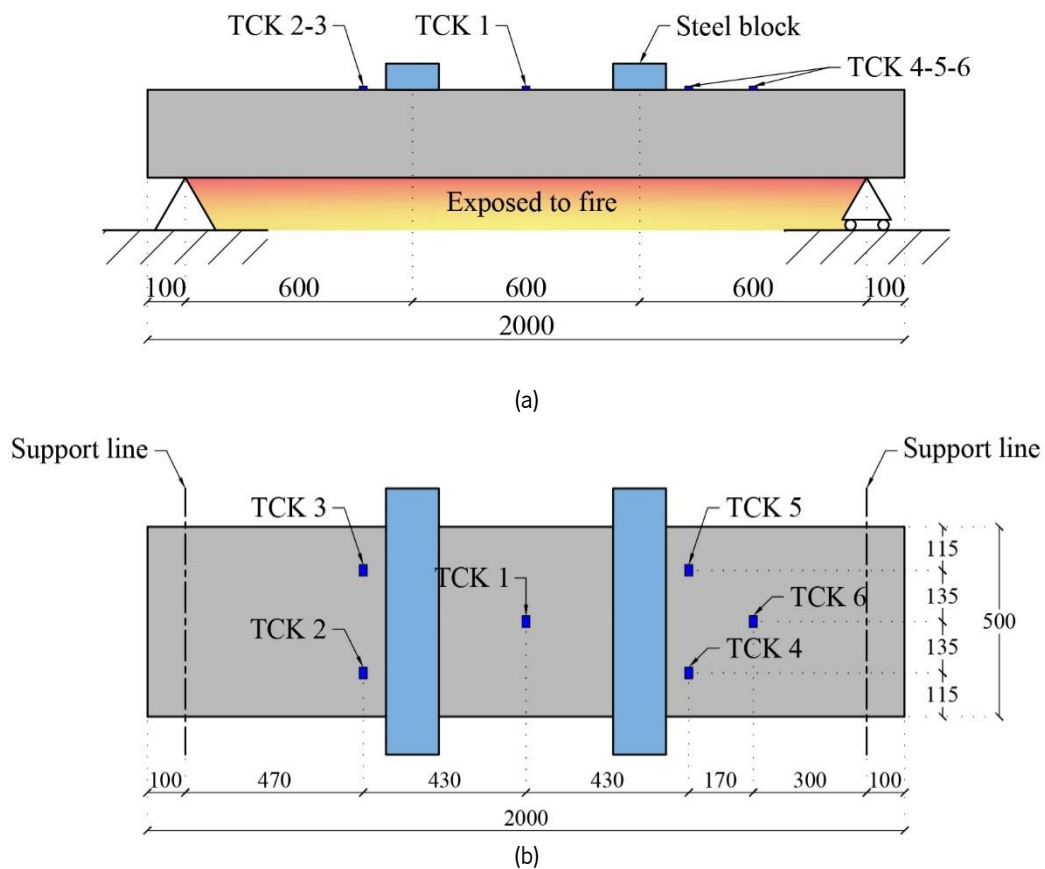


Figure 3.13. Fire resistance test setup and instrumentation: (a) lateral view; (b) top view. All units in [mm].

The specimen analysed in this experimental campaign is the final prototype described in Section 3.8.2 of this chapter. The sandwich panel has a length of 2000 mm, a total thickness of 170 mm, and a width of 500 mm. The experiment was carried out in accordance with EN1363-1 (2020) [129] using a four-point bending loading configuration over a clear span length of 1800 mm and a shear span of 600 mm. To apply a total dead load of 10 kN, steel blocks were placed on the unexposed face sheet, while the panel was subjected to the ISO 834-1 (1999) [130] fire curve. Six type K thermocouples (TCK) monitored the temperature of the unexposed face sheet (see Figure 3.13).

The temperature measured over the area of the top face sheet of the sandwich panels presented similar qualitative development. Therefore, the evolution of the mean temperature over time on the unexposed face sheet is presented in Figure 3.14.

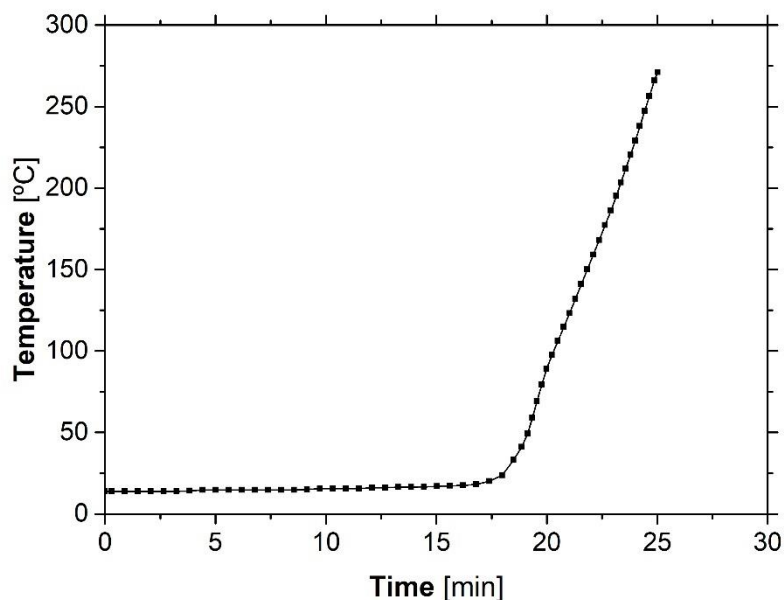


Figure 3.14. Thermal response of the sandwich panel.

The temperatures on the top face sheet of the panel present a very small increase until approximately 18 min, when a steep increase is registered. The higher heating rate is most likely due to the degradation of the upper part of the core.

Two fire safety criteria were assessed based on the test results, namely integrity (E) and insulation (I). However, since there was no measurement available for the vertical deflection, it was not possible to evaluate the load bearing capacity (R). For what concerns the (E) criterion, the sandwich panel ensured its separating function for 21 min when sustained flames could be observed at one of the ends of the specimen. Regarding criterion (I), the sandwich panel registered an increase of more than 140 K after

20 min at the location of Thermocouple 3. Thus, the panel did not comply with the minimum requirements imposed by the Portuguese fire safety code [108].

The panel showed poor resistance to fire, which highlights the importance of installing fire protective systems in view of its application in residential buildings. However, the use of suspended calcium silicate board as a passive fire protection system has proven to be an effective solution to increase the fire endurance of sandwich panels by 20 to 35 minutes [131], making it feasible for the sandwich panel to meet the minimum fire safety requirements.

3.6. SANDWICH PANEL SOLUTIONS

Three different sandwich panel solutions were considered by combining different face sheets and core materials. As stated previously, the different solutions are based on the web-core system. This architectural layout was deemed the most appropriate for the new flooring system. It is cost-efficient as it allows the use of economic and low-strength core materials that are currently available in the market and does not complicate the manufacturing process of sandwich panels [132]. In the preliminary design stage, mechanical connections were envisaged between the webs and the face sheets of all the solutions. However, in a later stage, due to a requirement made necessary by the manufacturing process, adhesive bonding was selected as a preferable option. A tailored polyurethane-based adhesive was developed by the team of the Department of Polymer Engineering of the University of Minho. Further information on the subject will be provided in Section 4.3 of this thesis. The different solutions are illustrated in Figure 3.15, Figure 3.16, and Figure 3.17. Their cross-section is the result of a preliminary optimisation process by trial-and-error method. The optimisation parameters were the total thickness of the panel and the number of reinforcing elements in the core. The target of the optimisation was to retrieve the architecture with the lowest cost in fulfilment of the structural safety requirements at the ULS and SLS.

Solution SP1 includes CFS face sheets and webs and a PUR foam core (see Figure 3.15). The outer webs enclosing the core are constituted by C-section profiles, whereas the inner webs are composed by Z-section profiles. The design considered both the face sheets and the webs composed of continuous hot-dip zinc-coated carbon steel sheets with a yielding and ultimate strength of 220 MPa and 300 MPa, respectively. The contribution of the PUR foam to the mechanical resistance was neglected as a conservative assumption. The governing design factor was the resistance of the cross-section to bending moment.

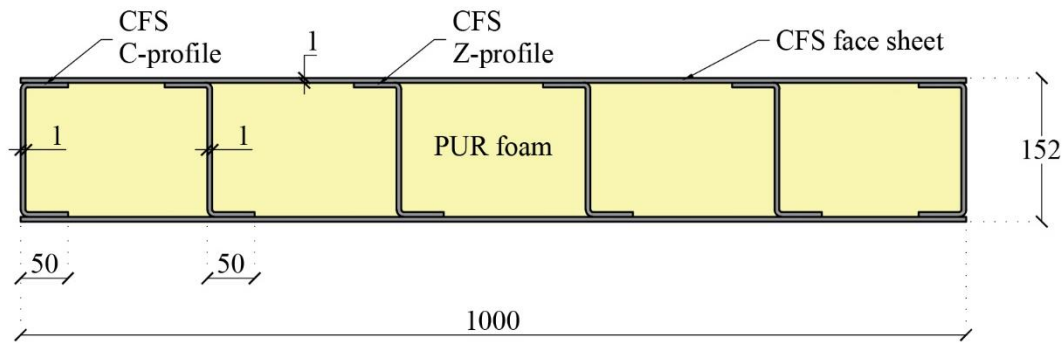


Figure 3.15. SP1 cross-section layout. All units in [mm]. Note: the thickness of the face sheets and webs are not to scale.

Solution SP2 includes CFS face sheets, GFRP webs and a PUR foam core (see Figure 3.16). The face sheets possess the same characteristics as those considered for solution SP1. The GFRP webs were selected from the commercial C- and I-profiles manufactured by the company Fiberline Composites A/S. The laminate architecture is symmetrical and balanced. The internal and external laminas are made up of unidirectional glass fibre roving and bi-directional fabric mats, respectively. The design considered the mechanical properties attributed to pultruded profiles of class E23 according to EN 13706-3 (2002) [133]. The contribution of the PUR foam was neglected as per the previous solution design. The structural safety verifications were carried out according to the Italian Guide for the Design and Construction of Structures made of FRP Pultruded Elements (2007) [105]. The design was carried out for the most stressed reinforcing element, namely the GFRP I-profile. The resisting cross-section included the effective part of the CFS face sheets calculated according to the Eurocode 3 Part 1-3 (2006) [80] and Part 1-5 (2006) [102]. Ultimately, the design was governed by the deflection limit at the SLS.

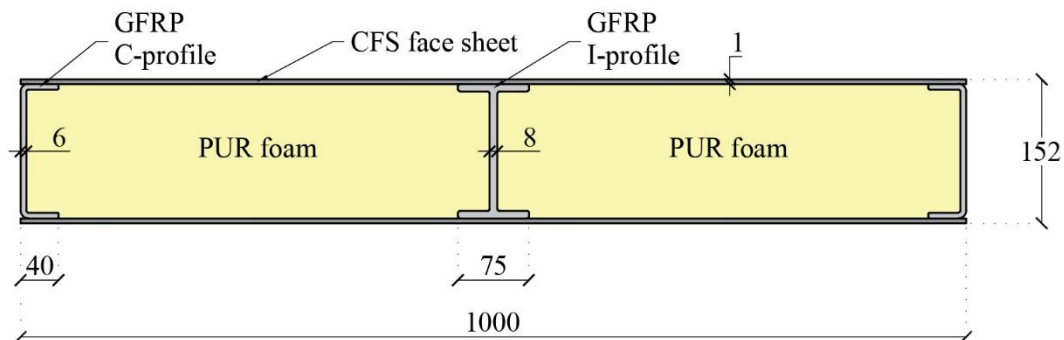


Figure 3.16. SP2 cross-section layout. All units in [mm]. Note: the thickness of the face sheets and webs are not to scale.

Solution SP3 includes CFS face sheets and webs and a balsa wood core (see Figure 3.17). The face sheets and webs have the same characteristics as those in the previous solutions. The balsa wood core consists of veneer layers with the grain oriented in the direction perpendicular to the face sheets of the panel. Average properties of balsa wood core and sandwich panel failure modes considered in the design were based on the guidelines reported in the CEN WG 4 Scientific and Technical Report (2014) [104].

The area of the compressed CFS face sheet was considered to be fully effective. However, the wrinkling stress accounting for the contribution of the balsa wood core was considered instead of the steel yield strength. As a conservative assumption, the shear action was considered to be carried only by the CFS web elements. Thus, their verification was carried out according to the Eurocode 3 Part 1-3 (2006) [80] and Part 1-5 (2006) [102]. The driving factor of the design was the shear resistance of the CFS webs.

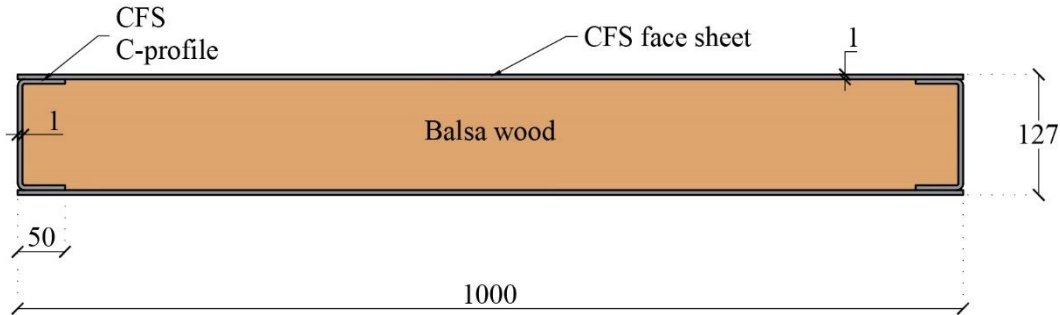


Figure 3.17. SP3 cross-section layout. All units in [mm]. Note: the thickness of the face sheets and webs are not to scale.

The outcome of the design process, including the imposed load carrying capacity with respect to bending moment ($R_{M,d,Q}$) and shear ($R_{V,d,Q}$), as well as the deflection caused by permanent and variable loads (δ_{max}), weight, and cost, are summarised in Table 3.1. The weight and cost of the sandwich panels are based on the nominal dimensions, mass density, and unit cost of the materials used. The unit costs are estimated based on the prices provided by the supplier of the sandwich panel manufacturer.

Table 3.1. Structural performance, weight, and cost of solutions SP1, SP2, and SP3.

Solution	$R_{M,d,Q}$ [kN/m ²]	$R_{V,d,Q}$ [kN/m ²]	δ_{max} [mm]	Weight [kg/m ²]	Cost [€/m ²]
SP1	3.1	8.4	13	33.4	32
SP2	3.4	8.6	13	30.6	93
SP3	4.2	3.4	19	36.2	128

The preliminary design and optimisation process yielded expected results. Solution SP2 is the lightest due to the presence of the GFRP webs, whereas solution SP1 resulted in the most economical prototype. Solution SP3 has a smaller thickness compared to SP2 and SP3, demonstrating that the inclusion of the core in the contribution to the overall structural performance leads to significant saving of materials. Due to its low cost and the experience of Ferpainel in the production of steel insulating sandwich panels, solution SP1 was selected for the next stage of the design process, namely the optimisation process by means of GA.

3.7. GENETIC ALGORITHM OPTIMISATION PROCESS

In the scope of this thesis, a new GA has been developed to optimise the design of solution SP1 in terms of weight, cost, and environmental impact. A primary target of the optimisation procedure is to ensure that it is easily accessible to non-experts. In order to achieve this, the GA should be straightforward to implement, flexible, and computationally efficient. With this aim, a code was developed and implemented in C language, based on the GA described in this section.

Despite the aim of approaching the solution of realistic engineering problems with GAs, it is necessary to discuss the strategies used in the evolutionary process in order to evaluate the reliability of the method. In particular, attention will be dedicated to the description and tuning of the penalty function as well as explaining its versatility and ease of implementation with the verifications of the Eurocode 3 Part 1-3 (2006) [80]. Finally, the result of the optimisation process in terms of optimal solutions will be presented.

3.7.1. Formulation of the optimisation problem

GAs operators, which will be described in detail in the next section, manipulate the genetic information of the candidate solution to improve it in the next generation. Thus, the first step in the formulation of the optimisation problem is the translation of the geometric and material properties of SP1 into a chromosome-like structure. This process, known as encoding, is shown in Figure 3.18 and converts the observable features of the sandwich panel, also called its phenotype, into its corresponding genetic constitution, called the genotype. The following design variables are considered in the optimisation process of the sandwich panel: i) the total thickness of the sandwich panel (t_{tot}); ii) the thickness of the CFS sheet (t_s); iii) the width of the web flange and (b_f); iv) the number of reinforcing webs in the core (n_w); and v) the density of the PUR foam core (ρ_{PUR}).

As mentioned earlier, when using the trial-and-error optimization process, the weight, cost, and GWP of the sandwich panel are determined based on the nominal dimensions and the mass density, unit cost, and carbon footprint of the steel, PUR foam, and adhesive (see Table 3.2). The assessment of the unit carbon footprints is based on the values found in the literature [12] and the life cycle assessment study (LCA) conducted by different hot-dip galvanized steel manufacturing companies [134]. The LCA studies considered in this work follow the cradle-to-gate approach. These studies assess the environmental effects of building materials starting from the extraction of raw materials, up to the completion of the finished product ready for shipment from the factory gate. However, they do not consider downstream activities

such as transportation, final use, disposal, and recycling. The impact category studied is global warming whose indicator is the GWP for a time horizon of 50 years. Furthermore, given the small amount of adhesive to be used in the panel its contribution to the overall mass and carbon footprint is neglected.

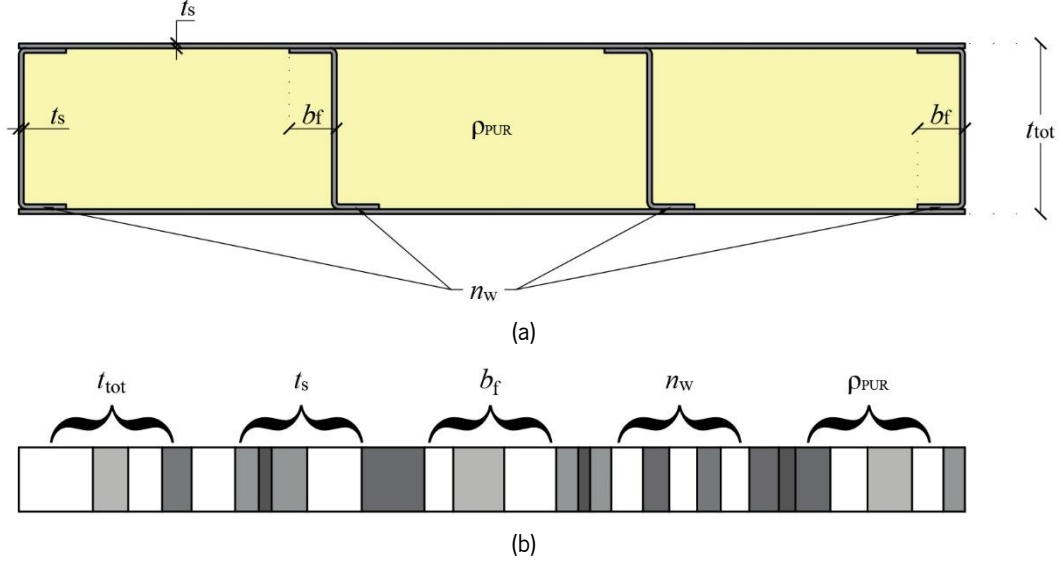


Figure 3.18. Encoding procedure of the sandwich panel: (a) phenotype; (b) genotype.

Table 3.2. Mass density, unit cost, and carbon footprint of the components of the sandwich panel.

Material	Mass density [kg/m ³]	Unit cost [€/kg]	Unit carbon footprint [kgCO ₂ -eq./kg]
Steel	78.5	0.82	2.8
PUR foam	35, 40, 50, 60, 70, 80, 90, 100, 110, 120	1.79	4.14
Adhesive	Neglected	9.54	Neglected

The weight (wt), cost (cst), and global warming potential (gwp) objective functions are aggregated into a single objective function ($f(\mathbf{x})$) according to the weighted-based genetic algorithm (WBGA) approach:

$$f(\mathbf{x}) = \sum_{i=1}^3 w_i \cdot f'_i(\mathbf{x}) \quad (3.29)$$

where $\mathbf{x}$ is the design variable vector, w_i is the weight assigned to the i – th objective function, and $f'_i(\mathbf{x})$ is the normalized value of the objective functions. Indeed, the magnitude of each objective function f_i is different, thus, a min–max normalization is implemented. The min-max normalization consists in using the weight, cost, and environmental footprint of the best and worst individual at each generation to calculate the normalized values of the objective functions. The objective function's minimum value is scaled to 0, the maximum value is scaled to 1, while every other value is scaled to a decimal between 0

and 1. The normalization process allows the comparison of objective functions of different magnitudes. Consider, for instance, the normalised value of the weight of an individual wt'_j :

$$wt'_j = \frac{wt_j - wt_{\min}}{wt_{\max} - wt_{\min}} \quad (3.30)$$

where wt_j is the weight of the j individual, $wt_{\max}$ is the weight of the heaviest individual, and $wt_{\min}$ is the weight of the lightest individual.

The WBGA approach aggregates the objectives, producing a single compromise solution according to the decision-maker [85]. It is a simple extension of the single objective GA and therefore, it is straightforward to implement. Taking into consideration the aforementioned aspect, the WBGA approach was deemed the most adequate to retrieve the optimal cross-section of the sandwich, which had to be ultimately manufactured and tested. Nevertheless, from a scientific standpoint and under the pressure of climate change issues related to the construction sector (or rather humankind itself), it is interesting to retrieve at least the extreme of the Pareto optimal set. Therefore, assigning different weights in Equation (3.29), different alternatives are sought, namely the lightest, the most economical and the least environmental impacting solutions. This requires the code to be run multiple times with different weights of the objective functions. However, if the decision-maker has no clear preference among the initial objectives the Pareto-based approach should be used [135]. This approach has the advantage of providing a Pareto set of solutions which should be uniformly distributed and diverse over the true Pareto front in a single run. A trade-off study has to be carried out to select a compromise solution from the Pareto set.

3.7.2. Constraints and penalty function

When solving an optimisation problem, the constraints can be divided into two categories: i) box condition constraints; and ii) behavioural constraints. The former group of constraints limits the range of values that the design variables can take, as detailed in Table 3.3.

Table 3.3. Box conditions and mutation steps of the design variable of the sandwich panel.

Design variable	Symbol	Unit	Minimum	Maximum	Step
Total thickness	t_{tot}	[mm]	50	200	1
Steel sheet thickness	t_s	[mm]	0.5	2.0	0.1
Width of web flange	b_f	[mm]	20	$50 \cdot t_s$	1
Number of webs	n_w	[mm]	3	4	1
PUR foam mass density	ρ_{PUR}	[kg/m ³]	35	120	1

The range of values for the design variables was limited for various reasons: i) t_{tot} can range between 50 mm and 200 mm, which is comparable to those of traditional floor systems; ii) the limit for ρ_{PUR} is set according to the manufacture capability of Ferpaine; iii) n_w (which includes the outer webs) upper limit is related to manufacturing constraints, whereas the lower limit is based on the research of sandwich panels that indicates that typically at least two webs in the core are necessary for adequate structural behaviour [59], [61]; iv) t_s and b_f are within the interval permissible by Eurocode 3 Part 1-3 (2006) [80].

Regarding the behavioural constraints, they are implicit functions of $\mathbf{x}$ and they follow from the structural, thermal, and acoustic requirements described in the previous sections. The behavioural constraints are incorporated in $f(\mathbf{x})$ employing an adaptive penalty function ($f_p(\mathbf{x}, t)$) proposed in [93] given in Equation (3.31):

$$f_p(\mathbf{x}, t) = f(\mathbf{x}) + (F_{\text{feas}}(t) - F_{\text{all}}(t)) \cdot \sum_{i=1}^6 \frac{d_i}{NFT_i} \quad (3.31)$$

where t is the number of generations, $F_{\text{all}}(t)$ the unpenalized value of the best solution yet found, $F_{\text{feas}}(t)$ the value of the best feasible solution yet found, d_i the distance metric of each constraint, and NFT_i is the near feasible threshold described later on in this section. The penalty function involves adding a penalty based on the difference between the solutions performance and the legal requirement of the corresponding performance. In the literature of GAs, this difference is known as cost to completion [136] and represents the distance of the individual from the feasible solution space (d_i). Regarding constraints stemming from structural engineering practice, verification is usually expressed in terms of inequality between the design resistance (R_d) and the design value of the relevant force (F_{Ed}). This inequality can be used to calculate d_i of the penalty function adopted in the GA:

$$d_i = \begin{cases} F_{\text{Ed}} - R_d & \text{if } R_d - F_{\text{Ed}} < 0 \\ 0 & \text{if } R_d - F_{\text{Ed}} \geq 0 \end{cases} \quad (3.32)$$

The implementation of the current design practice requirements is straightforward and demonstrates the versatility of the optimisation method for structural members other than sandwich panels. The definition of d_i for the constraint described in Section 3.1 is given in Table 3.4.

Table 3.4. Behavioural constraints of the sandwich panel.

Constraint	Unit	Category
$R_{Md,Q} = \frac{R_{Md} - \gamma_G \times G_k}{\gamma_Q} \geq 2.0$	[kN/m ²]	ULS
$R_{Vd,Q} = \frac{R_{Vd} - \gamma_G \times G_k}{\gamma_Q} \geq 2.0$	[kN/m ²]	ULS
$\delta_{max} \leq 10$	[mm]	SLS
$U_c \leq 0.30$	[W/m ² K]	Thermal
$D_{nT,w} \geq 30$	[dB]	Acoustic
$L_{nT,w} \leq 100$	[dB]	Acoustic

The amount of the penalty added to the value of the objective function varies according to the severity of the constraint violation. This is achieved by means of the definition of a near feasible threshold at each generation ($NFT_i(t)$):

$$NFT_i(t) = \frac{NFT_{0,i}}{\lambda \cdot t} \quad (3.33)$$

where $NFT_{0,i}$ represents the initial value of NFT_i , and λ is the dynamic parameter. NFT_i represents the distance from the feasible region of the solution space beyond which the GA is discouraged to search for a solution. Indeed, the inclusion of unfeasible regions at the beginning of the search is useful since the optimal solution often lies at the boundary of the feasible region [94]. In terms of realistic design problems, this corresponds to retaining in the population the design characteristics of solutions which are not currently meeting the requirements but present low value of the objective function, i.e. extremely lightweight, cost effective, or environmentally friendly designs. Generally λ takes a positive value so that as t increases NFT_i reduces, forcing the GA to favour designs which are in complete fulfilment of the requirements. A common problem in the use of penalty function is the choice of the constant parameters [29], which are represented in the proposed algorithm by $NFT_{0,i}$ and λ . Further details on the tuning of the parameters will be given in Section 3.7.4. Nevertheless, it is important to notice that in the scope of the application of the optimisation code in current engineering practice, $NFT_{0,i}$ may be interpreted as the initial difference allowed between the performance of a randomly initialized design and the performance required by the correspondent building code. For instance, considering the SLS constraint in Table 3.4, $NFT_{0,3}$ is the estimated deflection of a sandwich panel design at generation $t = 0$ which will not be heavily penalized by the GA. In other words, $NFT_{0,3}$ represents the required

percentage of the fulfilment of the SLS requirements in the beginning of the optimisation process as given in Equation (3.34):

$$NFT_{0,3} = \frac{\delta_{\max}}{5\%} = \frac{10}{0.05} = 200 \quad (3.34)$$

Finally, the adaptive feature of the penalty function is given by the difference between $F_{\text{feas}}(t)$ and $F_{\text{all}}(t)$. The severity of the penalty decreases if all the best individuals in the previous generation $t - 1$ were feasible, whereas it increases if they were all unfeasible.

3.7.3. Description of OSPANEL code and the evolutionary search process

As stated before, the target of the genetic algorithm is to retrieve the set of design variables $\mathbf{x}$ which minimizes one of the objectives selected by the decision-maker, namely weight, cost, and environmental impact of the sandwich panel. With this aim, a computational code is developed (OSPANEL) in the programming language C in the integrated development environment Visual Studio. The code is composed of different modules which are described in detail in the Annex Chapter. The main steps constituting the genetic algorithm (see Figure 3.19) are the following:

1. The input parameters, namely the size of the population (N), the maximum number of generations ($t_{\max}$), the weight vector of the objective function ($\mathbf{w}$), and the selective pressure (s) are initialised;
2. The set of design variables $\mathbf{x}$ (t_{tot} , t_s , b_f , n_w , and ρ_{PUR}) is randomly initialised for N individuals and the number of generations is set equal to $t = 1$;
3. The penalised fitness ($f_p(\mathbf{x}, t)$) of the individuals in P_t is calculated;
4. The individuals are ranked (see *Module SortMinMax* in Figure 3.19) according to the penalized fitness;
5. The exponential ranking fitness (f_{exp}) and the mating probability (p_{mat}) of the individuals in P_t is calculated (see *Module CalcMatProbability* in Figure 3.19);
6. A set of N individuals that will generate the offspring population, i.e. the mating pool M_t , is selected from the individuals in P_t (see *Module CalcMatPool* in Figure 3.19);
7. The best half ($N/2$) of individuals in P_t is copied into the population of the new generation P_{t+1} . Pairs of individuals in M_t are combined together through the cross-over operator to create the offspring population Q_t constituted by $N/2$ individuals. A mutation is applied to the set of design

variables $\mathbf{x}$ of the individuals in Q_t which is copied into the population of the new generation P_{t+1} (see *Module CalcNewGeneration* in Figure 3.19);

8. The number of generations is updated $t = t + 1$. Steps 2, 3, 4, 5, and 6 are repeated until $t_{\max}$ is reached.

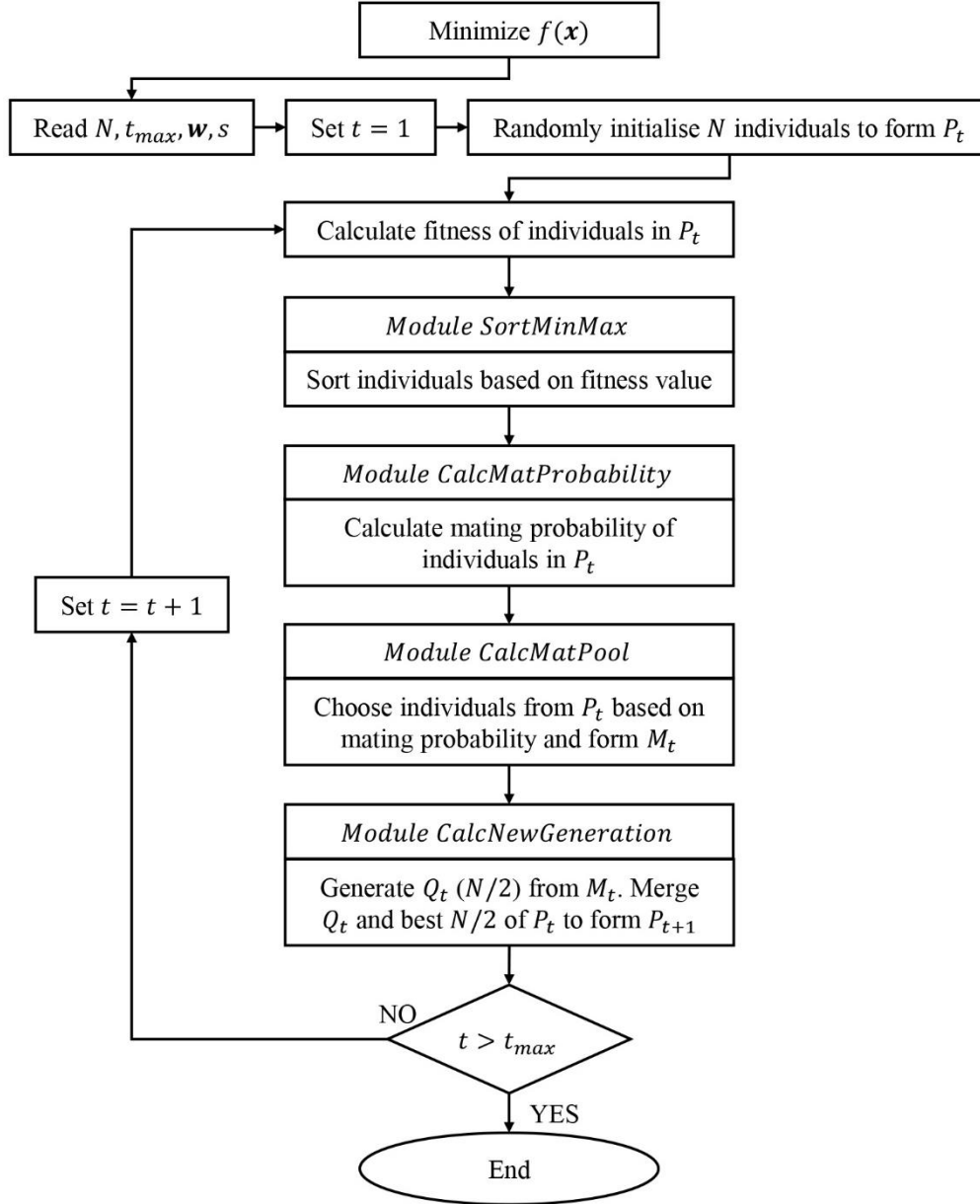


Figure 3.19. Implemented algorithm (OSPANEL) to optimise the design of the sandwich panel.

The developed procedure adopts a set of genetic operators in various steps of the above-described procedure: i) exponential ranking method [137]; ii) SUS method [138]; iii) crossover and mutation; iv) elitism [139]. The exponential ranking fitness assignment ($f_{\exp}$) is adopted in step 5, and it is given by:

$$f_{\exp} = s^r \quad (3.35)$$

where r is the ranking of the individual. The selective pressure (s) represents the degree to which the better individuals are favoured. The exponential ranking method ensures a constant selective pressure throughout the whole search in order to avoid premature convergence due to the presence of so-called “super individuals” in the starting population as well as slowing down the search at the end when the difference between the fitness of the individuals is small [92]. In the case of this work a value of 0.7, i.e. 30% of the offspring population inherits the genes of the best solution, is found to be optimal.

The SUS method is employed in step 6 to select the mating pool population (M_t). The method can be visualized as the result of a spinning roulette with slots proportional to the mating probability of each individual with multiple equally spaced pointers. Once it stops, the number of pointers over each slot corresponds to the number of copies selected for each individual for the mating pool. This method grants the minimum selection error which is caused by the fact that the number of copies is an integer number and must be rounded from the mating probability of the individual which is a real number.

Crossover and mutation are utilised in step 7 to manipulate the genetic information of the parent population in M_t to generate the new individuals. The offspring individuals inherit parts of the genetic information of both parents (see Figure 3.20), thus, accelerating the search for the optimal individual bringing together the feature of two individuals. On the other hand, mutation reintroduce diversity in the population, which is essential to overcome the problem of convergence to local optimal. Mutation is applied to all the genes of the new individuals with a rate of 0.8. The elitism strategy is also applied in step 7. This strategy consists in merging the best half of the parent population P_t with the offspring population Q_t to form the next generation P_{t+1} and, thus, preserving the best genetic information found yet in the search.

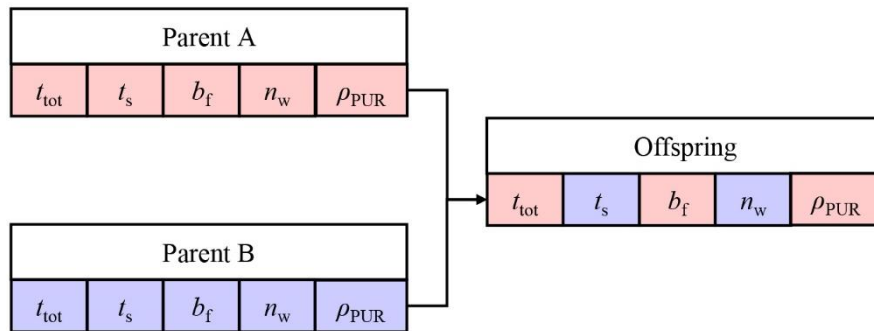


Figure 3.20. Crossover operator of the algorithm OSPANEL.

3.7.4. Tuning of the penalty function parameters

A parametric study conducted on $NFT_{0,i}$ and λ is used to verify the reliability of the results obtained from the GA. The optimal solutions presented in this section are compared in terms of weight with a preliminary known design. The relative difference is less than 5%, thus, the best designs are considered acceptable [140]. The weight vector is firstly set equal to $\mathbf{w} = \{1,0,0\}$, i.e. OSPANEL only optimises the weight of the sandwich panel. The results of multiple search varying $NFT_{0,i}$ are shown in Figure 3.21.

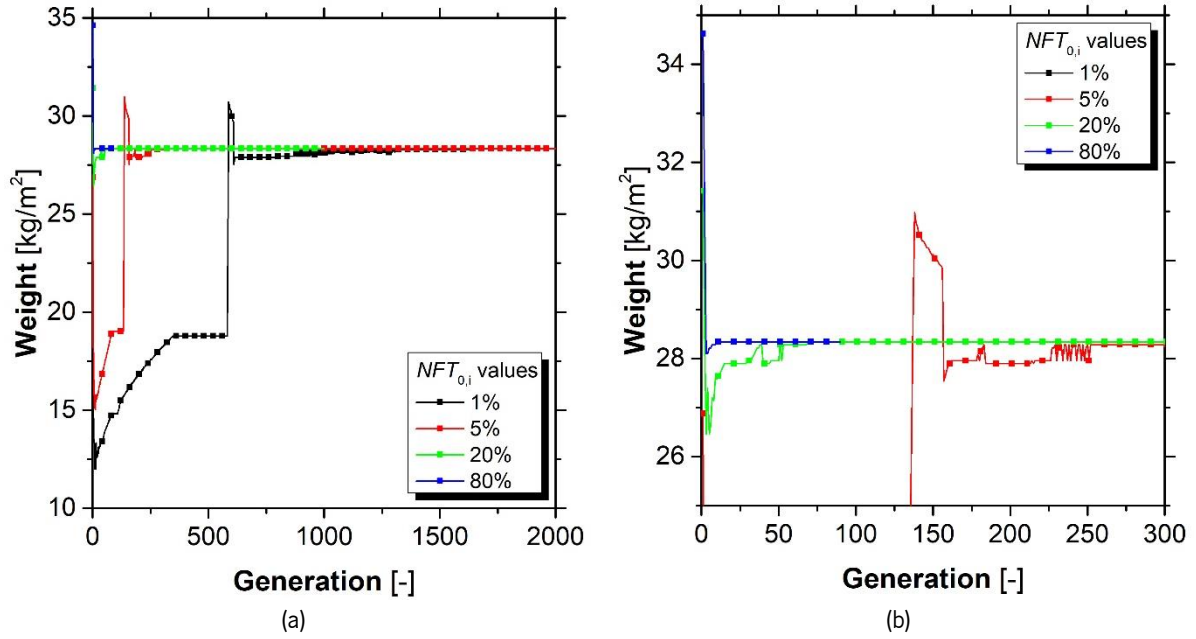


Figure 3.21. Weight of the lightest sandwich panel found by OSPANEL: (a) different runs varying the value of $NFT_{0,i}$; (b) zoom on the first 300 generations.

The weight per square meter of the lightest sandwich panel is plotted against the number of generations (Figure 3.21). It can be observed that with high values of $NFT_{0,i}$ the optimal solution, which weights 28.34 kg/m², is approached starting from very light sandwich panels which do not satisfy the constraints. On the other hand, with a low value of $NFT_{0,i}$ the starting solutions are heavier than the optimal one but in fulfilment of the requirements. In general, the higher the $NFT_{0,i}$ the more generations are required to reach convergence. However, computing speed may be relatively important.

In a second search, the weight vector is set equal to $\mathbf{w} = \{0,1,0\}$, i.e. OSPANEL only optimises the cost of the sandwich panel. Reducing $NFT_{0,i}$ speeds up the search but also reduces the solution space explored. The result is that for $NFT_{0,i}$ equal to 80% OSPANEL falls into a local optimum failing to reach the most economical solution, whose cost is 35.24 €/m², as shown in Figure 3.22a. For what concerns the coefficient λ , the results of the parametric study are reported in Figure 3.22b. The coefficient

influences the search in a similar way to that of $NFT_{0,i}$. The higher it is (coefficient λ), the faster OSPANEL reaches convergence. Nevertheless, the optimal solution is reached in all of the runs.

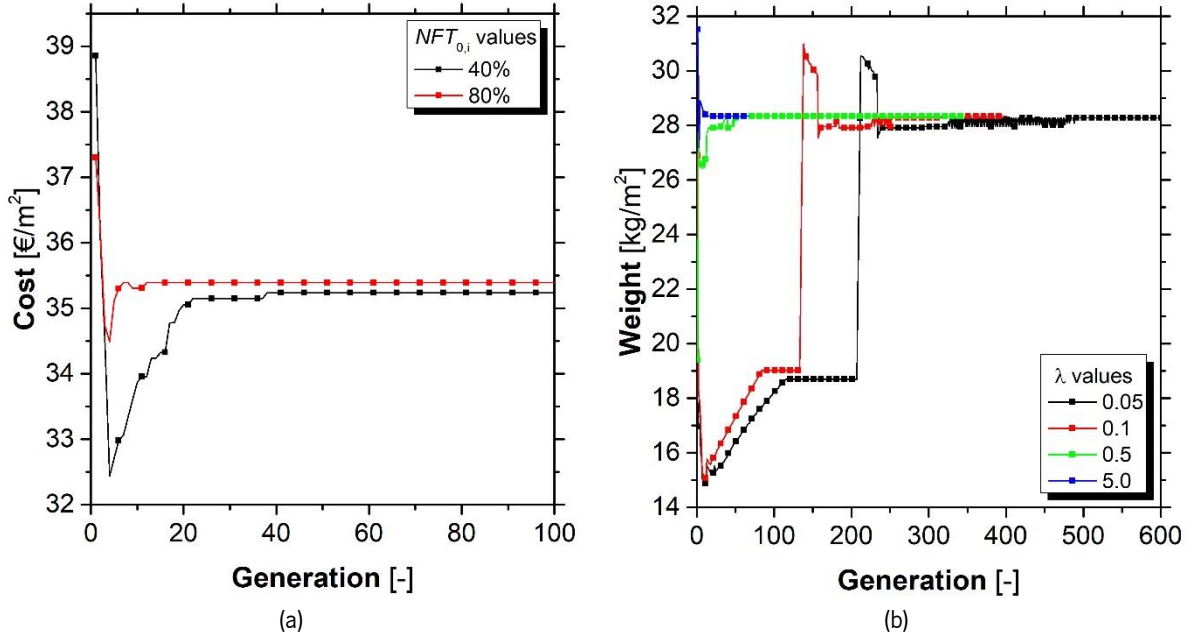


Figure 3.22. Sensitivity analysis on the adaptive penalty function parameters: (a) $NFT_{0,i}$; (b) λ .

The results of the tuning runs of OSPANEL suggest that it is preferable to search for a wider solution space at the expense of the computational speed. In this fashion, the starting solutions present extremely low values of the objective function and are subsequently adjusted to satisfy all the constraints. Therefore, in the next simulations $NFT_{0,i}$ and λ are set equal to 5% and 0.1 respectively. Since each iteration of the GA takes up approximately 0.05 s, 20000 iterations are recommended as a stopping criterion for the optimization procedure: in this way, the results of a single run can be retrieved within a period of about 17 min.

To compare the performance of the adaptive penalty function adopted in this work with other constraint handling techniques, the GA was properly modified to accommodate a static penalty function commonly found in the literature [19,24] and given by:

$$f_p(\mathbf{x}) = f(\mathbf{x}) + \sum_{i=1}^6 d_i \quad (3.36)$$

Several runs of the modified GA were performed, and their results compared to the solution previously found by OSPANEL. The modified GA retrieved the optimal solution only one out of five runs, i.e. the second run in Figure 3.23. Furthermore, only small variations can be observed during the early stages of

the search. It can be concluded that the proposed adaptive penalty function allows i) to explore a wider solution space, ii) avoid premature convergence, and iii) make the results of the GA less sensitive to the starting population.

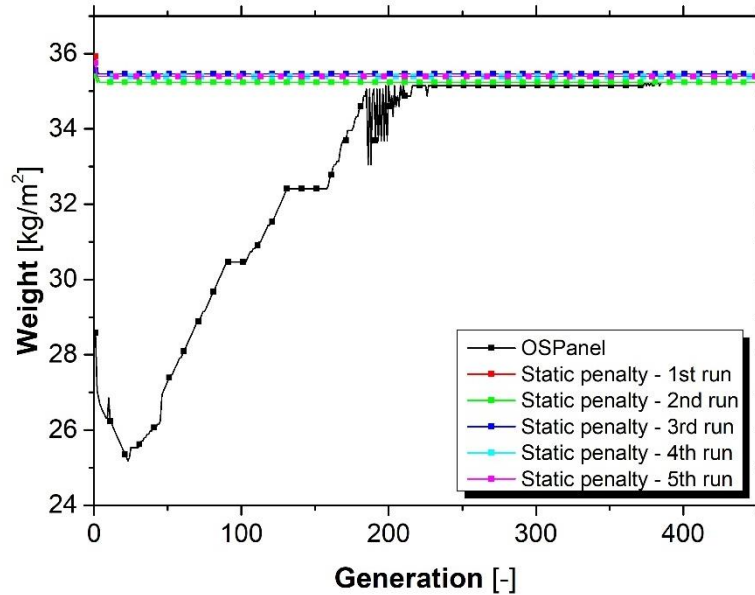


Figure 3.23. Comparison of the performance of the modified GA, including static penalty function, and OSPANEL.

In Table 3.5, the parameters of the genetic operators adopted in the search of the optimal solutions presented in the next section are summarised.

Table 3.5. The genetic parameters adopted in OSPANEL.

Genetic parameters	Symbol	Default value
Number of individuals in a population	N	1000
Near-feasible threshold upper bound	NFT_0	5%
Dynamic search parameter	λ	0.1
Selective pressure of exponential ranking method	s	0.7
Type of selection scheme	-	SUS
Type of crossover operator	-	Multi-point crossover
Crossover rate	-	1
Mutation rate	-	0.8
Elitist strategy	-	TRUE
Maximum number of generations	$t_{\max}$	20000

3.7.5. Optimal solutions

OSPANEL is used to retrieve the extreme of the Pareto front, namely the lightest (S_1), the most economical (S_2), and the least polluting (S_3) sandwich panel designs. A summary of the characteristics of the optimal solutions is illustrated in Figure 3.24a and Figure 3.24b. S_1 solution coincides with the

S_3 one which agrees with the result on the optimization study of sandwich panels [12]. This may be explained by the fact that the carbon footprint is proportional to the weight of the component materials. The steel weight makes up for 80% of the total weight of the sandwich panel, thus the GA minimizes its use. A different scenario may arise if a less conservative structural design approach would be available, and the PUR foam contribution to the resistance of the sandwich panel is considered. In such a case, a lighter solution may be obtained increasing the density of the foam to allow for the use of a thinner steel sheet with an improved buckling resistance resulting from the denser core support. However, this evolutionary strategy may not lead to a less polluting solution since the PUR foam has a greater environmental impact than steel according to Table 3.2.



Objective functions	Design variables	Constraints
$wt=28.34 \text{ kg/m}^2$	$t_{\text{tot}}=187 \text{ mm}$	$R_{\text{Md,Q}}=2.01 \text{ kN/m}^2$
$cst=35.24 \text{ €/m}^2$	$t_s=0.9 \text{ mm}$	$R_{\text{Vd,Q}}=2.48 \text{ kN/m}^2$
$gwp=88.03 \text{ kgCO}_2\text{-eq./m}^2$	$b_f=45 \text{ mm}$	$\delta_{\text{max}}=9.20 \text{ mm}$
	$n_w=4$	$U_c=0.20 \text{ W/m}^2\text{K}$
	$\rho_{\text{PUR}}=35 \text{ kg/m}^3$	$D_{\text{nT,w}}=33.4 \text{ dB}$
		$L_{\text{nT,w}}=98.6 \text{ dB}$

(a)



Objective functions	Design variables	Constraints
$wt=29.56 \text{ kg/m}^2$	$t_{\text{tot}}=173 \text{ mm}$	$R_{\text{Md,Q}}=2.01 \text{ kN/m}^2$
$cst=35.23 \text{ €/m}^2$	$t_s=1.0 \text{ mm}$	$R_{\text{Vd,Q}}=4.40 \text{ kN/m}^2$
$gwp=90.81 \text{ kgCO}_2\text{-eq./m}^2$	$b_f=41 \text{ mm}$	$\delta_{\text{max}}=9.86 \text{ mm}$
	$n_w=4$	$U_c=0.20 \text{ W/m}^2\text{K}$
	$\rho_{\text{PUR}}=35 \text{ kg/m}^3$	$D_{\text{nT,w}}=33.4 \text{ dB}$
		$L_{\text{nT,w}}=98.6 \text{ dB}$

(b)

Figure 3.24. Cross-section and characteristics of the optimal solutions: (a) S_1 and S_3 solution; (b) S_2 solution.

The normalized value of the constraints during the search of the S_1 and S_3 solutions is shown in Figure 3.25a and Figure 3.25b, respectively. The bending moment resistance is the active constraint [141], i.e. the requirements which is ultimately governing the design as it is the last one to be fulfilled during the

search. This depends mostly on the total thickness of the panel which is the last design variable converging to its optimal value as shown in Figure 3.25c. The shear resistance and the midspan vertical deflection are relatively less stringent as they are fulfilled earlier in the run. The value of the thermal resistance fluctuates when great variations in the total thickness are observed. This is an expected result as the thermal performance relies on the thickness of the core insulating layer which makes up most of the total thickness.

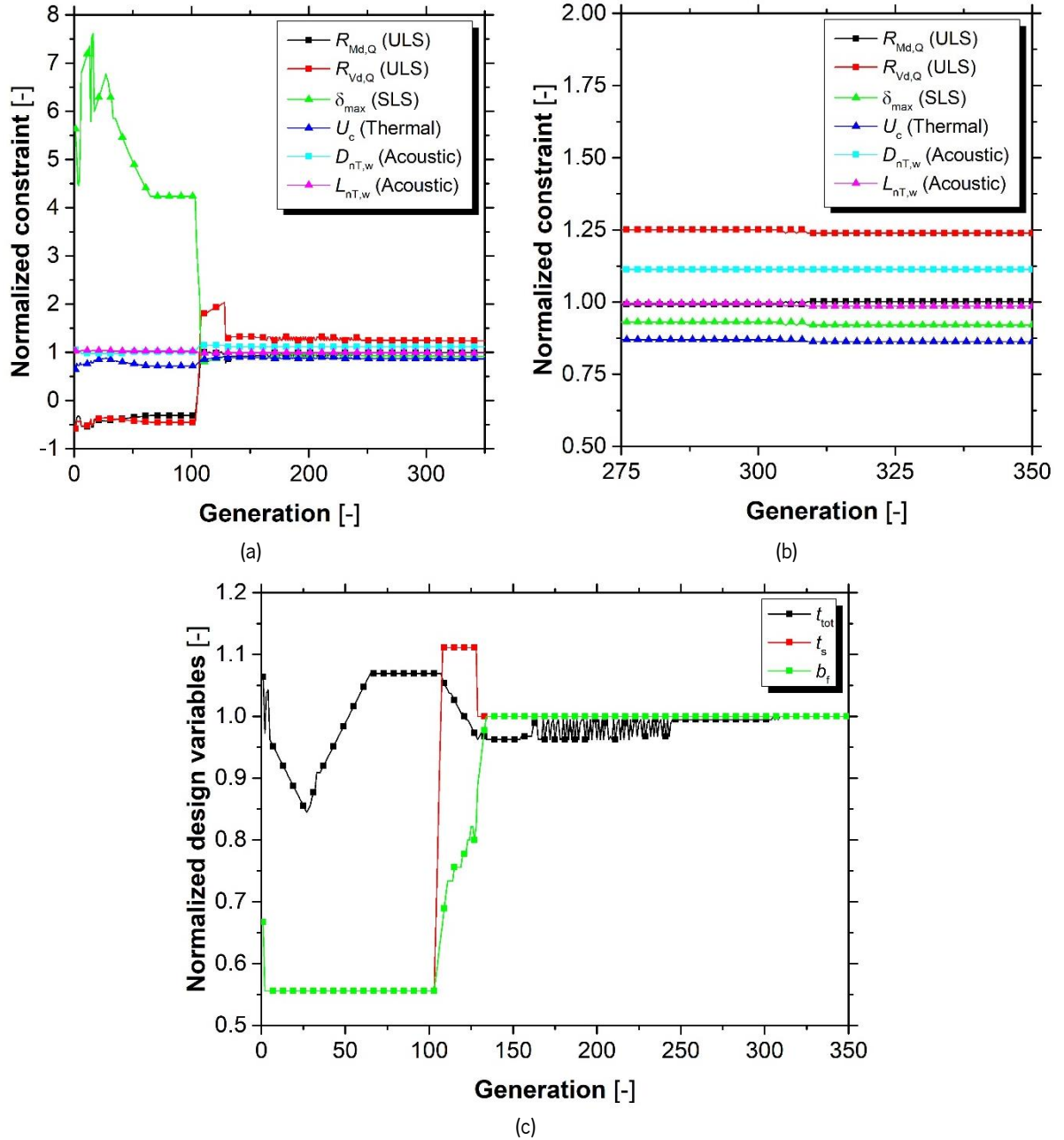


Figure 3.25. Search history of S_1 and S_3: (a) normalized constraints value; (b) zoom at the converged iteration of the normalized constraints value; (c) normalized design variable value. Note: Lines with triangle symbols are constraints that must be equal or less than 1, whereas lines with square symbols are constraints that must be equal or greater than 1.

Despite the large range of geometric design variables explored during the search, the sound insulation constraints show little variation. This corroborates the difficulty of improving the acoustic performance of sandwich panels. A 4 dB increase in the sound insulation can only be achieved by doubling the weight of the sandwich panel.

OSPANEL identifies in the 4 webs configuration, i.e. the maximum number allowable, a preferable option both for the S_1, S_2, and S_3 solutions. This confirms the findings of previous studies on the flexural behaviour of sandwich panels [59], [61]. The webs are fundamental in providing the required bending stiffness to the structural element. It should also be noted that in the lightest solution the flanges of the webs are at the limit of the geometrical requirements of the Eurocode 3 Part 1-3 (2006) [80]. This is due to the fact that they provide a constraint for the top face sheet against buckling.

The normalized values of the constraints and design variables in the search history of S_2 solution are illustrated in Figure 3.26a. The constraints and design variables search history (see Figure 3.26b) show a similar pattern to those of S_1 and S_3 solutions. In comparison to the latter, the S_2 solution presents thicker steel sheets and less wide flanges. This indicated that the price of the adhesive is relevant to the overall cost of the sandwich panel. The results of OSPANEL indicates that it is more economical to increase the flexural stiffness of the panel by thickening the steel sheets rather than adhesively joining together wider flanges and top face sheet.

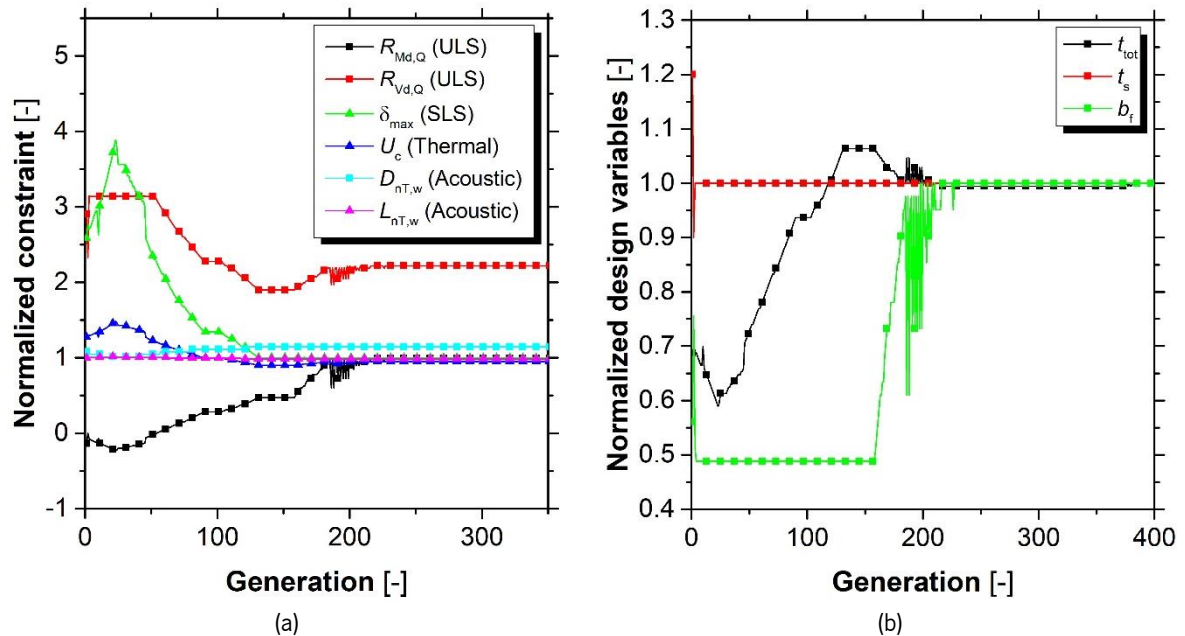


Figure 3.26. Search history of S_2: (a) normalized values of the constraints; (b) normalized values of the design variables.

Fluctuations and singularities can be observed in the plot of the design variables and the constraints in the search history of both optimal solutions (see Figure 3.25 and Figure 3.26). In the search for the

optimal solution, when a solution with a low value of the objective function, not fitting the constraints, ceases to be the best solution, two new designs might appear with very similar values of the penalized objective function. OSPANEL takes some generations to establish the best among the two competing designs which result in the fluctuations and singularities observed in the graphs. OSPANEL switches between the two competing solutions until the closest to meet the constraints emerges as the best one. This may be explained by the fact that both $NFT_{0,i}$ and λ values are set to be small (the former one in terms of percentage) so that the unfeasible region of the solution space that OSPANEL is encouraged to search is slowly decreasing through the generations. Indeed, in Figure 3.21 and Figure 3.22 it is possible to observe that fluctuations and singularities tend to disappear when the above-mentioned parameters are set in order to perform a faster search and explore an unfeasible region that is rapidly decreasing.

3.7.6. Parametric studies with OSPANEL code

To explore a wider solution space, the values of design variables and requirements are modified keeping the other factors constant. Firstly, the maximum number of webs in the core is progressively increased, disregarding the manufacturing constraints. The population is forced to maintain a constant number of webs during the whole search so that multiple optimal solutions with a different number of webs are retrieved. The result of the parametric study is shown in Figure 3.27.

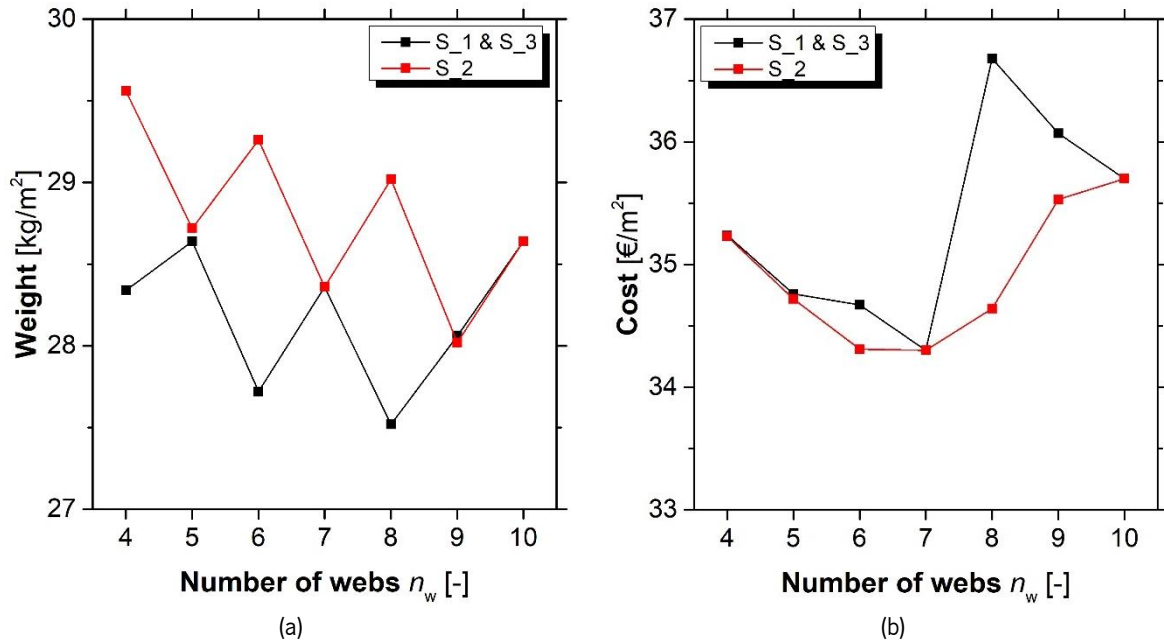
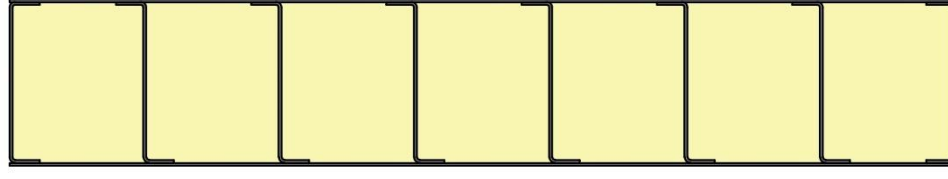


Figure 3.27. Parametric study on the number of webs: (a) weight of the optimal solutions; (b) cost of the optimal solutions.

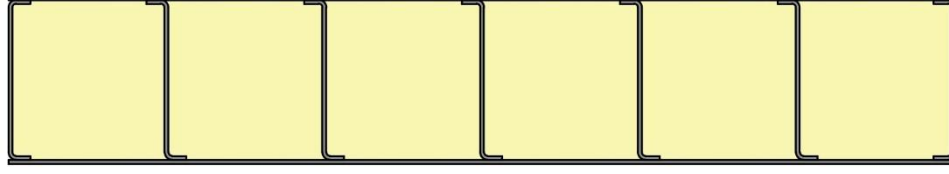
The S_3 solution with 8 webs is the lightest and least polluting sandwich panel (Figure 3.28). However, it is also the most expensive since a large amount of adhesive is required to bond the flanges with the

top face sheet. The S_2 solutions with 6 and 7 webs are both economically competitive. Furthermore, the 7 webs architecture appears to be a local optimum as the S_1, S_2, and S_3 solutions are identical.



Objective functions	Design variables	Constraints
$wt=27.52 \text{ kg/m}^2$	$t_{\text{tot}}=176 \text{ mm}$	$R_{\text{Md,Q}}=2.52 \text{ kN/m}^2$
$cst=36.68 \text{ €/m}^2$	$t_s=0.7 \text{ mm}$	$R_{\text{Vd,Q}}=3.02 \text{ kN/m}^2$
$gwp=85.27 \text{ kgCO}_2\text{-eq./m}^2$	$b_f=32 \text{ mm}$	$\delta_{\text{max}}=9.96 \text{ mm}$
	$n_w=8$	$U_c=0.28 \text{ W/m}^2\text{K}$
	$\rho_{\text{PUR}}=35 \text{ kg/m}^3$	$D_{\text{nT,w}}=33.4 \text{ dB}$
		$L_{\text{nT,w}}=98.6 \text{ dB}$

(a)



Objective functions	Design variables	Constraints
$wt=28.36 \text{ kg/m}^2$	$t_{\text{tot}}=177 \text{ mm}$	$R_{\text{Md,Q}}=2.02 \text{ kN/m}^2$
$cst=34.30 \text{ €/m}^2$	$t_s=0.8 \text{ mm}$	$R_{\text{Vd,Q}}=4.17 \text{ kN/m}^2$
$gwp=87.64 \text{ kgCO}_2\text{-eq./m}^2$	$b_f=23 \text{ mm}$	$\delta_{\text{max}}=9.85 \text{ mm}$
	$n_w=7$	$U_c=0.28 \text{ W/m}^2\text{K}$
	$\rho_{\text{PUR}}=35 \text{ kg/m}^3$	$D_{\text{nT,w}}=33.4 \text{ dB}$
		$L_{\text{nT,w}}=98.6 \text{ dB}$

(b)

Figure 3.28. Cross-section and characteristics of the optimal solutions obtained from the parametric study: (a) S_1 and S_3 solution; (b) S_2 solution.

In Figure 3.29 the imposed load-carrying capacities, deflection, and thermal transmittance of the S_1, S_2, and S_3 solutions are reported for each number of webs. $R_{\text{Md,Q}}$ for optimal solutions with less than 8 webs approaches the limit value, that is the bending moment is the active constraints for these sandwich panels. On the other hand, $R_{\text{Vd,Q}}$ constraint is always largely fulfilled. The assumption of a constant thickness for the whole section facilitates the effective width calculation but it seems to preclude the possibility of further improvement in the optimization of the webs, which govern the shear resistance of the panel. The vertical deflections increase with increasing number of webs. This may be explained by the fact that the optimal solutions with a greater number of webs have smaller total and steel sheet thicknesses that result in a lower lever of arm produced by the face sheets and reduced effective area,

respectively. Both these phenomena contribute to the decrease of the bending stiffness of the panel. Finally, thermal performances are illustrated in Figure 3.29d. The thermal transmittances of the S_1 and S_2 solution with more than 9 and 8 webs, respectively, are slightly above the limit. Thus, for these sandwich panel architectures the governing criterion of the design is not related to the structural performance but to the thermal one.

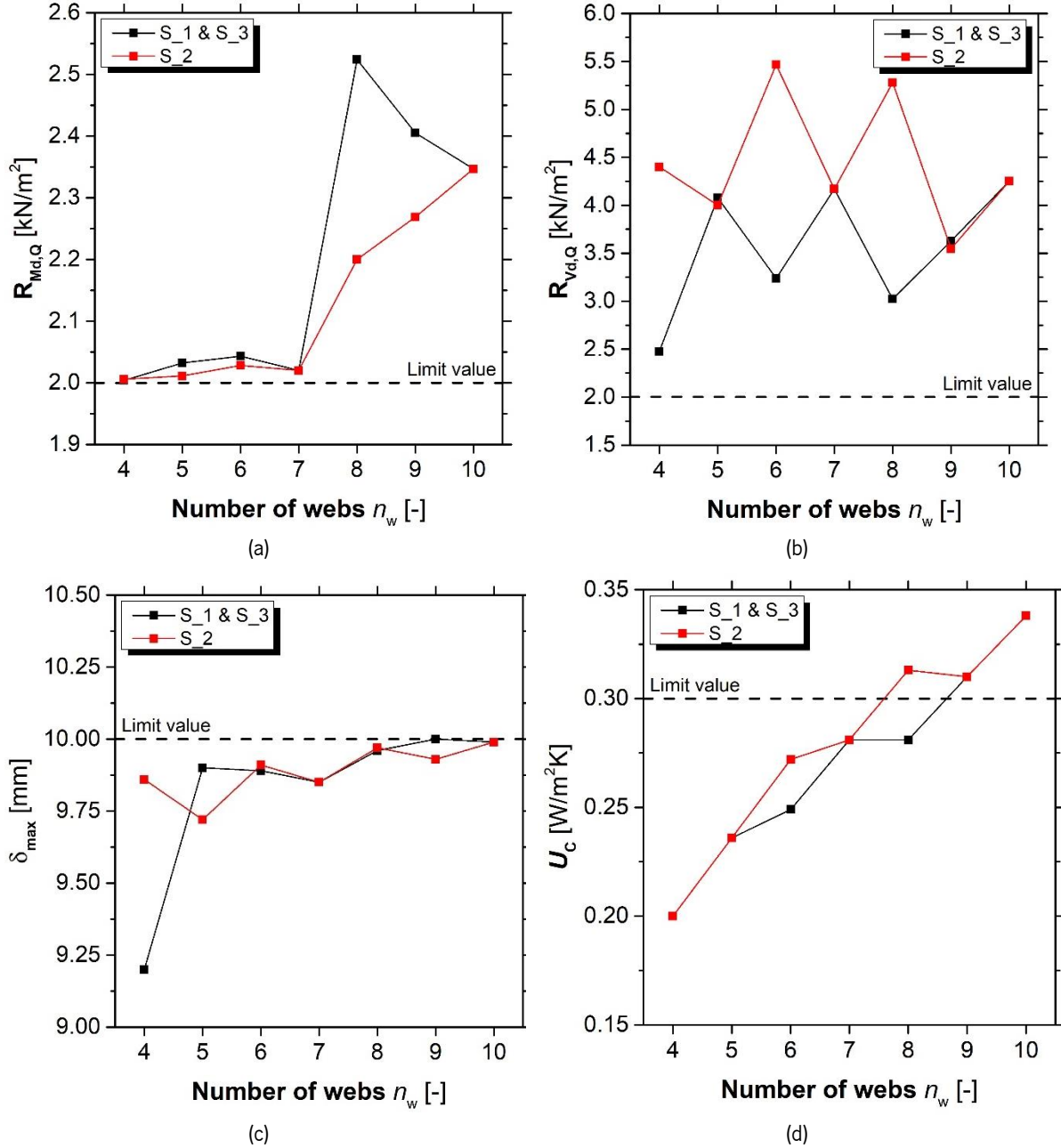


Figure 3.29. Values of the constraints of the optimal solutions: (a) $R_{M,d,Q}$; (b) $R_{V,d,Q}$; (c) δ_{max} ; (d) U_c .

The design variables for the different layouts are plotted in Figure 3.30. OSPANEL reduces the cost of the panel mainly by thinning the thickness of the steel sheet and narrowing the width of the web flanges. On

the other hand, S_1 and S_3 are obtained by reducing the total thickness as well as the steel sheet thickness.

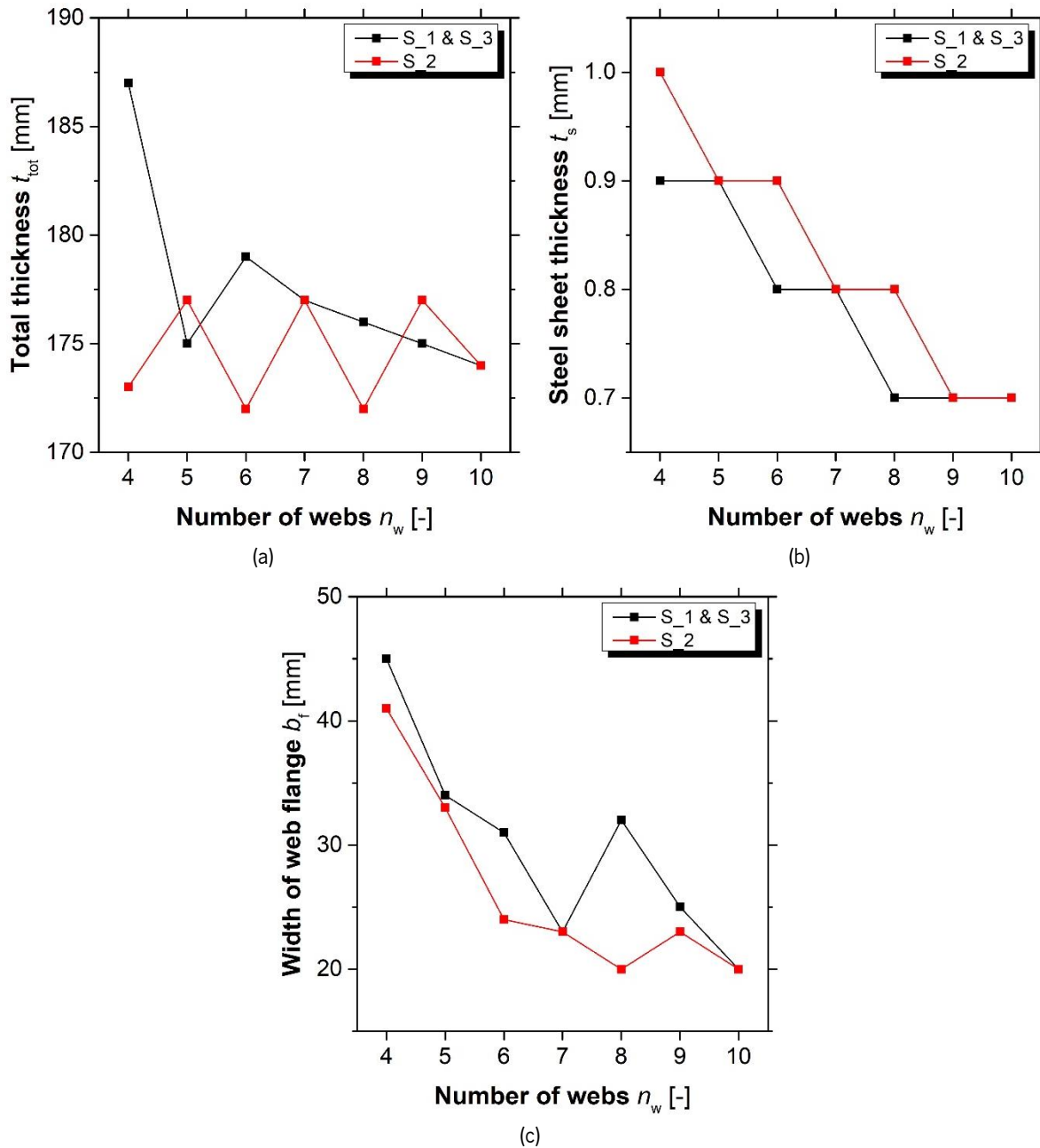


Figure 3.30. Values of the design variables of the optimal solutions: (a) t_{tot} ; (b) t_s ; (c) b_f .

In the second stage, the influence of the cost of the adhesive is also investigated offsetting the original price by +10% and -20%. A decrease in the price of the adhesive makes the S_1 and S_3 solution with 4 webs also the most economical solution. An increase in the price provokes instead a narrowing of the flanges in the S_2 solution, thus diminishing the surface area to be bonded. At the same time, the total thickness of the panel is increased to compensate for the loss of the moment of inertia. This suggests that for an adhesive with a unitary cost equal to or less than approximately 8.5 €/kg there is no relevant

difference in the overall price for a sandwich panel with an adhesive or mechanical connection between its face sheets and webs.

Finally, the full sound insulation requirements are imposed in the adaptive penalty function to verify the possibility of using sandwich panels as separating elements of dwellings in new construction. The highest acoustic insulation is achieved when OSPANEL is set to search for the S_1 and S_3 solution. The sandwich panel fulfils 80% and 65% of the airborne and impact sound insulation requirements, respectively. OSPANEL conceive a much heavier sandwich panel compared to the previous one making use of most of the steel mass allowable by the box conditions and the highest density PUR foam (see Figure 3.31).

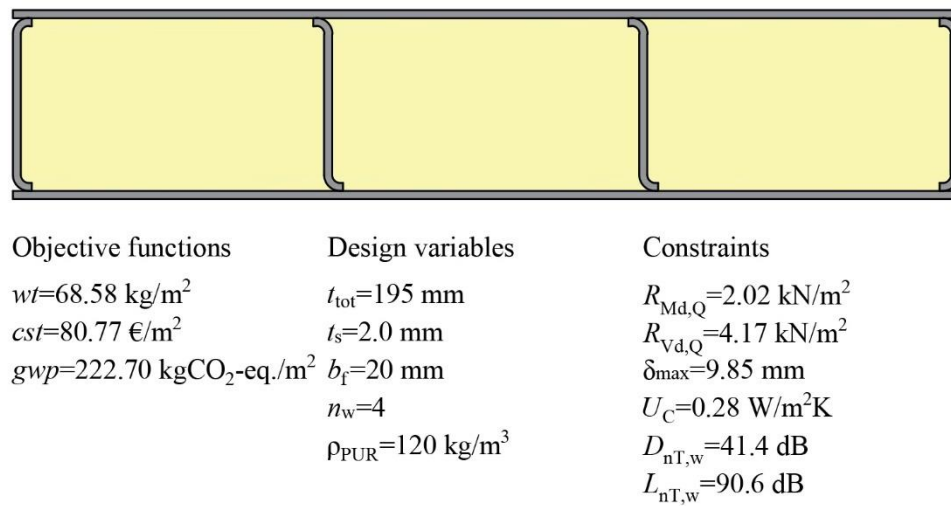


Figure 3.31. Most sound insulating sandwich panel retrieved by OSPANEL.

3.7.7. Discussion of the optimisation results with OSPANEL code

Based on the obtained solutions and the manufacturing constraints, the S_2 with 4 webs is selected as the optimal architecture for the sandwich panel to be produced in the scope of the Lightslab R&D Project. The nominal dimensions are modified to conform to more convenient and traditional sizes of floor and CFS profiles, i.e. the total thickness and the web flange width are set equal to 170 mm and 45 mm, respectively.

The lightest sandwich panels coincide with the least polluting as the carbon footprint is directly proportional to the mass of the sandwich panel. The difference between the S_1 and S_2 solutions is the use of the adhesive bonding connection. The least expensive sandwich panels have a reduced width of the web flanges to reduce the cost of the adhesive bonded area.

The plot of the constraint functions shows that the issue of the low acoustic insulation of the sandwich panel shall be addressed in view of their application as a residential floor system. Additionally, the thickness of the face sheet and the webs shall be considered as separate design variables to improve the efficiency of the design with respect to bending moment and shear resistances.

Disregarding the manufacturing constraint imposed by Ferpainel production line, optimal solutions with a number of webs greater than 4 present a total thickness ranging between 170 mm and 190 mm and a steel sheet thickness exclusive of the coatings varying between 0.7 mm and 1.0 mm. In particular, the 7 webs architecture present a good compromise between all the objective functions. On the other hand, sandwich panel architectures with more than 8 webs are not advisable both for economical and thermal issues.

3.8. MANUFACTURING PROCESS AND PANELS' CONNECTION

The manufacturing process of the optimised prototype, i.e. S_2 with the appropriate changes to fit standard measures of flooring systems and CFS profiles, consisted of three stages. The challenges encountered in the first two manufacturing attempts eventually led to the final cross-section of the sandwich panel. Furthermore, the final prototype included in its design considerations related to the panel-to-panel connection system. In the next sections, a detailed description of the three manufacturing attempts is presented.

3.8.1. Manufacturing of prototype S_2

In the first manufacturing attempt made by Ferpainel, a modification was applied to the recommendations provided by the DEC team of the University of Minho. The builder team of Ferpainel opted for a spot weld connection between the bottom face sheet and the webs (see Figure 3.32). The bottom part of the sandwich panel was then manually positioned on the conveyor belt for the foaming process. The top face sheet-to-web connection was achieved by means of adhesive bonding. The DEP team of the University of Minho developed the polyurethane-based adhesive for such connection. The adhesive was prepared by the builder team at Ferpainel and applied on the top flanges of the webs as the sandwich panels advanced on the conveyor belt to the injection system of the monomer of the PUR foam. However, the bottom face sheet-to-web connection did not ensure the necessary stability of the webs during the manufacturing process. As a result, the manufactured panels highlighted rather large deviation from straightness of the

webs. Furthermore, the presence of the webs represented an obstacle for the correct foaming process of the PUR foam. This resulted in large voids not filled by the PUR foam in the cavities of the built-up CFS section (see Figure 3.32c).

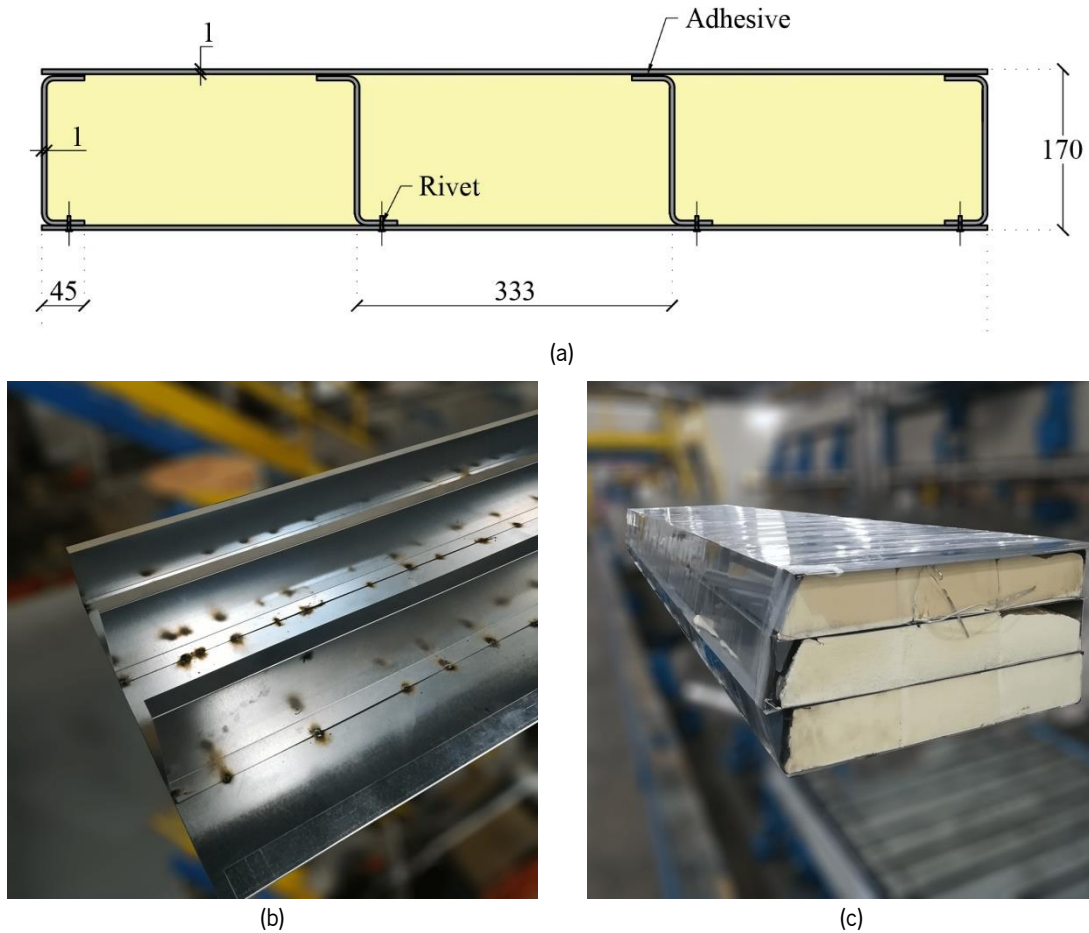
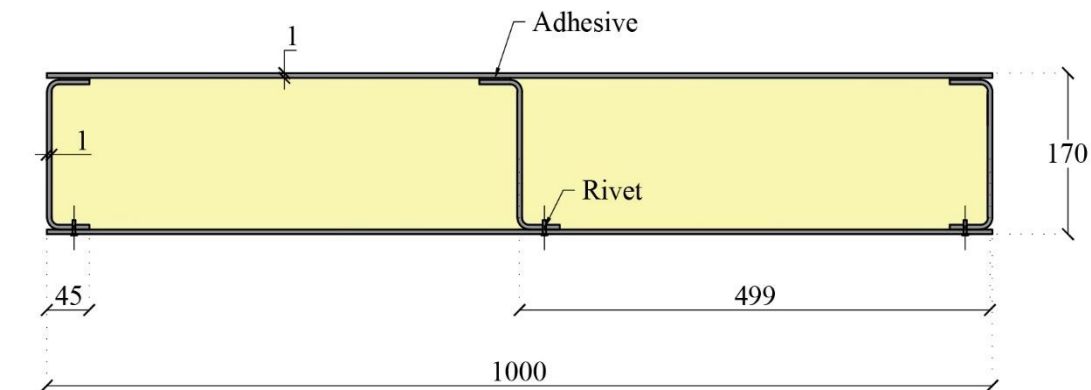


Figure 3.32. Prototype S_2: (a) Cross-section schematic; (b) bottom face sheet-to-web spot weld connection; (c) manufactured panels. All units in [mm].

3.8.2. Manufacturing of the intermediate prototype

In the second manufacturing attempt, the DEC team modified the cross-section of the optimised sandwich panel to mitigate the PUR foaming issue observed in the first attempt. A single Z-section internal web was envisaged in the new prototype to reduce the number of cavities to be filled by the PUR foam and facilitate the foaming process (see Figure 3.33). Nevertheless, the reduction in the number of Z-section webs resulted in a reduction of structural performance, which imposed a new limit on the maximum span of the new prototype, set equal to 4.0 m. A bracing system was also developed by the DEC team between the webs of the sandwich panel. The bracing system consisted of thin strips of CFS C-section riveted to

adjacent webs at a spacing of 1.0 m. However, due to the slenderness of such elements, the desired stabilising effect was not achieved (see Figure 3.33c).



(a)



(b)



(c)



(d)



(e)

Figure 3.33. Intermediate prototype: (a) cross-section schematic; (b) assembly of bottom face sheet and webs; (b) failed bracing system attempt between webs; (c) manufactured panels; (d) misalignment between the top face sheet and the web.

All units in [mm].

Furthermore, the builder team of Ferpainel followed the recommendations of the DEC team and used riveted connections for the bottom face sheet-to-web connection. The intermediate prototype showed that the foaming process took place successfully, thus, establishing an optimal width of the cavities between the webs of the sandwich panel of 0.5 m. Nevertheless, the issue of the straightness and initial imperfection in the webs remained unresolved, as highlighted in Figure 3.33e. The presence of initial

imperfections is particularly relevant in the case of CFS structural members, since they are susceptible to cross-sectional instabilities [64], [142].

3.8.3. Manufacturing of the final prototype

The cross-section of the final prototype may be visualised as a box beam constituted by the assemblage of a plain C-section and a lipped C-section oriented according to their weak bending axis (see Figure 3.34a). The width of the cross-section is 0.5 m since the intermediate prototype showed a correct foaming process for such a width of the cavities between the webs. The webs of the panel are constituted by the flanges of the stiffened C-section. The continuity of the bottom face sheet and the webs remove the problem of the cumbersome connection procedure and ensure improved stability of the latter elements during the manufacturing process. Furthermore, in the final prototype folds were added to the webs of the sandwich panel in view of their application as a modular flooring system. The panel-to-panel connection system developed will be explained in detail in Chapter 6, hereafter a brief description of the architecture is presented. The panel-to-panel connection is based on the tongue and groove joint commonly used in flooring system [143]. One edge of the panel is profiled in a way to provide a slot or a groove, whereas on the opposite edge the folds create a protruding element, i.e. the tongue. Two adjacent sandwich panels will closely fit together (see Figure 3.34b).

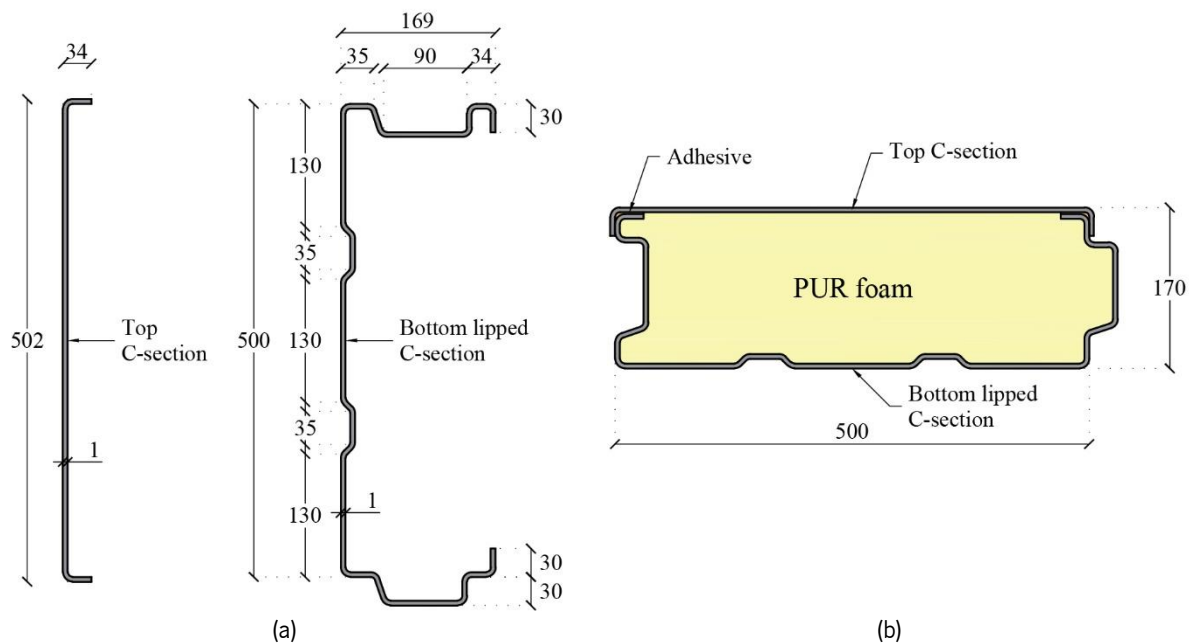


Figure 3.34. Final prototype layout: (a) CFS components cross-section; (b) sandwich panel cross-section. All units in [mm].

The first manufactured batch of the final prototype is illustrated in Figure 3.35. The sandwich panel presented a visually superior quality compared to the previous attempts, and, therefore, it was deemed adequate for further production of specimens for the experimental campaign.

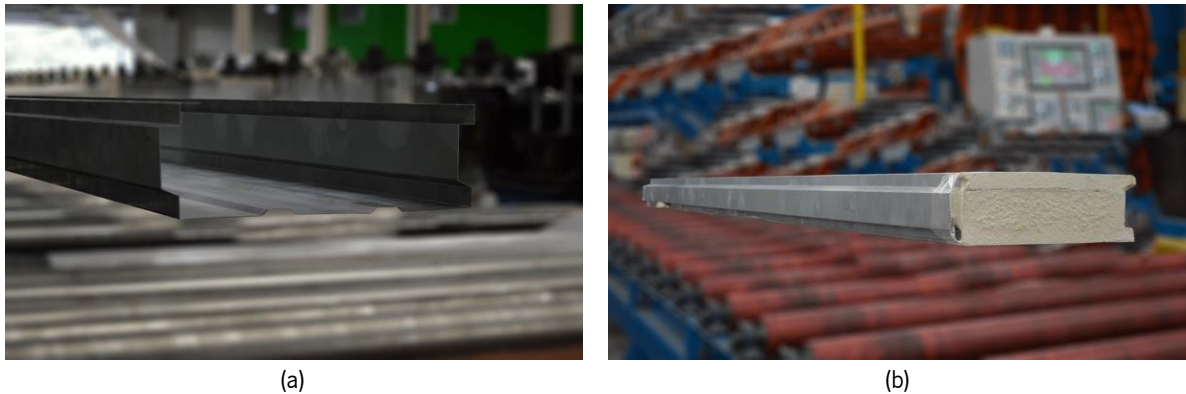


Figure 3.35. Final prototype: (a) lipped C-section on the conveyor belt; (b) manufactured panel.

3.9. CONCLUDING REMARKS

The aim of the work in this chapter was to develop and produce a cost-optimized sandwich panel that meets various requirements outlined in building codes, including structural safety, thermal and acoustic properties, and fire resistance.

Different sandwich panels architectures (with the different composing parts of different materials) were optimised by means of trial-and-error method, and their performance compared in terms of weight and cost of production. The sandwich panel constituted by CFS face sheet and webs, and a PUR foam core was selected for the next stage of the design process, i.e. the optimisation by means of genetic algorithms (GA). An original GA code was developed to retrieve designs minimizing weight, cost, and environmental footprint of the sandwich panel. The main feature of the GA procedure was described highlighting its versatility and potential for applications in current engineering design practice.

The most cost-effective solution retrieved by the GA code was selected as the final cross-section of the full-scale sandwich panel. The final prototype constitutes a relatively low environmental impact solution compared to its first design stage. Nevertheless, the use of insulation materials such as PUR foam with a high level of industrial processing represents a huge drawback in the achievement of low CO₂ emissions in construction systems [144]. A switch from traditional construction materials to wood-based product has been indicated as a solution that can lead to significant decrease in CO₂ emissions [145]. Since Portugal is a world leader producer of cork, with a 50% share of the worldwide raw cork production [146],

such natural material should be considered as an attractive alternative as core insulating material. Further benefits are added considering that the production of cork in Southern Europe is one of the most ecological production processes, involving extraction process from trees every 10 years [1]. The most significant figure is that the production of 1 kg of PUR foam produces almost 400 times the amount of CO₂ required to produce 1 kg of cork [147].

As the effect of climate change threaten not only human communities but life itself on Earth, private manufacturing companies participating in R&D Project must substitute their primary goal from cost-saving solutions to low environmental impact solutions, as the ultimate decision-maker power resides in them. At the same time, research teams from universities must be ready to stand their ground and challenge their partners as these issues cannot be longer overlooked.

For what concerns the manufacturing process, the production of sandwich foam technology helps to prevent dust and debris from affecting the quality of the face sheet-to-core interface. However, the manufacturing attempts mentioned earlier suggest that further research is required for the sandwich foaming process. It is crucial to ensure that the foaming is of high quality to guarantee that all the cavities of the sandwich panel's cross-section are filled and supported by the PUR foam core. The inclusion of the bottom face sheet and the webs in a single component improved the dimension tolerances of the final prototype. However, to profile the CFS sheets, modifications are required in the production line of Ferpainel. As the face sheets are unrolled from the coil, automated profiling equipment needs to be included. By ensuring quality control of the foaming process and including the profiling equipment, it is highly feasible to continuously produce the proposed sandwich panel.

CHAPTER 4

4. MATERIAL CHARACTERISATION AND SMALL-SCALE TESTS

This chapter addresses the material characterisation of the materials composing the sandwich panels and small-scale tests. As stated in the previous chapter, the analytical design of the sandwich panel considered only the mechanical properties of the face sheets and webs material. The sandwich panel manufacturing company provided different datasheets of hot dip galvanised steel coil purchased during the timeframe of the R&D Project. Generally, the mechanical characteristics reported in the datasheets satisfied the requirements for the lowest grade reported in the Eurocode 3 Part 1-3 (2006) [80], i.e. S220GD+Z. On the other hand, no technical specifications were available for the PUR foam core materials and the PUR-based adhesive developed by the DEP team for the face sheet-to-web connection.

Therefore, an experimental campaign was envisaged to obtain the values of the unknown material properties of the PUR foam and the PUR-based adhesive. Furthermore, the experimental programme was also required to verify the compliance of the mechanical properties of the steel coil with the technical datasheets. Finally, the experimental tests were carried out to investigate in detail the materials' behaviour up to failure, the failure modes, and the scatter in the properties. The stress-strain response of the constituent materials under different loading conditions can be later used to develop specific constitutive material models for the numerical analyses carried out in Chapter 7. Additionally, small-scale sandwich panels were manufactured to assess the integrity of the composite structure and provide a database to calibrate the numerical constitutive models.

The experimental campaign was carried out in the Structural Laboratory of Civil Engineering (LEST) facility of the University of Minho, and it included the following tests: i) coupon tensile tests of the steel sheet used as the face sheets and webs of the sandwich panel; ii) flatwise compressive and tensile tests and shear tests of the PUR foam core; iii) single lap shear tests of the PUR based adhesive; and iv) edgewise

compressive tests of small scale sandwich panel with steel face sheet and PUR foam core. Different PUR foam core densities (40 kg/m^3 , 50 kg/m^3 , and 60 kg/m^3) and steel sheet thicknesses (1.0 mm and 1.5 mm) were tested according to a preliminary study on the range of properties of interest for the application of sandwich panels in residential flooring systems. Thermogravimetric analyses (TGA) were also carried out to qualitatively assess the PUR foam microstructure. Indeed, information can be gathered about the main physical and chemical reactions needed to create a closed-cell microstructure by assessing foam degradation caused by temperature.

Civil engineering structures, particularly residential buildings, are designed to last 50 years. Polymeric materials used in the proposed sandwich panels, such as the PUR-based adhesive and the PUR foam, are prone to creep deformation under permanent loads [148], [149]. Shear stress-induced creep in PUR foam is less relevant because the CFS webs have greater shear stiffness and absorb most stress. On the other hand, the PUR-based adhesive transfers shear stresses from the plain CFS section to the lipped CFS section. Thus, the creep deformation and the effect of temperature on the adhesive's mechanical properties shall be assessed to ensure the integrity of the sandwich panel during its service life. However, obtaining experimental data on the creep behaviour and temperature's effect on the adhesive's mechanical properties is beyond the scope of this thesis and will be addressed in future works.

Finally, CFS specimens were extracted from the full-scale tested sandwich panels to perform additional tensile coupon tests. Especially during the Covid-19 pandemic, Ferpainel purchased steel sheets from multiple suppliers to save costs due to extreme fluctuations in raw material supply costs. Therefore, verifying the mechanical properties of the steel sheet with which the final prototypes were manufactured was deemed necessary. Furthermore, zinc coating thickness measurements by microscopical examination were carried out to establish the core thickness of the metal sheets.

4.1. COLD-FORMED STEEL

CFS specimens were laser cut out from the steel sheet used in the production of the steel insulating sandwich panels produced by Ferpainel and subsequently from the final manufactured prototypes. Specimens' dimensions were established based on the type of test to be carried out, i.e. tensile coupon tests and zinc coating thickness microscopical examinations.

4.1.1. Tensile coupon test

Tensile coupon tests were carried out according to ISO 6892-1 (2016) [150] on three sets of specimens. The first batch of specimens included two sets with a nominal thickness of 1.0 mm (TC#1-1.0) and 1.5 mm (TC#1-1.5), respectively. TC#1-1.0 and TC#1-1.5 were extracted from the steel sheet used in Ferpainel's line of steel insulating sandwich panels. The third set of specimens (TC#2-1.0) had a nominal thickness of 1.0 mm and were extracted from the full-scale panels described in Chapter 5. Therefore, sheets with different nominal dimensions (35 cm × 40 cm and 35 cm × 8 cm) were extracted out from the sandwich panels tested in a four-point bending configuration up to failure. Regions of low bending and shear stresses in the face sheets and webs of the panels were identified as the extraction zone for the sheets by numerically simulating the four-point bending test (see Chapter 7).

All the coupon specimens were extracted in the longitudinal direction of the steel sheets. The specimens were machined according to the specification of EN 10346 (2015) [151] and are illustrated in Figure 4.1a. Prior to testing, gauge marks were applied in the region of interest, and the cross-section dimensions of the specimens were measured with a digital calliper. The coupon tensile tests were carried out in a universal testing machine (UTM) with a capacity of 200 kN. The tensile load is applied by gripping the ends of the specimens with clamps. The axial strain was measured with an extensometer with a base length of 100 mm (precision of 0.01 mm) placed at the middle specimen height. The overall test setup is shown in Figure 4.1b.

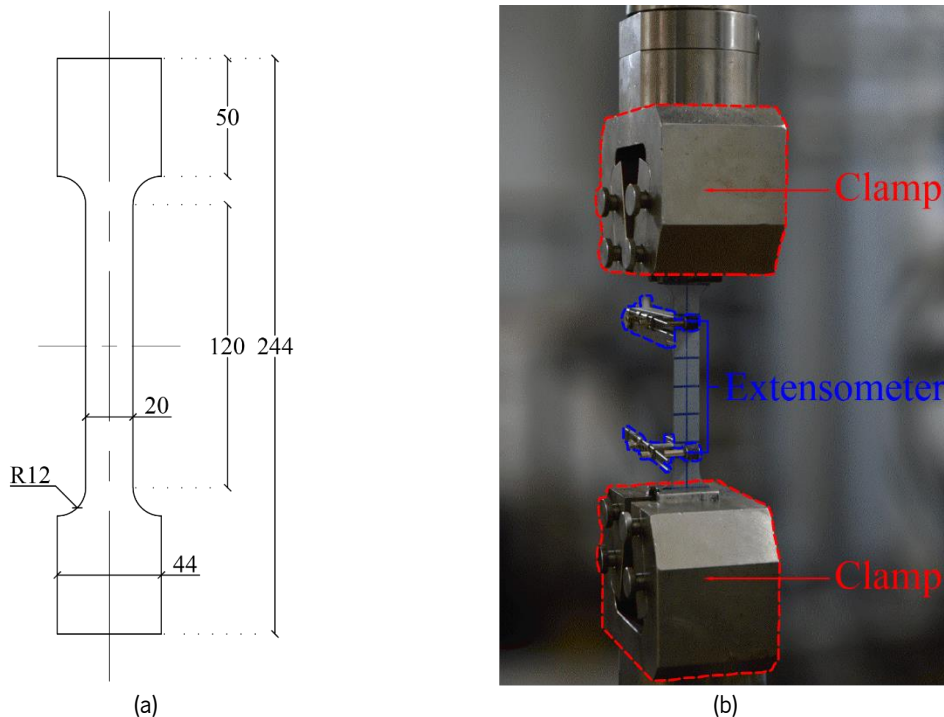


Figure 4.1. CFS tensile coupon test: (a) nominal dimensions of the specimen; (b) test setup. All units in [mm].

The coupon specimens were tested in a quasi-static monotonic-instantaneous loading up to failure under displacement control. The displacement-based protocol followed the suggestions of [152] to eliminate the influence of the strain rate on the test results. A displacement rate of 0.003 mm/min was applied until reaching the proportional limit. The displacement rate was then increased to 0.015 mm/min and kept constant until the yielding phase was completely developed. Finally, the displacement rate is increased to 0.066 mm/min until failure to keep the test duration within a reasonable time. The loading was paused for 100 s at different critical locations to obtain the static drops due to stress relaxation. The pauses were near the 0.2% proof stress, at the beginning of the hardening region, and near the ultimate strength.

As stated previously, despite the tensile coupon test being a highly standardised test procedure, errors may occur in the analysis and interpretation of the data. Therefore, to correctly estimate the properties of the CFS specimens, the influence of the loading rate had to be removed from the experimental stress-strain curves, i.e. the dynamic curves, according to the method described in [152]. The procedure to obtain the static material properties of the CFS coupons is the following (see Figure 4.2):

1. The Young's modulus (E) is calculated as the slope of the initial linear portion of the stress-strain curve in the range between 20% and 45% of the nominal proof strength ($\sigma_{0.2}$);
2. A straight line is drawn from the origin of the axis with the slope of the estimated E . The proportional limit (σ_p) is established at the point of separation of the straight line from the stress-strain curve;
3. A line parallel to the linear portion is drawn at a distance from it equivalent to 0.2% strain. The intersection between the line and the stress-strain curve represents the experimental value of $\sigma_{0.2}$;
4. The first part of the stress-strain curve from the origin to σ_p is left unchanged. The second part of the curve from σ_p to $\sigma_{0.2}$ is determined by reducing the stress of a quantity in proportion with the strains at $\sigma_p(\varepsilon_p)$ and at $\sigma_{0.2}(\varepsilon_{0.2})$ according to the following Equation:

$$\sigma_s(x) = \sigma_d(x) - \frac{\Delta_0(\varepsilon_x - \varepsilon_p)}{\varepsilon_{0.2} - \varepsilon_p} \quad (4.1)$$

where Δ_0 is the first static drop due to stress relaxation, and $\sigma_s(x)$ and $\sigma_d(x)$ are the static and dynamic stresses corresponding to the strain ε_x , respectively;

5. The part of the curve between the intermediate static drops is calculated according to Equation (4.2):

$$\sigma_s(x) = \sigma_d(x) - (\Delta_{i-1} - \Delta_i) \cdot \frac{(\varepsilon_x - \varepsilon_{i-1})}{\varepsilon_i - \varepsilon_{i-1}} \quad (4.2)$$

where Δ_i and Δ_{i-1} are the intermediate static drops due to stress relaxation, and ε_i and ε_{i-1} are the strains corresponding to the intermediate static drops;

6. The last part of the curve is obtained by reducing the stress of the amount corresponding to the last static drop (Δ_u).

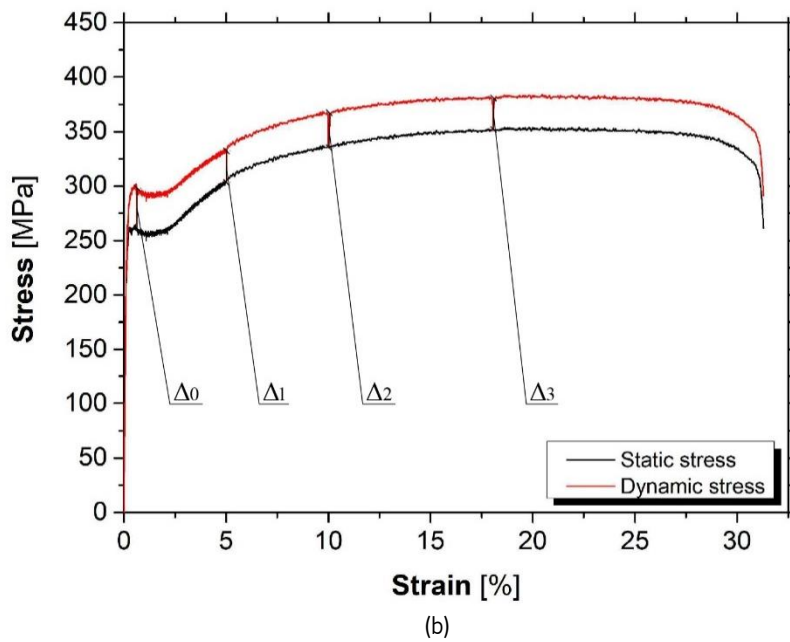
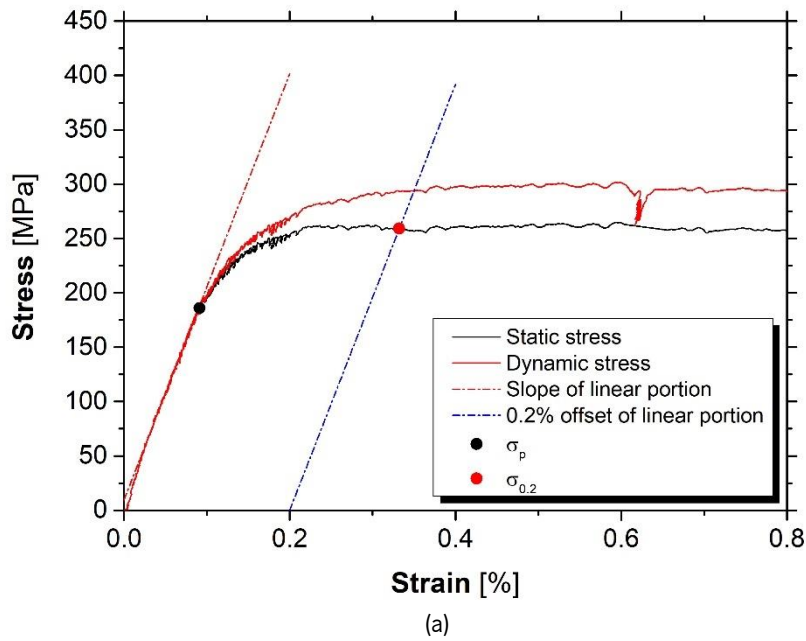


Figure 4.2. Procedure to obtain static material properties: (a) proportional limit and 0.2% proof strength; (b) dynamic stress and static stress-strain curves.

The static engineering stress-strain curves (see Figure 4.3a) indicate that all sets of specimens present a pronounced yielding phenomenon [151]. A failed specimen exhibiting the typical necking phenomenon before fracture is shown in Figure 4.3b.

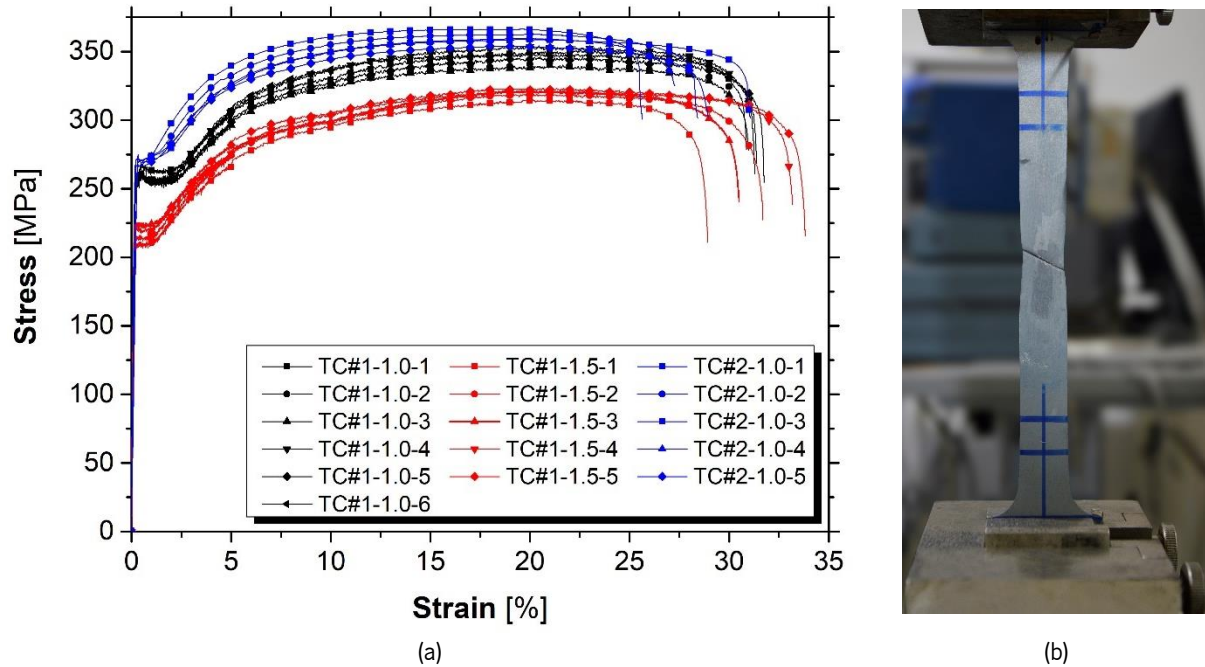


Figure 4.3. Uniaxial coupon tensile test result: (a) stress-strain curves; (b) typical failure mode.

The mechanical properties obtained from the tests are reported in Table 4.1, including the Young's modulus (E), the proof strength ($\sigma_{0.2}$), the ultimate tensile strength (σ_u), and the strain at fracture (ε_f). $\sigma_{0.2}$ is the value of the yield strength (σ_y) to be used in the structural safety verifications [80]. The three sets of specimens presented similar levels of ductility. Sets TC#1-1.0 and TC#2-1.0 satisfy the requirements of the steel grade S250GD+Z. A relatively small difference (less than 5%) in terms of $\sigma_{0.2}$ and σ_u is observed between the steel used in the production of Ferpaine steel insulated sandwich panels and that used to manufacture the final sandwich panel prototype. On the other hand, the set TC#1-1.5 does not meet the minimum requirements of the steel grade S220GD+Z in terms of yielding strength. Nevertheless, the values reported in the technical datasheet correspond to the static yielding strength. The mechanical properties reported in the technical datasheets usually refer to the dynamic values established according to the strain rates imposed by the standard [81]. A statistical analysis of the TC-1.0 and TC-1.5 sets of data using the interquartile range (IQR) method was carried out to detect outliers [153]. The elastic modulus of specimen TC#1-1.5-1 was below the lower inner fence and, thus, identified as an outlier. This may be due to the presence of out-of-straightness defects in the specimen resulting in the introduction of bending moments and a non-uniform state of tensile stresses.

Table 4.1. Summary of the mechanical properties of the CFS sheets.

Specimen [-]	$\sigma_{0.2}$ [MPa]	σ_u [MPa]	ϵ_f [%]	E [GPa]
TC#1-1.0-1	261	351	30.87	190
TC#1-1.0-2	253	342	31.54	222
TC#1-1.0-3	253	340	30.97	201
TC#1-1.0-4	259	354	31.29	194
TC#1-1.0-5	258	346	31.75	209
TC#1-1.0-6	254	349	31.39	214
Avg. (CoV)	256 (1.3%)	347 (1.6%)	31.30 (1.1%)	205 (5.9%)
TC#1-1.5-1	210	315	28.93	166*
TC#1-1.5-2	210	319	31.69	198
TC#1-1.5-3	225	321	30.51	203
TC#1-1.5-4	215	321	33.19	190
TC#1-1.5-5	219	323	33.83	187
Avg. (CoV)	216 (3.0%)	320 (1.0%)	31.63 (6.3%)	194 (3.8%)
TC#2-1.0-1	266.7	366.4	25.6	209
TC#2-1.0-2	271.4	362.9	28.4	208
TC#2-1.0-3	269.0	359.1	31.1	205
TC#2-1.0-4	270.8	360.0	27.3	209
TC#2-1.0-5	266.6	354.2	29.0	201
Avg. (CoV)	268.9 (0.8%)	360.5 (1.3%)	28.3 (7.1%)	206 (2.0%)

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation; the highlighted outliers with the symbol “*” were not considered in the average calculation.

4.1.2. Zinc coating thickness by microscopical examination

Hot-dip galvanisation involves submerging a steel element in a bath of molten zinc. The procedure is most used to provide long-term corrosion protection to the steel for up to 50 years [154]. Since the datasheets provided by Ferpainel reported different coating masses of the zinc coating, a microscopical examination of the CFS sheet cross-section was carried out to estimate the coating thickness. Three specimens were taken from the same sheets described in Section 4.1.1. A thin layer of PUR foam was present on the inner side of the sheets (see Figure 4.4a). Therefore, before cutting the samples, the sheets were immersed in a Dimethylformamide bath, allowing the PUR foam to soak the compound. The PUR foam was then carefully removed from the sheet using a spatula. Finally, the sheets were first rinsed with light petroleum and, subsequently, distilled water. The dimensions of the specimens were 1 cm × 1 cm. The samples were mounted in epoxy resin, and the examined cross-section was polished with diamond paste particles of 1 µm according to the standard ASTM B487-85 (2013) [155] (see Figure 4.4b).

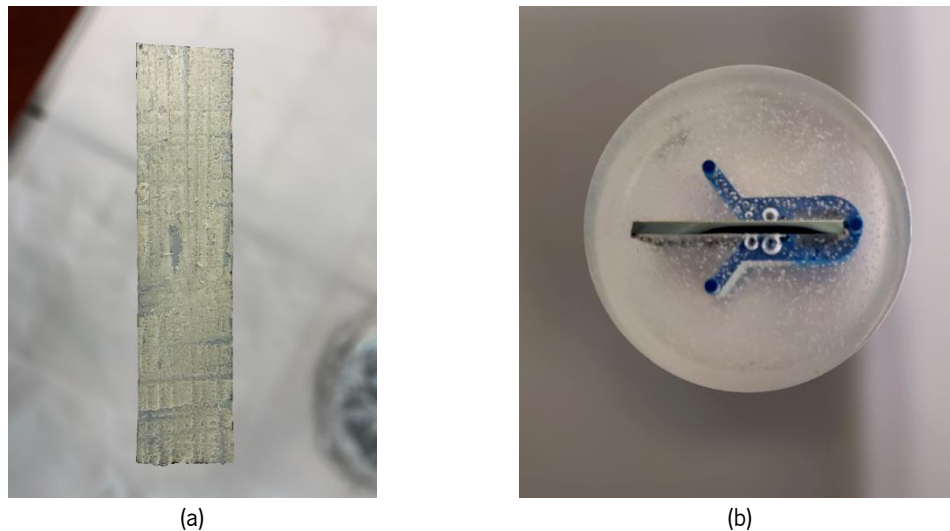


Figure 4.4. Specimen preparation for zinc coating thickness measurements: (a) CFS sheet cut out from the full-scale tested sandwich panel; (b) specimen cast in epoxy resin.

Digital pictures were taken using an optical microscope to capture the cross-section regions of each sample (see Figure 4.5). The thickness of the coating (t_z) was measured by employing the ImageJ software. Five measurements were carried out on both coated surfaces of each specimen. The results of the zinc coating thickness by microscopical examination are summarised in Table 4.2.

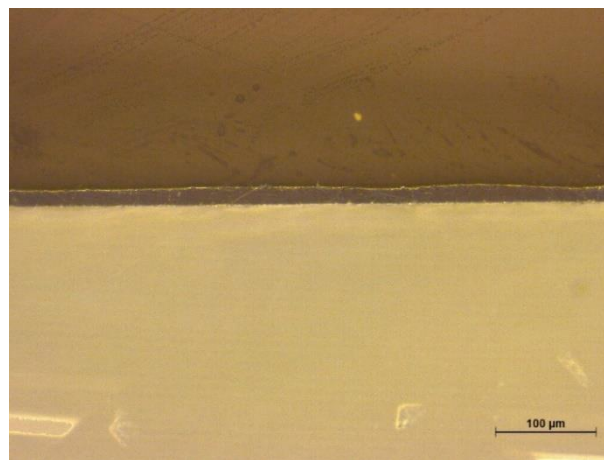


Figure 4.5. Optical microscope image from one of the samples.

Table 4.2. Thickness of the zinc coating on the steel sheet of the full-scale sandwich panel.

Specimen [-]	t_z [μm]										Avg. (CoV)
	#1	#2	#3	#4	#5	#6	#7	#8	#9	#10	
TZ01	17.1	15.2	17.1	16.0	19.3	18.7	17.0	15.4	14.7	17.5	17.7 (9.7%)
TZ02	18.8	19.0	19.3	19.9	19.3	19.1	20.0	20.2	19.4	17.6	18.4 (8.4%)
TZ03	18.8	20.3	18.8	15.6	18.0	17.2	18.4	16.0	17.6	17.6	17.6 (11.8%)

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation.

The average thickness of the zinc coating of the CFS sheets is 17.9 μm . The coating mass (m_z) may then be estimated according to the Equation provided in EN 10346 (2015) [151]:

$$m_z = 2 \cdot d_z \cdot t_z \quad (4.3)$$

where d_z is the density of the zinc and t_z is the thickness of the zinc coating. The estimated zinc coating mass is 247.8 g/m² considering a zinc density of 7.1 g/cm³ [151]. The CFS sheet coating mass is conservatively designated as the coating mass Z225 whose typical thickness is 16 μm . The typical zinc coating thickness value is used to calculate the core thickness (t_{cor}) of the steel as prescribed by the Eurocode 3 Part 1-3 (2006) and discussed further ahead in Chapter 7.

4.2. POLYURETHANE FOAM AND ADHESIVE

The objectives of the rigid closed-cell PUR foam employed as core material in the proposed sandwich panel are twofold: i) provide adequate thermal insulation; and ii) improve the resistance of the CFS sheets to local buckling phenomena. In this sense, it is interesting to characterize its compressive and tensile mechanical properties in the direction perpendicular to the face sheets. This is generally different from sandwich panel without webs, where the most relevant mechanical properties of the core materials concern its shear behaviour.

The macroscopic mechanical properties of PUR foams depend on its microstructure. Indeed, PUR foams are cellular materials, the man's attempt of reproducing materials that nature uses to build load-bearing structures, i.e. wood, bones, and coral [156]. Cellular materials are characterised by a porous microstructure comprising of solid and voids network. In the case of polymeric foams, the voids are called cells, and they are filled with gas. The solid are the faces surrounding the cells constituted by the polymer [157].

The microstructure of the PUR foam is a complicated process resulting from the interaction of different chemical reactions. The synthesis of polyurethane consists in the reaction of isocyanate groups and polyols to form urethane linkages. Simultaneously, physical (water) or chemical blowing agents (the infamous chlorofluorocarbon banned by the Montreal Protocol (1987) [158] and the slightly less harming hydrochlorofluorocarbons [159]) generate the gas to form the foamed structures. Urethane and urea linkages, resulting from the addition of diamines, react with the isocyanate groups to form allophanate and biuret linkages, respectively. These linkages are responsible for the branching and crosslinking of the

polymer. The gel point is the start of the crosslinking reaction which result in significantly increasing viscosity. The period after the gel point is defined as the curing period which generally involves heating. An imbalance between the polymerization-crosslinking and the blowing processes compromises the successful synthesis of the foamed product [160]. If gelation point is reached too slowly in foamed products, the foam structure may collapse or present imperfections which yield poor mechanical strength and performance.

The mechanical properties of PUR foam are influenced, among others, by mixture components, substrates, and production technologies [161]. In the proposed sandwich panel, the PUR foam is produced by the sandwich foaming process [157]. The liquid prepolymer (an incompletely polymerized polymer) is poured onto the bottom face sheet. As polymerization and crosslinking occur, an adhesive bonding is provided between the face sheets and the expanding PUR foam. The PUR foam is constrained from expanding laterally by the webs and by the material behind and in front of it on the face sheet. Therefore, the foaming process causes the material to rise vertically, resulting in an anisotropic cell morphology [31].

The influence of cell structure on macroscopic properties has been investigated through visual inspection and mechanical testing. Scanning electron microscope pictures are used to correlate the size, shape, and orientation of foam cells to the foam's strength and stiffness. It was found that foams with cells with a larger aspect ratio in the rise direction, obtained from the preparation of a 100 kg/m^3 PUR foam in narrow aluminium moulds, exhibited higher compressive stiffness and strength compared to the same foam prepared in wider moulds [162]. Additionally, cell orientation divergence from the rise direction was observed and attributed to the temperature gradient between the reacting polymer and the room temperature walls of the moulds. Furthermore, the tensile and compressive moduli of a 45 kg/m^3 PUR foam used to fill CFS C-section were found to be 3.5 and 1.5 times higher, on average, in the rise direction, respectively [33]. Once again, the study confirmed that the superior properties resulted from the stretching and alignment of the cells in the rise direction.

When different production technologies and testing methodologies are used, it may be more challenging to make a straightforward comparison. For example, when samples extracted from full-scale steel insulating sandwich panels were tested in flatwise compressive and tensile tests, they showed strength values 3.25 times higher in the longitudinal direction of the panel, which is perpendicular to the rise direction of the foam [163]. Additionally, the study indicates that 5.1% of the variation in the results of the

mechanical tests can be attributed to the heterogeneity resulting from the sandwich foam production technology.

In the process of sandwich foaming, the polymerization and crosslinking reaction occurring in contact with the metal face sheets constitute an interface possessing its own mechanical characteristics. The mechanical properties of the interface are rarely well-characterised. To the best of the author's knowledge, only one study in the literature attempted to characterise the galvanized steel-to-PUR foam interface in terms of critical strain release energy rate, maximum normal and tangential strengths, and normal and tangential displacements at maximum strength and failure for Mode I and II fracture [73]. Nevertheless, the estimated experimental properties present coefficient of variation of up to 90%. The difficulties arise both from the involved test setup required, i.e. a double cantilever beam test, as well as the variability of the properties of the PUR foam and the interface itself.

In this brief literature review of rigid closed-cell PUR foam used as filler of CFS cross-section, it is interesting to highlight the significant number of factors that can influence the macroscopic mechanical properties of the core material. The complete characterisation would require the assessment of the chemical composition and cell morphology in relation to the mechanical properties. However, due to the timeframe of the LightSlab R&D Project, the envisaged experimental campaign aimed at retrieving the main mechanical properties of the PUR foam required by commercial finite element software to reproduce numerically the behaviour of the foam filled sandwich panel: i) flatwise compressive and tensile tests; and ii) shear tests. Additionally, TGAs were carried out to assess the decomposition processes of the foam due to temperature.

4.2.1. Flatwise compressive and tensile tests

Flatwise compressive and tensile specimens were carried out according to ASTM C297 (2004) [164] and ASTM C365 (2003) [165], respectively, on three sets of PUR foam specimens with a nominal density of 40 kg/m³ (FC-40 and FT-40), 50 kg/m³ (FC-50 and FT-50), and 60 kg/m³ (FC-60 and FT-60). The mechanical properties studied are those perpendicular to the face sheets of the sandwich panels, which are particularly relevant in the prediction of local instability phenomenon such as wrinkling. Furthermore, depending on the observed failure mode it is possible to assess either the bulk mechanical properties of the core materials or those of the PUR foam-to-face sheet interface. The tests were performed on cubic specimens with 60 mm sides. The specimens were cut out from larger sandwich panels with a core thickness of 60 mm and face sheet thickness of 1.0 mm. Before testing, the specimens' dimensions

were measured, and their weight was recorded with a scale. Based on that information, the estimated densities of the PUR foam are 39.6 kg/m^3 , 46.9 kg/m^3 and 52.6 kg/m^3 , and their relative differences with the nominal densities were -1.0%, -6.7% and -12.3%.

The tests were carried out in a UTM under displacement control, and the applied load was recorded by a load cell with a capacity of 25 kN ($\pm 0.005 \text{ kN}$). The vertical displacements were measured by four linear variable differential transformers (LVDTs) installed on each face of the cubic specimen to account for possible rotations deriving from the misalignment of the loading plate [166]. The setup pieces accommodating the LVDTs were glued to the face sheets according to the following procedure: i) both the surfaces of the setup pieces and the face sheets of the specimens were polished with sandpaper and cleaned with acetone; ii) a PUR-based two components adhesive was mixed and applied to the setup pieces and the face sheets of the specimens; iii) the coupons and setup pieces were assembled, and the adhesive was cured for 2 h at 40°C and then for at least 12 h, at room temperature, before the test. Regarding the flatwise compressive test, spherical washers were placed between the specimen assembly and the loading plate to ensure that only axial loading was applied to the PUR foam (see Figure 4.6a).

In the case of the flatwise tensile test, 3D bearing joints were screwed to the setup pieces and connected to the frame and hydraulic actuator to avoid bending moments in the system (see Figure 4.6b). Furthermore, a speckle pattern was applied to one of the specimens' faces for selected specimens to gain insight into the mechanical behaviour of the PUR foam. Digital image correlation (DIC) was employed to monitor the evolution of strains over the selected faces.

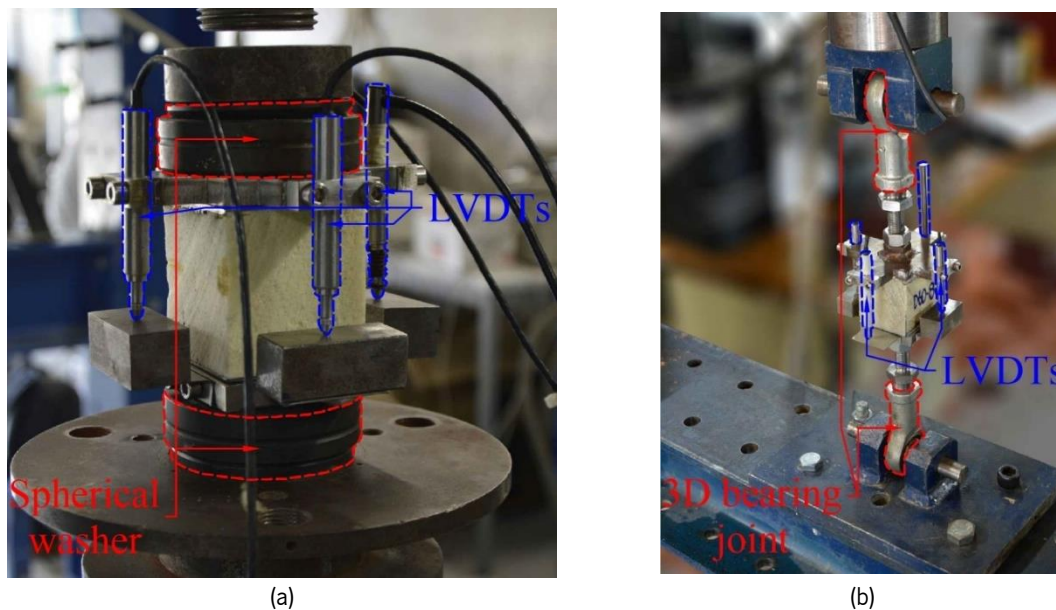


Figure 4.6. Mechanical characterisation of the PUR foam: (a) flatwise compressive test setup; (b) flatwise tensile test setup.

The flatwise compressive test was carried out at a speed of 2.4 mm/min, and the flatwise tensile test at 0.5 mm/min. Preliminary tests showed no significant differences between the two testing speeds. Therefore, the compressive tests were conducted at 2.4 mm/min to keep the test duration reasonable.

The stress-strain curves of the flatwise compressive tests are shown in Figure 4.7. The stress-strain curves of set FC-40 present three distinct regions: i) a linear elastic branch for low levels of strains; ii) a plateau where deformation occurs at a constant level of stress due to the buckling of the cell walls of the PUR foam; iii) a hardening stage involving the densification of the material as a result of the cell walls crushing [167]. On the other hand, the linear stage of sets FC-50 and FC-60 is followed by a hardening branch.

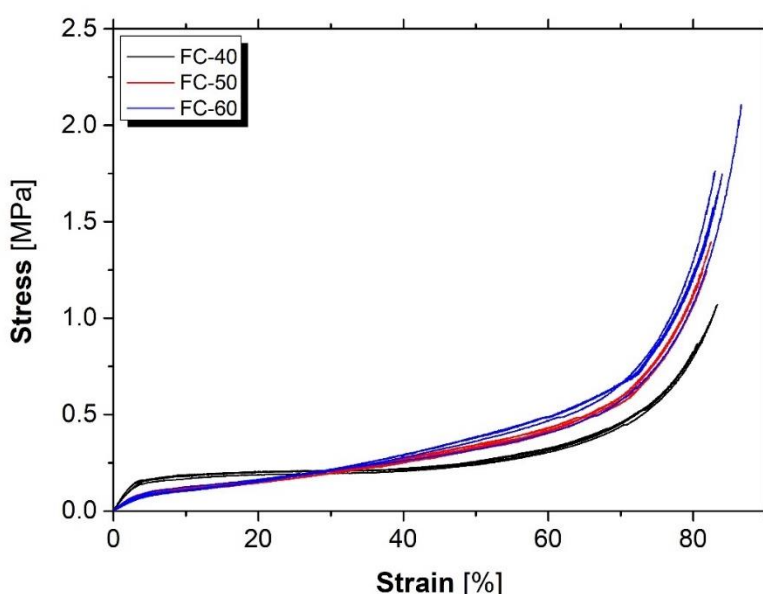


Figure 4.7. Flatwise compressive test: stress-strain curves.

Furthermore, the average compressive elastic modulus and yield strength of set FC-50 and FC-60 is lower than that of set FC-40 (see Table 4.3). This is an odd result considering that the increase in density corresponds to increasing mechanical properties of the PUR foam [22]. However, it is worth noting that the sets FC-50 and FC-60 present higher relative differences from the target nominal density. This may indicate that the synthesis of the polyurethane was not successful resulting in a microstructure with imperfections. The presence of thicker cell walls is attributed to the absence of the plateau region, since the thickness of the walls inhibits buckling and the consequent nonlinear elastic deformation [33]. Furthermore, thick cell wall was reported to be the cause of poor mechanical properties in the layers of the PUR foams studied by [162].

Table 4.3. Compressive properties of the PUR foam.

Specimen	Density [kg/m ³]	E_c [MPa]	$\sigma_{c,y}$ [MPa]	$\epsilon_{c,y}$ [%]
FC-40-1	40.6	6.14	0.15	2.50
FC-40-2	40.8	6.23	0.15	2.38
FC-40-3	37.5	5.38	0.15	2.84
FC-40-4	41.8	5.16	0.14	2.67
FC-40-5	39.1	5.99	0.15	2.49
Avg. (CoV)	40.0 (4.3%)	5.78 (8.3%)	0.15 (4.3%)	2.57 (7.0%)
FC-50-1	47.3	3.23	0.08	2.57
FC-50-2	45.6	3.47	0.09	2.61
FC-50-3	47.6	3.07	0.09	3.12
FC-50-4	49.4	3.50	0.09	2.64
FC-50-5	47.8	2.89	0.08	2.78
Avg. (CoV)	47.5 (2.9%)	3.23 (8.0%)	0.09 (6.9%)	2.75 (8.2%)
FC-60-1	50.8	3.48	0.08	2.35
FC-60-2	55.1	2.77	0.07	2.70
FC-60-3	52.4	2.20	0.07	3.09
FC-60-4	50.7	2.92	0.09	3.30
FC-60-5	54.2	2.76	0.07	2.56
Avg. (CoV)	52.7 (3.8%)	2.83 (16.2%)	0.08 (13.7%)	2.80 (13.9%)

Notes: Avg. – average results; CoV – coefficient of variation; E_c – compressive elastic modulus; $\sigma_{c,y}$ – compressive yield strength; $\epsilon_{c,y}$ – compressive yield strain

For what concerns the tensile stress-strain curves (see Figure 4.8), a linear behaviour is observed up to failure, which consisted in the cohesive rupture of the bulk PUR foam. Similarly to what has been observed for the compressive elastic modulus, the average tensile elastic modulus of FT-40 is higher than those of FT-50 and FT-60 (see Table 4.4). Additionally, FT-50 and FT-60 present higher extensibility at fracture. This may suggest that a low level of crosslinking was achieved during the synthesis of the PUR [160]. The large scatter found in the tensile properties of the PUR foam is within the range of the coefficients of variation reported in other studies [72], [162]. The production technology and the mould's size and shape significantly influence the foamed product's microstructure, which in turn determines the macroscopic mechanical properties of the PUR foam [160]. The extraction location of the specimen alone can contribute to a 5% scatter in the results [163]. Furthermore, defects such as cracks and the presence of large bubbles in the foam may lead to stress concentration in the cell walls and premature failure in the case of tensile tests [156].

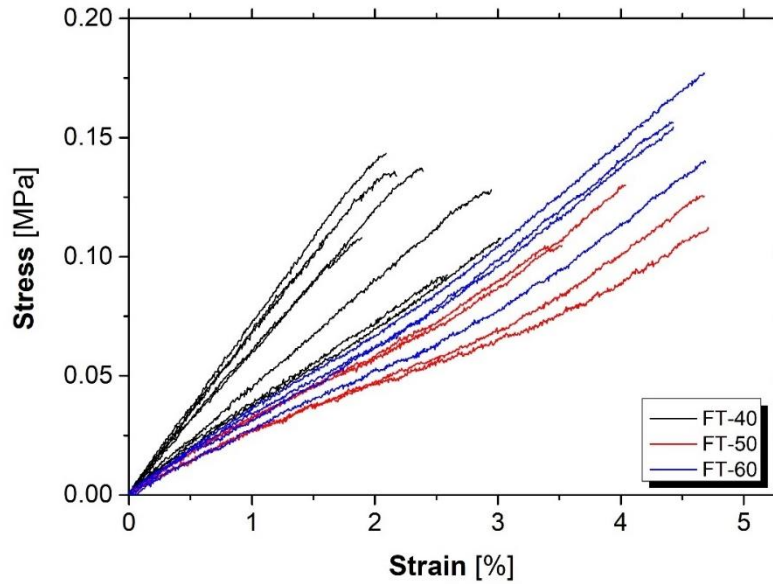


Figure 4.8. Flatwise tensile test: stress-strain curves.

Table 4.4. Tensile properties of the PUR foam.

Specimen	Density [kg/m ³]	E_t [MPa]	$\sigma_{t,u}$ [MPa]	$\epsilon_{t,u}$ [%]
FT-40-1	40.3	4.35	0.13	2.95
FT-40-2	38.0	4.09	0.09	2.59
FT-40-3	38.7	3.65	0.11	3.02
FT-40-4	38.9	7.07	0.14	2.09
FT-40-5	38.9	6.29	0.11	1.89
FT-40-6	40.5	5.89	0.14	2.40
FT-40-7	37.6	6.84	0.11	1.60
FT-40-8	40.7	6.85	0.14	2.18
Avg. (CoV)	39.2 (3.0%)	5.63 (24.6%)	0.12 (15.6%)	2.34 (21.2%)
FT-50-1	45.8	2.51	0.11	4.71
FT-50-2	45.1	2.55	0.13	4.68
FT-50-3	46.4	2.77	0.12	4.49
FT-50-4	46.6	3.23	0.13	4.04
FT-50-5	47.3	3.23	0.10	3.52
Avg. (CoV)	46.2 (1.8%)	2.86 (12.3%)	0.12 (8.6%)	4.29 (11.8%)
FT-60-1	50.9	3.15	0.16	4.42
FT-60-2	50.7	3.58	0.18	4.68
FT-60-3	53.0	3.31	0.15	4.43
FT-60-4	55.3	2.81	0.14	4.69
Avg. (CoV)	52.5 (4.1%)	3.21 (10.0%)	0.16 (9.7%)	4.55 (3.3%)

Notes: Avg. – average results; CoV – coefficient of variation; E_t – tensile elastic modulus; $\sigma_{t,u}$ – ultimate tensile strength; $\epsilon_{t,u}$ – ultimate tensile strain

The DIC technique can provide further insights into the mechanical behaviour of the PUR foam. The tracked area was divided into three layers, namely the top (TL), middle (ML), and bottom layers (BL) (see Figure 4.9). The average compressive and tensile strains over the whole tracked area (AVG-DIC), and over

the three layers are computed by comparing successive images using the DIC and compared to the measurements obtained from the LDVTs (AVG-LVDTs).

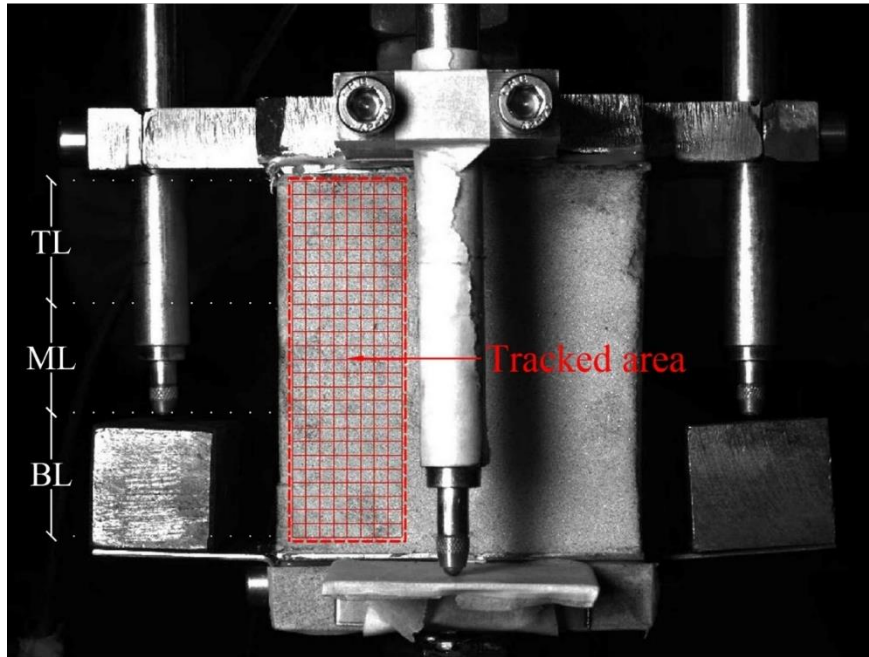
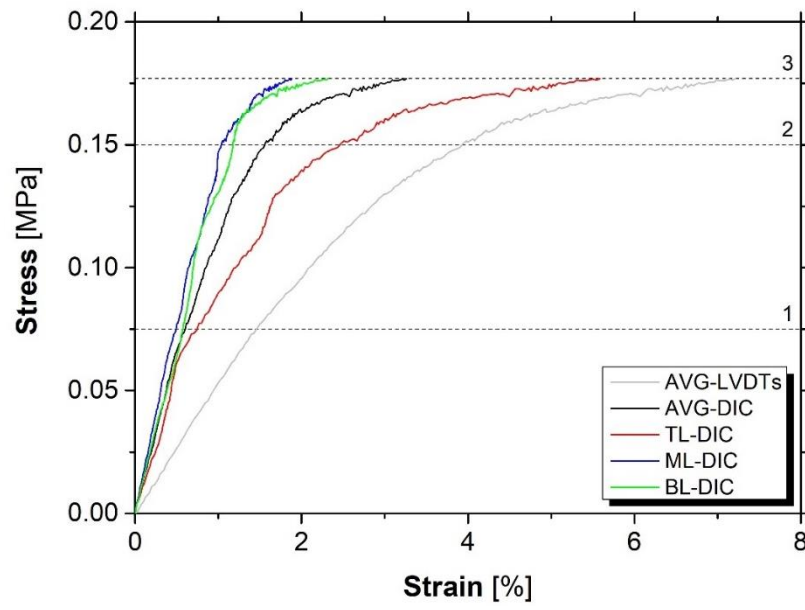
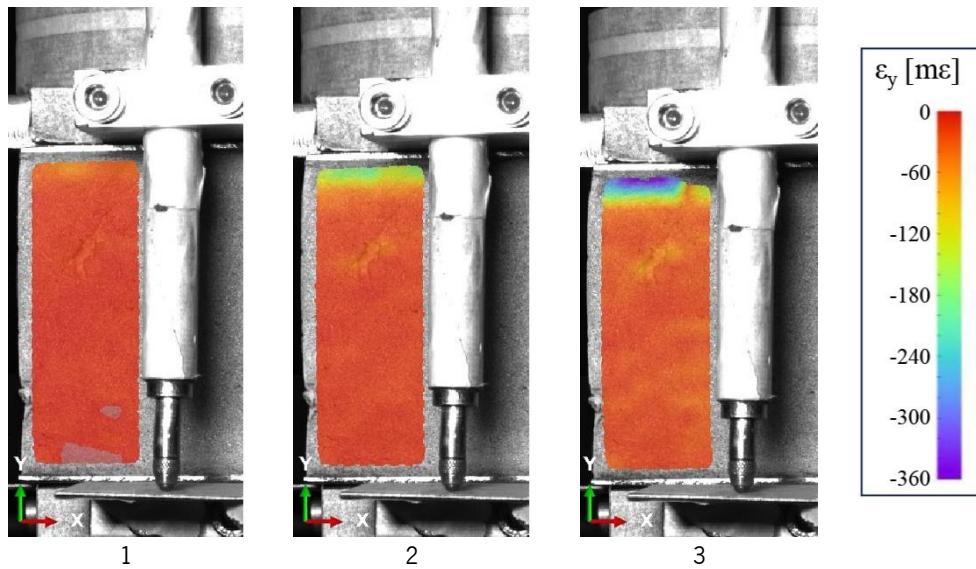


Figure 4.9. Area of the PUR foam specimen tracked by the DIC equipment with the corresponding layers.

Typical stress-strain curves obtained from the DIC recordings of one of the FC-40 are plotted in Figure 4.10a, along with the images captured by the DIC at different stress levels (see Figure 4.10b). A significant difference was noticed between the DIC and LVDT measurements during the flatwise compressive test. The reason for this is that the DIC was unable to track the speckle pattern near the interface PUR foam-to-CFS sheet. This area experiences a significant amount of compressive deformation, resulting in the formation of a compressive band. However, the DIC recordings do not take this deformation into account. As a result, the AVG-DIC curve appears to be stiffer than the AVG-LVDTs curve, which is not accurate. In contrast, the results of the flatwise tensile test show a good agreement between the measurements taken by the DIC and LVDTs (see Figure 4.11a). The area of stress concentration is accurately located within the DIC-tracked region (see Figure 4.11b). The data consistently indicates that the TL in the 40 kg/m³ PUR foam used in the steel insulated sandwich panel is less rigid compared to the ML and BL. This may be due to the cell orientation near the top face sheet, which may not align with the direction of the applied load, causing a decrease in mechanical properties [162].

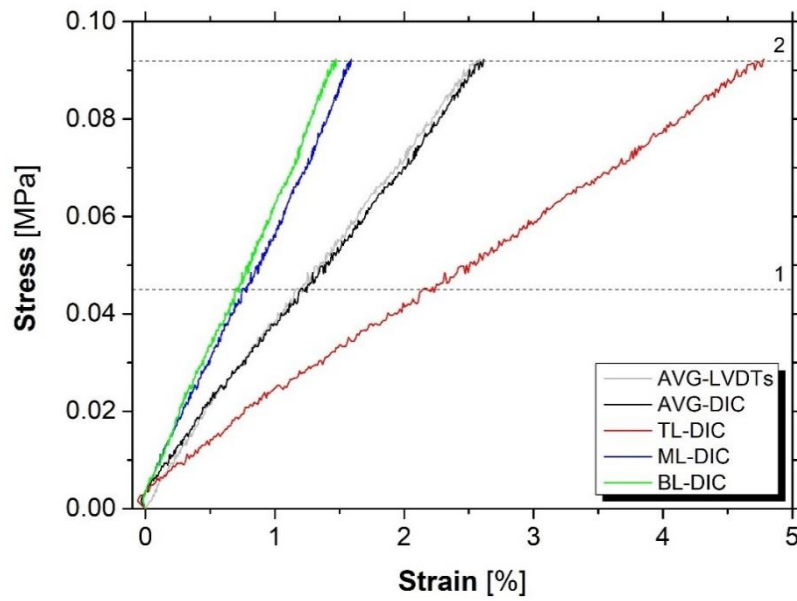


(a)

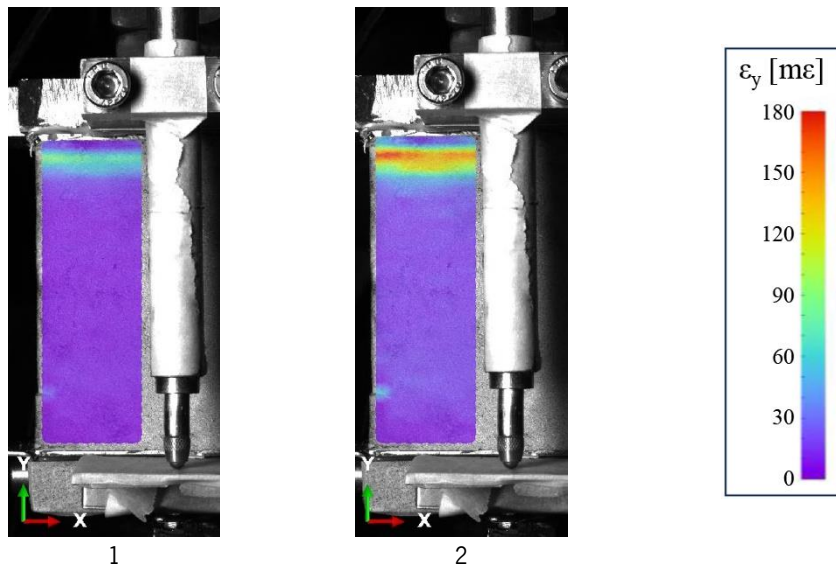


(b)

Figure 4.10. DIC measurements of FC-40 under compression: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial compressive strain at the linear stage, at the onset of yielding and after yielding. All units in $[\text{m}\epsilon]$.



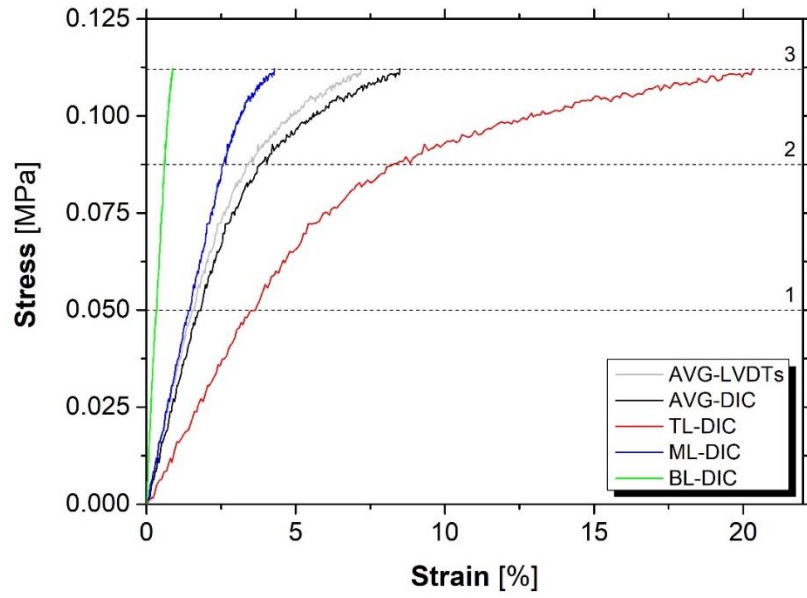
(a)



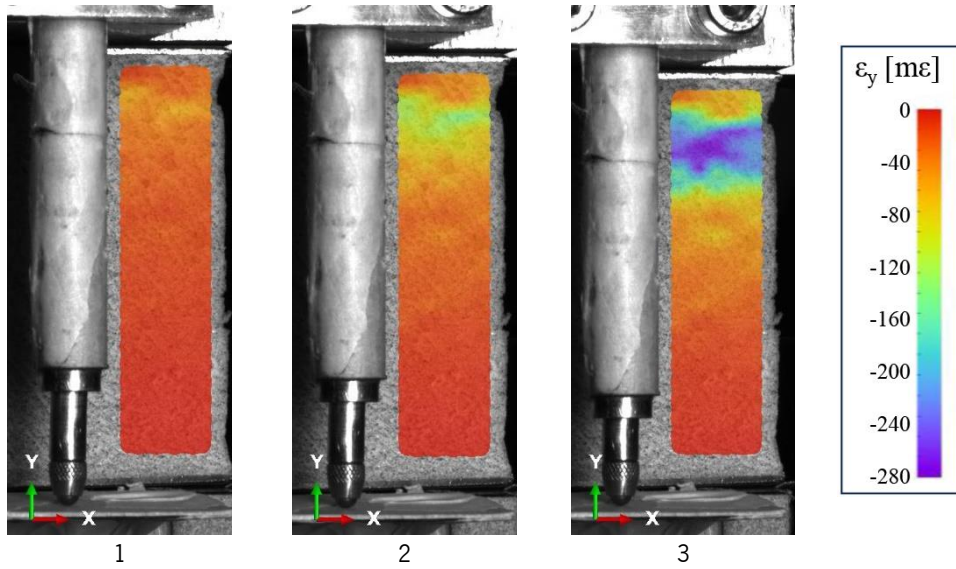
(b)

Figure 4.11. DIC measurements of FT-40 under tension: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial tensile strain at the linear stage and at fracture. All units in [mε].

In Figure 4.12a, the stress-strain curves for the tracked area of one of the FC-50 specimens using DIC are displayed. The compressive axial strains obtained from the LVDTs and the DIC show good agreement. The three layers exhibit different stiffnesses, with BL remaining in a linear stage after yielding. Plastic deformations are mainly observed in ML and TL. In Figure 4.13a, the tensile stress-strain curves computed from the LVDTs and the DIC images show negligible discrepancies. The tensile elastic modulus of BL is the highest, as observed in the compressive behaviour. ML and TL exhibit a small hardening branch before the specimen fails. This may indicate the presence of a non-optimized microstructure with thick wall cells possibly exhibiting plastic deformation before ultimate failure.



(a)



(b)

Figure 4.12. DIC measurements of FC-50 under compression: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial compressive strain at the linear stage, at the onset of yielding and after yielding. All units in [mε].

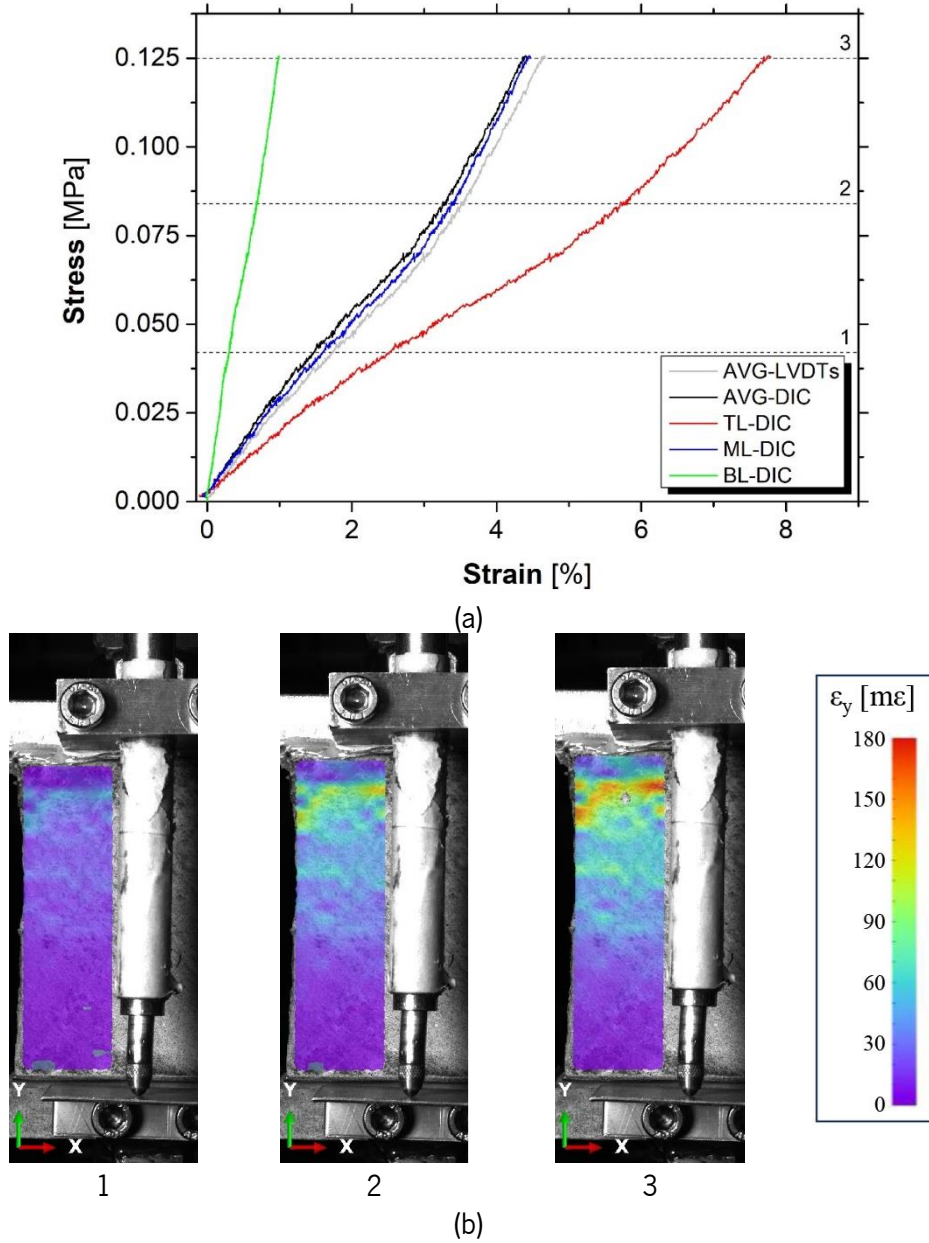


Figure 4.13. DIC measurements of FT-50 under tension: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial tensile strain at the linear stage, at the onset of the hardening stage and fracture. All units in [mε].

The stress-strain curves of a typical specimen from the FC-60 set, which were computed using DIC tracking, are displayed in Figure 4.14a. The axial compressive strains measured by LVDTs agree well with the DIC recordings. TL and ML experience plastic deformation at yielding, while BL behaves elastically during the recorded time interval. However, there is a notable difference between the tensile curves of AVG-DIC and AVG-LVDTs (see Figure 4.15a). This could be due to non-uniform contrast in the speckle pattern, which causes the analysis software to fail in correlating the area of high-stress concentration. BL is stiffer than TL and ML, and ML exhibits a hardening branch before tensile failure, indicating the presence of imperfections in its microstructure (see Figure 4.13b).

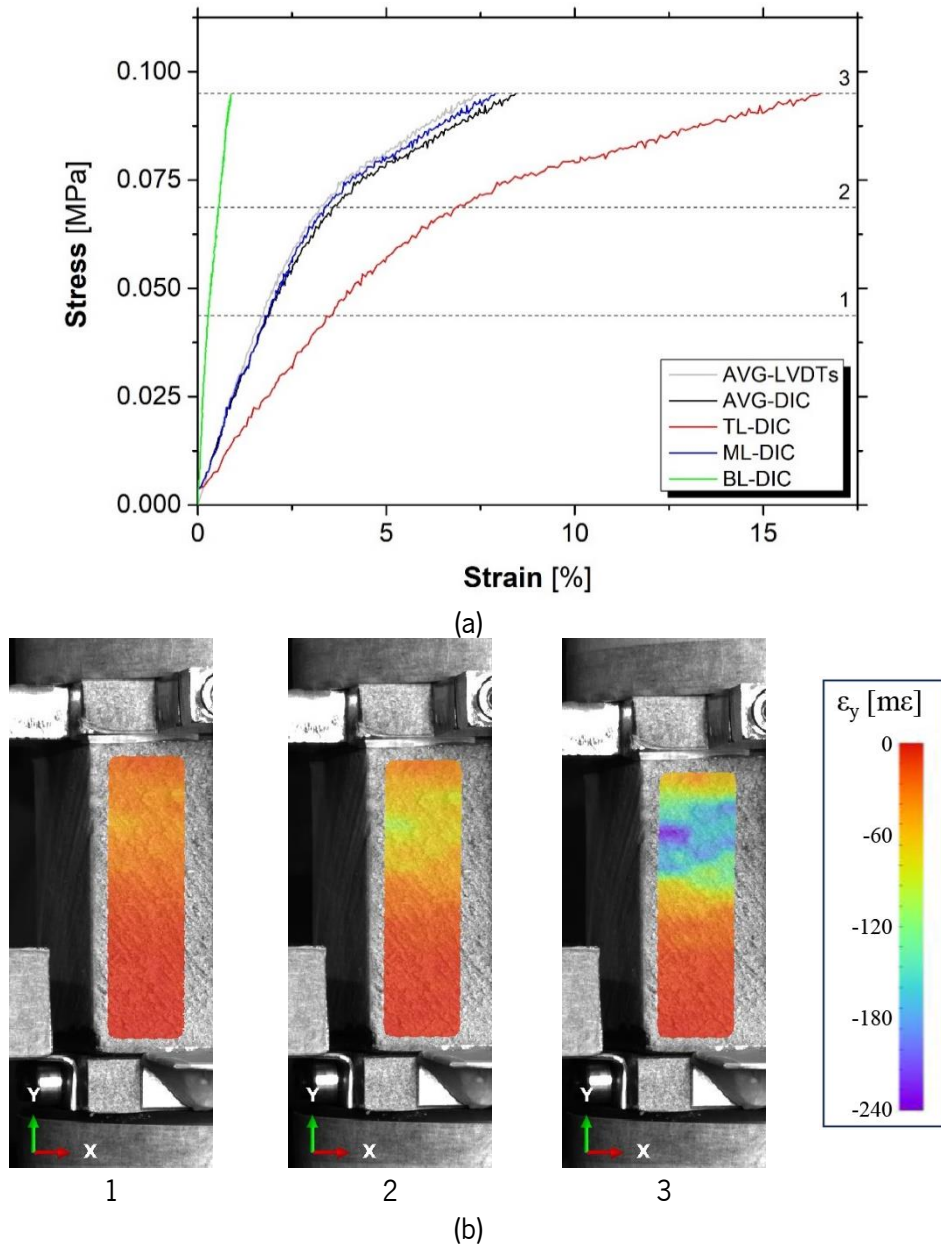


Figure 4.14. DIC measurements of FC-60 under compression: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial compressive strain at the linear stage, at the onset of yielding and after yielding. All units in $m\epsilon$.

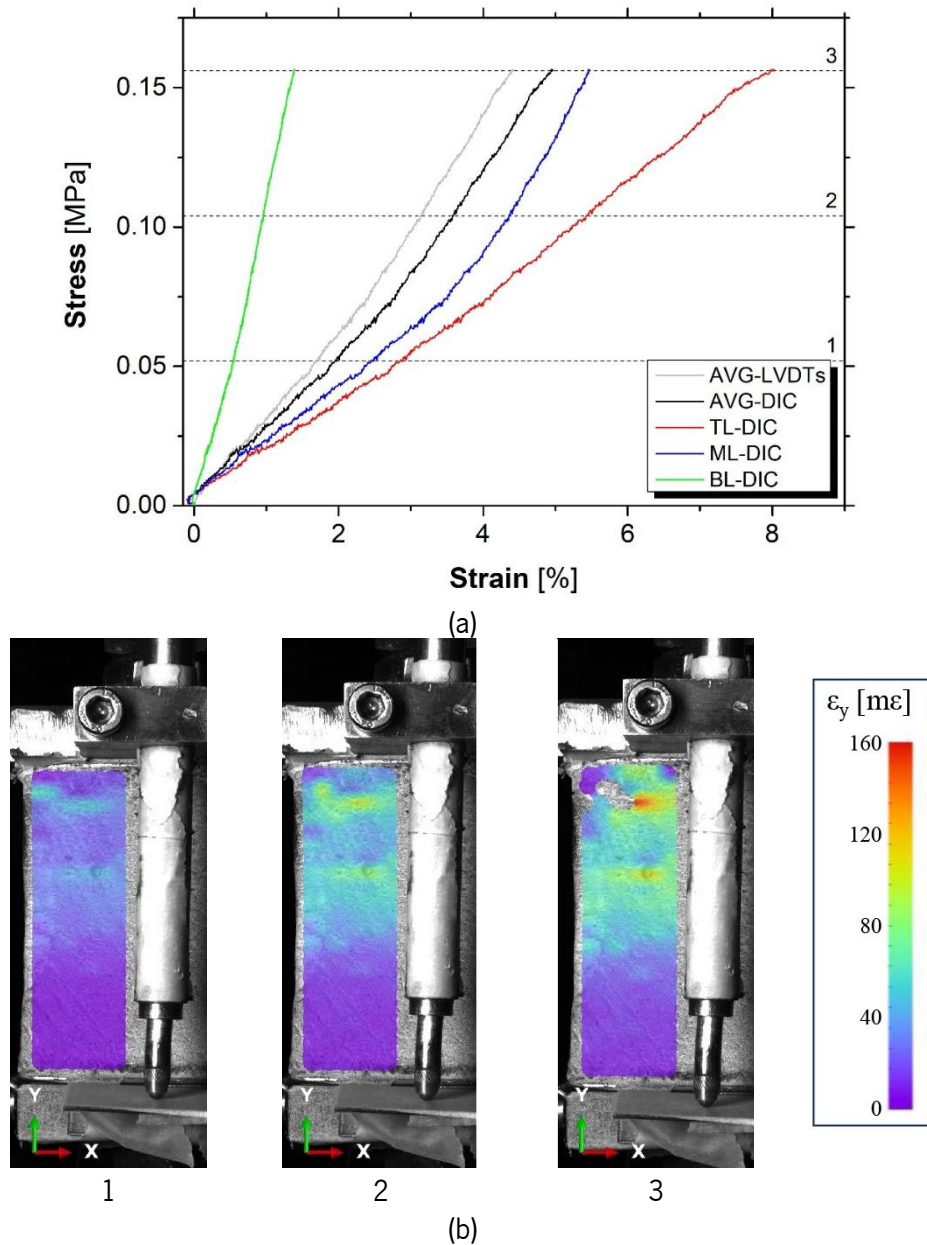


Figure 4.15. DIC measurements of FT-60 under tension: (a) stress-strain curve; (b) images extracted from the DIC recordings with the plot of the axial tensile strain at the linear stage, at the onset of the hardening stage and fracture. All units in [mε].

According to the analysis of the strain field using the DIC technique, there is a notable difference in the compressive and tensile mechanical properties between the 40 kg/m³ PUR foam and the 50 kg/m³ and 60 kg/m³ PUR foams. This difference is due to significantly different mechanical behaviour. The 40 kg/m³ PUR foam has weaker TL, while ML and BL have similar elastic properties. The 50 kg/m³ and 60 kg/m³ PUR foam have similar mechanical properties. There is an increasing gradient in the elastic properties from BL to TL that is easily recognizable. Additionally, the nonlinear tensile behaviour may suggest issues during the synthesis of the PUR foam.

4.2.2. Shear test

The difference in elastic properties between the CFS webs' shear modulus and the PUR foam core's shear modulus is significant. Therefore, the foam core material is not expected to have a significant influence on the overall shear performance of the sandwich panel. Nevertheless, it is worth noting that the mechanical properties of the bulk PUR foam material or the PUR foam-to-face sheet interface can be estimated based on the observed failure mode. This information can be utilised in the calibration of the constitutive models presented further ahead in Chapter 7.

Five prismatic specimens with nominal dimensions of 360 mm × 115 mm × 30 mm were tested using the approach suggested in ASTM C273 (2016) [168]. The specimens were extracted from full-scale sandwich panels with a 1 mm thick steel face sheet in the longitudinal direction. The full-scale sandwich panel had a nominal PUR foam density of 40 kg/m³. As the flatwise compressive and tensile tests on the 50 kg/m³ and 60 kg/m³ PUR foam yielded unexpectedly low mechanical properties, and Ferpainel experienced difficulties in synthesizing these foams, only the 40 kg/m³ PUR foam was tested as it had mechanical properties similar to those reported in the literature. Nevertheless, it is worth noting that the shear specimens showed visible manufacturing defects, namely large voids, concentrated near the interface, as shown in Figure 4.16.

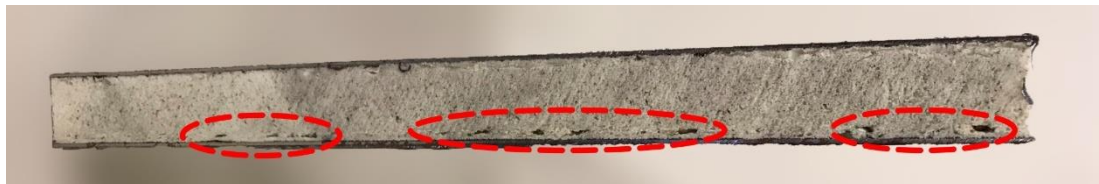


Figure 4.16. Shear specimens' defects: voids along one of the interfaces.

Before testing, the specimens' dimensions were measured, and their weight was recorded with a scale. Based on that information, the estimated average density was 36.4 kg/m³, and its relative difference with the nominal density was -9.0%. The specimens were bonded to steel L-shaped profiles made of two UNP 120 steel profiles with SikaForce 7710 L100, a two-component polyurethane-based adhesive. Before applying the adhesive, the surfaces of the steel pieces and the face sheets of the specimens were polished with sandpaper and cleaned with acetone. The adhesive was then cured for 24 h at room temperature before the test. Two 3D bearing joints were screwed to the L-shaped profiles to avoid the transmission of the bending moment through the supports (see Figure 4.17). A tensile load is applied, resulting in the plates moving in opposite directions. Thus, a transverse displacement of the planes parallel to the bonded face sheets of the PUR foam is imposed [169].

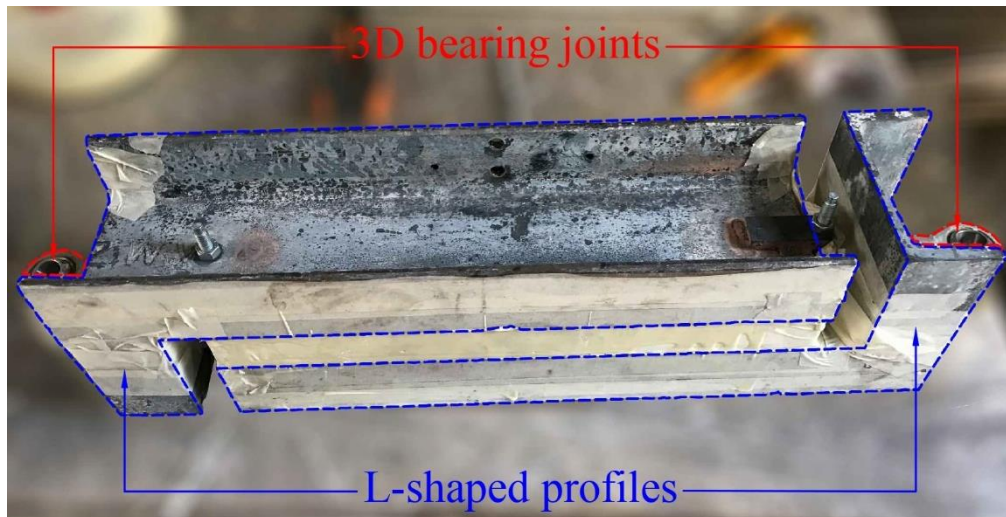


Figure 4.17. Test fixture for the assessment of PUR foam shear properties.

The shear tests were performed in a UTM, and the load was recorded by a load cell with a capacity of 200 kN (± 0.12 kN). The relative vertical displacement between the L-shaped profiles is measured by two LVDTs with a measuring stroke of 50 mm (± 0.12 mm). The overall test setup is illustrated in Figure 4.18. The specimens were tested under quasi-static loading up to failure, under displacement control at a rate of 0.5 mm/min.

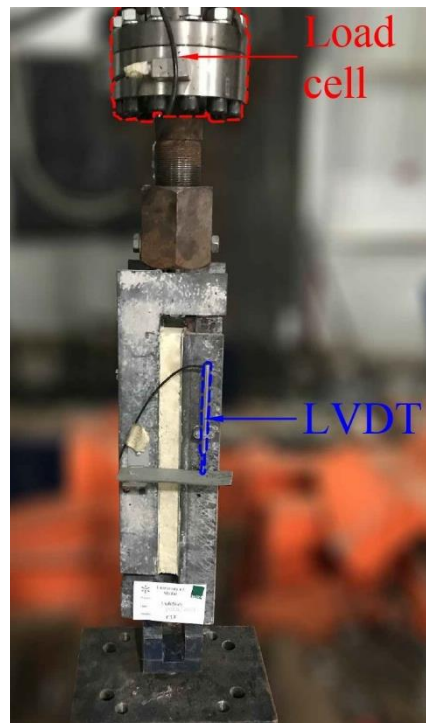


Figure 4.18. Shear test experimental setup.

The stress-strain curves for all the tested specimens tested are plotted in Figure 4.19. The shear strains are computed as the ratio between the relative displacement between the L-shaped profiles and the

thickness of the specimen. All the specimens presented an initial elastic stage followed by a hardening branch preceding the brittle failure. Two failure modes were identified based on tests performed on the five specimens: i) debonding at the PUR foam-to-face sheet interface (DI) (see Figure 4.20a); and ii) cohesive shear failure of the PUR foam (CS) with the formation of diagonal cracks (see Figure 4.20b).

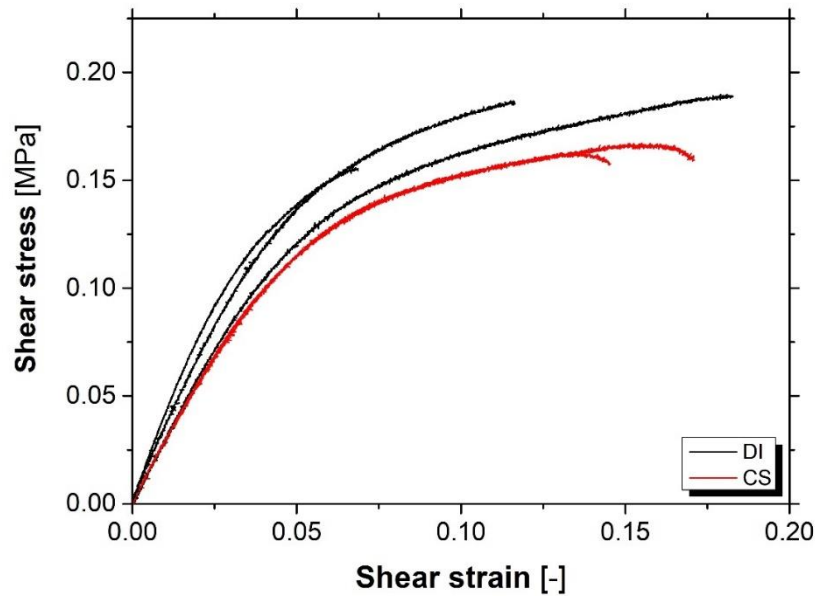


Figure 4.19. Stress-strain curves from the shear tests on PUR foam.

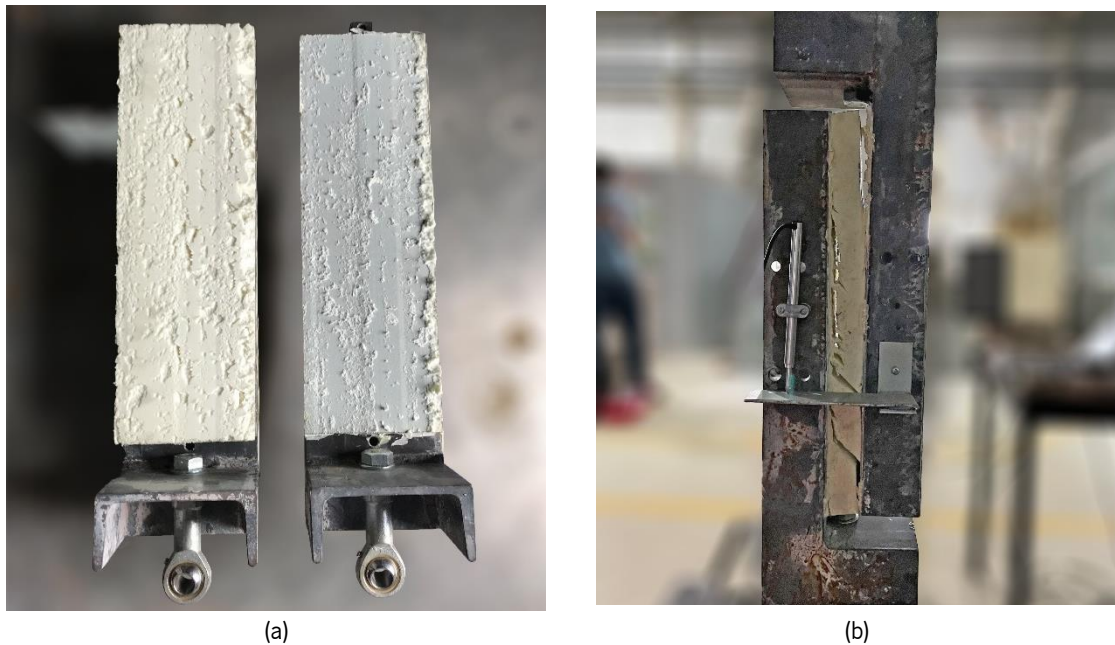


Figure 4.20. Failure modes identified in the shear tests: (a) debonding failure at the interface PUR foam-to-face sheet interface; (b) cohesive shear failure of the PUR foam.

The material properties of the PUR foam are summarised in Table 4.5. The shear modulus (G) is calculated as the slope of the initial part of the stress-strain curve (up to one-third of the shear yielding strength). The shear yield strength (τ_y) is estimated as the intersection of the extrapolation of the linear and hardening region in the stress-strain curves following the method described in [170]. The shear yield strain is the strain corresponding to the shear yielding strength (γ_y). Finally, in the table are included the ultimate shear strength (τ_u) and the corresponding ultimate strain (γ_u).

Table 4.5. Shear properties of the PUR foam.

Specimen	G [MPa]	τ_y [MPa]	γ_y [-]	τ_u [MPa]	γ_u [-]	Failure mode [-]
S01	2.53	0.14	0.055	0.17	0.155	CS
S02	2.69	0.15	0.054	0.19	0.179	DI
S03	3.77	0.14	0.037	0.16	0.067	DI
S04	2.62	0.14	0.052	0.16	0.131	CS
S05	3.11	0.16	0.050	0.19	0.115	DI
Avg. (CoV)	2.94 (17.5%)	0.15 (5.4%)	0.049 (15.0%)	0.17 (8.7%)	0.129 (32.8%)	All
Avg. CS	2.57	0.14	0.053	0.16	0.143	CS
Avg. DI (CoV)	3.19 (17.1%)	0.15 (6.1%)	0.047 (19.3%)	0.18 (10.5%)	0.120 (46.6%)	DI

Notes: Avg. – average results; CoV – coefficient of variation. Failure modes: DI – debonding at interface; CS – cohesive shear failure PUR foam.

The stress-strain curves of the specimens failed due to debonding present a larger scatter in terms of ultimate stress and strain. This may be due to the unstable propagation of cracks at the PUR foam-to-face sheet interface. It is also worth noting that the debonded specimens presented more noticeable defects, as depicted in Figure 4.16. The specimens that failed due to cohesive shear failure of the PUR foam presented lower mechanical strength than those that failed due to debonding. It may be argued that the specimens failed because the cohesive shear of the PUR foam presented a larger interface area, allowing for a uniform stress distribution through the thickness of the specimen.

According to theory of elasticity of homogeneous and isotropic materials [171], the Poisson's ratio (ν) of the PUR foam is given by Equation (4.4):

$$\nu = \frac{E - 2 \cdot G}{2 \cdot G} \quad (4.4)$$

According to the experimental data obtained from the flatwise compressive and tensile tests as well as the shear tests, ν of the PUR foam core material of the proposed sandwich panel present unusual values

of -0.02 and -0.04. Negative values of the Poisson's ratio are also reported for the PUR foam core material of the steel insulated sandwich panel investigated in [163]. The authors of the above-mentioned study suggests that the assumption of isotropy for PUR foams may not be correct. Furthermore, in the case of the PUR foam tested in this work, the influence of the heterogeneity of the PUR foam on the value of ν should also be considered. The difference of the estimated densities between the sets of specimens is approximately 8%. Additionally, the specimens for the flatwise compressive and tensile tests were extracted from sandwich panels with larger total thickness than those produced for the production of the shear specimens. This may also contribute to different cell microstructure in the PUR foams.

There is not a good agreement in the literature regarding the Poisson's ratio of PUR foams found in civil engineering applications as can be seen in Table 4.6. The comparison is also made difficult by the different production technologies employed to produce sandwich panels. According to [172], the definition of the Poisson's ratio of conventional PUR foams as a material constant is only valid for small strains. In their study, ν is estimated to be equal to 0.32 for small strains and approaching the value of 0.0 and 0.5 for large strain in compression and tension, respectively. In [173], the interval of Poisson's ratio for polymeric foams is estimated to be between 0.1 and 0.4. Even considering less stringent definitions, more than half of the estimated values in Table 4.6 fall outside the typical range. This may suggest that the PUR foam has anisotropic material behaviour due to the preferential evolution of the foam in the rise direction [72].

Table 4.6. Poisson's ratio of different PUR foams.

Reference	Production technology	Nominal density [kg/m ³]	E_c [MPa]	E_t [MPa]	G [MPa]	ν_c [-]	ν_t [-]
[36]	Slabstock foam	70	9.0 (± 1.0)	14.58 (± 1.0)	3.96* (± 0.37)	0.14	0.84
[174]	Slabstock foam	62	18.58 (± 0.24)	18.1 (± 0.457)	7.53 (± 0.01)	0.23	0.20
[33]	In-situ foam filling	45	12.44 (± 1.66)	19.0 (± 5.41)	4.46 (± 0.45)	0.39	1.13
[17]	Slabstock foam	100	26.8 (± 1.4)	31.3 (± 0.02)	8.7 (± 1.0)	0.54	0.80
[163]	Sandwich foaming	40	3.82 (± 0.44)	3.59 (± 0.17)	3.83 (± 0.09)	-0.50	-0.53
[18]	Slabstock foam	48	6.30 (± 0.57)		3.15 (± 0.38)	0.00	-1.00
LightSlab	Sandwich foaming	40	5.78 (± 0.48)	5.63 (± 1.39)	2.94 (± 0.52)	-0.02	-0.04

Notes: the values between parentheses are the corresponding standard deviation; ν_c – Poisson's ratio based on E_c ; ν_t – Poisson's ratio based on E_t ; the highlighted values the symbol “*” include debonding specimens in the average calculation.

4.2.3. Thermogravimetric analysis

The results of the flatwise compressive and tensile test highlighted the presence of layers with different mechanical behaviour. For this reason and to study the decomposition process underwent by the PUR foam subjected to temperature, TGA was performed on samples extracted from the TL, ML, and BL of each foam density (see Figure 4.21a). Each layer cut out from the cubic specimen was divided into 9 cubes. The sample for the TGA are extracted from the central cube (see Figure 4.21b) of each layer and the average weight is 2.2 mg. The tests were performed on a TA Instruments Q600 SDT (see Figure 4.21c), in an inert medium (N_2 atmosphere). TGA was conducted at a heating rate of $10\text{ }^{\circ}\text{C}/\text{min}$, from $30\text{ }^{\circ}\text{C}$ to $700\text{ }^{\circ}\text{C}$ and the flow rate of the gas was $100\text{ ml}/\text{min}$.

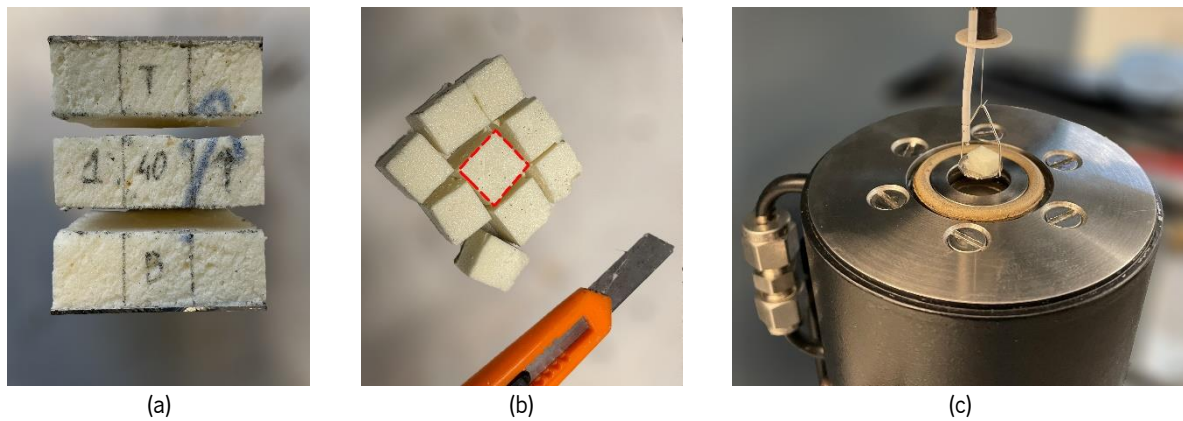


Figure 4.21. TGA sample preparation: (a) TL, ML, and BL of the cubic PUR foam specimen; (b) location of TGA sample extraction; (c) TGA sample in the pan of the TA Instruments Q600 SDT.

The TGA results are presented in terms of weight loss and differential weight loss (DTGA) curves in Figure 4.22, Figure 4.23, and Figure 4.24. The layers are the same as those used in Section 4.2.1 for the DIC measurements. The thermal degradation of all the samples is divided into two main stages, namely the dissociation of the urethane bonds into polyols and isocyanates and the scission of the polyols' groups [175]. The microstructure of the polymer matrix of the PUR foam is composed of hard segments (HS) deriving from isocyanate and chain extender and soft segments (SS) formed by polyols [176]. The first mass loss stage reaches its peak ($T_{\max,1}$) for all the samples between $314\text{ }^{\circ}\text{C}$ and $321\text{ }^{\circ}\text{C}$ and it is associated with the HS decomposition [177]. The second stage is attributed to the SS decomposition and reaches its peak ($T_{\max,2}$) between $451\text{ }^{\circ}\text{C}$ and $485\text{ }^{\circ}\text{C}$.

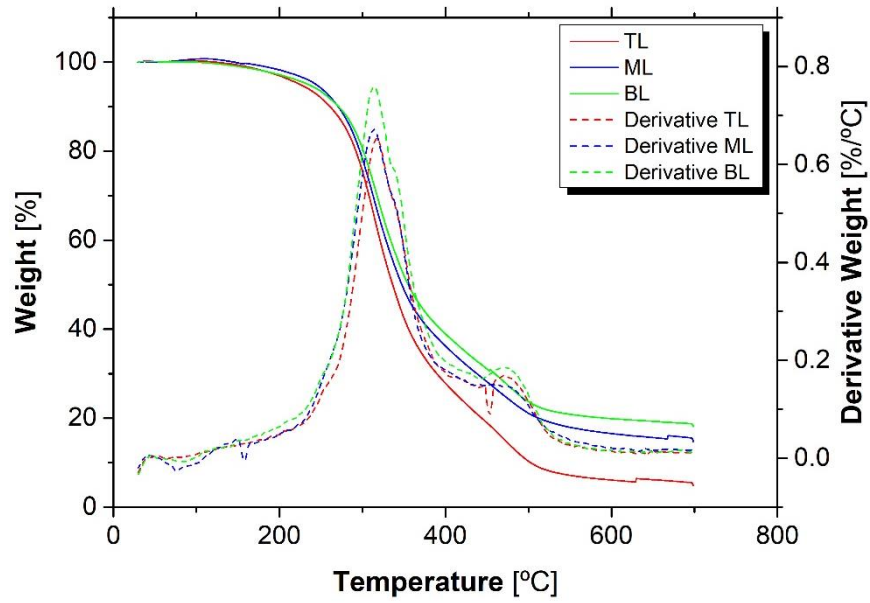


Figure 4.22. TGA and DTGA curves of the top (TL), middle (ML), and bottom layer (BL) of the 40 kg/m³ PUR foam.

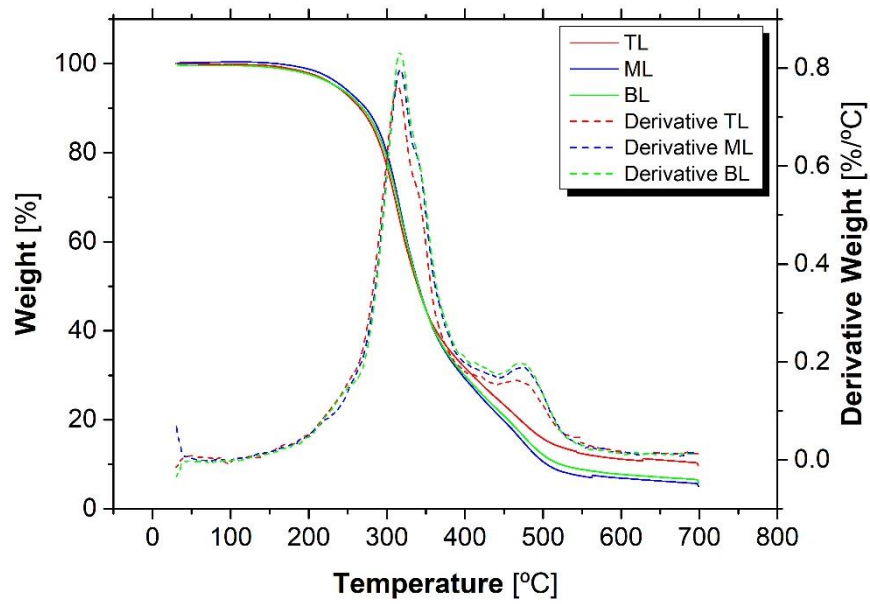


Figure 4.23. TGA and DTGA curves of the top (TL), middle (ML), and bottom layer (BL) of the 50 kg/m³ PUR foam.

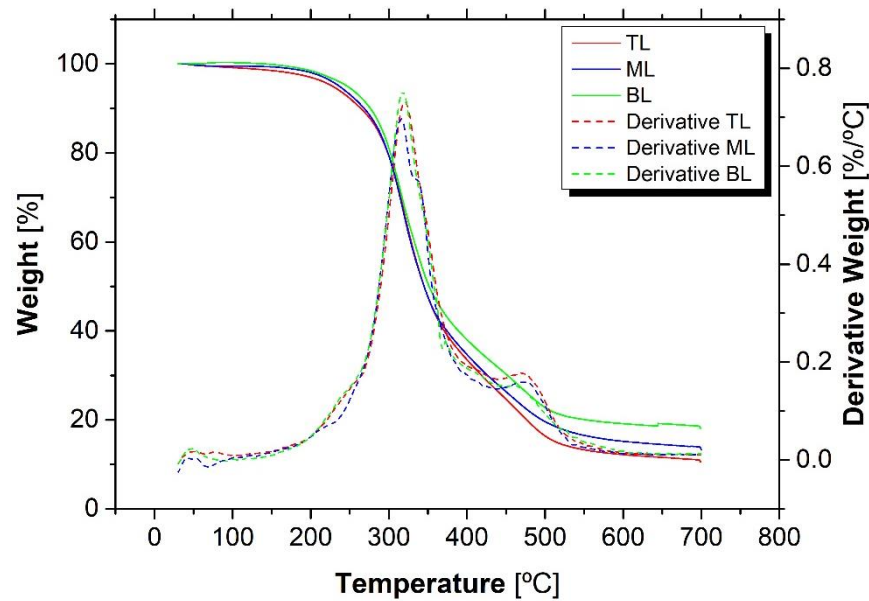


Figure 4.24. TGA and DTGA curves of the top (TL), middle (ML), and bottom layer (BL) of the 60 kg/m³ PUR foam.

The maximum peak of the first degradation stage is similar to other studies [178], [179] meaning that the urethane bond between the isocyanates and polyols is achieved. On the other hand, the second peak, and in general the range of the second stage is closer to the study presented in [175]. This may indicate the use of polyether in the synthesis of the PUR [177], as well as differences in reactions involved in the cross-linking of the polymer chains with the previous references. All the samples show a temperature corresponding to the 5% weight loss ($T_{5\%}$) included between 225 °C to 247 °C (see Table 4.7). The interval of values of $T_{5\%}$ is well above the temperature service conditions found in residential buildings. Nevertheless, such temperatures are slightly below the average value found in [180] for commercial PUR foams (about 250 °C).

Table 4.7. Thermal decomposition temperatures of the top, middle, and bottom layer of the PUR foams.

Density [kg/m ³]	Layer [-]	$T_{5\%}$ [°C]	$T_{\max,1}$ [°C]	$T_{\max,2}$ [°C]
40	Top	225	317	469
	Middle	244	314	451
	Bottom	235	314	475
50	Top	243	314	465
	Middle	235	321	485
	Bottom	234	317	472
60	Top	237	320	470
	Middle	247	315	474
	Bottom	229	318	459

The correlation between the TGA results and the mechanical properties is neither straightforward for the different layers of the PUR foam density nor for the layers of different PUR foams. This indicates the need for future studies involving a larger number of samples and possibly the integration of other experimental techniques such as the differential scanning calorimetry (DSC) and in situ Fourier Transformed Infrared spectroscopy (FTIR).

4.2.4. Single lap shear test on polyurethane adhesive

The adhesive connection is responsible to resist the in-plane shear sliding forces arising due to out-of-plane bending of the sandwich panel and to transfer the load between the different CFS sections composing the prototype. Beside the mechanical requirement, the PUR adhesive shall comply with another set of requirements stemming from the manufacturing process: i) compatibility with the PUR foam to avoid unwanted chemical reaction; ii) gelation time attainment after top bottom and top CFS sections are assembled together; iii) handling time attainment before the packaging process of the sandwich panels begins. The optimisation of the PUR adhesive with respect to these requirements is addressed in detail in [181] and [182]. It is worth mentioning that to ensure chemical compatibility the DEP team of the University of Minho utilise the same types of polyol and isocyanates for the synthesis of the PUR foam. An additive to prevent the pore formations was added to the chemical formulation.

The single lap shear test is selected to assess the strength of the metal-to-metal adhesively bonded joints of the sandwich panel. The single lap shear test is representative of the stress state of the adhesive in the structural members as well as adequate in the case of relatively thin adherends [183]. The test followed the specifications of ASTM D1002-10 (2010) [184]. The test was performed on 5 pairs of CFS rectangular sheets with nominal dimensions of 25.4 mm × 101.6 mm × 1.5 mm adhesively bonded together. To prepare the specimens, the following procedure was applied: i) the adherends faces were cleaned with acetone to remove any contaminants; ii) the pre-mixed adhesive was applied to one of the metal sheet; iii) the opposite sheet was then assembled, creating an overlap region of 12.7 mm; iv) a weight of about 50 g was applied over the joint; v) the assembled specimens were left to cure under controlled conditions (temperature of 23 °C, relative humidity of approximately 50%) with the applied pressure. The nominal thickness of the adhesive layer was 0.10 mm.

The tests were carried out in a UTM under displacement control, and the applied load was recorded by a load cell with a capacity of 10 kN. The relative displacements between the CFS sheets were measured by two LVDTs with a stroke of 10 mm installed on each face of the specimen to account for possible

rotations deriving from the misalignment of the sheets. M10 bolts inserted in the 5 mm radius hole drilled at a distance of 10 mm from the edge of CFS sheets were used to connect the specimen to the fixed and moving part of the test setup (see Figure 4.25). The tests were conducted in a quasi-static loading condition up to failure, at a displacement rate of 1.3 mm/min.

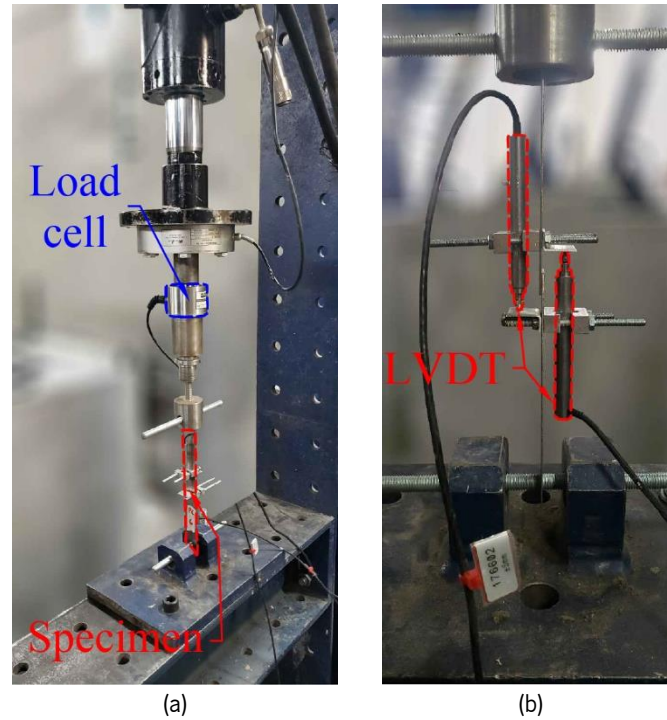


Figure 4.25. Single lap shear test: (a) overall test setup; (b) adhesive joint and LVDTs.

The load-slip curves of the single lap shear tests are illustrated in Figure 4.26. Specimens SLST-01, SLST-03, and SLST-04 presented a linear elastic behaviour with a stiffness decrease prior to failure. On the other hand, specimens SLST-02, SLST-05, and SLST-06 present plateau for small values of the displacement as well as lower stiffness values. This may be due to several reasons. During the preparation of the specimens unwanted misalignment between the CFS sheet may have occurred as suggested by the visual inspection of the specimens after failure (see Figure 4.27). Furthermore, deviation from the nominal thickness of the joint may result in different failure mode. The stiffer exhibited mixed failure mode involving interface failure at both adherends surfaces as well as cohesive failure. On the other hand, the more flexible presented interface failure with the adhesive left on one side of the joint.

The summary of the mechanical properties obtained from the single lap shear test are summarised in Table 4.8, including the stiffness of the joint (K) and the ultimate load (F_u) and its corresponding displacement (δ_u).

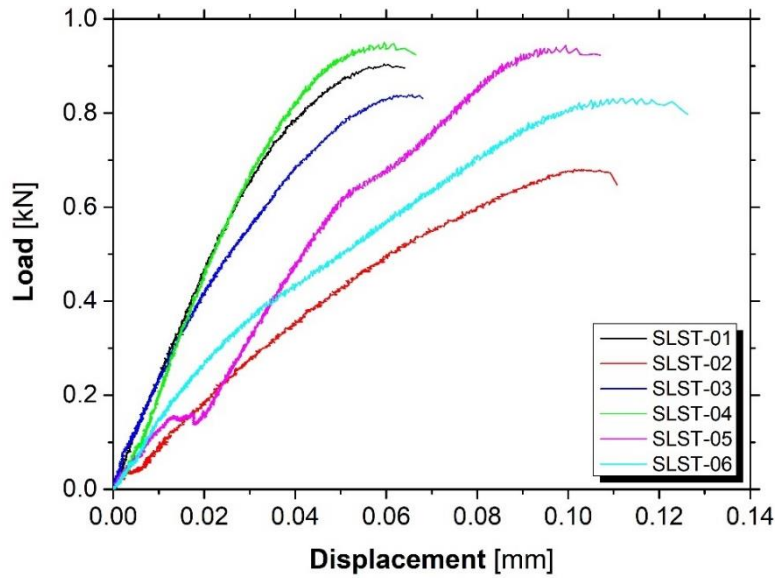


Figure 4.26. Load-displacement curves of the single lap shear test on the PUR adhesive.



Figure 4.27. Observed failure modes in the single lap shear test: (a) SLST-02; (b) SLST-04.

Table 4.8. PUR adhesive mechanical properties for the single lap shear test.

Specimen [-]	K [kN/mm]	F_u [kN]	δ_u [mm]
SLST-01	22.41	0.90	0.06
SLST-02	9.22	0.68	0.10
SLST-03	20.08	0.84	0.07
SLST-04	26.27	0.95	0.06
SLST-05	16.24	0.94	0.10
SLST-06	10.40	0.80	0.11
Avg. (CoV)	17.44 (38.7%)	0.85 (12.1%)	0.08 (29.3%)

Notes: Avg. – average results; CoV – coefficient of variation.

4.3. EDGEWISE COMPRESSIVE TEST

The experimental campaign carried out on small-scale sandwich panels aimed at assessing the mechanical behaviour of the sandwich panels produced by sandwich foaming technology utilised by Ferpaine. Small-scale specimens were extracted from larger panels. The edgewise compressive test was selected to assess axial load-carrying capacity of the sandwich panels. In civil engineering as well as

automotive and aerospace applications, such a test is often used to evaluate the mechanical properties of the composite structure, and in particular the capacity of the core to transfer stresses from one face sheet to another [36], [40], [50], [51], [185]. The axial stiffness characterisation is particularly relevant in seismic-prone areas where the floor shall act as a diaphragm, equally redistributing the horizontal actions to the vertical structures. Moreover, an analytical study was conducted for predicting the performance of the panels.

Edgewise compressive tests were carried out on two sets of specimens according to ASTM C364 (1999) [186]. The two sets of specimens had identical nominal dimensions and two different nominal thicknesses for the steel face sheet, namely 1.0 mm (EC-1.0) and 1.5 mm (EC-1.5). These specimens were extracted from larger sandwich panels with a core thickness of 100 mm and face sheet thicknesses of 1.0 mm and 1.5 mm, and had an area of 250 mm × 250 mm. Before testing, the specimens were measured and weighted, and an estimated density of 38.8 kg/m³ was calculated (relative difference from the nominal density of -3.1%). Visible defects were observed in the specimens upon delivery, which were caused by the extraction process from full-scale sandwich panels (see Figure 4.28).

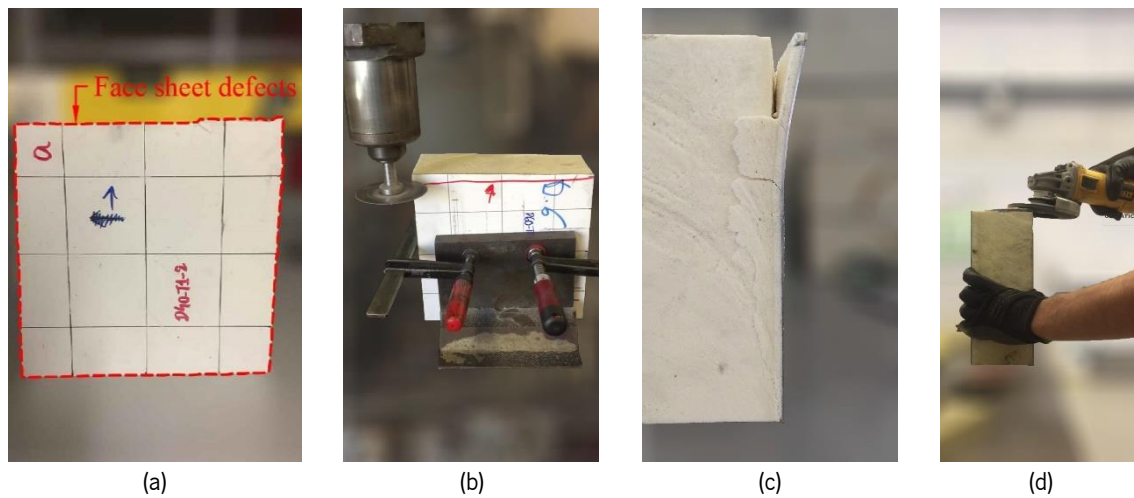


Figure 4.28. Sandwich panels specimens upon delivery: (a) face sheet defects; (b) milling machine treatment; (c) cohesive crack due to milling operation; (d) grinding wheel treatment.

To achieve flat end surfaces perpendicular to the length of the specimens, different methods were used. Although the milling machine produced smooth end surfaces, it also induced stress that caused a cohesive crack in the PUR foam parallel to the face sheets (see Figure 4.28c). Thus, a grinding wheel was found to be more appropriate for the smoothing process. The quality of the end surfaces was slightly lower than that produced by the milling machine, but no cracks were observed in the PUR foam.

The UTM used for testing had a load capacity of 200 kN. To ensure an even distribution of stress over the loaded area, a 50 mm thick solid steel plate was used to transfer the load. To prevent failure in the loading and support areas, 20 mm × 25 mm solid steel bars were attached to the plates on each side of the specimen. Four LVDTs were used to monitor vertical displacement on the bottom face of the loading plate (to remove the effect of possible rotations), and DIC was used to monitor one of the lateral faces of the specimen. The overall test setup is illustrated in Figure 4.29a. The tests were performed under displacement control at a speed of 0.5 mm/min.

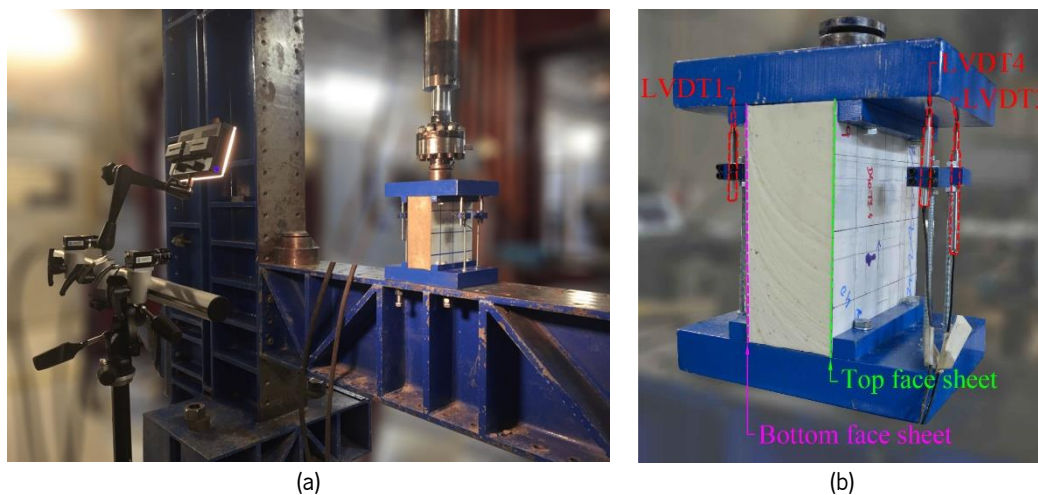


Figure 4.29. Edgewise compressive test: a) overall setup and DIC equipment; b) specimen.

The load-displacement curves of the edgewise compressive tests conducted on the EC-1.0 and EC-1.5 sets of specimens are shown in Figure 4.30. Typically, both sets exhibit an initial nonlinear section of the curve, which can be attributed to the closing of gaps between the loading plates and the ends of the samples. Once the end cross-sections are entirely engaged, a linear elastic behaviour is observed until failure initiation. The EC-1.0 specimens exhibited various failure modes, which resulted in high scatter in peak load results. These modes included debonding between the core and face sheet (D), global buckling (GB), wrinkling of the face sheets (W), and local buckling near the loading plate (LB). Conversely, the EC-1.5 specimens demonstrated a consistent failure mode due to global buckling. During the edgewise compressive test of the EC-1.0 specimen set, there was a significant deviation in the results, which could be attributed to several factors. These factors include the extraction process of the specimens from full-scale sandwich panels, resulting in irregular end surfaces and possible cracks in the PUR foam.

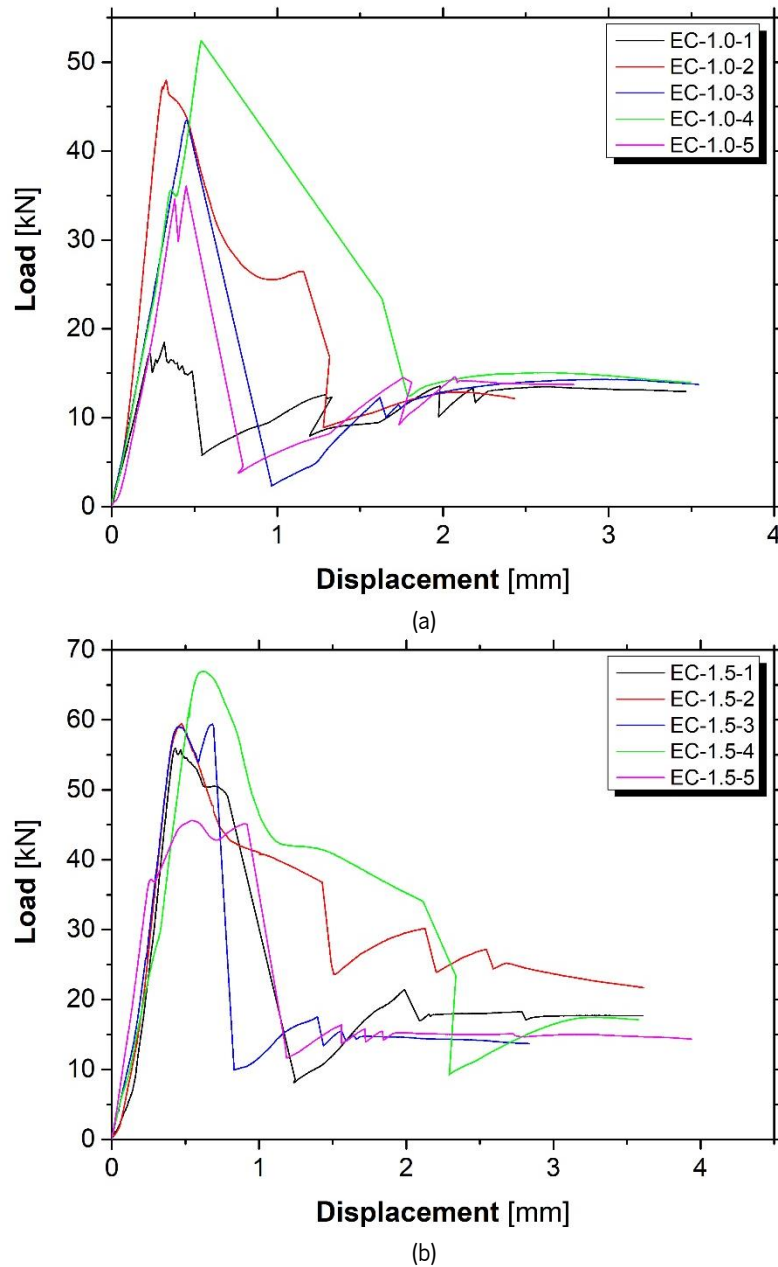


Figure 4.30. Edgewise compressive test load-displacement curves: a) EC-1.0 set; b) EC-1.5 set.

The premature debonding of specimen EC-1.0-1 may have been caused by pre-existing cracks due to this process. Smooth end surfaces were difficult to achieve due to stress in the bulk of the PUR foam, leading to the concentration of stresses and localised end failure. The scatter in results could also be attributed to the heterogeneity of the PUR foam and the face sheet-to-core interface. This interface is crucial for the composite structure's monolithic behaviour [73]. In Figure 4.31, the compressive response of an EC-1.0 specimen is presented alongside its deformation history as recorded by the DIC.

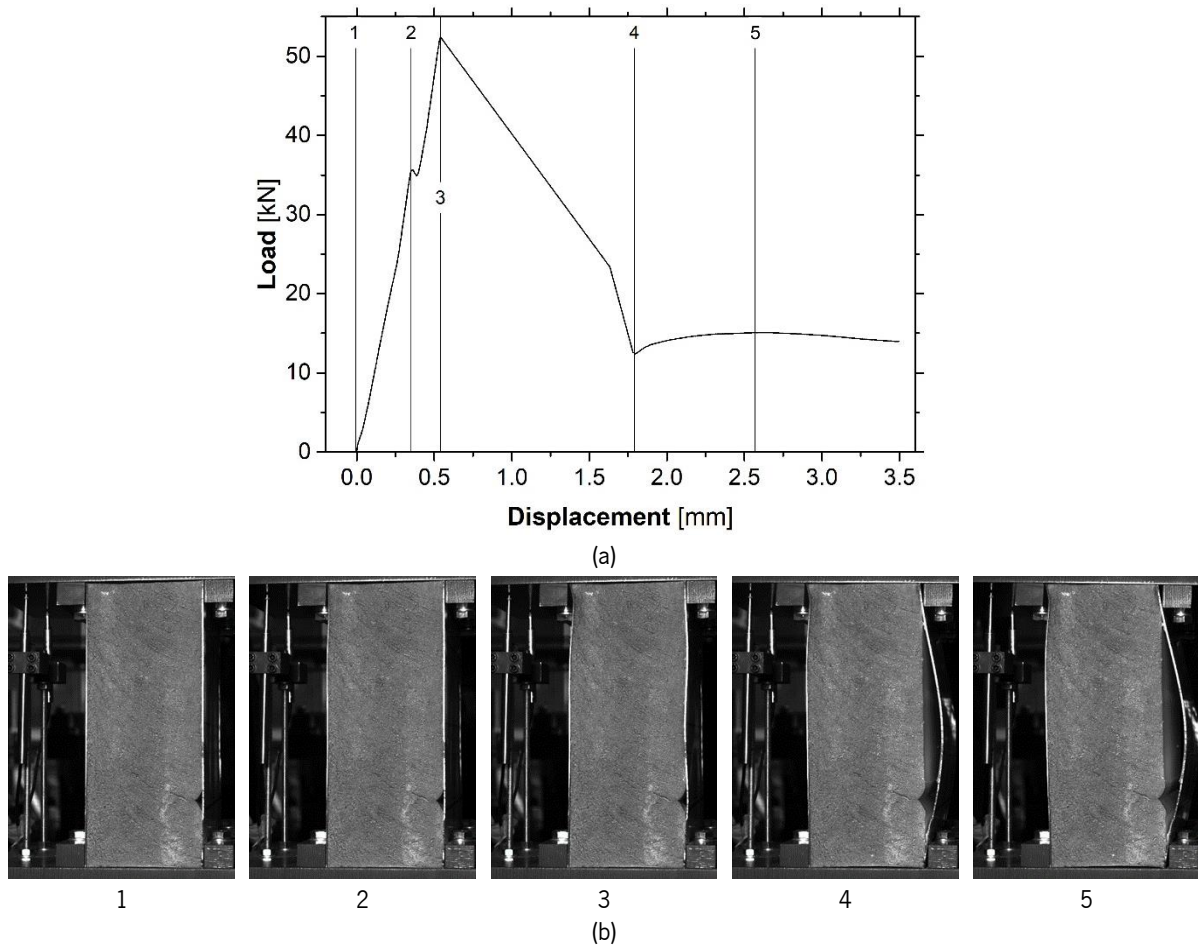


Figure 4.31. Edgewise compressive test DIC results for EC-1.0 sample: a) load-displacement curve; b) deformation history.

The load increases steadily until 0.3 mm, where a small load drop occurs due to local instability of the face sheet. When peak load is reached one of the face sheet wrinkles. The load drop after failure initiation is due to debonding at the top face sheet-to-core interface. The following constant load plateau corresponds to debonding propagation through the top face sheet-to-core interface. An average half wrinkling wavelength of 65.5 mm was estimated from the DIC measurements (see Figure 4.32).

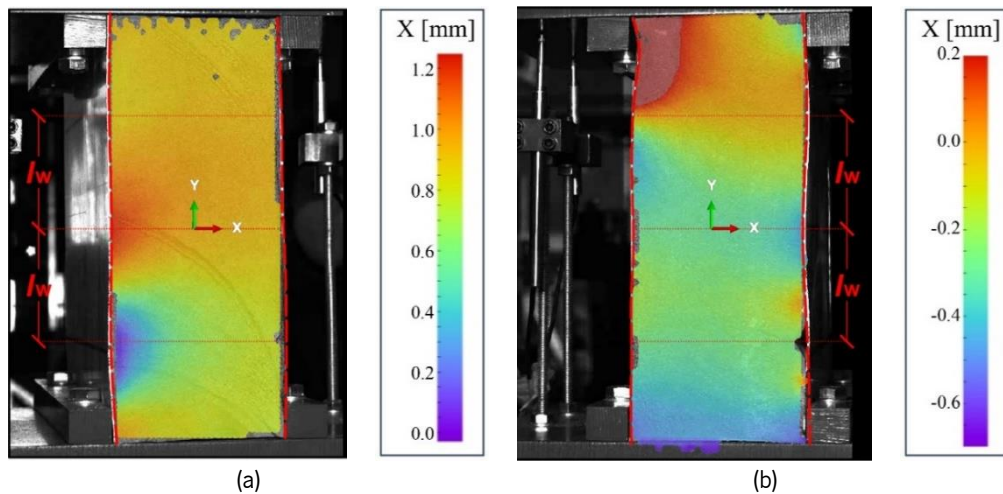


Figure 4.32. Wrinkling half wavelength estimation using DIC: a) EC-1.0-4; b) EC-1.0-5. All units in [mm].

The typical deformation and collapse of an EC-1.5 sample is shown in Figure 4.33. Before failure initiation, two distinct linear branches are visible. Following the attainment of peak load due to global buckling, a softening branch appears, indicating the formation of a plastic hinge in the top face sheet located near the loading plate, and the simultaneous crushing of the PUR foam core. The out-of-plane bending of the top face sheet is transferred to the bottom face sheet through the core. However, the PUR foam core's tensile strength is insufficient to hold the face sheets together with the core. Ultimately, a significant drop in load is observed as the bottom face sheet separates from the core due to debonding. It is worth noting that most specimens failed due to the debonding of the top face sheet. After testing, the specimens were examined, revealing a thin layer of PUR foam remaining on the debonded face sheet. These findings are consistent with the results of the flatwise tensile test conducted on cubic sandwich panel specimens. The tensile tests highlighted the presence of a more flexible layer of PUR foam next to the top face sheet, where the cohesive tensile crack ultimately occurred.

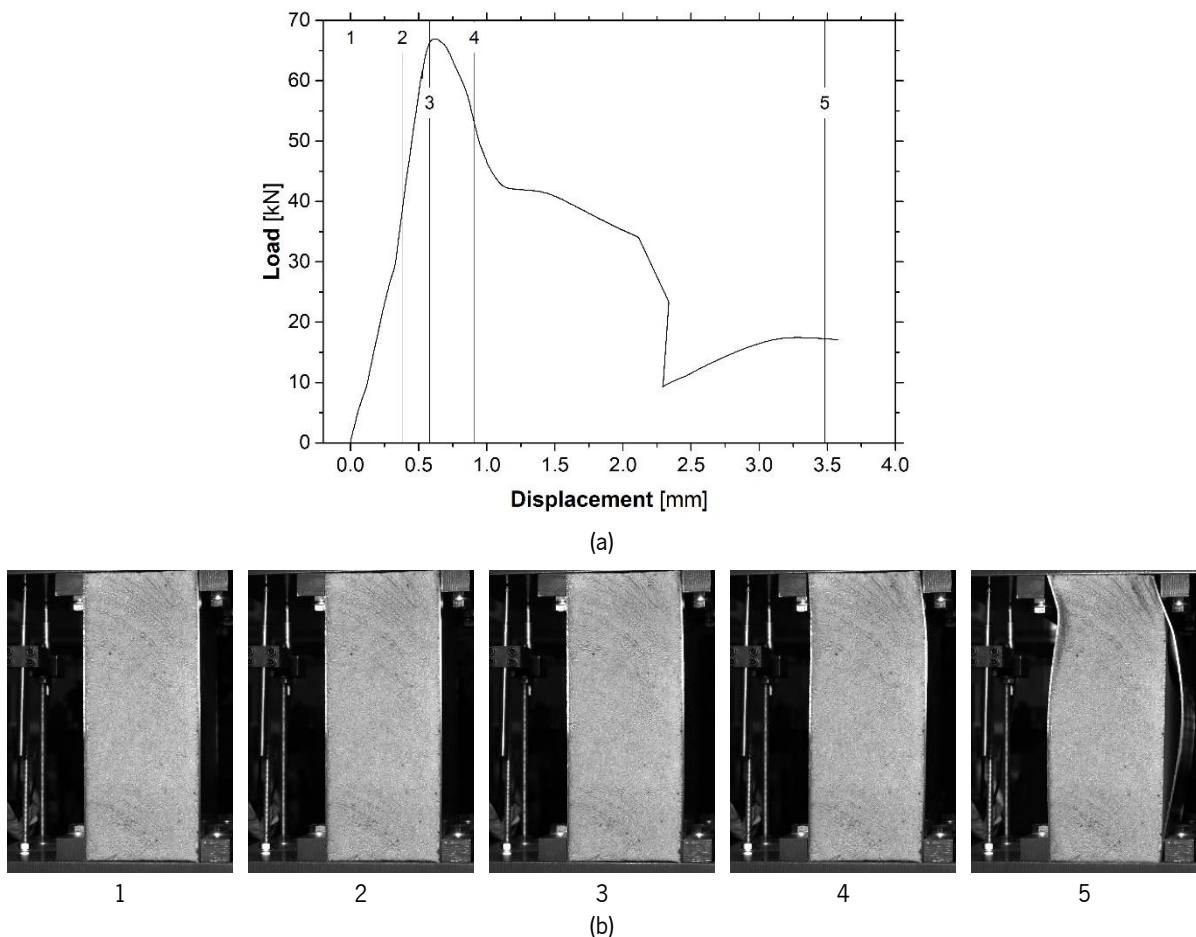


Figure 4.33. Edgewise compressive test results for EC-1.5 sample: a) load-displacement curve; b) deformation history.

In Table 4.9 a summary of the obtained results, including ultimate load (F_u), axial stiffness (K), failure initiation, and failure propagation is presented.

Table 4.9. Summary of the results of the edgewise compressive tests.

Specimen [-]	F_u [kN]	K [kN/mm]	Failure initiation [-]	Failure propagation [-]
EC-1.0-1	18.5	488.3	D (T)	D (T)
EC-1.0-2	48.0	395.1	LB + FC	D (T)
EC-1.0-3	43.5	-	W (T)	D (T)
EC-1.0-4	52.4	520.7	W (T) + FC	D (T)
EC-1.0-5	36.1	592.0	W (T)	D (T)
Avg. (CoV)	39.7 (33.5%)	468.0 (17.5%)		
EC-1.5-1	56.0	533.4	GB	D (T)
EC-1.5-2	59.4	503.7	GB	FC
EC-1.5-3	59.4	-	GB	D (T)
EC-1.5-4	67.0	571.2	GB	FC and D (B)
EC-1.5-5	45.6	555.7	GB	FC and D (B)
Avg. (CoV)	57.5 (13.5%)	541.0 (5.4%)		

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation; the symbol “-” indicates DIC measurements are unavailable for the specimen.

The displacement of the cross-sections in Figure 4.34 was tracked by the DIC system. The sampling points were located in the PUR foam near the face sheet. It is assumed that the deformation of the face sheet and the PUR foam close to the interface is the same until debonding occurs.

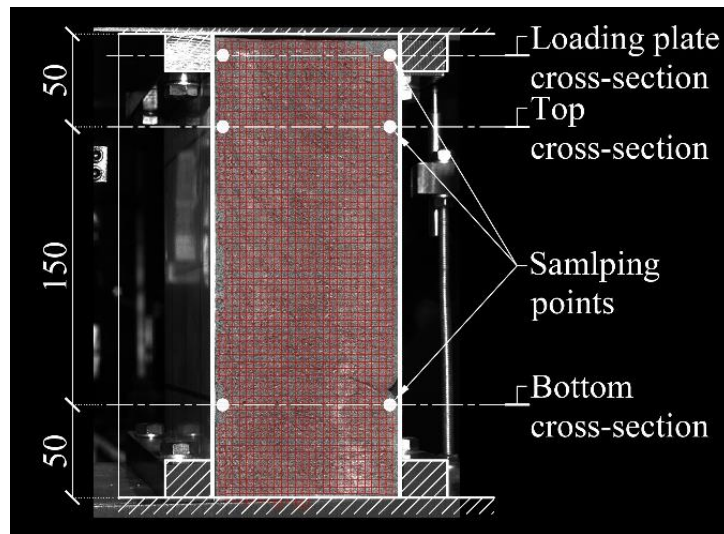


Figure 4.34. Location of the monitored points by the DIC in the edgewise compressive test. All units in [mm].

To validate the accuracy of the DIC system, the vertical displacement of the cross-section near the loading plate was compared with the results from LVDTs. The procedure is shown in Figure 4.35 for specimen EC-1.0-4. The DIC readings could not capture the rotations of the loading plate, causing a small difference (about 10%) from LDVTs measurement, as the DIC only monitors one of the vertical surfaces of the

specimen. Additionally, the reading of the loading plate cross-section was disrupted due to the crushing of the PUR foam, causing damage to the speckle pattern.

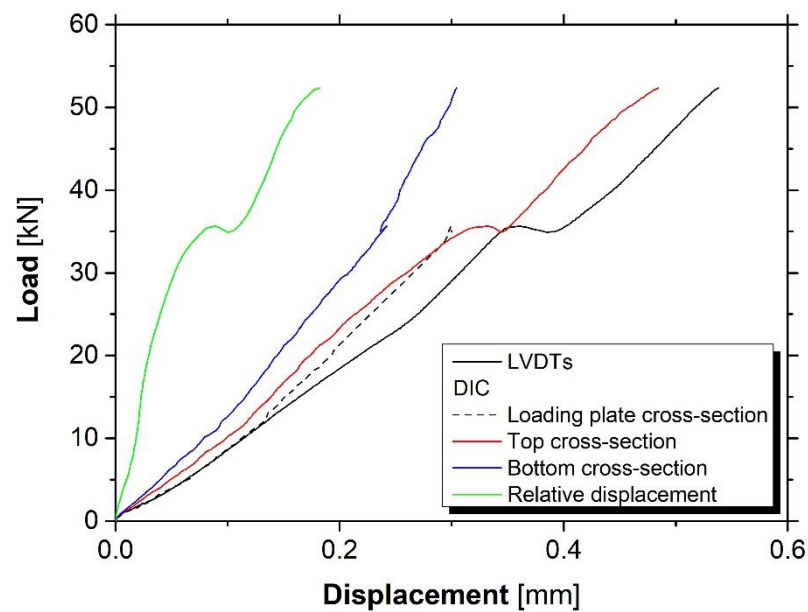


Figure 4.35. Comparison between LVDTs and DIC measurements for specimen EC-1.0-4.

The vertical displacement of the top and bottom cross-sections and their relative displacement (see Figure 4.36) showed that a significant component of the displacement occurred near the loading and supporting areas of the specimens due to the uneven end surfaces of the specimens.

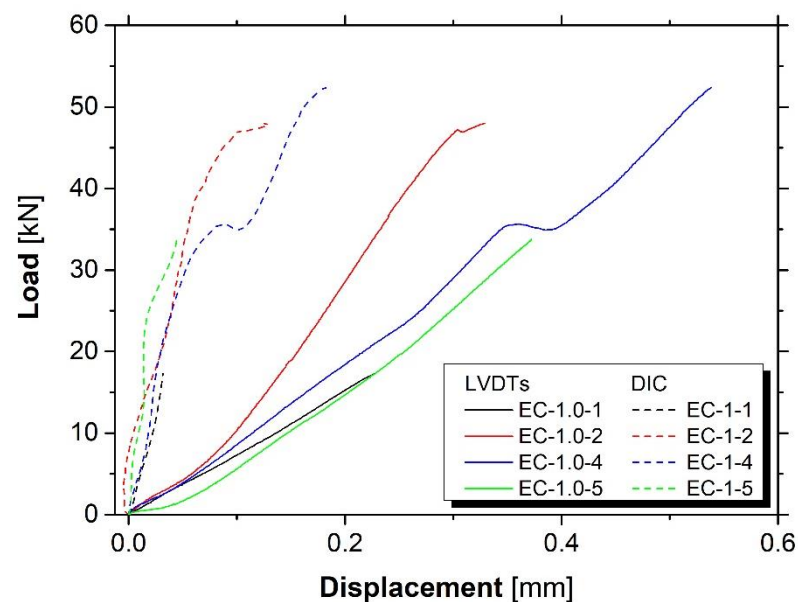


Figure 4.36. Comparison between LVDTs and DIC relative displacement between top and bottom cross-section for set EC-1.0.

The LVDTs measured the relative displacement between the loading and support plates, including the closing of gaps between the specimens and the test setup, as well as localised deformations. Therefore, to estimate the axial stiffness of the sandwich panels, load-displacement curves recorded by the DIC system were considered. However, the axial stiffness values for specimens EC-1.0-3 and EC-1.5-3 could not be estimated due to difficulties encountered during the image-matching process. These challenges were likely caused by varying lighting conditions in the laboratory facility due to changes in weather, resulting in insufficient pattern contrast for the algorithm to carry out the image correlation process [187].

4.4. ANALYTICAL MODELLING OF EDGEWISE COMPRESSIVE TESTS

To investigate the suitability of existing models to simulate the edgewise compressive behaviour of sandwich panels and to further understand the mechanical behaviour of the tested specimens, the analytical study on a sandwich panel with clamped ends is carried out. The panel has a length of L , width of b , and is subjected to compressive load P . It consists of face sheets with a thickness of t_f and a core with a thickness of t_c . The material properties of the face sheets and core materials are given in Table 4.1, Table 4.3, and Table 4.4. During the experiment, the sandwich panel's response up to the point of failure initiation was generally linear. The panel stiffness during this linear response is expressed as K . Assuming that the face sheet-to-core interface is perfectly bonded, the axial deflection of the sandwich panel is the same in the face sheet and the core, as given in Equation (4.5):

$$\delta_{\text{tot}} = \delta_f \Leftrightarrow \frac{P \cdot L}{E_e \cdot A_{\text{tot}}} = \frac{P \cdot L}{E_f \cdot A_f} \quad (4.5)$$

where δ_{tot} is the sandwich panel deformation, δ_f is the axial deformation of the face sheets, A_{tot} is the total cross-section area of the sandwich panel, E_e is the effective Young's modulus of the sandwich panel [188], E_f is the Young's modulus of the face sheets, and A_f is the cross-section area of the face sheets. Keeping in mind that the axial stiffness of a beam under compression is calculated by multiplying the Young's modulus of the material by the beam's area and dividing it by its length, the stiffness of the sandwich panel is determined according to Equation (4.6):

$$K = \frac{E_e \cdot A_{\text{tot}}}{L} \quad (4.6)$$

When predicting the ultimate load of a sandwich panel, two failure modes are identified: global buckling and wrinkling. It is assumed that both the EC-1.0 and EC-1.5 sets consist of sandwich panels with thin

faces, where most of the bending stiffness (more than 99%) comes from the bending of the face sheets around the centroidal axis. The cores are weak, and their bending stiffness contributes to less than 1% of the total stiffness [189]. In this case, the global buckling load (P_{cr}) is a combination of Euler buckling (P_E) and buckling of the core due to shear (P_s) as shown in Equation (4.7):

$$\frac{1}{P_{cr}} = \frac{1}{P_E} + \frac{1}{P_s} \quad (4.7)$$

The Euler buckling and the core shear buckling are defined by the following expressions:

$$P_E = \frac{\pi^2 \cdot (E \cdot I)_{eq}}{(k \cdot L_e)^2} \quad (4.8)$$

$$P_s = b \cdot t_c \cdot G_c \quad (4.9)$$

where G_c is the core shear modulus, and L_e is the free length of the sandwich panel between the clamps. Wrinkling happens when periodic waves with a similar thickness as the core layer appear all over the face sheet. There are three types of wrinkling: i) rigid base (single-sided); ii) anti-symmetric; and iii) symmetric. In this study, symmetric wrinkling is the most relevant type since periodic waves can be seen on both face sheets. According to [190] the wrinkling half wavelength (l_w) and the wrinkling critical stress (σ_w) in the face sheet are given by:

$$l_w = 1.65 \cdot t_f \cdot \sqrt[6]{\frac{E_f^2}{E_c \cdot G_c}} \quad (4.10)$$

$$\sigma_w = 0.91 \cdot \sqrt[3]{E_f \cdot E_c \cdot G_c} \quad (4.11)$$

The critical wrinkling load (P_w) will be reached when the face sheets reach the critical wrinkling stress according to Equation (4.12):

$$P_w = 2 \cdot b \cdot t_f \cdot \sigma_w \quad (4.12)$$

A summary of the analytical results, along with the experimental values, is presented in Table 4.10. The axial stiffnesses for specimens EC-1.0 and EC-1.5 are 413.3 kN/mm and 581.0 kN/mm, respectively, according to Equation (4.6). The experimental results are within 10% of the analytical predictions, indicating the assumption of a perfectly bonded interface during the linear stage of axial response is reasonable. As mentioned in Section 4.2.2, the estimated values of ν based on the mechanical

characterisation of the PUR foam yield unusual negative values and possible explanations have been provided. Based on that discussion, it was deemed adequate to estimate the value of G_c according to [171] assuming that the ν of PUR foam for small strains may be taken as 0.3.

Table 4.10. Summary of the geometrical properties and predicted and measured failure loads.

Set	L [mm]	t_c [mm]	t_f [mm]	K [kN/mm]	K_{exp} [mm]	P_{cr} [kN]	l_w [mm]	P_{wr} [kN]	P_{exp} [kN]
EC-1.0	250	100	1.0	413.3	468.0	55.6	64.1	62.9	44.0*
EC-1.5	250	100	1.5	518.0	541.0	55.6	94.2	92.5	57.5

Notes: K_{exp} – average experimental value of the axial stiffness of sandwich panels; P_{exp} – average experimental value of the peak load of sandwich panels; highlighted with “*” is the average wrinkling load of specimens EC-1.0-3, EC-1.0-4, and EC-1.0-5.

Equation (4.7) results in a value of 55.6 kN for both sets of specimens since the buckling of the core due to shear governs the overall failure mode. The estimated global buckling load agrees well with the average experimental ultimate load of EC-1.5 specimens (-3.4% relative difference). The critical wrinkling load is calculated using Equation (4.12), and the values of 62.9 kN and 92.5 kN are obtained for the EC-1.0 and EC-1.5 sets, respectively. The critical wrinkling load for EC-1.5 is higher than the global buckling load, indicating consistency in the failure mode. The predicted critical wrinkling load differs from the average experimental ultimate load of EC-1.0 specimens that failed due to wrinkling (44.0 kN). Nevertheless, several authors have proposed different values for the constant coefficient of Equation (4.11), which range from 0.5 to 0.91 [190], [191]. However, these coefficients were altered to fit the experimental results and may not be appropriate for the sandwich panel studied in this work. If a coefficient of 0.65 were used, the EC-1.0 and EC-1.5 specimens would have wrinkling loads of 44.9 kN and 66.1 kN, respectively. These results would suggest that global buckling occurred in the EC-1.5 specimens and would also decrease the difference between the experimental average peak load of the EC-1.0 specimens. The EC-1.0 specimens had a wrinkling half wavelength of 64.1 mm, according to Equation (4.10), which matches the analytical prediction (-2.2% relative difference) despite the discrepancy in ultimate load. This difference may be due to the presence of geometrical imperfections, which affect the ultimate load instead of the instability mode's shape.

4.5. CONCLUDING REMARKS

The mechanical characterisation of the constituent materials of the sandwich panel was carried out, namely cold-formed steel, PUR foam, and PUR adhesive. Furthermore, small-scale sandwich panels were tested in edgewise compression. The following conclusions may be drawn:

- The CFS sheets with a nominal thickness of 1.0 mm satisfy the minimum requirements imposed by the Eurocode 3 Part 1-3 (2006) [80]. Furthermore, the CFS sheets possess a zinc coating thickness of class Z225 according to EN 10346 (2015) [151]. The CFS sheets, which have a nominal thickness of 1.5 mm, fail to meet the requirements of the lowest grade, i.e. S220GD+Z. As a result, there is a considerable difference between the static and dynamic curves of CFS coupon specimens;
- The mechanical properties of the 40 kg/m³ PUR foam produced by Ferpainel agree with the expected values for this type of material. On the other hand, the 50 kg/m³ and 60 kg/m³ PUR foam show unexpectedly lower mechanical properties. Additional studies should be conducted to establish the influence of the chemical formulation, cell morphology, and production technology on the macroscopic properties of the PUR foam;
- The estimated Poisson's ratio values indicate that the PUR foam produced through sandwich foaming is anisotropic. DIC measurements also show that the PUR foam exhibits different mechanical properties across its thickness. This outcome confirms that the size and shape of the mould have a significant influence on the properties of polymeric foams;
- The results of the flatwise tensile test and edgewise compressive test are consistent with the hypothesis that failure initiation of the composite structure is located in the layer of the PUR foam near the top face sheet. The PUR foam undergoes brittle failure when subjected to tensile stresses in the direction perpendicular to the face sheet. The crack propagation generally causes the debonding of the CFS face sheet.

CHAPTER 5

5. FLEXURAL RESPONSE OF SANDWICH PANELS

An experimental study was conducted on small- and full-scale sandwich panels to determine their suitability as a residential building floor system. This comprehensive campaign was developed in two stages. First, due to initial issues that arose during the manufacturing process (see Chapter 3), a first batch of small-scale sandwich panels was produced to validate the analytical design. After verifying that the prototype's quality was sufficient and its behaviour met the expected analytical design, full-scale sandwich panels were manufactured to evaluate their performance and ensure compliance with the requirements at the SLS and ULS.

Single and jointed small-scale sandwich panels were subjected to four-point bending tests - details of the panel-to-panel connection system can be found in Chapter 6. In the present chapter, the tests on jointed sandwich panels were conducted only on the small-scale panels and the objective was to confirm that the dimensional tolerances allowed for the proper execution of the connection systems and to preliminarily assess the effectiveness and contribution of the connection to the overall behaviour of the structure.

The full-scale sandwich panels were investigated using three different types of tests: i) static three-point bending test up to serviceability state load; ii) dynamic test; and iii) static four-point bending test up to failure. The static serviceability tests aimed at assessing the effective flexural properties of the prototypes and validating the analytical design methods for the SLS. The dynamic test's objective is to assess the natural frequencies of the sandwich panels and verify their compliance with the limit on the vibration related to the comfort of the occupants. The main aims of the static flexural tests up to failure were to assess the ultimate capacity of the panels and their failure mechanisms and validate the analytical design methods for the ULS.

Finally, the results of the experimental campaign provided important information on the mechanical behaviour of the sandwich panel that were compared with numerical predictions carried out in Chapter 7.

5.1. SMALL-SCALE FOUR-POINT BENDING TEST

For the preliminary experimental campaign, six panels measuring 2.0 m in length were produced to confirm the analytical design and evaluate the flexural load-bearing capacity of the sandwich panels. The panels were tested in a four-point bending configuration. Two tests were conducted on individual panels, and two were performed on sets of two panels joined together with adhesive (slabs).

The panels were connected using an epoxy structural adhesive (S&P Resin 220 HP) after cleaning the surfaces with acetone and sandpaper. The jointed panels were then secured with ratchet tie-down straps and left to cure at room temperature for 72 h before starting the test. A thin layer of bitumen-based polyester putty was applied to the load application zones to ensure an even distribution of the load between the two connected panels. The overall test setup is presented in Figure 5.1.

The panels were tested over a clear span of 1.80 m with a 0.60 m shear span in simply supported conditions. To allow for free rotation at the supports, steel plates with a cross-section of 10 mm x 100 mm and 10 mm x 50 mm were placed on top of steel rollers for the single and joint sandwich panels, respectively. To distribute the load across the width of the single and joint panels, a IPE140 and a HEB120 steel beam were used, respectively. Vertical deflections were measured using LVDTs with a stroke of 75 mm (± 0.075 mm) and 100 mm (± 0.1 mm) located under the loaded section and at midspan, respectively. Additionally, two support bars were added to the sandwich panels' sides, and LVDTs were installed to measure vertical deflection, excluding any support settlement displacements. TML PFL10-11 strain gauges (SG) were bonded to the top and bottom face sheet of the sandwich panels to register the longitudinal strains. A schematic view of the instrumentation used on the tests is shown in Figure 5.2. The tests were conducted under quasi-static monotonic-instantaneous loading up to failure, under displacement control at a speed of 1.8 mm/min. A load cell with a capacity of 200 kN (± 0.3 kN) was used to measure the load.

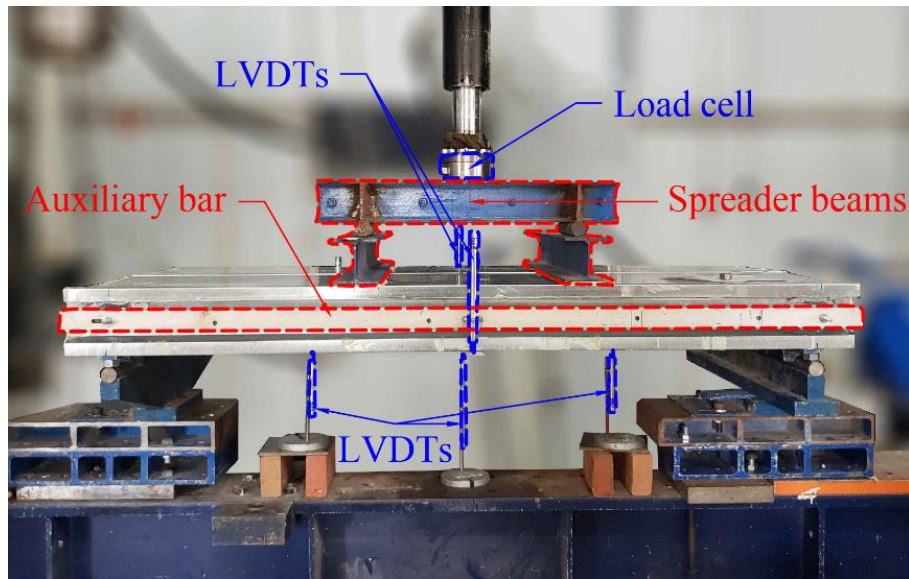


Figure 5.1. Small-scale four-point bending test setup.

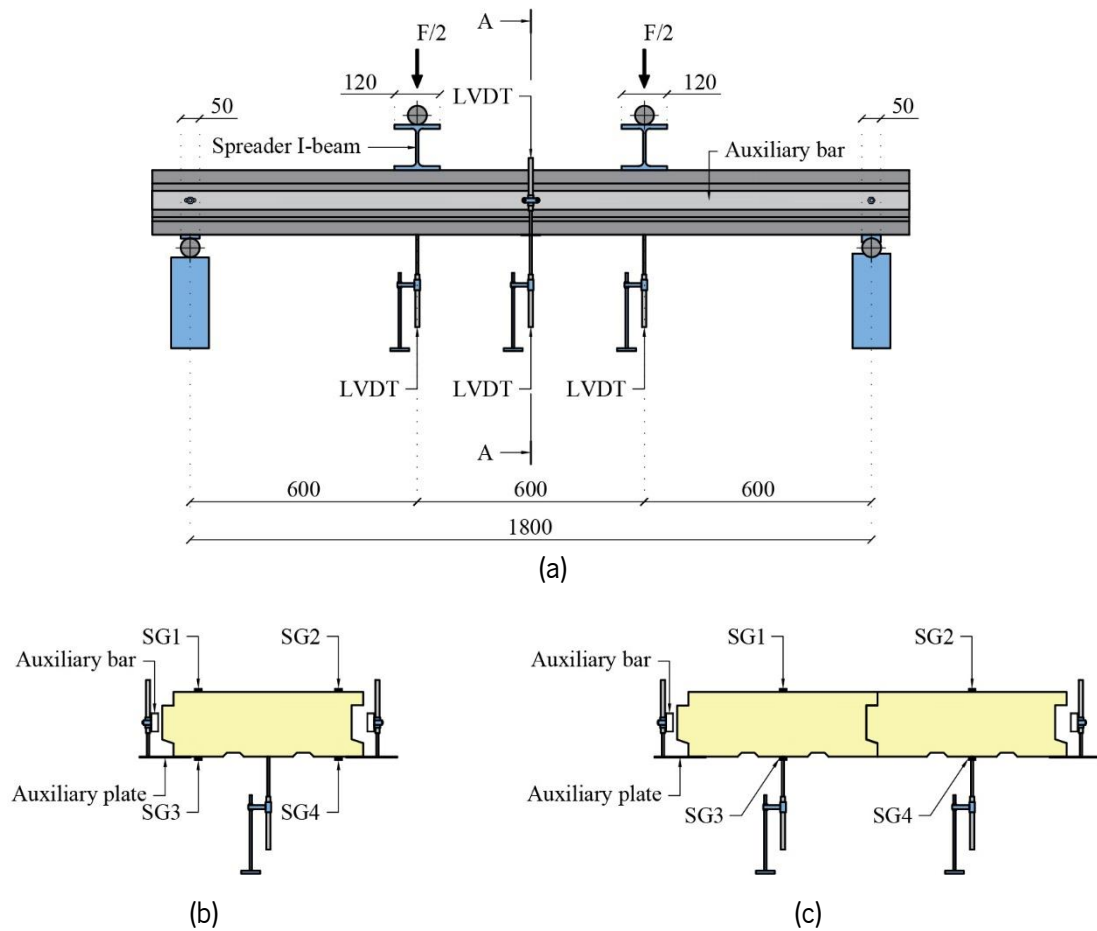


Figure 5.2. Schematic representation of the small-scale four-point bending test: (a) lateral view of the joint panels; (b) A-A section of the single panel; (c) A-A section of the joint panels. All units in [mm].

5.1.1. Single sandwich panel

The load-midspan displacement curves for the two single sandwich panels (SP1 and SP2) are presented in Figure 5.3a. The midspan displacement is calculated as the average of the readings of the LVDTs installed on the auxiliary bars at midspan to disregard possible settlements at the supports. The load-longitudinal strain of all the panels is illustrated in Figure 5.3b. The progress of the failure of sandwich panel SP1 is presented in Figure 5.4.

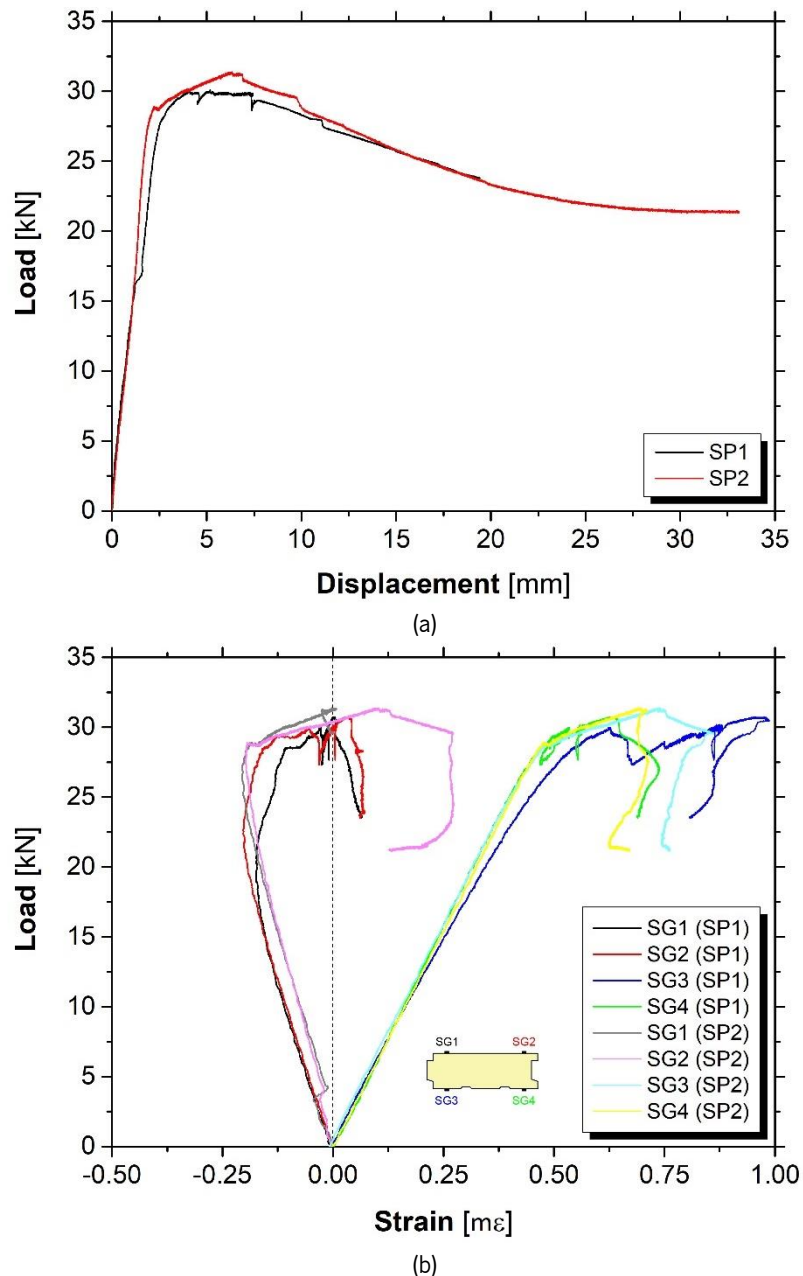


Figure 5.3. Small-scale four-point bending test of single sandwich panels: (a) load-displacement curve; (b) load-longitudinal strain curve.

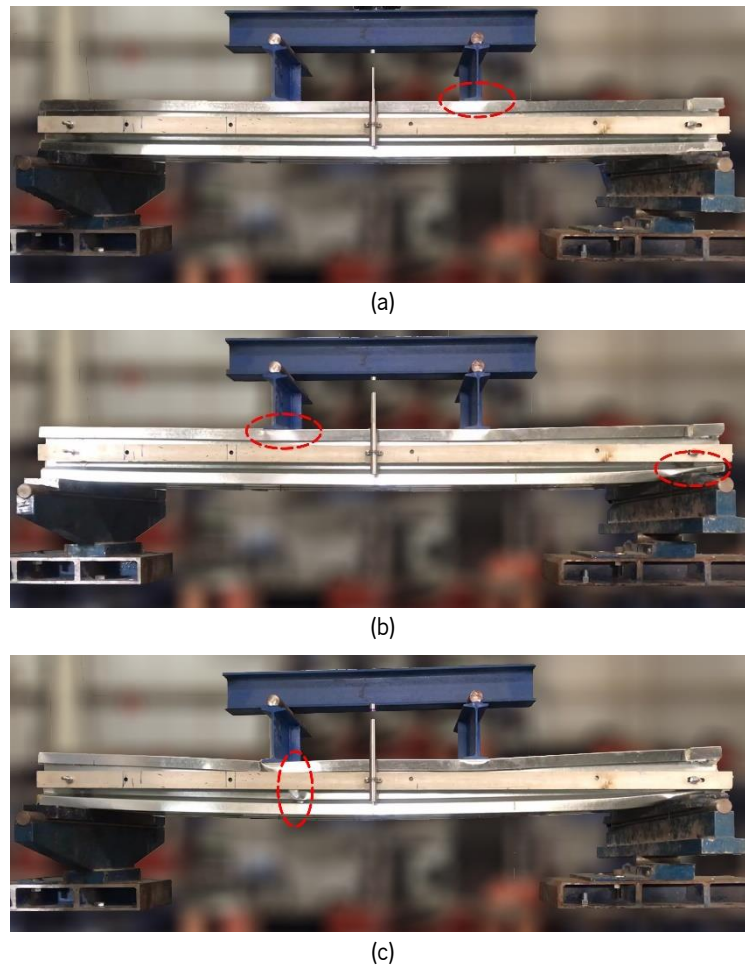


Figure 5.4. Failure progress of single sandwich panel SP1: (a) onset of top face sheet wrinkling; (b) PUR foam crushing under loaded sections and end bearing failure of the web; (c) web local buckling under the loaded section.

Both panels presented an approximately linear behaviour with a stiffness reduction before reaching the ultimate load. The reduction in stiffness is also attested by the strain gauges measurements of both panels. The strain gauges SG1 and SG2, located on the compressed face sheet, revert direction at a high load level, reflecting the onset of outward wrinkling phenomenon [59]. This may suggest the fracture of the PUR foam near the top face sheet as observed previously in Chapter 4. The sandwich panels continued to carry load, though with reduced stiffness, as crushing of the top face sheet at the loaded sections progressed. This may indicate that the PUR foam densification in the loaded region allowed the bulk of the core to sustain additional load. The crushing of the top face sheet resulted in the outward deformation of the flanges of the top plain C-section as shown in Figure 5.4a and Figure 5.4b. Panel SP1 also presented bearing failure at one of the supports [62], i.e. a combination of web crippling and crushing of the PUR foam core material.

The average ultimate load registered was 30.74 kN at a displacement of 5.76 mm. Final failure is triggered by local buckling of the webs adjacent to the loaded sections, most likely due to stress

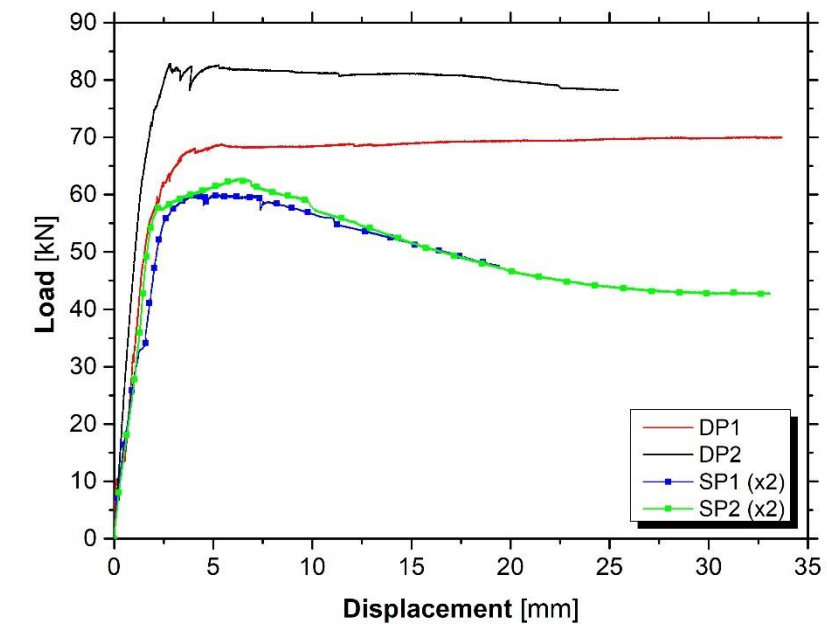
concentration in those regions. After the ultimate load is reached, the load decreases gradually. The sandwich panels retain about 70% of the maximum load at 30 mm displacement, thus displaying a desirable ductility. The concentration of plastic deformation in the buckled webs is reflected by a partial recovery of the elastic deformation measured by the strain gauges SG3 and SG4 located on the bottom face sheet.

5.1.2. Joint sandwich panels

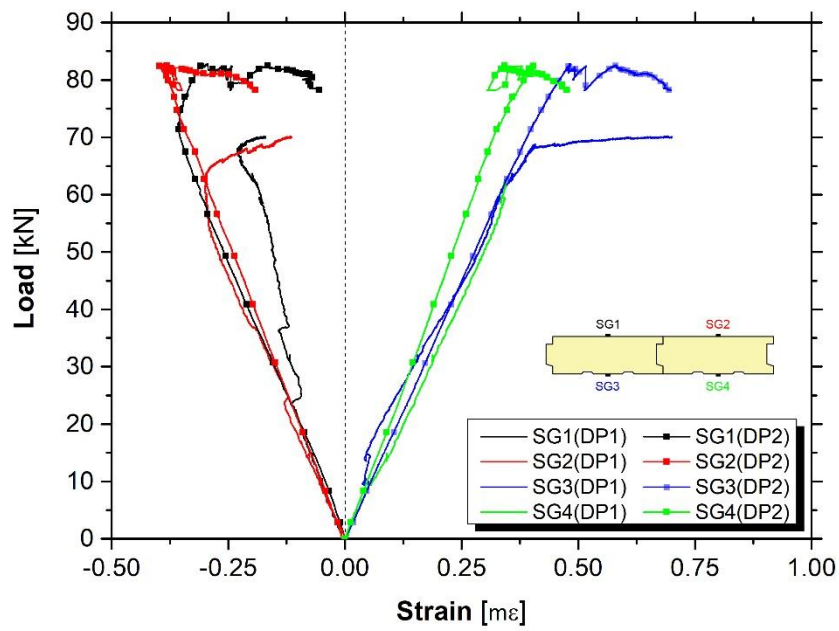
The load-midspan displacement curves for the jointed sandwich panels DP1 and DP2 are illustrated in Figure 5.5a, including the projection of the curve of the single panels with double the corresponding force obtained in the single panel four-point bending test. The load-longitudinal strain curves are presented in Figure 5.5b and the progress of the failure of sandwich panel DP2 is presented in Figure 5.6.

Both panels presented an approximately linear behaviour with a stiffness reduction coinciding with failure initiation. The onset of outward wrinkling, similarly to what has been observed for the single sandwich panels, is reflected in the strain gauges measurements of SG1 and SG2 of both panels. Nevertheless, after failure initiation DP1 and DP2 panels presented a slightly different behaviour. The DP1 panels presented bearing failure of the webs at both supports. Plastic hinges formed at the supports where the deformations concentrated. DP1 continued carry load until the end of the test reaching an ultimate load of 70.18 kN at a displacement of 35.70 mm. The progressive failure of DP2 panels showed a similar pattern to that observed for the single sandwich panels. The ultimate load registered for DP2 panels was 82.85 kN at a displacement of 2.86 mm. After the maximum load was reached, the damage concentrated in the buckled web of one of the loaded regions and at one of the supports.

A summary of the obtained results of the small-scale four-point bending test of the single and joint sandwich panels is given in Table 5.1. The joint sandwich panels showed ultimate load and stiffness greater than two times the value of those of the single sandwich panels. This suggests the presence of synergic effect due to the adhesive connection. The stiffness is determined as the slope of the load-displacement curve in the initial (elastic) stage. The slope of the curve is calculated by performing a linear regression of the data in the load interval between 20% and 45% of the ultimate load.



(a)



(b)

Figure 5.5. Small-scale four-point bending test of joint sandwich panels: (a) load-displacement curve; (b) load-longitudinal strain curve.

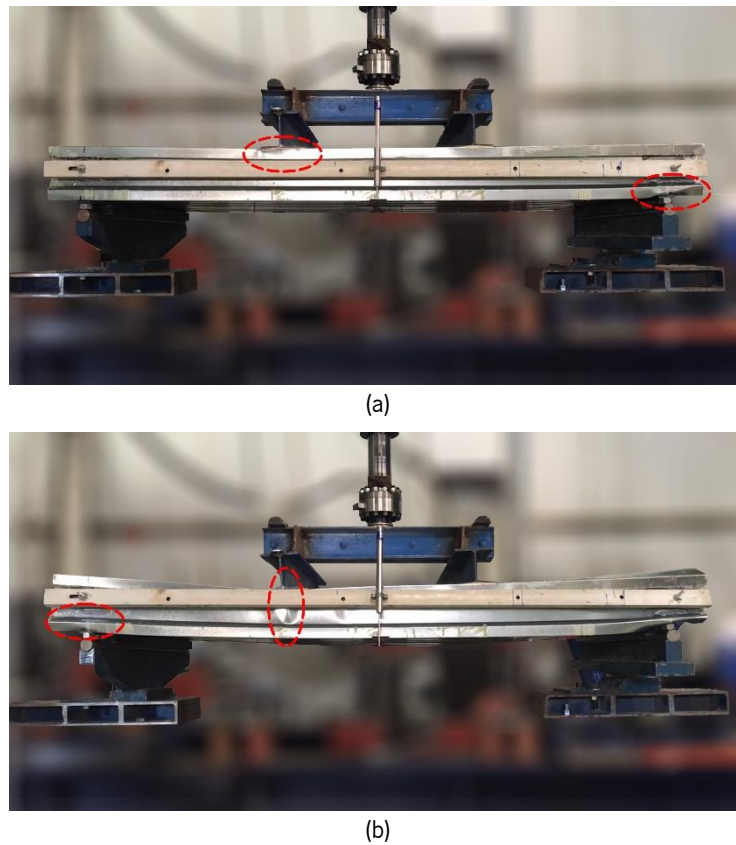


Figure 5.6. Progress failure of set of joint sandwich panels DP2: (a) face sheet wrinkling and end bearing failure of the web; (b) buckling of the web under the loaded section and end bearing failure of the web.

Table 5.1. Summary of the results of the small-scale four-point bending tests on single and joint panels.

Specimen	K [kN/mm]	F_u [kN]	δ_u [mm]
SP1	11.66	30.08	5.18
SP2	12.28	31.39	6.35
Avg.	11.97	30.74	5.76
DP1	45.55	70.18	35.70
DP2	48.98	82.85	2.86
Avg.	47.27	76.51	19.28

Notes: Avg. – average value per set of specimens; K – stiffness; F_u – ultimate load; δ_u displacement at ultimate load.

5.1.3. Analytical modelling

To validate the results of the analytical design at the ULS and SLS described in Chapter 3, the analytical load-carrying capacity and stiffness of the sandwich panel is compared to the experimental results of the small-scale four-point bending test of the single panels. The experimental test setup may be approximated to a simply supported beam subjected to two concentrated loads applied at thirds of the span length. It is assumed that the sandwich panel attains its maximum load when the midspan cross-section reaches

its ultimate bending moment. The ultimate bending moment is a characteristic of the cross-section, and it is independent of the loading configuration and span length. Therefore, utilising the ultimate bending moment estimated for the structural model considered in the analytical design and substituting it in the equation of the bending moment diagram of the simply supported beam subjected to two concentrated loads yields:

$$F_{ULS} = \frac{6 \cdot M_{c,Rd}}{L} \quad (5.1)$$

where F_{ULS} is the analytical prediction of the ultimate load-carrying capacity of the panel, $M_{c,Rd}$ is the design bending moment resistance based on the effective properties of the cross-section according to the Eurocode 3 Part 1-3 (2006) [80] and Part 1-5 (2006) [102].

For what concerns the stiffness of the sandwich panel, three different approaches are considered. The first method (M1) considers the gross cross-section properties along the length of the beam for the calculation of the moment of inertia (I_{M1}). M1 is based on the hypothesis that the PUR foam core is sufficiently rigid to prevent local instabilities in the plate elements of the CFS cross-section, and an elastic distribution of stresses exists in the cross-section (see Figure 5.7a).

The second method (M2) considers the effective cross-sectional area determined at the maximum stress level along the beam span to estimate the moment of inertia (I_{M2}). M2 does not take into consideration the contribution of the PUR foam. Therefore, it assumes that local instabilities will occur before the cross-section develops its elastic moment resistance. These local instabilities cause an uneven stress distribution across the width of the plate elements. The effect of local and distortional buckling is accounted for according to the EMW (see Figure 5.7b).

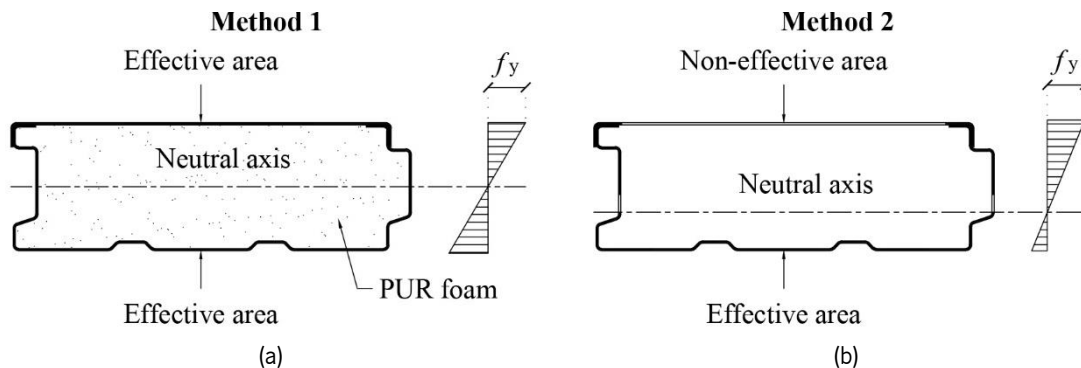


Figure 5.7. Design method for the estimation of the sandwich panel's deflection: (a) M1; (b) M2.

Finally, the third method (M3) follows the guidelines set out in Eurocode 3 Part 1-3 (2006) [80], which doesn't consider the contribution of the PUR foam. However, the bending stress variation that causes

local buckling is accommodated by calculating the effective cross-section properties at the midspan cross-section, where the maximum stress occurs, and interpolating them with the gross cross-section properties. The interpolated moment of inertia (I_{M3}) is calculated according to Equation (5.2):

$$I_{M3} = I_{M1} - \frac{\sigma_{gr}}{\sigma} \cdot (I_{M1} - I_{M2}) \quad (5.2)$$

where σ_{gr} is the maximum compressive bending stress determined based on the gross cross-section properties and σ is the maximum compressive bending stress determined based on the effective properties considering the maximum level of bending moment within the span length.

In summary, for the calculation of the bending stiffness of a sandwich panel cross-section, three methods are considered. M1 is the least conservative, as it assumes that the sandwich panel cross-section is equivalent to a hot-rolled steel section that can develop its full elastic moment resistance. On the other hand, M2 is the most conservative, as it assumes that the sandwich panel cross-section is equivalent to a cold-formed steel built-up member without core material. M3 represents an intermediate state between the previous two methods.

The validation of the SLS analytical design is carried out in terms of vertical deflection and sandwich panel's stiffness. The equivalent SLS load level (F_{SLS}) for the structural model of the four-point bending test is obtained by considering the characteristic load combination (q_{SLS}) in Equation (3.7) and applying Equation (5.3):

$$F_{SLS} = q_{SLS} \cdot L \quad (5.3)$$

where L is set equal to the distance between the support in the four-point bending test. The experimental vertical midspan deflection (δ_{SLS}) for the corresponding SLS level may then be extracted from the load-mid span displacement curve. The analytical prediction of the of the vertical deflection of the sandwich panel is calculated according to the structural model of a simply supported subjected to two concentrated loads applied at thirds of the span:

$$\delta_{Mi} = \frac{23 \cdot F_{SLS} \cdot L^3}{1296 \cdot E \cdot I_{Mi}} \text{ with } i = 1,2,3 \quad (5.4)$$

where δ_{Mi} and I_{Mi} are the vertical deflection at midspan and the moment of inertia of the cross-section determined according to M1, M2, and M3. Finally, the analytical prediction of the bending stiffness is obtained as follows:

$$K_{Mi} = \frac{1296 \cdot E \cdot I_{Mi}}{23 \cdot L^3} \quad (5.5)$$

where K_{Mi} is the analytical flexural stiffness according to M1, M2, and M3.

The comparison of the analytical design results and the results of the four-point bending test are reported in Table 5. 2 and Table 5.3.

Table 5. 2. Comparison between analytical predictions of ultimate load and experimental results.

Specimen	F_u [kN]	F_{uLS} [kN]	Exp/AD [-]
SP1	30.73	22.22	1.38
SP2	31.39	22.22	1.41
Avg.	31.06	22.22	1.40

Notes: Avg. – average value per set of specimens; Exp/AD – ratio between experimental result and analytical design prediction.

Table 5.3. Comparison between analytical predictions of stiffness and experimental results.

δ_{SLS} [mm]	Analytical prediction [mm]			K [kN/mm]	Analytical prediction [kN/mm]		
	δ_{M1}	δ_{M2}	δ_{M3}		K_{M1}	K_{M2}	K_{M3}
0.20	0.21 (0.96)	0.26 (0.77)	0.24 (0.83)	11.8	16.5 (0.72)	13.2 (0.89)	14.2 (0.83)

Notes: values between parentheses are the corresponding ratio with experimental measurements.

The experimental load-carrying capacity is about 1.4 times higher than the analytical prediction. It is important to note that previous research has shown that the ratio between predictions based on the EWM and the experimental flexural resistance of hollow built-up CFS box section is generally around 1.0 when the ratio of width to thickness is less than or equal to 150 [77], [82], [83]. The CFS C-sections constituting the sandwich panels have a width-to-thickness ratio of approximately 500. Therefore, it is evident that neglecting the PUR foam contribution leads to a conservative prediction of the flexural load-carrying capacity of the panel. Furthermore, the results highlight the efficiency of the adhesive joint in transferring the loads between the top and bottom components of the sandwich panels.

Regarding the vertical midspan deflection corresponding to the SLS load level, all the methods considered overestimate the experimental result, which is well within the limit of the Portuguese version of the Eurocode 3 Part 1-1 (2000) [103] for the span length considered (9 mm). Therefore, the analytical

prediction of the vertical deflection was conservative. On the other hand, for the load interval considered for the estimation of the flexural stiffness (20% - 45%), all the methods overestimate the sandwich panel's stiffness. Therefore, the predictions were unconservative. The assumption of the PUR foam as a rigid medium is not adequate since M1 predicts a stiffness which is about 40% higher than the experimental one. On the other hand, M2 which does not include the PUR foam in the calculation of the bending stiffness present a smaller relative difference (about +12%). Additionally, all the analytical methods, do not consider geometrical nonlinear effect which may be significant for the load interval selected for the estimation of the experimental stiffness.

5.2. FULL-SCALE SANDWICH PANELS

The testing of small-scale sandwich panels through four-point bending tests confirmed the good quality of the manufacturing process used by Ferpainel in producing the prototypes. The results of the analytical design were found to be consistent with the experimental results in terms of load-carrying capacity and vertical deflection. Therefore, it was concluded that the sandwich panel layout was suitable to produce full-scale panels. The final prototypes were subjected to different loading configurations over a span length of 4.0 m, and the results of the experiments were compared with the predictions of the analytical design, which are provided in the following sections.

5.2.1. Three-point variable span bending test

The variable span length flexural test follows the recommendations of EN 13706-2 (2002) [192]. However, modifications were adopted specifically for the sandwich panel object of this study. According to the test methodology, it is assumed that the deflection caused by bending contributes to the overall deflection for large span lengths, whereas the deflection due to shear is the largest part of the total deflection for short span lengths. If the panel span length is varied, both effective bending and shear stiffness can be estimated based on the Timoshenko beam theory.

Three sandwich panels with a length of 4.5 m were tested. The panels were weighed before the test and a density of 44.4 kg/m³ was estimated (relative difference with nominal density +11%). Simply supported boundary conditions were materialised by steel plates with a cross-section of 10 mm × 100 mm placed on top of steel rollers to allow free rotation. The three-point bending loading scheme was employed for the following span lengths: i) 4.0 m, ii) 3.5 m; and iii) 3.0 m. The load was applied by a hydraulic jack

and monitored by a load cell of 200 kN with a precision of 0.05%. A steel IPE beam was used to distribute the load across the width of the panel. Vertical deflections were monitored at midspan by LVDTs with a stroke of 50 mm. Vertical displacements were also registered at the supports to monitor possible settlements in these locations. Longitudinal strains were recorded on the cold-formed steel top and bottom face sheets across the width of the panel. The overall test setup and the schematic of the employed instrumentation are shown in Figure 5.8 and Figure 5.9, respectively.

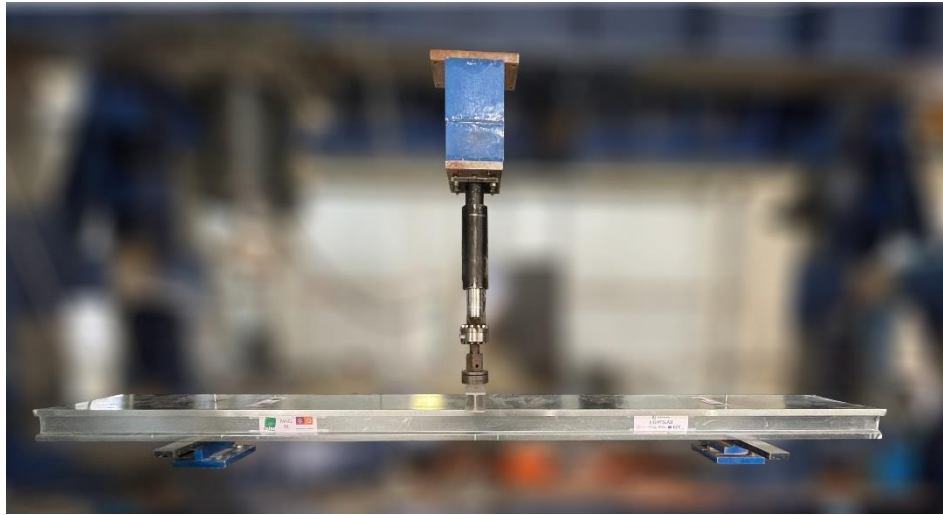


Figure 5.8. Three-point variable span bending test setup.

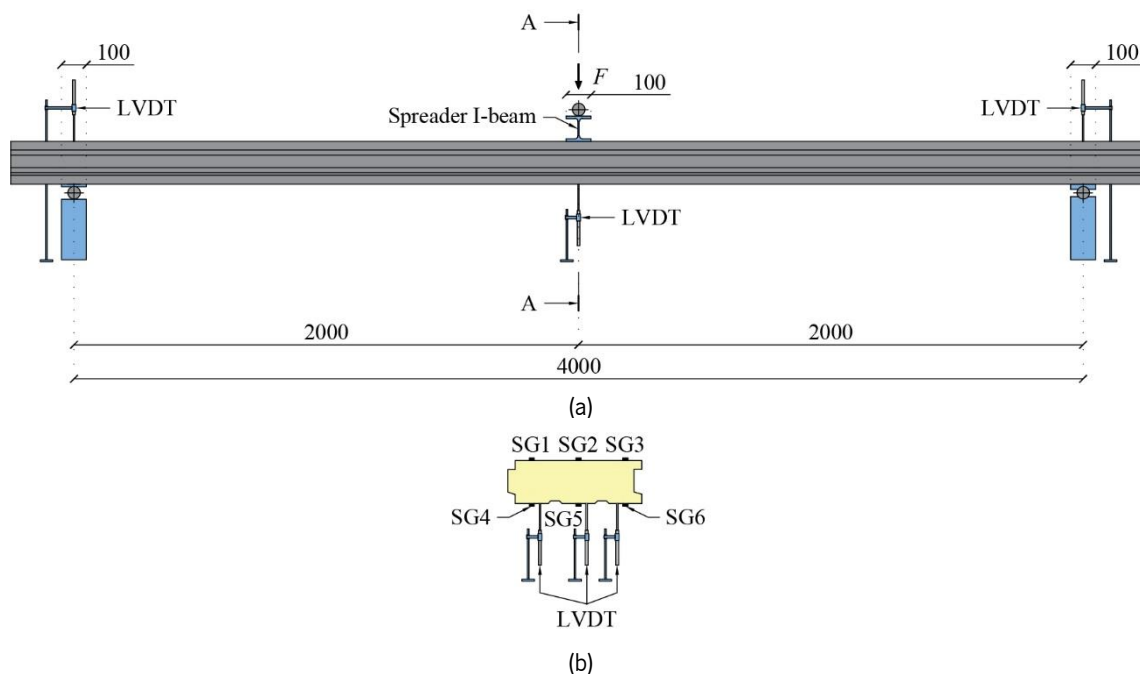


Figure 5.9. Three-point variable span bending test instrumentation: (a) lateral view; (b) AA section. All units in [mm].

The design of CFS structural members assumes that the effective bending stiffness of the sandwich panel depends on the stress level in the compressed elements of the cross-section. To ensure that the effective

bending stiffness remains constant during the experiments, the load was adjusted for different span lengths to produce the same value of the bending moment at midspan.

The value of the bending moment was established based on the preliminary four-point bending test result described in Section 5.1. The bending moment was established as half of the ultimate bending moment ($M_{4PB,u}$) registered during the small-scale four-point bending test to ensure that the sandwich panel would stay within its elastic regime. The corresponding value of the concentrated force (F_{3PB}) is obtained by substituting $M_{4PB,u}$ in the equation of the bending moment diagram of a simply supported beam subjected to a concentrated force at midspan:

$$F_{3PB} = \frac{2 \cdot M_{4PB,u}}{L} \quad (5.6)$$

In order to investigate the assumption of the EMW proposed by Eurocode 3 Part 1-3 (2006) [80], which suggests that the effective bending stiffness is dependent on the stress level in the compressed element of the cross-section, the sandwich panels were subjected to three different levels of concentrated forces for each span length. The aim was to produce three different levels of bending stresses at the midspan (M_{3PB}) cross-section, namely 4.5 kN·m, 3.0 kN·m, and 1.5 kN·m.

The loading procedure consisted of three loading-unloading cycles with a pause of 20 s between them. The loading rates were estimated to reach the maximum load in 90 s, ranging from 1.0 kN/min to 4.0 kN/min. The summary of the applied forces for each span length is summarised in Table 5.4.

Table 5.4. Summary of applied forces and loading rates of the three-point variable span bending test.

	$L = 4.0 \text{ m}$		$L = 3.5 \text{ m}$		$L = 3.0 \text{ m}$	
M_{3PB} [kNm]	F_{3PB} [kN]	v [kN/s]	F_{3PB} [kN]	v [kN/s]	F_{3PB} [kN]	v [kN/s]
4.5	4.50	3.00	5.14	3.43	6.00	4.00
3.0	3.00	2.00	3.43	2.29	4.00	2.67
1.5	1.50	1.00	1.71	1.14	2.00	1.33

Notes: L – span length; v – loading rate.

The deflection at midspan (δ) of a simply supported beam subjected to a concentrated load (F) is given by Equation (5.7):

$$\delta = \frac{FL^3}{48D} + \frac{FL}{4U} \quad (5.7)$$

where D and U are the effective bending stiffness and shear stiffness, respectively. To plot the experimental results obtained for different span lengths, Equation (5.7) can be rearranged as follows:

$$\frac{\delta}{FL} = \frac{L^2}{48D} + \frac{1}{4U} \quad (5.8)$$

This results in a graph with L^2 as the x-axis and δ/FL as the y-axis. A linear pattern with a slope equal to $1/48D$ and an intercept equal to $1/4U$ should be followed by the points obtained for different span lengths. The results obtained by applying different bending moments are illustrated in Figure 5.10.

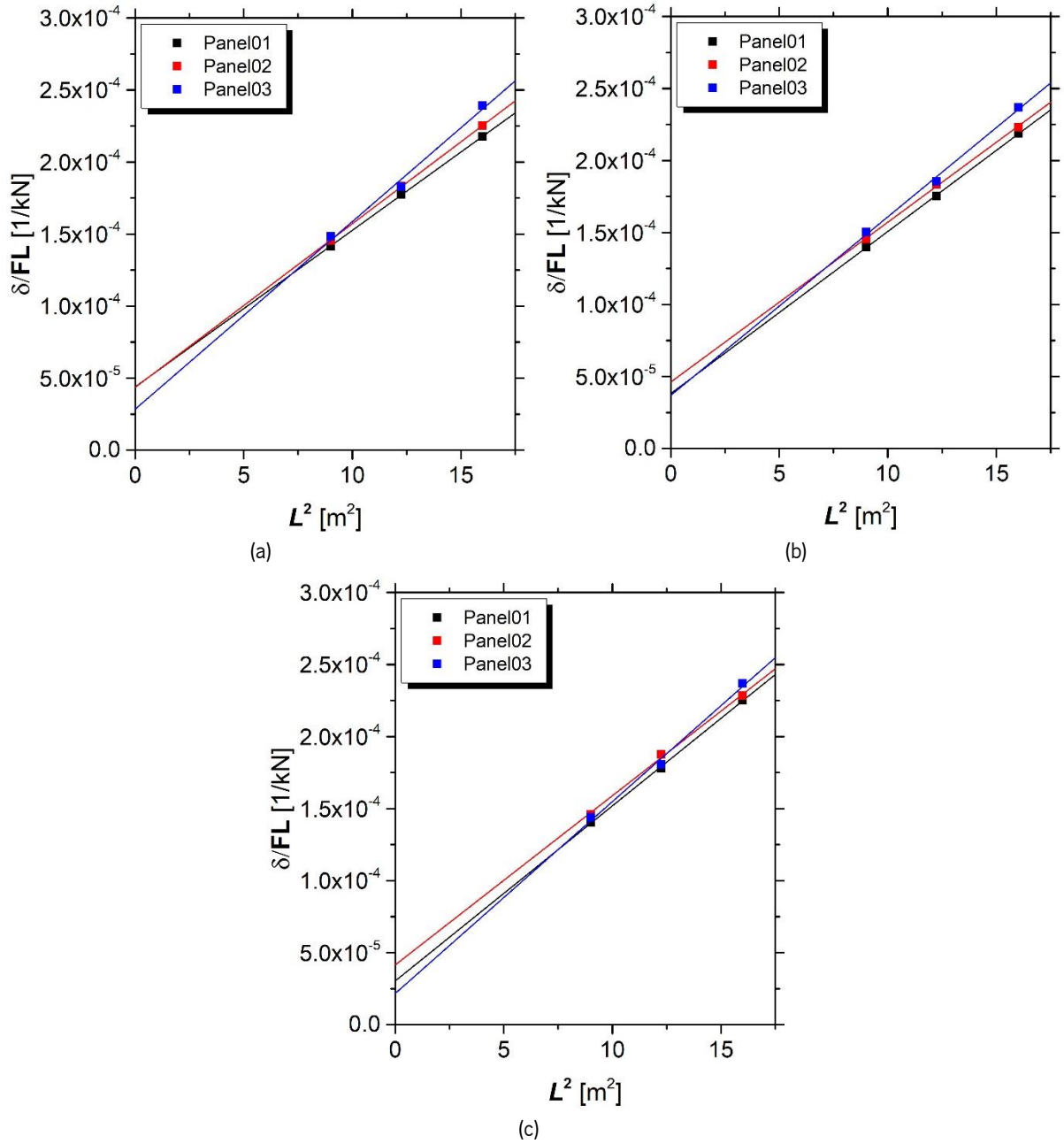


Figure 5.10. $L^2 - \delta/FL$ axes plot of the results: (a) 4.5 kN·m bending moment test; (b) 3.0 kN·m bending moment test; (c) 1.5 kN·m bending moment test.

The experimental data points of each panel were subjected to a linear regression analysis, yielding a coefficient of determination (r^2) greater than 0.99 [193]. The correspondent values of the effective bending stiffness and shear stiffness based on the linear regression analysis and the comparison with the analytical estimations are reported in Table 5.5.

Table 5.5. Comparison between analytical predictions of bending and shear stiffnesses and experimental results.

		Panel01	Panel02	Panel03	Avg. (CoV)	Exp/AD
4.5 kNm	D [kNm ²]	1915.9	1833.1	1600.9	1783.3 (9.2%)	1.04
	U [kN]	5698.3	5741.7	8749.0	6729.7 (26.0%)	0.80
3.0 kNm	D [kNm ²]	1851.2	1877.0	1681.6	1803.3 (5.9%)	1.06
	U [kN]	6533.6	5412.6	6750.8	6232.3 (11.5%)	0.74
1.5 kNm	D [kNm ²]	1714.5	1774.2	1565.5	1684.7 (6.4%)	0.99
	U [kN]	8227.7	6013.7	11461.1	8567.5 (32.0%)	1.01
Analytical	D [kNm ²]	1707.4				
	U [kN]	8458.2				

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation; Exp/AD – ratio between experimental result and analytical design prediction.

The effective bending stiffness and shear stiffness of the sandwich panel may be calculated according to the following equations:

$$D = EI \quad (5.9)$$

$$U = \frac{A_v G}{\alpha} \quad (5.10)$$

where A_v is the cross-sectional area of the webs, E is the elastic modulus of steel, G is the shear modulus of steel, I is the moment of inertia of the cross-section, and α is a form factor expressing the distribution of the shear stresses throughout the web of the section [194]. The analytical predictions (see Table 5.5) considered a Young's modulus of 210 GPa and a shear modulus of 81 GPa according to the Eurocode 3 Part 1-1 (2005) [113]. Furthermore, based on the result of the preliminary four-point bending test described in Section 5.1, the calculation of I considers the properties of the gross cross-section.

It appears that the level of stresses in the compressed elements does not have an influence on the effective bending stiffness of the sandwich panel, as the estimated values for the different applied bending moments are similar (with a coefficient of variation of 3.6%). Additionally, the experimental results demonstrate good agreement with the analytical predictions of D (effective bending stiffness).

The average ratio between the experimental results and the analytical calculations is 1.03. This indicates that the design is slightly conservative. This finding agrees with the theory of sandwich panels with a weak core. According to the theory, the contribution of the core to the bending stiffness is less than 1% [189], thus justifying the decision to neglect the contribution of the PUR foam in the preliminary design.

A larger scatter in the experimental results of the shear stiffness of the sandwich panels is observed (overall coefficient of variation of 17.1%). Since the shear stiffness of the panel is mostly attributable to the CFS webs, the relatively large scatter could be decreased by considering shorter span lengths in future studies. Indeed, for the shortest of the span length adopted (where the shear contribution to the overall deflection is the highest), the ratio between the experimental and analytical calculations is 1.01. Furthermore, the calculation of the form factor based on the assumption that the distribution of the shear stresses resembles those of a box beam may be oversimplistic due to the presence of the stiffeners.

The load-longitudinal strain curves for the first loading cycle of Panel01 and Panel03 are compared to the analytical predictions in Figure 5.11 and Figure 5.12, respectively. The analytical compressive (ε_{top}) and tensile strains ($\varepsilon_{\text{bottom}}$) of the face sheets were calculated according to the Euler-Bernoulli beam theory as follows:

$$\varepsilon = \frac{M_{3PB}y}{EI} \quad (5.11)$$

where y is the distance between the neutral axis and the extreme fibres of the cross-section. Panel02 presented a similar behaviour to that of Panel01. Therefore, its load-strain curve is omitted. Panel03 presents a bending stiffness consistently lower than Panel01 and Panel02 for all the applied bending moments. The difference is reflected in the measurements of the strain gauges in the compressed face sheet. SG1 and SG3 present higher levels of longitudinal strain in Panel01 compared to SG2. This is an expected result since, according to the EWM, the highest stresses in the plate cross-section are located at the supported edges. On the other hand, the longitudinal strains registered in Panel03 show a gradient of strains across the cross-section. This may indicate that one of the adhesive joints between the top plain C-section and the bottom-lipped C-section was not able to transfer the stresses adequately.

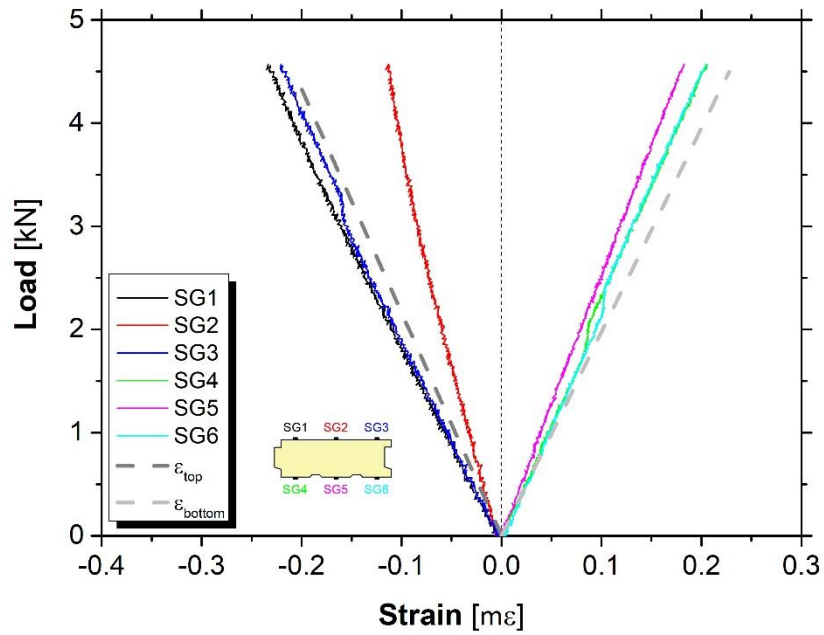


Figure 5.11. Load-longitudinal strain curve of Panel01 for a span length of 4.0 m and an applied bending moment of 4.5 kN·m.

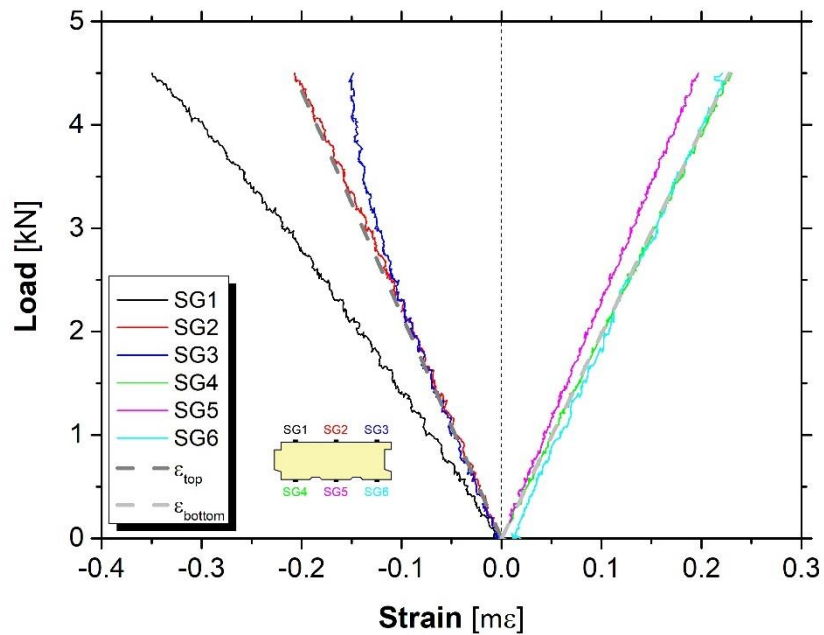


Figure 5.12. Load-longitudinal strain curve of Panel03 for a span length of 4.0 m and an applied bending moment of 4.5 kN·m.

5.2.2. Flexural dynamic test

The same sandwich panels used for the estimation of the effective bending and shear stiffness were subjected to flexural dynamic tests. The natural frequencies of the sandwich panel were assessed according to the operational modal analysis. This modal identification technique relies on the

measurements of the response of the structure to ambient vibration caused by natural excitation sources near the structure [195]. It is a cost-effective technique often used in civil engineering applications, although it does not allow to fully characterise the modal parameters as the ambient forces are not being measured [196].

Simply supported boundary conditions were adopted for the flexural dynamic tests similar to the three-point variable span bending test. The panels were tested over a span length of 4.0 m. The vertical accelerations were monitored by means of nine accelerometers with a stroke ranging from 0.1 Hz to 4800 Hz. The response was monitored using a sampling frequency of 600 Hz. Numerical simulations, described in Chapter 7, were carried out to establish the monitoring locations of the accelerometers on the sandwich panels. Accelerometers were placed at midspan, quarter-span, and at the supports to capture the vibration frequencies of the sandwich panels. Each monitored section had at least two accelerometers, one per panel side, to capture the first torsional frequency. A summary of the test setup and instrumentation are shown in Figure 5.13 and Figure 5.14, respectively.

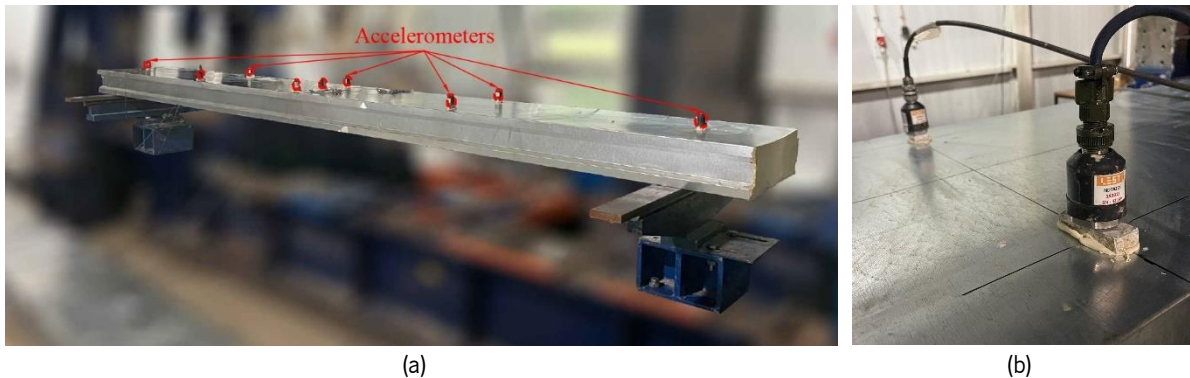


Figure 5.13. Flexural dynamic test setup: (a) overall view; (b) accelerometers.

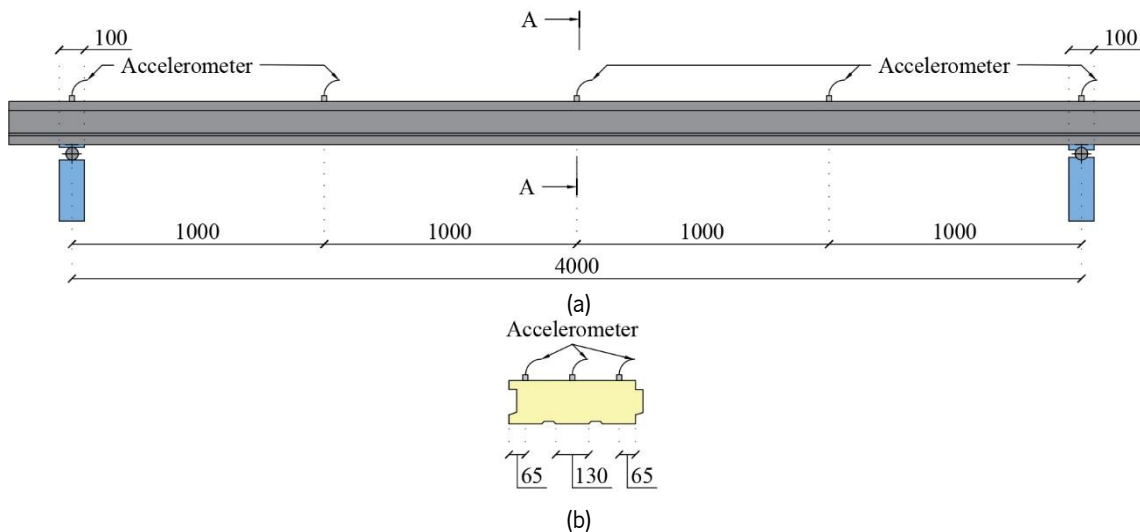


Figure 5.14. Flexural dynamic test instrumentation: (a) lateral view; (b) section A-A. All units in [mm].

The accelerometer recordings were processed using the commercial software Artemis Modal (Version 6.0). The typical singular values of the spectral density function matrix are plotted in Figure 5.15.

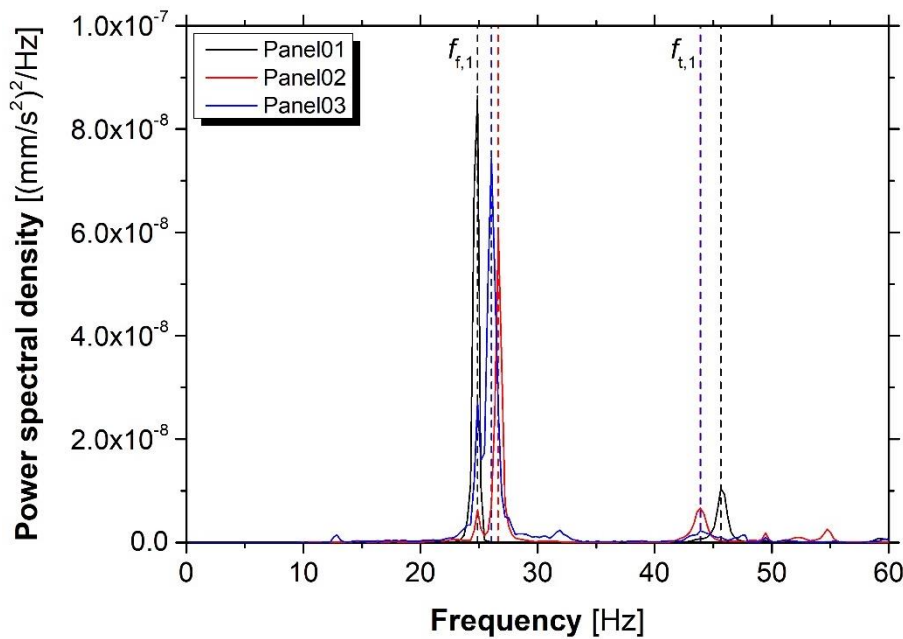


Figure 5.15. Typical power spectral density curves of the sandwich panels.

The frequency domain decomposition technique is adopted to obtain the mode shapes and natural frequencies of the sandwich panel. The method assumes that the natural frequencies are well-spaced in frequency and that the contribution of other modes in the vicinity of the natural frequency is null [197].

The identified peaks above 60 Hz did not meet the assumption of closed-mode orthogonality [197]. Therefore, the results were not included. The mode shapes of the first two vibration frequencies are shown in Figure 5.16. The first vibration mode is of pure bending, whereas the second mode presents torsional mode shape characteristics. The results of all the repetitions for the three sandwich panels, as well as the analytical prediction of the first flexural frequency ($f_{f,1}$), are presented in Table 5.6.

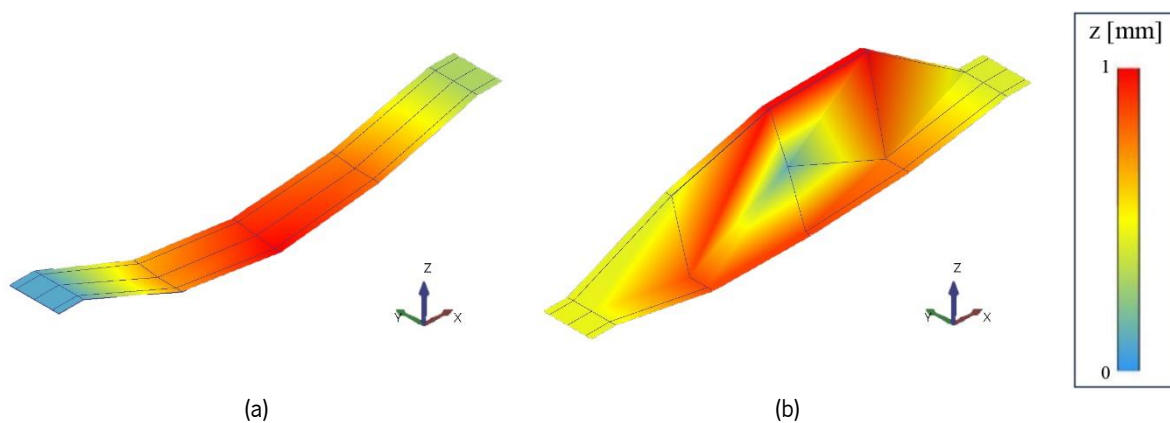


Figure 5.16. Typical shapes obtained from Artemis Modal (Version 6.0): a) first mode; b) second mode.

Table 5.6. Summary of the flexural dynamic test and analytical predictions.

		Panel01		Panel02		Panel03		Avg. (CoV)	Exp/AD
		Test #1	Test #2	Test #1	Test #2	Test #1	Test #2		
Dynamic test [Hz]	$f_{f,1}$	26.4	24.5	27.0	26.4	25.8	25.5	25.9 (3.3%)	0.78
	$f_{t,1}$	44.8	45.5	43.9	43.7	43.7	44.8	44.4 (1.7%)	-
Analytical [Hz]	$f_{f,1}$	33.2							

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation; $f_{f,1}$ – first flexural frequency; $f_{t,1}$ – first torsional frequency.

Regarding the analytical estimation of $f_{f,1}$, the same cross-section and material properties are considered as those used in Section 5.2.1. The solutions of the equation of the dynamic equilibrium of the vibrating beam using the method of separation of variables [198] yields the first natural frequency of the beam (ω_1) according to Equation (5.12):

$$\omega_1 = \pi^2 \cdot \sqrt{\frac{EI}{\bar{m}L^4}} \quad (5.12)$$

where $\bar{m}$ is the mass per unit area of the sandwich panel. The flexural frequency $f_{f,1}$ may then be estimated according to Equation (5.13):

$$f_{f,1} = \frac{\omega_1}{2\pi} \quad (5.13)$$

The results showed a first vibration mode of pure bending with a frequency of 25.9 Hz. The analytical estimation of the first vibration frequency is 21.9% higher than the experimental result. This may be due, among other reasons, to the fact that the sandwich panels presented a core material with a density 11% higher than the nominal density considered in the estimation of $\bar{m}$. Therefore, the greater overall weight may decrease the vibration frequency.

According to the Portuguese version of the Eurocode 3 Part 1-1 [103], structural horizontal elements are required to meet the minimum natural frequency requirements for residential floors, which should be greater than 3 Hz. Based on this requirement, it can be concluded that the sandwich panels largely fulfil the limit imposed by the code.

5.2.3. Four-point bending test

After carrying out the previous tests (three-point variable span bending test and flexural dynamic test), the one-way static behaviour of sandwich panels was studied up to failure. Four-point bending tests were carried out with the loaded section located at thirds of the span, with a clear span of 4.0 m. Simply supported boundary conditions were adopted, similar to the three-point variable span bending test.

The load was applied by a hydraulic jack and monitored by a load cell of 200 kN with a precision of 0.05%. A steel IPE beam, steel rollers and prismatic steel plate with a cross-section of 10 mm × 100 mm were used to transfer the load to the panels. Additionally, a thin layer of bitumen-based polyester putty between the panel and the steel plates was applied to ensure full and even contact in the loading zones. The sandwich panels were loaded monotonically up to failure at a rate of 1.8 mm/min.

Vertical displacements were recorded by six LVDTs with a stroke ranging from 50 mm to 100 mm, located at midspan and under the loaded sections. Furthermore, two LVDTs with a stroke of 10 mm were placed on the support to measure possible settlements. Six strain gauges were used to monitor the longitudinal strain at midspan, three at the top face sheet and three at the bottom face sheet. The overall experimental setup and the employed instrumentation are illustrated in Figure 5.17 and Figure 5.18, respectively.

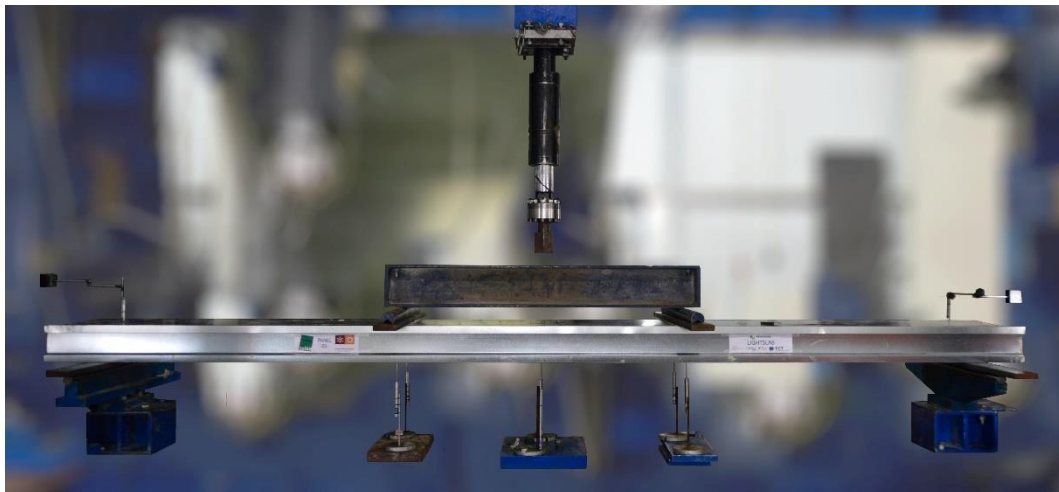


Figure 5.17. Four-point bending test setup.

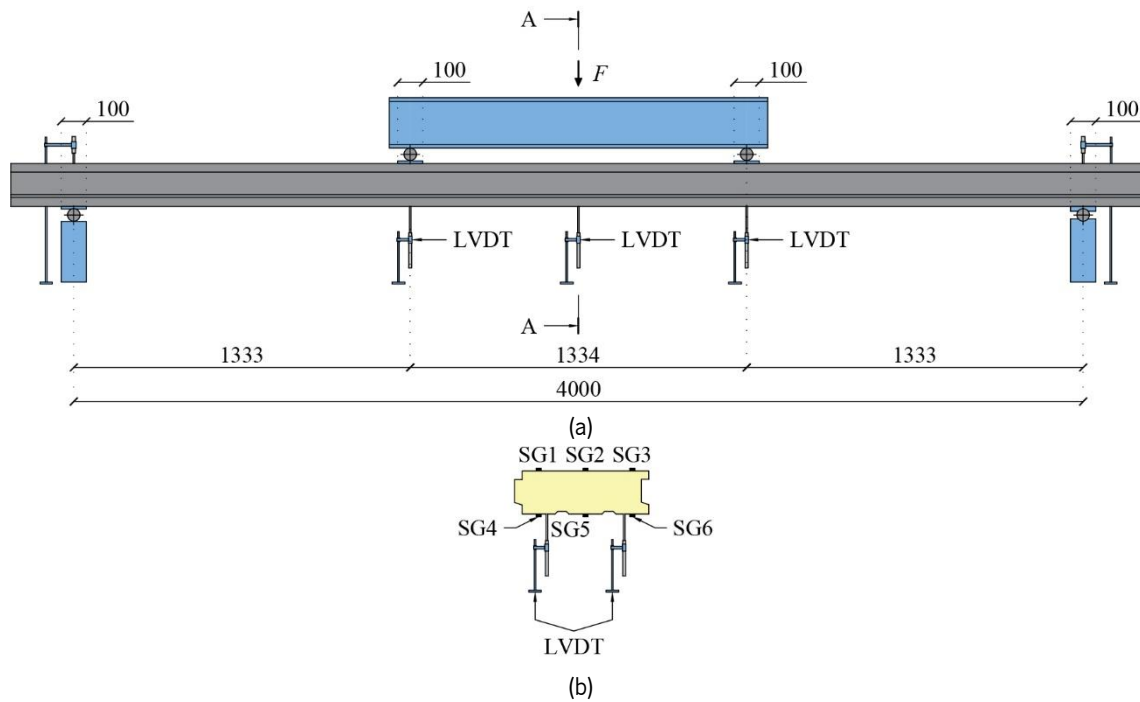


Figure 5.18. Four-point bending test instrumentation: (a) lateral view; (b) AA section. All units in [mm].

The load-midspan displacement curves and the typical load-longitudinal strain curves of the sandwich panels are presented in Figure 5.19 and Figure 5.20, respectively.

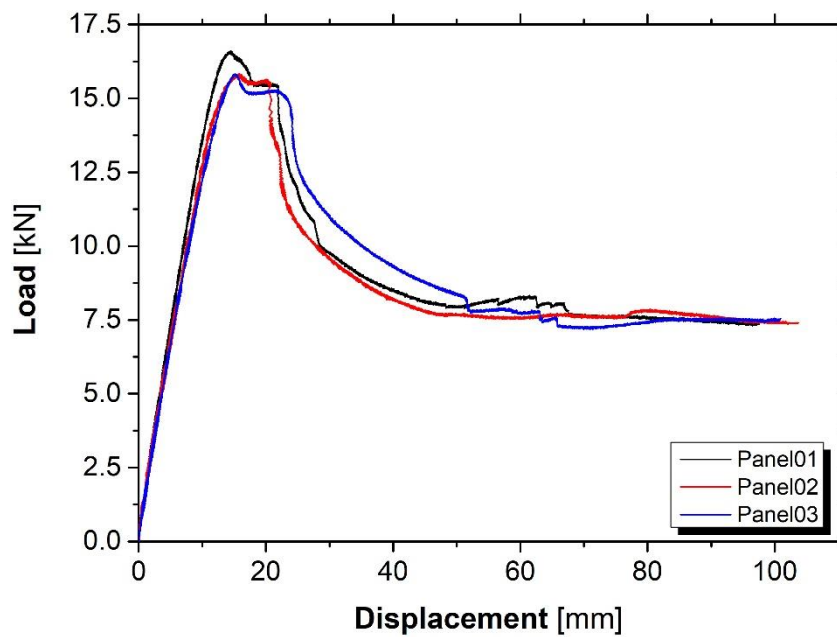


Figure 5.19. Load-displacement curves of the sandwich panels.

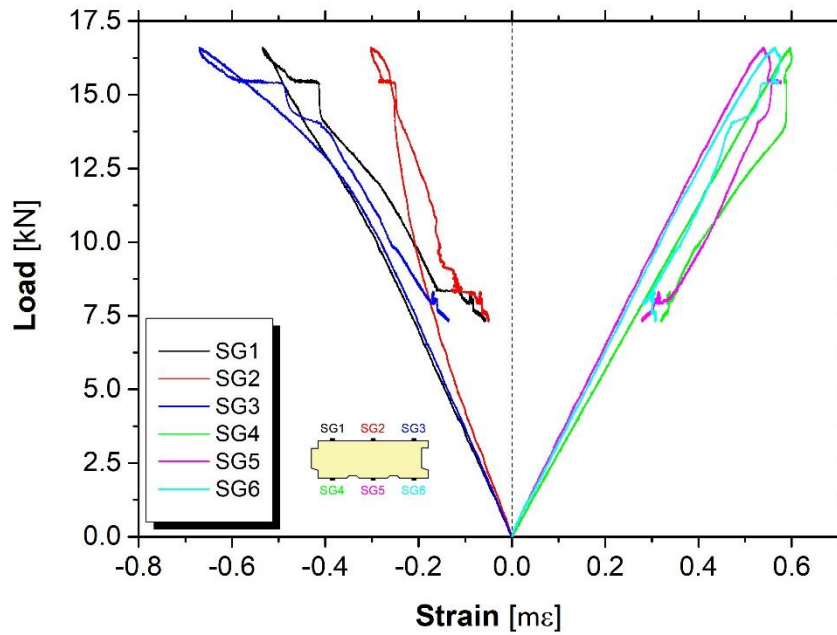


Figure 5.20. Typical load-longitudinal strain curve of the sandwich panels.

The typical failure progression of the sandwich panel is shown in Figure 5.21. A linear elastic behaviour is observed up to approximately 10 kN. As the loading increased, the sandwich panels presented a gradual loss of stiffness up to the attainment of the ultimate load. The reduction in stiffness is reflected by the nonlinear load-longitudinal strain relationship highlighted by the strain measurements in the compressed face sheet. This may be attributed to the wrinkling of the top face sheet between the loaded points. At the ultimate load the sandwich panels failed due to local buckling at one of the loaded sections highlighted by the outward deformation of the flanges of the top plain C-section (see Figure 5.21b-1). The plateau after the ultimate load is attributed to the crushing of the PUR foam under the loading regions (see Figure 5.22a and Figure 5.22b). The second load drop corresponds to the local buckling of the webs of the sandwich panel at one of the loaded sections (see Figure 5.21b-2). Local buckling appears gradually and results in the formation of a plastic hinge. The outward deformation of the top face sheet is most likely due to the failure of the top face sheet-to-core interface (see Figure 5.22c and Figure 5.22d). This result appears to be consistent with the findings of the flatwise tensile and edgewise compressive tests where cohesive cracks of the PUR foam were located near the top face sheet.

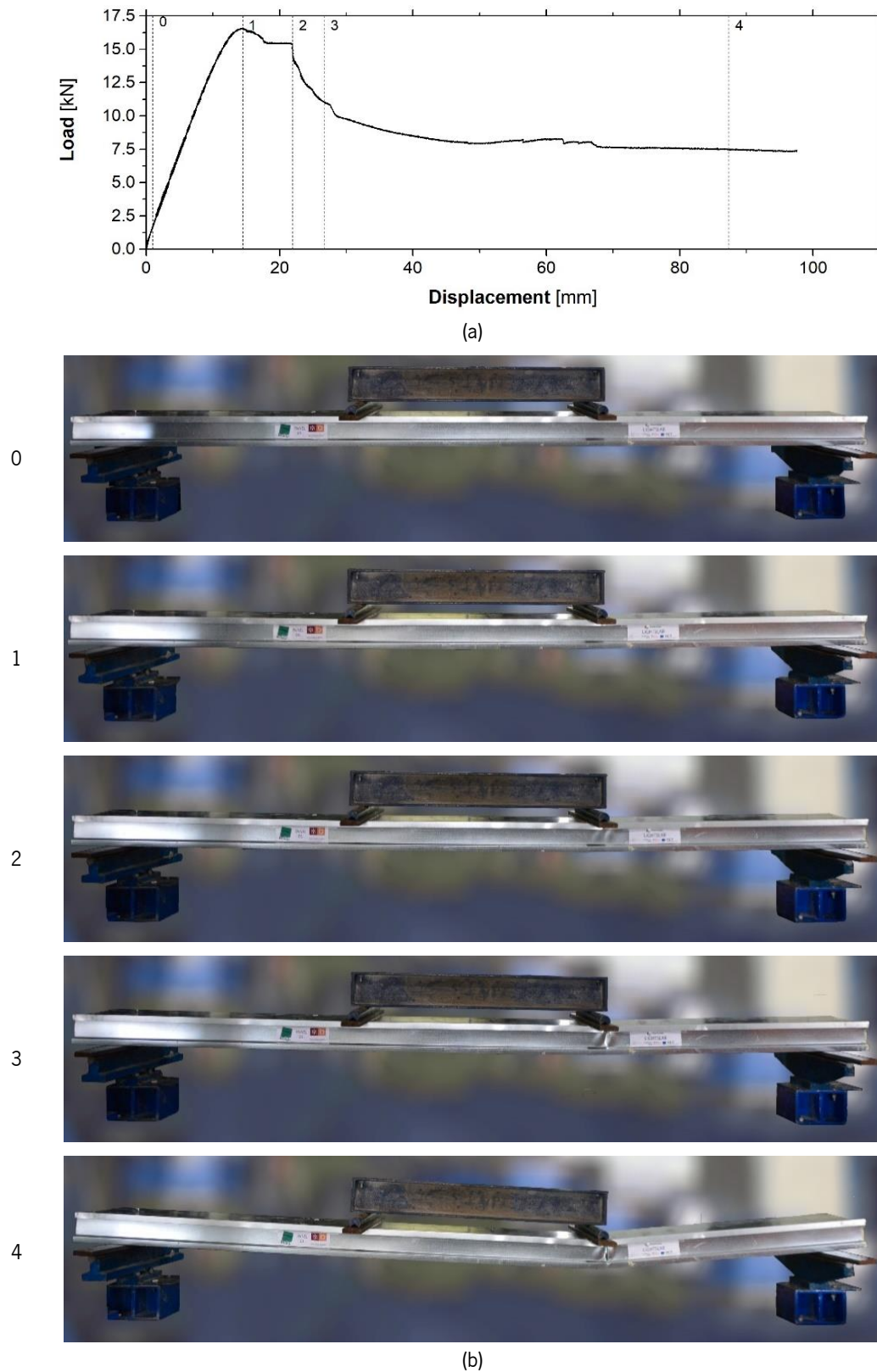


Figure 5.21. Evolution of failure mode of Panel01: (a) load-midspan displacement curve; (b) failure progression.

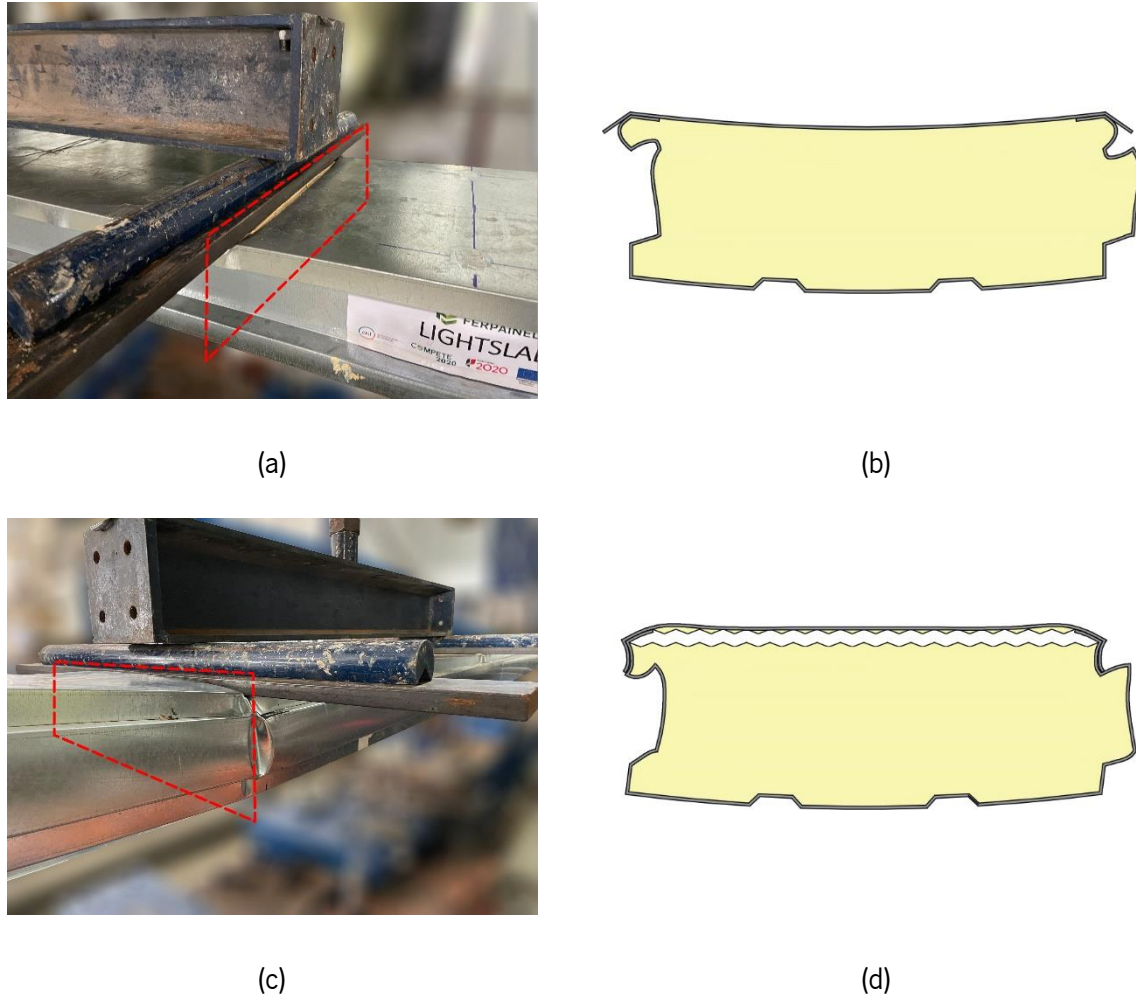


Figure 5.22. Progressive failure of the sandwich panel: (a) interest zone at ultimate load; (b) local buckling of the top plain C-section with outward deformation of the flanges; (c) interest zone at the second load drop; (d) local buckling of the webs and cohesive PUR foam crack near the top face sheet.

The summary of the obtained results from the four-point bending test is presented in Table 5.7, including the initial stiffness calculated as the slope of the load-midspan displacement curve in the load interval between 20% and 45% of the ultimate load (K), the ultimate load (F_u), the deflection at ultimate load (δ_u), the compressive strain at ultimate load (ϵ_{top}), and the tensile strain at ultimate load (ϵ_{bottom}).

The coefficient of variation of the initial stiffness, ultimate load and displacement at ultimate load are less than 5%. This result highlights the consistency in the flexural performance of the sandwich panels and the high repeatability of the manufacturing process. On the other hand, a high coefficient of variation is observed in the measured compressive strains at ultimate load. The experimental compressive strains represent a localized measurement which is more influenced by the presence of initial geometrical imperfections. The scatter is mostly attributed to the performance of Panel03 which had already shown

a different strain pattern in the three-point bending test. The possible reasons for the anomalous behaviour are provided in Section 5.2.1.

Table 5.7. Summary of the results of the four-point bending test.

Specimen	K [kN/mm]	F_u [kN]	δ_u [mm]	ε_{top} [mε]	ε_{bottom} [mε]
Panel01	1.40	16.60	14.49	-0.502	0.567
Panel02	1.28	15.83	15.81	-0.302	0.569
Panel03	1.30	15.82	15.21	-0.691	0.630
Avg. (CoV)	1.33 (4.7%)	16.08 (2.8%)	15.17 (4.3%)	-0.498 (39.0%)	0.589 (6.1%)

Notes: Avg. – average value per set of specimens; CoV – coefficient of variation.

5.2.4. Analytical modelling

As previously stated, the objective of the flexural test up to failure was the investigation of the ultimate load-carrying capacity of the sandwich panel and its collapse mechanisms, as well as the validation of the analytical design method. The comparison follows the same approach used for the analysis of the results of the preliminary four-point bending test described in Section 5.1.3. It assumes the same structural model, i.e. a simply supported beam subjected to two concentrated loads applied at thirds of the span length. The ultimate load-carrying capacity (F_{ULS}), the equivalent SLS load level for the structural model of the four-point-bending test setup (F_{SLS}), the analytical prediction of the vertical deflections (δ_{Mi}), and the experimental and analytical stiffness (K_{Mi}) of the sandwich panel are determined according to Equation (5.1) to (5.5). The experimental vertical deflection corresponding to F_{SLS} (δ_{SLS}) and the experimental stiffness (K) are extracted from the load-displacement curve in the same fashion as described in Section 5.1.3. Furthermore, the longitudinal strains on the top and bottom face sheets at the ultimate load are estimated according to Equation (5.14) considering the gross cross-section properties:

$$\varepsilon = \frac{M_{4PB,u} \cdot y}{EI} \quad (5.14)$$

The measured and calculated deflection at the SLS, initial elastic stiffness, ultimate load, and compressive and tensile strains at the ultimate load are compared in Table 5.8. For what concerns the comparison with the analytical prediction, the ultimate force (F_u) registered in the experiments is almost 50% higher than that estimated in the preliminary design. These results are consistent with those of the preliminary four-point bending test (see Section 5.1.3). Therefore, a conservative estimation of the ultimate load-carrying capacity of the panel is obtained by neglecting the contribution of the PUR foam.

Regarding the vertical deflection at the SLS load level (δ_{SLS}) and the sandwich panel stiffness (K), the experimental results are included in the range delimited by the analytical values obtained considering the PUR foam as a rigid medium (M1) and neglecting the contribution of the PUR foam (M3). It is also worth noting that the prediction of the longitudinal strains according to M1 is in good agreement with the experimental results. This suggests that the location of the neutral axis, as determined by the properties of the gross cross-section (I_{M1}), is reliable. The discrepancy found in the estimation of the δ_{SLS} and K of the sandwich panel according to M1 may be due to the presence of initial geometrical imperfections which are not considered in the analytical model.

Table 5.8. Comparison between analytical predictions and experimental results obtained from the four-point bending test.

Parameters	Experimental	Analytical	Exp/AD
δ_{SLS} [mm]	5.44	5.06 (M1)	1.08 (M1)
		6.15 (M3)	0.88 (M3)
K [kN/mm]	1.33	1.50 (M1)	0.88 (M1)
		1.24 (M3)	1.07 (M3)
F_u [kN]	16.08	11.03	1.46
ε_{top} [mε]	-0.498	-0.502 (M1)	0.99 (M1)
ε_{bottom} [mε]	0.589	0.549 (M1)	1.07 (M1)

Notes: Exp/AD – ratio between experimental result and analytical design prediction.

5.3. CONCLUDING REMARKS

Small-scale and full-scale tests were carried out according to different loading configurations to evaluate the flexural performance of the sandwich panels. The data collected from these tests was then used to verify the accuracy of the analytical models employed in the initial design phase.

The small-scale four-point bending test results validated the predictions of the analytical design in terms of midspan deflection and ultimate load. Neglecting the contribution of the PUR foam for the estimation of the load-carrying capacity of the sandwich panel lead to conservative predictions. On the other hand, considering the stabilising effect of the PUR foam on the plate elements of the cross-section lead to accurate estimation of the vertical deflection at midspan for SLS load level.

The effective bending stiffness of the sandwich panel was estimated through analytical methods and compared to the results of a three-point variable span bending test. The findings showed a good agreement between the two, suggesting that the adhesive joints were successful in transferring the load between the two CFS components of the cross-section. Additionally, the PUR foam acted as a rigid

medium, preventing any instability phenomena in the compressed element of the cross-section, without causing any reduction in the gross cross-section properties. However, the shear stiffness of the sandwich panel yielded less consistent results. The experimental and analytical values were found to differ, which could be attributed to oversimplifying assumptions made during the calculation of the shape factor for the shear stress distribution in the webs of the sandwich panel. To obtain more accurate results, future studies should assess span lengths shorter than 3.0 m, as well as a more accurate estimation of the shear stress distribution in the webs of the sandwich panels.

The first flexural and torsional frequencies of the sandwich panels were identified through dynamic testing. The obtained vibration frequencies are higher than the common CFS floor system found in the literature for tested span lengths slightly higher [199], [200] and lower [201] than that considered in this research. The average first natural frequency of the sandwich was found to be more than eight times greater than the limit prescribed by the Portuguese version of the Eurocode 3 Part 1-1 [103]. This implies that the sandwich panel floor system should ensure the occupants' comfort with respect to vibration responses.

In the four-point bending tests, the various sandwich panels exhibited similar failure modes. Local buckling of the top face sheet and webs at one of the loaded sections occurred gradually, ultimately leading to the formation of a plastic hinge. Additionally, debonding between the top face sheet and the core was noticeable. The sandwich panel's top face sheet-to-core interface is a weak point, as seen in the test results for flatwise tensile and edgewise compressive tests. Further studies should focus on the PUR foam-to-metal interface to accurately predict the strength of the sandwich panel. Generally, the three tested panels ultimately registered similar ultimate loads and longitudinal strains at failure, indicating a manufacturing process of high repeatability.

The analytical predictions that were carried out without taking into consideration the contribution of the PUR foam led to a conservative estimation of the ultimate load of the sandwich panels, which was about 50% smaller than the actual value. On the other hand, the prediction of the flexural stiffness, considering the gross cross-section properties, generally resulted in an accurate estimation. Overall, the experimental results obtained from the small- and full-scale sandwich panels validated the predictions carried out in the preliminary design.

CHAPTER 6

6. CONNECTION SYSTEMS FOR FLOOR SANDWICH PANELS

Prefabricated modular structural systems have a limited size imposed by the manufacturing process and transportation requirements. Therefore, connection systems are crucial to ensure the safe and efficient functioning of such systems. Two main categories of connection systems can be identified: panel-to-panel connection and panel-to-wall connection. The former is concerned with the joint between the sandwich panels and should ensure the load-transferring capacity of the floor system, as well as the compatibility of deformations. The latter is mostly concerned with transferring shear forces to the supports [202]. In both cases, safety, serviceability, and constructability aspects should be considered during the design process.

In the following sections, existing examples of panel-to-panel connections and panel-to-wall connections found in the literature will be presented. The developed connection technologies of the sandwich panel floor system will also be described, along with their motivation. Finally, the results of the experimental campaign conducted on them will be presented.

6.1. PANEL-TO-PANEL CONNECTION

The connection between panels is a crucial aspect that ensures the monolithic integrity of the floor system. This is especially important in areas that are prone to earthquakes. In such regions, the floor must provide the necessary diaphragm behaviour to distribute the earthquake forces proportionally to all the shear walls [203], [204]. When developing the connection system, it's important to consider construction details and practical aspects such as the ease of maintenance, assembly, and installation in confined spaces for rehabilitating existing degraded floors. A survey of current connection technologies will be presented in

the next section to highlight the feasibility of production, considering Ferpainel's manufacturing equipment and expertise, as well as compatibility with the target applications of the floor system.

6.1.1. Connection systems and technical considerations

In the literature of panel-to-panel joints, there are four most used types of joints. These are categorised as follows: i) tongue and groove joints; ii) z-shaped joints; iii) side lap joints; and iv) insert of profiles or cover plates.

Tongue and groove joints are a widely used solution for connecting both FRP and steel sandwich panels in civil engineering applications. This type of joint consists of an element edge with a protrusion and a mating element with a profiled groove on the corresponding edge (see Figure 6.1a). Research has shown that tongue and groove adhesively bonded joints are effective for connecting FRP sandwich panels in various applications, such as ship and marina decking, bridge and pavement for highway structures, and housing and temporary shelters [205]. Indeed, this type of connection was successfully employed for the rehabilitation of a through truss iron cast bridge with an FRP bridge deck system [206]. Tongue and groove joints are also used for connecting steel sandwich panels in cladding wall applications (see Figure 6.1b). The face sheets are folded back into the core, and a soft sealant is applied to the groove so that after assembly, the joint ensures adequate waterproofing and airtightness [22]. Nevertheless, tongue and groove joints present an installation challenge in confined spaces, as the panel needs to be horizontally slid into position. This may be particularly problematic when rehabilitating existing buildings [207].

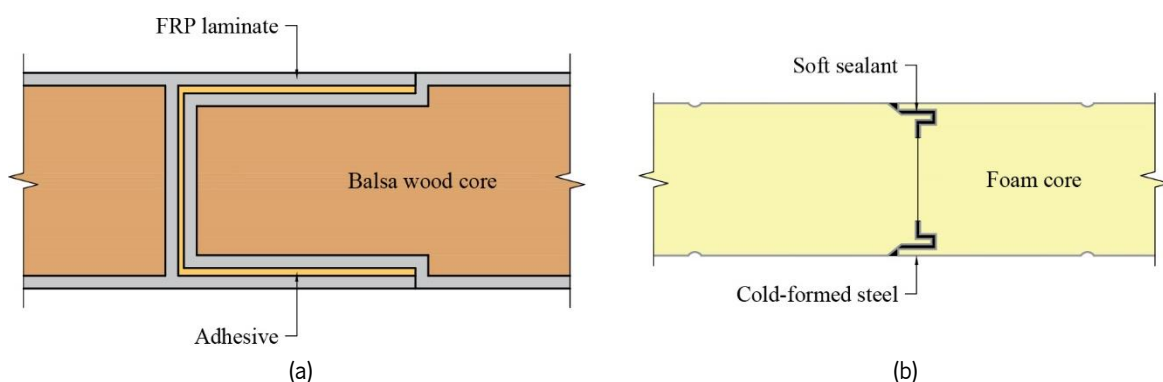


Figure 6.1. Tongue and groove joint system: (a) bridge deck FRP sandwich panel [205]; (b) panel-to-panel connection with soft sealant in steel cladding wall sandwich panels [22].

Z-shaped joints are illustrated in Figure 6.2. The main advantage of using this type of connection is that the panels can be lowered vertically into position without the need for any jacking system. This makes the assembly process easier, especially in confined spaces such as when rehabilitating degraded floors

in existing buildings. Z-shaped joints have been used for panel-to-panel connections of composite bridge deck profiles [208]. Both mechanical and adhesive joining techniques have been investigated. In the case of bolted connections, a cover plate needs to be added at the bottom of the profiles. In the context of sandwich panel floor systems used in building floor rehabilitation projects, adhesive Z-shaped joints are preferred for panel-to-panel connections, as discussed in [207]. These joints have been subject to experimental investigation, revealing their significant contribution to shear stiffness while minimally affecting bending stiffness.

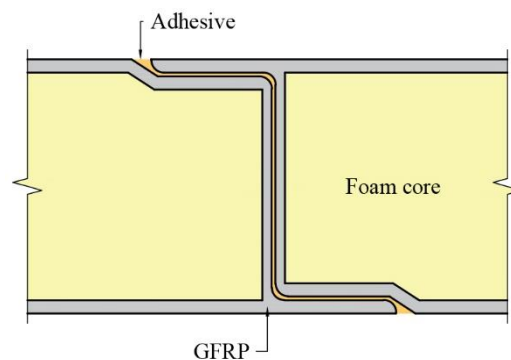


Figure 6.2. Z-shape adhesive joint in GFRP sandwich floor panels [207].

The previous methods of joining sandwich panels involved shaping the sides of the panels so that they can fit together when placed adjacent to each other. These systems have the advantage of not requiring any additional components other than structural adhesives. However, the production process for tongue and groove profiles in FRP sandwich panels requires forming dies, while specific roll-forming equipment is needed for steel sandwich panels. Integrating these changes into the production system can complicate the process and potentially lead to lower-quality sandwich panels.

Another type of joint involves using an additional element, other than the sandwich panels themselves, to assemble them together. This additional element can be made of thin-walled profiles and/or cover plates. Fitting composite profiles in the designated cavities of sandwich panels is used as a joint system for the GFRP sandwich panel floor system for emergency housing [19]. The investigation analysed two types of joining techniques: i) a friction technique without the use of mechanical fasteners and ii) a bolted connection. Steel hollow rectangular profiles are also common practice in the maritime industry to join the longitudinal sides of steel web-core sandwich panels [209]. Cover plates overlapping the adjoining sandwich panels are also used (see Figure 6.3). In both cases, the joint is achieved by means of welding.

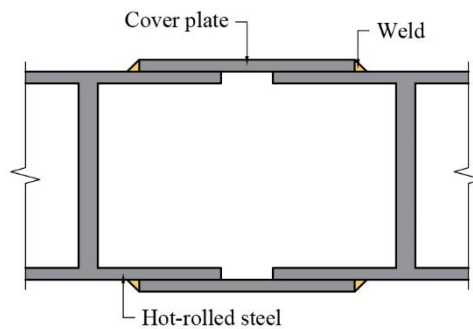


Figure 6.3. Cover plate joint for panel-to-panel connection in steel web-core sandwich panel [209].

Finally, regarding the assembly of CFS profiled sheet as well as steel sandwich roof panel, side lap joints are the most common employed strategy (see Figure 6.4). The overlapping portions of adjacent panels (side laps) are fastened together using screws or blind rivets [22].

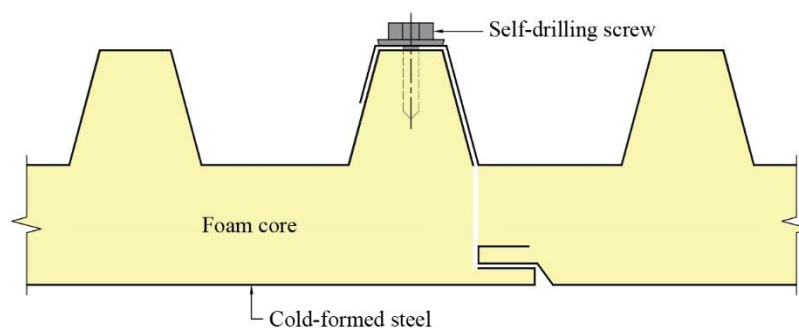


Figure 6.4. Side lap joint in steel roof cladding sandwich panels [22].

6.1.2. Proposed connection

After reviewing the current practices in panel-to-panel connection systems, the tongue and groove geometry joint was selected for the floor system developed in this study. This type of joint can be continuously produced by Ferpainel, with some necessary changes to the current roll-forming equipment. While it may pose challenges in confined spaces, the tongue and groove joint geometry also enhances the mechanical resistance of the panel by acting as a stiffener for the webs. Additionally, as explained later, this joint geometry is combined with mechanical fasteners, making it easier for repairs in service and material recovery at the end of the panel's life [210].

In a preliminary stage, three different mechanical joining techniques were considered: i) a cover bottom plate fastened to the sandwich panels using self-drilling screws (SB); ii) a top and bottom cover plates fastened to the sandwich panel using self-drilling screws (SBT); iii) self-drilling screws through the webs of the sandwich panels (SW). Mechanical fasteners were preferred because of their ease of disassembly

and repair. However, it is worth noting that adhesive connections offer more efficient shear and bending moment transfer, as well as greater fatigue resistance [206].

The spacing of the self-drilling screws was designed according to the specifications for mechanical fasteners of the Eurocode 3 Part 1-3 (2006) [80], which encompass verifications for the shear strength of the screws, the bearing failure of the screw holes, and the net section resistance of the cover plate. The screws chosen for the three joining techniques are made of stainless-steel and have a characteristic shear strength of 4.82 kN according to their data sheet. The self-drilling screws of connection SW have a nominal diameter of 6.3 mm and a nominal length of 160 mm. The self-drilling screws of connections SB and SBT have a nominal diameter of 5.5 mm and a nominal length of 21 mm. The cover plates are 1 mm thick and 80 mm wide and were designed assuming the material properties corresponding to the steel grade S250GD+Z. The overall geometry of the panel-to-panel joint systems is illustrated in Figure 6.5.

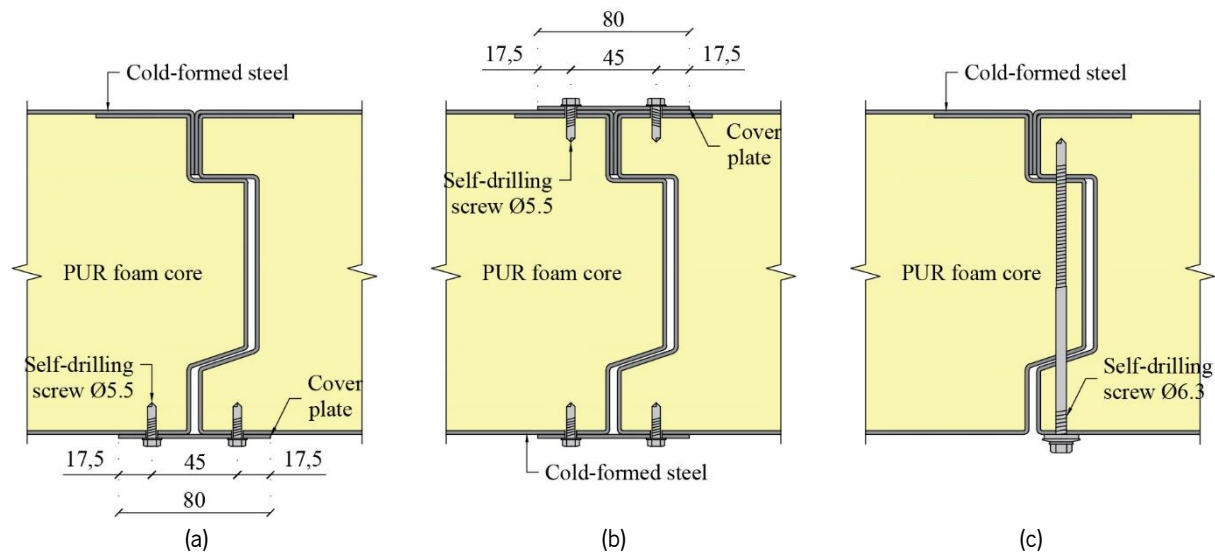


Figure 6.5. Panel-to-panel connection systems: (a) SB; (b) SBT; (c) SW. All units in [mm].

The panel-to-panel connection was designed to withstand the shearing stresses caused by non-uniform bending in the longitudinal direction. Although the panels were designed as a one-way slab, under specific loading conditions, i.e. concentrated loads, moments occur in the span and the transverse direction [211]. According to the design of concrete one-way slabs defined in the Eurocode 2 Part 1-1 (2004) [212], it is recommended to use a steel reinforcement that is equal to 20% of the reinforcement used in the principal direction for the transverse direction. Based on this standard, 20% of the ultimate bending resistance of the sandwich panel in the longitudinal direction was applied in the transverse direction. The

design actions acting on the connection are determined considering the structural model of a simply supported beam subjected to a uniformly distributed load (see Figure 6.6).

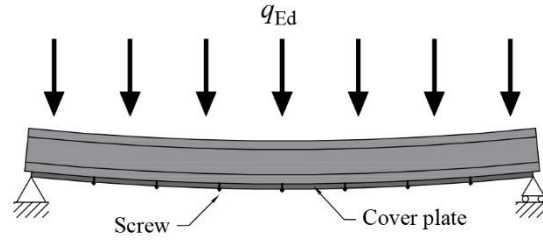


Figure 6.6. Structural model adopted for the design of the panel-to-panel connection.

Regarding the longitudinal direction, the shear force at the supports (V_{Ed}) is given by Equation (6.1):

$$V_{Ed} = \frac{q_{Ed} \cdot L}{2} \quad (6.1)$$

where q_{Ed} is the design value of the uniformly distributed load which was set equal ultimate load-carrying capacity (q_{Rd}) of the sandwich panel. V_{Ed} is assumed to be applied at the mid-width of the connection. The influence area of one SB panel-to-panel connection is depicted in Figure 6.7a. For the sake of simplicity, the web stiffeners of the cross-section are not considered, so V_{Ed} acts in the plane of symmetry of the cross-section. It is important to note that this assumption should be investigated further because the presence of a torque resulting from the asymmetry of the cross-section may cause additional shearing stresses.

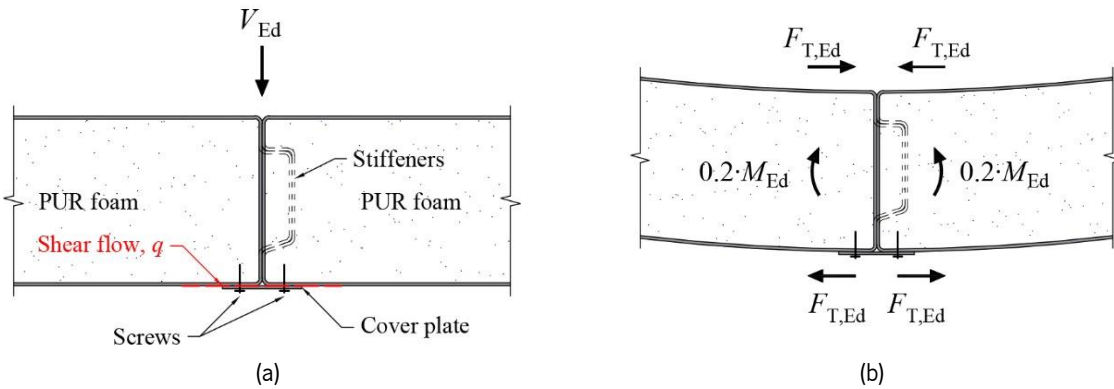


Figure 6.7. Detail of the cross-section for the panel-to-panel connection design: (a) longitudinal direction; (b) transverse direction.

The shear flow (q) at the upper surface of the cover plate is given by:

$$q = \frac{V_{Ed} \cdot Q}{I} \quad (6.2)$$

where Q is the first moment of area about the neutral axis, and I is the moment of inertia of the cross-section. As previously mentioned, the design bending moment (M_{Ed}) applied in the transverse direction is set equal to 20% of the ultimate bending moment resistance of the sandwich panel in the longitudinal direction (M_{Rd}), despite the one-way nature of the sandwich panel. Considering the previous simplified cross-section (see Figure 6.7b), the shear force ($F_{T,Ed}$) acting on the screws due to the transverse bending moment is given by:

$$F_{T,Ed} = \frac{0.2 \cdot M_{Ed}}{t_{tot}} \quad (6.3)$$

where t_{tot} is the total thickness of the sandwich panel.

The analytical design considers the shear resistance of the screw ($F_{v,Rd}$), bearing failure of the hole ($F_{b,Rd}$), and the net section resistance of the cover plate ($F_{n,Rd}$), whose values are given by the following Equations:

$$F_{v,Rd} = \frac{F_{v,Rk}}{\gamma_{M2}} \quad (6.4)$$

$$F_{b,Rd} = \frac{\alpha \cdot f_u \cdot d \cdot t}{\gamma_{M2}} \quad (6.5)$$

$$F_{n,Rd} = \frac{A_{net} \cdot f_u}{\gamma_{M2}} \quad (6.6)$$

where A_{net} is the net area of the cross-section of the cover plate, d is the diameter of the hole, f_u is the ultimate strength of the cover plate, $F_{v,Rk}$ is the shear characteristic strength of the screw, t is the thickness of the cover plate, and γ_{M2} is the partial factor for the resistance of screws.

It is important to note that each SB panel-to-panel connection requires two rows of screws. Therefore, the number of screws per linear meter of sandwich panel required in the longitudinal direction is calculated according to Equation (6.7):

$$n = \frac{q \cdot L}{2 \cdot \min(F_{v,Rd}, F_{b,Rd}, F_{n,Rd})} \quad (6.7)$$

On the other hand, in the transverse direction the shear force is acting on only one row of screw at a time, thus, the number of screws is determined according to Equation (6.8):

$$n = \frac{F_{T,Ed}}{\min(F_{v,Rd}, F_{b,Rd}, F_{n,Rd})} \cdot \frac{1}{L} \quad (6.8)$$

The result of the analytical design of the connection is summarised in Table 6.1. Based on the design, the failure mechanism of the connection system is governed by the bearing failure of the screw hole. The minimum spacing between screws is determined by the shear force in the longitudinal direction and it is found to be 0.55 m. In order to be conservative, a screw spacing of 0.5 m was chosen for the panel-to-panel connection.

Table 6.1. Analytical design of the panel-to-panel connection.

Design action		Design resistance		
q_{Ed} [kN/m]	$F_{T,Ed}$ [kN]	$F_{v,Rd}$ [kN]	$F_{b,Rd}$ [kN]	$F_{n,Rd}$ [kN]
7.18	11.20	3.86	1.99	18.22

6.1.3. Experimental characterisation: transverse direction

Three connected panels for each connection system, 0.5 m long, were tested in a three-point bending configuration over a clear span of 0.9 m. The specimens were prepared according to the following procedure (see Figure 6.8): i) the jointed panels were secured together using ratchet tie down straps; ii) the location of the cover plates and/or self-drilling screws were marked on the sandwich panels and the cover plates; iii) the panels were fastened together by means of self-drilling screws.

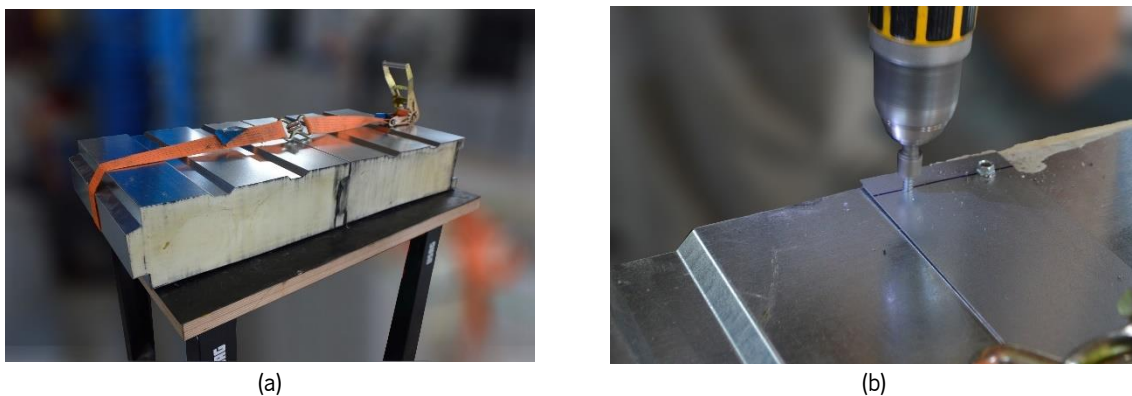


Figure 6.8. Panel-to-panel connection assembly process: (a) panel fastened using ratchet tie down straps; (b) drilling of the screws through the cover plate and sandwich panel.

The experimental test setup and the instrumentation are illustrated in Figure 6.9 and Figure 6.10. Simply supported boundary conditions were adopted for the test. The supports consisted of steel rollers on top of which were placed steel plates with a rectangular cross-section of 10 mm x 50 mm. The load was

applied by a hydraulic jack and monitored by a load cell of 10 kN. A hollow rectangular steel profile was used to transfer the load to the panels. LVDT 2 and LVDT 3 had a stroke of 50 mm and were placed at the midspan to measure the vertical displacements. Additionally, LVDT 1 and LVDT 4 had a stroke of 10 mm and were placed on the support to measure possible settlements. The specimens were tested in quasi-static monotonic-instantaneous loading up to failure, under displacement control, at a rate of 2.4 mm/min.

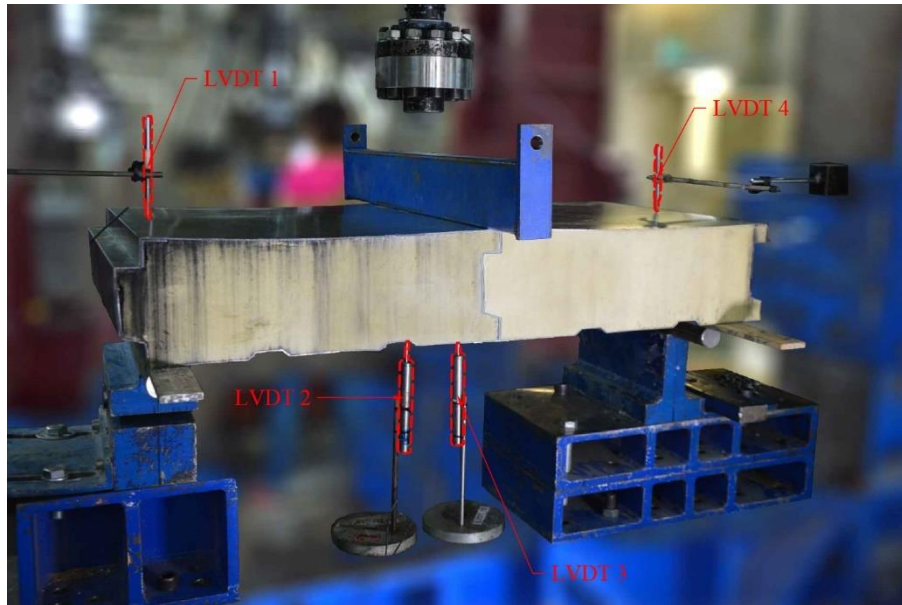


Figure 6.9. Transverse direction bending test setup.

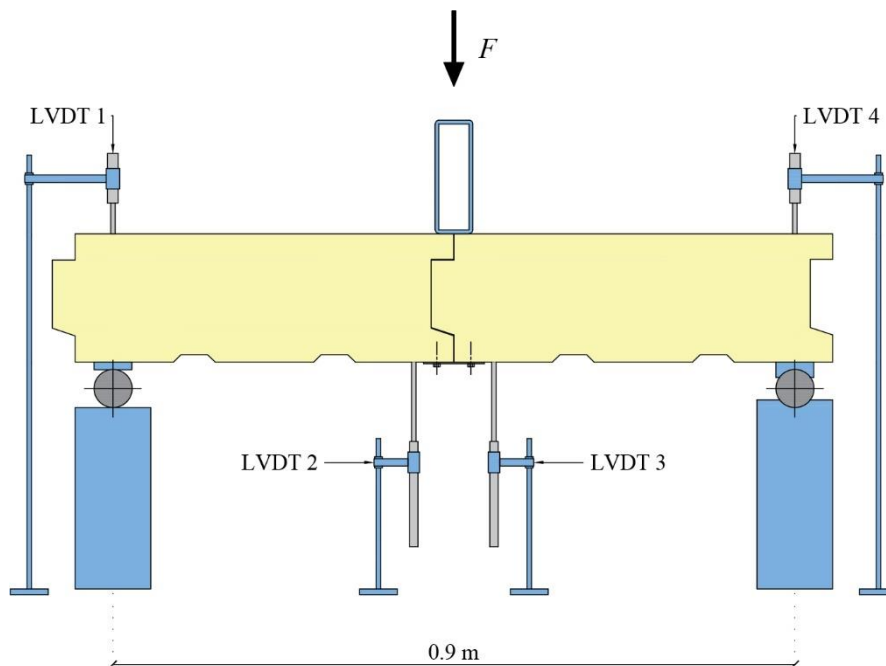


Figure 6.10. Transverse direction bending test instrumentation.

The load-displacement curves for the various panel-to-panel connection systems are displayed in Figure 6.11. The displacement is calculated as the difference between the average displacement measured at midspan (LVDT 2 and LVDT 3) and the average settlement recorded at the support (LVDT 1 and LVDT 4). The observed failure modes are illustrated in Figure 6.12 to Figure 6.15. For connection SB, an initial nonlinear branch is observed in the load-displacement curve, which is attributed to the closing of the gaps between the sandwich panels. Once the adjoining panels are in full contact, a linear part with increased stiffness is observed until a load of approximately 4 kN. At this point, a gradual loss of stiffness is observed in specimens SB-1 and SB-2, corresponding to the onset of the bearing failure of the screw hole in the sandwich panel bottom face sheet, as shown in Figure 6.12.

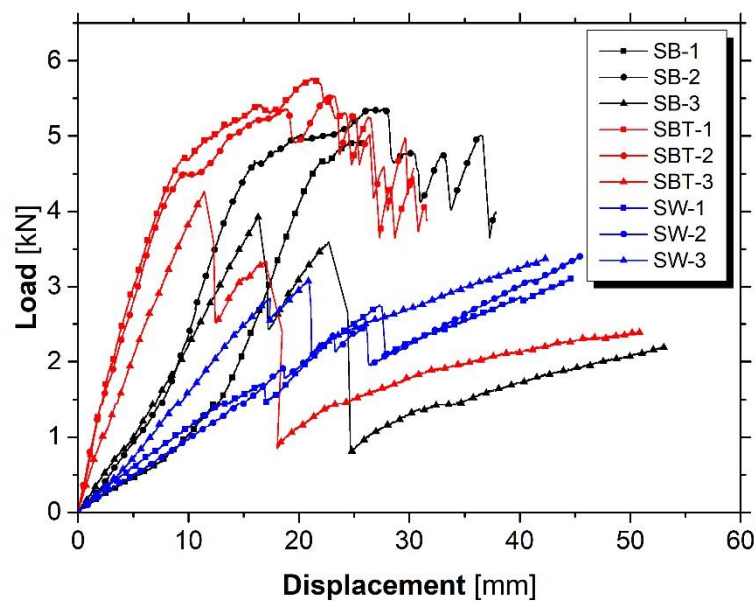


Figure 6.11. Load-displacement curves of the flexural test in the transverse direction.

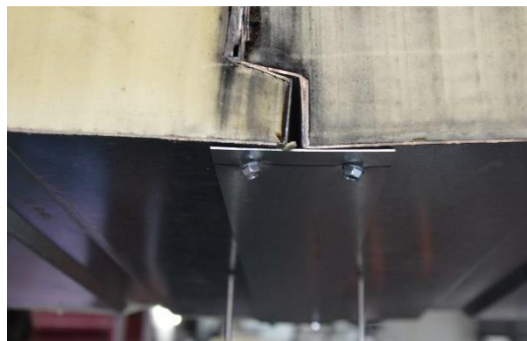


Figure 6.12. Bearing failure of the screw hole in specimens SB-1 and SB-2.

On the other hand, specimen SB-3 showed a smaller ultimate load with an abrupt load decay corresponding to the cohesive shear failure of the PUR foam core, as illustrated in Figure 6.13. It appears that the heterogeneities of the PUR foam core may have contributed to the observed differences in results.

This is because the mechanical properties of the PUR foam can vary across the length of the full-scale sandwich panels from which the 0.5 m long specimens were extracted. Specifically, in the case of specimen SB-3, one of jointed sandwich panel most probably had lower mechanical properties compared to the others. Additionally, based on the material properties outlined in Section 4.2.4, the predicted ultimate load for the shear failure of the PUR foam is expected to be more than two times the ultimate load obtained experimentally, determined by the product of the shear area of the core and the shear strength of the core.

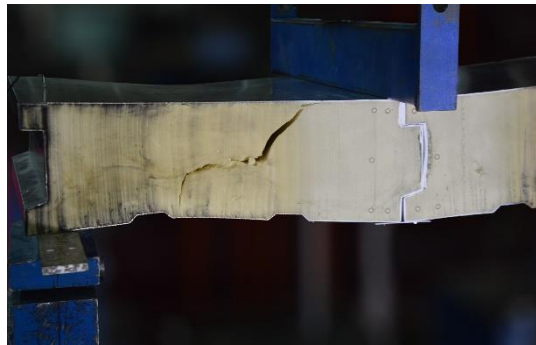


Figure 6.13. Cohesive shear failure of the PUR foam in specimen SB-3.

The behaviour of the connection system SBT was observed to be similar to that of SB. However, the presence of the top cover plate eliminated the gap between the sandwich panels and the non-linear initial part of the load-displacement curve. The collapse of specimens SBT-1 and SBT-2 occurred due to the bearing failure of the screw hole in the bottom face sheet of the sandwich panel. The consecutive load drops after reaching the ultimate load were caused by the tilting of the screw, which ultimately led to the screw pull-out (see Figure 6.14). On the other hand, specimen SBT-3 failed due to cohesive shear failure of the PUR foam, with a diagonal crack forming at mid-height at a quarter of the span length, similarly to what was observed for specimen SB-3. The crack later propagated to the top and bottom face sheets-to-core interface, leading to a complete loss of resistance.

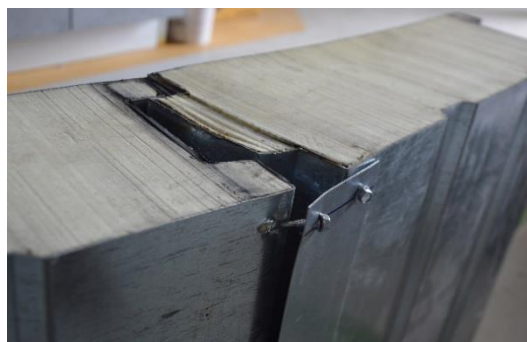


Figure 6.14. Tilting and pull-out of the screw in specimens SBT-1 and SBT-2.

During the testing of connection SW, it was observed that the load capacity increased linearly up to a displacement of about 15 mm. However, after this point, there were noticeable load drops due to the tensile cohesive failure of the PUR foam at midspan (see Figure 6.15). Despite this, the specimens were still capable of supporting additional load up to a displacement of 50 mm before the test was halted due to the LVDTs reaching their maximum stroke.



Figure 6.15. Tensile cohesive failure of the PUR foam near the face sheet-to-core interface.

A summary of the results of the three-point bending test for the transverse direction, including the ultimate load (F_u), the ultimate bending moment (M_u), the initial stiffness (K) obtained by means of linear regression of the data in the interval between 20% and 45% of F_u , and the identified failure mode is presented in Table 6.2.

Table 6.2. Summary of the results of the flexural test in the transverse direction.

Specimen [-]	F_u [kN]	M_u [kNm]	K [kN/mm]	Failure mode [-]
SB-1	4.92	1.11	0.19	B
SB-2	5.37	1.21	0.28	B
SB-3	3.93	0.88	0.22	CS
Avg. (CoV)	4.74 (15.5%)	1.07 (15.5%)	0.23 (19.7%)	
SBT-1	5.76	1.30	0.56	B
SBT-2	5.53	1.24	0.47	B
SBT-3	4.26	0.96	0.37	CS
Avg. (CoV)	5.18 (15.6%)	1.17 (15.6%)	0.47 (21.1%)	
SW-1	2.75	0.62	0.11	T
SW-2	2.48	0.56	0.10	T
SW-3	3.08	0.69	0.18	T
Avg. (CoV)	2.77 (10.8%)	0.62 (10.8%)	0.13 (36.4%)	

Notes: Avg. – average value for set of specimens; CoV – coefficient of variation; B – bearing failure; CS – shear cohesive failure of PUR foam; T – tensile failure of the PUR foam.

The results obtained from the tests show that connection SBT exhibits the highest value of ultimate load and stiffness, while the stiffness of connection SW is almost half and one-quarter of that of connection SB and SBT. This is to be expected since the lever arm between the shear planes of connection SW is smaller than those of connection SB and SBT. The substitution of M_u in Equation (6.3) determines the experimental value of the bearing failure of connection SB and SBT. The resulting ultimate load was calculated to be 3.67 kN, which is more than 50% higher than the analytical design results. Therefore, the preliminary design of the panel-to-panel connection is on the safe side.

Based on these results, connection SB was selected as the connection system for the flexural test in the longitudinal direction, as it presents the fastest and simplest execution process and a non-significant difference in terms of F_u compared to SBT.

6.1.4. Experimental characterisation: longitudinal direction

Two sets of full-scale connected panels (DP01 and DP02) were tested in a four-point bending configuration over a clear span length of 4.0 m. Two panels (Panel A and B) were assembled using the SB connection following the procedure described in Section 6.1.3. In the experiment, simply supported boundary conditions were achieved by placing steel plates on top of steel rollers. The load was applied at thirds of the span by a hydraulic jack with a capacity of 200 kN. A load cell of 200 kN with a precision of 0.05% was used to monitor the load. The load was transferred to the panels through a steel IPE beam, steel rollers and a prismatic steel plate with a cross-section of 10 mm × 100 mm. To ensure full and even contact in the loading zones, a thin layer of bitumen-based polyester putty was applied between the panel and the steel plates. The sandwich panels were loaded monotonically up to failure at a rate of 1.8 mm/min. The experimental setup and the employed instrumentation are illustrated in Figure 6.16 and Figure 6.17.

Eight LVDTs were utilized to monitor displacements at various locations along the tested span length. LVDT 1 and LVDT 8, with a 10 mm stroke, were positioned at the supports to measure possible settlements. LVDT 2, LVDT 3, LVDT 6, and LVDT 7, each with a 50 mm stroke, were situated under the loaded cross-sections of the sandwich panels. LVDT 4 and LVDT 5 were employed to measure the vertical deflection at the midspan. Four strain gauges located at mid-width of the cross-section were used to record the longitudinal strains at the top and bottom face sheets of each panel at midspan.

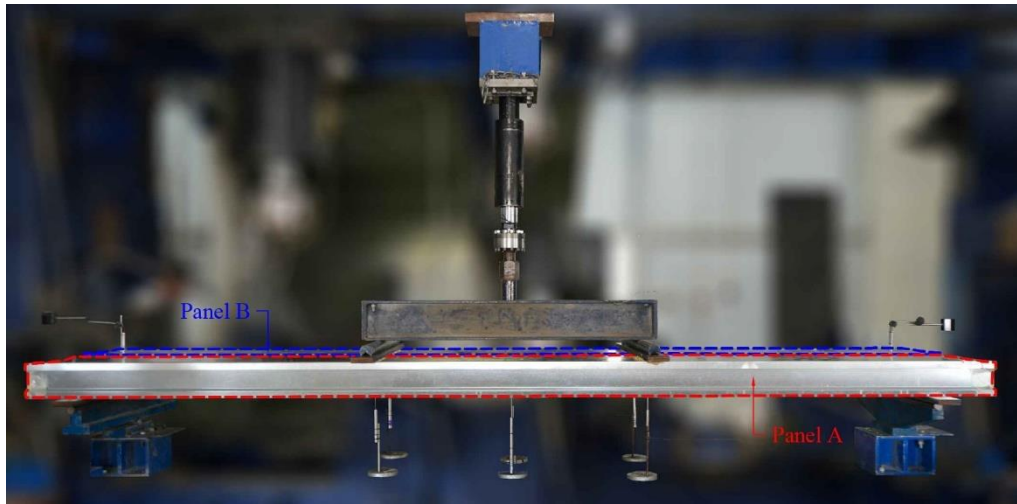


Figure 6.16. Test setup of the flexural test in the longitudinal direction.

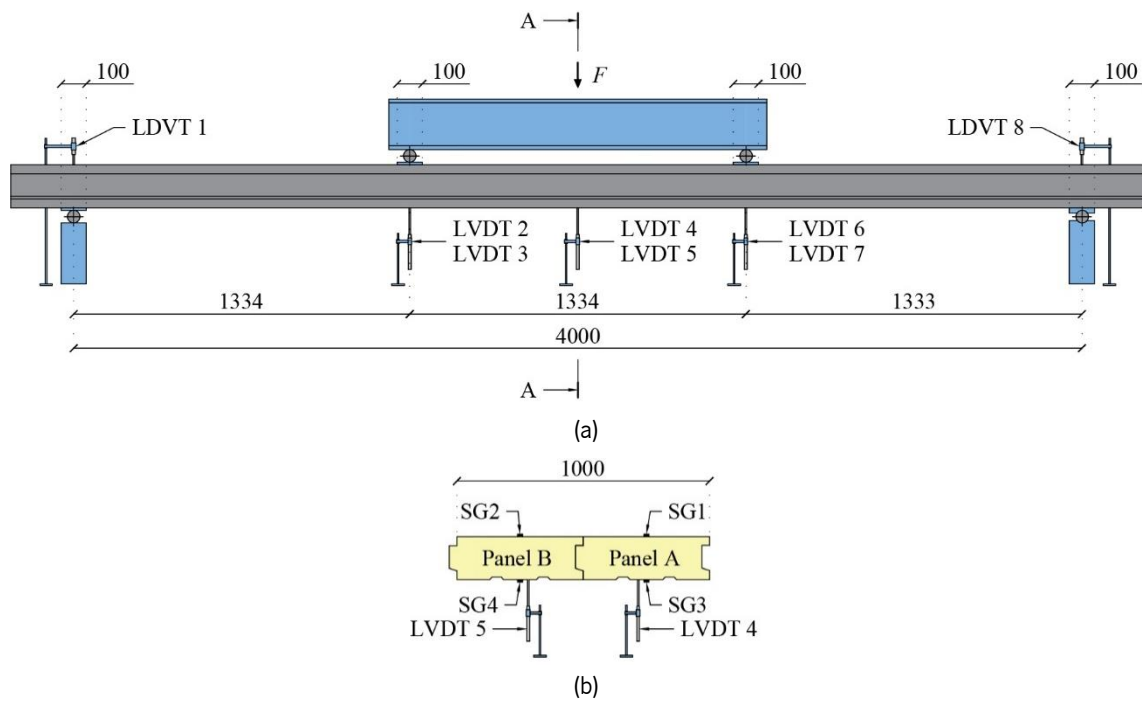


Figure 6.17. Test instrumentation of the flexural test in the longitudinal direction: (a) lateral view; (b) AA section. All units in [mm].

The load-displacement curves of the connected panels are plotted in Figure 6.18. The plotted displacement represents the difference between the average measurements obtained from LVDT 4 and LVDT 5, which recorded the midspan deflection, and the average value of the settlements recorded by LVDT 1 and LDVT 8. The vertical displacements recorded by LVDT 2, LVDT 3, LVDT 6, and LVDT 7 showed a similar qualitative development to those measured by LVDT 4 and LVDT 5, and are therefore not included here. The load of both sets of panels increased linearly up to approximately 20 kN. Beyond this point, the stiffness of the panels gradually decreases until the ultimate load is reached. In both sets of panels, the progressive failure was similar to that observed in the four-point bending test of the single

panel. The ultimate load is reached when local buckling of the top plain C-section of the sandwich panel occurs under the loaded section. This is noticeable by the outward deformation of the flanges. After this, another sharp load drop is observed, which is attributable to the buckling of the webs under the loaded section. A plastic hinge is formed, and failure at the top face sheet-to-core interface is also noted.

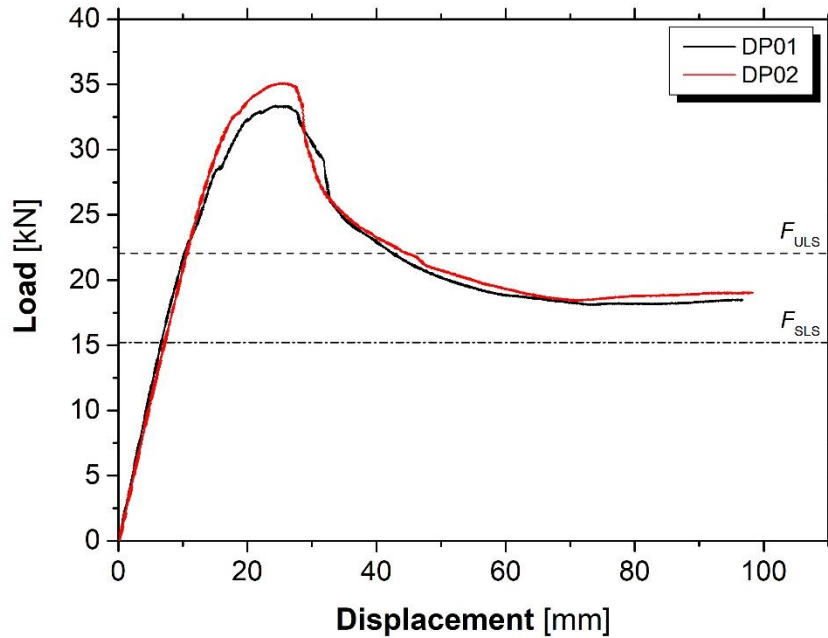


Figure 6.18. Load-displacement curves of flexural test in the longitudinal direction.

The deformed shape at the end of the test is similar in both jointed panels and resembles the failure mode of the single panels (see Figure 6.19a). Upon inspection and disassembly of the panel after failure, no sign of bearing failure in the screw holes of the cover plate and the bottom face sheet of the sandwich panel were detected (see Figure 6.19b and Figure 6.19c).

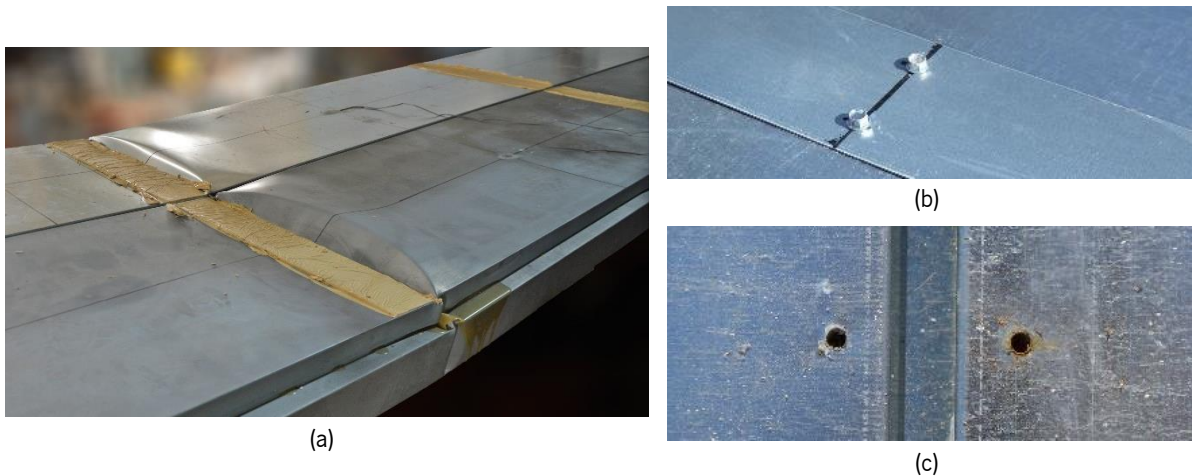


Figure 6.19. Failure and specimen inspection after the flexural test in the longitudinal direction: (a) failure mode at one of the loaded cross-sections; (b) screws appearance after failure; (c) screw holes appearance after failure.

The evolution of displacement of the monitored cross-sections of Panel A and Panel B for load levels corresponding to the ultimate experimental load, the ULS load, and the SLS load is plotted in Figure 6.20a and Figure 6.20b. The monitored cross-sections correspond to the midspan (A-A) and the sections where the load is applied to the sandwich panel (B-B and C-C) as illustrated in Figure 6.20c and Figure 6.20d.

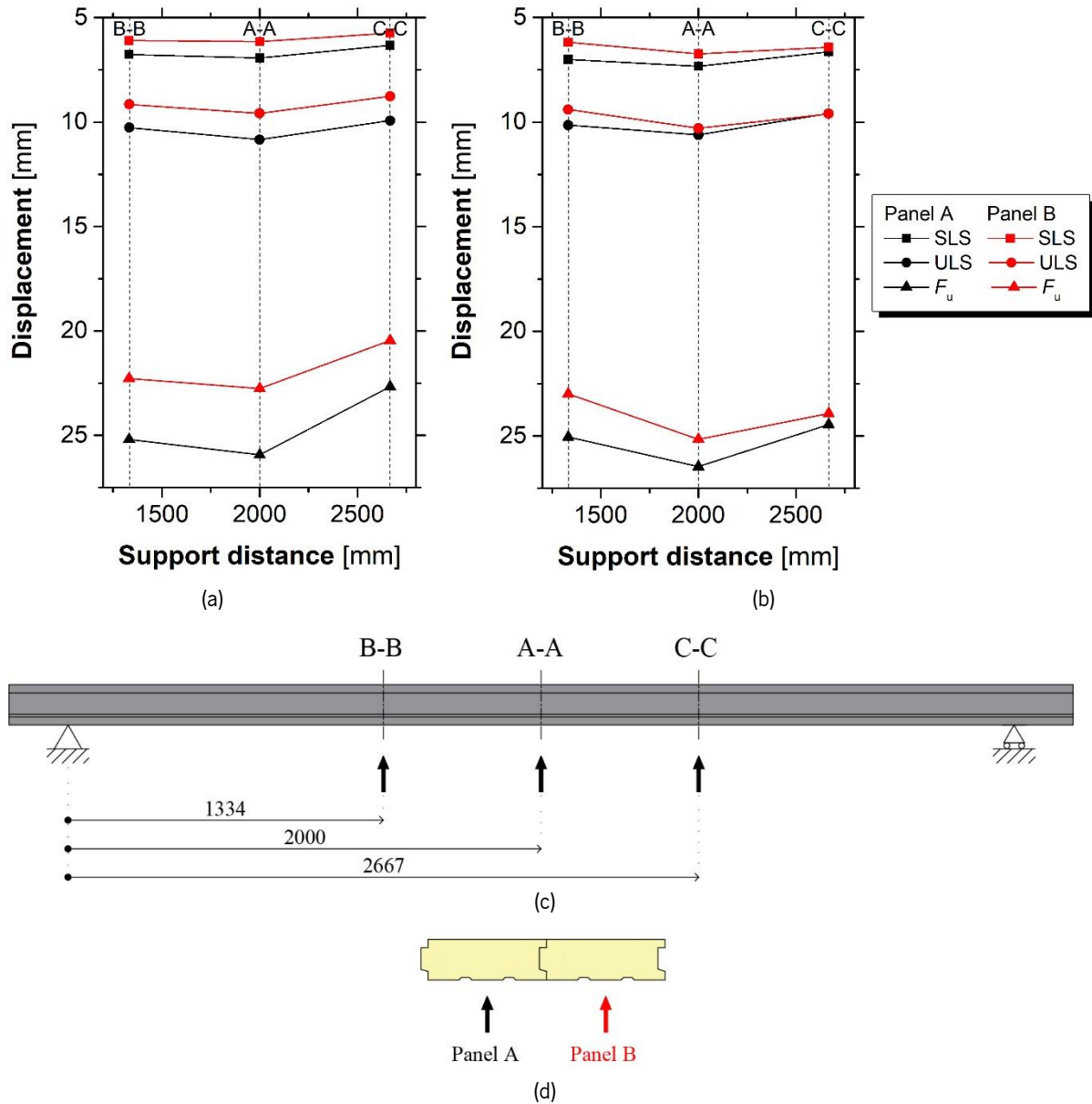


Figure 6.20. Displacement evolution for the three monitored cross-section of Panels A and B: (a) specimen DP01; (b) specimen DP02; (c) schematic of lateral view; (d) schematic of cross-section. All units in [mm].

Notably, a difference in the evolution of the displacement can be observed between Panel A and Panel B of the assembly. Specifically, in both joint specimens DP01 and DP02, Panel A exhibits greater displacement than Panel B, by 11.2% and 5.1%, respectively. This difference may be attributed to the rotation about the longitudinal axis of the sandwich panel assembly resulting from the asymmetry of the

cross-section. As previously stated, the difference is twice as high in specimen DP01. After executing the connection, an average gap of 12 mm and 6.5 mm was measured between the adjoining panels of specimens DP01 and DP02, respectively, due to a small deviation in the straightness of the panels. Therefore, the larger difference between sandwich Panels A and B in DP01 may be due to the larger gap between the jointed panels.

The load-longitudinal strain curves of specimen DP01 are presented in Figure 6.21. The longitudinal strain evolution of specimen DP02 is qualitatively similar therefore the curves are omitted. According to Figure 6.21, the linearity in the strain measurements of the top face sheet is lost at around 12.5 kN. This could be due to a non-uniform distribution of the load that resulted in the early onset of wrinkling in the top face sheet of Panel A in both specimens DP01 and DP02. This observation is consistent with the more rapid evolution of the displacement of Panel A plotted in Figure 6.20. Alternatively, it is possible that varying levels of imperfections present in the adjoining panels caused this behaviour, given that the longitudinal strain measurements in the bottom face sheets in tension show good agreement.

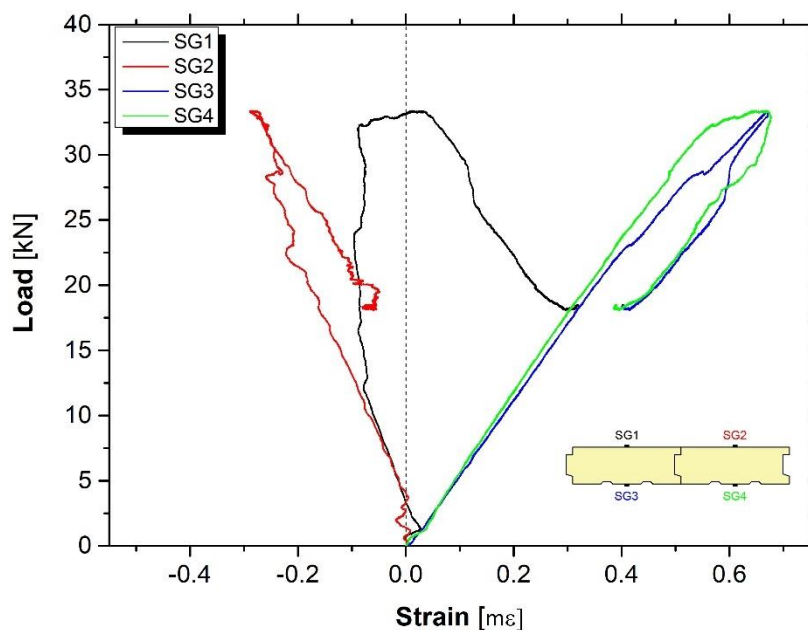


Figure 6.21. Load-longitudinal strain curves of the flexural test in the longitudinal direction.

The results obtained from the four-point bending test on the connected panels including the deflection at the SLS load level (δ_{SLS}), the initial stiffness (K), the ultimate load (F_u) and the corresponding compressive (ε_{top}) and tensile strains (ε_{bottom}) are summarised in Table 6.3. Furthermore, a comparison with the average results obtained from the same type of test on single panels is also presented.

The midspan deflection in both the single and connected panels present similar values for the SLS load level, with the corresponding loads being 7.61 kN and 15.21 kN, respectively. The stiffness of the connected panels is almost twice that of the single panels, which is as expected, since the area of steel mostly contributing to the bending stiffness is twice that of the single panels. The average ultimate load of the joint panels is slightly higher than twice that of the single panels, which could be due to some level of restraint provided against local buckling from the contact between the adjoining sides of the panels.

Table 6.3. Summary of the results of the four-point bending test on connected panels.

Item	δ_{SLS} [mm]	K [kN/mm]	F_u [kN]	ϵ_{top} [mε]	ϵ_{bottom} [mε]
Single panel*	5.44	1.33	16.08	-0.498	0.589
DP01	5.49	2.67	33.38	-0.136	0.657
DP02	6.03	2.36	35.11	-0.125	0.669
Avg.	5.76	2.52	34.25	-0.131	0.663
Jointed/Single	1.06	1.89	2.13	0.26	1.13

Notes: highlighted with “*” is the row corresponding to the average values obtained from the four-point bending test of the single panel (see Sections 5.2.3 and 5.2.4).

The results from the compression face sheets exhibit a significant variation in longitudinal strain, whereas the results from the face sheets in tension show good agreement. The strain measurements are localised and thus susceptible to the influence of local imperfections in the compression zone of the sandwich panel. Within the results of the four-point bending test of the single panel, a larger scatter was already observed in the compressive strains at the ultimate load. The difference with the single panel results could also be due to the presence of a stiffer zone, that is, the joint between adjacent sandwich panels, which absorbs a larger portion of the bending strains than the slenderer part of the cross-section.

6.2. PANEL-TO-WALL CONNECTION

The panel-to-wall and panel-to-panel connections must ensure the transfer of vertical and horizontal loads from the floor to the vertical load-bearing structures, as mentioned in Section 6.1. However, care should be exercised when dealing with existing buildings. Indeed, recent earthquakes demonstrated the inefficiency of obtrusive intervention or overall substitution of degraded timber floors with reinforced concrete slabs [213]. The introduction of reinforced concrete elements such as ring beams and rigid diaphragms in the depth of the wall due to the incompatibilities in terms of mass and stiffness between the new interventions and the original structure may lead to local or global failures [214]. In the following sections, a survey of the limited literature on panel-to-wall connection technologies will be presented,

followed by the illustration of the different proposed solutions for the floor system object of this study and the discussion of the results of the experimental campaign.

6.2.1. Introductory aspects

The literature on the rehabilitation of degraded floors in existing buildings has mostly focused on in-plane strengthening techniques for timber floors [213], [215], [216] and the development of structural members to replace them. However, for the effective utilisation of these new structural members, connection systems must be developed, and their strength and stiffness must be evaluated [217].

A connection system between composite sandwich floor panels and load-bearing walls is proposed in [218]. The system comprises steel angles anchored to walls supporting sandwich panels, similar to top-and-seat angle connections used in the beam-to-column joint of steel frame structures [219]. The connection technique comprises the use of bolts, adhesive, or a hybrid bolt-adhesive connection. To evaluate the mechanical behaviour of the connection systems, flexural cantilever tests were carried out. The results indicated that the use of the top steel angle generally provided increased rotational stiffness. Adhesively bonded connection presented higher stiffness, but lower deformation capacity compared to the bolted connection.

The use of steel angles to improve the connection between the masonry walls and the timber floors was investigated in [220]. The process involved bolting the steel angles to both the walls and floors. A layer of mortar between the masonry surface and the steel angle leg was applied to ensure a uniform contact surface. This method significantly improved the lateral strength and deformation capacity of the building as compared to the unreinforced structure.

The diaphragm behaviour of steel-insulated sandwich panels for roofing and cladding systems has also been assessed. An experimental campaign was conducted, which revealed that the in-plane stiffness of the system is strongly influenced by the number of self-drilling screws used to fasten the sandwich panels to the load-bearing structures [221].

6.2.2. Proposed connection

The connection systems investigated in the scope of this work are illustrated in Figure 6.22. The connection systems are based on the common practice in the rehabilitation of degraded floors in existing

buildings and they include the following components: i) one bottom steel angle supporting the bottom face sheet of the panel and connected by bolts throughout the thickness of the panel (1L); ii) two steel angles, being one positioned at the bottom and one on the top face sheet of the panel, both connected to the panel by bolts throughout the thickness of the panel (2L); iii) two steel angles, being one positioned at the bottom and one on the top face sheet of the panel, both connected to the panel by mechanical means with bolts throughout the thickness of the panel and adhesive between the angles and the panel (2LA). The gap between the sandwich panel and the vertical structure (see Figure 6.22) is necessary for installation procedures and to accommodate any irregularities in the existing masonry wall.

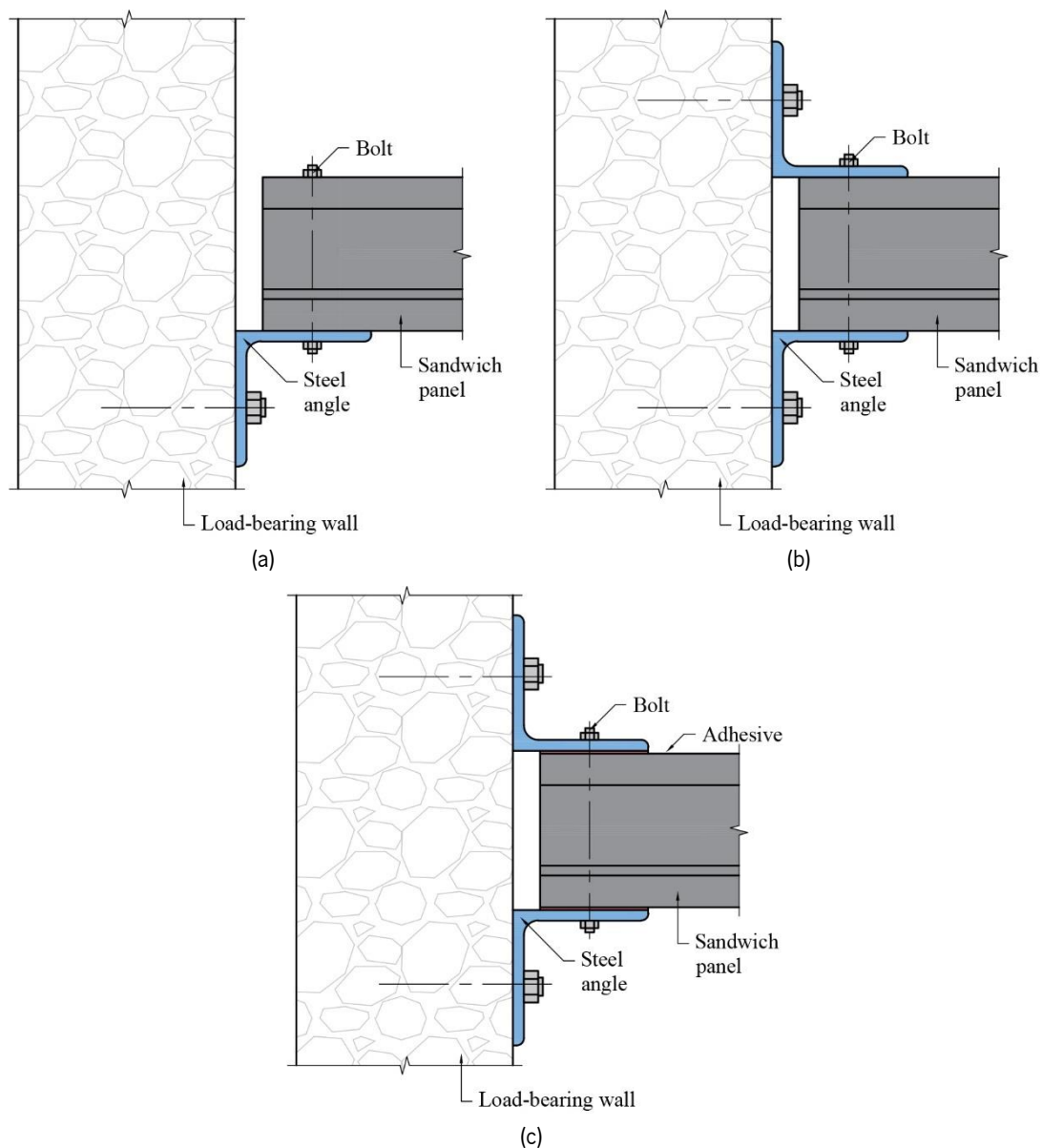


Figure 6.22. Panel-to-wall connection layouts: (a) 1L; (b) 2L; (c) 2LA.

6.2.3. Experimental characterisation

To assess the mechanical behaviour of the connection systems, full-scale 4.5 m long sandwich panels were connected to HEB 450 columns simulating load-bearing walls (see Figure 6.23a). Steel L-profile equal angles with a height of 150 mm and a thickness of 12 mm were cut to a width of 500 mm to accommodate the full width of the sandwich panels. 10 mm diameter holes were pre-drilled into the L-profiles according to the dimensions shown in Figure 6.23c. M10 bolts were cut to a length of 250 mm to connect the sandwich panels to the L-profiles. In the case of connection 2LA, the S&P resin 220 Epoxy structural adhesive was used to adhesively bond the sandwich panel with the L-profiles. The nominal thickness of the adhesive layer is 1 mm. For simulating service life conditions, the sandwich panels were installed 30 mm away from the HEB450 profiles. The panels were tested in a four-point bending configuration as illustrated in Figure 6.24. Due to limitations of materials, it was only possible to test two specimens for each connection.

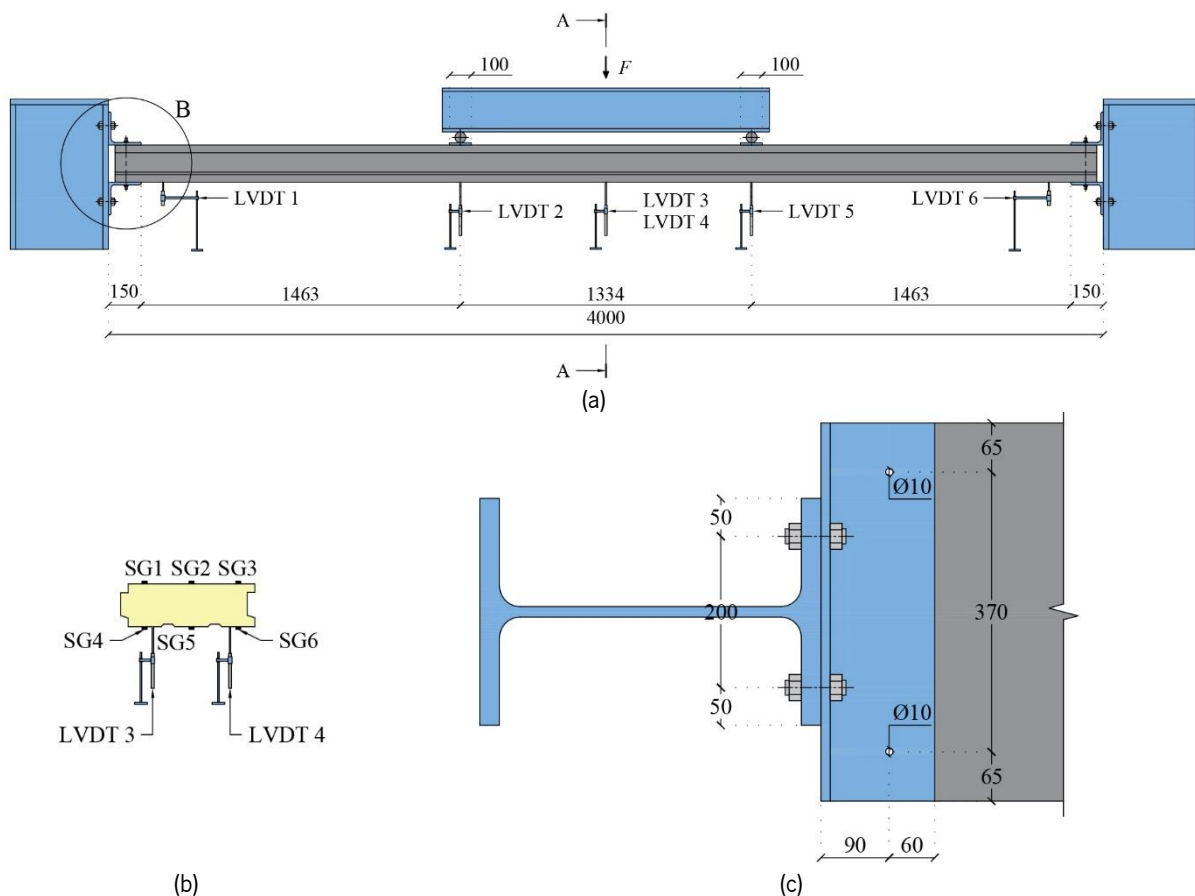


Figure 6.23. Panel-to-wall connection bending test instrumentation: (a) lateral view; (b) section AA; (c) detail B. All units in [mm].



Figure 6.24. Panel-to-wall connection four-point bending test.

Considering the distance between the edge of the L-profiles as the supports, the panels were tested over a clear span of 4260 mm and a shear span of 1463 mm. The details of the support conditions are shown in Figure 6.25 for the different connection systems. The distance between the loaded sections was 1334 mm, which corresponds to one-third of the span length of the four-point bending test carried out on the single sandwich panels. The load was applied by a hydraulic jack with a capacity of 200 kN. A load cell of 200 kN with a precision of 0.05% was used to monitor the load. The load was transferred to the panels through a steel IPE beam, steel rollers and a prismatic steel plate with a cross-section of 10 mm × 100 mm. The sandwich panels were loaded monotonically up to failure at a rate of 1.8 mm/min.

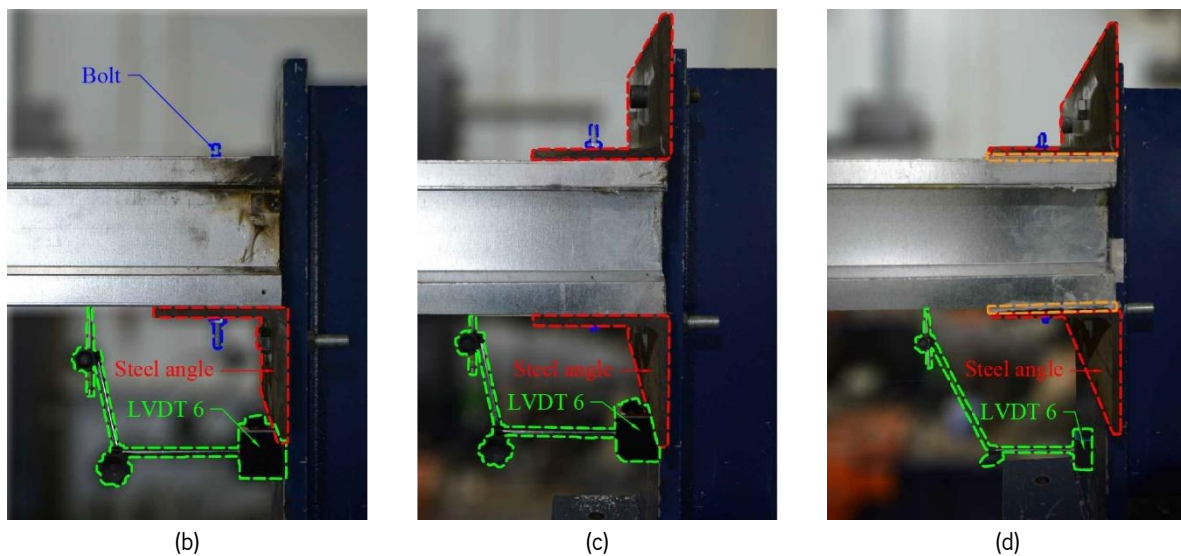


Figure 6.25. Panel-to-wall connection bending test: (a) overall setup; (b) 1L; (c) 2L; (d) 2LA.

LVDT 1 and LVDT 6, which had a stroke of 5 mm, monitored the vertical displacements at a distance of 100 mm from the edge of the steel angle profiles. LVDT 2 and LVDT 5, with a stroke of 50 mm, were

located under the loaded cross-sections. LVDT 3 and LVDT 4, with a stroke of 100 mm, measured the deflection at midspan.

To evaluate the flexural stiffness of the different connection systems and compare it to analytical predictions, it's important to note that the overall stiffness of the system depends on the HEB450 profiles and the assembly comprising steel angles, bolts, and the sandwich panel. To estimate the contribution of the HEB450 profiles to the overall stiffness, the simplified structural model shown in Figure 6.26a is considered. In this model, appropriate boundary conditions are applied to the axis of symmetry of the system. When a concentrated load (F) is applied to the sandwich panel, it results in a bending moment (M_{Ed}) and an axial force on the HEB450 profile. The free end rotation (θ) of the HEB450 profile due to M_{Ed} is given by:

$$\theta = \frac{M_{Ed} \cdot h}{E \cdot I} \quad (6.9)$$

where h is the length of the HEB450 profile, E is the Young's modulus of steel and I is the moment of inertia of the HEB450 profiles about the Y local axis. The free end rotation of the HEB450 profile result in a rigid body movement of the sandwich panel at midspan (δ_{HEB450}) (see Figure 6.26c).

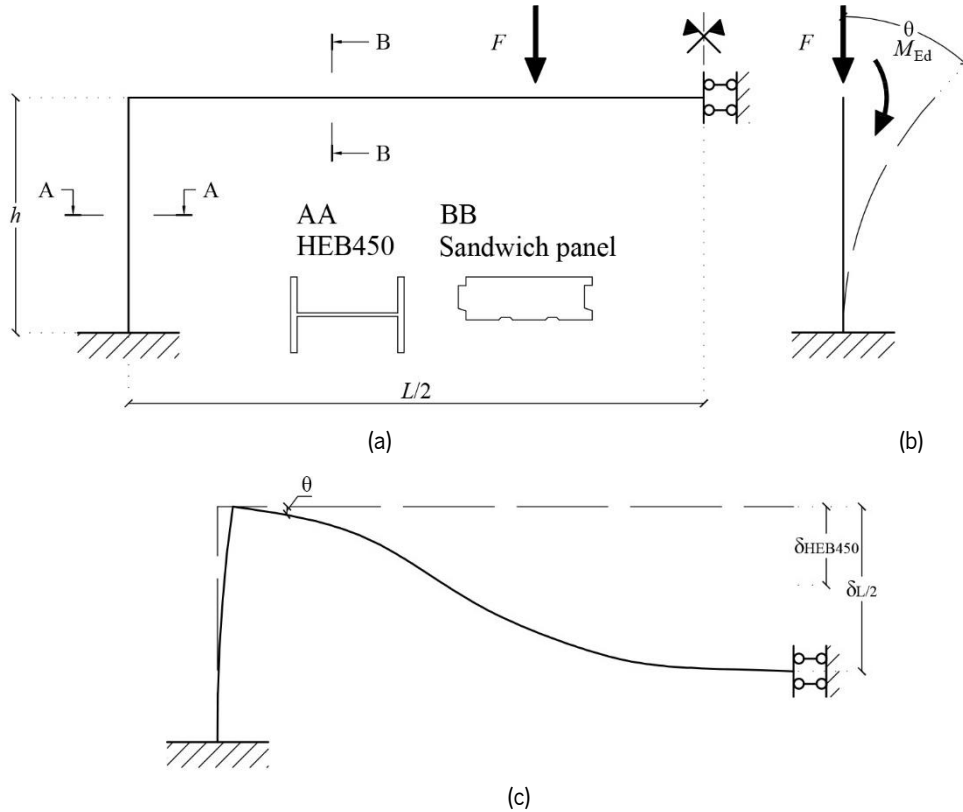


Figure 6.26. Estimation of the flexural stiffness of the connection systems: (a) structural model of the panel-to-wall flexural test; (b) actions on the HEB450 profiles; (c) contribution to the overall deflection.

The deflection due to the free end rotation of the HEB450 profile is given by:

$$\delta_{\text{HEB450}} = \frac{L}{2} \cdot \tan(\theta) \quad (6.10)$$

where L is the span length of the sandwich panel. As explained later in this section, a linear regression is used to estimate the flexural stiffness of the system in a load range between 20% and 45% of the ultimate load. Within this range, for all the specimens considered, δ_{HEB450} represents less than 2% of the overall midspan deflection ($\delta_{L/2}$). Therefore, the measured midspan deflection is mainly attributed to the flexural stiffness of the assembly, which consists of the steel angle profiles, bolts, and sandwich panel.

Two analytical models were used to evaluate the flexural stiffness of the connection systems. These models do not consider the contribution of the HEB450 profiles since they are considered to be much stiffer than the assembly of steel angles, bolts, and sandwich panels. The structural models make use of the symmetry of the boundary conditions and loading configurations (see Figure 6.27). They represent a simply supported beam subjected to two-point loads and a fixed beam subjected to two-point loads.

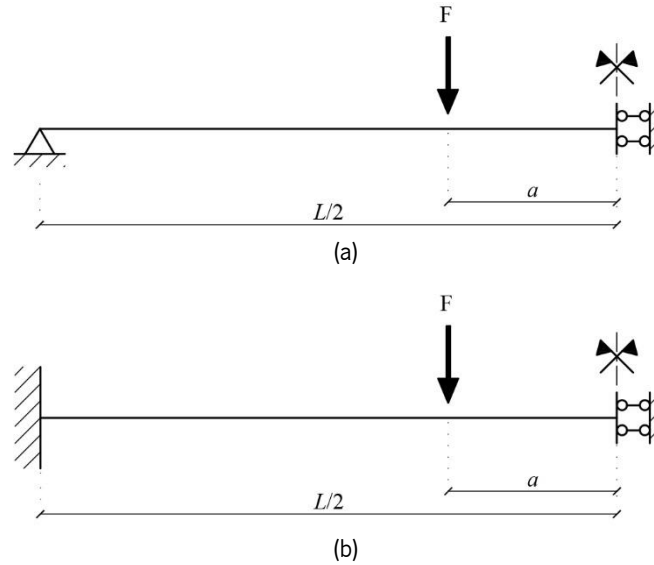


Figure 6.27. Analytical models used to predict the flexural stiffness of the connection systems: (a) simply supported beam; (b) fixed beam.

The experimental stiffness was expected to fall between a lower bound, expressed by the simply supported boundary conditions, and an upper bound, represented by the fully clamped boundary conditions, due to the finite stiffness of the steel angle, bolts, and sandwich panels. If the structural models and the design method M1 (see Section 5.1.3) are considered, the flexural stiffness of the simply supported sandwich panel (K_{SS}) is given by the following equation:

$$K_{ss} = \frac{6 \cdot E \cdot I_{M1}}{(L - a)(2L^2 + 2aL - a^2)} \quad (6.11)$$

where a is the distance between the midspan and the point load and L is the span length. On the other hand, the flexural stiffness of the fixed sandwich panel (K_f) is given by Equation (6.12):

$$K_f = \frac{12 \cdot E \cdot I_{M1}}{(L - a)^2(L + 2a)} \quad (6.12)$$

In Table 6.4, the summary of the results obtained from the four-point bending test on the connection is presented, including the stiffness (K), the ultimate load (F_u) and the analytical predictions of the stiffness considering the simply supported and fixed structural models. The experimental stiffness is determined as the slope of the load-displacement curve in an interval between 20% and 45% of the ultimate load.

Table 6.4. Summary of the results of the four-point bending test on the connection systems.

Connection system [-]	Specimen [-]	K [kN/mm]	Exp/AD	F_u [kN]
1L	1L-01	0.87		13.25
	1L-02	0.83		14.96
	Avg.	0.85	1.39 (SS) 0.31 (F)	14.11
2L	2L-01	0.95		15.66
	2L-02	1.00		14.86
	Avg.	0.98	1.60 (SS) 0.35 (F)	15.26
2LA	2LA-01	1.73		23.41
	2LA-02	1.41		17.26
	Avg.	1.57	2.58 (SS) 0.57 (F)	20.34
Analytical	SS	0.61		
	F	2.76		

Notes: Avg. – average value per set of specimens; SS – simply supported boundary conditions; F fixed boundary conditions; Exp/AD – ratio between experimental stiffness and analytical design prediction.

The experimental flexural stiffness of connection system 2LA was found to be the closest to the fixed boundary conditions, achieving 60% of the analytical value. In contrast, connection systems 1L and 2L showed a flexural stiffness 1.5 times higher than that obtained from simply supported boundary conditions, but still less than half of the stiffness expected from fixed boundary conditions.

The load-midspan displacement curves for various connection systems are shown in Figure 6.28. However, due to an issue with the acquisition system, data for the connection specimens 1L-02 and 2LA-

01 are only available until a displacement of 50 mm. As the displacement measured by the LVDTs near the supports displayed a qualitatively similar development to those registering the deflection at the midspan cross-section, they are not included.

During the test, the 1L connection specimens showed a nearly linear-elastic behaviour until the ultimate load was reached. However, there was a difference in the behaviour of specimen 1L-01 and specimen 1L-02. Prior to the peak load, the stiffness loss of specimen 1L-01 was due to the bearing failure of the web at one of the supports. On the other hand, the decrease in stiffness of specimen 1L-02 was caused by the wrinkling of the top face sheet between the loaded sections (see Figure 6.29).

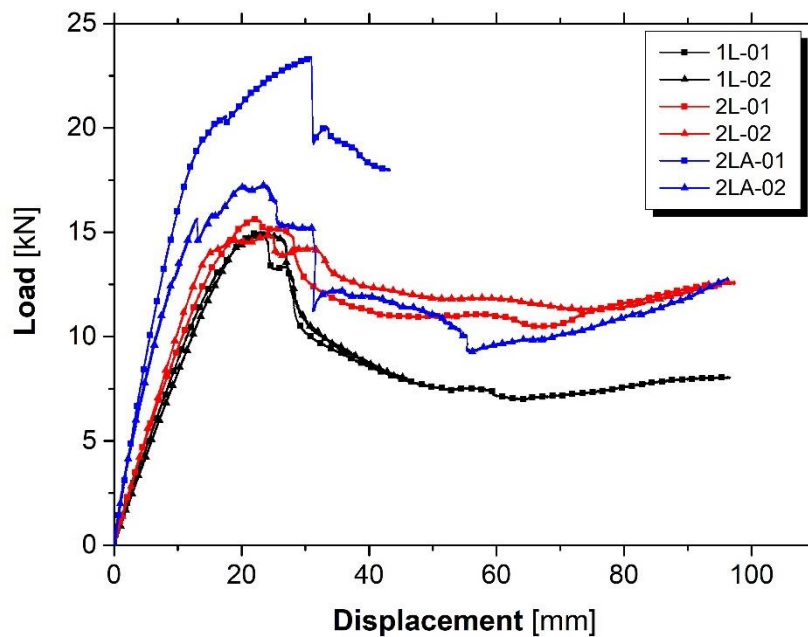


Figure 6.28. Load-displacement curves of the flexural test on the panel-to-wall systems.

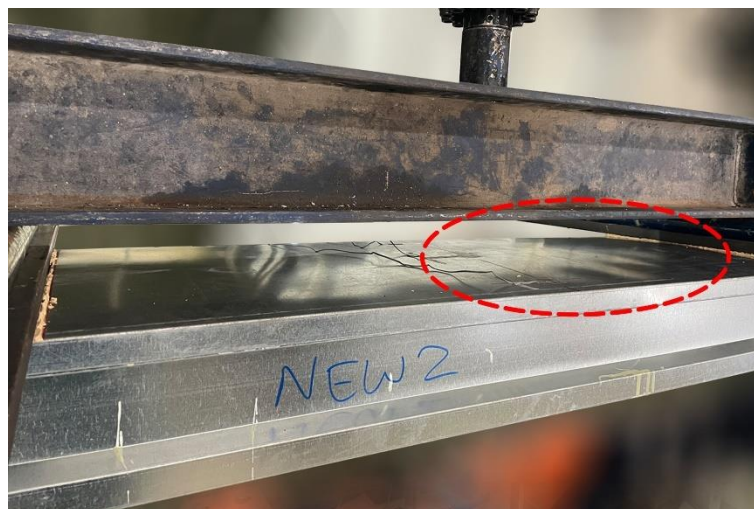


Figure 6.29. Wrinkling of the top face sheet of specimen 1L-02.

Both specimens showed a failure mode that was typical of a single panel tested with simply supported boundary conditions. Specifically, there was a local buckling of the top plan C-section, followed by outward deformation of the flanges at one of the loaded sections, and then local buckling of the web. Tilting of the bolts was observed at the support angles (see Figure 6.30a). However, after the test, inspection of the specimens revealed no signs of bearing failure in the holes of the sandwich panel face sheets (see Figure 6.30b).



Figure 6.30. Failure progression and inspection after the flexural test of connection system 1L: (a) tilting of the bolts at the end of the test; (b) undamaged bolt holes after disassembly.

The analysis and comparison of the tests conducted on 2L connection specimens revealed two significantly different behaviours. In the case of specimen 2L-01, the curve's shape was similar to the ones observed for the single panel tested with simply supported boundary conditions. The failure of the specimen was caused by the local buckling of the top face sheet and webs under one of the loaded sections. On the other hand, specimen 2L-02 presented multiple peaks in the load-displacement curve. This was due to local buckling being observed in both the loaded sections of the specimen (see Figure 6.31).

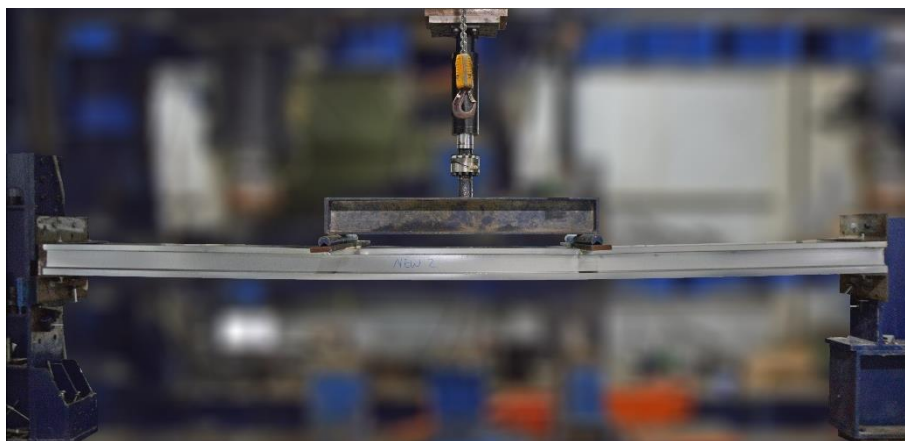


Figure 6.31. Local buckling at both loaded sections of specimen 2L-02.

Both specimens showed a hardening branch at midspan after about 70 mm of deflection. The sandwich panel's cross-section rotation caused gaps to form between it and the steel angles (see Figure 6.32a), leading to the bearing failure of the bolt holes in the top and bottom face sheets (see Figure 6.32b). However, the top steel angles increased the initial stiffness (+15.0%) and ultimate load (+8.2%). Additionally, the top steel angle improved the connection system's post-peak behaviour, allowing the sandwich panel to withstand a load of around 73% of the ultimate load at a midspan displacement of 100 mm.



Figure 6.32. Inspection after the flexural test of connection system 2L: (a) rotation at the support; (b) bearing failure of the bolt holes after disassembly.

The hybrid mechanical-adhesive connection system had a larger variance between the two specimens compared to the previous ones. The assembly process for connecting the sandwich panels involved several steps: i) the adhesive was prepared and applied to the bottom angle; ii) the panel was quickly installed onto the bottom angle; iii) the adhesive was applied to the top face sheet of the sandwich panel; iv) the top angles were installed, and the bolts were fastened. Despite the lightweight nature of the sandwich panel, the assembly process was still complicated and required the use of the laboratory's 2-ton capacity crane. As is often the case, the experience gained from preparing the first specimen resulted in a higher level of precision when assembling the second specimen. The first specimen had a significant amount of adhesive, which increased the bonding area between the column of the frame, the steel angles, and the sandwich panels (see Figure 6.33). This possibly led to an increase in initial stiffness and ultimate load.

The bonded area of specimen 2LA-02 showed no significant difference from the nominal bonded area of the connection (see Figure 6.34). It was observed that both specimens failed progressively, and failure started with the opening of a crack at the interface of the top steel angle and top face sheet (see

Figure 6.35a). As the load increased, the rotation at the supports also increased, leading to the opening of cracks at the interface of the bottom steel angle and bottom face sheet (see Figure 6.35b).



Figure 6.33. Support conditions of specimen 2LA-01: (a) adhesive excess in the bottom steel angle; (b) adhesive spill in the end cross-section of the sandwich panel.

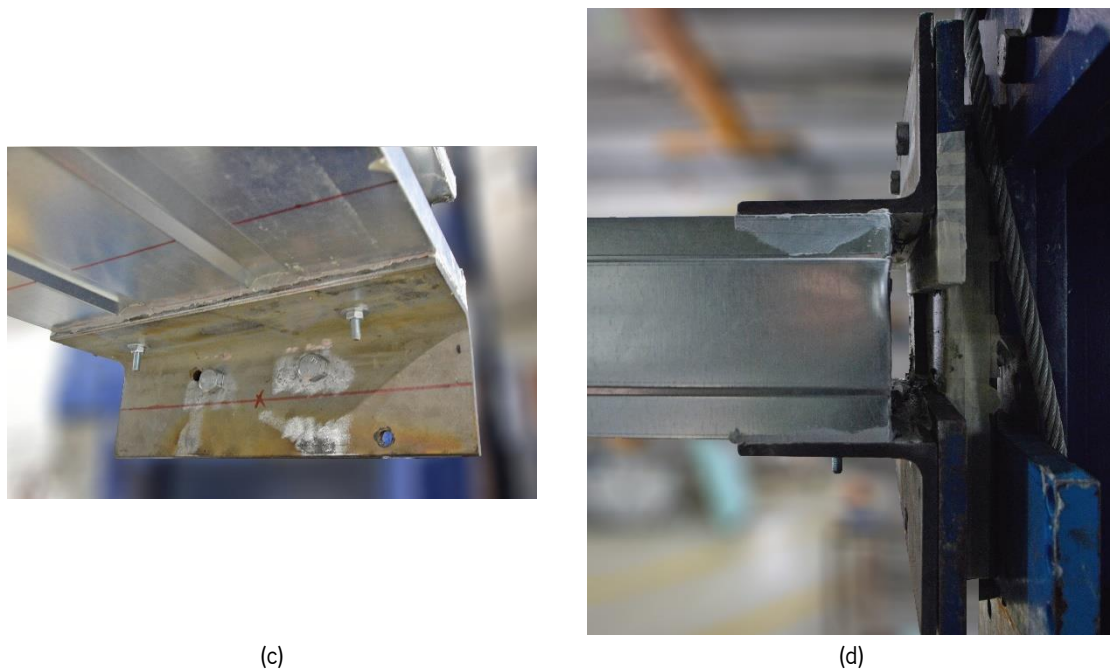


Figure 6.34. Support conditions of specimen 2LA-02: (a) bottom steel angle; (b) gap between the sandwich panel and the support column.

The test results for specimen 2LA-01 showed a gradual loss of stiffness throughout the test. At both loaded sections, the local buckling of the top plain C-section occurred, leading to a significant drop in the load when the webs buckled at one of the cross-sections. On the other hand, in specimen 2LA-02 local buckling was concentrated in only one of the loaded cross-sections, and a failure with complete loss of resistance was observed. The adhesive bond between the top plain C-section and the bottom lipped C-section failed abruptly, causing the separation of the constituent parts of the built-up cross-section of the

sandwich panel (see Figure 6.36). This failure may be due to the variability of the properties of the PUR-based adhesive and may be an indication of the presence of defects in the adhesive bond between the CFS components of the cross-section.

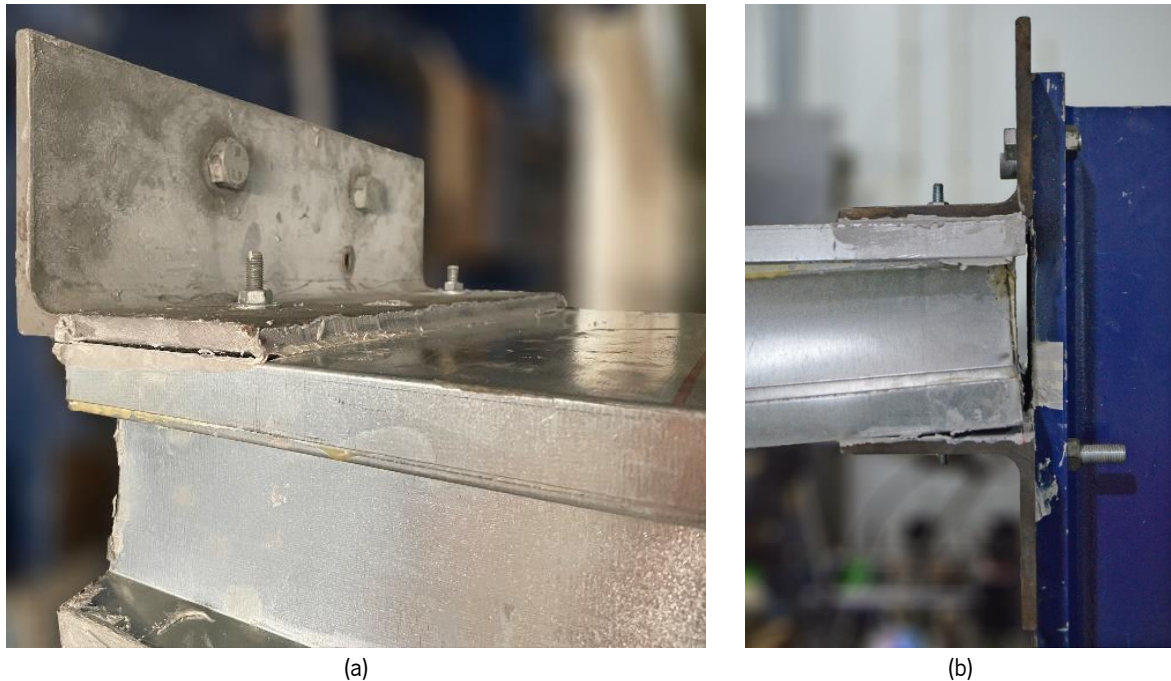


Figure 6.35. Failure progression of connection system 2LA: (a) crack at the adhesive-to-top face sheet interface; (b) crack at the bottom face sheet-to-adhesive interface.



Figure 6.36. Debond between the top plain C-section and bottom lipped C-section of the sandwich panel.

After examining the specimen following the test, it was found that there was a significant amount of adhesive remaining on the surface of the steel angle in both specimens. This was likely due to the rough surface of the hot-rolled steel angle profiles, even after the face sheet of the sandwich panel had been sanded. Furthermore, there were signs of bearing failure in the bolt holes of the sandwich panel face sheet. This suggests that as the cracks in the adhesive progressed, the bolts were activated, providing additional stiffness and ductility to the connection.

6.3. CONCLUDING REMARKS

Different technologies for panel-to-panel and panel-to-wall connections were developed and experimentally and analytically assessed to gain insight into the local and overall behaviour of the floor system replicating service life conditions.

Regarding the panel-to-panel connection, different failure modes were observed during the transverse direction connection tests. Some of the specimens showed premature cohesive shear failure of the PUR foam. As Chapter 4 highlights, future research should address the heterogeneity of the mechanical properties of the PUR foam based on its location within the sandwich panel. To improve the thermal and acoustic insulation, eliminate gaps between adjoining sandwich panels, and enhance the connection's stiffness, a soft sealant could be added to connection system SB, which was selected for the longitudinal connection test.

The panel-to-panel connection system proposed for the longitudinal connection tests involves a bottom cover plate and self-drilling screws (SB). This system is cost-effective and easy to assemble. However, during the longitudinal direction connection tests, the possible presence of a torsional effect was highlighted due to the asymmetrical tongue and groove joint geometry. Further investigation is required to address this issue. Moreover, it is recommended that flexural tests be conducted on more than two jointed sandwich panels, with the load applied to only one of the connected sandwich panels. This experimental campaign will provide insight into the capacity of the connection system to distribute the load among adjoining panels.

For what concerns the panel-to-wall connection, the experimental tests have demonstrated that the ultimate failure occurs outside of the connection system. Therefore, all the connection systems guarantee that the sandwich floor panel meets the mechanical requirements. Connection systems 2L and 2LA have exhibited signs of bearing failure as the panel's rotation at the supports increases. The use of stiffening timber blocks can be considered to enhance the stiffness and bearing resistance at the supports [221]. Additionally, the connection systems and sandwich floor panels' in-plane behaviour should be evaluated for their application in buildings situated in seismic-prone areas where the floor's diaphragm behaviour is critical to ensuring the structure's monolithic behaviour.

CHAPTER 7

7. NUMERICAL MODELLING AND DESIGN TOOLS

The finite element (FE) method is widely used for the prediction of the mechanical behaviour of civil engineering structures subjected to different types of loading conditions. In the scope of this work, the FE analyses are conducted using the ABAQUS software to replicate the structural response of a cold-formed steel sandwich panel with a PUR foam core.

The numerical models are developed to reproduce i) the behaviour of the constituent materials, ii) the behaviour of the small-scale steel sandwich panel with PUR foam core, and, finally, iii) the dynamic and static structural response of the sandwich floor panel. It should be noted that the behaviour of the connection systems (panel-to-panel and panel-to-wall) is crucial in determining the service life behaviour of the sandwich floor panels, but it is not included in this study and will be explored in future research.

The simulation of the sandwich panel's static and dynamic behaviours under simply supported boundary conditions can be used to gain insights that are relevant to their service life conditions. The experimental results from Chapter 6 showed that damage in the connection system is usually detected only after reaching the ultimate load, which is often caused by instability phenomena observed in the sandwich panel.

The procedure to obtain the constitutive material models from the mechanical properties obtained during the material characterisation campaign is described in the first part of this chapter. This is followed by the material and geometrical nonlinear analyses necessary to reproduce the behaviour of the sandwich floor panel. Critical aspects affecting the reliability of numerical models and their accuracy in reproducing the physical model are identified, including limitations arising from simplifying assumptions.

In the second part of this chapter, guidelines are proposed for designing the newly developed sandwich panel using the simplified analytical design method developed during this research work. The design tools are intended to promote the practice of rehabilitating existing buildings where the original timber floors require a new flooring system due to degradation.

7.1. CONSTITUTIVE MATERIALS MODELS

The aim of this study is the development of numerical models that can accurately replicate the mechanical behaviour of sandwich floor panels. To achieve this, it is necessary to calibrate the constitutive models of the cold-formed steel face sheets and the PUR foam core. This section explains the calibration methods that were used, including a comparison with the material characterisation curves. Nonlinear FE simulations were conducted to match the stress-strain response obtained from the uniaxial tensile test of steel, as well as the flatwise tensile and compressive tests of PUR foam.

7.1.1. Cold-formed steel face sheets

When modelling steel structures that undergo large deformations, it's important to estimate an accurate true stress-strain curve based on the measured engineering stress-strain curve. In this study, the approach proposed by [222] was followed to develop the true stress-strain model. The engineering strain (ε_e) and stress (σ_e) can be related to the true strain (ε_t) and stress (σ_t), up to the ultimate tensile strain, according to the following equations:

$$\varepsilon_t = \ln(1 + \varepsilon_e) \quad (7.1)$$

$$\sigma_t = \sigma_e \cdot (1 + \varepsilon_e) \quad (7.2)$$

The ultimate tensile strength is considered the start of the necking phenomenon in the central region of the specimen [223]. Beyond this point, highly localised stresses appear in the area near the cross-section that is undergoing necking. The true stress in the hardening region was obtained using the power law given by Equation (7.3):

$$\sigma_t = \sigma_{t,u} \cdot \left(\frac{\varepsilon_t}{\varepsilon_{t,u}} \right)^n \quad (7.3)$$

where n is a constant parameter, $\sigma_{t,u}$ is the true ultimate tensile stress, and $\varepsilon_{t,u}$ is the true ultimate tensile strain. The same power law is also used to extrapolate the true stress from the true ultimate strain up to a true strain of 1.0, a typical value for true fracture strain in structural steel, according to [222] and [224]. Therefore, the steel material properties used in the FE simulations are listed in Table 7.1.

Table 7.1. Cold-formed steel properties for the simulation of the tensile coupon test.

Parameter [-]	TC#1-1.0	TC#1.5-1.0
ρ [kg/m ³]	7850	7850
E [GPa]	205	194
ν [-]	0.3	0.3
$\sigma_{t,u}$ [MPa]	417	386
$\varepsilon_{t,u}$ [-]	0.19	0.19
n [-]	0.22	0.21

The predicted true stress-strain curves are plotted along with the experimental average engineering curves for the specimen sets TC#1-1.0 and TC#1-1.5 in Figure 7.1.

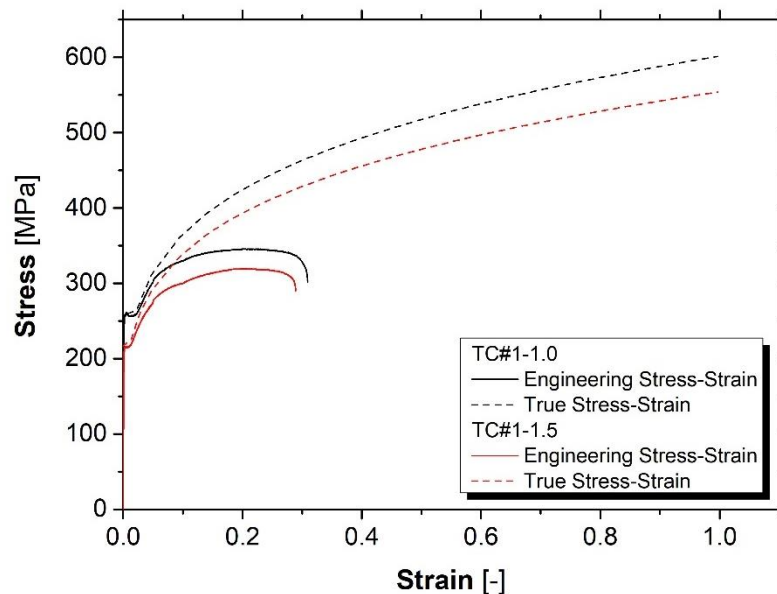


Figure 7.1. Engineering stress-strain curves and predicted true stress-strain curves.

A FE model was developed to validate the constitutive model, which simulates the stress-strain experimental curves. A four-node shell element (S4R) based on linear interpolation and a reduced integration scheme was used to model the steel coupon test. The Von Mises yield criterion with isotropic hardening was chosen to replicate the metal plasticity. The finite element model consists of quadrilateral elements with an approximate side length of 5 mm. In the middle of the model, where the necking

phenomenon was observed during experiments (see Figure 7.2), there is a more refined area with a side length of 2.5 mm. The end regions' nodes (top RBC and bottom RBC in Figure 7.2) are tied to reference points (top RP and bottom RP in Figure 7.2) using the rigid body constraint in the ABAQUS software. As a result, the reference point's motion governs the motion of the nodes in the selected regions, and their relative positions remain constant throughout the analysis. Therefore, these regions do not undergo any deformation. The bottom reference point was fully constrained, while a vertical displacement of 35 mm was applied to the top reference point, resulting in the tensile loading of the specimen. The chosen solver scheme for the analysis, which accounted for material and geometrical nonlinearities, was the Newton-Raphson method.

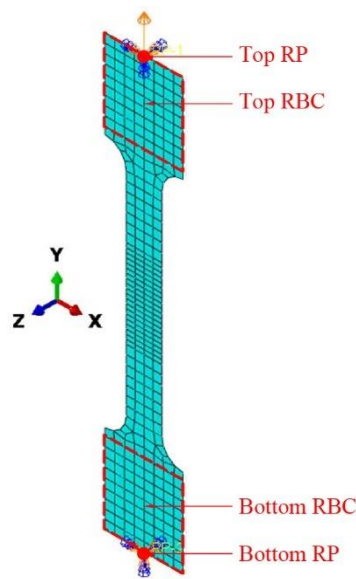
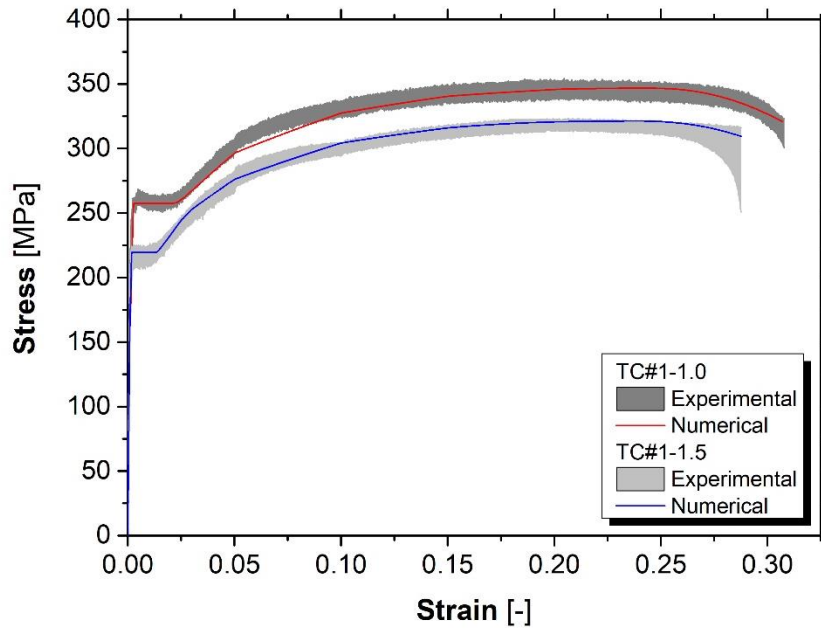
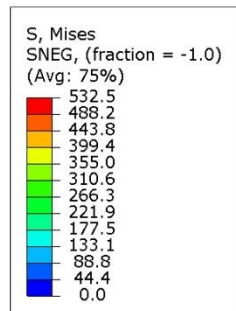


Figure 7.2. Tensile coupon test: mesh, loading, and support conditions.

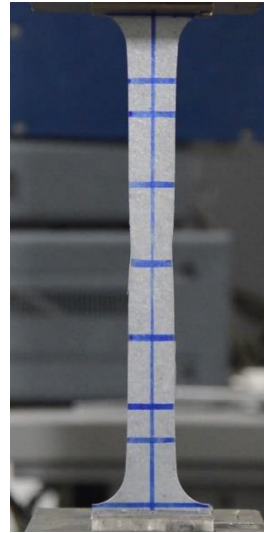
The stress-strain curves of the FE models for the sets of specimens TC#1-1.0 and TC#1-1.5 are presented in Figure 7.3a and compared with the average experimental curve. The numerical model strains were determined at the same locations as the extensometer readings during the experiments. The predicted stress-strain curves are in good agreement with the experimental ones. Similar failure patterns were observed before fracture, as shown in Figure 7.3b and Figure 7.3c. The numerical analyses terminated when the top reference point reached the prescribed vertical displacement. The face sheets of the small-scale sandwich panel and the full-scale prototypes did not exhibit fracture failure. Therefore, reproducing the coupon specimen's fracture failure mode is beyond this work's scope.



(a)



(b)



(c)

Figure 7.3. Numerical results of tensile coupon test simulations: (a) stress-strain curves; (b) deformed shape of the FE model before failure; (c) coupon specimen before failure.

7.1.2. PUR foam core

The compressive and tensile behaviour of the core of the sandwich panel is particularly relevant for correctly predicting the instability phenomena, such as wrinkling of the face sheet. The crushable foam material model available in the ABAQUS software was selected to reproduce the mechanical behaviour of the PUR foam. It requires defining the yield surface in the hydrostatic pressure-Von Mises stress space and the true stress-strain curve for uniaxial compressive loading. The parameters needed to define the yield surface were obtained through the method proposed by [170]. It is a simplified calibration process

where the yield surface is determined based on the values of the compressive yield strength ($\sigma_{c,y}$), the ultimate tensile strength ($\sigma_{t,u}$), and shear yield strength (τ_y).

In Chapter 4, the results of the experimental campaign conducted on the PUR foam with a nominal density of 40 kg/m³ were discussed. However, values obtained from the campaign were found to be inconsistent with the literature regarding the Poisson's ratio of the PUR foam. Also in Chapter 4, several explanations were proposed for the discrepancy between the literature values and the experimental campaign values, such as the heterogeneity of the PUR foam and the influence of the overall dimensions of the sandwich panel on the foam microstructure. It was also suggested that the assumption of isotropy of the PUR foam may not be correct due to the preferential evolution of the foam in the rise direction.

Therefore, due to the restriction of the crushable foam material model on the assumption of the linear isotropic elasticity of the material, it was deemed necessary to disregard the results of the shear test on the PUR foam. Only the elastic properties obtained from flatwise tensile and compressive tests are considered in the developed numerical models, compliant with the assumption of the isotropic behaviour of the crushable foam material model. The value of the Poisson's ratio was estimated based on typical average values for polymeric foams found in the literature [172], [225]. To estimate the yield surface, the value of (τ_y) was estimated based on the value of the compressive yield strength. According to the values found in the literature on PUR foam core sandwich panels [17], [18], [36], [42], [226], the $\sigma_{c,y}/\tau_y$ ratio ranges from approximately 1.0 to 2.0. Therefore, τ_y was estimated by dividing $\sigma_{c,y}$ by 1.5.

The expression of the yield surface for the crushable foam constitutive model in the hydrostatic pressure (p)-Von Mises stress (q) space is given by Equation (7.4):

$$q = \alpha \cdot \sqrt{\left(\frac{p_c + p_t}{2}\right)^2 - \left(p - \frac{p_c - p_t}{2}\right)^2} \quad (7.4)$$

where p_c is the hydrostatic compressive strength, p_t is hydrostatic tensile strength, and α is a coefficient for the definition of the densification process.

It is possible to predict the values of p_c and p_t by fitting Equation (7.4) with the known points identified by $\sigma_{c,y}$, $\sigma_{t,u}$, and τ_y . The resulting yield surface is illustrated in Figure 7.4a. The evolution of the yield surface as a result of the compaction of the cell wall material in the PUR foam is defined in the ABAQUS software by the true stress-plastic strain curves in uniaxial compression.

The true stress and true strain are estimated according to the formulations provided in [227]:

$$\varepsilon_t = -\ln(1 - \varepsilon_e) \quad (7.5)$$

$$\sigma_t = \frac{\sigma_e}{(1 + \nu \cdot \varepsilon_e)^2} \quad (7.6)$$

The true plastic strain is obtained by subtracting the elastic part of the deformation from the true strain. It should be noted that a large compressive strain in PUR foam causes the Poisson's ratio to tend towards 0 [172]. As a result, in the plastic stage, the engineering stress coincides with the true stress. The obtained yield surface and the predicted true stress-true plastic strain curve are plotted in Figure 7.4.

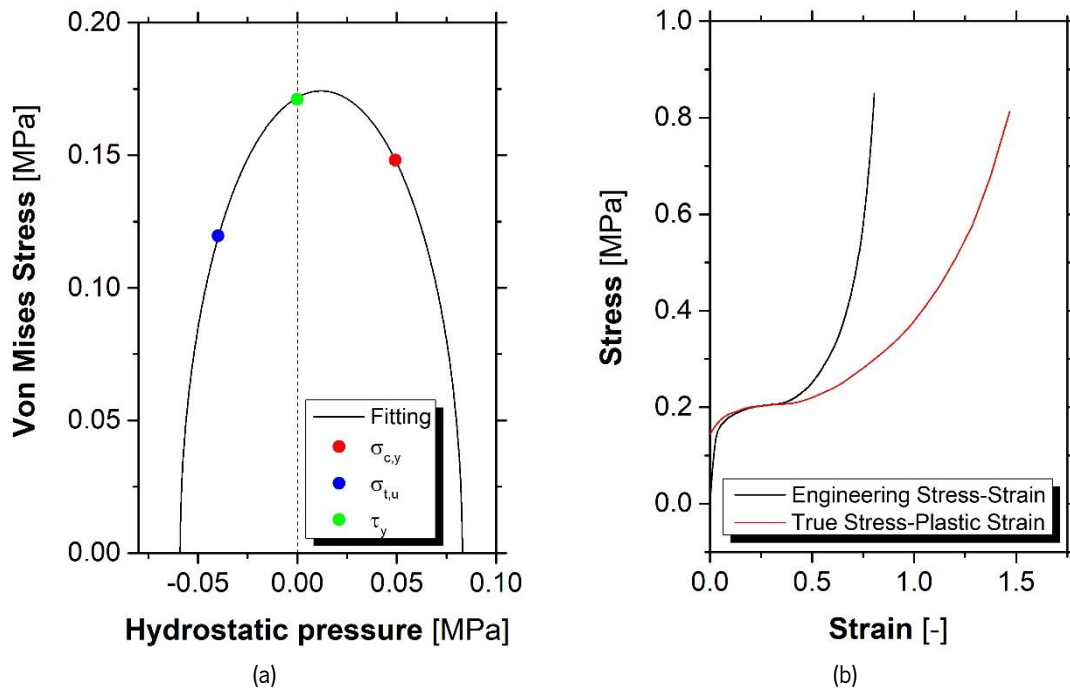


Figure 7.4. Crushable foam constitutive model: (a) predicted yield surface; (b) engineering stress-strain curve and predicted true stress-plastic strain curve.

The PUR foam material properties used in the FE simulations are listed in Table 7.2. The predicted p_c and p_t , are introduced in ABAQUS as the ratio of the uniaxial compressive yield strength to hydrostatic compressive strength (K) and hydrostatic tensile strength to hydrostatic compressive strength (K_t), respectively.

Table 7.2. PUR foam properties for the simulation of the flatwise compressive and tensile tests.

ρ [kg/m ³]	E [GPa]	ν [-]	K [-]	K_t [-]
40	5.7	0.3	1.78	0.71

A numerical model, including material and geometrical nonlinearities, was built to simulate the flatwise compressive and tensile test stress-strain curves (see Figure 7.5).

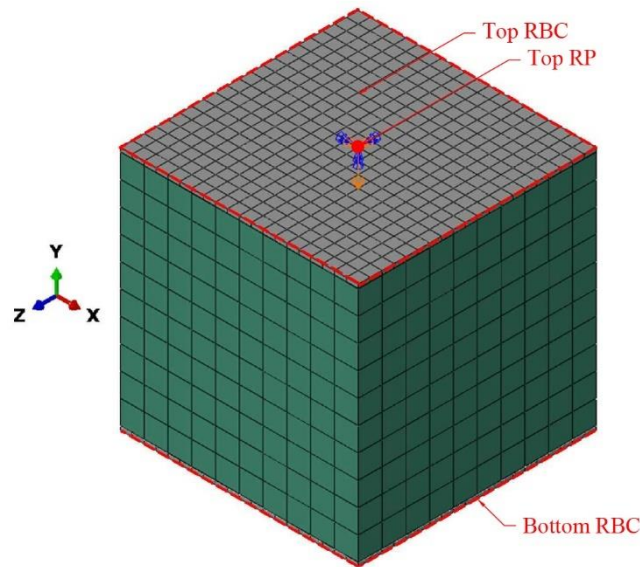


Figure 7.5. Flatwise compressive test: mesh, loading, and support conditions.

To model the PUR foam core and the steel face sheet, 3D continuum solid elements (C3D8R) were selected based on a reduced integration scheme. The core was meshed using hexahedral elements with a side length of 6 mm, while the face sheets were meshed using hexahedral elements with a side length of 3 mm. The steel face sheet was assumed to exhibit a linear elastic material behaviour. The nodes of the bottom face sheet were tied to a reference point and fully constrained. To apply compressive and tensile loading, a vertical displacement was imposed on the top reference point (top RP in Figure 7.5), which controlled the motion of the nodes of the top face sheet (top RBC in Figure 7.5). The same solver scheme as the coupon tensile test model was used.

The plot of the numerical simulations of the flatwise compressive and tensile tests against the experimental curves is presented in Figure 7.6. The average compressive behaviour of the PUR foam is accurately reproduced up to 0.6 strain (see Figure 7.6a), which is deemed sufficient for the simulation of the mechanical behaviour of the sandwich panel. On the other hand, the numerically predicted tensile behaviour shows a nonlinear part of the curve, which was not observed during the experiments (see Figure 7.6b).

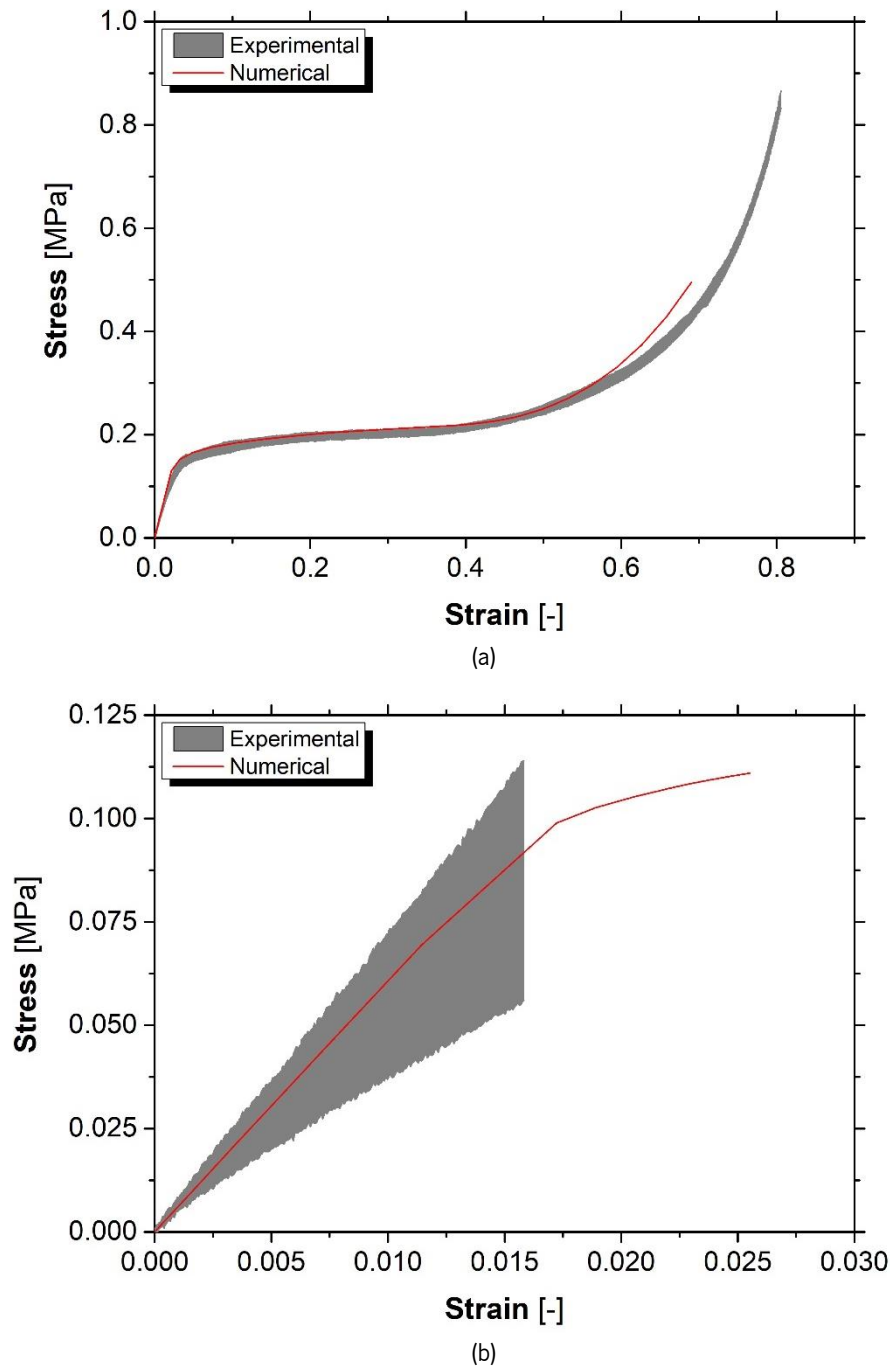


Figure 7.6. Comparison between experimental and numerical results: (a) flatwise compressive test; (b) flatwise tensile test.

The analyses of the flatwise compressive and tensile tests terminated because the Newton-Raphson solver scheme failed to converge. This was due to excessive distortion in the corner elements of the model. Convergence difficulties caused by excessive element distortion are common when using the crushable foam material model in simulations involving large compressive and shear deformation [157]. To address this issue in future studies, special attention should be given to meshing the corner region so that the shapes of the elements become more regular as deformation occurs.

It is important to conduct further research on the brittle behaviour of foam cores in tension as the experimental results on small- and full-scale sandwich panels indicate that the developments of cracks inside the core can influence the ultimate failure and post-peak behaviour in some cases. Crack propagation has been observed to be parallel and close to the interface in the edgewise compressive test, while crack kinking occurred in some specimens subjected to shear and flatwise tensile tests, with the crack propagating from the core to the core-to-face sheet interface. To include these phenomena in the numerical environment, future studies should focus on two strategies found in the literature.

A first approach involves using zero-thickness cohesive elements to simulate the interaction between the core and face sheet during crack propagation [50], [228]. The cohesive elements' constitutive relationship is described by a traction-separation law. Often, damage initiation is set equal to the tensile and shear strength of the foam cores. Specific testing is usually required to determine the critical fracture energy (G_c) that describes the damage evolution. The double cantilever beam (DCB) test is the most common testing method for mode I fracture [183], while the single lap shear test [229], end notched flexure test [230], and modified DCB test are used for mode II [73]. Analytical methods can be used to obtain G_c from the experimental load-displacement curves. The parameter can also be determined by fitting the FE model with the experimental load-displacement curves.

A second approach involves defining failure criteria for the face sheet and the core materials [231], [232]. Maximum principal stress or strain failure criteria can be included in the ABAQUS software by means of a user subroutine. Damage evolution for the foam core may require evaluating input parameters such as the fracture toughness of the foam core through experimental testing, specifically the Single Edge Notch Bending (SENB) test.

A combination of the two previously mentioned strategies can simulate both interface failure and possible crack kinking into the core and/or face sheet of the sandwich panel [233], [234]. Future research should focus on exploring the advantages and disadvantages of different modelling strategies to strike a balance between accuracy and computational cost.

7.2. EDGEWISE COMPRESSIVE TEST

A FE analysis was conducted to replicate the results of the edgewise compressive tests in terms of initial stiffness and ultimate load. The sandwich panels' face sheets and core were modelled as described in Sections 7.1.1 and 7.1.2, respectively (see Figure 7.7). As a simplified strategy is adopted, it is assumed

that the face sheets and core are perfectly bonded. Therefore, a surface-based tie constraint was applied to ensure that the core's outer surface had the same translational motion as the face sheet's inner surface.

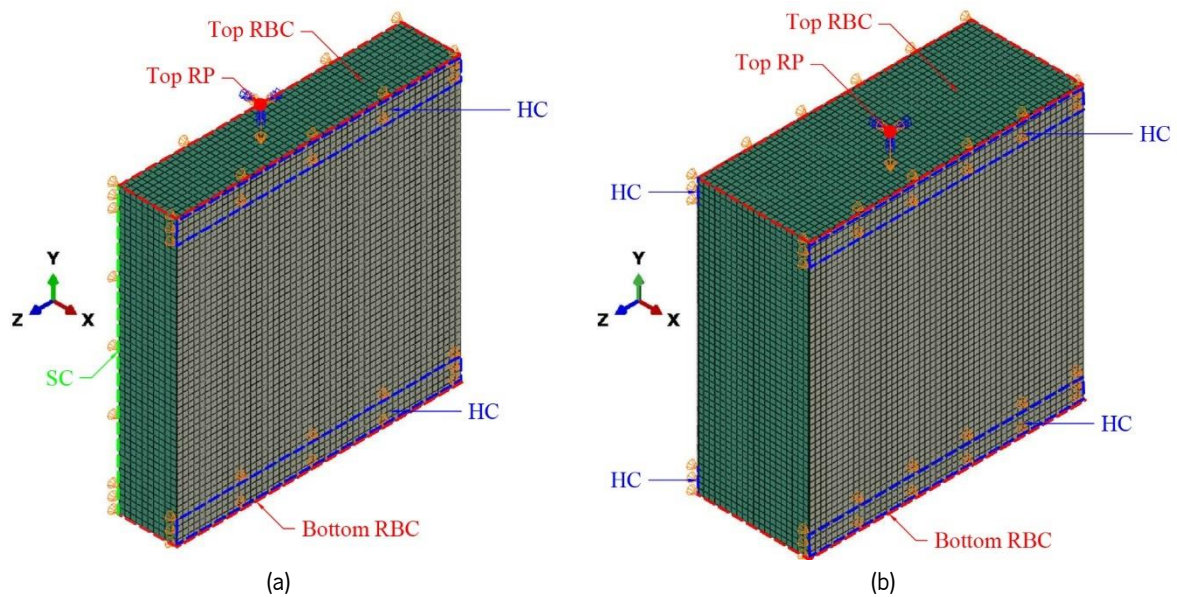


Figure 7.7. Edgewise compressive test numerical model: (a) EC-1.0 set; (b) EC-1.5 set. Notes: HC – horizontal constraint; RBC – rigid body constraint; RP – reference point; SC – symmetry constraint.

The top and bottom nodes of the assembly (top RBC in Figure 7.7) are tied by means of rigid body constraint to two reference points to which the boundary conditions are applied. The bottom reference node is completely constrained, while the top reference node (top RP in Figure 7.7) is constrained in all degrees of freedom, except for vertical translation. Moreover, the nodes of the face sheets in contact with the solid steel lateral bars can only move vertically and not horizontally (HC in Figure 7.7). Due to the symmetry of the panel and the failure mode, only half of the EC-1.0 specimens were simulated, and appropriate boundary conditions were applied to the plane of symmetry (SC in Figure 7.7a).

Preliminary buckling analyses were conducted on numerical models with different mesh sizes. The optimal mesh size was determined based on the critical load of the first buckling mode and computational time. For the nonlinear analyses a global mesh size of 5 mm was chosen (see Figure 7.8). To induce buckling and failure in the mode shape observed during the experiment, which was wrinkling and global buckling, initial geometrical imperfections were introduced in the model.

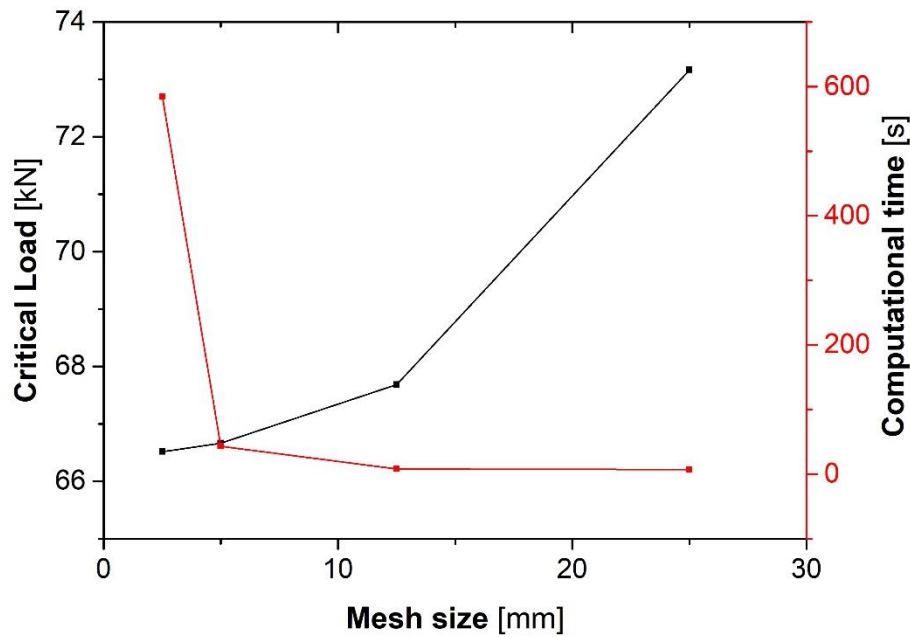


Figure 7.8. Mesh convergence study of the numerical model of specimens EC-1.0.

In order to simulate imperfections, a linear buckling analysis was performed. To retrieve the first tenth-order eigenvalues, a unitary load was applied to the top reference point along the vertical direction. The obtained buckling mode shapes were then identified and used to perturb the initial geometry in the subsequent analysis, considering nonlinear material and geometrical effects.

The linear buckling analysis of specimens EC-1.0 captured buckling mode shapes with fundamentally similar half wavelengths attributed to local buckling. Therefore, the first buckling mode with the lowest critical load, which is shown in Figure 7.9a, was applied to the model. The half wavelength of the 1st buckling mode (63.8 mm) is in good agreement with the experimental results (-2.7% relative difference). On the other hand, the first two buckling mode shapes of specimens EC-1.5 presented significant differences in terms of critical load and half wavelength, as shown in Figure 7.9b and Figure 7.9c. Therefore, the two modes were summed together following the recommendations provided by [235].

To evaluate the sensitivity of the model, three magnitudes of imperfections were taken into consideration. These magnitude values were determined using the method proposed by [236]. As per [235], cumulative distribution function (CDF) values were estimated based on data collected on geometric imperfections of cold-formed steel members. The CDF values connect the probability of occurrence ($\mathbf{P}$) with a particular imperfection magnitude. A CDF value is represented as $\mathbf{P}(\Delta < d)$ and indicates the likelihood of a randomly chosen imperfection (Δ) being smaller than a particular imperfection value (d) [235]. For instance, $\mathbf{P}(\Delta < d_1) = 75\%$ corresponds to $\frac{d_1}{t} = 0.54$ mm, namely a CFS structural member with a cross-section thickness (t) of 1.0 mm is expected to have an initial local imperfection (d_1) less than 0.54

mm 75% of the time. The magnitudes of the imperfections considered for the wrinkling and global buckling are reported in Table 7.3.

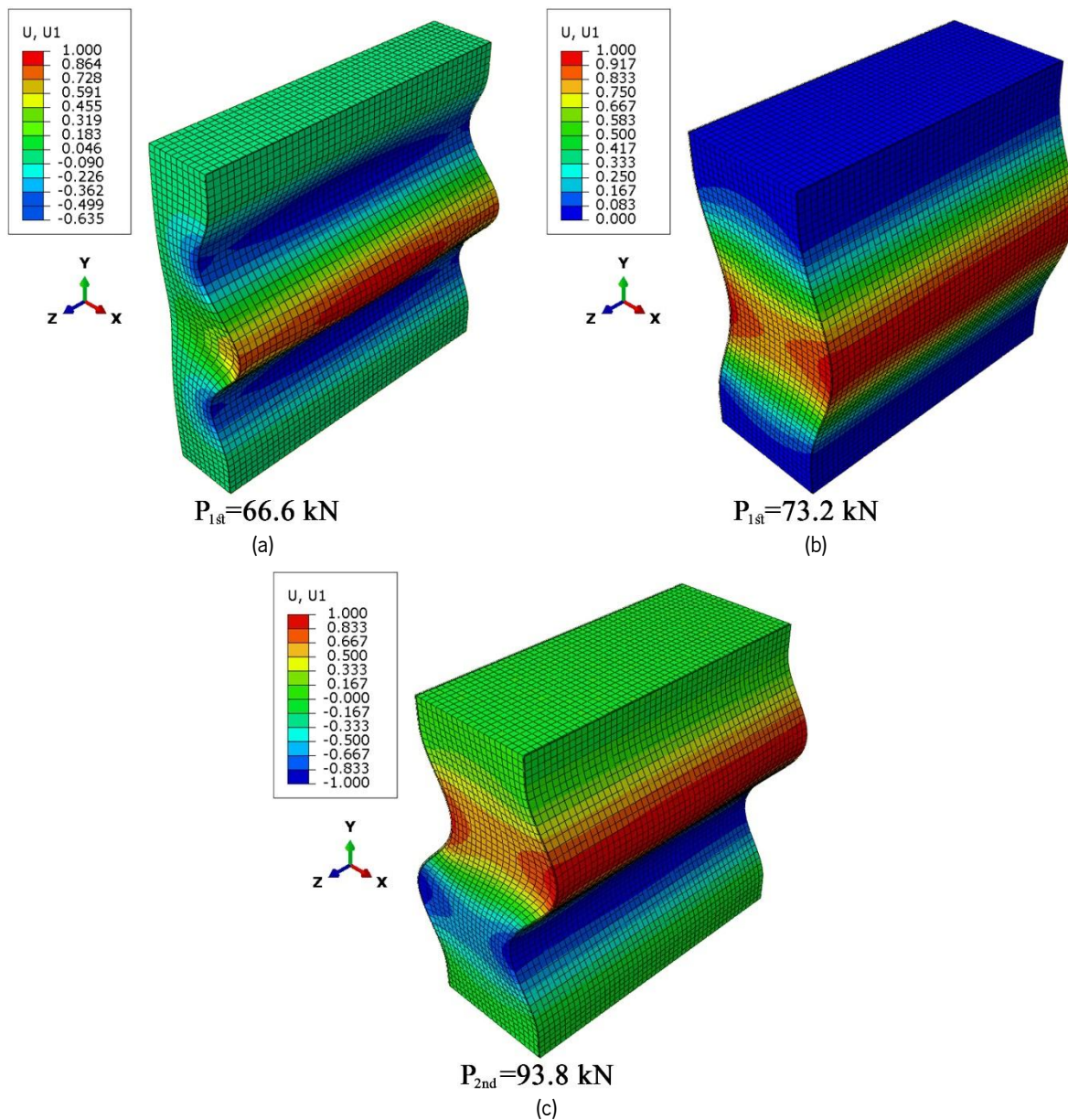


Figure 7.9. Buckling eigenvalues and mode shapes: a) 1st mode of EC-1.0 set; b) 1st mode of EC-1.5 set; c) 2nd mode of EC-1.5 set.

Table 7.3. Statistical magnitude of imperfections.

$P(\Delta < d)$	Wrinkling	Global buckling
	d_1/t	L/d_2
25%	0.17	4755
50%	0.31	2909
75%	0.54	1659

Notes: d_2 – global imperfection.

Numerical models were created to simulate the axial response of sandwich panels until failure initiation, by incorporating nonlinear material properties and initial geometrical imperfections. To replicate the outcomes of the edgewise compressive tests, a vertical displacement of 0.3 mm was assigned to the top reference node. In order to determine the impact of the geometrical imperfections on the axial stiffness and peak load of sandwich panels and to establish the most appropriate magnitudes and combinations, the numerical results were compared to the experimental curves depicted in Figure 7.10. The vertical displacements were calculated from nodes located in the same positions as those used for the plot of the results obtained from the DIC readings in Chapter 4.

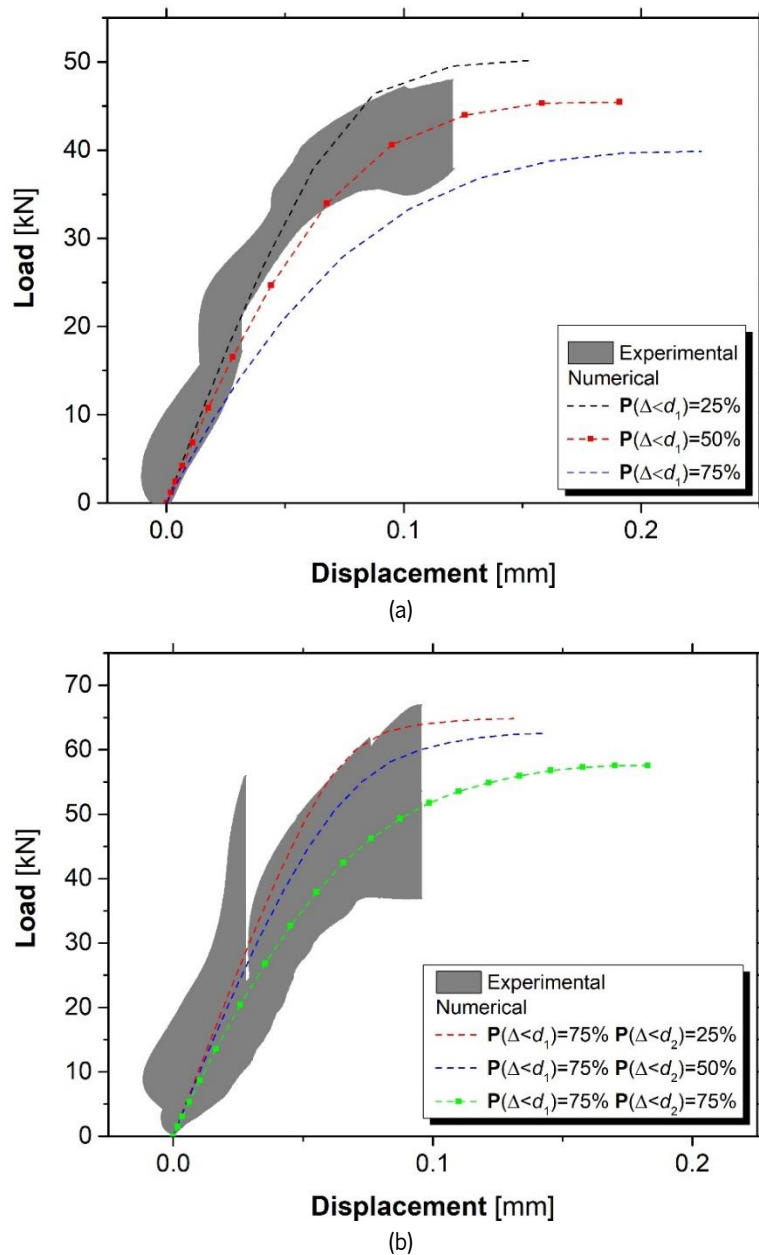


Figure 7.10. Comparison between experimental and numerical load-displacement curves: a) EC-1.0; b) EC-1.5. Note: The numerical curves with the smallest relative difference from the experiments are highlighted with square symbols.

All the numerical analyses terminated when the top reference point reached the prescribed vertical displacement. For what concerns specimens EC-1.0, $\mathbf{P}(\Delta < d_1) = 50\%$ yields the most accurate results both in terms of axial stiffness (460.1 kN/mm) and ultimate load (45.5 kN), with a relative difference of -1.7% and +3.2%, respectively. Regarding specimens EC-1.5, after a preliminary study on the influence of the 1st buckling mode shape, $\mathbf{P}(\Delta < d_1) = 75\%$ was selected as the adequate magnitude to reproduce the behaviour of the sandwich panels. In the second stage, keeping the magnitude of the 1st buckling mode shape constant, the 2nd buckling mode shape was added to the initial configuration. The amplitudes $\mathbf{P}(\Delta < d_1) = 75\%$ and $\mathbf{P}(\Delta < d_2) = 75\%$ provide an axial stiffness of 589.9 kN/mm and a peak load of 57.6 kN with a relative difference of +8.3% and less than +0.1%, respectively. Generally, the FE and experimental results show a relative difference of less than 10% and 5% in terms of axial stiffness and peak load, respectively, as summarised in Table 7.4.

Table 7.4. Comparison of experimental and numerical results of the edgewise compressive test.

Approach	K [kN/mm]		P_{wr} [kN]	l_w [mm]	P_{cr} [kN]
	EC-1.0	EC-1.5	EC-1.0	EC-1.0	EC-1.5
Experimental	468.0	541.0	44.0*	65.5	57.5
Numerical	460.1 (-1.7%)	589.9 (+8.3%)	45.5 (+3.2%)	63.8 (-2.7%)	57.6 (+0.2%)

Notes: the values between parentheses are the corresponding relative differences; highlighted with “*” is the average wrinkling load of specimens EC-1.0-3, EC-1.0-4, and EC-1.0-5; K average value of stiffness; l_w is the wrinkling half wavelength; P_{wr} is the critical wrinkling load; P_{cr} is the global buckling load.

Nevertheless, small drops and changes in slope are not observed in the numerical curves of both specimens EC-1.0 and EC-1.5 (see Figure 7.10). This could be due to various reasons related to the modelling strategies adopted. The ununiformed ends of the specimens and the corresponding gaps with the loading plate were not reproduced in the numerical environment. The compressive load is applied through vertical displacement of the top reference node. Additionally, the perfect bond between the face sheets and the core imposed in the numerical model may not be representative of the real interface behaviour.

Stresses developed in the core of the sandwich panel at peak load for the numerical models presenting the least difference with the experimental curves are shown in Figure 7.11. The maximum tensile stress in the normal direction of the sandwich panel is 0.11 MPa which exceeds the linear stage of the PUR

foam according to the results of the calibration of the crushable foam constitutive model (see Figure 7.6b). The maximum tensile stress corresponds to the region near the face sheet-to-core interface where the face presents an outward buckling wave. The location of the maximum tensile stress corresponds to the initiation of debonding observed in the experiments.

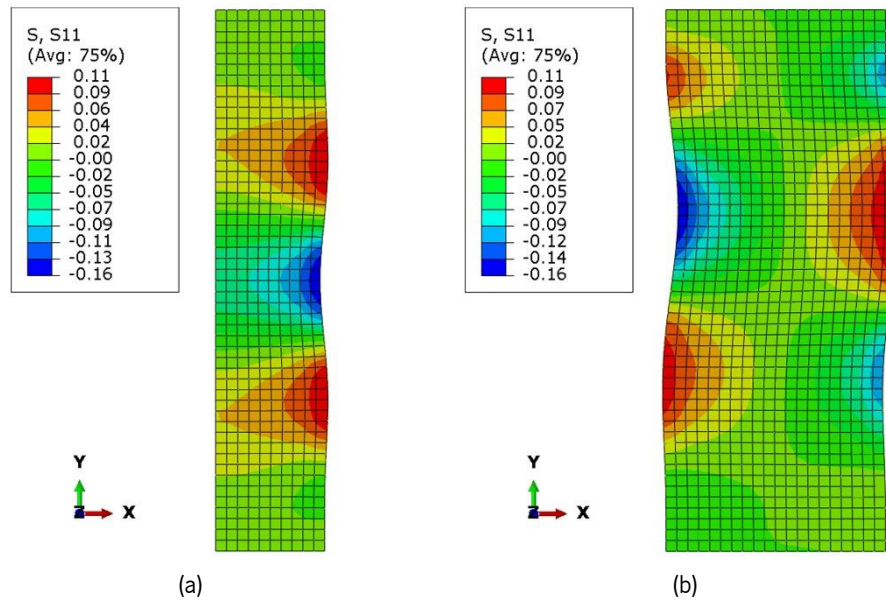


Figure 7.11. Normal stresses in the PUR foam core: (a) EC-1.0; (b) EC-1.5. All units in [MPa].

Despite the reasonable agreement in terms of deformed shapes at peak loads, it appears that further studies shall be required that not only account for the effect of the initial imperfections but also for the tensile cohesive failure of the PUR foam, as mentioned already in Section 7.1.2. Such studies are required to clearly identify the initiation of failure in the sandwich panel and the possibility of cracks in the bulk of the PUR foam and/or at the interface with the face sheets occurring before the attainment of the peak load. The inclusion of the PUR foam tensile cohesive failure, debonding at the face sheet-to-core interface, and layers of PUR foam through the thickness of the sandwich panel with different mechanical properties shall be considered in future studies to accurately reproduce the behaviour of the composite structure.

7.3. FULL-SCALE SANDWICH PANEL

FE models were developed to simulate the static and dynamic behaviour of the sandwich panel. The same modelling strategy for the face sheet, core and face sheet-to-core interface as those for the edgewise compressive test was used to model the full-scale sandwich panel. Regarding the cross-section, similarly to what has been done for the analytical calculations, a built-up box beam is considered instead of explicitly modelling the CFS plain and lipped C-sections as well as the adhesive joining the two profiles.

The overlapping parts of the C-section profiles are considered by assigning a section property with a shell element thickness of 2 mm. On the other hand, the flange of the plain C-section (top cover of the panel) is disregarded as a conservative assumption. The overall FE model cross-section is illustrated in Figure 7.12.

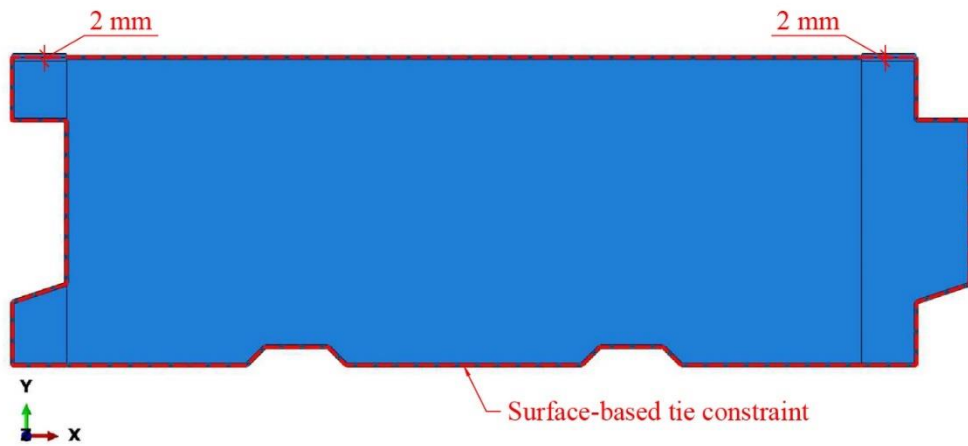


Figure 7.12. Cross-section of the FE model of the sandwich panel.

After some preliminary analyses it was found that both S4R shell elements and C3D8R solid elements with an approximate size of 40 mm of side were optimal in terms of accuracy, convergence, and computational cost (see Figure 7.13). The stiffener and web regions of the cold-formed steel profiles were discretised with a finer mesh to minimise geometrical and, hence, numerical inaccuracies [237].

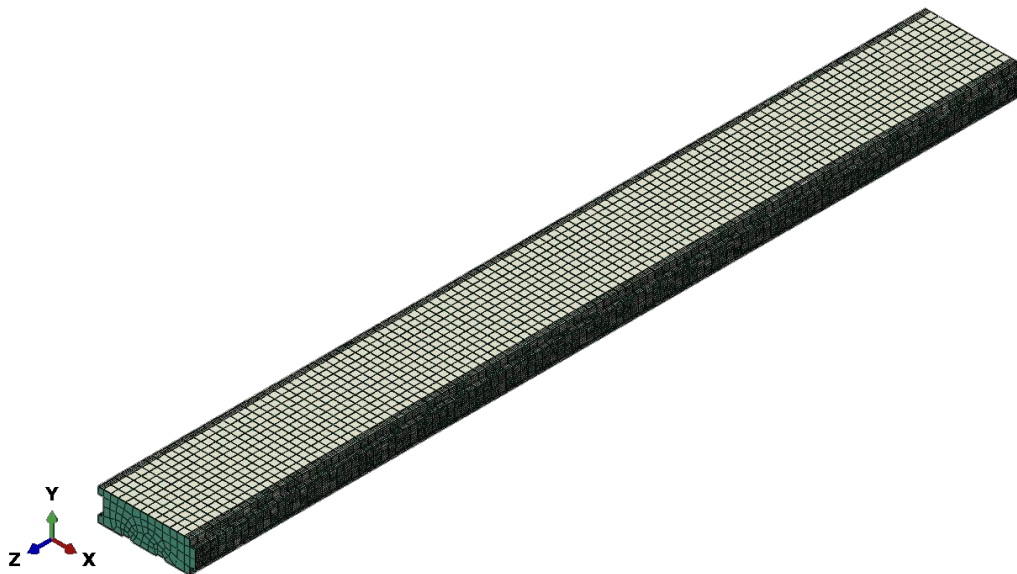


Figure 7.13. Mesh of the FE model of the sandwich panel.

7.3.1. Three-point variable span bending test

In the first stage of the numerical study a linear elastic analysis was performed with the definition of the elastic material properties of the components of the sandwich panel to replicate the results of the variable span flexural test. In the analysis the 3D FE model was supported and loaded by 10 mm thick and 100 mm wide metal plates modelled with 3D continuum solid elements. Linear isotropic elastic material properties were assigned to the plate elements with an elastic modulus of 210 GPa and a Poisson's ratio of 0.3. In order to reproduce the experimental boundary conditions the nodes of the bottom face of the metal plates resting on the roller supports were constrained in the vertical direction, while rigid body motion was prevented by constraining the horizontal translations of one support (see Figure 7.14). A tie constraint was applied between the support and loading plates and the face sheets of the sandwich panels. The mesh of the CFS box beam was refined at the support and loading regions with an offset equal to the thickness of the sandwich panel to better resolve the strain contours. The load was applied in displacement control on the top face of the loading plate. The Newton-Raphson solution scheme is employed.

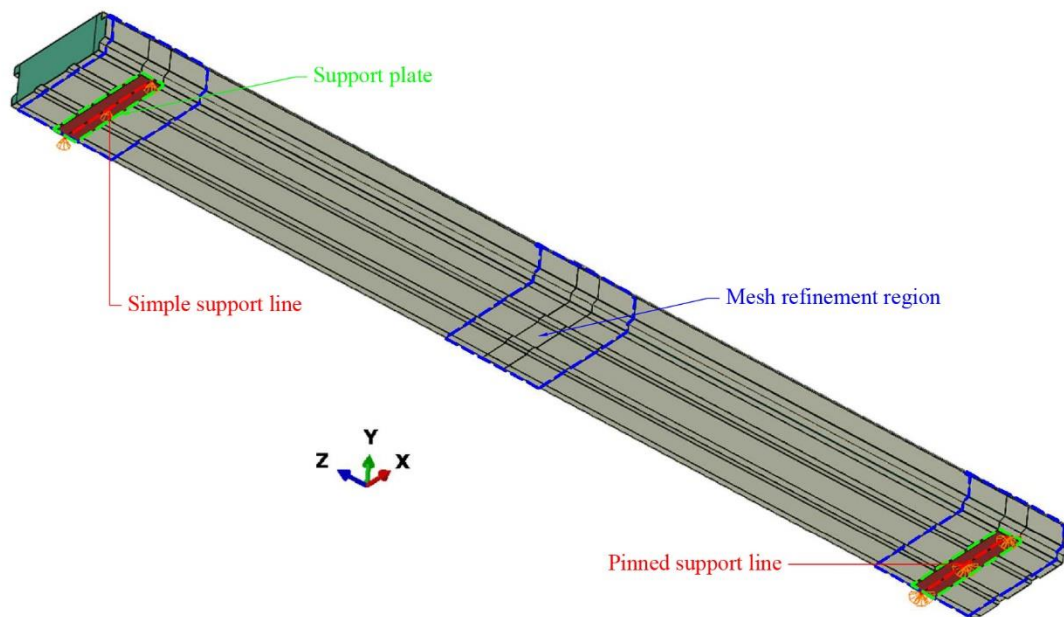


Figure 7.14. Details of the FE models of the variable span flexural test.

The results of the linear elastic analyses for the different span lengths are illustrated in Figure 7.15 and compared to the experimental results. In Table 7.5 the comparison between the measured and calculated bending and shear stiffness of the tested panel for both analytical and numerical predictions are reported.

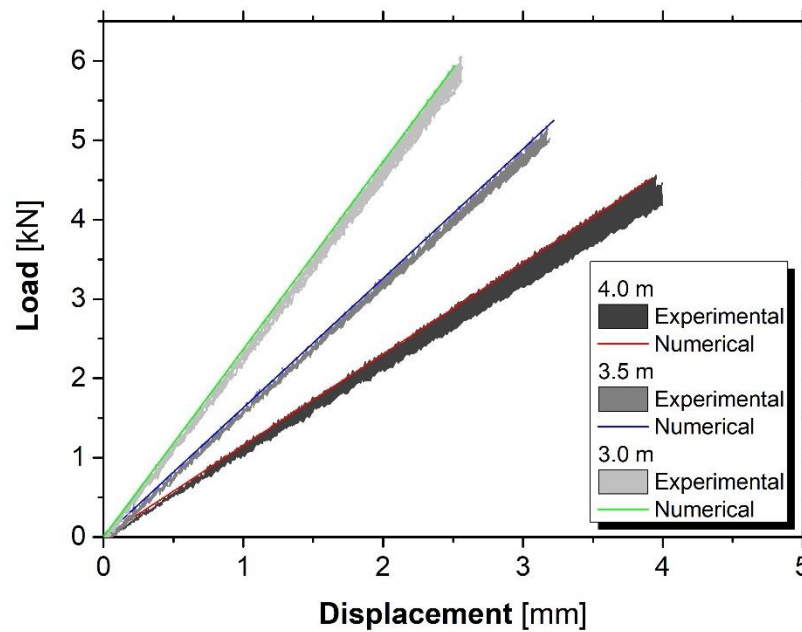


Figure 7.15. Load-displacement curves of the variable span flexural test: comparison of experimental and numerical.

Table 7.5. Experimental, analytical, and numerical bending and shear stiffness.

Approach	D [kNm ²]	U [kN]
Experimental	1757.1	7176.5
Analytical	1707.4 (1.03)	8458.2 (0.85)
Numerical	1919.2 (0.92)	5828.9 (1.23)

Notes: the values between brackets represent the ratio between experimental measures and analytical/numerical predictions.

There is a general good agreement between the measured and predicted values of stiffness with the prediction of bending stiffness generally being more accurate than that of the shear stiffness. Different factors may contribute to this discrepancy. In the FE models as a conservative assumption, the flanges of the top C-section were disregarded as the adhesive joint is located between the lips of the bottom C-section and the web of the top C-section. Nevertheless, the numerical results suggest that they may contribute significantly to the overall shear stiffness of the sandwich panel.

The numerical model is overestimating the bending stiffness of the sandwich panel in comparison to both experimental and analytical values. The analytical values do not account for the presence of the foam, which means that the numerical model is expected to be stiffer. However, the numerical model does capture the varying levels of strains across the width of the sandwich panel, as shown in Figure 7.16a.

In Figure 7.16b, the distribution of bending stresses for the span length of 4.0 m and an applied bending moment of 4.5 kNm at the cross-section midspan is compared to the numerical results.

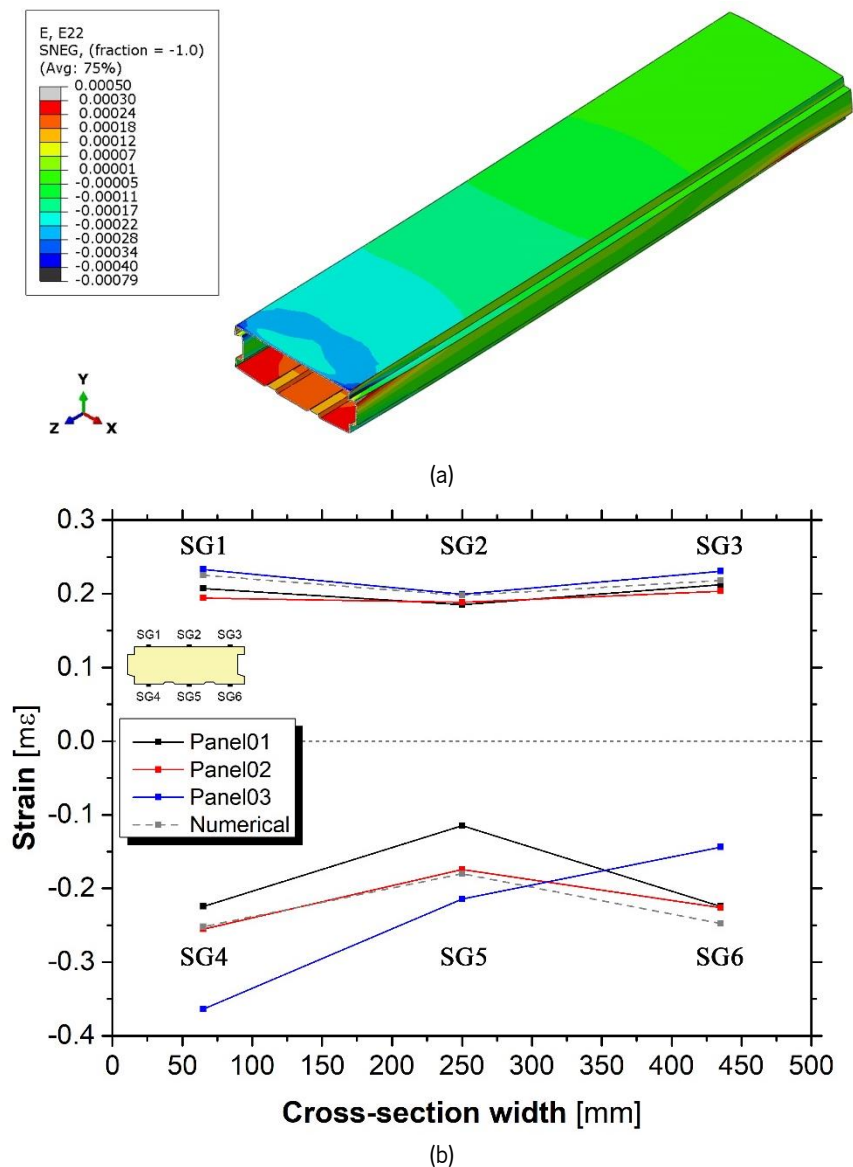


Figure 7.16. Bending strains in the sandwich panels: (a) FE model plot for half panel (magnification factor x20);
(b) comparison with experimental measurements.

The FE model is able to capture the overall trend observed in the experiments and are in line with the theory behind the effective width method [114]. This means that higher strains are present near the webs where the top face sheet plate is supported, except for Panel03 whose behaviour is explained in Chapter 5. However, the FE model generally predicts smaller strain differences between the mid-width and the edges of the top face sheet. The higher strain gradient measured in the experiments may be due to the presence of initial imperfections in the CFS cross-section resulting from the cold-forming process, which were not considered in the FE model.

The experimental and numerical results show a good agreement, which implies that the modelling strategy is reasonable for load levels within 50% of the ultimate load of the sandwich panels. The FE model's cross-section is a box beam, indicating that there is negligible slip between the top and bottom CFS C-section profiles that make up the sandwich panel. Additionally, since the face sheet and the core of the FE model's interface are assumed to be perfectly bonded, it may be suggested that no damage is present up to 50% of the ultimate load of the sandwich panel at the face sheet-to-core interface.

7.3.2. Flexural dynamic test

The FE model described in 7.3 is used to perform an eigenvalue extraction procedure to obtain the natural frequencies and the corresponding mode shapes of the sandwich panel. Simplified boundary conditions were considered for the simulation. Vertical displacements of the nodes of the bottom face sheet corresponding to the location of the axis of the steel roller were constrained. Furthermore, to prevent rigid body motions, the horizontal translations of one of the supports were constrained. The first flexural and torsional vibration modes of the sandwich panels obtained with the numerical models and the comparison with the experimental data are illustrated in Figure 7.17.

The mode shapes for the first two natural frequencies show a qualitative good agreement with those obtained from the treatment of the experimental data with the software Artemis Modal (Version 6.0). Results presented in Table 7.6 show a reasonable agreement for what concerns the first flexural frequency, whereas a larger difference is observed for the first torsional frequency. As shown in the previous section, the FE model appears to be slightly overestimating the sandwich panel bending stiffness, thus higher flexural natural frequencies are expected. The assumption of the simplified box section may not be representative of the torsional rigidity of the sandwich panel. Coarse mesh density and the use of elements with linear interpolation functions may also affect the accuracy of the estimation of the natural frequency [238].

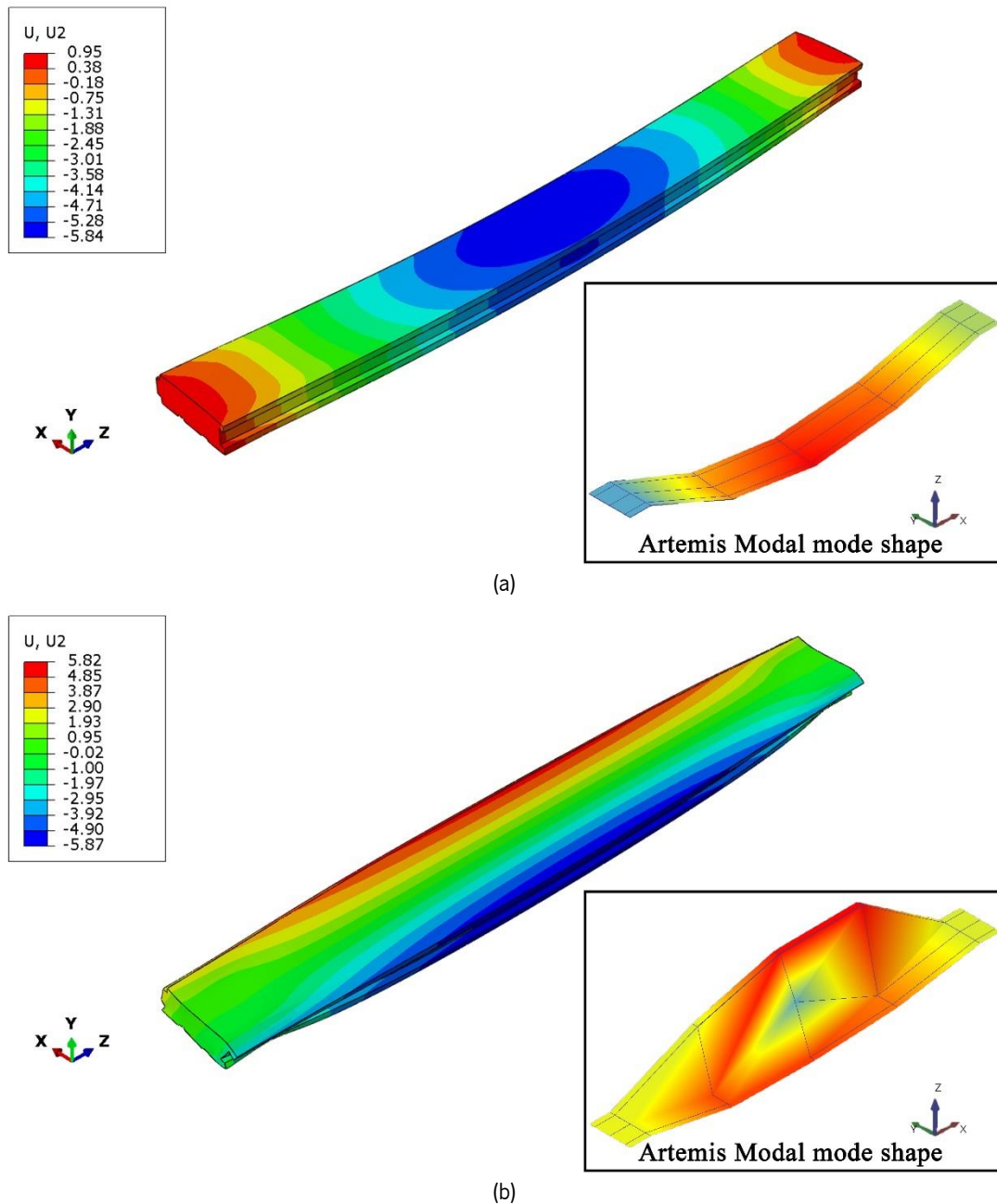


Figure 7.17. Mode shapes extracted from the FE model (magnification factor x20): (a) flexural mode; (b) torsional mode.

Table 7.6. Measured and predicted natural frequencies (in [Hz]) for the sandwich panel.

Approach	Flexural mode	Torsional mode
Experimental	25.9	44.4
Analytical	33.2 (0.78)	-
Numerical	29.68 (0.87)	58.77 (0.76)

Notes: the values between brackets represent the ratio between experimental measures and analytical/numerical predictions.

7.3.3. Four-point bending test

The modelling strategy is analogous to that of the three-point bending test. A mesh with an approximate size of 10 mm of side is adopted at the loading and support regions to capture the local instability and consequent plate folding observed during the experiment. Particular attention shall be given to the number of elements required for the stiffener and lips of CFS member. At least four elements are necessary to replicate bending deformations of such cross-section elements [239].

Since rotations of the loading plates was observed during the experiment and ultimate failure concentrated in one of the loaded sections, a displacement control analysis previously conducted for the three-point bending was no longer an option. Therefore, a pressure load on the top surface of the loading steel plate was applied instead (see Figure 7.18).

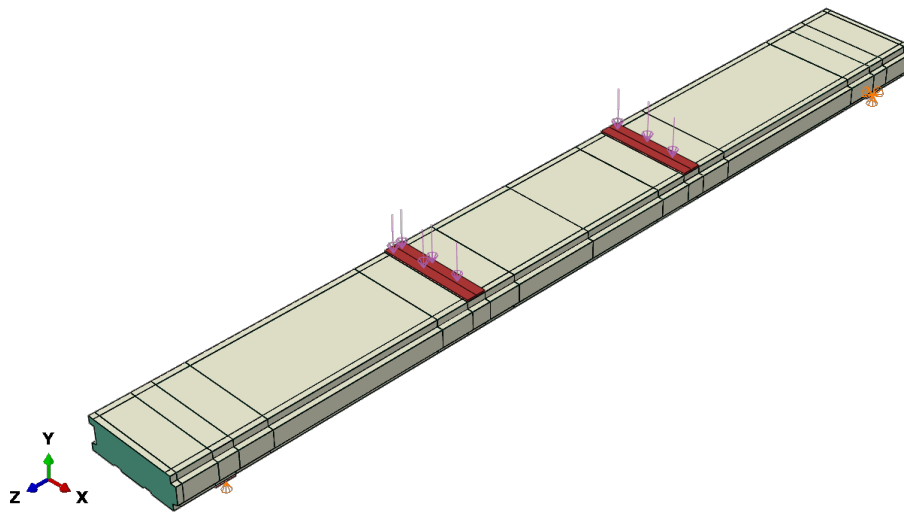


Figure 7.18. Loading setup of the FE model of the four-point bending test.

In order to follow the post-peak response of the sandwich panel, namely a load path displaying a negative stiffness, the modified Riks method as a solver scheme is adopted. The modified Riks method, also known as the arc-length method, employs increments of work rather than increments of displacement or force alone [240]. The developed numerical model accounted for geometrical and material nonlinearities.

The load-displacement curve obtained from the FE model is plotted against the experimental results in Figure 7.19.

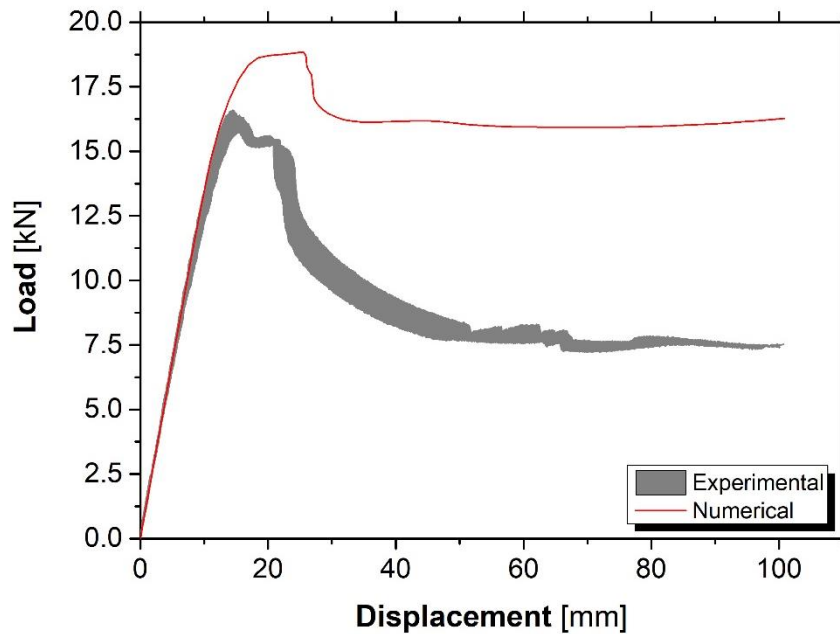


Figure 7.19. Comparison of experimental and numerical results of the four-point bending test.

In Table 7.7 a comparison between the measured and calculated deflection at SLS load level (δ_{SLS}), initial elastic stiffness (K) and ultimate load (F_u) for the analytical and numerical predictions is reported. The numerical predictions show a very good agreement in the linear elastic domain of the sandwich panel. However, for larger deformations the FE model does not correctly represent the behaviour of the sandwich panel, specifically the ultimate load predicted is about 15% higher than the experimental value. Nevertheless, the difference provide insight into the mechanical features of the problem that should be considered to replicate the structural behaviour of the sandwich panel.

Table 7.7. Experimental, analytical, and numerical flexural properties of the sandwich panel.

Approach	δ_{SLS} [mm]	K [kN/mm]	F_u [kN]
Experimental	5.44	1.33	16.08
Analytical (M1)	5.06 (1.08)	1.50 (0.88)	35.3 (0.46)
Analytical (M3)	6.15 (0.88)	1.24 (1.07)	11.03 (1.46)
Numerical	5.55 (0.98)	1.37 (0.97)	18.84 (0.85)

Notes: M1 – design method 1; M3 – design method 3; the values between brackets represent the ratio between experimental measures and analytical/numerical predictions.

The FE model uses an equivalent simplified box section which efficiently predicts stiffness and deflection within the linear elastic regime. However, at approximately 16.0 kN, the FE model captures the wrinkles

near the loading plate but cannot reproduce the outward buckling of the top C-section (see Figure 7.20b) since the two CFS components of the sandwich panel are not explicitly modelled. Furthermore, this may suggest that damage in the adhesive layer joining the two CFS components is taking place in the critical cross-section.

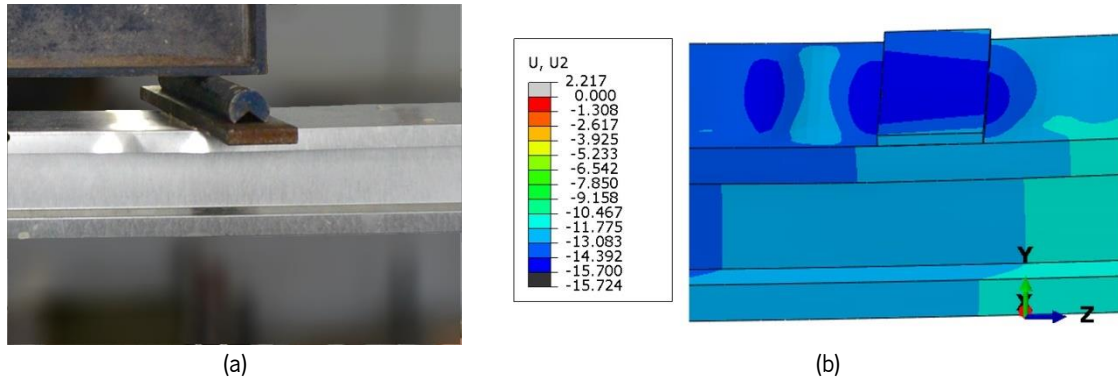


Figure 7.20. Sandwich panel failure mode: (a) experimental; (b) FE model plot of the vertical displacement (magnification factor x5). All units in [mm].

The FE model predicts a gradual loss of stiffness until the attainment of the ultimate load. The wrinkling deformation develop progressively due to the perfect bond enforced at the interface by means of the surface-based tie constraint. On the other hand, a first load drop is observed in the experiment likely due to the failure of the face sheet-to-core interface since the PUR foam is not able to accommodate the deformations resulting from the wrinkling near loading plate. The FE model can capture the significant load drop once local buckling of the webs under one of the loaded sections occurs (see Figure 7.21), qualitatively similar to that observed during the four-point bending test. Nevertheless, the FE model retains about 85% of its ultimate load as opposed to the 50% of the experimental measurements. This may be due to the crack progression within the PUR foam near the top face sheet-to-core interface.

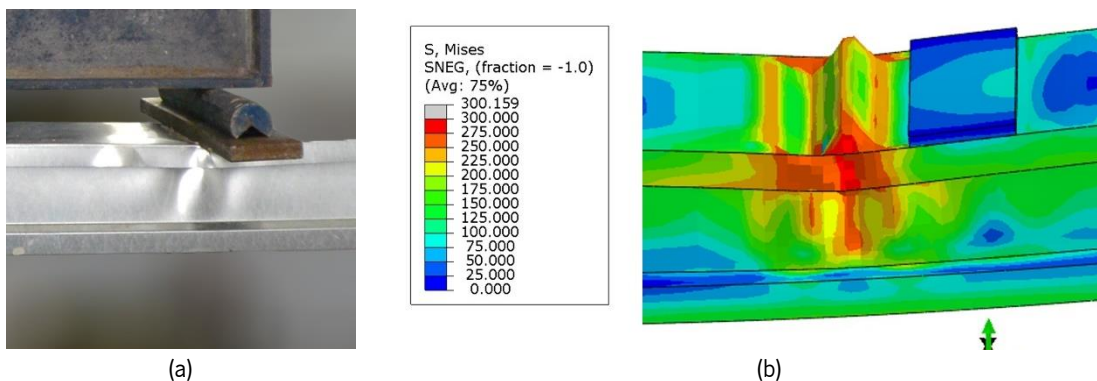


Figure 7.21. Web local buckling: (a) experimental; FE plot of the Mises stress (magnification factor x5). All units in [MPa].

Although a significant difference in terms of load is observed, the descending branch of FE model curve corresponds to a plastic mechanism observed also in the experiments. The web plates folding form yield

lines meeting in the top face sheet. The evolution of the plastic hinge shows a similar pattern to that of the experiment, with the deformed shape matching the experimental behaviour (see Figure 7.22).

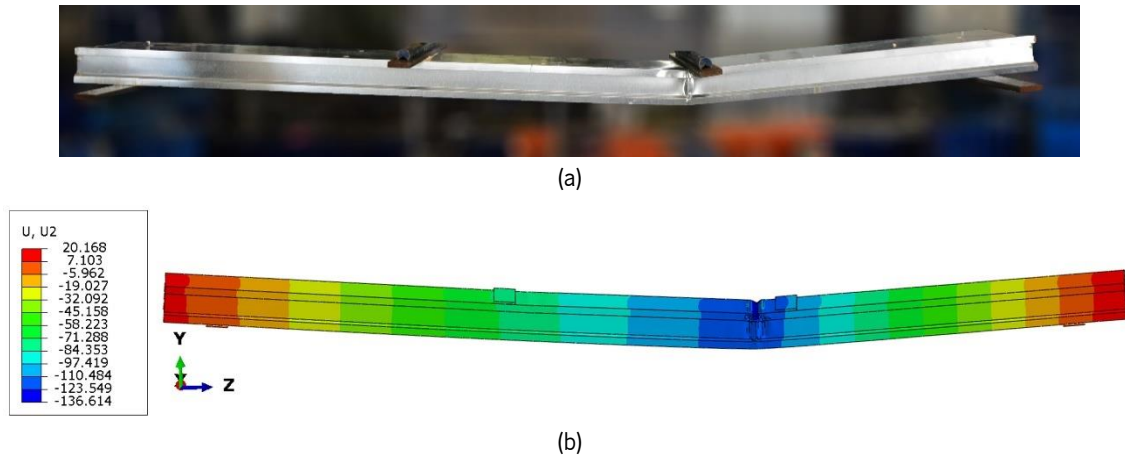


Figure 7.22. Deformed shape of the sandwich panel at 100 mm displacement: (a) experimental; (b) FE model plot of the vertical displacement. All units in [mm].

In the elastic regime, there is a very good agreement between the numerical and experimental results. As discussed in Chapter 5, the contribution of the PUR foam to the stiffness in the elastic regime is negligible. However, its contribution becomes more significant when predicting the peak load of the sandwich panel, where the largest difference between experimental and numerical behaviour is observed. The PUR foam acts as an elastic foundation, supporting the plate elements of the cross-section and increasing their critical buckling load compared to unsupported elements. Therefore, the difference between the numerical and experimental ultimate load of the sandwich panel may indicate the presence of defects in the PUR foam. This issue will be further investigated by sawing the tested sandwich panel to inspect the quality of the PUR foam and assess the possible presence of defects such as voids and cavities within the core.

The differences observed in terms of failure progression and ultimate load between the numerical model and the experimental tests also suggest that reproducing the structural behaviour of the sandwich panel requires a more complex approach. The numerical model accounted for geometrical nonlinearities; however, it did not include initial imperfections in the analysis. Furthermore, the failure progression in the numerical model suggests that the influence of certain physical features should be assessed in future studies: i) explicit model of both CFS components of the sandwich panel cross-section; ii) a failure criterion for the face sheet-to-core interface; iii) a failure criterion for the adhesive joint between the CFS components of the sandwich panel. The above-mentioned physical features were not included in this

initial study to strike a balance between accuracy and convergence of the solution due to the highly unstable behaviour of CFS members [64].

7.4. DESIGN TOOLS

Experimental, analytical, and numerical studies have been conducted to determine the behaviour of the adhesive joint between the CFS profiles and the PUR foam core present in the sandwich panel. The studies have shown that within the range of up to 50% of the measured ultimate load, the joint can be considered perfectly bonded. However, further studies are needed to accurately identify and quantify the contribution of the PUR foam and the adhesive joint in terms of ultimate load. This is due to the complexity of the physical phenomena involved, which makes it difficult to account for their effect in analytical procedures.

The simplified analytical method that uses Eurocode 3 Part 1-3 (2006) [80] and Part 1-5 (2006) [102], based on an equivalent hollow built-up box beam, was validated against the results of the experimental campaign conducted at LEST. This validated model was then used to analyse the structural performance of the proposed sandwich panel under different span lengths and support conditions. To simplify the design process for the industrial partner of the LightSlab R&D Project and ease the dissemination and use of this innovative solution, design tables were created that include imposed load carrying capacity and permissible span lengths of the sandwich panels. Additionally, the design tables provide information on the governing failure criteria of the panels under different support configurations.

The cross-section of the CFS sandwich panel infilled with PUR foam with a density of 40 kg/m^3 considered for this study is presented in Figure 7.23b. The panel has a total width of 500 mm and a height of 170 mm. It is comprised of two C-sections, one of which is unstiffened while the other is stiffened. These sections are joined together in their overlapping regions by an adhesive made of polyurethane-based material. To simplify the design calculations, the following assumptions were made: i) the mechanical resistance of the panel is calculated without considering the contribution of the core PUR foam; ii) the adhesive joint between the top and bottom cold-formed steel C-sections provides a perfect bond; iii) the inner corners of the C-sections have a radius of 1.5 mm; iv) the thickness of the compressive flange is calculated as the weighted average of the thickness of the overlapping regions and non-overlapping regions (see Figure 7.23c). The design tables were developed by means of parametric studies considering steel thickness ranging between 0.8 mm and 1.2 mm and two steel grades according to the Eurocode 3 Part 1-3 (2006) [80], namely S220GD+Z and S250GD+Z.

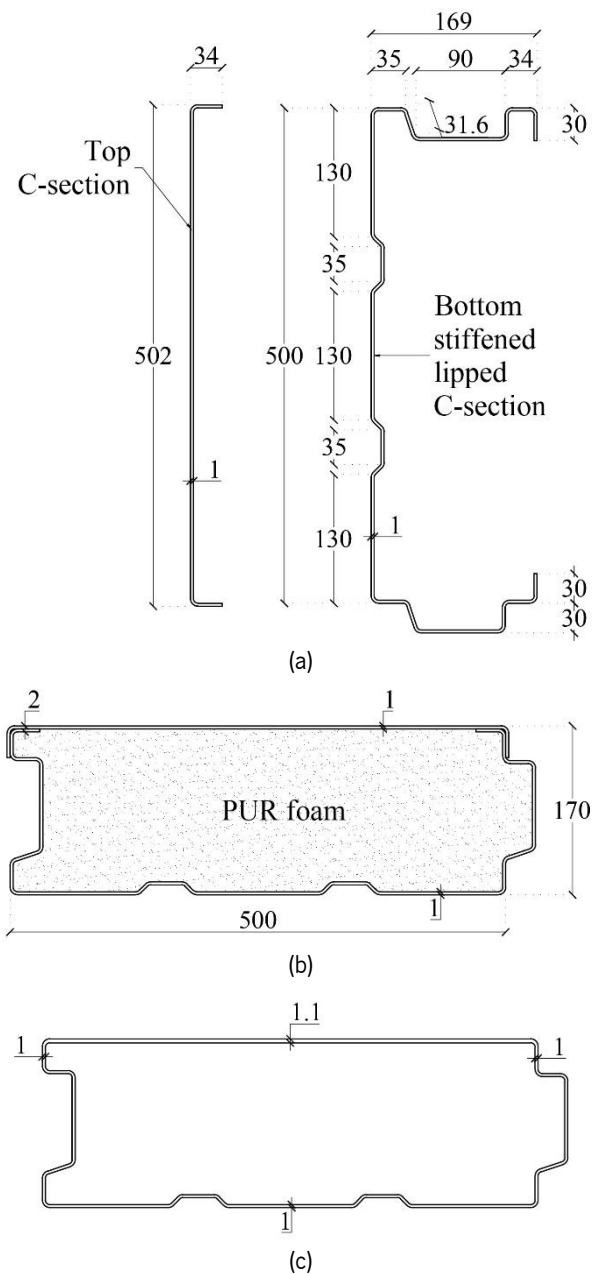


Figure 7.23. Dimensions of the structural member: (a) CFS components of the sandwich panel; (b) cross-section of the sandwich panel; (c) cross-section employed for the design tables. All units in [mm].

Besides the simply supported beam already considered in the preliminary design, a continuous beam with three equally spaced supports is also considered as a structural model for the design tables (see Figure 7.24).

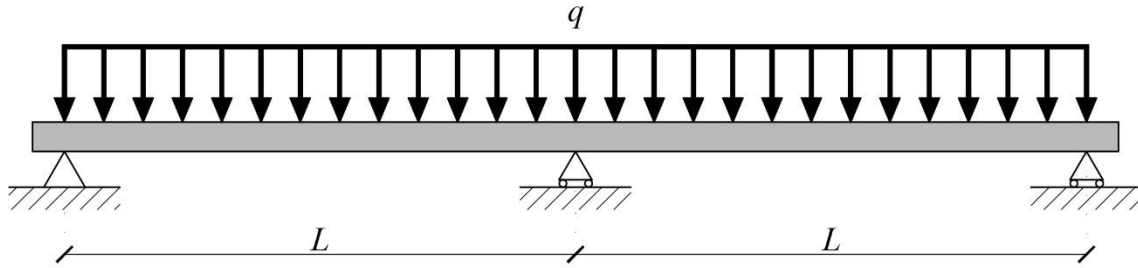


Figure 7.24. Structural model of a continuous beam with three equally spaced supports.

The load-carrying capacity for the verification of the ULS considers the resistance to shear and bending moment for the single panel supported at both ends. The interaction between the bending moment and shear at the supports influences only the load carrying capacity of the continuous panel resting on multiple supports (negative bending moment at the intermediate supports) and is given in Equation (7.7):

$$\frac{M_{Ed}}{M_{Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2 \leq 1.0 \quad (7.7)$$

where M_{Ed} is the design bending moment acting at the internal support, $M_{f,Rd}$ is the moment resistance of the cross-section considering only the effective area of the flanges, $M_{pl,Rd}$ is the plastic moment resistance of the cross-section, and V_{Ed} is the design shear action at the internal support. The calculations are carried out assuming a linear distribution through the thickness of the cross-section. The yielding stress is reached in either the top or bottom fibre of the cross-section depending on the sign of the bending moment.

In the calculation of the cross-section's effective properties, the design calculations exclude the coating layer. For the calculations, a coating thickness of 16 μm , which corresponds to a coating mass Z225 as per EN 10346 (2015) [155], is assumed. This is based on the results of the microscopic examination described in Section 4.1.3.

The same SLS verifications used for the preliminary design of the sandwich panel are also applied to the design tables. Additionally, design method M3 is used to estimate the vertical deflection, as it is the most accurate and conservative approach based on the experimental results presented in Chapter 5.

7.4.1. Design tables

In the following tables, the maximum load carrying capacity of the sandwich panel is reported for span lengths ranging from 2.0 m to 5.0 m at intervals of 0.5 m. The 4.0 m span length is a typical value found in old masonry buildings across Europe, which is one of the target applications of the newly developed sandwich panel.

The values reported in the table correspond to the characteristic imposed load carrying capacity of the sandwich panel. The self-weight of the panel and the weight of other permanent loads (i.e. finishing, suspended ceiling, partition walls, with a value of 1.5 kN/m^2), are already accounted for in the calculations. The imposed load carrying capacity corresponds to the maximum imposed load applicable to the sandwich panel. The value is determined as the minimum among the resistance verifications for the span length considered.

All the tables provide the main geometrical and material characteristics of the sandwich panel, namely steel sheet thickness and structural steel grade (Table 7.8). Table 7.9 to Table 7.12 present the different tables developed, in terms of characteristic maximum load carrying capacity (imposed load) as function of the steel grade, CFS thickness, structural system and span length.

Table 7.8. Geometrical characteristics of the cross-section.

t_{nom} [mm]	t_c [mm]	Mass [kg/m ²]	Sectional area [cm ²]	W_{el} [cm ³]			A_v [cm ²]
				Top compression	fibre in compression	Bottom compression	
0.80	0.77	26.1	11.97	89.52		70.50	2.56
1.00	0.97	30.9	15.06	98.32		93.22	3.22
1.20	1.17	35.8	18.14	137.38		101.17	3.86

Notes: t_{nom} – nominal thickness of the steel sheet; t_c – core thickness of the steel sheet excluding the coating layer; W_{el} – elastic section modulus; A_v – shear area.

Table 7.9. Imposed load carrying capacity of the panel between two supports for steel grade S220GD+Z in kN/m².

t_c [mm]	Span length [m]						
	2.0	2.5	3.0	3.5	4.0	4.5	5.0
0.77	10.71	6.29	3.88	2.44	-	-	-
0.97	15.43	9.29	5.96	3.95	2.64	-	-
1.17	20.75	12.68	8.30	5.65	3.94	2.76	-

Notes: the symbol “-” indicates the sandwich panels did not meet the minimum requirement according to the Eurocode 1 Part 1-1 [101].

Table 7.10. Imposed load carrying capacity of the continuous panel over three equally spaced supports for steel grade S220GD+z in kN/m².

t_c [mm]	Span length [m]						
	2.0	2.5	3.0	3.5	4.0	4.5	5.0
0.77	12.74	7.59	4.78	3.09	-	-	-
0.97	18.36	11.16	7.26	4.90	3.37	2.32	-
1.17	24.27	14.94	9.86	6.80	4.82	3.46	-

Notes: the symbol “-” indicates the sandwich panels did not meet the minimum requirement according to the Eurocode 1 Part 1-1 [101].

Table 7.11. Imposed load carrying capacity of the panel between two supports for steel grade S250GD+Z in kN/m².

t_c [mm]	Span length [m]						
	2.0	2.5	3.0	3.5	4.0	4.5	5.0
0.77	11.99	7.10	4.45	2.85	-	-	-
0.97	17.20	10.42	6.75	4.52	3.08	2.10	-
1.17	23.01	14.13	9.30	6.39	4.50	3.21	2.28

Notes: the symbol “-” indicates the sandwich panels did not meet the minimum requirement according to the Eurocode 1 Part 1-1 [101].

Table 7.12. Imposed load carrying capacity of the continuous panel over three equally spaced supports for steel grade S250GD+z in kN/m².

t_c [mm]	Span length [m]						
	2.0	2.5	3.0	3.5	4.0	4.5	5.0
0.77	12.72	8.42	5.37	2.33	-	-	-
0.97	20.24	12.37	8.09	5.52	3.84	2.70	-
1.17	26.78	16.54	10.98	7.62	5.45	3.95	2.89

Notes: the symbol “-” indicates the sandwich panels did not meet the minimum requirement according to the Eurocode 1 Part 1-1 [101].

The primary factor in designing the structural model of a panel supported at both ends is its moment resistance at midspan. For the continuous sandwich panel resting over three supports, the limiting factor in its design is the combination of bending moment and shear action at the internal support. This finding

is consistent with the results of the four-point bending test of full-scale sandwich panels, which showed that failure occurred at one of the loaded sections. Local instabilities were observed in both the compression face sheet and webs due to the combined action of bending moment and shear force.

It has been observed that a sandwich panel with a core thickness of 0.77 mm can only be used to cover a span of 3.5 m maximum, regardless of the steel grades used. On the other hand, it has been found that a sandwich panel with a core thickness of 1.17 mm, utilizing the steel grade 250GD+Z, can cover a maximum span length of 5.0 m under any of the support conditions studied.

7.4.2. Calculation example

The sandwich panel shall be designed to cover the floor of a residential building. The structural model considered is the simply supported panel at both ends covering a span of 4.0 m. The steel grade S220GD+Z is utilised for the CFS profiles constituting the sandwich panel.

After a preliminary assessment, the floor requires additional acoustic and thermal insulation layers. The value of the permanent loads of the finishing now amounts to 2.5 kN/m². Thus, the following loads shall be considered to design the sandwich floor panel: i) self-weight of the panel; ii) permanent loads due to finishing in the amount of 2.5 kN/m²; iii) imposed load for residential floors in the amount of 2.0 kN/m² according to Eurocode 1 Part 1-1 (2002) [101].

The case study requires the use of Table 7.9. However, the other permanent loads are greater than the value considered to build the table. Thus, the value of the admissible imposed load must be updated. The characteristic value of the imposed load (Q_k) is given by Equation (7.8):

$$Q_k = \frac{R_d - \gamma_g \cdot (G_{k,1} + G_{k,2})}{\gamma_q} \quad (7.8)$$

where $G_{k,1}$ and $G_{k,2}$ represent the self-weight of the panel and the permanent loads, respectively, R_d is the design ultimate load carrying capacity of the panel, and γ_g and γ_q are the partial factor for permanent actions and variable actions, respectively. Solving Equation (7.8) for R_d yields:

$$R_d = \gamma_g \cdot (G_{k,1} + G_{k,2}) + \gamma_q \cdot Q_k \quad (7.9)$$

Using Equation (7.9) it is possible to calculate the ultimate load carrying capacity of the panel considering the original value of the permanent loads equal to 1.5 kN/m². For a span length of 4.0 m two steel core

thicknesses can be chosen, namely 0.97 mm and 1.17 mm. The ultimate load carrying capacities are 6.19 kN/m² and 8.17 kN/m² for the 0.97 mm and 1.17 mm sandwich panels, respectively. Based on the values of R_d it is now possible to determine the updated imposed load carrying capacity of the panel considering the current value of the permanent loads equal to 2.5 kN/m². Equation (7.8) yields the following results:

$$\text{For } t_c = 0.97 \text{ mm} \rightarrow Q_k = \frac{6.19 - 1.35 \cdot (0.155 + 2.5)}{1.5} = 1.74 \text{ kN/m}^2 \quad (7.10)$$

$$\text{For } t_c = 1.17 \text{ mm} \rightarrow Q_k = \frac{8.17 - 1.35 \cdot (0.175 + 2.5)}{1.5} = 3.04 \text{ kN/m}^2 \quad (7.11)$$

The minimum value of Q_k according to the Eurocode 1 Part 1-1 (2002) [101] is 2.0 kN/m². Thus, the final solution for the residential floor building is the sandwich panel with a steel sheet thickness of 1.20 mm. The panel can sustain a imposed load up to 3.04 kN/m².

7.5. CONCLUDING REMARKS

Numerical models were developed to gain insight into the mechanical behaviour of the sandwich panel under different loading and boundary conditions, including edgewise compressive tests, three-point variable span bending tests, flexural dynamic tests, and four-point bending tests up to failure. Information on the key aspects governing the mechanical behaviour of the sandwich panel was provided through a comparison between the numerical analyses, analytical predictions, and the experimental results. The most conservative approach, i.e., the analytical formulas, was selected to develop design tables for the sandwich floor panel system based on such comparison.

Before modelling the whole sandwich panel assembly, numerical analyses were conducted to calibrate the material model for the steel and the PUR foam. The Von Mises yield surface was used to model metal plasticity, and it accurately replicated the behaviour of steel coupon specimens until they failed. However, the crushable foam constitutive model had limitations in replicating the brittle behaviour of PUR foam in tension. Strategies to model the cohesive tensile and shear failure of the PUR foam core, as well as the crack at the face sheet-to-core interface, were highlighted, but they were not included in the developed FE model to improve convergence.

The adopted simplifying modelling strategy resulted in good agreement of the axial and flexural stiffness with the small-scale sandwich panel in edgewise compression and the full-scale sandwich panel in bending, respectively.

The agreement in the elastic regime and the simultaneous difference in the ultimate load between the numerical analysis and the bending tests may indicate manufacturing issues in the core of the sandwich panels, such as the presence of voids and cavities.

It is important to note that in future studies, the modelling of both C-sections that make up the sandwich panel cross-section should be explored. The current method of simplifying them into a single box beam profile with web stiffeners does not account for the outward buckling of the top C-section profiles observed during the four-point bending test of the full-scale sandwich panel. Additionally, to capture the post-peak behaviour, it is important to investigate failure criteria for the PUR foam core and the face sheet-to-core interface. On the other hand, the constitutive model used to replicate the large deformation of the CFS components aligns with the experimental observations. In the post-peak regime, the FE model exhibits local instabilities and folding of the plate elements, as well as the development of plastic hinges, which match the deformed shape of the panel.

Design recommendations have been developed based on simplified analytical methods used in the preliminary design of a sandwich panel. However, future studies are required to gain more insight and accurately quantify the contribution of the face sheet-to-core interface and the adhesive joints between the CFS profiles. The design procedure assumes the cross-section to be a box beam bending about its weak axis and disregards the mechanical contribution of the PUR foam core. The model has been validated against experimental results and is conservative in terms of deflection at the SLS and ultimate load. Design tables have been created to verify different span lengths and support conditions, and the results suggest a good correlation with the findings of the four-point bending test.

CHAPTER 8

8. CONCLUSIONS

Construction practices such as reusing existing buildings and modular construction help to reduce the environmental impact of the construction industry. Reusing existing buildings prevents the production of waste from demolition, while modular construction involves using prefabricated structural components designed to minimize the use of natural resources. However, these construction practices are weight-sensitive applications. Furthermore, the use of concrete or composite steel-concrete solutions presents different challenges: when rehabilitating existing buildings, degraded timber floors often need to be replaced with solutions of similar mass to avoid overloading the existing vertical load-bearing structures; on the other hand, modular construction requires the use of lightweight structural components that can be easily transported to the construction site.

The lightweight floor system proposed in this thesis falls within the scope of the LightSlab R&D Project, which aims to develop a new structural system based on sandwich panels for applications where weight is a critical design factor. The prefabricated floor system is made up of modular units, specifically PUR foam-filled CFS sandwich panels with web core, as well as technologies for panel-to-panel and panel-to-wall connections. The adjacent panels are connected using cover plates attached to the bottom face sheet and fastened with self-drilling screws. For connection to load-bearing walls, steel angles are used for bolting and/or adhesive joining the sandwich panels.

To develop the proposed sandwich floor panel system, this thesis addressed the following topics:

- i) assessment of available design methods for PUR foam-filled CFS sandwich panel with web core;
- ii) development of design optimisation procedures for the cross-section of sandwich panels based on genetic algorithms;
- iii) characterisation of the sandwich panel mechanical performance and verification of its compliance with structural safety requirements;
- iv) development and characterisation of the panel-

to-panel and panel-to-wall connection systems; v) numerical modelling of the sandwich panel constituent materials and the final prototype; and, vi) development of simplified design guidelines for the developed sandwich floor panel. A summary and concluding remarks of the chapters of this thesis are presented below.

8.1. SYNTHESIS AND RESULTS

In Chapter 3 the requirements and design standards for the sandwich floor panel were identified. The initial solution was designed using CFS face sheet and webs, and a PUR foam core according to the available standards for CFS structural members based on the EWM. As a conservative assumption, the preliminary design did not consider the contribution of the PUR foam. The sandwich panel's thermal and acoustic performance were assessed based on the available analytical tools. An innovative optimization procedure was developed based on genetic algorithms to minimize cost, weight, and environmental footprint. The penalty function strategy was used to include structural, thermal, and acoustic constraints in the search process of the optimal solution. The optimization procedure showed efficiency and versatility, with potential applications in civil engineering structural elements other than sandwich panels.

The final sandwich panel prototype resulted from modifying the cross-section after failed attempts to create one with minimum dimensions tolerances. The change to a layout with a bottom stiffened C-section and a top plain C-section proved beneficial for the panel's rigidity in the assembly phase and during the PUR foam foaming process. Continuous production of these panels is highly feasible, contingent on Ferpainel's ability to introduce a profiling machine capable of bending the sheets as they are unrolled from the coil.

In Chapter 4, an experimental campaign was conducted to test the constituent materials and small-scale sandwich panels. To confirm the mechanical properties of the technical datasheet, CFS coupon tensile tests were carried out. Meanwhile, a comprehensive mechanical characterisation was required for the PUR foam. Flatwise and compressive tests were performed on the PUR foam. These tests were coupled with DIC measurements and thermogravimetric analyses which highlighted the presence of layers of PUR foam with different mechanical properties across the thickness of the sandwich panel. Furthermore, the estimation of the Poisson's ratio for the PUR foam object of this study, as well as for other PUR foams used as the core of sandwich panels, suggests that the sandwich foaming process introduces some level of anisotropy in the foam due to the preferential evolution of the foam in the rise direction.

The results of the flatwise compressive and tensile tests, shear tests and edgewise compressive on small-scale sandwich specimens without longitudinal reinforcements indicate the presence of a top layer of PUR foam with relatively weaker mechanical properties. According to the type of stress state, the sandwich panel generally failed due to cohesive tensile or shear crack of the PUR foam located in such a layer. The crack evolution followed a path parallel to the face sheet. Occasionally, crack kinks were observed, leading to the crack propagating through the top face sheet-to-core interface. Ultimately, failure is caused by the crack evolution, leading to the debonding of the face sheet.

In Chapter 5 the results of the flexural tests on single and joint sandwich panels were used to validate the assumptions of the preliminary analytical design. It was found that the simplification into a box beam cross-section with a top face sheet equal to the weighted average of the thicknesses of the overlapping and non-overlapping parts of the original built-up cross-section was valid. It may be suggested that no significant slip occurs in the adhesive joint in the initial elastic regime. The use of gross properties of the CFS cross-section for the estimation of deflection at the SLS load level provided accurate results. The PUR foam reduced the slenderness of the plates of the CFS cross-section and prevented premature local instability phenomena. However, the stiffening effect decreased rapidly as the load increased beyond the SLS load level. Neglecting the presence of the PUR foam while predicting the sandwich panel's load-carrying capacity was deemed adequate as it led to a conservative estimation which was approximately 70% of the experimental average value.

The results of the flexural tests on full-scale sandwich panels agreed quantitatively with those of the small-scale sandwich panels. However, in the case of the full-scale sandwich panel, damage was concentrated in the loaded sections rather than at the supports. Debonding of the top face sheet was also observed, confirming the findings of the mechanical characterisation campaign. Ultimately, the panel failed due to web crippling and crushing of the PUR foam core at one of the loaded sections. The post-peak behaviour was characterised by the gradual formation of a plastic hinge. Approximately 50% of the peak load was retained at a displacement of about 100 mm, highlighting the ductile behaviour of the sandwich panel.

In Chapter 6 the development and characterisation of the connection systems for sandwich floor panels was presented. This involved panel-to-panel and panel-to-wall connections. The development of connection systems was based on the literature review of current bridge deck solutions, roofing, and cladding systems. The connections considered criteria such as ease of installation in confined spaces and facilitating maintenance works.

For the panel-to-panel connection, a tongue and groove joint system coupled with mechanical fasteners was selected. Three strategies to fasten adjacent panels together were investigated by means of flexural tests in the transverse direction. The solution with a cover plate fastened with self-drilling screws to the bottom face sheets of the adjacent panels (SB) was selected based on its load carrying capacity and ease of execution. The SB connection was used to connect two full-scale panels and investigate the connection's behaviour in the longitudinal direction. The joint sandwich panels presented approximately twice the stiffness and ultimate load of the single sandwich panel.

Regarding the panel-to-wall connection, three solutions based on steel angle profiles bolted and/or adhesively joined (1L, 2L, and 2LA) to the sandwich panels were considered. For all the tested connections, failure was always triggered at the loaded sections of the sandwich panels. However, in the post-peak regime, bearing failure of the holes of the bolts in the sandwich panels was observed for the connections involving bottom and upper steel angles. Overall, the hybrid bolted-adhesive connection outperformed the bolted connections in terms of stiffness and ultimate load. Furthermore, through a comparison with analytical solutions of simply supported and fixed beams at the supports, all the connection systems presented stiffness values included in the interval defined by the analytical prediction. The hybrid mechanical-adhesive connection (2LA) reached 60% of the theoretical value of a beam with fixed supports.

In Chapter 7, numerical simulations were conducted to replicate the mechanical characterization of the constituent materials and the full-scale sandwich panel. Based on the accuracy of the analytical and numerical results in predicting the stiffness and load-carrying capacity of the sandwich panel, the former method was chosen to develop simplified design tools for the sandwich floor panels.

The numerical models of the CFS coupon tensile test matched the experimental results very well. However, the constitutive material model selected for the PUR foam could only satisfactorily capture the compressive response of the core. The simplifying assumption of the CFS box beam cross-section and the perfect bond between the CFS plates and the PUR foam core was found to be adequate only in the elastic regime. The numerical predictions of the axial and bending stiffness of the small- and full-scale sandwich panels, respectively, showed good agreement with the experimental results. However, the ultimate load-carrying capacity was overestimated by about 17%. This suggests the presence of defects in PUR foam core such large voids and cavities. It also highlights that the crack propagation of the PUR foam and possibly its evolution through the face sheet-to-core interface are crucial physical features of the sandwich floor panels.

The development of the simplified design tools was based on the analytical predictions validated in Chapter 5. Design tables were provided for two different support conditions, namely a simply supported panel and a panel with an intermediate support. The manufacturer of the sandwich panels was provided with the design tools to promote their use and assist practitioners in their design.

8.2. FUTURE DEVELOPMENTS

During the mechanical characterisation of PUR foam, some inconsistencies were observed in the estimation of its Poisson's ratio. These discrepancies were also estimated based on the values reported by other researchers in the literature. Additionally, the use of Digital Image Correlation (DIC) recordings revealed that layers of PUR foam parallel to the face sheet had different mechanical properties. These findings suggest that PUR foam may possess anisotropic properties as a result of its manufacturing process. To validate this hypothesis, an experimental campaign is necessary. It would be useful to couple mechanical testing with scanning electron microscopy to establish a possible correlation between the porous microstructure and its macroscopic properties, as other researchers have attempted to do. Furthermore, both the viscoelastic properties of the PUR foam and the PUR-based adhesive should be studied in the context of the long-term behaviour of the sandwich panel and its durability.

Further investigation should be carried out on the inclusion of the core material stiffening effect against the instability of CFS plates in analytical models, with a better understanding of the PUR foam's mechanical behaviour.

Regarding the connection systems, suggestions for enhancements have already been provided in the relevant chapters. These recommendations include using a soft sealant between adjacent panels and using reinforced timber blocks to connect with the vertical structures. It is important to test these improvements experimentally. Additionally, tests should be performed using an eccentric loading configuration to gain a better understanding of the panel-to-panel connection's load transfer capacity.

The comparison between the numerical analyses and the experimental result of the full-scale prototype suggests that further inspection on the quality of the PUR foam core is required. This is consistent with the findings of the mechanical characterisation of the PUR foam and the small-scale sandwich panels. Furthermore, the numerical simulations showed that physical features such as the cohesive tensile crack of the PUR foam, as well as debonding between the core and the face sheet and between the overlapping parts of the CFS cross-section adhesively joined, need to be included to improve the understanding of the

mechanical behaviour of the full-scale sandwich panel. To include such features, different modelling strategies are available, such as using failure criteria based on maximum principal stress or strain for the foam and explicitly modelling the interfaces by means of cohesive elements. Implementation of these strategies would require a new set of parameters that could be obtained either by calibrating the numerical models with the results of the flexural test of the sandwich panel or through experimental tests. In the latter option, tests such as the double cantilever beam and the single edge notch bending test should be carried out to obtain the traction-separation law of the interfaces and the fracture energy of the core material, respectively.

Finally, the proposed sandwich panel floor system's applicability depends on future evaluations of its performance in terms of building physics functions, such as acoustic and thermal behaviour, as well as fire resistance. The low mass of the sandwich panel poses a disadvantage in terms of acoustic insulation. To address this, future efforts should focus on characterizing the airborne and impact sound insulations of the sandwich panels to identify potential measures for sound transmission dampening. It is also essential to quantify the thermal insulation of the sandwich panel due to the presence of thermal bridges represented by the steel webs. Additionally, understanding the thermal properties of the sandwich panel and its constituent materials is crucial for assessing fire resistance performance. Preliminary fire resistance tests have raised concerns about the panel's behaviour under fire exposure. Therefore, it is important to characterize the behaviour of fire protection systems that can be used with the sandwich panel floor system and assess their compliance with building code requirements in the future.

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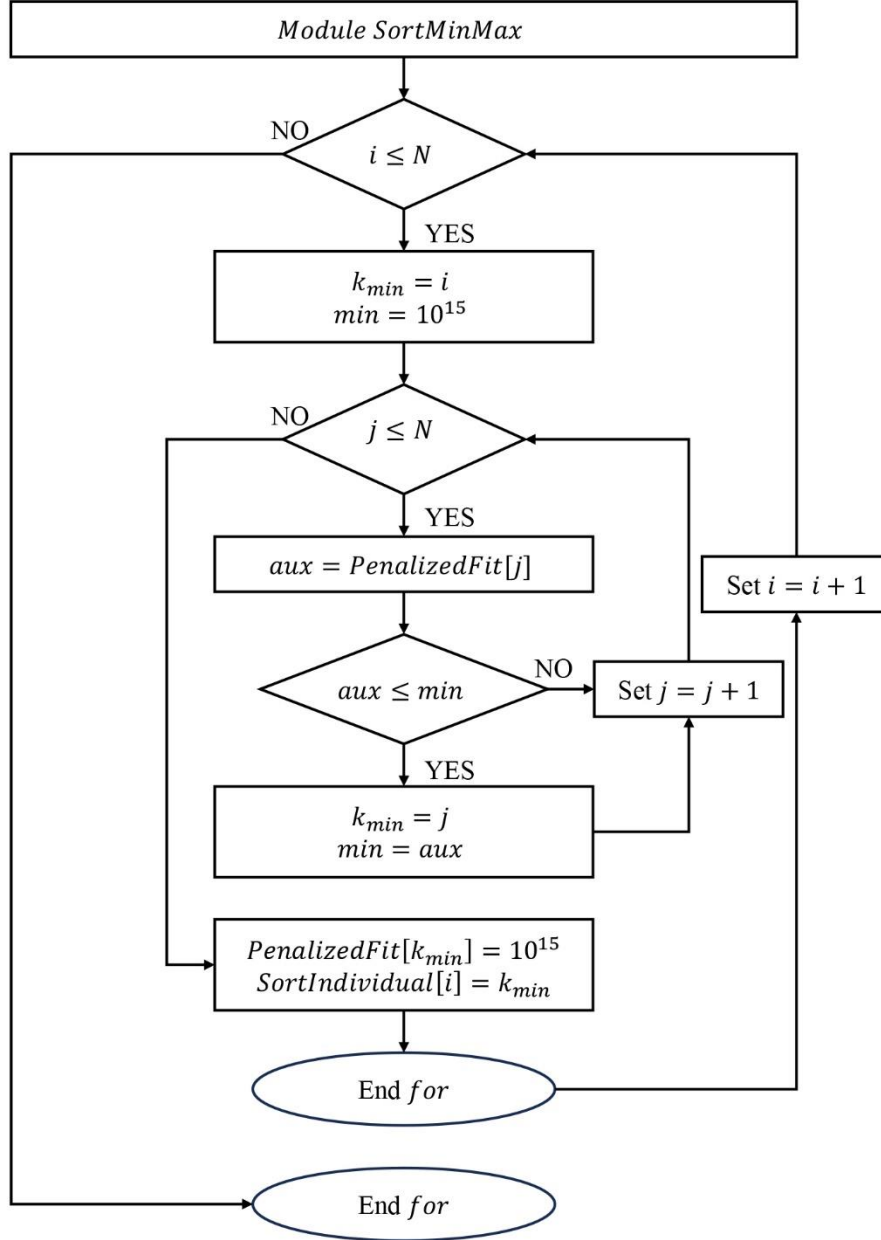
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ANNEX

The OSPANEL code comprises several modules which contribute to the convergence of the solution of the optimisation problem. The $i - th$ individual in the OSPANEL code are represented by a structure which is a database containing variables of different types: i) the set of design variables ($\mathbf{x}[i]$); ii) the cost ($cst[i]$), weight $wt[i]$, and environmental impact ($gwp[i]$); iii) the penalised fitness ($f_p[i]$), the ranking ($k[i]$), and the exponential ranking fitness ($f_{exp}[i]$); iv) the mating probability ($p_{mat}[i]$) and the cumulative mating probability ($p_{cumulmat}[i]$). The structure of each individual is stored in an array ($Individual[i]$) whose size (N) may be chosen by the user. The user, through an input file, may also select the component w_i of the weight vector ($\mathbf{w}$) of the objective function, the maximum number of generations (t_{max}), and the selective pressure of the best individual (s).

The *Module SortMinMax* is used to assign the ranking of each individual so that the individual with the smallest value of the penalised fitness is assigned a rank of 0 and the worst performing individual is assigned a rank of $N - 1$ (see Figure A.1). Firstly, the penalised fitness of each individual ($f_p[i]$) is stored in an auxiliary float array ($PenalizedFit[i]$). The *for* loop with the counter variable j compares the value of the $j - th$ individual with the value of the variable min , which is the minimum value found so far of the penalised fitness. If the value of the current $j - th$ individual is less than min , its location in the array of individuals is stored in k_{min} . At the end of the *for* loop with the counter variable j , the location of the best individual is store in an auxiliary integer array ($SortIndividual[i]$) and its value in $PenalizedFit[i]$ is assigned a fictitious large value. This way, as the instruction are repeated in the *for* loop with the counter variable i , the best performing individuals found in the $i - 1$ loop is automatically discarded from the comparison. At the end of the *for* loop with the counter variable i , the location of the individuals in the array are saved in $SortIndividual[i]$ from the smallest to the largest. The last step consists in transferring this information from the $SortIndividual[i]$ array to the $k[i]$ variable in the structure array $Individual[i]$.

Figure A.1. *Module SortMinMax* of the algorithm OSPANEL.

The *Module CalcMatProbability* is used to determine the mating probability of each individual ($p_{mat}[i]$). In the first *for* loop the new fitness value of the $i - th$ individual ($f_{exp}[i]$) is estimated (see Figure A.2). In the same loop the sum of the exponential ranking fitness of all the individuals is stored in the auxiliary float variable *Sum*. Finally, in the second *for* loop the mating probability ($p_{mat}[i]$) of each individual is obtained as the ratio of the exponential ranking fitness of the single individual between the exponential ranking fitness of the entire population.

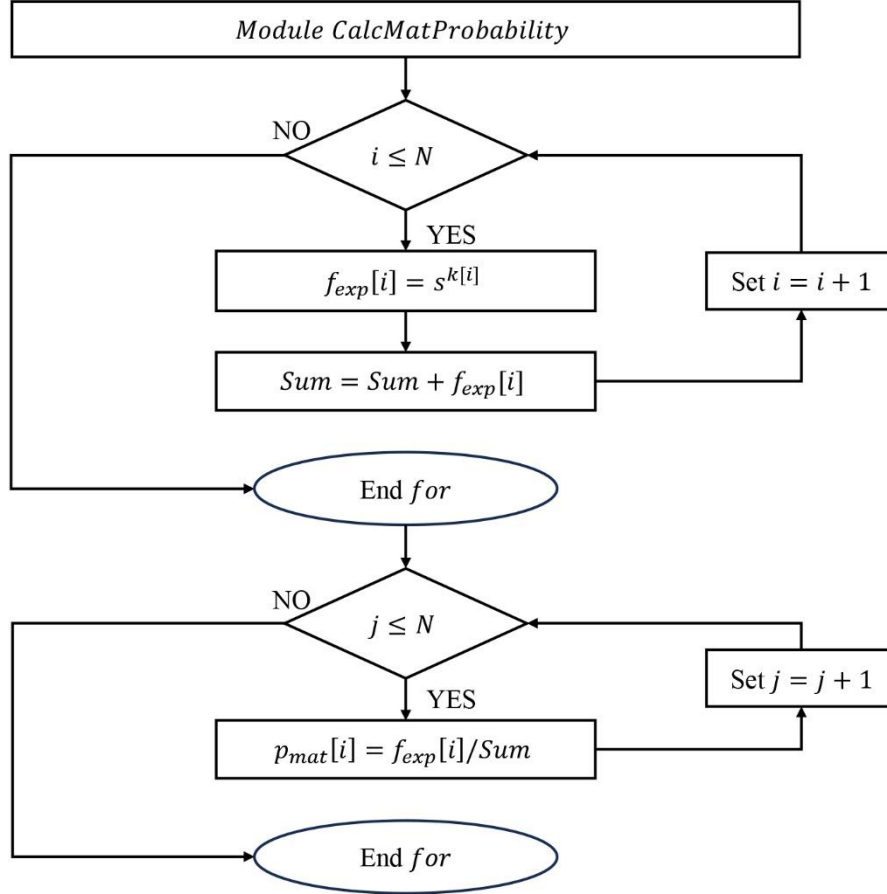
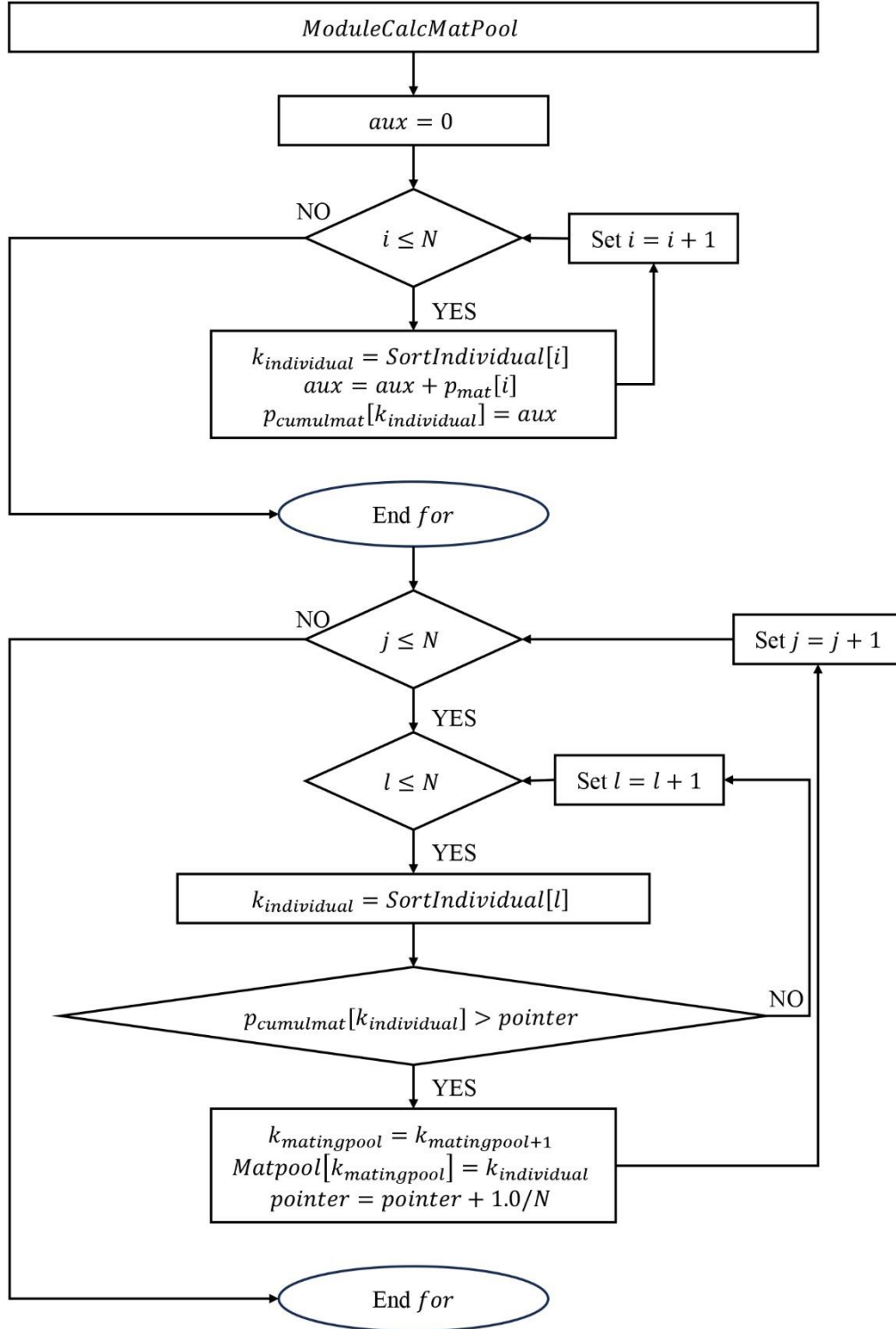


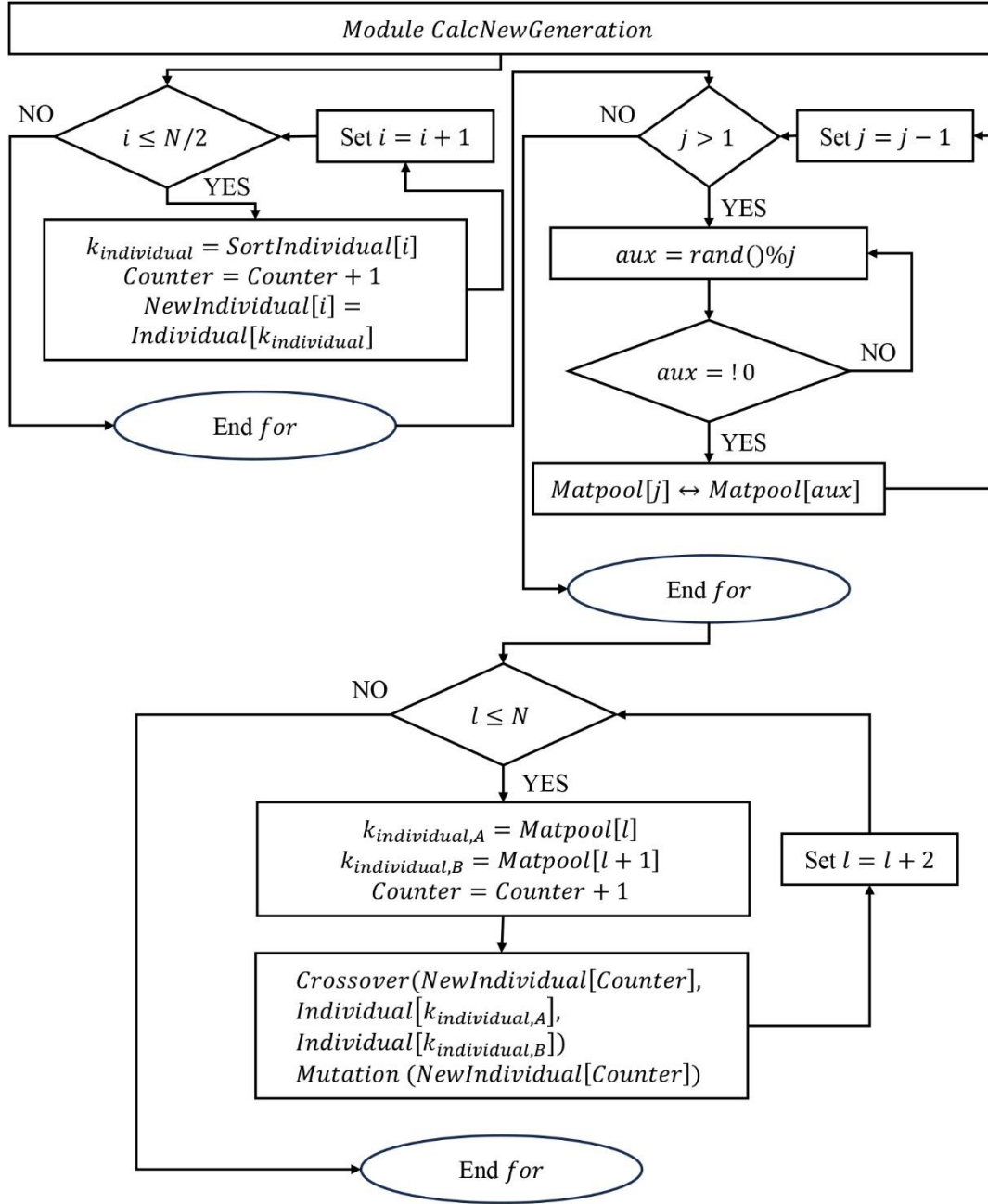
Figure A.2. *Module CalcMatProbability* of the algorithm OSPANEL.

In the *Module CalcMatPool* the auxiliary integer array $MatPool[i]$ is filled with the location of the individuals selected for reproduction in the structure array $Individual[i]$ to form the mating pool M_t (see Figure A.3). The selection is achieved by means of SUS, which can be visualised as the result of a spinning roulette wheel with the slots proportional in width to $p_{mat}[i]$ with multiple, equally spaced pointers. The slots of the roulette wheel correspond to the cumulative probability ($p_{cumulmat}[i]$) of the individuals. The first *for* loop consists in locating the position of the individual ($k_{individual}$) through the auxiliary array $SortIndividual[i]$ and constructing the roulette wheel. The cumulative probability is calculated by means of the auxiliary variable *aux*, which stores the sum of the mating probability of the individuals at the end of each loop. Once the roulette is built, the auxiliary variables $k_{matingpool}$ and *Pointer* are initialised. The cycle with the counter variable *l* is used to locate the position of the pointers within the roulette slots. If the cycle is broken the last stored value of $k_{individual}$ represent where the pointer landed on the roulette wheel. The position of the individual selected by the pointer is stored in the array $MatPool[i]$, the *Pointer* location is updated and the *for* loop with the counter variable *j* is repeated until all the $MatPool[i]$ array is filled.

Figure A.3. *Module CalcMatPool* of the algorithm OSPANEL.

The *Module CalcNewGeneration* is utilised for the creation of the next generation of individuals P_{t+1} (see Figure A.4). According to the elitist strategy the best genetic information of the previous generation is retained in the next one. Therefore, in the first *for* loop, the best half ($N/2$) of the parent generation is copied into the auxiliary array *NewIndividual*[*i*]. The best individuals are located

through the integer array *SortIndividual*[*i*]. Furthermore, the auxiliary variable *Counter* is used to keep track of the position of the individual in the structure array *NewIndividual*[*i*]. In the second nested *for* loop the mating pool M_t is shuffled. The *for* loop with the counter variable *i* starts from the value *N*. The location stored in *MatPool*[*i*] is swapped ($\leftrightarrow$) with the location in *MatPool*[*j*]. The *j* – *th* location is a random value ranging from 1 to *i* – 1. The random location is obtained by using the *rand*() operator which generates a pseudo random number and the modulus (%) operator which returns the remainder of the integer division between *i* and *rand*(). The shuffled *MatPool*[*i*] array is then used to generate the offspring population. Two consecutive individuals ($k_{individual,A}$ and $k_{individual,B}$) are selected from the *MatPool*[*i*] and their genetic information is combined together through the cross-over operator (*Crossover*). The offspring individual inherits a combination of the design variables of the parent individuals. A mutation operator (*Mutation*) with a predefined rate of 0.8 is applied to the set of design variables $\mathbf{x}[i]$ of the offspring individual which is finally copied into the *NewIndividual*[*i*] at the location determined by the *Counter* variable. The last step consists in copying the information from *NewIndividual*[*i*] to the original structure array *Individual*[*i*].

Figure A.4. *Module CalcNewGeneration* of the algorithm OSPANEL.