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Stabilised Compressed Earth Block Masonry: a mechanical behaviour analysis

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Abstract

Earthen construction is a vernacular building technique which adopts locally sourced geo-based materials and can provide an overall comfort of living. Despite earth being generally a weak structural material, it can offer an appealing solution for construction in terms of energy consumption, emissions and costs. Reinforcement can be introduced in earthen structures to improve mechanical response of load bearing elements, especially when subjected to horizontal forces. It is therefore of interest to study its structural behaviour.

In this study, an earthen masonry system developed in Senegal for reinforced earth walls is presented. It is composed of concrete-stabilised compressed earth blocks (CEB) and earth-based mortar. The blocks are moulded with hollows which allow a minimum reinforced concrete frame to be cast in them. However, the use of concrete reinforcement in earth-based materials presents challenges not only in terms of eco-compatibility but also from a mechanical perspective. Concrete is a rigid material, whereas earth exhibits significantly lower mechanical performance. The disparity in stiffness between the two systems renders the extensive use of concrete reinforcement for earth structures incompatible.

Consequently, this research investigates alternative reinforcement methods, such as Fiber Reinforced Polymers (FRP), which are compatible both ecologically and mechanically, assessing their effectiveness. Experimental tests have been carried out to identify mechanical properties of stabilised CEB and mortar (compressive strength, elastic modulus). Three masonry configurations were tested: unreinforced masonry (UM), concrete-reinforced masonry (CRM), and a mixed-reinforced masonry (MRM) system combining BFRP nets with vertical concrete elements. Experimental results demonstrated that the MRM configuration significantly improved structural performance, enhancing load resistance, stress distribution, and post-peak behaviour, while the CRM system showed mechanical incompatibility and premature failure.

Additionally, a homogenised orthotropic FE model was developed and validated through experimental data to study the nonlinear response of the wall. This work provides both experimental and numerical contributions towards optimizing earthen masonry systems reinforced with composite materials, highlighting their potential for sustainable and structurally efficient construction.

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Chapter 1 - Introduction

Thesis Outline

The current research is organized in eight chapters.

Chapter 2 provides essential information for comprehending the research objectives and the corresponding decision-making process. The chapter includes an overview of earth as a construction material, traditional applications, and construction strategies. Since traditional masonry has been taken as a reference for earthen masonry, Chapter 2 discusses its mechanical behaviour, its standard for experimentations and explores the application of reinforcement in this context.

Chapter 3 introduces the methodology approach used to investigate experimentally and numerically the earthen masonry system, focusing on the challenge of the earth-based material.

Chapter 4 extensively studies the Construction System. It consists of a custom stabilised compressed earth block masonry structure with a reinforcement frame cast within for enhanced mechanical properties. Additionally, a modification of this system is introduced to replace materials with high ecological impact (e.g., steel elements) with fibre-reinforced polymer alternatives (FRP). The following chapters (5 and 6) delve into the experimental program.

Chapter 5 presents the experimental campaign executed on earth detailing the procedure adopted for the mechanical characterization. Given the singularity of earthen materials, the campaign is conducted first at a constituent materials level - compressed earth blocks and earth mortar in compression and bending - to define essential mechanical properties. (Different typologies of mortar are tested). Then the construction system is investigated when subjected to a compression load. First the unreinforced panels and the reinforced one. Strength and deformation are analysed in both configurations.

Chapter 6 reports the experimental campaign on Glass Fibre Reinforced Polymers bars. The tests then focused on the effectiveness of the bond between the GFRP bars and a cementitious matrix through pull-out tests. Bars of different diameters and surface treatments are tested to investigate effectiveness of the bond.

In both chapter 4 and 5 chosen specimens, test setup, interpretation and elaboration of results are outlined – DIC configuration was tailored for the specific experiments.

Chapter 7 presents the numerical models and analysis. First different Representative Element Volumes (REV) are identified to describe different mechanical responses depending on different configurations of the masonry panel with the reinforcements. A homogenised model, for both unreinforced and reinforced masonry panels, was then built with boundaries and loading conditions presented in the experiments in order to establish a comparison between the numerical and experimental data. Further, both linear and nonlinear analyses were conducted. Subsequently, a numerical model incorporating GFRP reinforcement elements was developed to assess the mechanical response. The models were initially calibrated for compression and later extended to simulate shear behaviour.

Chapter 8 concludes with observations of the main results of the research itself. It comments on the use of this construction system, based on Compressed Earth Masonry Blocks, to build structures able to withstand loads, leading to construction buildings techniques that present a low ecological impact. Consideration for future research on this construction system are also addressed.

1.1 Research Topic

In response to the growing understanding of the climate crisis, research across various fields has surged to address the demand of sustainable solutions. This awareness has led to an increasing demand for environmentally friendly practices and materials in several sectors, among which is the construction industry. The research is driven by the needs of a more sustainable way of constructing, focusing on the use of materials that present a low impact in terms of energy emission and energy consumption. There is an increasing expectation for these materials to be renewable, biocompatible, and capable of concluding their life cycle through biodegradation or recovery.

Adopting earth as a construction material leads to several advantages in this matter. It has an inherent sustainability due to the abundance of resources, together with a low extraction and processing process. Earth presents good thermal mass properties providing natural insulation, reducing the need for heating and cooling system leading to energy efficient systems. Moreover, the production of earthen structure requires a lower amount of energy when compared to conventional materials such concrete or steel. These structures employ a straightforward construction system and, as earth is often locally sourced, transportation costs are also cut. This promotes a sustainable approach

to construction, considering both environmentally friendly system and a positive impact on a social level. Additionally, earth being one of the oldest construction materials, it could meet and preserve architectural traditions.

The central theme of this research revolves around the development and evaluation of a sustainable construction system for masonry structures, utilizing earth block and earth mortar. Compressed earth block (CEB) technology aligns seamlessly with these requirements. Its straightforward production and implementation, abundant availability of raw materials and low overall cost, make compressed earth a compelling subject of study, also considering its potential social implications (e.g., affordable housing, training and re-training of artisans).

Although there are many positive aspects to using earth in construction, it presents lower mechanical performance compared to conventional building materials. This, together with the heterogeneity of the material itself and the limited presence of standardised regulation for its construction, implicates the requirement for an in-depth understanding and study of the material.

In particular, limited research has explored the mechanical optimization of CEB systems through the integration of reinforcement strategies that are both structurally effective and environmentally compatible. This study contributes to this area by assessing different reinforcement configurations - including concrete and fibre-based systems - and by providing experimental and numerical evidence of their performance.

For this reason, the research initially delves into the analysis of the mechanical properties of the constituent materials (CEB and earth mortar). Subsequently, it shifts its focus toward the construction system.

1.2 Research Hypothesis

Despite being an inherently sustainable construction material, earth exhibits limitations in overall strength, leading to low mechanical performance in earthen structures. Physical and chemical stabilisation of earth material can help to meet construction standards; however, challenges arise due to the diverse compositions and production methods of earth mixes. An exhaustive analysis of the mechanical characteristics is imperative to comprehend its response effectively.

In this research is introduced a construction system capable of resisting loads, based on stabilised compressed earth block with earth mortar. As know, masonry constructions have capability to resist to compressive loads, but a poorer performance is shown when subjected to shear, which is a further disadvantage for earthen masonry. To address this,

a minimal reinforcement frame, comprising concrete and steel elements, is adopted. This choice, although effective in enhancing structural strength, contradicts the eco-friendly nature of earth.

Experimental results, however, revealed that the use of traditional concrete reinforcement may lead to mechanical incompatibility, due to the significant difference in stiffness between concrete and earthen materials. The introduction of composite reinforcement, particularly BFRP nets combined with minimal concrete elements, demonstrated improved load-bearing capacity, ductility, and post-peak behaviour.

To validate these findings, a homogenized finite element model was developed, capable of capturing the nonlinear behaviours of the reinforced masonry panels. The integration of experimental data and numerical modelling forms a core contribution of this research, offering tools for the design and optimization of earthen masonry systems using innovative reinforcement strategies.

This research aims to understand the mechanical response of this construction system, setting the groundwork for substituting high-impact materials like steel and concrete with natural alternatives such as Basalt Fiber Reinforced Polymers (BFRP) and Glass Fiber Reinforced Polymers (FRP). Experimental testing campaigns and numerical models are thus employed for this investigation, addressing the issues presented by composite systems, particularly the integration of reinforcement into masonry. This complexity arises from combining various materials within the structure.

Chapter 2 - Earthen Masonry: An Overview

This chapter explores earthen masonry construction. It presents earth as a construction material, providing a background of the research to fully understand the different choices that have been made throughout the research. Since earth is a locally sourced material that dates back to the earliest forms of construction, various use, and applications of earth in the construction field coexist at the present day, developed in response of different needs. Given that the construction technique applied in this research involves reinforced masonry, this chapter also delves into topics related to the mechanical behaviour of masonry, its strengths and weaknesses, and the various types of reinforcement that can be applied to improve the mechanical response of the system.

2.1 Earth as a Construction Material

Since ancient times, one of humanity primary goals has been to build shelter. This led to the development of the earliest forms of construction using local resources. Earth has been used as a building material in many parts of the world where other materials, such as stone, were not available, taking advantage of its abundance. This artisanal approach allowed earth to be used to meet local needs, leading to the development of a wide range of different construction methods.

The earliest documented forms of construction using earth see this material with the presence of additives, such as natural fibres, or together with a principal structure, usually made out of wood or stone, which were then coupled with earth-based mixtures. This led to the first forms of constructions that have been documented across numerous Neolithic centres of origin, including regions like the Fertile Crescent, Mesoamerica, and China, leading to the formation of the earliest villages. Earth was first used as a primary building material, where local clays were mixed with water and organic additives like straw or other natural fibres. [1], [2], [3], [4], [5], [6], [7], [8].

The earliest construction techniques developed, that adopted earth, include wattle and daub, as well as cob, followed by the sun-dried earth bricks technology. Over time, these approaches continued to evolve, eventually leading to the well-known techniques of rammed earth and, in the 19th century, compressed earth block [9], [10].

Despite the suitability of earth-based materials for building construction, durability issues - particularly those related to weather exposure - presented significant challenges.

Historically, various techniques and supplementary materials were explored to address these limitations, (e.g., fired bricks masonry) [11]. Traditional earthen construction techniques remained in use until the advent and widespread adoption of industrialized materials. The emergence of metals and cement-based products - whose use expanded markedly during the Industrial Revolution - precipitated a decline in the application and further development of conventional earth-based building methods. This shift has led to a progressively reduced use of earth in the construction industry, associating it with an outdated lifestyle, and resulting in a significant loss of construction traditions and knowledge transfer among younger generations.

Nowadays, almost 40% of the population lives in earth buildings [12], [13], but there is still a general reluctance for adopting earth. This is shown in a lack of codes and guidelines, a lack of education of know-how to earth construction and a largely generalised image of earth being costly [13], [14].

Additional challenges encountered in construction practices can be attributed to a diverse array of contributing factors: economically, challenges related to costs hinder the widespread adoption of innovative construction practices. Organisationally, effective project management and coordination among stakeholders remain critical yet difficult to achieve. Sociologically, public perception and cultural attitudes exert a significant influence on construction practices. Technically, issues concerning material properties, design considerations, and construction methods present intricate barriers. Politically, the absence of sufficient regulations and incentives for sustainable practices further complicates the implementation of advanced construction methods. Moreover, earthen materials require careful design, a highly skilled construction team, and robust logistical planning to address challenges associated with their scale and handling on construction sites. [15].

Despite that, people which opt for earthen construction do so for various reasons, such as the absence of alternative materials, the presence of vernacular heritage that they want to preserve, or deliberate choice of earth as a construction material [16]. In the latter case, some of the driving factors for the adoption of earth for construction include minimizing ecological impact, reducing water consumption, and fostering a sense of responsibility for a more sustainable way of living [13].

2.1.1 What is earth?

Earth, as a geo-based material, is characterized by a three-phase composition comprising multiple soil types, water, and air. In nature different typologies of soils can be found but not every soil is suitable to constructions. Depending on the intended use, the

composition of the earth mixture is a key factor to the selection of the material: gravely soils are more draining while clayey soil tend to be used in sealing systems [17]. In most cases, earth materials derive from subsoil layers with additional sourcing from alterites or deposit or softer rock (see Figure 2.1). On the other hand, topsoil is avoided because of susceptibility to shrinkage and degradation. Soil is defined as an aggregation of weathering products derived from parent materials - including mineral deposits, colluvium (gravity-transported material), alluvium (water-deposited material), glacial deposits, and aeolian deposits (wind-transported material) - combined with products of biotic degradation [9], [18]. In the past excavation concerned the subsoil but later, along with the use of the excavators, deeper substrates were reached. Particle size, or grain size, is a fundamental factor to describe and categorise different soil and sediments because it influences the physical and mechanical properties of the soil. Different size scales can be used to define different particles of the soils, Atterberg size scale is used as an international scale. Atterberg fixed a size limit of 2 mm to define sand and the categories above and below based on a logarithmic scale (base 10) [19], [20]. From Atterberg scale years later the International Society of Soil Science defined the “International” size scale. Imagining that soil particles can be simplified to a sphere; the different categories can be described according to their diameter as follows:

- Gravel, particles diameter in a range of 2 to 20 mm.
- Sands, particles diameter in a range of 0,06 to 2 mm.
- Silt, particles diameter in a range of 0,002 to 0,06 mm.
- Clay, particles diameter smaller than 0,002 mm.

These categories can be divided into stricter ranges by diving each of into three subgroup, fine, medium, and coarse.

This classification is predicated not solely upon variations in particle size, but also to different behaviour characteristics. Consequently, these distinctions define distinct material entities. A first distinction can be made in regard of the reaction of the different typologies when exposed to water, clay reacts to it making it an active material, while the others are to be considered inert materials.

As succinctly described by Rigassi [18], gravels are rocks of different nature (size, hardness, etc). There are no changes in the mechanical properties of it in reaction of the water content in the mixture. Sands, mineral particles. As gravels, they are also a stable material of the earth mix, meaning the capability to endure alternating levels of humidity and dryness while maintaining its properties unchanged. Even if they don't exhibit

cohesion when dry, sands are characterised by a high degree of internal friction, leading to a high mechanical resistance to external stresses.

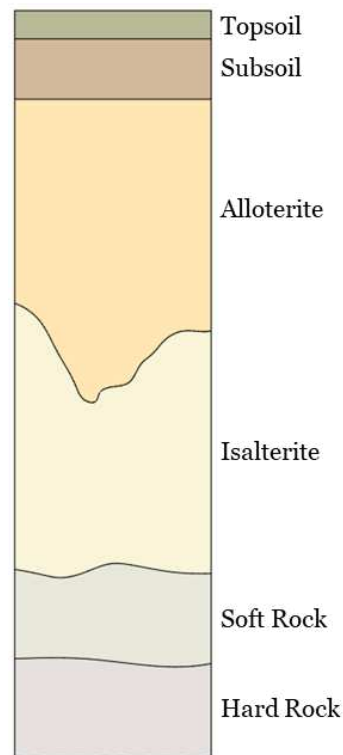


Figure 2.1: Vertical cross-section of soil layers and weathering profile.

When water is added to the mix, they do not change state but a change in the behaviour is to be noticed, which is attributed to the surface tension of the water that infills the voids between the sand particles. On the other hand, silts are characterised by a lower mechanical resistance, but an increase of cohesion is registered when wet. Different humidity levels lead to changes in volume of the silts, because of the swelling and shrinking effect. Due to their minimal reactivity to water, gravel, sand, and silt can be classified as inert materials, using them alone in an earth mixture for construction purposes poses a challenge.

In contrast, clays are typically defined as the soil fraction composed of microscopic clay mineral particles, which exhibit distinct physicochemical properties (such as, kaolinites, illites, etc). These particles are covered in a thin layer of absorbed water, making them exceedingly lightweight in comparison to the surface tension forces within the water film. This film leads to a strong adhesion of the clay particles, that results in a high cohesion of the material and mechanical strength. Because of this clay component can be defined as a binder in the soil mix. High capacity of water absorption of clays leads to volumes changes in relation to the variation of humidity levels. Only advanced desiccation can

eliminate all the water, but general drying phases can result in the formation of cracks, leading to a significant reduction of mechanical performance of the material.

The plastic properties of a soil mix are primarily determined by its clay content, which can be quantified using the Atterberg limits [21]. Based on these properties, earth mixtures can be classified as sandy (up to 70% sand content), silty (up to 50% silt content), or clayey (up to 45% clay content). Consequently, soil mix selection for construction applications should account for these classifications to optimize both workability and performance.

Soil mix selection, in the construction field, is a key factor. Usually soil mixes adopted are a mix of gravel, sand, silt and clay, with 20 to 40% of clay and silt and up to 80% of sand and gravel [22]. The texture of an earth mix, defined by its particle size distribution, significantly influences its mechanical and physical performance. Consequently, not all available earth is suitable for construction; for example, topsoil is typically avoided due to its high water retention capacity, which increases susceptibility to shrinkage and decay.

Soil presents some specific characteristics that should be considered when building with earth is intended, among these: Cohesion, Colour, Compaction, Plasticity and Organic content [23], [24].

Cohesion in earth building materials is due to the strong bond forces that develop between films of adsorbed water bound to clay particles. In certain types of earth construction, clay acts as the primary binder, and its cohesive strength is crucial for maintaining the material's integrity. When chemical binder agents are used in the soil blending, natural soil cohesion helps maintain the form during the initial production stages until the binder hardens.

The colour of soil can indicate its constituents and influences its selection for earth construction based on aesthetic considerations. Soils that are black or dark brown often have high organic matter content, while red, reddish-brown, yellow, and yellowish-brown colours suggest the presence of iron. It is important to note that cement and lime stabilization can lighten the final colour of the soil. For some construction techniques soils with high organic matter should be avoided.

Compaction refers to the densification of soil by packing solid particles closer together and expelling air voids. Increased density directly improves strength and durability, making it essential to compact earth building materials at their optimum moisture content to achieve maximum dry density. Adding particle sizes that are missing in poorly graded soils can improve their dry density. The Proctor test is used to determine the

maximum dry density and the corresponding optimum moisture content under a given standard compaction effort [25]. The use of additives can change the optimum moisture content.

Plasticity refers to a soil's ability to undergo permanent deformation at a constant volume without crushing or cracking, which is primarily due to the presence of clay minerals and organic matter. Depending on its moisture content, soil can exist in a liquid, plastic, or solid state. The moisture content thresholds that define these transitions are known as the liquid limit (LL) and the plastic limit (PL). The liquid limit marks the transition from liquid to plastic, while the plastic limit marks the transition from plastic to solid. The range of moisture content over which soil exhibits plastic behaviour is quantified by its plasticity index (I_p), calculated as $I_p = LL - PL$.

Organic content is referred to as the decaying material found in the upper layer of natural soils, known as the vegetable layer or "topsoil." This layer often contains remnants that can be sizable and interfere with the structure of the earth. Additionally, decomposing material within walls can create significant weak points, making soils with organic matter unsuitable for construction. Typically, construction-grade earth is defined as subsoil, which lies beneath the vegetable layer. While laboratory procedures can determine the organic matter content, but even a simple smell test is sufficient to reject soils with high organic content. Checking for the presence of organic matter is crucial to avoid undesirable instability. Generally, soils with organic matter are categorically discarded without needing to know the exact content or type of organic material.

Earth in today construction

Increasing climate awareness has sparked renewed interest in sustainable practices, particularly in the construction industry. This sector is a major contributor to energy consumption, greenhouse gas emissions [26], [27], [28], [29]. Housing deficit is a current problem that affects almost 1 million people. Moreover, an increase of population in the urban areas is expected in the future years, with a particular focus on a low-income household. The current housing deficit, affecting nearly 1 million people, is compounded by the anticipated population growth in urban areas, particularly among low-income households. However, it is to be noted that earthen building techniques transcend socioeconomic boundaries, proving suitable for high-income countries as well [15], [30].

Accordingly, it is advisable to research and adopt building materials and construction methods that meet sustainability criteria, thereby promoting positive social, environmental, and economic outcomes [31].

In recent years, the use of earth as a construction material has experienced a resurgence of interest, as earthen constructions are increasingly being seen as a reference point for transitioning towards sustainable and eco-friendly building practices. Earth has a long history in building construction, and it is estimated that at least 30% of the world population is living in earthen homes still. This is intrinsically linked to sustainable development, addressing environmental, social, and economic dimensions by facilitating local construction with indigenous materials and technologies, thereby promoting local economic growth and equitable, sustainable housing [14]. By combining traditional earth building techniques with contemporary methods, there is significant potential to create an eco-friendly approach in the construction field. However, a challenge that may arise from the historical use of earth is a lack of understanding of its mechanical properties, which can lead to poor structural performance [13].

Given its widespread local availability, earth offers cost-effective transportation and a low environmental impact due to its low embodied energy. In contrast to fired bricks, which are characterized by high embodied energy, earth-based construction solutions emit up to 80% less carbon dioxide and require significantly lower energy inputs. This natural material is not only reusable, capable of being disaggregated to produce new products, but also supports sustainable practices by reducing the consumption of non-renewable resources and waste production.

Earthen constructions provide numerous benefits, such as good sound insulation, heat insulation, and fire resistance. They also improve indoor air quality, making them an eco-efficient choice for building. The economic feasibility of using earth supports less skilled labour and encourages self-help construction, further promoting sustainability.

While many studies aim to reduce the operational energy of buildings, the choice of construction materials is crucial in determining overall energy consumption. Earth stands out as an environmentally friendly option due to its abundance and the low energy required for its extraction and processing using para-industrial or artisanal technologies. Due to its inherent link to local conditions, this material is typically sourced locally, with construction techniques tailored to specific climatic requirements. Additionally, earth construction has low transportation requirements from raw material extraction to the manufacturing site and low embodied energy due to simpler manufacturing processes, resulting in reduced potential environmental impacts. Therefore, adopting earth as a construction material not only addresses ecological concerns but also enhances economic and social sustainability. [32], [33], [34], [35], [36], [37], [38], [39], [40], [41], [42], [43], [44], [45], [46], [47], [48], [49]. Moreover, when earth is used without adding

any other material in the mixture, it can be reused indefinitely by just adding water to the mix and mould new construction elements [50].

2.1.2 Overview of earthen construction techniques

In construction, earth-based materials typically consist of clay, silt, sand, and gravel in varying proportions, combined with water. When earth is used exclusively or partially in building elements, the clay–water interaction acts as a binder, underscoring the critical role of water in the production process. Different materials and different construction techniques can be achieved employing earth mixtures with different clay-water proportions, hence different hydric state [18]. The water content determines the transition thresholds between soil states - from solid to plastic and from plastic to liquid. This leads to the development of different materials and, consequently, different construction techniques that respond to different needs. In plastic-state processes, the earth mixture is used in a plastic state and gains mechanical strength through drying shrinkage densification. Solid-state processes involve the use of the earth mixture at an optimum water content, with strength provided by compaction densification. Liquid-state processes employ an earth slip that functions as a binder for particles - typically plant particles - added during a subsequent phase. The implementation can occur right after earth mixture production [9]. Among the different construction techniques and construction method that can be used, a classification among them needs to be made on the ability to resist external loads, meaning load bearing and non-bearing elements, but in the case of earthen mixture it is necessary to first classify the technique based on the state of the earth mix (see Figure 2.2). As describe in [9] they can be classified as follow:

Plastic state techniques:

- Mortar: earth-based mixture to lay brick in masonry
- Plaster and render: earth mixture applied in its plastic state. It is used for coating internal and external surfaces
- Wattle and Daub: ancient construction method, involves weaving a framework of wooden branches (wattle), which acts as the structural element, and then covering it with a mixture of mud, clay, or straw (daub).
- Cob: is a mixture of earth in a plastic phase and straw. Builders shape cob into monolithic walls stacking layers of the earth mixture.
- Adobe: it consists of sun-dried bricks made from a blend of clay, sand, and straw. These bricks are stacked and bonded with mud mortar to construct free-standing and load-baring walls.

Solid state techniques:

- Compressed Earth Block (CEB): this technique is usually considered an evolution of the Adobe one. The earth blocks are formed by mechanically compressing moist earth or soil.
- Rammed earth: this technique involves compacting layers of earth or soil within wooden forms to create solid walls. It can be used to build free-standing and load-bearing walls. It is a construction process which has limited possibilities of mechanisation.

Liquid state technique:

- Light Earth: earth at a liquid state. It is generally used as an insulation material.

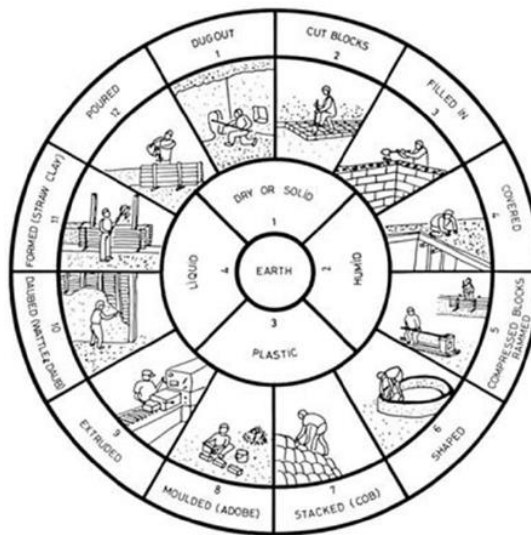


Figure 2.2: Identification of earth building technique based on the material state [51].

Cob

Cob is one of the earth-building techniques that define the vernacular earthen architecture. It is usually adopted to build monolithic walls by piling up clods which are made of an earth mixture in a plastic state (see Figure 2.3), while mechanical strength is achieved through drying process [38]. As stated in [52] there is no apparent discrimination in the use of a specific soil mixture instead of general earth composition that can be sourced locally. Different water content can be used but is found an inverse proportional relation between the water content of the mixture and the compressive strength of the structure, additionally high-water content can lead to cracks development due to shrinkage [53]. Traditionally earth mix for cob construction can be added with fibres to reduce shrinkage and improve the mechanical response of the wall, although

compressive strength might decrease after a certain fibre percentage content. All these variables make it difficult to define precise mechanical properties. The thickness of cob walls can be up to 100 thick at the ground floor [54]. Generally historic earth walls were thicker than walls build with different materials to overcome the low mechanical resistance. Usually, four stages of the cob construction process can be identified [38]: material supply; soils mixing; implementation (addition of natural fibres); rectification and drying. Earth for mixture was locally sourced, but the source itself is not yet clearly identified. Water is a key element in the preparation of a suitable mix for cob construction, which is why areas close to water sources and periods of construction when there was no shortage of water were preferred. Traditionally natural fibres were added to the mix, they were locally sourced and usually added directly to the mixture, small fibres, or after with a length about the width of the wall, long fibres. Meanwhile, other additives were scarcely used.



Figure 2.3: Cob building with extended shrinkage crack pattern [53]

Wattle and Daub

Wattle and daub is one of the one of the oldest construction technique ever adopted. It developed by using tree branches to from a skeleton structure that was later covered in earth in a form of mud plaster (see Figure 2.4). In this way protection from weather conditions was assured [55]. Wattle and daub system mainly evolves around composite walls construction. The wattles are the frames of the walls, usually realised with wooden frames or interwoven wooden elements, whereas the daub is an earth-based mixture is applied as a thick render layer [56]. Clayey soil mixtures were preferred. There are few indications of structures dating back to the Neolithic with roofs build were with the same technique. In this scenario, reeds were preferred to build the supporting structure.

Wattle and daub can still be found in today constructions. As described in [57], first an underneath structure is realised by excavating the ground for a depth of 60 to 100 cm and then infilled with stones and soil to simulate a foundation structure. After a wooden main structure is built, using the foundation layer to fix the vertical elements to the ground, usually wooden battens. Stave realised from tree branches locally sourced are positioned, intertwined around the vertical structure. This defines the skeleton of the walls. The earth is then mixed to obtain a plastic mix (mud like consistency) and pressed against the lattice staves. There is no precise indication on the soil mix preparation, but usually red soil was preferred, with cow dung often added to the earth mix. Cow dung could be also used as a protective layer for the wall, but also lime wash and cement mortar can be found to realise the external plaster. The plaster can improve the durability of the wattle and daub, not only does it provide outer protection, but it also allows for the filling of any cracks caused by excessive shrinkage of the daub [58].

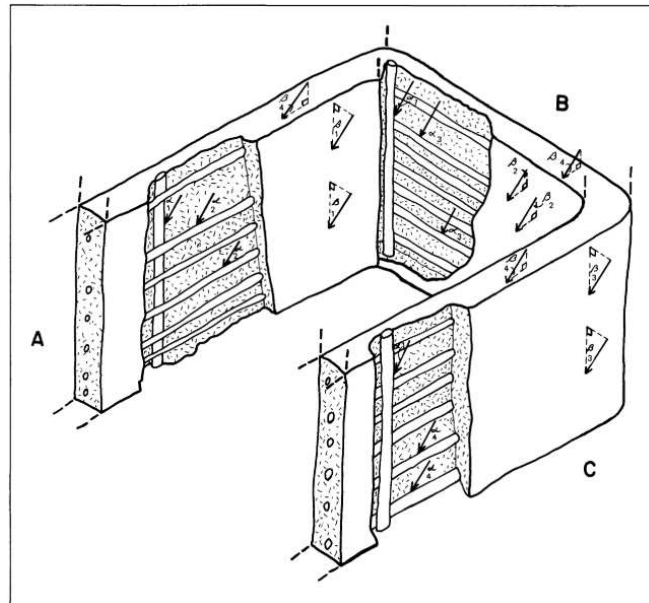


Figure 2.4: Representation of wattle and daub structure from [56] with identification of the wooden structure.

Rammed Earth

Rammed earth (RE) is among the most diffuse technique to build with earth. It is usually adopted to build monolith walls, by a compaction process of several layer of soil mix piled up (see Figure 2.5). Usually, walls are built with a thickness comprise in a rage of 30 to 60 cm. A temporary formwork is used, layers of earth mix roughly 7 to 15 cm are laid inside and the rammed to achieve soil consolidation [59]. The process does not require skilled labour and can be realised on site without major limitations [60]. The earth mix

is poured and rammed until the height reached is approximately half of the height of the uncompressed soil and no other mixtures are added between a layer and the next. In order to assure an optimal realisation of the walls the framework must be tightly fixed, to prevent deformation while pouring and hammering the mix.



Figure 2.5: Rammed earth prism.

During the compaction process, the moisture in the earth mix is mechanically expelled. This increases the walls resistance to loads and enhances their durability. Therefore, compaction is crucial for making earth walls suitable for building construction. During soil compaction, particles are forced closer together, reducing interparticle void space. Consequently, smaller-diameter particles fill the voids between larger particles. RE walls also gain strength during the drying phase, it can take up till two year for a rammed wall to completely cure [13]. The soil adopted is usually a sub soil low in clay with higher percentage of sand and gravel (up to 80% of the mass of the dry mix). Having no binders added to the earth mix (e.g., cement, lime, or natural binders) strength of the structure depends on the internal friction of the soil particle interlock and the water content [61]. Rammed earth structure can reach a compressive strength of 4 MPa c.a. which can be considered an acceptable value for housing construction [13]. Rammed earth (RE) remains in use, and various adaptations of the original technique have emerged. One such modification of the monolithic RE wall entails producing regular blocks that are stacked and bonded using mud slurry. Another variation involves the introduction of mechanical ramming to expedite the construction process and enhance mechanical

properties. Innovation has been made also in the pouring phase by adopting 3D printing technologies to pour regular layer of the earth mix [59].

Adobe

Adobe is one of the oldest building techniques among earthen constructions. It employs the production of earth blocks, or mud-blocks, moulded and left to dry in the sun (see Figure 2.6). Once cured, the blocks are used in masonry work traditionally laid with earth-based mortar. This method is primarily adopted for construction load-bearing walls, suitable for low rise buildings. The production procedure can be summarised in few fundamental steps: selection and preparation of the soil, preparation of the mixture by adding water, moulding of the blocks, and sun drying. The drying process can take from two to four weeks [62]. Moisture control is necessary to avoid swelling on one hand and weak blocks production on the other [63]. The optimum moisture content should be up to 30%, slightly higher compare to other construction methods [23]. Fibres can be added to the earth mix. The addition of fibres to the soil can help to prevent the formation of cracks in the adobe blocks by shrinking during the curing phase [64]. Moreover, an earth mix blended with fibres results in a less dense, lighter block if the dimensions remain the same facilitating easier masonry installation. Being adobe a local technique there was no general guidelines on the proper dimensions of blocks. They were moulded in different size, depending on the intended use of the structure, with common formats being roughly $450 \times 300 \times 100 \text{ mm}^3$. Generally, clayey soil were not used because it can lead to cracks development due to water retention properties of the clayey particles [65]. When the soil is high in clay content is good practice to add sand to the mixture [23]. Studies indicate a soil mixture for adobe preparation generally consists of 40-70% sand, 20-40% silt, and 10-25% clay by weight, essentially requiring a sandy soil [66]. Adobe blocks made from a pure soil mixture can be prone to degradation when exposed to weathering. Stabilization processes can improve the mechanical properties of the blocks, making them more durable. However, construction with unstabilised earth blocks is still possible and commonly used.



Figure 2.6: Adobe masonry wall with coarse aggregates, from [9].

Compressed Earth Block

Compressed earth block (CEB) is a construction technique that can be taken as a progression of the adobe. Similar to adobe, it consists in soil-based blocks that can be arranged mainly to build masonry. CEB derives its strength from a compaction process, either manual or mechanical (see Figure 2.7), allowing for an industrial scale of production [67]. By doing so, the quality and performance of earth masonry can be improved. Compared to other earth-based technique, compressed earth block is relatively new. The compaction process began in the 1700s but became commonly used in construction in the mid-1900s, following the invention of the first mechanical press, the CINVA-RAM, in 1952 [18]. The use of a mechanical press led to a certain amount of regulation in regards of the block dimensions. The CINVA block has dimension of $295 \times 140 \times 90 \text{ mm}^3$, but the common size adopted are $380 \times 250 \times 100 - 125 \text{ mm}^3$. Compaction strength and compressive strength of the blocks are closely correlated in a linear proportion. The minimum compressive strength required for a CEB is $f_c = 2 \text{ MPa}$ [68]. An increased density from higher compaction values results also in an increased thermal conductivity – because of the reduction of the void in the block [69]. Density of the CEBs can vary from 1800 to 2100 kg/m^3 [70]. Quality of CEB is strictly correlated with the selection of the soil for the mixture. A high percentage of clay can lead to lower mechanical performances due to clay physical properties, a good amount of clay ranges from 5 to 25% whereas sand and gravel can go up to 80% [23]. The earth mix required has a water content lower than the one for adobe blocks, obtaining a humid mixture [9]. Different typologies of CEB can be produce with geometric variations, depending on the different masonry work required (i.e., infill masonry, reinforced masonry, decorative masonry). A special type can be moulded to build dry joints masonry, the interlocking block [18]. It is common to enhance the properties of CEB by using additive or stabilisers

in the mixture, cement and lime are largely used in the Stabilised compressed earth block production [71].



Figure 2.7: CEB compaction process, hydraulic press.

Earth-based mortar

Mortar is a material used to bind together construction elements such as bricks. It typically derives its strength from binders like cement or lime. While raw earth is not a binder on its own, it can become a natural binder with the inclusion of clay in the mixture [72]. Earth-based mortars are usually employed to build masonry work (see Figure 2.8), but it is also adopted to repair and protect rammed earth walls [73], [74], [75]. It can be applied using a trowel but also with bare hands [23]. Type and proportion of different clay used in the mix is a key factor in clay-based mortar, most common clay used are illite and kaolin because of their lower leaning towards shrinkage due to their specific structures [76]. The mixture is prepared with a water content close to the plastic limit of the soil used. Variation of water content proved to be an effective and inexpensive method to improve workability of the mortar [77]. Curing process is fundamental, if plastic mortars are high in clay content dimensional variations during the curing phase might occur, leading to shrinkage and cracks [78]. The addition of aggregate can help to reduce its shrinkage effect [79]. In masonry structure mortar strength is a fundamental factor because is strictly correlated to the strength bond in the structure [80]. Earth-based mortar has poorer mechanical performances compared to most common binder used. Various option can be adopted to enhance them, among these the mechanical stabilisation (i.e., optimization of the mechanical skeleton) to increase the dry density of the mix and, by consequence, the mechanical properties [81]. Other procedures to

improve them can be found in blending additives to the mix (i.e., fibres, cement, lime) in small percentage to provide improved mechanical performances.



Figure 2.8: Earth-based mortar in earthen masonry house, Senegal.

Render and Plaster

Render and plaster can be realised with an earth-based mixture (see Figure 2.9). They can be used as protection layer for earth walls – like cob o rammed earth - to assure protection from the weathering. Especially when earthen structures are built in rainy areas the water can lead to damage of the structure, an external plaster can help to prevent the direct contact with water and preserve the structure. Earth based plaster is used to provide consistency and conformity with the materials and their properties, they have hygroscopic properties and the mechanical behaviour is similar to the one of the earth walls, making them a better match if compared to cement based plaster with a much higher stiffness [82], [83]. Mechanical properties compatibilities, especially for the elastic modulus, is a fundamental factor, differences in that can lead to a differential strain capable of compromising the surface bond [84]. Earth render can be of use not only for earth walls but also for different types of masonry due to the ease of application, efficiency and durability, if protect from direct rain [83]. Most of the mixes for earth renders blend tighter two part of sand and one part of clay (in this proportion silts are counted as clay). Higher clay content can lead to crack formation and a poor bond with the wall surface [85]. Sand, on the other hand, being geometrically stable, provides structure to the mix, which is required to produce a good plaster [86]. It is important that the curing phase does not result in cracks due to shrinkage, as this could lead to water penetration, undermining the purpose of the external coat as a protective layer

[82]. Fibres or other additives can be blended in the mixture to provide shrinkage control and higher physical and mechanical performances [84].



Figure 2.9: Earth-based plaster from [9].

Light earth

Light earth is a method that uses earth slip i.e. clayey soil mixed with water until the liquid state is reached, aggregates and fibres can be blended in (see Figure 2.10). The earth slip is bounded from the cohesion generate between the clays particles [87].



Figure 2.10: Light earth specimen from [89].

There are different ways by which the earth slip is mixed with the natural fibres. Three can be identified as in [88] i) spray method, in which the fibres are evenly distributed in a layer and the earth mix is poured over. The process is repeated until 10 to 15 layers are piled up with a height from 50 to 70 cm; ii) dipping method, here both fibres and chipped wood can be used to realise the light earth. The earth mix is poured inside a container so

that after fibres can be added and mixed, the slip should be thick and viscous; iii) a compulsory mixer can be adopted to mechanically blend the light earth. Light earth is usually adopted as an insulation material because of its thermal and hygroscopic properties. It is a lightweight mixture with a density ranging from 200 to 1200 kg/m³. A lower density of the hardened material is associated with reduced thermal conductivity [89].

2.1.3 Stabilization process: altering earth properties for improved performance

Earth as a construction material is a mixture of different soil types, gravel, sand, silt, and clay, in different proportions – depending on the construction technique - and water. This composition makes earthen structures highly ecological, cutting down overall costs of materials, production and energy consumption with a minimal waste production. Even though some mechanical strength is acquired from the soil properties (i.e., friction of the sand particles and cohesion from the clay content) the performances are quite limited, being these mechanical or physical, and they rarely meet the standard for today construction practices. Not less the durability of earthen-based product when exposed to weathering, especially rain. From earthen construction it is required to adequately resist loads, withstand moisture ingress, avoid excessive swelling and ensure durability [71]. Stabilisation is a process that allows for earth to acquire higher physical and mechanical properties by modifying the structure of the earthen-based mixture to address limitations of soil. This can happen by reduction the volume of the voids (it affects porosity), blocking the voids that are responsible for the permeability of the system, and by improving the bond of the soil particles [18]. Three different processes of stabilisation can be identified (see Figure 2.11):

- Mechanical: consists in compressing soil particle to reduce porosity and increase density.
- Physical: consist in the modification of the soil particles by introducing the missing size fraction.
- Chemical: properties are here modified adding a binder or agent to the soil mix.
- This result into an alteration of the structure of the soil and its texture so that mechanical properties (i.e., compressive strength) can be enhanced, reducing porosity – affecting shrinking capacity of the material and by improving the durability of the material to wind and rain. Stabilisation is strictly necessary when the earthen structure is directly exposed to weathering agents. Chemical stabilization is commonly used in rammed earth and compressed earth block

techniques, whereas physical stabilization is more frequently applied in techniques like adobe, cob, and wattle and daub. In these cases, physical stabilization often involves the use of natural fibres [9]

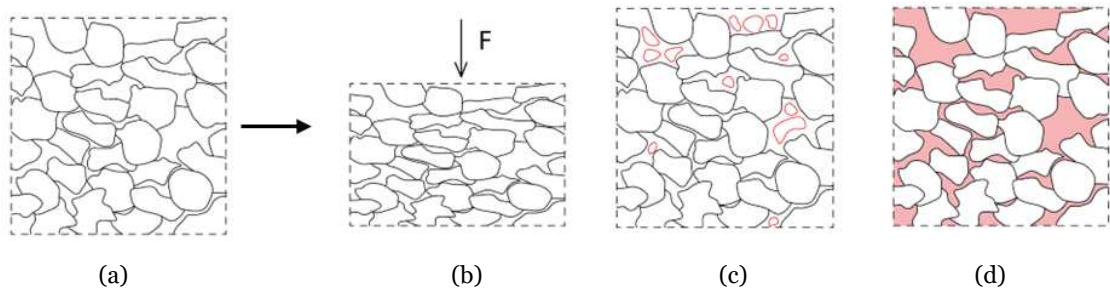


Figure 2.11: Stabilisation process of soil: (a) unstabilised soil, (b) mechanical stabilised soil, (c) physical stabilised soil and (d) chemical stabilised soil.

Mechanical stabilisation

Mechanical stabilization does not alter the soil mix composition but modifies its structure by eliminating interparticle voids through compression, thereby increasing overall density. A moistened earth mix is composed of earth, air voids and water, through compaction - densification process - the voids volume is reduced in proportion to the compaction energy applied [71]. A theoretical maximum dry density is obtained when all air voids are eliminated. Soil composition and clay mineral type can also affect the maximum dry density, with expansive clay minerals like montmorillonite yielding higher moisture content and lower maximum density, compared to less expansive minerals like kaolinite and sand that shows an opposite trend.

Physical stabilisation

Another method to affect soil density is by altering its texture through a physical stabilisation process. In this process, voids are filled by introducing soil particles of various sizes into the soil mix, resulting in a denser material. This is the most inexpensive among the stabilisation methods, and it consists of mixing different soil types together to get a soil mix apt for construction practices. Soils found naturally tend to be or sandy or more clayey, by mixing the missing particles the properties are altered ad hoc. Different purpose can lead to this choice. For example, blending clayey soils with sandy soils improves drainage in earthen bunds, while mixing sand with clayey soils prevents shrinkage cracks in adobe block production and earth-based plasters. For compressed earth blocks made from highly silty soils, adding coarse sand or clayey soil aids in compaction and enhances initial strength. Additionally, adjusting the clay content by mixing with sand makes the soil suitable for cement-stabilised compressed earth blocks

[71]. Fibres can be also added to the soil mix. The use of fibres (organic or inorganic) can be categorised under physical stabilisation, this is because fibres, while they might add some chemical interactions, primarily improve the physical properties of the soil. They help to reinforce the soil matrix, reduce shrinkage and cracking, and improve tensile strength, much like how missing particle sizes fill gaps and enhance the overall structure of the soil [18]. However compressive strength does not directly benefit from the use of fibres [23]. The incorporation of natural fibres is an eco-friendly approach that also reduces the environmental impact of construction practices by utilizing renewable and often waste materials. The use of natural fibres is characteristic of some earthen construction technique like cob, wattle and daub or light earth mixes. Plant fibres (e.g., straw or husks) and plant aggregates (e.g., corn cob particles, olive pit, and cork particles) are used to enhance the physical properties of earth materials in construction. By adding these fibres, the mechanical stability of the earth is improved, making it more durable and resilient to environmental stresses. Historically, fibres and plant derivatives like Nopal mucilage, mauve leaves, and gaucima bark have been employed to strengthen earth buildings in Latin America and the Mediterranean. Attention has to be paid when using fibres because an excess of it may create discontinuities in the earth matrix, which results in voids and leads to weakness in the structural element.

Chemical stabilisation

Chemical stabilisation process employs external agents - commonly binders - to enhance interparticle bonding, thereby significantly improving material properties and fundamentally altering the soil characteristics. Stronger bonds between particles lead to higher compressive strength, enhancing durability. Enhanced interparticle bonding results in a significantly higher compressive strength, improving the mechanical resistance of the material when subject to external loads. Additionally, the improved waterproof properties ensure that the material is impermeable to water, blocking the pores inside the earth matrix it prevents infiltrations. Moreover, the process enhances water dispersal characteristics, allowing the material to efficiently shed or spread-out water. The material exhibits a marked reduction in both water absorption and adsorption, minimizing the amount of water that can penetrate or cling to the surface, thus maintaining the material's structural and functional properties over time. Binding agents of different nature can be blend in the soil, among the most common there are cement and lime. However, other materials, depending on local traditions or necessities, such as cow dung, fly ash, rice husk, bitumen, and resins, can also be used [23]. Chemical stabilization also involves using natural polymers and other organic compounds to modify the bond structure within the soil. This includes the use of animal waste, such as

cow dung, which contains organic compounds that chemically interact with soil particles to improve binding, reduce shrinkage, and enhance durability. Natural polymers like starch, cactus mucilage, agave, algae, cereal flour, and natural resins contribute to the soil's strength and water resistance. Historically, civilizations have used bitumen mixed with vegetable oil to increase water resistance in humid areas, and recent studies confirm the effectiveness of mucilage and bitumen in enhancing adobe durability by limiting shrinkage and reducing rain erosion. Polysaccharides, biopolymers made of long-chain sugar molecules, and proteins like caseins and tannins further enhance stabilization by binding clay particles and forming durable gels. Biomineralization techniques using microbes or enzymes are also being explored to reduce water absorption and improve the strength of earth materials, showcasing the diverse chemical strategies available for earth stabilization.

Portland cement is widely employed to stabilise compressed block and rammed earth constructions by binding sand and gravel; however, its efficacy is reduced when used for clayey soils stabilisation. For this reason, cement stabilisation works best in sandy soils with less than 15-20% clay. Additionally, less than 2% organic matter should be found in the mix so that cement hydration is not altered. Typical cement content ranges from 2.5% to 15%, with 4% to 10% being most common. Adequate amount of water is needed to assure complete hydration of the cement

Cement stabilisation works best on CEBs, the cement is added before compaction enhancing durability and mechanical properties of the block, especially when saturated [71]. Since cement reacts with water, it should be thoroughly blend in the dry mixture and water should be poured as close as possible to the compaction phase. When soil, cement, and water are combined, the cement hydrates, leading to two main outcomes, the formation of cementitious gel such as calcium silicate hydrate and calcium aluminate hydrate (CSH), and the release of calcium hydroxide (CASH). These outcomes result in several effects on the soil: aggregation of soil particles or grains, and the cementitious gel forms water-insoluble bonds, binding the gravel, sand, and silt particles together. A pozzolanic reaction occurs between the released calcium hydroxide and the clay minerals, resulting in the formation of additional quantities of CSH/CASH gel [71]. When water is added to the cement grains, the reaction begins from the surface of the grains, then progresses inwards. This results in the formation of gels and ettringite. The C₃S (tricalcium silicate) and C₂S (dicalcium silicate) form CSH gels, while the C₃A (tricalcium aluminate) forms ettringite. Due to the increased contact between the formed gels on adjoining grains, and due to the interlocking of the ettringite crystals, cohesion develops signifying the start of the setting period. As each grain sprouts a multitude of

these fibres and as they continue to grow and multiply, they start to interlock even more closely and rigidly than before. The paste, however, remains initially weak and therefore of low compressive strength. The hardening process, involving further hydration and pozzolanic reactions, continues to develop strength and is effectively 'completed' about 28 days after the initial contact with water, resulting in a significant increase in compressive strength [90]. Curing should be done in a humid environment for the first 7 days to prevent rapid drying, shrinkage, and cracking. A preservation of moisture helps to complete the hydration of cement content and fully develop strength. After curing, a 14-day drying phase is needed to release water retained in the clay. During this time, avoid direct sun and wind exposure to prevent quick shrinkage [18]. Strength of cement-stabilised earth elements is strictly correlated to the amount of cement used, density and curing phase.

Lime stabilisation is used for higher clay content soils, often in combination with cement when cement alone is insufficient, but it is not unusual for lime to be used on its own. Lime reacts with clay minerals to improve stability and strength, although this process is slower and less pronounced than with cement, making lime stabilisation suitable for more clayey soils. Lime content ranges from 3% to 12%, increasing with clay content. Both hydraulic and non-hydraulic lime can be used, with non-hydraulic lime being especially effective for clayey soils. Lime stabilisation can tolerate high organic matter content, up to 20%. Limestone is the main source of lime. The limestone is burnt in order to obtain calcium oxide (CaO) or quick lime. After that the quick lime is added with water – slacking process- to obtain $\text{Ca}(\text{OH})_2$ or hydrate lime, an unstable chemical product, which as to be stored in airtight bag to avoid an early carbonation process. Mixing soil with calcium hydroxide results in four key reactions: cation exchange, flocculation, agglomeration, and pozzolanic reaction. The first three reactions alter plasticity, shrinkage, and workability, while carbonation slightly contributes to strength gain over time [71]. The pozzolanic reaction is the primary contributor to strength development in lime-soil mixtures. Lime reacts with clay minerals, forming a water-insoluble gel of silicates and silicate-aluminates, which crystallizes into CSH/CASH gel over time. This cementitious gel coats the soil particles, creating strong bonds, like those in hydrated cement. Moreover, swelling and shrinkage effect are reduced due to the lime-clay links [18]. The formation of CSH/CASH products occurs gradually and can take several years to fully develop, but the curing process can be considered complete after six months. High temperatures can accelerate the process.

2.1.4 *Building earthen masonry*

Earth blocks and rammed earth are the most commonly used techniques among earthen constructions, with blocks generally offering greater cost efficiency compared to other techniques [91]. Earth blocks, including adobe blocks or compressed earth blocks, can be used to construct masonry structures similar to more conventional materials such as stones, fired blocks, and cement blocks. Historically, adobe was extensively utilized in brick construction; however, in recent decades, compressed earth blocks have demonstrated sustained viability [47]. This is due to improved mechanical and physical properties of the blocks, allowing for an expansion of the area of application from simple walls to more structurally complex constructions such as domes, vaults and shells [48], [92] and a reduced energy consumption if compared to more commonly used materials like extruded blocks or fired bricks [93]. In these constructions, mortar is necessary for binding the blocks together, preventing moisture ingress, and accommodating dimensional imperfections of the blocks. Additionally, the mortar enhances the flexural and shear strength of the masonry, contributing to the overall stability and durability of the structure. In earthen masonry is common to find unconventional mortar mixtures instead of the more commonly used cement-based and lime-based mixture. Earth-based mortars are common, but composite mixtures combining soil with cement, lime, or both, can be also used [94], [95], [96]. Cement-soil or cement-lime mortar are preferred to purely cementitious mortar mixes due to higher bond strength, however reduced compared to unit-mortar bond in more traditional masonry system [80], [97], with cement-soil mortar being especially common in compressed earth block (CEB) masonry [98], [99], [100], [101], [102], [103], [104], [105]. In earth-based mortar clay acts as a binder. In this case mechanical strength at a fresh state is relatively low, leading to a wall construction rate of 1 m in height per day [78]. Clay content in mortar can be challenging because it can lead to a disrupt bonding on the interfaces [80]. In CEB masonry work, using a composite mortar (soil-cement) allows for a better mechanical response of the masonry in respect of CEB-cementitious mortar masonry [71]. For cement-stabilised mortars, it is suggested to use the same proportion of stabiliser as for the blocks. Different composition can also be found, where lime is also blended into the mix to get a weak mortar [106]. When cement-stabilised earth-based mortar is used in cement-stabilised earth block masonry, better performances can be reached compared to the one with cementitious mortar. If low percentage of cement are used in clayey soil mix the overall strength of the masonry is reduced but a higher deformation capacity is achieved. However, no significant differences in the masonry compressive strength capacity are found when different types of mortar are used, on the other hand is the block strength to largely influence it [107]. On the other hand, higher percentage of cement can be used

in mortar when high shrinkage effect is expected, like in soil mixture with clay content higher than 20% [80].

Stabilised earth blocks are appealing for masonry construction due to their higher mechanical strength and enhanced physical properties compared to adobe blocks. The production of these blocks, using mechanical presses, marks a significant improvement over traditional methods, ensuring consistent quality, also leading to an enhanced social acceptance of building with earth. The process involves soil preparation, often mixing with stabilizers, and compression to form dense, uniform units. The quality of a CEB is closely dependent on the selection of the soil and its preparation. As explained by Rigassi [18] cement or lime stabilised compressed blocks can be organized in key phases: extraction, preparation, mixing, compression, curing, and stocking (see Figure 2.12).

- Extraction: Soil is sourced from the quarry or pit. The process can be either manual or mechanical. Organisation of the tools depends on the typology of the quarry.
- Preparation: soil preparation is a key factor for an optimal block production
 - Drying: The soil is dried by either spreading it in thin layers or using a hot-air cyclone.
 - Pulverising: The dried soil is pulverized to break lumps or unsuitable grain size particles can lead to the formation of weak points. Clay lumps are crushed, and gravel is grinded. The process can be either manually done or mechanised.
 - Screening: Undesirable elements are removed through screening after initial preparation, like organic matter or gravel. Meshes are used to filter coarser soil particles, usually done through a manual process, but mechanical screening can be found.
- Mixing:
 - Measuring: The dry soil is measured by weight or volume to prepare for mixing with water and, if necessary, a stabiliser. The quantities are calculated based on dry weights, accuracy is crucial, ensuring efficiency and reliability. Water, on the other hand, is not easy to calculate in advance, it is strictly correlated to each soil properties. Tests, as the drop test, must be performed, variations on the optimum water content can lead to significant alteration of the physical properties and/or workability.
 - Dry Mixing: Soil mixing is necessary to get a homogeneous distribution of the particles. If stabiliser is used, the dry soil is mixed thoroughly to ensure the stabiliser in powder form is effective. The more uniform the

mixture, the less stabilization is needed, allowing for reduced costs without compromising quality

- Wet Mixing: Mixing should start in the dry state, then water must be added gradually to assure good mixing of the components. Water can be added by spraying after dry mixing, or a liquid stabiliser can be directly incorporated. Lumps must be avoided for a high quality CEBs production.
- Reaction: The mixture is allowed to react for varying times, depending on whether a stabilizer is used; cement requires a short time, while lime requires longer.
- Compression:
 - Measuring Mixed Material: The prepared mixture is measured for optimal block density. Moulds are usually designed to be filled to the top, the soil can be levelled manually or with mechanical procedure. Any gravel or lump still present must be removed before compaction.
 - Compression: The measured material is compressed into blocks with careful attention to the quantity of soil introduced into the mould. In manual processes, a lever is used for compaction, requiring precise force to prevent equipment damage and to achieve adequate block strength. Conversely, automated processes apply a uniform force, ensuring consistent block quality.
 - Demoulding: The compressed block is removed from the mould. The same piston used to compact the block can be used, or a specific piston mounted to eject the CEB.
- Curing:
 - Wet Curing: The blocks undergo wet curing, with the duration depending on the climate and stabilizer used. Block should be sheltered from direct sunlight and high temperatures to avoid cracks formation, whereas in case of stabilised block the presence of water is crucial also to let fully develop the stabiliser-soil.
 - Drying: The blocks are dried to achieve the desired quality.
- Stocking: The finished blocks are stored and ready for use.

The compaction process is fundamental to produce CEB, chemically stabilised or not. Undergoing mechanical stabilisation, the blocks acquire higher density, and consequently higher physical and mechanical properties. The compaction process can be carried out manually or motor-driven, depending on the typology of press used. Although the most common type of block is the solid one, similar to traditional fired brick units,

the use of specific moulds allows for extensive flexibility in producing blocks with precise geometries that can vary depending on the intended use during construction.

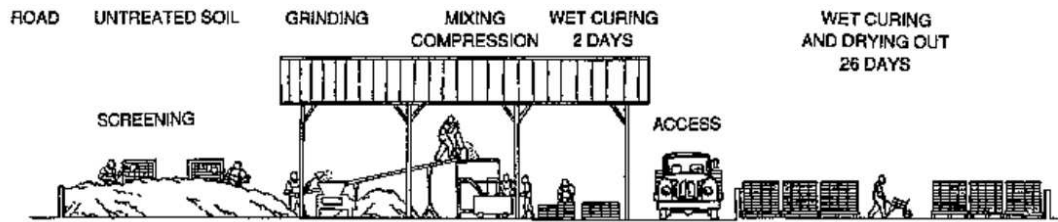


Figure 2.12: CEB manufacturing process from [18].

Three major categories can be identified, considering the shape and construction method for which they are being used. Solid blocks, even if with specific shapes, are used to build masonry work by stacking the unit with mortar. Hollow blocks, usually moulded with void, they required advanced techniques. The hollow part is required for reducing the weight of the block itself, improving mortar-unit bond or to use a permanent formwork or beams construction. A perforated block, similar to a hollow block, has voids inside to allow for reinforcement to be built inside, which is common in seismic areas. Interlocking blocks are specifically designed to build masonry with dry joints, since the shape of the block serves as a mechanical interlock. They are commonly found in non-bearing structures.

Mechanical properties of a masonry are correlated with the mechanical properties of the unit, mortar and the unit-mortar bond.

Cement-stabilised CEB masonry: mortar-unit bond

Compared to traditional masonry structures like burnt brick masonry, stabilised compressed earth block masonry has unique properties that need to be considered. The soil composition and type of stabiliser used have a substantial impact on physical and mechanical properties of the blocks. The moisture content significantly influences the mechanical properties of the blocks, such as strength and elastic modulus. Additionally, these blocks often exhibit higher deformation capacity, particularly when dry. For these reasons, the characteristic of stabilised compressed earth block (SCEB) must be undergoing thorough examination to understand the mechanical behaviour, especially when earth-based mortars are used. The bond between unit and mortar relays not only the texture and microstructure of the surface and mortar composition, but also on moisture content and water absorption properties of the SCEB [71]. Soil composition also affects the bond strength of the unit-mortar interface. It is observed that the higher bond strength is reached when the clay content in the mixture is within the range of 14% 16%

[97]. The bond between masonry units and mortar is due to mechanical interlock, chemical bonding, or both. During construction, blocks are placed on fresh mortar, which usually consists of a binder (cement or lime), fine aggregates, water, and possibly additives. The binder hydration results in the formation of cementitious material. Dry or partially saturated bricks absorb water from the mortar, depositing cementitious products in their pores. These products grow into the brick pores at the interface. As the mortar sets and the hydration products harden, a strong bond is created through mechanical interlocking [108], [109], [110]. Chemical bonding occurs when pozzolanic materials on the block surface react with calcium hydroxide released during mortar hydration. However, this reaction may not occur if masonry units lack surface pozzolanic materials [71].

Additionally, the development of the mortar-unit bond is influenced by the surface characteristics of the unit. A rougher surface tends to allow for a stronger bond than the one with a finer texture [101]. On cement-stabilised CEB, the physical appearance of the surface is influenced also by the amount of stabiliser used. Studies show that increasing the cement content enhances unit strength, increase the density and leads to a definition of smoother unit surfaces, which reduces the mechanical interlock bond strength [71]. Usually the porosity of a cement-stabilised block ranges from 10% to 20% [71]. Among clay content, density, porosity, another factor that highly influences the water absorption rate is the quantity of cement used to stabilise the blocks influence. Higher percentage of cement result in lower absorption capacity of the SCEB [101], [106], [111], affecting the strength development of the unit-mortar bond. Water transport from fresh mortar to the block helps create the bond, but if too much water is rapidly absorbed by the block, the cement in the mortar may not set properly, resulting in a weaker bond. Pre-saturation of blocks before construction helps prevent this issue, facilitating strong bond development. However, excessive block saturation can inhibit water migration, thereby hindering the formation of an effective interlocking bond The strength of the bond is closely tied to the moisture content of the blocks; partially saturated blocks appear to be ideal for achieving strong masonry structures [95], [107], [112].

Cement-stabilised CEB masonry: the unit

A fundamental aspect influencing the behaviour of masonry structures is the choice of building units. In earthen masonry, various materials and construction techniques are employed, including well-known types such as adobe, compressed earth blocks (CEB), stabilised compressed earth blocks (SCEB), and extruded blocks. Non-conventional masonry unit, as for the stabilised compressed earth block, requires a comprehensive understanding of its properties. While CEBs have undergone mechanical compaction

process, SCEBs require a further stabilization process, to enhance strength and durability. Main stabilizations derive from adding to the soil mix fibres of different natures (both organic and non-organic) and chemical binders. Physical stabilization often involves choosing fibres that enhance the block's tensile strength and resistance to cracking. Organic fibres, such as straw or coconut coir, provide natural reinforcement, while synthetic fibres offer specific performance enhancements under varying environmental condition. Chemical stabilization methods primarily involve the addition of binders like cement or lime, which chemically react with the soil particles to improve cohesion and strength. Organic products can also be chosen for their environmental benefits or specific performance characteristics. Careful consideration must be given to the percentage of stabilizer added: an excess of fibres can create voids in the earth matrix, weakening the block's overall structure. Similarly, adding more than 10% of chemical stabilizers to the soil mix can alter the block's properties, affecting its compressive strength and durability. On the other hand, a stabilisation process with less than 5 % of cement content can lead to the production of overly friable products. It is not advisable to use the stabiliser in this way because results in losing the inherent properties of the soil, as economical sustainability, ease of production, carbon efficiency etc., while also failing to gain the mechanical and physical benefits of cement stabilization as well [42]. In addition, cement content higher than 10% makes CEB production not sustainable from economical point of view [113].

Compressive strength is one of the most common mechanical properties investigated to define a block. Given the wide variability in the types of soil mixtures that can be used, the different types of stabilizers that can be employed, the various percentages of these stabilizers, and the different reactions that stabilizers may have with different soil types, the data regarding compressive strength and elastic modulus vary according to the different studies conducted by the authors. Additionally, CEB products are lacking an international standard or code to give uniform guidelines on testing procedure, resulting in non-uniform testing procedure across different studies leading to further variation in these data.

According to the RILEM state-of-the-art report on testing and characterisation of earthen materials [9], any form of earthen construction should be regulated with respect to: earth/soil composition; moulds and machinery; production and manufacturing processes; testing and quality control; design guidance; construction procedures; earthquake-resistant design; and durability and maintenance. Although several existing codes are referenced (e.g., Brazilian, Indian, Spanish, and Sri Lankan codes), not all the aspects mentioned above are comprehensively addressed. Reddy and Gupta [101] noted

that while efforts have been made to draft codes for the manufacture and application of cement-stabilised CEBs, a comprehensive regulatory framework is still lacking. Furthermore, as highlighted by Silva [114], the inherent complexity of soil introduces uncertainties, suggesting that all standards related to earthen materials should be interpreted as guidelines for common building practices rather than strict regulations.

This issue has been also addressed in [45], where different testing procedures to evaluate compressive strength of CEB are evaluated, also considering geometry of the blocks. Direct compressive strength tests are typically performed on 5 to 10 masonry units without corrections for platen confinement, with the average strength determined via statistical analysis. Although standard block sizes exist, they vary considerably due to customizable, non-industrial production methods. Compression tests on cubic specimens are feasible if the cubes are directly cut from the blocks rather than moulded; however, such tests may require geometric correction factors to yield representative results. Moreover, while cubic samples can mitigate some issues, the cutting process may induce micro-damage that lowers the measured compressive strength. [115]. Another approach for direct testing is the RILEM test. To double the slenderness ratio of test specimens, blocks are halved and stacked with an earth mortar bed joint, mimicking masonry construction and ensuring uniform stress transfer. Capping systems are used to minimize friction between the platens and blocks. Half blocks may be prepared following a splitting strength test, aiming to replicate the compressive strength of a cylinder with a 1.5:1 aspect ratio. The RILEM procedure for compressive strength has been validated by research laboratories in France and North Africa. Results show compressive strengths for manually pressed blocks typically ranging from 2–3 MPa, with higher strengths achievable using hydraulic presses or increased cement content. However, this testing setup primarily reflects the performance of the combined unit-mortar system rather than the individual masonry block. Indirect tests may be also performed, like a three-point bending test in order to correlate the compressive strength with the flexural strength obtained.

In addition, compressive strength of earth products can be influenced by physical-chemical factors. Being earth sensitive to water absorption different performances and range of strength values are recorded when the material is in wet or dry conditions. This fluctuation of values depends on different material properties as density, soil composition, clay content and stabiliser used. Walker showed in [106] how an increase of clay content is strictly correlated to a decrease of compressive strength, independently of the percentage of cement used to stabilise the earth. Hence, clay content is a sensitive factor especially for cement-stabilised blocks. Venkatarama Reddy in [116] showed how

the optimum clay content for cement-stabilised CEB is between a range of 10% to 15%, depending on soil particles size. Earth blocks with higher clay percentage exhibit lower strength capacity. In addition higher clay content, as well as lower than optimum range, lead to the production of less durable blocks. Murmu in [113] also reported that optimum clay content falls within the same range. Clay content can be found up to 30% for block production but higher percentage of cement could alter the performances of cement stabilisation due to the presence of unstabilised cohesive soil, clay, limiting soil-cement bonding.

In most cases, both “dry” compressive strength and “saturated” compressive strength are measured to understand the behaviour in different conditions and predict long-term performance. Heathcote in [117] states that there is no advantage on testing the earth blocks in wet conditions. Earth materials inherently vary in their properties, and moisture content adds to this variability. Testing wet blocks (as per the Australian Code) does not offer any advantage. It is suggested to test the entire block to determine compressive strength, because cutting specimen from larger blocks can lead to specimen with different moisture content, therefore fluctuation in the mechanical performances. However, allowing the cut specimen to dry out prior testing can help to ensure consistency in the results. Capping of specimen is also discussed, the author states that capping does not alter the strength results in any appreciable way, hence the no capping was used while testing. Many standard do not mentioned capping as necessary, but [115] states that is necessary when the surfaces are not parallel and regular, plywood sheets is commonly used for this purpose.

Density is another factor that influences the strength of compressed earth blocks. Since the blocks undergo a compaction process, their level of compactness, and consequently their density, affects their mechanical response to stresses. Because the production process is not standardised, different pressures can be applied during mechanical stabilisation using various types of presses and moulds. This results in a range of densities within which the characteristic average values of the blocks are expected to be obtained. It is found that as dry density increases compressive strength increases as well. When cement is used a stabiliser, this can be related to the reduction of voids in the soil mixture, with less porosity, the particles in the compacted specimen are in closer contact with each other. This closer contact results in better bonding, as the cement hydration products have more surface area to adhere to, leading to a stronger overall structure [104].

Stabiliser also plays a key role on the mechanical response of earth blocks. Different stabiliser can be used in respect of the clay minerals in the soil. Among chemical

stabilisers cement is more suitable for sandy soil with a coarse texture and less-expansive clays, enhancing block strength as cement content increases, typically between 5% and 10%. The strength gain varies with the clay content and block density. On the other hand, lime, especially calcium hydroxide, is crucial for soils with expansive clays to manage swelling and shrinkage, with an optimum lime content specific to each soil type for maximum strength. Lime is also effective for non-expansive clays, improving block durability and performance. Wet compressive strength of cement stabilised specimen and lime stabilised specimen has been compared in [104]. The increase of the quantity of stabilisers used was reflected in an increase of the wet compressive strength. However, a higher compressive strength was measured with blocks stabilised with 7% of cement, while lime-stabilised blocks with 8–10% lime exhibit a lower wet compressive strength of 1.5–2 MPa.

The use of type of cement or lime as stabilisers for has been discuss in several studies. In [118] a suitable stabiliser for rural housing is investigated. Plasticity index (PI) of soil is one factor to determine the choice between cement and lime. Cement is most suitable for soil with an index lower than 15, while lime for PI ranging from 15 to 30. CEB compacted at 5 MPa, with different percentage of lime and cement, were tested to determine compressive strength. Lime-stabilised CEB showed a dry compressive strength of 0.81 MPa with 3% of lime and 1.3 MPa with 15%, while cement-stabilised showed a dry compressive strength of 1.12 MPa with 3% of cement and 3.4 MPa with 9% of cement. It was observed that not only cement stabilised blocks exhibited higher compressive strength requiring less stabiliser, but cement-stabilise block gained strength faster than the lime-based one. Moreover, when wet compressive strength was evaluated, cement blocks were found to have lost 37% of their dry compressive strength, proving to be more effective than lime blocks, which lost 50% of their strength. Mortar was also tested; various proportions of lime-cement mixes were investigated with different ratios of soil and sand. Although the highest compressive strength values for mortar were recorded when soil was not included in the mix, stabilized earth-based mortar achieved strength values comparable to cement-stabilized CEB, meeting Ghana's requirement of 1.4 MPa for wet compressive strength in mortars.

In [119] the authors assert that using more than 4% of mineral binder, lime or cement indifferently, is not advisable for environmental reasons. Low-binder stabilised CEB are proposed as an alternative, adopting 2 to 4% of stabilisers. Different soil mix were used, both can be classified as clayey soil but with different mineralogical composition. One containing kaolinite, and the other, with chlorite and montmorillonite. The compression tests were performed on cylindrical specimen with 1:1 geometric proportion. Both soils

stabilised with different percentage of lime did not show noticeable improvement in dry strength, with first soil type the mechanical performance decreased actually. Cement proved to notably enhance the block strength from 80% to 100%. When wet compressive strength was evaluated a sharp drop in compressive strength value was recorded, leaving only 4% cement-stabilised CEB as a valid option for construction building.

A significant distinction between cement and lime as stabilizers is the differing behaviour of stabilized earth products when exposed to water. Lime-stabilised products tend to break down due to a loss of cohesion during water absorption tests. In contrast, cement, whether used alone or in combination with lime, enhances the block resistance to water. Increasing cement content reduces water absorption by filling soil voids and lowering porosity. Although lime more effectively reduces soil water affinity, its stabilizing effect is limited when used alone. Hydrated cement only produces about 25% lime, resulting in less available lime to reduce water affinity. Consequently, adding 6% lime decreases water absorption more than adding 6% cement. This indicates that even if lime helps reduce water absorption, it is insufficient alone for water resistance, unlike cement-based stabilizers [120].

In his studies, Kerali proposed that a portion of the cement used in producing compressed earth blocks (CEB) can be substituted with mineral additives, such as fly ash, ground granulated blast furnace slag, silica fume (CFS), volcanic ash, with CFS being particularly common in cement mixes due to its high reactivity as a silica source [90]. CFS has a particle size range approximately three orders of magnitude smaller than that of ordinary Portland cement, this allows CFS to fill the pores more effectively, preventing moisture penetration and enhancing the strength properties by ensuring denser packing of the binder. Cubic specimens were cut from the block after complete curing. Compressive strength showed to be higher in the silica fume-cement-stabilised CEB compared to cement-stabilised CEB (90% greater). A linear increase was recorded with the increase of cement content in both mixtures. Compacting stress also proved to influence the mechanical properties of the blocks. An increase from 6MPa to 10 MPa compaction stress led to an increase in strength of 16% c.a.

Oti [40] investigated engineering properties of earth blocks, from clayey soil, when different stabilisers are employed, such as Ground Granulated Blast furnace Slag (GGBS), a cementitious material derived from blast furnace waste, activated with lime (quicklime and hydraulic lime) or cement. Application of GGBS alone as a stabiliser can be found, however, in these cases, strength development occurs over extended periods. Lime was used in a quantity of 1.5% of the total mix, while cement comprised 1.4%. GGBS was included at 5.5%. Blocks were moulded using a hydraulic press, at a compaction

pressure of 15 MPa. The entire block was used as sample in the compression tests. At 28-day the compressive strength of the different soil mixes did not show significance difference. The cement-GGBS stabilised blocks showed an overall minor strength capacity compared to the lime-GGBS one. The difference was appreciable at 90-day curing time (60% increase), where lime had more time to develop strength. The use of the by-product GGBS can reduce the quantity of lime and cement for soil stabilisation, lime offered a better performance with clayey soil thanks to the formation of calcium silicate hydrate gel.

Again, in [121] strength and microstructure of earth blocks was investigated, when lime-GGBS mix and cement-GGBS mix were used as stabilisers. The study demonstrated that bricks produced with lime-activated GGBS exhibited superior strength compared to those made with Portland cement-activated GGBS. This difference is attributed to the enhanced hydraulic reactivity of the lime system in clay, likely resulting from elevated alkali/pH levels. The reactivity and formation of strength-contributing silicates and aluminates were not enhanced to the same degree in the cement-stabilised mix, leading to lower strength values. This is not strictly correlated to the better performance that lime might exhibit with clayey soil, indeed lime works effectively with GGBS by enhancing its properties through hydration in a more effective way. Lime creates an alkaline environment that activates and hydrates the GGBS, leading to the formation of cementitious compounds like Calcium Silicate Hydrate gel, which are necessary to improve strength and durability of the earth blocks. In the context, lime is preferred over cement for activating GGBS.

Kwon in [122] similarly examined the stabilization of compressed earth blocks (CEB) using ground granulated blast furnace slag (GGBS). In this study 20% of blast furnace slag is mixed with 10% cement and soil is compacted at 10 MPa to achieved higher performances of earth blocks. Soil with clay content higher than 25 % were selected and mixed with different percentage of sand (from 10% to 100%). Cylindrical samples were specifically moulded to test compression strength. Higher strength values were obtained when 65% of sand was added to the mixture. The addition of sand (finer particles) in GGBS- cement stabilised soil resulted in enhanced mechanical properties of the blocks. Similarly, a decrease in strength was recorded for clay content higher than 14%.

Multiple studies show how cement stabilisation can be a more effective process compared to lime stabilisation when to improve earth block performances. Silva in [114] compared unstabilised CEB with 8% cement-stabilised and 4% lime - 4% cement mix stabilised blocks. The compressive test was conducted using two half blocks stacked on top of each other to create a prism. The trend in compressive strength of the blocks

indicates an increase as the percentage of cement increases. Even if unstabilised blocks exhibit higher thermal performances, it is advised to stabilise with cement due to mechanical advantages. However, if high load-bearing capacity is not strictly required, a mix of lime-cement stabilisation can be adopted because of the similar water erosion properties of cement-stabilised CEB, even though the mechanical resistance is lower.

Again, in [123] Tran states that cement content is one of the most crucial factor for mechanical strength development. Different percentage of cement, based on dry soil weight, are studied. Cornsilk fibres are also added to the mix to enhance the properties of the block using a by-product. Compression strength was tested using cylindrical specimens. Highest mechanical strength was reached with 12% cement content and 0.25% of fibre (9 MPa), a more than acceptable compressive strength value was reached at 8% cement content with both 0.25% and 0.5% of fibre (5.5 MPa and 6 MPa). With higher percentage of fibre, no visible improvement was recorded in terms of strength. However, it was visible that the driving factor for strength was cement-stabilisation. Nonetheless, fibres can be used to achieve good engineering properties with a reduced cement content, provided they are properly balanced. Mechanical strength evaluation was also performed at different curing time period but is not particularly striking to see that higher strength values were recorded at 28-day curing time, since cement particles need time to fully hydrate and develop strength.

Regardless of the mechanical properties of the block, durability is a significant parameter in defining the quality of a CEB. Rainfall can be highly damaging for earthen construction [2]. Indeed, Heathcote asserts that durability is one of the primary concerns associated with this type of material. In this study a ratio between wet/dry compressive strength is proposed as a method to evaluate earth-based blocks durability, following CraTerre specification, as minimum dry compressive strength of 2.4 MPa (for single storey building) minimum saturated strength of 1.2 MPa with a ratio higher than 0.5. If cement is used as stabiliser the ratio should be higher, approximately 0.3 or 0.4. Compression tests were performed on cement-stabilised CEB (2, 5 and 7.5% cement content) that were used to build a small construction and left exposed to the weathering for roughly two years, mimicking real house-construction conditions. Different soil types were used as well, with a maximum clay content of 10%. Bricks were removed and tested, but only 7.5% cement-stabilised ones were suitable for testing. Half blocks were used to investigate oven dry/wet compression strength. The ratio found was 0.5 which was considered acceptable.

Ngowi also states durability as crucial aspect of earthen construction. In [124] different stabilisation material are investigated to improve strength and durability of earth blocks.

Compressive strength is considered as an indication of durability of the block, too. Cement and lime materials were compared with other stabilisers as bitumen and cow—dung, which is an indigenous method, to some villages in Boswana, used to produce adobe blocks. The blocks were moulded at 2 MPa compaction stress, adopting two different soil mixtures, one sandy soil and one clayey soil. Cement and lime were used at concentrations of 5%, 7.5%, 10%, and 15%, while bitumen and cow dung were applied at 10% and 20%. The use of 5% cement resulted in a compressive strength increase of 2.5 times compared to unstabilised blocks. Notably, there was no significant difference in compressive strength between the two soil types when they were either unstabilised or stabilised at the lower percentages. However, sandy soil stabilised with 15% cement exhibited a 25% greater strength than clay soil stabilised with the same percentage. Lime-stabilised blocks generally showed lower compressive strength compared to cement-stabilised ones, although they performed better with clay soil, achieving a 34% increase at 15% lime content. Both cow dung and bitumen were found to be ineffective for soil stabilisation; at 10%, minor reductions in mechanical resistance were observed, and at 20%, the blocks experienced a significant drop in compressive strength (reduction of 40%). While lime-stabilised blocks can be a viable construction option, cement-stabilised blocks proved to be more effective in terms of strength and durability. In contrast, unstabilised CEB, cow dung, and bitumen blocks did not meet the minimum strength requirements.

Walker in [125] investigated the differences in mechanical performance among stabilised CEB with 5% and 10% of cement in both wet and dry conditions. The CEBs were produced with a manual press at compressive strength of 2 to 4 MPa. The entire block was tested to assess its compressive strength. When saturated blocks were tested a reduction in compressive strength was measured as the clay content increased (compressive strength of 2 MPa). This is primarily due to the weakening effect of clay minerals on the bonding between cement paste and the sand/fine gravel matrix. While cement does react with and stabilize clay minerals, the strength of this colloidal/cement structure is significantly weaker compared to the granular/cement matrix. Additionally, the presence of cohesive soil aggregations formed during mixing impairs the effectiveness of cement, or any stabilizer. When dry blocks were tested the cement content showed to be determinant to the compressive strength value, while clay content had less influence (compressive strength of 5 MPa). Increased cement content enhances both the saturated strength and durability of cement-stabilised soil blocks, while higher clay content impairs them.

Morel in [45] states that, for optimal CEBs production, the clay fraction should be lower than 25% of the dry mass with cement percentage from 4 to 10%. Manually pressed blocks - the most common form of earth blocks - exhibit compressive strengths ranging from 2 to 3 MPa. One of the factors that might influence mechanical strength is cement content, which is found to directly improve the mechanical strength. Despite this higher compressive strength levels can be reached if hydraulic presses (hence, higher compaction values) are used in the production process, instead of manual presses. Higher compaction stress results also in higher density. It is reported that as the density increase the compressive strength increase, independently from the mixture composition.

Hence, compaction strength during the manufacturing phase (mechanical stabilisation) is another factor that has been investigated to understand its influence on stabilised compressed earth blocks CEB strength. Guettala in [126] tested CEBs produced with increasing levels of compaction stress (5, 7.5, 10, 12.5, 15, 17.5, and 20 MPa). Regardless of the variation in stabiliser content, the study demonstrated an increase in the compressive strength of the material up to a compaction pressure of 17.5 MPa. Beyond this pressure, no significant differences in strength were observed, identifying 17.5 MPa as the optimal compaction stress to achieve enhanced mechanical performance. This trend was also observed in wet compressive strength, highlighting the substantial impact of mechanical stabilisation on the final strength of the earth blocks.

In Guettala work [127], the compaction pressure was set at 15 MPa to investigate the influence of the stabiliser used. To investigate compressive strength, the specimens were created by stacking half-blocks with a mortar layer, as described in [128]. Cement-stabilised blocks showed an increase in strength as the cement content increased, with a compressive strength of 18.4 MPa in dry conditions and 12.7 MPa in wet conditions with 8% cement. Similarly, cement-lime-stabilised blocks also exhibited increased strength. Higher compressive strengths of 21.5 MPa in the dry state and 15.6 MPa in the wet state were recorded when cement and lime were used together to stabilise the blocks (8% cement and 4% lime). Mixtures of cement and resin (8% cement with 50% resin) were also tested to stabilise the earth blocks. Although the compressive strength obtained in wet conditions was notable (14 MPa), and the resin-stabilised blocks showed higher durability performance, the values obtained do not significantly justify the use of resin mixtures in the earth mix.

Mahdad and Benidir in [129] investigated lower compaction pressures (2, 5, and 7 MPa) for both cement-stabilised blocks and lime-cement-stabilised blocks. Prismatic samples were tested without the mortar joint layer, using displacement control. Various

percentages of cement content were examined (2%, 4%, 6%, and 8%). At lower cement content levels, differences in compressive strength between samples with different compaction stresses were not significant. However, as the cement content increased (6% and 8%), samples subjected to higher compaction stress exhibited much higher compressive strength (with 8% cement, 8 MPa at 7 MPa compaction pressure versus 4 MPa at 2 MPa compaction pressure). Lime-stabilised blocks did not show this improvement, while a mix of cement (5%) and lime demonstrated a significant increase in compressive strength at higher compaction energy with a lime content of 8%. Non stabilised CEB exhibit lower compressive strength (less than 1 MPa) at a compaction stress of 2 MPa, and almost doubled, 2 MPa, at both 5 MPa and 7 MPa compaction pressure. This demonstrates that a higher cement content, combined with a high initial compaction pressure, enables the production of earth-based blocks with enhanced mechanical performance.

Bahar in [130] studied the different properties of CEB using different methods for mechanical stabilization and together with the chemical stabilization of soil. A soil enriched with sand was used as material to produce the blocks, with cement was chosen as stabiliser, different quantities of stabiliser (4%, 6%, 8%, 10%, 12%, 15%, and 0% of cement by weight of soil) were adopted. Cylindrical specimens were specifically moulded for testing. Static, vibro-static and dynamic compaction methods were investigated. Dynamic compaction involves dropping a weight from a prefixed height onto a cylindrical specimen (120 mm diameter, 180 mm height), number of drops was calibrated in order to achieve the energy levels set at 3.0, 5.5, 8.3, and 10.3 J/cm³. Static compaction is achieved by applying static pressure using a universal testing machine on a cylindrical mould (100 mm diameter, 165 mm height) at a strain rate of 1.27 mm/min. Vibro-static method involves vibrating specimens on a shaking table for one minute before applying static compaction, which leads to higher compressive strengths compared to static compaction process alone. The dynamic compaction method shows that increasing the energy level leads to compressive strength gains exceeding 50%, although a reduction in optimal water content was also noted. Testing with cement at 2% and 12% revealed a twofold increase in strength. The static method demonstrated a direct relationship between compaction stress and strength, while the vibro-static method showed only a minor improvement (5% increase in strength) but was effective for earth mixes with higher water content. Furthermore, compressive strength was observed to increase with higher cement content, recording 1.5 MPa at 0% cement, 4 MPa at 8%, 4.5 MPa at 10%, and 6.5 MPa at 20%. A comparison between dry and wet strength identified 10% cement content as optimal; above this percentage, a reduction in compressive

strength exceeding 50% was recorded for wet specimens, which does not justify using such a high amount of stabiliser

Fibres may be incorporated into the mix to enhance block strength. Ismail and Yaacob in [131] investigated cement-stabilized blocks reinforced with oil palm fibres. The dry mix comprised 70% soil, 24% sand, and 6% cement, with fibres incorporated (100 to 260 mm long) at ratios of 1% to 5% of the cement content. A pressure of 0.1 MPa was applied to compact the blocks. To evaluate compressive strength, the entire block was used as a specimen. The highest compressive strength values were observed with fibre contents of 2% and 3%, yielding strengths of 10.6 MPa and 10.65 MPa, respectively. In comparison, cement-stabilised blocks without fibres exhibited a compressive strength of 10.22 MPa, indicating an increase of approximately 4% with fibre inclusion. However, higher fibre percentages resulted in reduced mechanical performance, with compressive strengths of 9.5 MPa and 9.2 MPa, which were lower than that of the fibre-free blocks. This decrease may be attributed to reduced density, as the fibres are lighter than the soil, as well as a reduction in the block's effective cross-sectional area. The contact between fibres can create discontinuities in the soil matrix, acting like voids. Since voids in soil mixtures are typically eliminated through mechanical and chemical stabilisation processes, excessive fibre addition can be counterproductive.

Namago work [132] employed a manual press to produce earth blocks at compaction pressures ranging from 2 to 4 MPa. Sisal fibres were incorporated into the cement-stabilised compressed earth blocks (CEB), investigating varying cement percentages (5%, 9%, 12% by weight) along with different fibre percentages (from 0% to 1.25%). The compressive test was performed on prisms cut out from the block. A linear relation was defined between the cement content and compressive strength (compressive strength in a range of 3.5 to 7 MPa), as well as for the tensile strength. When only fibres were used content a decrease in mechanical resistance was registered with fibre content higher than 0.75%, indeed, an excess of fibre content can lead to a formation of fibres clots, which leads to weakness points in the block. A combination of cement and sisal fibres can be used to achieve a suitable product to use in constructions by reducing cement content.

Mostafa studies [133] investigate the mechanical performances of cement-stabilised CEB mixed with banana fibres (5% banana fibres in relation to the cement content). The mixture used to produce these earth blocks consisted in 35% of clay, meaning a slightly clayey soil, 35% of sand and 7 % of cement. Small prismatic blocks were produced to test compression strength. The compression strength achieved was relatively low in respect of the cement content (3.8 MPa), this can be due to the high amount of clay particles in the clay mixture, which is more suited for lime-stabilised products. On the other hand,

the banana fibres allowed for an increase of the mechanical properties of the blocks itself (compressive strength of 6.5 MPa) with 60 mm long fibres. Flexural strength also benefited from the addition of banana fibres, doubling the strength compared to the samples without fibres. Although compression tests may not be entirely representative of the actual strength of regularly sized blocks, due to the specially formed specimens used for compression testing [45], the study confirms that the use of fibres in cement-stabilised earth blocks can be beneficial. It is important to consider the length of the fibres, as excessively long fibres can lead to alignment, thereby reducing the isotropic effect provided by randomly dispersed fibres, which allows for the compression strength increase.

Galindez in [134] evaluated mechanical properties of CEB in respect of changes in plasticity index of the soil. To focus on that cement was not added to the mixture. Clayey soil, as stated, are not suitable for cement stabilisation, hence they required higher cement content to achieve comparable mechanical strength of cement-stabilised sandy soils. Different compaction pressure levels were adopted (0.2, 0.4 and 0.8 MPa). The compaction process also aims to prevent the clayey block to excessive swelling by reducing the space between clay particles and therefore limit water intrusion. Compression tests were conducted to investigate the strength of blocks as compaction pressure and plasticity index varied. The compaction strength was found to be directly proportional to both the compaction pressure and the plasticity index. Soil with a medium plasticity index (15.81), compacted at 0.8 MPa, showed a compression strength of 4 MPa, whereas soils with lower plasticity indexes (at the same compaction pressure) exhibited lower compressive strengths. The relationship is not linear; for instance, a plasticity index of 7.48 corresponds to a strength of 2.5 MPa, while a plasticity index of 13.38 corresponds to a strength of 3.5 MPa. Nevertheless, it has to be considered that an excess of clay in the soil mix might lead to excessive swelling and shrinkage, effectiveness and limitations of compacting high percentage of clays are influenced by how these clays behave when they lose moisture, including their tendency to shrink, crack, and therefore affecting the mechanical performance.

High sandy soil CEBs were studied in [135], focusing on the variation of mechanical behaviour, at the variation stabilisation processes. A sandy soil was used to produce earth blocks, which was chemical stabilised using cement or lime, and further mechanical stabilised at 2, 5, 7 MPa compaction stresses. Lime-stabilised CEB were let cure further the cement one to let strength fully develop. Blocks were used as specimen for compression test. In compliance with previous studies strength increases with the increment of percentage of stabiliser and compaction pressure. Compaction pressure

alone allows for an increment of 3 times. Significant increases are observed with the rise in cement content at lower percentages (2%, 4%, and 6%), with the increase between 6% and 8% being just of 9%. Consider reducing the cement amount in favour of higher compaction pressure if the available machinery permits. As expected, lime-stabilised CEBs exhibit poorer performance at the same binding quantity and compaction level as cement blocks. A mixture of lime and cement was used as a stabiliser. The mixture with 5% cement and 8% lime exhibits the same strength as 8% cement-stabilised blocks at the highest compaction stress. Again, cement proves to be a more effective binding agent to use as a stabiliser. The mechanical behaviour was analysed not only for the blocks itself, but also for masonry panels. An earth-cement mixture was adopted for mortar, same soil was used with 15% of cement content. Mortar cubes were tested in compression, a compressive strength of 2 MPa was recorded. Triplets were studied when tested in compression. It has been observed that the use of 8% cement content in blocks leads to an increment of triplets' strength of 58% compared to 4% cement stabilised triplets, which exhibit the same strength level of the unit alone. For 6% and 8% stabilised triplets no noticeable difference can be found in compression strength. An increasing reduction of strength, compared to the block strength, is indeed recorded as the cement content increase. Wallets were also tested, exhibiting brittle behaviour and a compression strength reduced of roughly 50% of the same block triplets.

In summary, advancements in the production of compressed earth blocks (CEBs), different mortar composition, and the development of diverse stabilisation and reinforcement techniques have markedly enhanced the mechanical performance of earthen masonry. Optimized soil compositions, tailored stabiliser contents (e.g., cement, lime, or their combinations with mineral additives such as GGBS and silica fume), and controlled compaction pressures have led to improved compressive and flexural strengths. The incorporation of fibres further enhances mechanical performance when applied in optimal proportions that preserve the soil matrix integrity. Despite these improvement, earthen systems continue to exhibit inherent vulnerabilities - most notably, soil composition, moisture sensitivity, and reduced resistance to horizontal loads - which underscore the variability in testing protocols and material performance. These challenges highlight the necessity for standardised testing methods and ongoing research into reinforcement strategies and structural optimization to fully realize the potential of sustainable, earth-based construction in modern practice.

2.2 Mechanical Behaviour of Masonry Structures

Masonry is an arrangement of strong elements – the unit – of different material nature, organised with specific patterns. Mortar can be used to bind the units, providing cohesion and stability to the structure [136]. The use of two material with distinct properties defines masonry as a composite material. When mortar is not used the surface of contact between the bricks are defined dry joint [137].

Mortar can be defined as a mixture of binder (lime, gypsum, cement) combined with sand (inert material) and water. The fresh mixture remains workable for a varying period and hardens within different time frames depending on the binder used. The sand serves to:

- prevent excessive shrinkage, which could lead to cracking,
- increase the volume of the mixture without compromising its mechanical properties,
- effectively and economically fill joints and cavities.

For mortars used in structural masonry, medium-grain sand ($d < 1\text{ mm}$) is typically used, while fine-grain sand ($d = 0.2\text{--}0.5\text{ mm}$) is preferred for plastering. Water facilitates the chemical reaction with the binder, allowing the sand particles to bond and ensuring adhesion of the mortar to the masonry components. It must be pure to avoid undesirable reactions that could negatively affect mechanical strength, cause efflorescence, or alter the colour.

Traditional masonry constructions do not have a distinction between bearing and not bearing element, unlike engineered modern structure (e.g., frame structures). Masonry can be used to build structural element like walls, pillars, arches and vaults.

The mechanical behaviour of masonry is influenced by the mechanical response of both the units and the mortar, each exhibiting distinct properties (see Figure 2.13). Consequently, the overall mechanical behaviour of masonry can be considered as a composite, reflecting an average of the characteristics of both the units and the mortar [138].

Masonry, because of its nature, is a heterogeneous orthotropic material which with good mechanical performance when subjected to compressive loads. A poorer performance is exhibited in tensile and shear stresses [139], [140], [141].

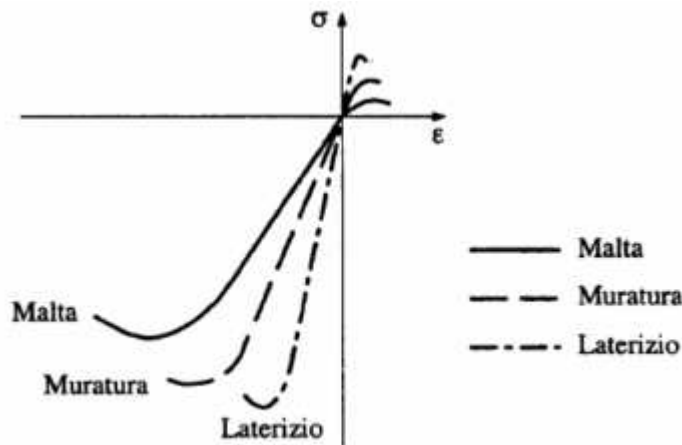


Figure 2.13: Mechanical behaviour of masonry constitutive elements (mortar and unit) and masonry system.

Masonry, when subjected to vertical loads, typically exhibit failure with vertical cracking, this happens due to the different level of deformability of the materials. The mortar, which usually has higher deformation capacity, is in a state of confinement between the layers of bricks, which creates a triaxial compression state. Due to the reaction on the faces of the bricks that adhere to it, a biaxial tensile state is generated, leading to failure as the load increases [136].

The tensile strength of masonry is very low, to the point that it cannot be relied upon. When the wall is subjected to in-plane stress, tensile forces develop through the mortar joints, and the resistance depends on the adhesive forces between the bricks and the mortar [137].

The shear strength of masonry is involved in the response to seismic or other horizontal forces. In these scenarios the walls (shear walls) are subjected to horizontal forces transmitted from the floors they support at the top, and reaction forces from the lower supports at the base. In buildings there is never only horizontal force, but a combination of actions:

- There is always a vertical compressive component (self-weight of the masonry, floor loads, and the weight of the overlying walls).
- The horizontal force at the top tends to rotate the wall: the resisting mechanism is due to the vertical component and the restraining action of any rigid top constraints (such as reinforced concrete floors).

- Very tall walls cannot be considered stocky and must be analysed taking into account bending effects, which become predominant due to the horizontal force [140].
- In stocky walls, failure, attributable to shear, can occur in various ways, not always following the pattern of an isotropic panel that cracks due to the principal tensile stress [139].

When horizontal forces are stressing the masonry three failure mechanisms are typically observed, depending on the compression forces which act at the top of the masonry wall [141]:

- Mortar joint failure, due to lower vertical compression values
- Shear failure in tension of the units, intermediate values of vertical compression
- Bending failure, failure due to crushing of the masonry; vertical stress values close to the compressive strength

The standards consider masonry as an isotropic continuous material; however, this does not allow for an accurate mechanical description of the material. Masonry is a heterogeneous and discontinuous material. If it is to be simplified to a continuous material, it should be considered as an orthotropic continuous material.

2.2.1 Reinforcement in masonry structures

Because of the poor performance of masonry to horizontal loads, reinforcement system can be used to improve the mechanical response [136], [142].

Reinforcement systems can be adopted to overcome these limitations, rebars or meshes elements are typically used to improve the overall strength and durability of the structure. The principal methods of reinforced masonry include:

- Reinforced Masonry with Vertical Reinforcement: In this method, vertical steel reinforcement bars (rebar) are embedded in the masonry units, typically placed within the mortar joints or cores of hollow concrete blocks. The reinforcement helps resist tensile stresses and bending moments [142], [143], [144].
- Reinforced Masonry with Horizontal Reinforcement: Horizontal reinforcement, often in the form of steel bars or mesh, is placed within the bed mortar joints. This reinforcement helps distribute tensile forces and provides resistance to cracking, enhancing the masonry's performance under shear and bending loads [143], [145].
- Reinforced Masonry with frame: A more comprehensive form of reinforced masonry involves the use of a "cage" element, consisting of both vertical and

horizontal reinforcement that is placed inside the wall before the masonry is constructed [142], [146].

- **Steel Plate Reinforced Masonry:** In some cases, steel plates or mesh can be placed on the exterior or between layers of masonry to provide additional tensile and shear strength, particularly in areas subject to high loads. This is usually a reinforcement [147], [148]
- **Composite Reinforced Masonry:** This method involves using a combination of different materials, such as steel reinforcement and concrete, integrated into the masonry structure to optimize both the compressive strength of the masonry and the tensile strength of the reinforcement technique for existing masonry [149], [150].

Chapter 3 - Investigation on Earthen Masonry: Background and Methodological Choices

This chapter explores the methodologies used to investigate earthen masonry. Due to the high heterogeneity of earth-based construction, the lack of standard productions systems and common guidelines to investigate these materials lead to the use of existing standard procedure, not related to earth, adapting them to the specific case study. An overview of the experimental procedure and numerical approach is here presented.

3.1 Adaptation of Experimental Standard Procedures

The lack of standardized procedures for assessing the mechanical properties of earthen elements and structures represents a significant limitation to the use of this material in construction. The absence of a common regulatory framework is closely related to the heterogeneous nature of the material. It is incorrect to approximate earth as a single material, as the composition of the soil, water content, and potential stabilization processes lead to the creation of a multitude of materials with varying physical and mechanical properties. The high specificity of the material is thus the cause of these limitations. In this research, to investigate the mechanical properties of the materials used and the construction system, existing regulations were referenced, enabling direct comparisons. For earth blocks, the regulations pertaining to concrete were used, due to the nature of the compressed earth blocks stabilized with a small percentage of cement (8%) under study. UNI EN 12390-1 [151] was adopted as guideline for selecting the geometry of the specimens, while UNI EN 12390-3 [152] was adopted to investigate the different failure typologies of the specimens. Cubic specimens were selected due to their ease of preparation compared to cylindrical ones. The dimensions of the cubes were constrained by the original block size, which had a height of 90 mm; therefore, each cube was cut to a dimension of 90 mm per side (see Figure 3.1). It should be noted that these dimensions do not conform to those specified in standard testing procedures. The prepared specimens were subsequently subjected to uniaxial unconfined compression testing.

For the mortars, the relevant standards were also applied - UNI EN 1015-11: 2007, [153] - although the properties obtained did not allow for direct comparisons with the data described in the standards. As a result, variations in the standard testing methods were

introduced to reach a more comprehensive understanding level of the mechanical parameters for the mortars used.

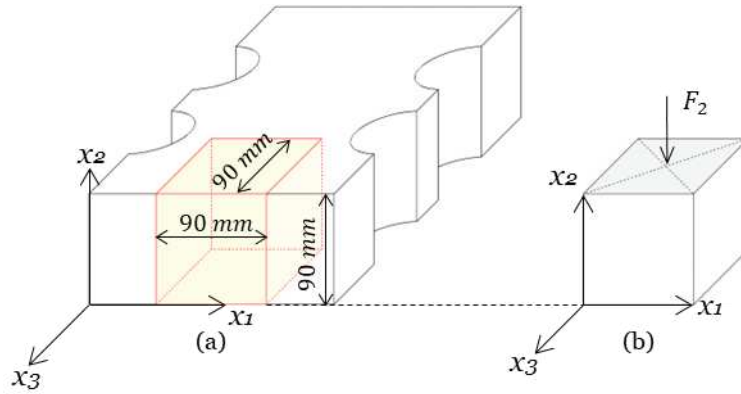


Figure 3.1: Specimen extraction (a) from SCEB for uniaxial unconfined compression test (b).

While the three-point bending test setup was performed in accordance with the standard specification, the setup for the compressive test was modified to allow for the extraction of both compressive strength and elastic modulus. Rather than following the procedure outlined in UNI EN 1015-11—which prescribes testing mortar halves using a dedicated frame that applies load over a $40 \times 40 \text{ mm}^2$ surface—cubic specimens with a side length of 40 mm were cut from the mortar halves (see Figure 3.2). These specimens were tested without the use of the frame to facilitate the measurement of deformations. The full procedure is detailed in Section 5.1.2.

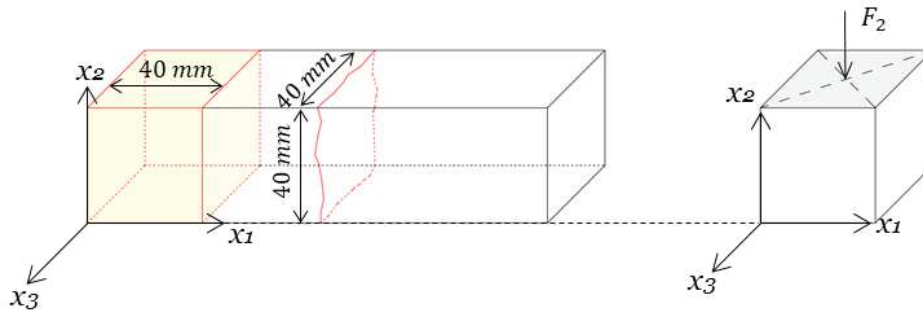


Figure 3.2: Identification of mortar cubic specimen for uniaxial unconfined compression test.

Regarding compressed earth block (CEB) masonry, as indicated by Reddy in [71] in the absence of a dedicated design code specifically addressing stabilised CEB masonry, the guidelines provided in conventional masonry codes may be appropriately interpreted and adapted to inform the design of cement-stabilised CEB structures.

Since the construction system is load bearing masonry, testing procedure for masonry panels followed the standards for masonry UNI EN 1052-1:2001 [154]. However, variations in specimen dimensions and proportions were required due to the non-standardized size of the blocks and the minimum wallettes dimensions needed to assess different construction configurations. To allow for the building of a minimum frame the panel dimensions were set as $1225 \times 945 \times 305 \text{ mm}^3$. The standard requires that $h_s \geq l_s$ where h_s is the height of the wallette, and l_s is the length of the wallette. However, in this study, the specimen length was increased to accommodate a representative minimum reinforcement frame.

3.2 Measuring deformations: Digital Image Correlation technique (DIC)

Not only the experimental procedure, but also the instrumentation used, was the subject of study. Due to the surface type and the inability to predict the material behaviour, traditional instrumentation methods for recording deformations (such as strain gauges and LVDTs) were found to be insufficient and/or ineffective for this series of tests. To overcome the limitations imposed by traditional instrumentation, an adaptation of the optical technique Digital Image Correlation (DIC) was adopted to allow for better control of the specimens.

3.2.1 Introduction and background

Developed in the early 1980s, Digital Image Correlation or DIC is a non-contact optical technique that analyses deformation and motion through correlations between digital images captured before and after loading. In experimental mechanics, DIC enables full-field strain and deformation analysis by comparing a set of images of the specimens under various loading conditions, presenting their detailed strain characterization.

The accurate measurement and analysis of deformations, strains, displacements, and motion are critical for understanding the mechanical behaviour of materials and structures. While traditional methods such as strain gauges and extensometers provide valuable insights into simpler problems, increasing complexity necessitates more advanced tools [155]. Among various optical methods, Digital Image Correlation (DIC) has emerged as a powerful, non-invasive technique for whole-field, contact-free, and highly sensitive deformation measurements, effectively addressing the challenges of recording complex strains.

The development of Digital Image Correlation (DIC) began in the early 1980s with Peters and Ranson [156] who introduced a method for computer-based image acquisition and deformation measurement using ultrasonic waves. This approach analysed deformation by comparing subsets of images and applying continuum mechanics principles. Sutton [155] advanced this technique by developing numerical algorithms and conducting experiments, establishing the feasibility of 2D-DIC, which uses speckle patterns and cross-correlation algorithms to measure deformations (see Figure 3.3).

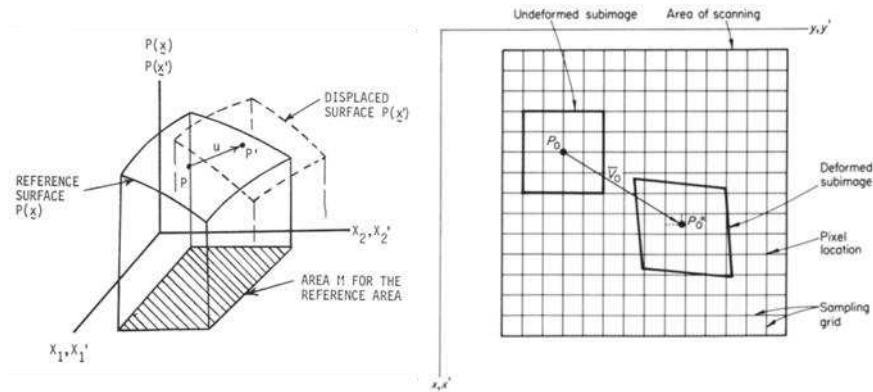


Figure 3.3: Reference and deformation of surface for correlation and sampling grid, and the correlation presented in Peters and Ranson [156] and Sutton [155] respectively

Subsequent studies, as presented by Chu [157] expanded DIC applicability to rigid rotations and rotational deformations, addressing challenges like interpolation models for subsets in deformed states. From the late 1980s to the mid-2000s, DIC saw significant improvements in image capture, algorithm accuracy, and efficiency. Techniques like digital speckle correlation and texture correlation emerged, sharing principles with DIC. Efforts also focused on addressing systematic errors and refining interpolation methods.

By the late 2000s, critical reviews of DIC algorithms identified limitations, leading to innovations like nested coarse-fine algorithms, which improved the measurement of large deformations and rigid rotations [158], [159]. Commercial and open-source DIC systems have since been developed for diverse applications, solidifying DIC as a robust tool for experimental mechanics. Over the next two decades, DIC techniques saw extensive applications across different specimens, scales, environmental conditions, and through application of various imaging technologies. This growth was fuelled by advancements in computational processing, improved imaging devices, and the recognized advantages of DIC over traditional and optical methods. Research focused on enhancing the accuracy, efficiency, and robustness of DIC, addressing equipment-related errors, refining correlation algorithms, assessing measurement errors, and expanding

applications (see Figure 3.4). A significant development was the innovation of methodologies to capture and analyse out-of-plane and 3D movements, broadening DIC capabilities [160].

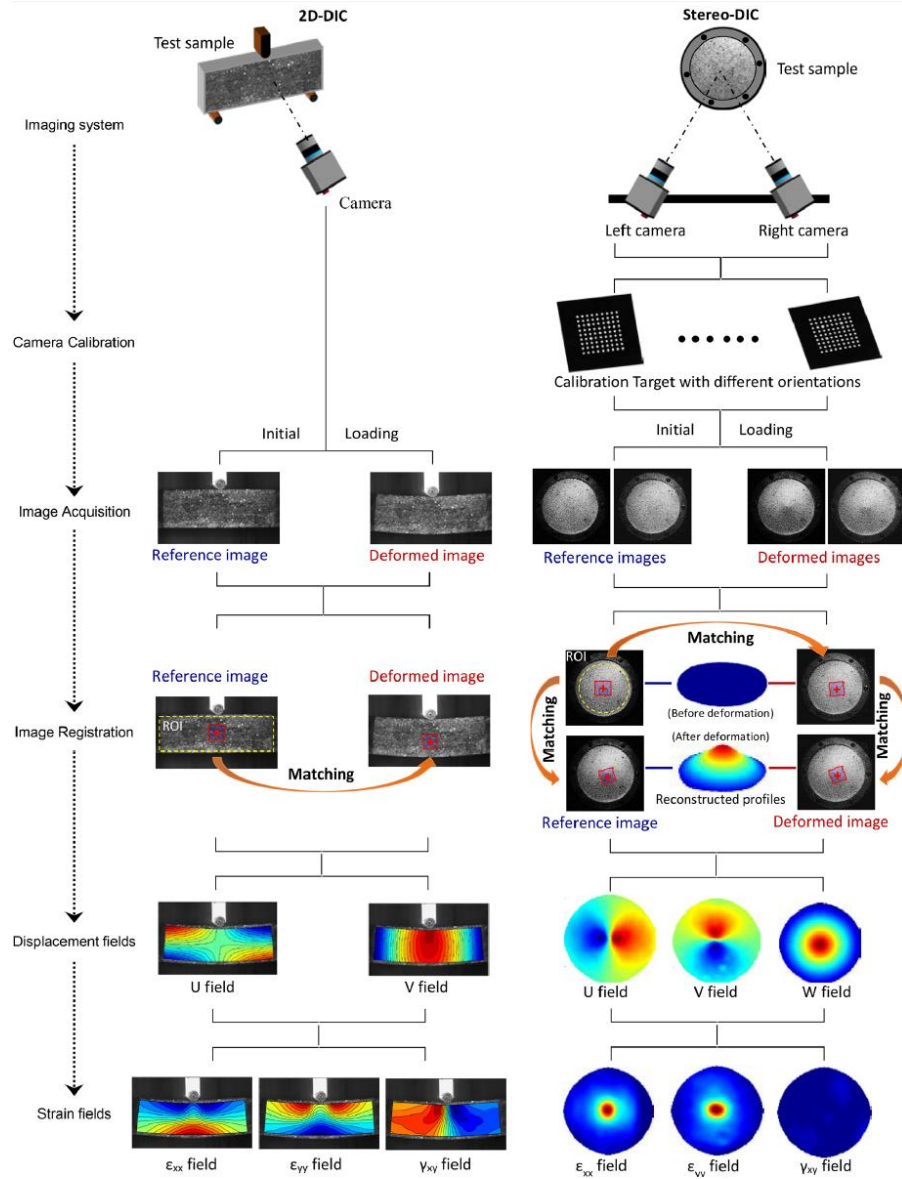


Figure 3.4: Implementation procedure for DIC techniques from [158].

The advancements in DIC, expanding across various fields, scales, and challenging environments, move parallel with the evolution of the technique, from a single image sequence captured, to stereo or multi-Camera [161] (see Figure 3.5) and 3D-DIC [162] to capture out of place and three-dimensional deformations, and to techniques such as Video-DIC capturing high-frequency image sequences for real-time analysis of dynamic events, like impacts and vibrations [163].

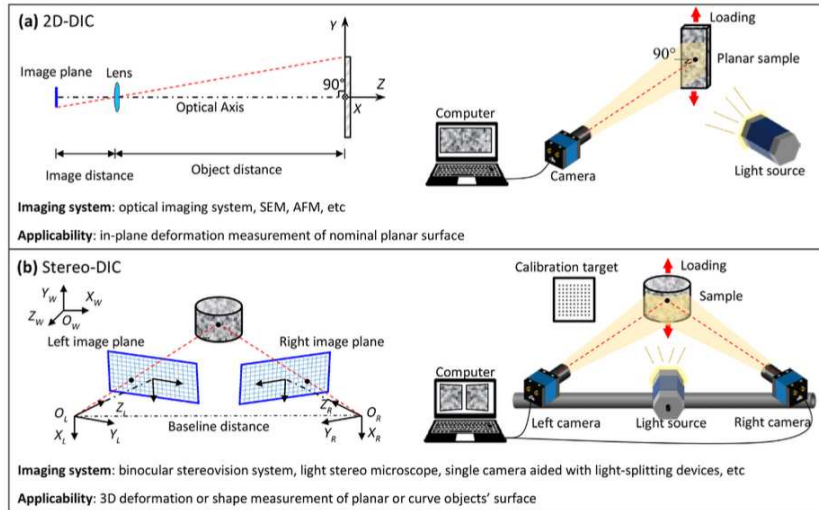


Figure 3.5: Comparison between 2D-DIC and stereo-DIC techniques setup [158].

Further, applications at large scales, such as wind turbines (see Figure 3.6), employ DIC to separate rigid body motions from internal strains, while tools like motion magnification enhance insights into operational deflection shapes under varying frequencies [164], [165]. At the micro-scale, DIC is used for studying elements such as MEMS, microfluidic devices, and biological tissues, providing critical data on mechanical behaviour and aiding optimization [166]. Adaptations for high-temperature and high-pressure environments allow DIC to measure deformation in extreme conditions, including underwater, where specialized techniques address challenges like fluid refraction [167]. Furthermore, on a territorial scale, DIC has been applied to monitor geological phenomena such as landslides, avalanches, and glacier movements, offering precise and repeatable measurements of large-scale deformations [168]. These innovations underline the versatility, efficiency and growing importance of the DIC technique in diverse scientific and engineering domains.

DIC has seen wide-ranging applications in the context of mechanical characterization of earthen constructions and earth-based masonry units. Several studies have demonstrated its utility in evaluating early-age properties, such as drying, shrinkage, and bond strength in earth block settings. In addition to these, DIC has proven to aid in recording deformations, fractures, crack propagations, and the effect of various additives, such as cementitious materials or natural fibres. This section presents few recent studies that illustrate the diverse applications of DIC in understanding the mechanical behaviour of earthen materials.

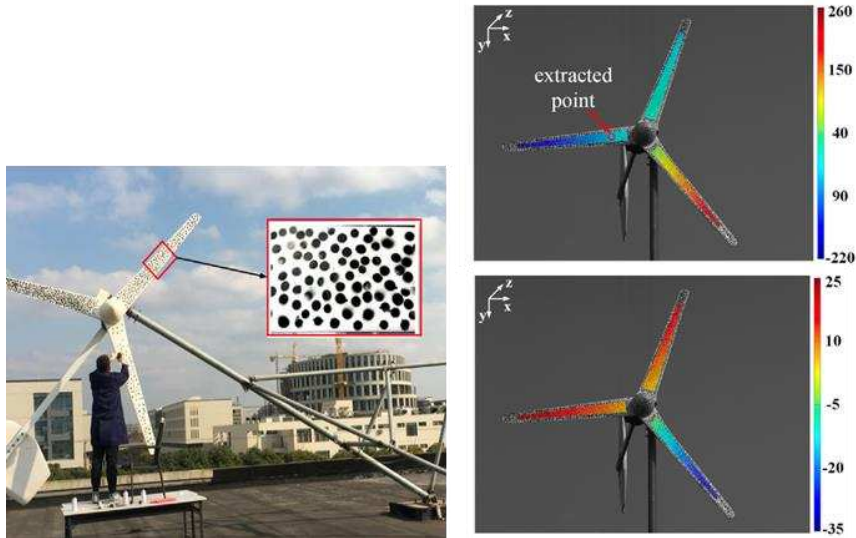


Figure 3.6: Texturized surface on the front surface of the blades and full-field displacement of the blades [165].

Eudave [169] explored the use of DIC for monitoring early-age shrinkage in strengthened rammed earth specimens. While the primary focus was on shrinkage, the study also demonstrated the ability of DIC in capturing spatial distribution of strain. By comparing the deformed surface of the specimens with their initial states, DIC identifies the areas with concentration of stresses, providing a comprehensive understanding of strain distribution. The detailed contour maps generated by DIC also allowed for the visualization of principal strains, offering insights into the stress–strain relationship and the localized behaviours in earth specimens.

Chitimbo [170] conducted a drying experiment in which DIC was used to assess the stiffness of earth-based samples. The study revealed that the maximum compressive strength occurred within the first three weeks of drying, after which the strength plateaued. Within this study, DIC was employed to track the evolution of strain distribution during drying, and the results were cross-referenced with unconfined compressive strength tests conducted at different stages, including 0, 2, 3, 4, 6, 8, and 19 weeks. This combination of techniques provided a detailed picture of the material's evolving mechanical properties over time, highlighting the applicability of DIC in understanding the early-age behaviour of earthen materials.

Kouta et al. (2021) [171] investigated the flexural fracture behaviour of earth concrete in the context of sustainable construction. By performing three-point bending tests on prismatic specimens with varying percentages (0%, 0.3%, and 0.6%) and lengths (12 mm, 24 mm, 50 mm) of flax fibres, DIC was used to monitor crack propagation and the fracture process (see Figure 3.7). The study identified the mechanisms of damage in

compressed earth blocks (CEBs) and demonstrated the advantages of incorporating cement and fibre additives to enhance the material mechanical performance. DIC's ability to capture full-field strain distributions and crack patterns was crucial for elucidating the fracture behaviour and evaluating the effectiveness of fibre reinforcement in improving the sustainability of earth-based construction materials.

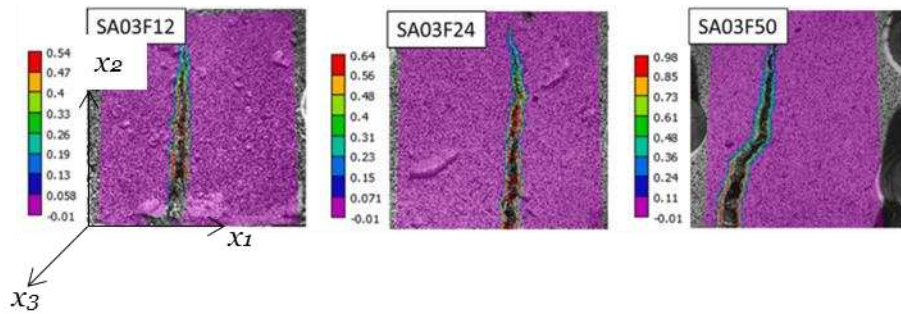


Figure 3.7: Full field strain ε_2 localisation maps from three-point bending test in [171].

Miccoli in [172] applied DIC to study the shear loading behaviour of earthen-based structural elements, such as cob walls. By using DIC to track the strain distribution under shear loading, the researchers were able to define the stress–strain relationships, characterize the nonlinear behaviour of the material, and identify failure modes (see Figure 3.8). The precision of DIC in detecting strain concentrations and monitoring the initiation and propagation of cracks provided critical insights into the structural behaviour of earthen materials under load, reinforcing its value as a tool for understanding the mechanical performance of earth-based construction.

Across these applications, the implementation of 2D-DIC allows for the extraction of full-field strain distribution data, identifying zones of strain concentration and detachment with high precision. This capability is particularly valuable when working with fragile, largely deformable, and heterogeneous materials such as earth. Furthermore, DIC ability to monitor full-field strain and crack patterns ensures its continued relevance in advancing research on earthen masonry and sustainable material performance. By enabling real-time data capturing of the mechanical behaviour of these materials under various loading conditions, DIC contributes significantly to the development of more efficient and sustainable construction techniques using earth-based materials.

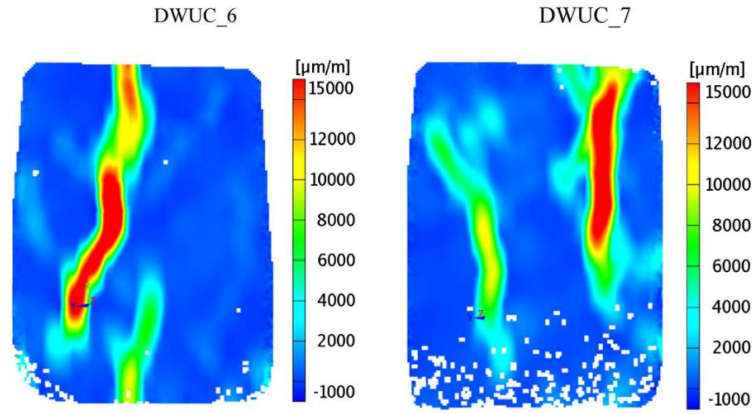


Figure 3.8: Comparison of maximum principal strain of COB panels subjected to shear loads [172].

3.2.2 DIC versus traditional methods for earth

In the case of earth-based material and earthen construction non-contact methods such as Digital Image Correlation (DIC) offer significant advantages over traditional measuring systems. While traditional systems like transducers, extensometers, and strain gauges have long been used to capture specific strain typologies, they face notable limitations. Transducers and mechanical extensometers provide single-point measurements but are vulnerable to damage from specimen failure, detachment during tests (especially within the current context), or the necessity of removal prior to failure. Similarly, strain gauges enable localized strain measurement with a direct correlation to stress but are restricted to point-wise data and often fail in materials like masonry or earth due to delamination and detachment issues. Additionally, most strain gauges are single-use and cannot reliably capture distributed or complex deformations.

In the specific case of earthen masonry, the inherent material characteristics further highlight the shortcomings of traditional contact-based measuring systems. The irregularities and heterogeneous nature of earth-based materials often result in uneven surfaces, making it challenging to establish consistent and secure contact between the specimen and the measuring instruments, such as strain gauges or extensometers.

In contrast to this, the DIC-based non-contact technique address these limitations effectively. They enable both full-field measurements and point wise strains, capturing deformations across the entire specimen surface (see Figure 3.9), which is particularly beneficial for understanding the heterogeneous and anisotropic behaviour of earthen materials. Moreover, DIC provides high-resolution imaging and a comprehensive assessment of material responses under load, making it a more reliable and versatile

technique for analysing the structural performance of earth. These advantages make non-contact methods superior for studying the intricate and distributed deformations often exhibited by earthen materials during experiments.

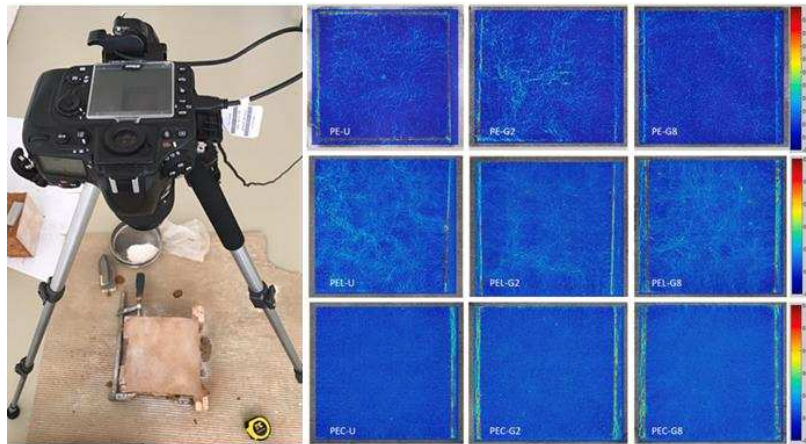


Figure 3.9: Contour map of the tensile principal strain of earth-based mortar [169]

3.2.3 Adaptation of DIC

While the conventional Digital Image Correlation (DIC) method has seen several applications in the context of earthen constructions and in the experimental characteristics of the earth, the current research adopts an “adapted DIC” technique. With a primary goal to evaluate, and especially quantify the strains across the specimen, an adaptation based on defining a virtual system of extensometers was considered. This adaptation stems from the understanding of the response of the material tested, and the nature of the anticipated deformations [173].

Here, as opposed to relying on complex speckle patterns, strategically selects points on the specimen's surface, where high-contrast targets, existing surface features such as the natural texture, change in materials, or any surface imperfections present are tracked [174].

By capturing images at regular intervals, typically using a DSLR camera placed in a well-lit environment, the movement of these markers or features is captured. This approach simplifies the traditional DIC methodology, which requires specialized setups like high-powered cameras or elaborate speckle patterns, making it more accessible for testing earthen specimens in various experimental settings [173].

The underlying theory of the adapted DIC technique relies on tracking the displacement of a pair of blocks or specimen surfaces. In a typical scenario, two blocks with distinct markers or high-contrast features are placed next to each other, and their relative movements are monitored as they undergo deformation (see Figure 3.10). The technique tracks the displacement of these markers in consecutive images, calculating the strain based on the pixel shifts. This allows for a precise quantification of strain across the specimen, including areas where surface features are either natural or artificially introduced. Unlike traditional DIC, which has limitations in non-homogeneous materials, this adaptation minimizes errors by focusing on selected points instead of trying to cover the entire field, thus improving accuracy in capturing the local strain responses, and drastically reducing both the effort that goes into surface preparation and the computational time. Even in the absence of high-contrast markers, the adapted DIC technique can still be effective by utilizing existing surface features, as long as they are discrete enough to be traced through correlation algorithms [173].

The adapted DIC technique, tailored to the specific experiments conducted in this research, along with the nature of the extracted strains and deformations, are presented in the following sections. Additionally, various considerations and modifications made to the adaptation process are outlined.

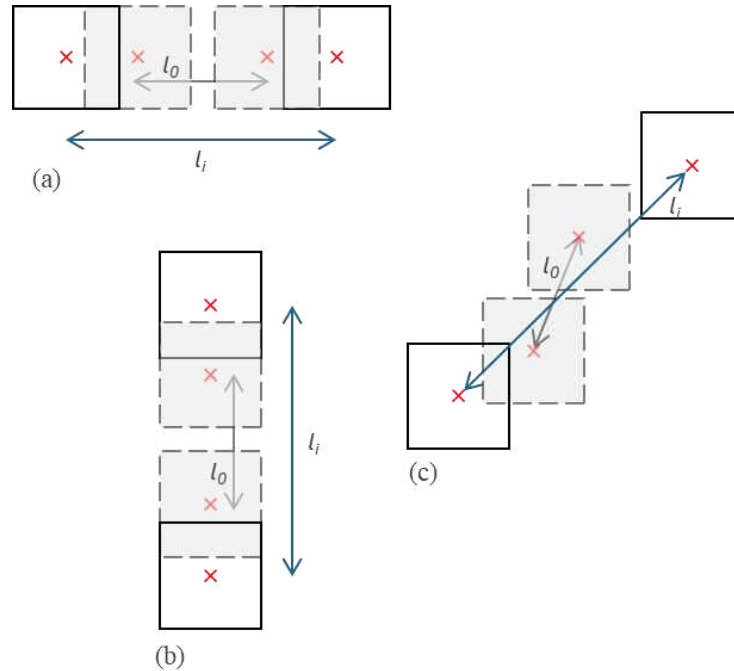


Figure 3.10: Relative movement between two reference points.

3.3 Numerical Modelling of Masonry Structures

A variety of masonry modelling techniques have been proposed in the literature, each offering a unique insight into its structural behaviour. These methods range from representing masonry as a geometric entity and applying plasticity-based approaches to employing non-linear computational models through finite element (FE) and discrete element (DE) analyses. The selection of a suitable modelling strategy depends on several factors, including the structure scale and typology, the required level of detail and accuracy, the availability of experimental data and material characterization, as well as the computational resources [175], [176], [177], [178].

3.3.1 Approaches on masonry modelling

Two main modelling approaches can be found to model masonry: micro—modelling and macro-modelling approaches. Micro-modelling addresses the behaviour at the level of individual masonry units and mortar joints, while macro-modelling treats the masonry as a homogeneous composite material (see Figure 3.11). First finite element studies in micro-modelling treated units and joints as a homogeneous anisotropic continuum, only to later represent joints as interface elements and thus model masonry as elastic blocks with fracture at the joints (see Figure 3.12) [179], [180].

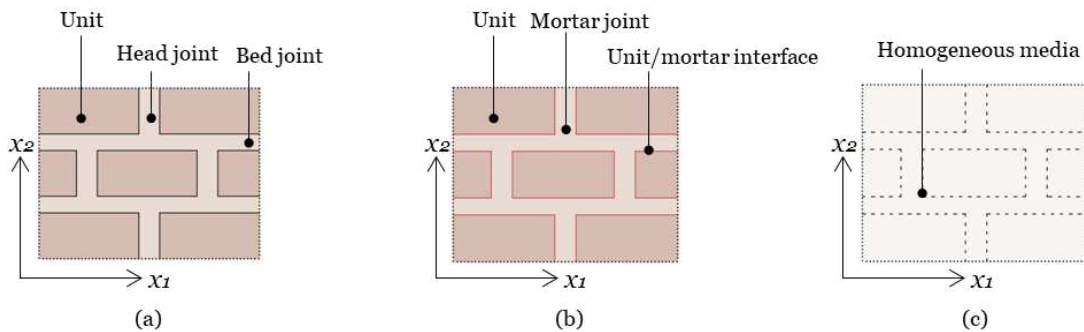


Figure 3.11: Masonry modelling approaches, (a) masonry sample, (b) detailed micro-modelling, and (c) macro-modelling.

Finite element and discrete element methods model approach have been developed to describe the nonlinear behaviour of masonry by representing the masonry system as a collection of discrete bodies, able to move and deform independently. These approaches automatically detect and update inter-element contacts during simulations, allowing for large displacements, rotations, and complete detachment of the blocks. The micro-modelling approach is here adopted to describe linear and non-linear behaviour of the

unit, mortar and unit-mortar interaction, requiring for a high computational effort [181], [182], [183], [184].

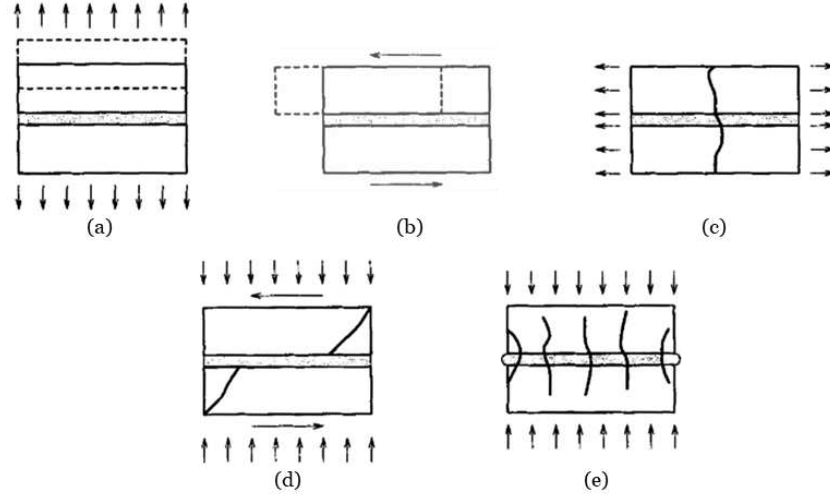


Figure 3.12: Failure mechanism in masonry from [180]: (a) Joint tension cracking, (b) Joint slip, (c) Unit direct tension, (d) diagonal tension crack, and (e) masonry crushing.

On the other hand, macro-modelling treats masonry as a continuous material equivalent, with the continuous equivalent obtained via homogenisation approaches. This approach proves to be an effective alternative to detail micro-modelling strategies, it reduces the complexity that characterises the micro-modelling approach, making it especially advantageous method for large-scale simulations and resource-efficient simulations [185], [186], [187], [188].

The Cauchy continuous model is a well-established framework in the literature that integrates both the linear and non-linear behaviours of constituent materials (unit and mortar) through periodic homogenization techniques [187], [189], [190], [191], [192], [193].

3.3.2 Homogenised models and Identification of the Representative Element Volume (REV)

Homogenization methodologies in masonry aim to establish equivalent macroscopic constitutive relations from the heterogeneous microstructure of brick units and mortar joints.

The early analytical approaches by Pande [194] and Pietruszczak and Niu [195] implemented two-step procedures to derive homogeneous material properties.

Pande (1989): uses a two-step process beginning with stacked brick-mortar systems (bed joints only), later extended to include head joints in 2D and 3D, yielding anisotropic properties with multiple elastic constants (see Figure 3.13).

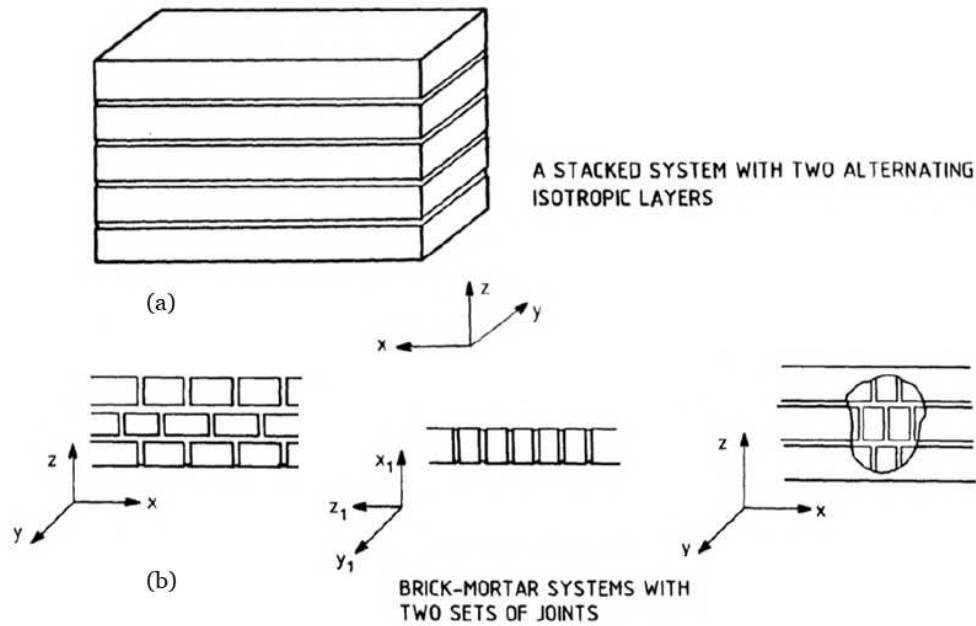


Figure 3.13: (a) stacked masonry system with two alternating isotropic materials, (b) unit-mortar systems with two sets of joint from [194].

Pietruszczak and Niu (1992): Also employ a two-step procedure but first homogenize the brick matrix with head joints (as dispersed weak inclusions) before incorporating bed joints, highlighting directional dependency and differing material behaviours (see Figure 3.14).

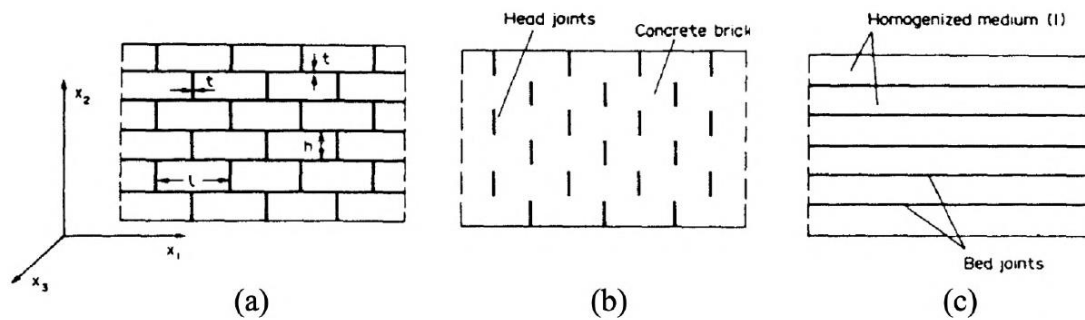


Figure 3.14: (a) geometry of a masonry panel, (b) medium with head joints, and (c) medium interrupted by bed joints [195].

However, these methods are limited by their lack of accounting for the masonry arrangement (or texture), their simplified geometric assumptions and sensitivity to the

order of homogenization. By adopting these procedures for different masonry pattern the same equivalent media will be derived.

To overcome these limitations, Anthoine in [196] proposed a periodic media homogenisation approach by identifying a representative elementary cell. By doing so, the texture of the masonry analysed becomes a variable of the homogenisation problem to derive the macroscopic mechanical properties.

In the work De Buhan and De Felice (1997) of a continuum model for masonry failure prediction within a yield design framework is introduced. The approach begins by formulating a macroscopic strength criterion that is expressed in terms of the material's kinematics. Specifically, the method involves quantifying velocity jumps across joints or characterizing the rigid block velocity field. This kinematic description is then used to derive an equivalent medium that effectively represents the composite behaviour of the masonry. Subsequently, the methodology is extended to determine the overall elastic equivalent properties for masonry in De Felice (2001) [197].

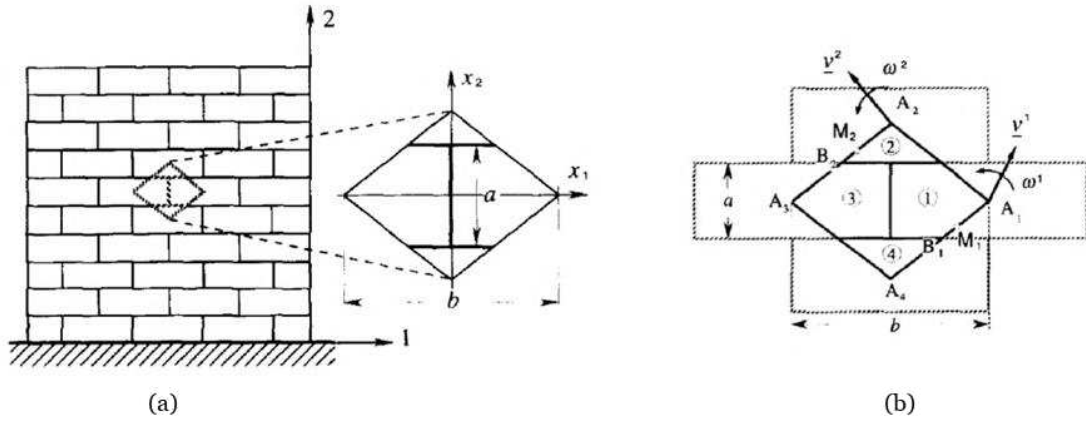


Figure 3.15 Homogenisation process from [187], (a) masonry panel geometry with identification of the representative cell, (b) rigid block kinematics of the masonry unit cell.

Later study of Cecchi and Di Marco [198] analyse the masonry structures using an asymptotic homogenization approach for finite element discretization. They developed two distinct homogenized models: the first assumes that both blocks and mortar exhibit linear elastic behaviour, while the second considers elastic mortar joints with infinitely stiff blocks. This latter assumption, involving rigid brick units, forms the basis for simplified structural modelling in the non-linear domain, where the overall response is governed by the non-linear constitutive law of the mortar joints.

Cecchi and Sab [189], [199] introduce an alternative finite element discretization method by employing multi-parameter homogenization models that utilize the Love–Kirchhoff

plate model to capture both in-plane and out-of-plane behaviours of periodic brickwork. In the first study, they incorporate three perturbation parameters—scale, elastic modulus ratio, and mortar joint thickness—to comprehensively represent a variety of masonry typologies. Cecchi and Sab (2002b) further extends the methodology to explicitly account for out-of-plane actions, ultimately leading to the derivation of a constitutive model for masonry based on Cauchy continuum framework (see Figure 3.16).

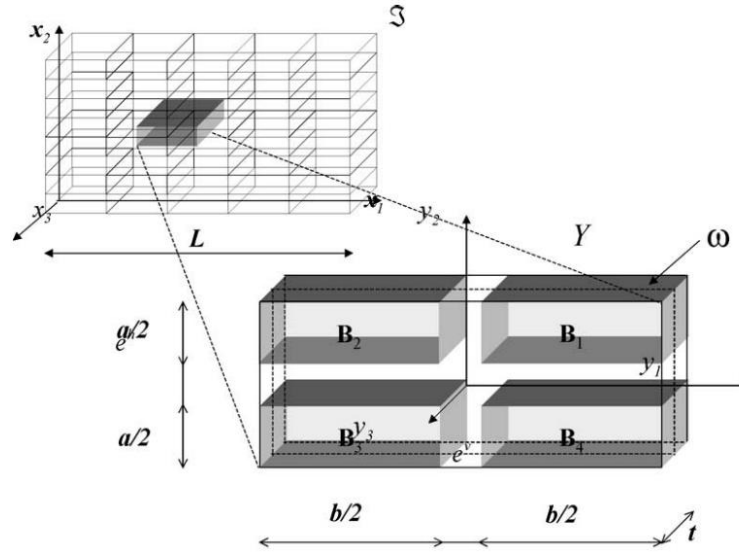


Figure 3.16 Identification of 3D cell geometry for out-of-plane model, stack-bond case [199].

Representative Element Volume - REV

The Representative Elementary Volume (REV) is a fundamental concept in the homogenization approach, serving as a small, statistically representative sample of the heterogeneous material. The REV captures all the essential features of the material microstructure, such as the geometry and properties of the constituent phases (e.g., bricks and mortar in masonry).

How to identify the cell:

1. Definition: The REV is a small, carefully chosen volume of a heterogeneous material that represents the behaviour of the entire periodic medium. It captures both the microstructure and the macroscopic mechanical behaviour of the material. Different REVs can be identified for the same masonry pattern; no single unit cell is exclusively valid (see Figure 3.17).

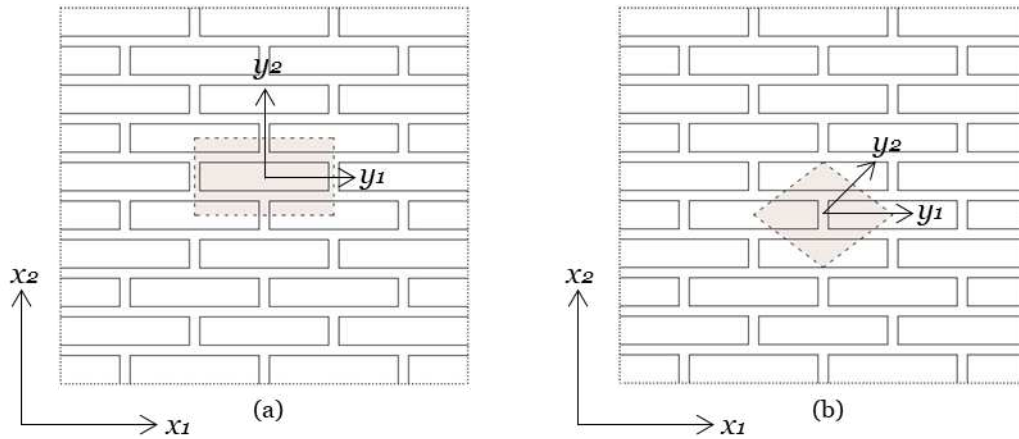


Figure 3.17 Running bond masonry pattern, definition of REV geometry in respect of y_1 and y_2 axis.

2. Characteristics:

- Heterogeneity Representation: The REV includes the different phases of the material, such as bricks and mortar in masonry, and their geometric arrangement.
- Size: The size of the RVE must be large enough to capture the statistical properties of the microstructure but small enough to be considered a point in the continuum sense on the macroscopic scale.
- Boundary Conditions: Proper boundary conditions must be applied to the REV during numerical simulations to ensure that the macroscopic response is accurately captured.

3. Mathematical and Computational Representation:

- Geometry: The REV geometry should include detailed features of the microstructure. For masonry, it consists of a minimum number of unit and joints so that shape and arrangement of bricks and the distribution of mortar joints at a macroscopic scale, are effectively represented.
- Material Properties: The material properties of each phase within the REV must be defined (e.g., elastic modulus, Poisson's ratio, and strength properties for both bricks and mortar).
- Numerical Methods: Finite element (FE) analysis or other numerical methods are typically used to simulate the behaviour of the REV under different loading conditions.

4. Homogenised material properties:

- Elastic Moduli and Poisson Ratios: The homogenization process provides effective macroscopic properties, including homogenized elastic moduli and Poisson ratios, which characterize the overall mechanical behaviour of the material.
- Anisotropy Considerations: In materials like masonry, these properties are often anisotropic or orthotropic, meaning they vary in different directions due to the heterogeneous nature of the microstructure.

In the context of masonry, the REV typically consists of one or more bricks surrounded by mortar joints. The arrangement should reflect the typical pattern found in the masonry wall (e.g., running bond or stack bond). The boundary conditions applied might include periodic boundary conditions to simulate an infinite array of bricks and mortar. By analysing the REV, it is possible to simulate the behaviour of materials at a larger scale, without having to delve into micro-analysis and the computational costs it requires.

Within this framework, the primary objective is to determine the effective mechanical properties of periodic media or a continuum by accounting for their microstructural features, including the geometric and mechanical characteristics of their constituent materials. This is accomplished by defining a Representative Elementary Volume (REV), which captures both local heterogeneity and global mechanical behaviour, enabling accurate yet computationally efficient modelling. By analysing the REV, the macroscopic stress-strain relationships of the material can be derived, reducing the complexity of full-scale simulations while preserving physical accuracy.

A fundamental aspect of this approach is stress and strain periodicity, which ensures that the microstructural response correctly translates into the macroscopic behaviour [186], [189], [196], [198].

A material is considered macroscopically homogeneous if all unit cells experience identical loading conditions. However, boundary effects must be considered, as cells positioned at the specimen edges undergo different stress distributions than those at its centre. To maintain the integrity of numerical and theoretical models, it is crucial that adjacent cells remain compatible even under deformation, ensuring that no artificial discontinuities arise.

This requirement imposes compatibility conditions on the stress and strain fields at the REV boundaries. According to Anthoine (1995) and Cecchi & Sab (2002a), these conditions are expressed as

$\boldsymbol{\sigma} \cdot \mathbf{n}$ is anti-periodic on ∂S

u^{per} is antiperiodic on ∂S

where $\boldsymbol{\sigma}$ is the stress field, $\mathbf{n}$ the outward normal, u^{per} is the strain field, and δS define the boundary condition of the cell [189], [196].

Anti-periodicity of the stress vector $\boldsymbol{\sigma} \cdot \mathbf{n}$ ensures that opposite boundary stresses are equal and opposite, maintaining global equilibrium. Meanwhile, periodicity of the strain field u^{per} guarantees compatibility, preventing artificial gaps or overlaps in the deformation field.

Furthermore, the effective mechanical properties of the REV are determined by integrating micro-stresses and micro-strains over the unit cell area (S), as defined by:

$$\langle \varepsilon_{\alpha\beta}(\mathbf{u}) \rangle = \frac{1}{|S|} \int_S \varepsilon_{\alpha\beta}(\mathbf{u}) \cdot dS$$

$$\langle \sigma_{\alpha\beta} \rangle = \frac{1}{|S|} \int_S \sigma_{\alpha\beta} \cdot dS$$

where $\langle \sigma_{\alpha\beta} \rangle$ and $\langle \varepsilon_{\alpha\beta}(\mathbf{u}) \rangle$ represent macroscopic stress and strain fields, while $\sigma_{\alpha\beta}$ and $\varepsilon_{\alpha\beta}(\mathbf{u})$ describe their microscopic counterparts. These formulations enable the accurate prediction of macroscopic responses under external loading, ensuring that local heterogeneities are properly accounted for.

By employing the REV approach, the micromechanical behaviour of periodic media such as masonry can be systematically incorporated into macro-scale models. This method strikes a balance between computational efficiency and physical fidelity, making it an essential tool for studying complex materials under various loading conditions while ensuring reliable structural predictions.

Chapter 4 - Stabilized Compressed Earth Block Reinforced Masonry: A Construction Technique

This chapter delves into the description of an innovative construction system currently being refined by Ditta Felino, an Italo-Senegalese construction company. The system consists of earthen masonry built using concrete-stabilised compressed earth blocks (CEBs), paired with an earth-based mortar. At its current stage of development, the masonry is reinforced with a concrete frame cast integrally within the blocks. This research explores an alternative approach to this system, aimed at reducing the use of concrete and steel in the construction of the reinforcement frame, while maintaining an adequate structural performance.

4.1 Construction System

This chapter introduces an innovative construction system for earthen masonry walls, currently under development in Senegal. The system features reinforced earth masonry panels constructed using concrete-stabilized compressed earth blocks (CEBs) bonded with earth-based mortar. CEBs mark a significant evolution from traditional adobe blocks, undergoing a compaction process that enhances their mechanical performance and physical properties. In this case study, stabilisation is achieved via hydraulic press (mechanical stabilisation process) and by incorporating cement into the mix (chemical stabilisation process). The combined stabilisation process further improves the blocks resistance to stress and their overall durability. The blocks are arranged in a header bond masonry pattern, joint together with an earth-based mortar. To address the inherent limitations of earth masonry, particularly its poor resistance to horizontal loads and lateral deformations, the system integrates a minimal concrete frame cast within the masonry. This reinforcement significantly enhances the structural integrity of the system, making it more suitable for seismic zones and enabling the construction of taller buildings.

The CEBs are produced using a mixture of sieved red earth, fine sand, and cement in a 12:6:1 ratio. The dry mix is combined with water to achieve a humid state, ensuring proper soil particle cohesion (reacting with silt and clay components) and optimal cement hydration. Care is taken to avoid excessive watering, which would cause the mix to enter a plastic state, and therefore resulting in excessive shrinkage, cracking during

the drying phase and a general poor compaction level resulting in weaker blocks. In addition cement will underperform in an overly wet mix, making the chemical stabilisation process ineffective.

The earth mix is compacted with a mechanical hydraulic press which acts via geometrical stop (see Figure 4.1a). The process is semi-automatised: the mould is manually filled with the earth mix until it is reached 150 mm height, then the press is activated and compacts the earth mix to get 90 mm block high (roughly 3 MPa of compaction stress is reached). This process ensures consistent block dimensions (305 × 200 × 90 mm) and a mean density of $\langle \rho \rangle = 1840 \frac{kg}{m^3}$. However, alterations in the filling process can lead to inconsistencies in block properties, such as increased volume/void ratio and decreased density. Once pressed, the blocks are cured for 28 days to allow the cement to achieve full strength.

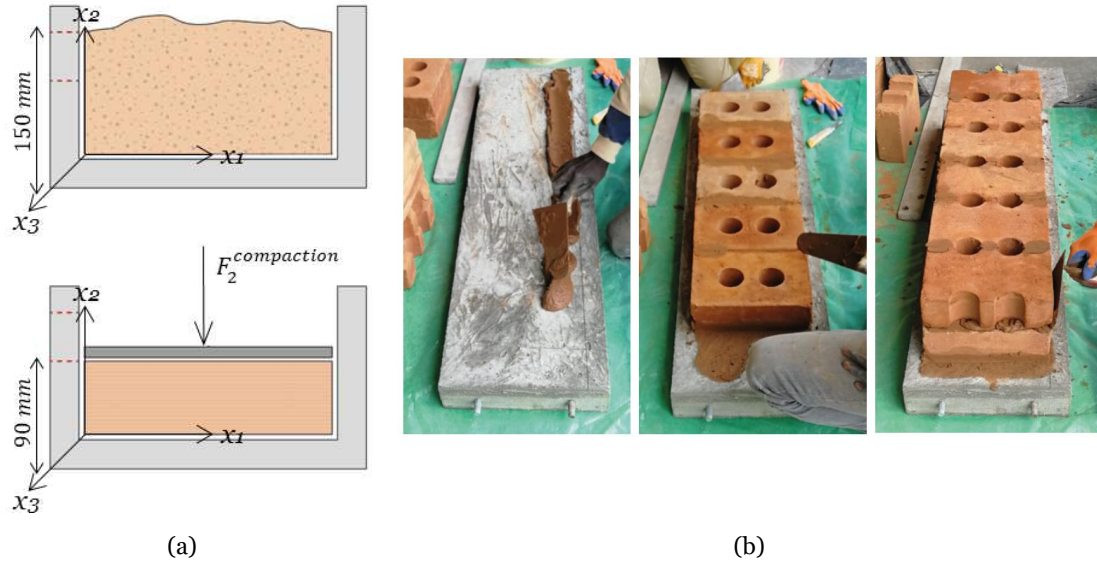


Figure 4.1: CEB (a) compaction process and (b) masonry arrangement.

To integrate a concrete reinforcement while preserving the masonry visual and structural pattern, three specific block geometries have been designed (see Figure 4.2):

- Block-1 features two circular holes (diameter equal to 70 mm) along its length, allowing for vertical reinforcement to be casted.
- Block-2 includes lateral semi-circles to accommodate vertical reinforcement.
- Block-3 serves as a channel, acting as permanent formwork for the reinforced beam frame (beam height $h^{beam} = 50 \text{ mm}$).

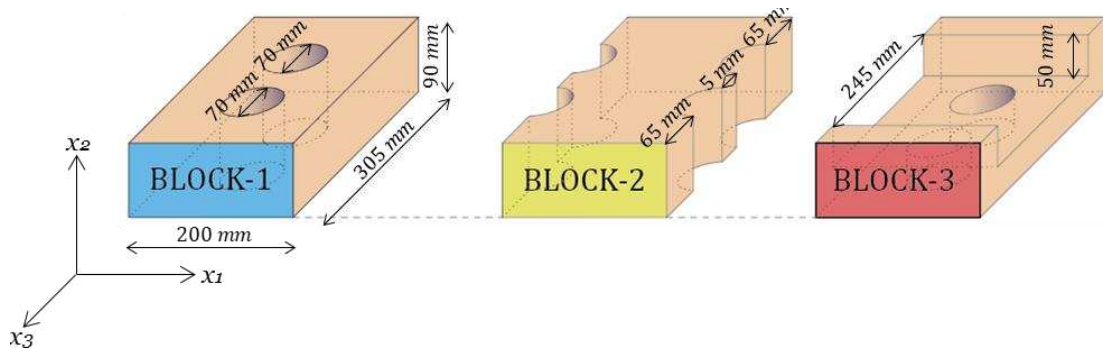


Figure 4.2: SCEB typologies geometrical configuration.

The blocks as designed enable the construction of both reinforced and unreinforced masonry walls. Blocks-1 and Block-2 alone are suitable for the construction of the unreinforced systems. In the reinforced configuration, the specific geometry of Blocks-1 and Block-2 enables vertical reinforcement without disrupting the masonry header-bond pattern, while Block-3 is specifically required for reinforced masonry system (see Figure 4.2).

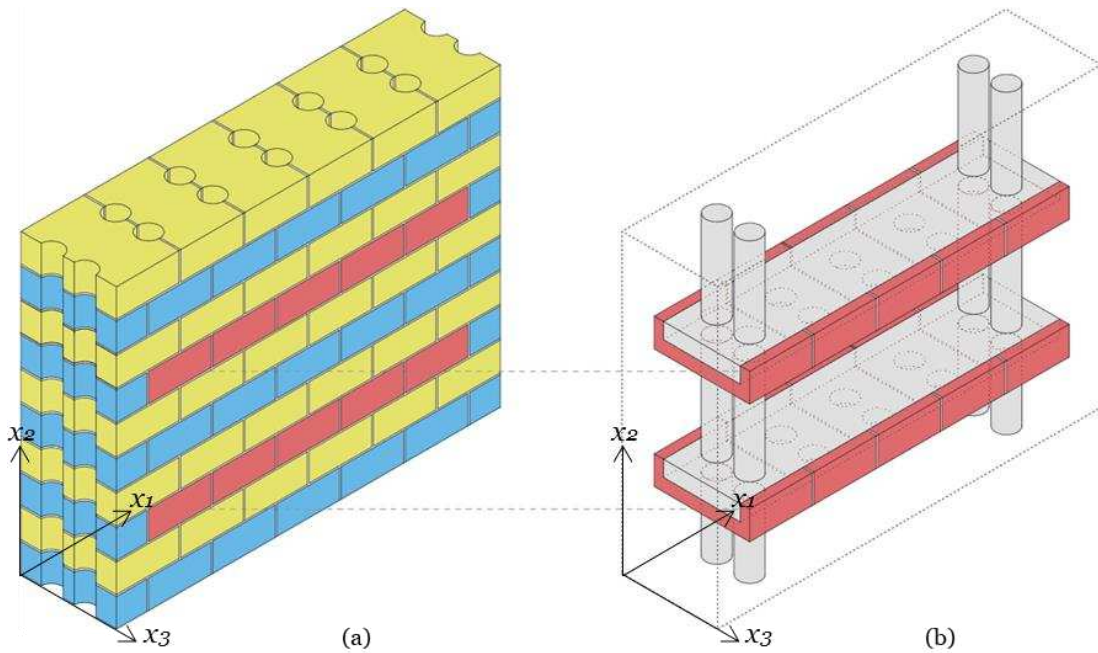


Figure 4.3: CEB masonry panel (a) masonry texture and (b) concrete reinforcement frame, masonry/frame proportions are indicative of the masonry panels realised for the experimental campaign in Chapter 5.

The frame comprises a vertical reinforcement element, consisting of two coupled slender 70 mm-diameter cylinders, cast within Blocks-1 and Block-2 and a horizontal concrete beam with a cross section of 50 × 245 mm (see Figure 4.3b). Block-3 typology serves as a permanent formwork for the concrete beam (see Figure 4.4). The masonry panels are

bonded with earth-based mortar joint (thickness $e = 5\text{mm}$), composed of sieved red earth, sand, and cement in a 3:6:1 ratio.



Figure 4.4: Construction process of concrete frame reinforced masonry panel.

In the proposed system, the earthen masonry primarily resists vertical loads through compression, while the reinforced frame, here composed of cast-in-place concrete, is designed to absorb horizontal actions, such as wind or seismic loads. This composite load-bearing mechanism reflects a structural logic already seen in traditional systems for example the “muratura borbonica”, where timber frames were used to provide lateral strength and ductility to weaker masonry infills [200], [201], [202]. In this case, concrete was adopted mainly for practical construction reasons, enabling the reinforcement frame to be cast directly within specially moulded CEB units. However, this material pairing introduces potential challenges in terms of mechanical compatibility and long-term durability. The significant difference in stiffness between concrete and earth may lead to stress concentrations and cracking at the interface, especially under lateral loading. Moreover, the combination of earth and cementitious materials raises concerns about moisture movement, shrinkage, and degradation over time, particularly in the absence of protective treatments. These aspects must be carefully considered in both design and future maintenance strategies.

4.2 Optimisation of the Construction System – Introduction of Fibre Reinforced Polymers (FRP)

The construction system incorporates a material with high stiffness – concrete - to reinforce a much weaker material - earthen masonry.

From a mechanical perspective, this introduces significant incompatibility. Concrete and earthen masonry have markedly different stiffness values, that can lead to stress concentrations and potential failure at the interface under load. This difference in stiffness can result in poor load transfer between the materials, compromising the overall performance of the system. Despite the blocks being stabilized with cement, only 8% cement is added to the mix - a minimal quantity aimed at maintaining the sustainability of the earth material while improving their mechanical performance and durability. However, the extensive use of concrete within the reinforcement frame negates the sustainability benefits of using earthen masonry due to the high carbon emissions associated with concrete production. Additionally, steel is used to reinforce both the concrete used in the vertical elements of the frame and the horizontal concrete beams, further contributing to the system high emissions and ecological impact. Steel, characterized by high stiffness and high embodied emissions, exacerbates the mechanical incompatibility and undermines the ecological and sustainable advantages of the construction system.

For these reasons, this research explores alternatives to the concrete reinforcement:

Horizontal reinforcement element

Horizontal reinforcement has been completely replaced with BFRP (Basalt Fiber Reinforced Polymer) nets, which are embedded within the mortar joints (see Figure 4.5b). This approach eliminates the need for concrete and steel in horizontal reinforcement, reducing the total concrete usage by approximately 60%. Furthermore, it allows for the exclusion of Block-3, a specific block type with inherent fragility due to its geometry. The removal of this block type optimises the production process and facilitates greater flexibility in the placement of horizontal reinforcement within the masonry system (see Figure 4.6). This also simplifies the construction process, as it eliminates the careful positioning required for Block-3 and significantly accelerates the building timeline by removing the need to prepare, mix, and pour concrete for the horizontal reinforcement.

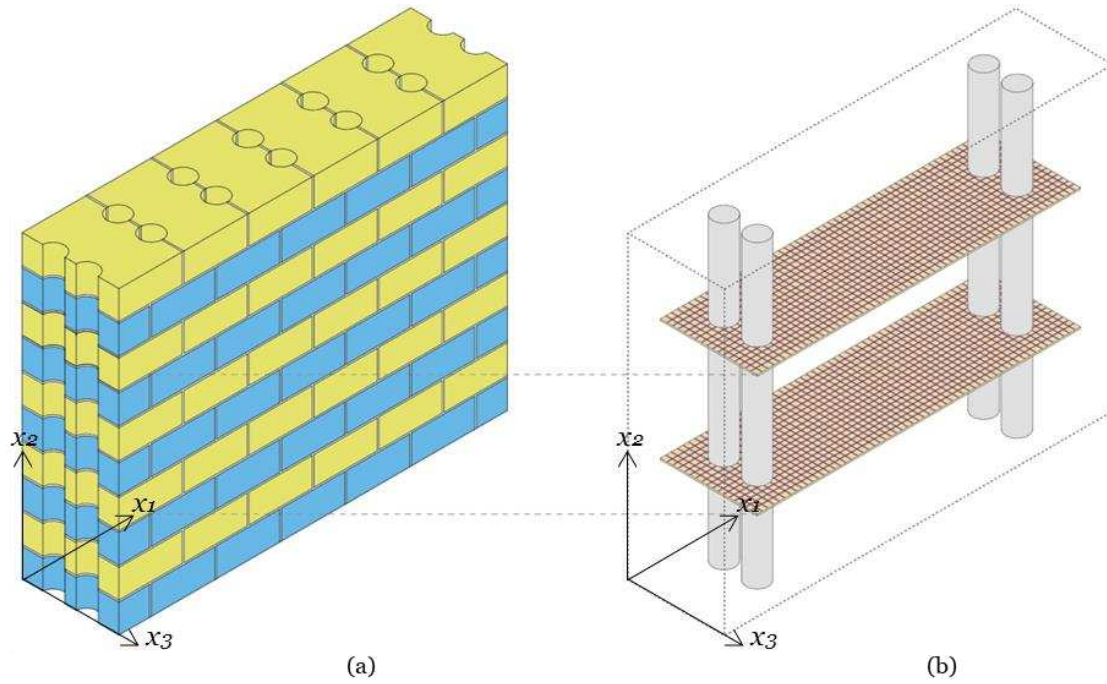


Figure 4.5: CEB masonry panel (a) masonry texture and (b) mixed reinforcement frame, masonry/frame proportions are indicative of the masonry panels realised for the experimental campaign in Chapter 5.



Figure 4.6: Positioning of the BFRP reinforcement net within the bed mortar joint.

Vertical reinforcement element

For vertical reinforcement, the transition was more incremental. Steel rebar was replaced with GFRP (Glass Fiber Reinforced Polymer) bars. This material was selected for its lower mechanical properties compared to steel, which makes it more compatible with the mechanical characteristics of the earthen masonry. However, a challenge arises when integrating GFRP bars with a cementitious matrix. Unlike steel, GFRP bars rely on the bond between glass fibres and a polymer resin matrix for structural integrity. GFRP

bars are specifically engineered for use in cementitious materials, and research has been conducted to evaluate their bond behaviour, durability, and load-bearing capacity. While their bond mechanisms differ from steel (they rely more on surface texture and chemical adhesion with concrete), their performance is largely investigated. This separates them from steel rebars, which naturally bond well with concrete through mechanical interlock and chemical adhesion. A detailed study of the adhesion properties of GFRP bars within a cement matrix was therefore necessary to ensure their effectiveness in the system (see Figure 4.7).



Figure 4.7: Preparation of GFRP rebars for pull-out test.

To evaluate the effectiveness and compatibility of each reinforcement element, a stepwise approach was adopted. Initially, specimens were produced using horizontal reinforcements with BFRP nets, while maintaining the existing vertical reinforcement system. This allowed for targeted assessment of the impact of BFRP nets on the mechanical behaviour of the masonry. Prior to integrating GFRP bars into the system, a comprehensive adhesion study was conducted to assess the performance of GFRP elements within a cementitious matrix. This step was critical to confirm their feasibility and optimize their application in the reinforced masonry system.

Chapter 5 – Mechanical Characterisation of Earthen Masonry

Masonry is a complex composite material that exhibits varied behaviour under external forces, depending on the properties of its constituent elements—namely, the unit and mortar—and the manner in which these materials are arranged to form the masonry system. To fully understand how a specific masonry system reacts to different loads, it is essential to analyse its unique composition. When earth-based masonry is used, both the unit and mortar mix are composed of earth, a material not defined by any common standard due to its inherent variability. Soil properties can differ based on the excavation site, and variations in the percentage of water and air—factors influenced by the climatic conditions at the building site—further complicate the definition of earth in construction practices. Consequently, earth in construction is not to be considered as a single material but rather a range of materials composed of soil, water, and air, in different proportions. The stabilization process, which also lacks standardization and includes various methods, introduces additional uncertainties in defining earthen products. Different mixes result in materials with distinct mechanical properties, even within the same earthen technique. For example, in the case of compressed earth blocks, variations in earth mix, stabilization processes, and press capacities lead to a unique material each time. Therefore, it is necessary to first investigate the mechanical properties at the material constituent level before moving on to the construction system level (masonry). The experimental campaign was conducted with this approach in mind, adapting existing standards from the closest standardized materials to the earthen products used in this research, as well as drawing from scientific publications on similar topics.

In the chapter that follows, a Euclidian 3D system is used where x_1 is the horizontal direction, x_2 is the vertical direction and x_3 is the depth.

5.1 Characterisation of Constituent Materials

This section explores the mechanical properties of the constituent material of earthen masonry: Earth Blocks and Earth Mortar. It is essential for anticipating the behaviour of a composite material (masonry) in both experimental campaign and numerical modelling. Sourced from Senegalese soil (Dakar area) and produced in a semi-artisanal way, these construction materials possess distinct characteristics. Given the singularity

of these elements, it is necessary to investigate their key mechanical properties such as compressive strength, elastic modulus, tensile strength. The following sections presents the mechanical characterisation of block unit (SCEB) and mortar joint.

5.1.1 Stabilised Compressed Earth Block (SCEB)

The earth block was subjected to compression and bending tests to evaluate its structural properties. Both cubic and prismatic specimens were extracted from the original block for these tests. The characteristics and testing conditions of the specimens are as follows:

- mean density: $\rho^{SCEB} = 1840 \text{ kg/m}^3$ (coefficient of variation COV = 0.04)
- specimen sizes:
 - cubic specimens $90 \times 90 \times 90 \text{ mm}^3$
 - prismatic specimen: $90 \times 90 \times 270 \text{ mm}^3$

The dimensions of the specimens were constrained by the size of the original brick itself (see Chapter 4 for further details).

Compression test

In order to determine compressive strength f_c and Elastic Modulus E of SCEB, (in the hypothesis of isotropic material) the response of the blocks under uniaxial unconfined compression loads was analysed.

Being earth a non-standardised material there is no common regulation of how to experimentally investigate these materials. Different specimen dimensions and sizes are explored in various research. A largely adopted method to study the compressive strength of earth blocks involves the use of the entire block. This can lead to an altered (usually higher) results because of confinement effect. The confinement effect refers to the phenomenon where the compressive strength of a material appears higher when tested within a constrained environment. In earth blocks, lateral confinement during testing restricts lateral expansion, leading to an apparent increase in compressive strength. This can result in an overestimation of the block actual compressive strength. Other research indicates the use of prismatic specimens or stacked halves with a mortar bed joint (for further details see Paragraph 2.1). However, the presence of a mortar joint can lead to altered data, as it is more reflective of a composite material than the unit itself.

For earth-based materials, cube-shaped specimens can be used to investigate compressive strength. In the present study, the earthen material is stabilized with 8% cement. Consequently, concrete testing procedures were also taken into consideration.

According to UNI EN 12390-1 [151], specific guidelines are provided for the geometry of specimens used in testing the compressive strength of concrete. The standard covers cubic, cylindrical, and core-extracted specimens. Cubic specimens are commonly adopted in concrete testing due to their capacity of allowing for a more uniform distribution of stresses across the specimen, resulting in more consistent and reliable data. Although some confinement effects may be present, these standardized procedures account for them, ensuring the accuracy of the results. For cubic specimens, the accepted dimensions are 100, 150, 200, and 300 mm, depending on the granulometric curve of the concrete. Similar considerations are made for cylindrical specimens. Additionally, UNI EN 12390-3 [152] offers evaluations on different crack patterns and failure typologies based on the shape of the specimen. In this research, instead of moulding specimens specifically as recommended by UNI EN 12390-1, cubes were cut from the original blocks to avoid the confinement effects that can occur due to compaction pressure in smaller moulds. The dimensions of the cubes also differ from the standard due to the smaller size of the blocks, which only allowed for 90 x 90 x 90 mm cubes of cement-stabilized compressed earth blocks (CEB). The crack patterns and failure mechanisms described in UNI EN 12390-3 were then used as a reference to understand the failure mechanisms of these cubes.

In this study, 13 cubic specimens, cut from the original block, were tested (see [96], [203]), no specimen confinement or capping was adopted to assure reliability of data.

Standard UNI EN 12390-3 procedure for evaluating compressive strength was adopted for SCEB specimen. The specimens were subjected to controlled displacement loading, mapping out the full load progression, at a velocity of 0.5 mm/min. The applied force F was recorded by the actuator. Strain was recorded with a 30-mm strain gauge. Given the high heterogeneity of the material, 3 strain gauges were fixed to register overall deformation. Two parallel to the loading direction (vertical) at opposite cube faces, and one perpendicular to the loading direction (horizontal) so that potential changes in strain could be registered. Strain gauges were arranged according to Configuration A and Configuration B setups. This test configuration was used on total of 10 specimens (see Figure 5.1).

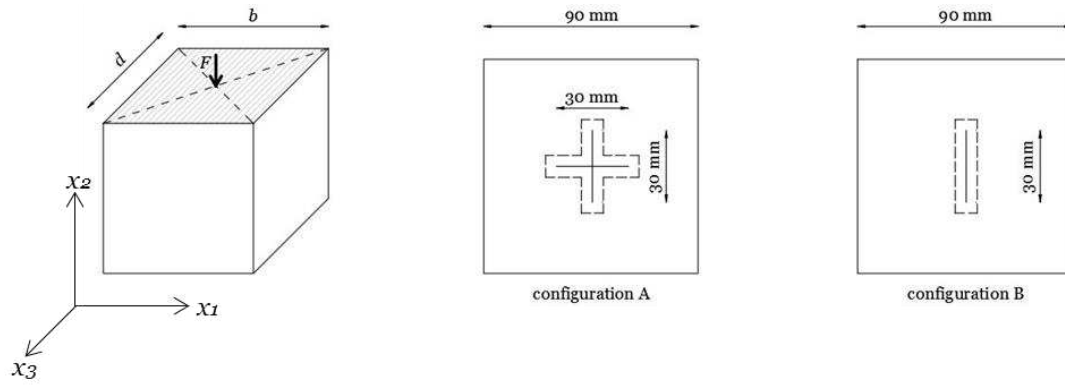


Figure 5.1: Compression test set-up and strain gauges configurations for cubic specimens.

Three additional specimens were tested under the same conditions as previously described. The observation of the specimen surface, where 30-mm long strain gauges were initially applied, revealed elements up to 10-15 mm in diameter. Since these larger elements were randomly distributed and it was not always possible to avoid placing the instrumentation over them, larger 60-mm strain gauges were applied. This adjustment was made to consider a larger portion of the specimen, aiming to balance out the aggregate size of the mixture and to overcome potential data alterations caused by the dimensional relationship between the granularity of the material and the instrumentation. No significant differences were noted on strain recording (see Figure 5.2 and Figure 5.8).

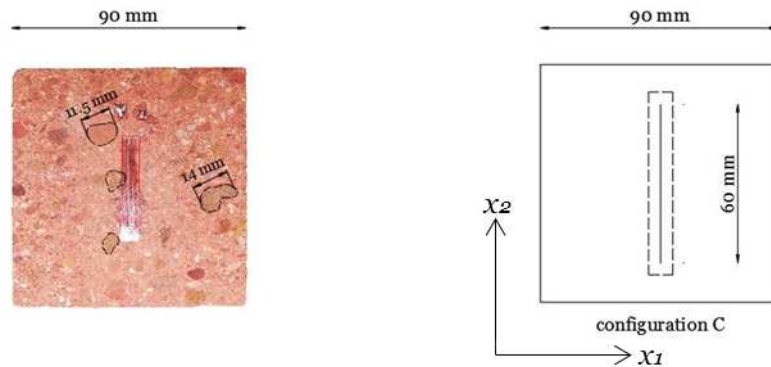


Figure 5.2: 60-mm-long strain gauges configuration for cubic specimens.

UNI EN 12390-3 crack pattern and failure type were considered to analyse earth specimen and understand the mechanical behaviour. The crack pattern observed in the specimens typically forms a cone shape. This pattern can appear in the top or bottom regions of the specimen, where the top region is defined as the area near the load application surface, and the bottom region is the opposite end. In some cases, this cone-shaped cracking can develop in both regions simultaneously, resulting in an X-shaped

crack pattern, similar to those often observed in concrete cube specimens under compressive loading (see Figure 5.3). It is an indication of complex stress distribution, not uniformly distributed. X-shaped cracks are usually formed due to combination of vertical compressive stress and lateral tensile stress. Vertical load causes the material to expand laterally, creating tensile stresses perpendicular to the applied load. These tensile stresses are responsible for the diagonal crack lines that intersect, forming the X-shape. The failure mode observed throughout the specimens was a crushing failure typical of brittle materials, where the material compacts and disintegrates at the load application point. It is a typical failure for cone-shaped crack pattern (see Figure 5.4), this occurs because the material is subjected to high compressive forces, causing the material to compress and form fine cracks and crumbling in the affected zone. This failure type is indicated as “satisfying” in the standard.

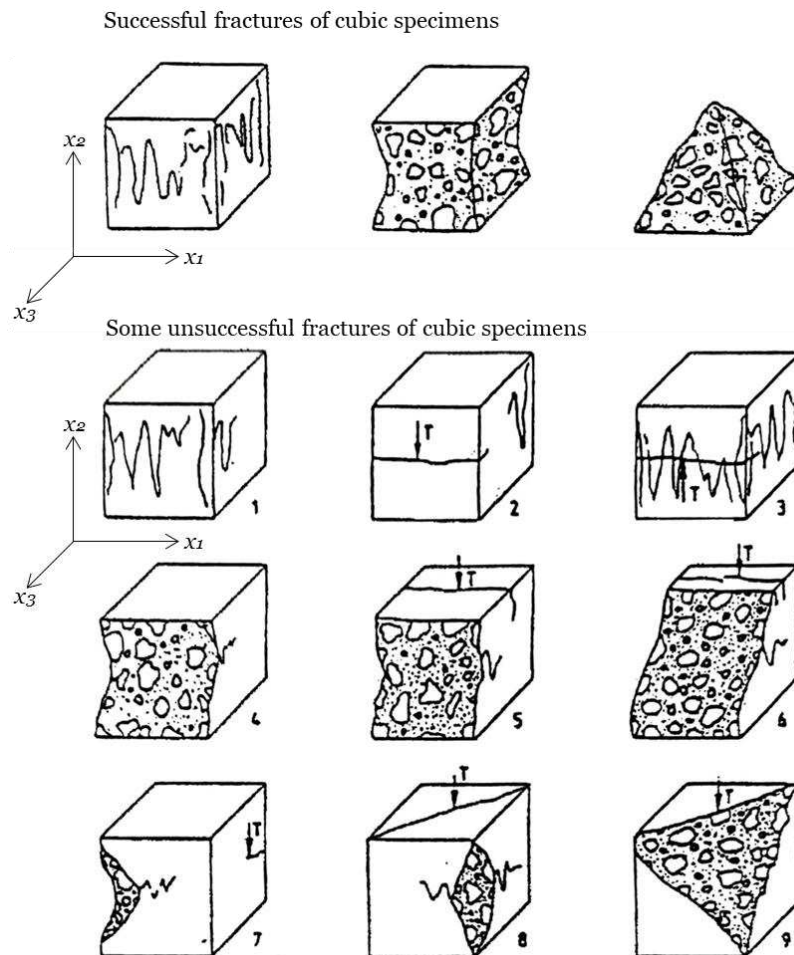


Figure 5.3: Fracture modes of concrete cubes tested in compression, from UNI EN 12390-3 [152].

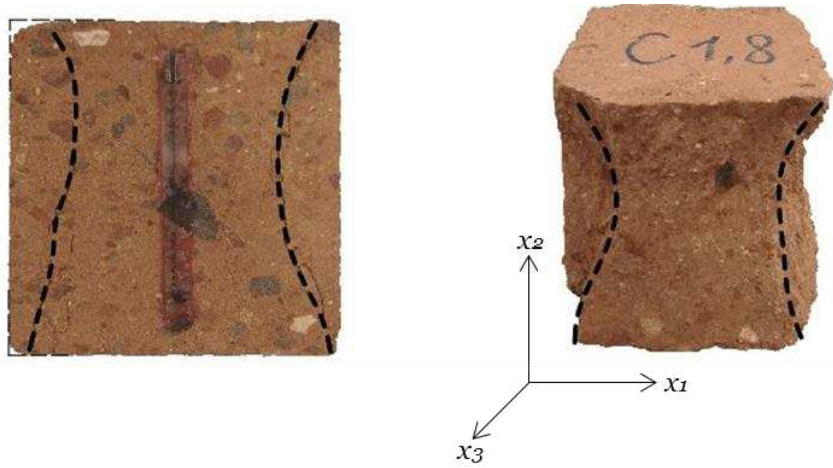


Figure 5.4: Typical failure mode obtained in SCEB cubic specimens tested in compression.

Some specimens experienced failure due to the high tensile strength in the material, which can be commonly observed in brittle material subjected to compressive loads, due to the weakness of the material in tension compared to its performance in compression. This can be observed when vertical cracks start to develop within the specimen surface, resulting in fracturing and eventually splitting (see Figure 5.5). Because of the even distribution of these vertical cracks along the surface of the cubic specimen, absence of crack on the top surface and no visible horizontal crack development this failure mechanism has been identified as the first one of the “satisfying” mechanism in the standard (see Figure 5.3).

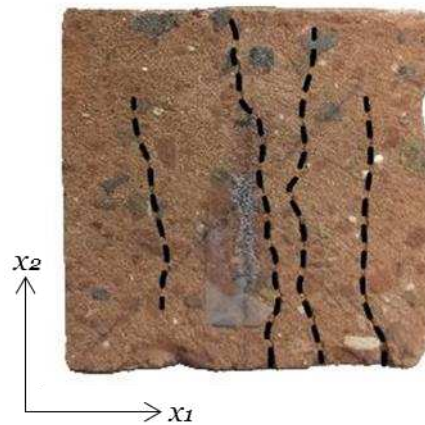


Figure 5.5: Typical failure mode obtained in SCEB cubic specimens tested in compression, due to high tensile stress.

The following equation was used to compute compressive strength f_c . Where F being peak force, expressed in N , b and d width and depth of the specimen at the contact area,

expressed in *mm*. Coefficient of variation was then calculated considering all the tests strength values.

$$f_c = \frac{F}{bd} \quad (1)$$

Specimen were named as C1-*n* and C2-*n*, where the number 1 refer to 30-mm strain gauges and 2 refer to 60-mm strain gauges instrumentation. To ensure the reliability of the data and minimise the impact of potential errors arising from manual production or human errors during the preparation of the tests, a data trimming approach was employed. This method involves the systematic exclusion of data points that fall outside a specified range, defined in relation to the average ($\langle \cdot \rangle$, average operator) and standard deviation (s) of the dataset. First the mean value was calculated to establish a reference point, then the standard deviation was computed. The thresholds (T) values were defined as lower threshold $T^{low} = \langle \cdot \rangle - s$, and upper threshold: $T^{high} = \langle \cdot \rangle + s$. Only the data within the interval described by the two thresholds were considered (see Figure 5.6). Being $\langle f_c \rangle = 5.83$ and $s = 1.12$, the final mean compressive strength was calculated excluding the value of the samples C1-1, C1-5, C1-8, C2-2 and C2-3 (see Table 5.1).

<i>Specimen</i>	<i>Compressive strength</i> [N/mm ²]	<i>Mean compressive strength</i> [N/mm ²]
C1-1	7.78	5.96 c.o.v. = 0.11
C1-2	6.67	
C1-3	6.70	
C1-4	5.31	
C1-5	7.15	
C1-6	5.12	
C1-7	4.95	
C1-8	4.56	
C1-9	6.30	
C1-10	6.54	
C2-1	6.32	
C2-2	4.04	
C2-3	4.37	
*Considering all values: $\langle f_c \rangle = 5.83$; c.o.v. = 0.19		

Table 5.1: Compressive strength determined for SCEB cubic specimens.

Elastic modulus of cement-stabilised CEB, E^{SCEB} , was determined as the initial slope of the stress-strain curve obtained from the compression tests (see Figure 5.8). The stiffness of the material is described by the linear proportion at the initial stage of loading.

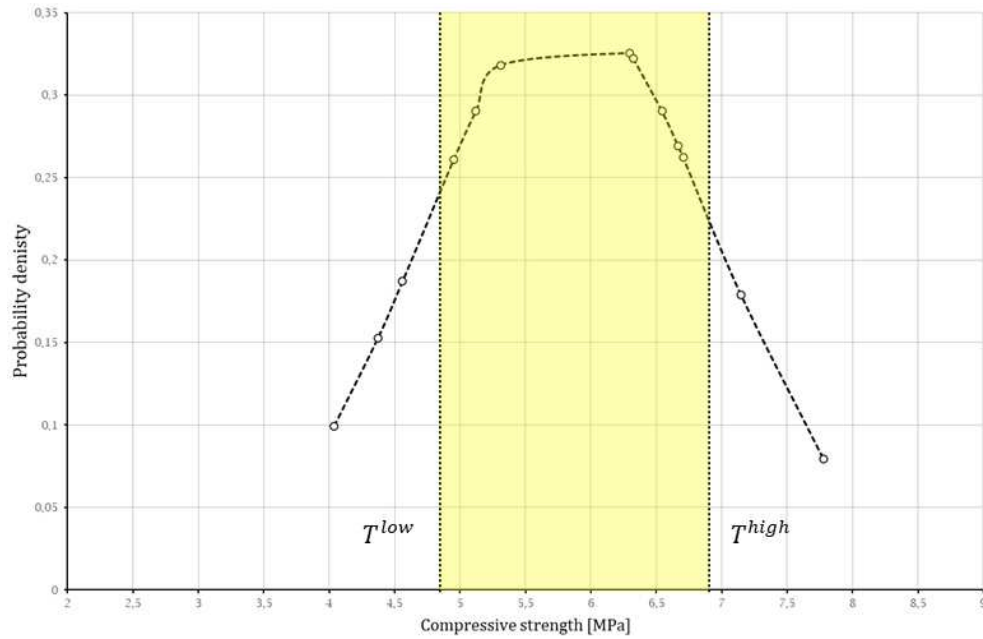


Figure 5.6: Normal distribution of compressive strength values of SCEB cubic specimen.

<i>Specimen</i>	<i>Elastic modulus</i> [N/mm ²]	<i>Mean elastic modulus</i> [N/mm ²]
C1-1	3343	6989 c.o.v. = 0.4
C1-2	9968	
C1-3	3473	
C1-4	5879	
C1-5	14115	
C1-6	7713	
C1-7	6374	
C1-8	10375	
C1-9	3248	
C1-10	28991	
C2-1	7557	
C2-2	2691	
C2-3	11964	
*Considering all values: $\langle E^{SCEB} \rangle = 8900$; c.o.v. = 0.7		

Table 5.2: Elastic modulus E_2 for SCEB cubic specimens.

Regarding compressive strength, the trimming procedure was applied to minimise potential unreliable data. Values considered to calculate the tangent Elastic modulus were defined discarding the upper and lower value of the thresholds (see Figure 5.7). By doing so, C1-5, C1-10 and C2-3 values were excluded to calculate the mean tangent elastic modulus $\langle E^{SCEB} \rangle = 6989 \text{ MPa}$ and $s = 6755$ (see Table 5.2).

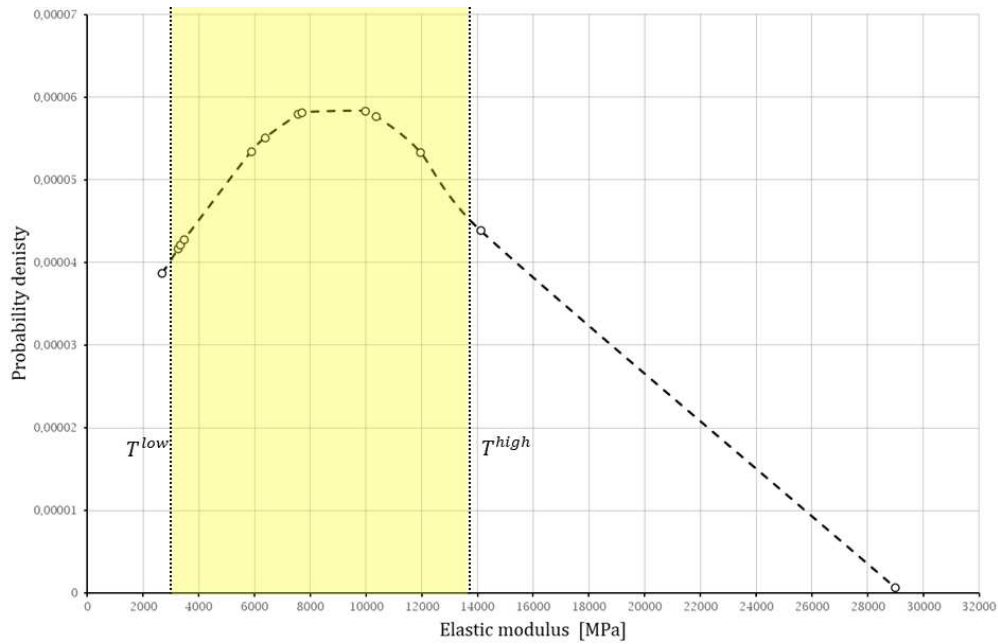


Figure 5.7: Normal distribution of Elastic modulus E^{SCEB} for SCEB cubic specimens.

The plot shows the behaviour of the specimen subjected to compressive load, correlating axial deformations with respective loads. Since two vertical strain gauges were fixed for each specimen an average of the data obtained was employed. The deformation observed is relatively low, which indicates limited deformation when subjected to stresses, therefore a higher stiffness level of the material. Majority of peak stresses occurs at 0.15 to 0.2% strain. The stress level dropped instantly after reaching peak stress value, tighter with an abrupt failure. The near-complete absence of post peak behaviour is an indicator of the brittleness of the cement-stabilised CEB specimens. Some of the samples showed higher peak levels compared to the overall experimental campaign (e.g., C1-2 and C2-2) as well as different maximum strain level (e.g., C1-5 and C1-6). This can be attributed to the high heterogeneity of the material and the semi-artisan way of production. While cement increase the mechanical performance and resistance of the material, the brittleness which is observed can lead to the necessity of the production of an earth-mix which can bear higher deformations levels.

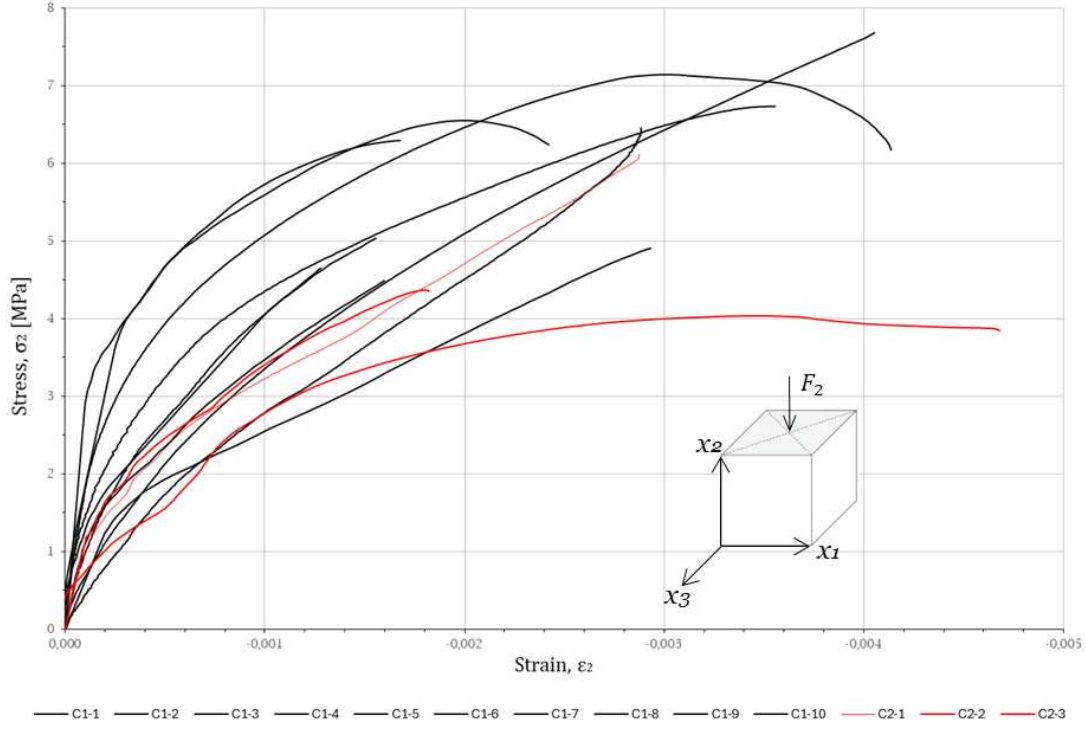


Figure 5.8: Stress-strain plot in compression of SCEB cubic specimens.

Poisson ratio ν , which correlates the horizontal and vertical strain, was determined from these tests. It was calculated by correlating the horizontal strain ε_1 (lateral deformation) and vertical strain ε_2 (axial deformation) observed during the compression tests as follows (the number of the subscript refers to the direction considered, given the 3D Euclidian reference system x_1, x_2, x_3).

$$\nu = - \frac{\varepsilon_2}{\varepsilon_1} \quad (2)$$

The mean value obtained was $\langle \nu \rangle = 0.34$. This gives further indications of the mechanical properties of the material. However, brittle the behaviour in compression, cement-stabilised CEB, Poisson ratio indicates that some degree of lateral expansion can be tolerated.

Bending test

In order to better understand the mechanical behaviour of the earth blocks, bending tests were carried out to assess the flexural strength and fracture of the SCEB. UNI EN 1015-11 [153] was first used as a guide for the evaluation of the flexural strength. Although usually used for ductile materials, the three-point bending test can also being used on brittle materials and soil specimens as a first approach for material characterisation. The three-point bending test is usually employed to determine the flexural properties of materials by applying a concentrated load at the midpoint of a sample supported at both

ends, which creates a bending moment along the specimen length (see Figure 5.9). It is useful for measuring the flexural strength and modulus, particularly for brittle materials. However, the test concentrates tensions at the loading point and supports, which can influence the results. Supports were fixed symmetrically at 230 *mm* distance. Load was applied at the middle section in displacement control at a constant velocity of 0.5 *mm/min*. Three specimens were tested with this configuration B1, B2, B3. Force values were obtained from machine data. According to the standard, flexural strength f_f was then calculated using the following equation, where F_2 is the maximum force applied, expressed in *N*, L is the distance between the support in *mm* and b and d are width and depth of the specimen, in *mm*.

$$f_f = \frac{3F_2L}{2bd^2} \quad (3)$$

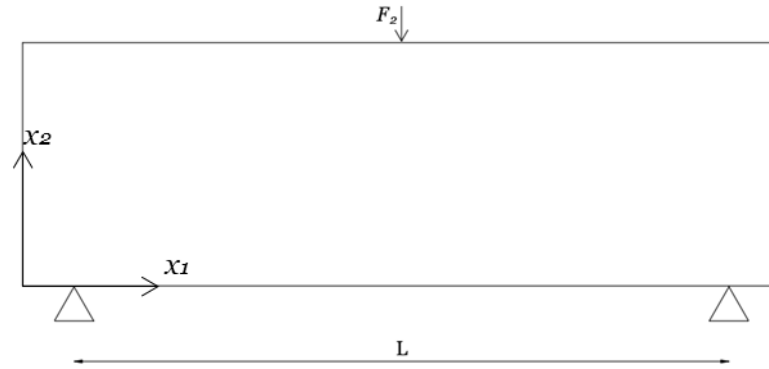


Figure 5.9: Three-point bending test set-up and for prismatic specimens.

As expected, all three specimens exhibited brittle behaviour with associated a low tensile strength, with higher stress concentration directly under the load point. For this reason, a four-point bending test (usually performed on materials sensitive to shear stresses, as brittle materials) was later performed on two more specimens with unaltered geometric properties, to determine if different results could be achieved. In contrast to the three-point bending test, the four-point bending test distributes the load more evenly, by applying two equal loads at equal distances from the centre of the sample. This setup reduces possible shear stress effects and provides a more uniform bending moment in the middle section, leading to more accurate measurements of flexural properties. This usually assure a failure due to bending and not to shear, because of the pure bending stresses in the region between the two-loading point, known as the load span.

Supports were place at distance $L = 240$ *mm* (support span), force F was applied via two-point at a distance $a = 70$ *mm* in the upper face (see Figure 5.10). This configuration allowed the placement of two 60-*mm* long strain gauges to register strain in the upper

and lower face on the specimen where maximum deformations (tension and compression) are exhibit. Two specimens, B4 and B5, were tested with this set-up.

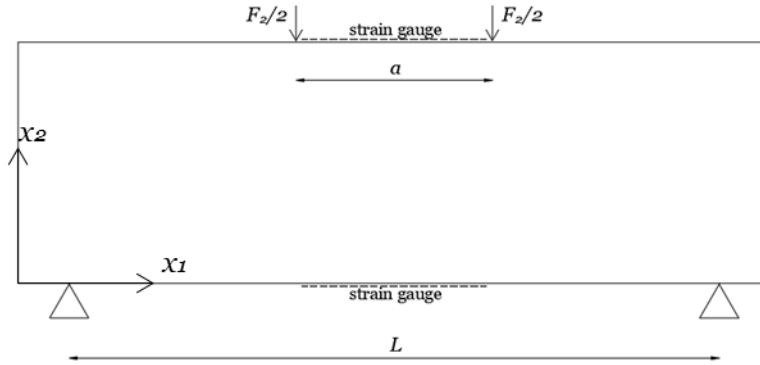


Figure 5.10: Four-point bending test set-up and strain gauges configurations for prismatic specimens.

Because the proportion between support span L and load span a is much closer to $\frac{1}{3}$ rather than $\frac{1}{2}$ the tensile strength was computed as followed, with b and d being width and depth of the specimen expressed in mm .

$$f_f = \frac{FL}{bd^2} \quad (4)$$

No significant differences were recorded in flexural strength and failure mode of the specimens between the two different tests (see Figure 5.12).

<i>Specimen</i>	<i>Flexural strength</i>	<i>Mean flexural strength</i>
	[N/mm ²]	[N/mm ²]
B1	1.06	
B2	1.24	
B3	1.13	1.16
B4	1.34	c.o.v. = 0.1
B5	1.03	

Table 5.3: Flexural strength determined for prismatic SCEB specimen.

The mean flexural strength evaluated is roughly 1.1 MPa (see Table 5.3). The moderate strength can be attributed to cement stabilisation which enhances the mechanical properties (see Chapter 2). Compared to concrete materials the strength is almost one-third of it, but it can be considered a noticeable achievement for earth-based materials. As observed in [204] adobe and rammed earth exhibit a flexural strength between

0.13 to 0.2 MPa and only one case registered of 1.03 MPa. Similarly in [133] tensile strength of CEB was investigated, with and without the addition of banana-fibre reinforcement. CEB showed a flexural strength of 0.5 MPa while 50-mm-long banana fibre reinforcement led to a strength of 0.95 MPa.

From the correlation of strain and flexural tension, in specimen B4 and B5, the Flexural modulus, E^{f-SCEB} , was determined as the initial slope (see Figure 5.11). By examining the initial linear portion of the stress-strain curve, the flexural modulus provides a measure of the material stiffness, further characterizing the mechanical performance of the cement stabilized CEBs. The mean value of $\langle E^{f-SCEB} \rangle = 5600 \text{ MPa}$ was obtained. The maximum bending moment obtained was $M^{max} = 15000 \text{ MPa}$ of calculated for the load span as follows.

$$M^{max} = \frac{F_2}{2} a \quad (5)$$

The test results showed a sharp failure mechanism for the specimen, with a maximum strain of approximately 0.02%, suggesting that the material is quite brittle and has a low deformation capacity. While the flexural modulus of the cement-stabilized CEB indicates high stiffness, which can be advantageous for its structural behaviour, the observed low deformation capacity means the material cannot withstand significant bending before failure occurs.

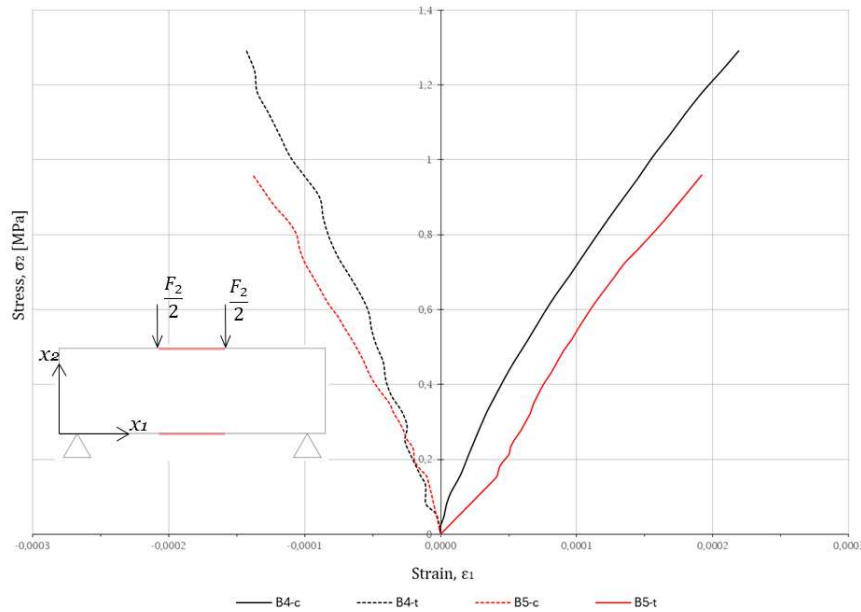


Figure 5.11: Stress-strain plot in flexion of SCEB prismatic specimens.



(a)



(b)

Figure 5.12: Failure mechanism of (a) three-point bending test and (b) four-point bending test for prismatic SCEB specimens.

5.1.2 Earth mortar

Mortar plays a key role in the masonry mechanics. It acts as a binder for block units and determine how effectively loads are transfer between them. Achieving optimal binding capability is essential for ensuring good mechanical performance in masonry construction.

For earth-based mortars, the soil mix typically consists of clay and sand. Since soil alone lacks sufficient binding properties, the mix must include clay, a binding agent, or both. As stated in Chapter 2, clay can lead to issues related to water absorption and retention and eventually leading to excessive shrinkage, so only a low percentage of clay (10 to 15%) is usually suitable when cement is blended into the soil mix. Other chemical binders, such as lime, can ensure good physical and chemical performance with a higher clay content. However, there are always some limitations compared to the mechanical properties obtained with cement stabilised earth-products.

Since earth-based mortar, as a construction material, lacks standardisation and the wide range of earth mix and percentage of stabilisers, it was necessary to assess basic mechanical properties of this new mortar mix. It was done through flexural and compressive tests. Prismatic specimens were prepared in accordance with the standards and cured for 28 days, considering the cement content in the mixture.

As earth mortar is a brittle material, two additional mixes were produced by adding natural fibres with the aim of enhancing the flexural strength of the mixture, and eventually positively influence the post peak behaviour of the material. These fibres – coconut and hemp – were selected which as common to the Senegalese area (see Figure 5.13). Three mortar mixes were produced as follows:

1. Mix-1: Earth mortar composed of sieved red earth, sand, and cement in a ratio of 3:6:1.
2. Mix-2: Earth mortar with hemp fibres, consisting of sieved red earth, sand, and cement in a ratio of 3:6:1, with 1.8% of fibres by weight of dry mass.
3. Mix-3: Earth mortar with coconut fibres, comprising sieved red earth, sand, and cement in a ratio of 3:6:1, with 1.8% of fibres by weight of dry mass.

Fibers, cut to a length of approximately 20 mm, were blended into the dry mix before water was added. No treatment was done on the fibres prior mixing.



Figure 5.13: Mortars prismatic specimens.

Specimen characteristics are given as:

1. Mix-1: mean density $\rho^m = 1600 \text{ kg/m}^3$, COV=0.029, size: $40 \times 40 \times 160 \text{ mm}^3$.
2. Mix-2: mean density $\rho^m = 1566 \text{ kg/m}^3$, COV=0.012, size: $40 \times 40 \times 160 \text{ mm}^3$
3. Mix-3: mean density $\rho^m = 1558 \text{ kg/m}^3$, COV=0.026, size: $40 \times 40 \times 160 \text{ mm}^3$

Geometric characteristics of the specimen before testing are here reported, shrinkage effect can be observed in the three dimensions. The samples are name as $Mn-m$, where n indicates the type of mixture (1,2,3) and m the specimen number.

Understanding mortar shrinkage is necessary to anticipate eventual cracks formation which compromise integrity of the material and of the masonry work and to assure sufficient bond-strength between unit and mortar and further mechanical performance of the composite system. In this study the mould dimensions have been taken as a reference for the initial length, while dimensions measured after the curing period as final length. The mean shrinkage level ε^s , expressed in percentage, of the three mortar mixes in the three directions are:

- Mix-1: $\varepsilon_1^s = 0.4\%$; $\varepsilon_2^s = 2.5\%$; $\varepsilon_3^s = 1.09\%$;
- Mix-2: $\varepsilon_1^s = 0.19\%$; $\varepsilon_2^s = 2.08\%$; $\varepsilon_3^s = 1.04\%$;
- Mix-3: $\varepsilon_1^s = 0.11\%$; $\varepsilon_2^s = 0.64\%$; $\varepsilon_3^s = 0.4\%$;

Mix-1 and Mix-2 exhibited the highest overall shrinkage levels among the three mixes, with Mix-1 showing the greatest shrinkage. The addition of fibres in the mixes demonstrated an improved response in terms of reducing volume contraction. However, compared to traditional cementitious mortars, these shrinkage levels are significantly higher. This increased shrinkage can be attributed to the presence of clay and organic materials. In this case, the fibres proved beneficial in mitigating some of the shrinkage effects.

Bending test

The flexural strength of a mortar is a parameter to be investigated in masonry construction because it influences structural behaviour, durability and crack formation of masonry walls. A mortar with sufficient flexural strength can withstand loads without cracking, which weaken the structure, or failure.

To determine flexural strength of earth-based mortar used in this study a three-point bending test was performed; reference was made to the UNI EN 1015-11: 2007 [153], specimen size was set accordingly. As per standard, lower supports were fixed at a 100 mm distance symmetrically to the middle section of the specimen. The tests were conducted in displacement control. Load was applied in the centreline at a steady velocity of 0.5 mm/min. Six specimen of Mix-1, nine of Mix-2 and four of Mix-3 were tested. Majority of Mix-3 specimens produced exhibited pre-testing cracks, or even presented already split, and felt fragile to the touch (see Figure 5.14). This could be attributed to the presence of coconut fibres, which hindered their optimal curing.



Figure 5.14: Pre-testing split specimen of Mix-3 mortar mix.

Flexural strength was calculated using eq. (3) where F is the peak load applied, obtained by machine data.

Samples M3-3, M3-5 and M3-6 were not suitable for testing because of pre-existing cracks. Mortar mixes proved to be quite limited in flexural strength if compared with traditional mortars. Despite expected results, specimen of Mix-1 experienced higher mechanical performance compared to Mix-2 and Mix-3. While the specimens from Mix-2 exhibit reduced performance compared to Mix-1, the data remains comparable (see Table 5.4). However, an increase in strength as usually expected from hemp fibres reinforcement was not found, which can be influenced by the amount of fibres introduced in the mixture. Despite the reduction in strength, Mix-2 specimens showed an increase in vertical displacement (u_2) capacity of 160% compared to Mix-1 specimen (see failure mode at Figure 5.12). Due to the mechanical response deviating from the average tendency of Mix-2 specimens, samples M2-4 and M2-6 were not considered to define u_2 (see Table 5.5).



(a)



(b)

Figure 5.12: Three-point bending test failure mechanism of (a) unreinforced mortar mix (b) fibre-reinforced mortar mix

<i>Specimen</i>	<i>Flexural strength</i> [N/mm ²]	<i>Mean flexural strength</i> [N/mm ²]
M1-1	0.52	0.59 c.o.v. = 0.06
M1-2	0.62	
M1-3	0.57	
M1-4	0.62	
M1-5	0.63	
M1-6	0.56	
M2-1	0.31	0.36 c.o.v. = 0.14
M2-2	0.26	
M2-3	0.14	
M2-4	0.39	
M2-5	0.35	
M2-6	0.15	
M2-7	0.40	
M2-8	0.41	
M2-9	0.40	
M3-1	0.039	0.04 c.o.v. 0.22
M3-2	0.038	
M3-3	-	
M3-4	0.053	
M3-5	-	
M3-6	-	
M3-7	0.029	

Table 5.4: Flexural strength determined for mortar specimens.

<i>Specimen</i>	<i>Vertical displacement</i> [mm]	<i>Mean vertical displacement</i> [mm]
M1-1	0.63	
M1-2	0.48	
M1-3*	0.25	0.6
M1-4	0.68	c.o.v. = 0.11
M1-5	0.58	
M1-6	0.62	
*Considering M1-3 in the: $\langle u_2 \rangle = 0.54$; c.o.v. = 0.27		
M2-1	1.50	
M2-2	1.45	
M2-3	-	
M2-4	1.63	
M2-5	1.60	1.57
M2-6	-	c.o.v. = 0.016
M2-7	1.35	
M2-8	1.78	
M2-9	1.65	

Table 5.5: Vertical displacement u_2 determined for Mix-1 and Mix-2 mortar specimens.

In order to compare the data, the displacement was observed only at the peak force. Mix-1 experienced instant failure right after reaching peak load F_2 . On the other hand, Mix-2 samples continued to show a certain level of deformation due to the presence of hemp fibres. Following a force-displacement plot of two sample representative of the earth-mortar mixes different behaviour (see Figure 5.16).

In contrast, the specimens from Mix-3 experienced a drastic decrease in mechanical response. Generally, a reduced strength in coconut fibres can be observed at high water content levels. Coconut fibres have a show moisture absorption effect that tends to reduce their tensile and flexural strength [205], which is critical for structural purposes. The high-water absorption of coconut coir fibre can increase the water content in the earth mix, leading to swelling, shrinkage. When dry, fibres tend to contract, resulting in cracks overall weaker material due to deterioration (as observed with Mix-3 mortar specimens). Moreover, a high content level in coconut fibres can increase the porosity of the material, which affects strength. The presence of coconut coir fibre appears to not be beneficial to the earth-based mortar mix stabilised with cement. Similar response can be observed in different mortar mixes as cementitious mortar [206] where the use of different natural fibres led to a reduced tensile strength of the mortar, as well as for compressive strength. Natural fibres have been used to reinforce concrete structure with good results, as well as for earth-based materials.

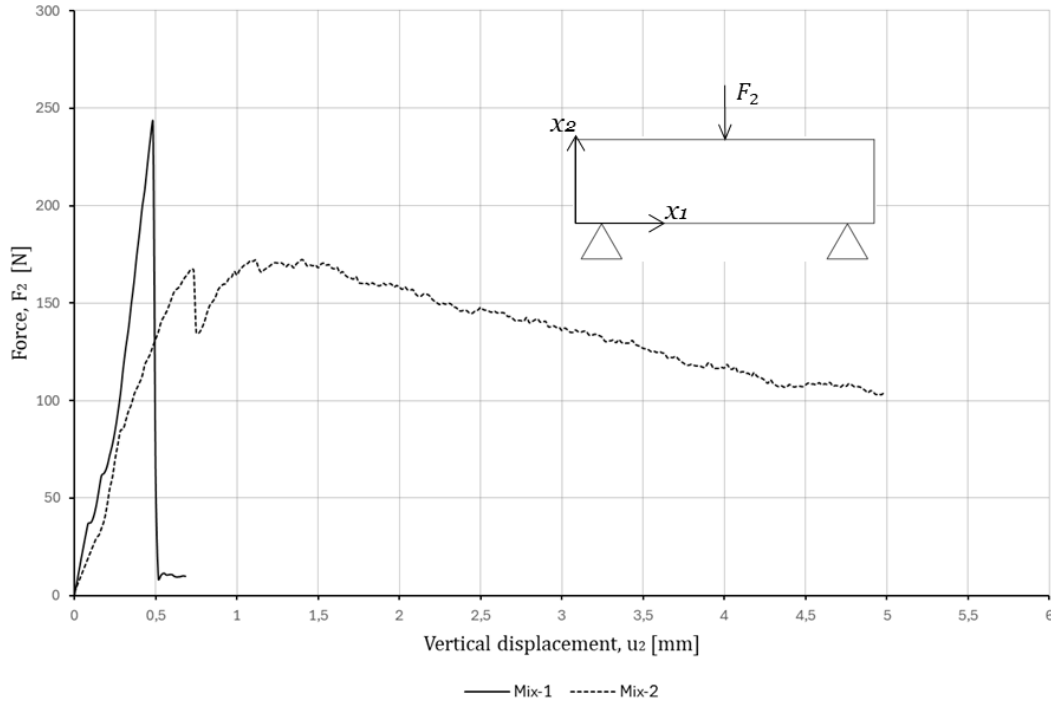


Figure 5.16: Force-displacement plot illustrating the representative behaviour of Mix-1 and Mix-2 mortars under three-point bending tests.

In [207] rammed earth blocks stabilised with cement and coconut coir fibres is investigated, it is shown that tensile strength tends to increase with the increasing of fibre content. This difference in trend can be attributed to the different nature of the material (block instead of mortar) and consequently different proportion mix and production process. Mix-3 mortar was therefore discarded for further testing.

Compression test

To further investigate the mechanical properties of Mix-1 and Mix-2 mortars, the half-specimens resulting from the flexural tests were subjected to compression testing. Contrary to the standard procedure, the half-specimens were not tested directly in the frame, as this would have posed significant challenges in strain measurement. Instead, $40 \times 40 \times 40 \text{ mm}^3$ cubic specimens were cut from the mortar halves and tested using an unconfined uniaxial compression test (see Figure 5.17). This specimen size was chosen based on standard guidelines, where mortar halves are typically tested to determine the maximum compressive strength using a frame. The standard specifies that force should be applied to a surface area of $40 \times 40 \text{ mm}^2$ to calculate compressive strength (f_c). Given the initial mortar specimens height of 40 mm, the mortar cubes were prepared with identical dimensions to ensure consistent testing conditions.

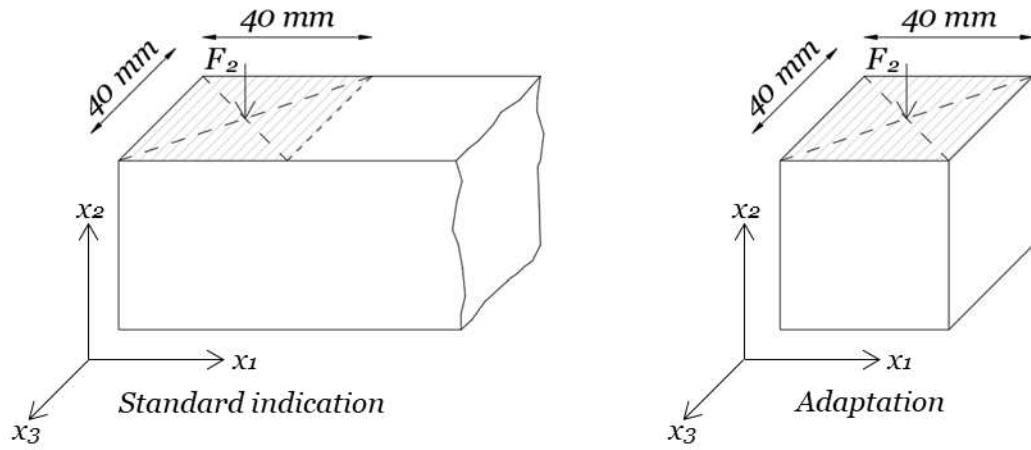


Figure 5.17: Compression test for mortar, variation of the specimen geometry.

Given the dusty/sandy texture and friability of the specimen surfaces, the use of strain gauges was not considered optimal for recording the deformations of the mortar specimens under vertical loading. Consequently, the optical Digital Image Correlation (DIC) technique was employed. As detailed in Chapter 3, DIC was used as an optical extensometer to measure deformations between defined points. However, another challenge arose due to the sandy surface texture, complicating the application of markers, and/or the reliability of the marker to represent the actual deformation of the specimen and not a surface collapse of the material. Utilising the physical characteristics of the surface to define control points proved difficult due to the homogeneity of colour and texture. To address this, a textured element was placed on the same plane as the specimen, with the plate for load transfer from the actuator chosen as the first reference (see Figure 5.18).

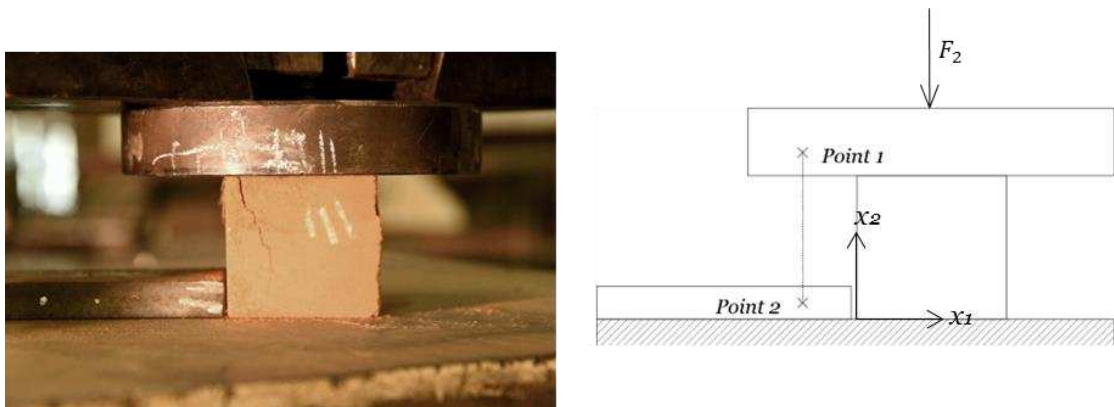


Figure 5.18: Set-up of compression test for mortar cubic specimens.

The force was applied via an actuator in displacement control at a velocity rate of 0.5 mm/min . However, since the failure mode and the time required to reach it were not

known, it was not possible to set an optimal image capture rate (i.e., the sampling frequency). To address this, a video was recorded during the test, allowing frames to be extracted later at the optimal intervals. This approach enabled a representative description of the mechanical behaviour while optimizing the data used. Vertical strain was extracted using DIC as a virtual extensometer with a time interval of $\Delta t = 1$ s. A total of six specimens of Mix-1 and seven of Mix-2 were tested. The compressive strength was determined using eq. (1), where F represents the peak force, and b and d represent the width and depth of the specimen at the contact area, respectively. To account variability of natural based materials, the coefficient of variation was calculated. Given the friability of the material and the manual preparation of the specimens, consistent dimensions were not achieved. Below are the measurements of the contact surface used for calculating compressive strength. In the hemp-reinforced specimens (Mix-2) a higher fluctuation of the data was observed (see Table 5.6). Because this trend was not showed in the Mix-1, it can be attributable to the presence of natural-fibres reinforcement and not to eventual damages for the manual production of the mortar specimens. Hence, outliers were excluded from the analysis when their values significantly deviated from the standard deviation of the Mix-2 group (from MC2-1 to MC2-3).

<i>Specimen</i>	<i>d</i>	<i>b</i>	<i>Compressive strength</i>	<i>Mean compressive strength</i>
	[mm]	[mm]	[N/mm ²]	[N/mm ²]
MC1-1	40	38	1.74	1.3 c.o.v. = 0.18
MC1-2	40	38	1.1	
MC1-3	40	39	1.5	
MC1-4	38	39	1.23	
MC1-5	43	38	1.2	
MC1-6	40	38	1.03	
MC2-1	44	39	0.35	0.89 c.o.v. = 0.21
MC2-2	40	42	0.22	
MC2-3	40	45	0.4	
MC2-4	40	43	1.01	
MC2-5	40	43.5	0.76	
MC2-6	42	40	0.65	
MC2-7	45	40	1.14	
*Considering all values: < <i>f_c</i> >= 0.64; c. o. v. = 0.49				

Table 5.6: Compressive strength of Mix-1 and Mix-2 mortar cubic specimens.

Compressive strength of the two mortar mixes is comparable, despite the Mix-2 reduction in strength. The primary failure mechanism observed in Mix-1 mortar specimens was characterized by the development of vertical cracks parallel to the load

direction, accompanied by crumbling at the corners, leading to sudden failure (see Figure 5.19). These vertical cracks are indicative of tensile splitting, where tensile stresses within the specimen exceed the material tensile strength, causing it to crack along the direction of the applied load. This type of tensile failure is typical of brittle materials, confirming the brittle nature of the earth-based mortar mix. The abrupt failure and lack of plastic deformation further highlight the material limited ductility. All specimens broke with a failure mode ascribable at the acceptable failure modes described in UNI EN 12390-3 of concrete cubes.



Figure 5.19: Failure mode of Mix-1 mortar cubic specimens.

For the Mix-2 mortar specimens, the presence of hemp fibres resulted in a different failure mode compared to the Mix-1 specimens. Initially, vertical cracking was observed, similar to the behaviour of the Mix-1 specimens, but the failure was not immediately reached (see Figure 5.20). The hemp fibres embedded within the mix contributed to an increased internal tensile strength, as evidenced in a second phase of multiple vertical cracks development and the delayed final crushing of the specimen. In the final phase of loading, the specimen remained intact primarily due to the reinforcing effect of the fibres, demonstrating an improvement in mechanical behaviour. Despite the reduction observed in compressive strength, the addition of fibres enhanced the ductility of the mortar, which can be particularly advantageous in masonry applications, where a more ductile mortar can help with the brittle behaviour of the element.



Figure 5.20: Failure mode comparison between Mix-1 and Mix-2 mortar cubic specimens.

Reconducting the failure typology to concrete standard was not a reliable choice for this mortar mix due to the presence of fibres which alters the stress distribution of the material. Modulus of elasticity of mortars E_2^{Mix-n} was later investigated to better understand the mechanical properties (see Table 5.7). It was determined as the initial slope of stress-strain curve of specimen when tested in compression. The stiffness of the material is then described by the linear proportion at the initial stage of loading.

<i>Specimen</i>	<i>Elastic modulus</i>	<i>Mean elastic modulus</i>
	[N/mm ²]	[N/mm ²]
MC1-1	218.9	
MC1-2	132.2	
MC1-3	155.7	214.14
MC1-4	233.6	c.o.v. = 0.22
MC1-5	145.5	
MC1-6	248.3	
MC2-4	10.6	
MC2-5	25.4	20.35
MC2-6	25.1	c.o.v. = 0.29
MC2-7	20.2	

Table 5.7: Elastic modulus of Mix-1 and Mix-2 mortar cubic specimens.

Despite the comparable compressive strength of specimens from the Mix-1 and Mix-2 mortar mixes, the elastic modulus of Mix-2 is significantly lower, particularly at higher levels of deformation. The elastic modulus of Mix-1 ($E_2^{Mix-1} = 214 \text{ MPa}$) indicates that this mortar is considerably stiffer compared to Mix-2 ($E_2^{Mix-2} = 20 \text{ MPa}$). This increased stiffness in Mix-1 means that the mortar can withstand higher loads while experiencing less deformation. This behaviour aligns with the observed failure mechanisms under compression, which exhibited a brittle response.

In contrast, Mix-2 samples demonstrate much lower stiffness (see Figure 5.21), with significantly higher deformations (maximum mean strain $\langle \varepsilon_2^{Mix-2} \rangle = 0.19$ versus $\langle \varepsilon_2^{Mix-1} \rangle = 0.011$). However, the increased deformations of Mix-2 mortar alone are not sufficient to account for the reduced stiffness and compressive strength. Consequently, Mix-2 mortar was considered unsuitable for earth masonry construction, leading to its exclusion in favour of Mix-1. Unexpectedly, the incorporation of fibres into the earth matrix in Mix-2 introduced discontinuities, effectively creating voids within the material. These voids adversely affected the mechanical strength, contrary to the

anticipated reinforcement provided by the fibres. As a result, the presence of fibres in Mix-2 did not enhance the material performance as expected and instead compromised its structural integrity. Despite the lower deformations exhibited by the Mix-1 mortar specimens, they demonstrate an increased strain capacity compared to the cement-stabilised blocks, which have an elastic modulus of $E_2^{SCEB} = 6900 \text{ MPa}$ with maximum mean strain of $\varepsilon_2^{SCEB} = 0.003$ (see Figure 5.22).

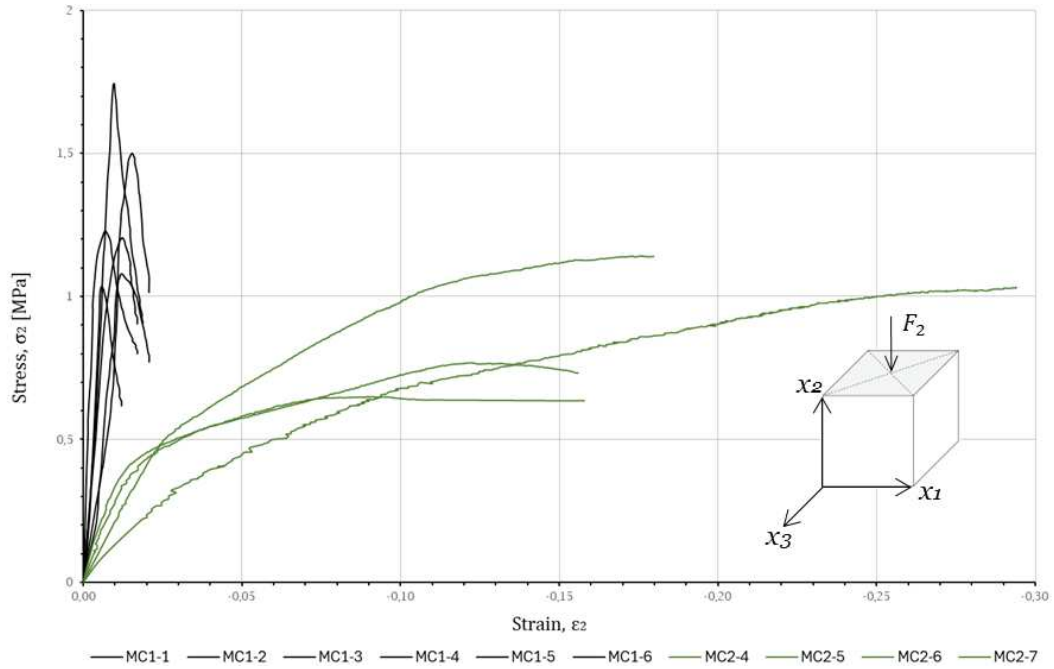


Figure 5.21: Stress-strain plot in compression of Mix-1 and Mix-2 mortar cubic specimens.

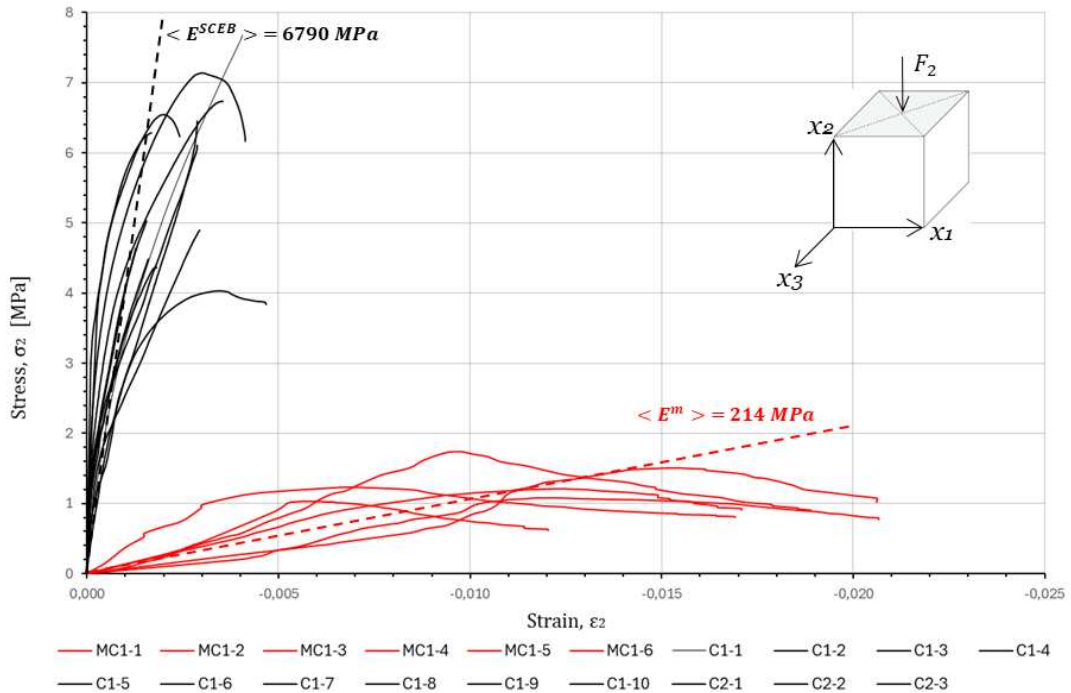


Figure 5.22: Stress-strain plot in compression of Mix-1 and SCEB cubic specimens.

This indicates that, while Mix-1 is stiffer than Mix-2, it still allows for some flexibility within the masonry system. The significant difference in elastic modulus between the mortar and the blocks means that Mix-1 mortar can absorb and dissipate stress more effectively, accommodating a degree of deformation within the masonry elements.

5.2 Characterisation of the Construction System: Earth Masonry Panels

This section investigates the mechanical properties of cement-stabilised earth block (SCEB) masonry. It is crucial to investigate the mechanical behaviour of a composite structure, in an experimental setting, to further study it through a numerical modelling phase. The earth panels are built with SCEB laid with an earth-based mortar. The high number of variables present in earthen construction - such as the various soil mixtures that can be used, the different types and percentages of stabilisers, the diverse production processes, the varying sizes and geometries of the bricks, and the different types of mortar employed, including their composition and thickness - results in specific mechanical interactions between the mortar and the block unique to each construction system. Consequently, this leads to a characteristic response to stress that is not directly comparable to case studies found in the literature. Therefore, it is required to thoroughly assess their mechanical performance, focusing on key mechanical parameters such as compressive strength, elastic modulus, and failure typology are examined. Both unreinforced and reinforced specimens were subjected to uniaxial unconfined compression tests, to evaluate these properties. The subsequent sections detail the mechanical characterization of the SCEB masonry panels, providing insights into their structural capabilities.

Specimens

The construction system under investigation consists of a masonry structure with a header bond pattern, featuring mortar joints of equal thickness ($e = e_1 = e_2 = 5 \text{ mm}$). The specific characteristics of the masonry panels are as follows:

- Panel Dimensions:
 - $1225 \times 945 \times 305 \text{ mm}^3$, determined by the configuration of the construction system.
- Panel Variants:
 - Unreinforced (UM).
 - Reinforced: with an embedded concrete reinforcement frame (CRM).
 - Reinforced: with an embedded mixed frame (BFRP fibre-net and concrete elements, MRM).

The presence of the reinforcement frame imposes a minimum size below which it is not feasible to create a specimen that accurately replicates, during testing, the behaviour of the construction system (see Figure 5.23).

The masonry panels were built in the Laboratory of strength of Materials (LabSCo) of IUAV university of Venice, by Felino Company. The blocks were wetted prior specimen building, a higher water content allows for the mortar to cure properly. The earth block properties of water absorption can lead to excessive shrinkage of the mortar, due to quick moisture migration, and therefore a weaker bond in the masonry. Attention was paid to keep the mortar joint thickness constant. A final layer of cementitious mortar was laid to assure evenness of the structure. Because of the presence of cement inside the earth-based mortar and the concrete frame, a 28-day period was necessary to fully cure the specimen, in laboratory conditions.

Test set-up

To investigate the behaviour of the earth masonry panel, reference was made to UNI EN 1052-1:2001 [154], which provides guidelines on specimen geometry, test setup, and the evaluation of the compressive strength of masonry systems. However, due to the unique shape and dimensions of the blocks, as well as the presence of a reinforcement frame within the masonry system, it was not entirely possible to adopt the fixed geometric proportions outlined in the standard. Although the block proportions within the masonry pattern were in agreement with the standard, and the proportion between the block length and the panel length was maintained, the ratio between the height and length of the panels did not conform to the standard, with the specimen height being lower than its length. However, because of the proportion between block dimension and panels dimension was respected the representative behaviour of the masonry was assured. Moreover, this specific shape allows for a proportional representation of the real structure and by doing so, reliable strain measurement and stiffness. The specimen geometry, and consequently test data, can be therefore considered acceptable and reliable even if not entirely in line with the standard. Additionally, strain measurement was conducted using the Digital Image Correlation (DIC) technique to achieve more comprehensive control over the failure mechanisms and crack development within the panel. The elastic modulus was also evaluated as the initial tangent of the stress-strain curve obtained, rather than using the formula indicated in the standard.

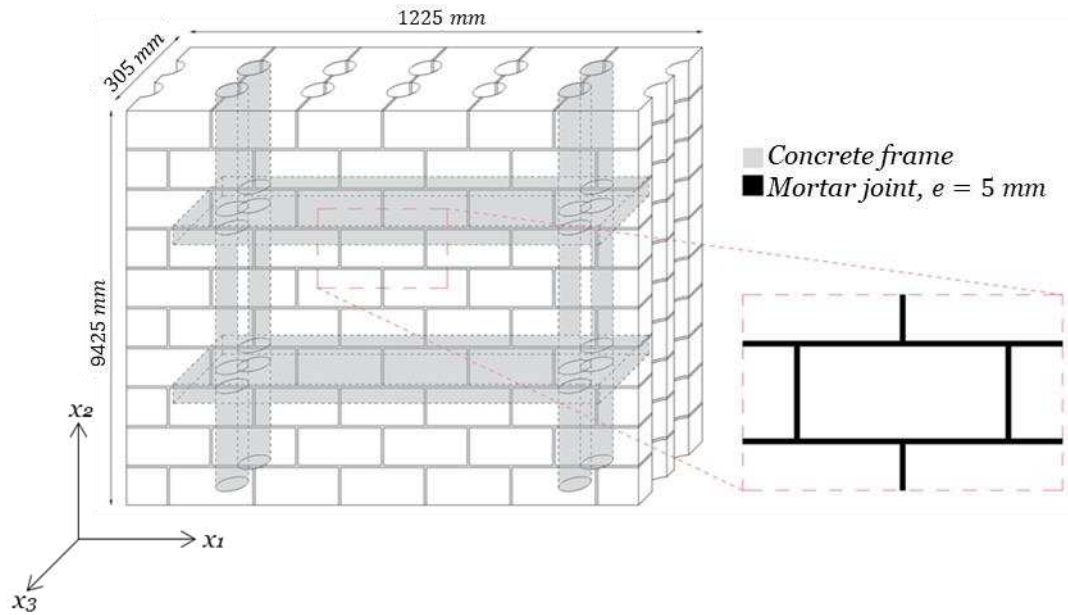


Figure 5.23: Reinforced earth masonry panel configuration.

The unconfined compression test was conducted on the earth masonry panel using the MetroCom press under displacement control. The test procedure involved the following steps:

- Load Application: The load was applied at a velocity rate of $\frac{du_2}{dt} = 1.5 \text{ mm/min}$ through a steel beam.
- Failure Time: Structural failure was reached within approximately 10 minutes.

Due to the heterogeneous nature of masonry and the unique properties of earth-based materials, it was crucial to focus on:

- Identification of strain zones.
- Detection of asymmetric deformations.
- Monitoring of crack development.

To achieve precise displacement recording and deformation analysis, the Digital Image Correlation (DIC) method was employed. The methodology included:

- Marker Placement: High-contrast targets were symmetrically placed on the block heads to characterize each masonry unit (see Figure 5.24). Two markers were used to record the failure behaviour of individual blocks, which was anticipated due to the characteristic hollows in the units which define a weaker region due to the reduction of section.

- **Grid Definition:** This arrangement resulted in a regular grid to describe the masonry panel.

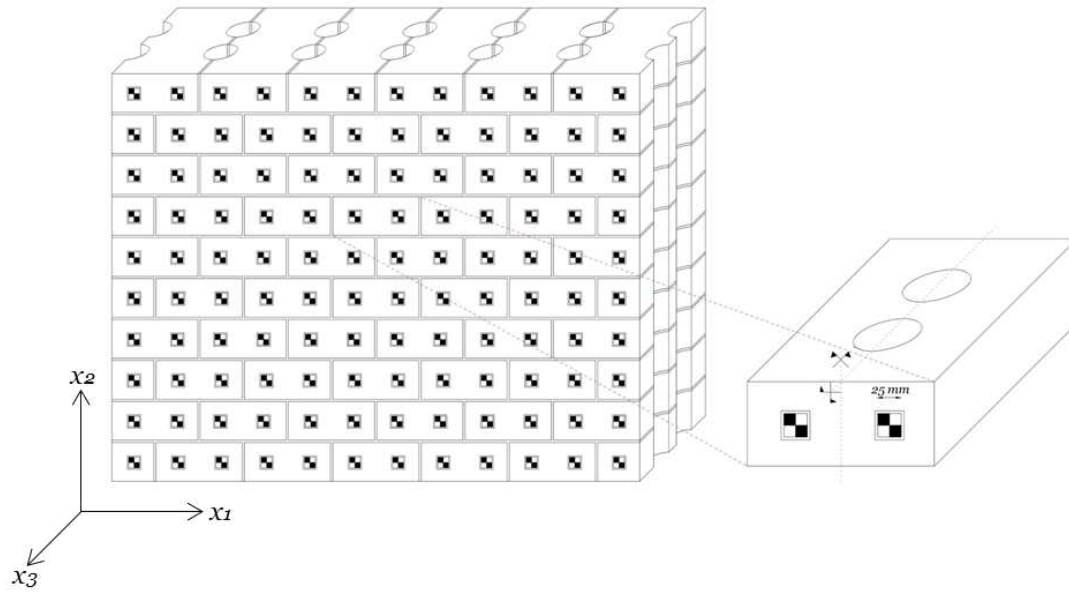


Figure 5.24: Reinforced earth masonry panel configuration.

Furthermore, the earth texture was used as an alternative for tracking points (see Figure 5.25). Control markers were fixed on the concrete support beam of the specimens to ensure stable and unreformable reference points.

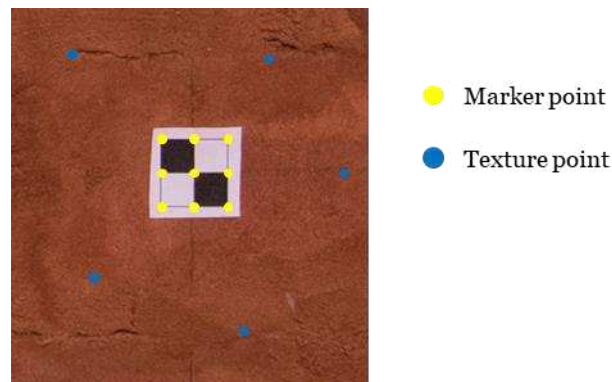


Figure 5.25: Identification of tracking points.

Data acquisition involved target tracking with a 50×50 *pixel* correlation subset size and displacement tracking of target centres and additional points on the target edges and specimen surface, providing a comprehensive analysis of compressive behaviour, strain zones, and crack development in the earth masonry specimens.

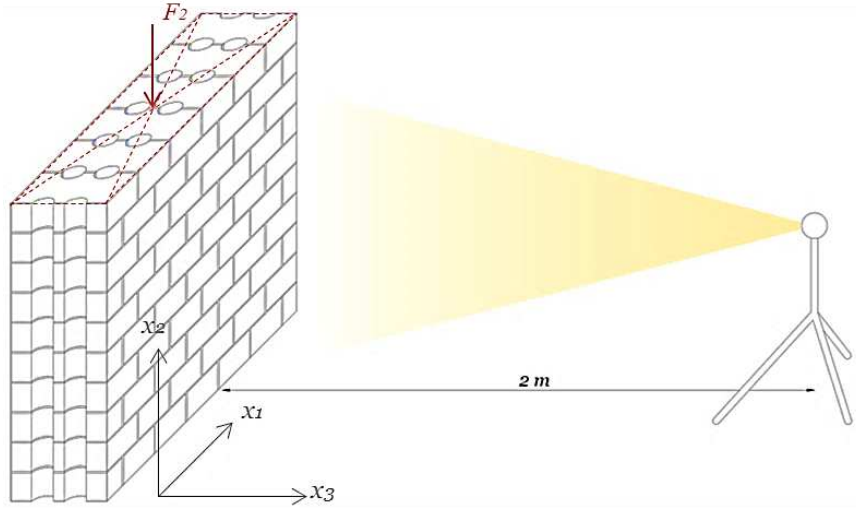


Figure 5.26: Test set-up for uniaxial unconfined compression test on earth masonry panels.

A digital single-lens reflex DSLR camera was used to record pictures at a time interval of $\Delta t = 2 \text{ s}$, with image resolution of $6000 \times 4000 \text{ pixel}$. Manual focus configuration was set to maintain uniform focal length. The camera was fixed on a tripod at a 2 m (see Figure 5.26) distance from the specimen surface, while camera height was fixed to ensure alignment with the barycentre of the surface specimens. A cool light source was used to assure marker and texture details readability.

All specimens were capped with a thin layer of a weak cementitious mortar to assure evenness of the loading surface.

5.2.1 Unreinforced earth-masonry panels

Two unreinforced masonry panels were tested. One of the masonry panels (UM1) was instrumented with two transducers in order to obtain an initial reference measurement for the deformations derived from DIC (see Figure 5.27). These were mounted on the face opposite to the one with the targets and arranged, one in the direction of load application and the other transverse to the load application.

During test both specimen UM1 and UM2 vertical cracks, characteristic of compressive loads, were observed. In UM1 the first visible crack was recorded at $t^1 = 654 \text{ s}$ at 98.15% of the maximum load. The crack appeared of the first block (from left to right) of the 8th row. Differently from usual vertical cracks observed in masonry panels subjected to vertical load, it did not originate at the midpoint of the unit but slightly to the right side. This is relevant to this masonry system because of the different shapes of SCEB used. The 8th row is composed of Block-1 typology, featuring semi-cylindrical voids on the sides. The external portions of the block, where reduction in material mass happens, could be identified as a weakness area when subject to vertical loads.

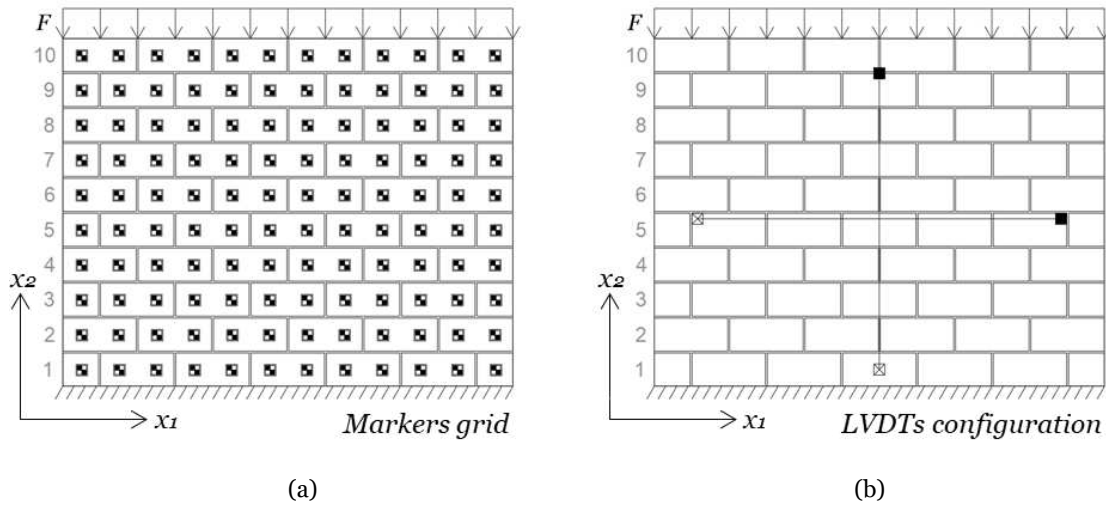


Figure 5.27: UM1 specimen surfaces instrumentation, frontal surface (a) marker grid and back surface (b) with displacement transducers (LVDT).

The first crack was observed at a position that confirmed the vulnerability of the block. Subsequently, crack propagated towards the 7th row where the second block failed. This row is composed by Block-2 typology, characterised by cylindric voids along the midspan (see Figure 5.28). The crack in this row advanced along the central area of the voids, indicating a relatively weaker portion of the block.

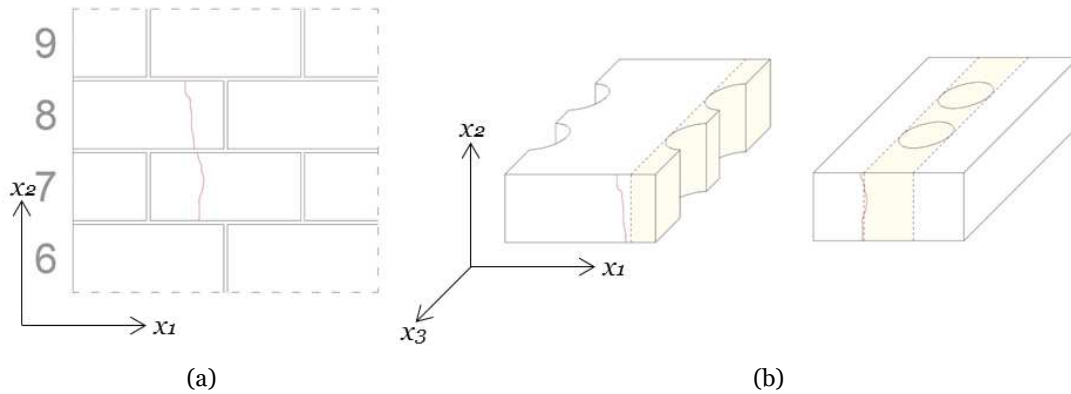


Figure 5.28: UM1 first visible crack, localisation on the wallet surface (a) localisation on each block isolated block (b)

Peak load was reached at $t^{peak} = 666 \text{ s}$, with a maximum compression strength of $f_c = 2.14 \text{ MPa}$ (evaluated with eq. 1). Just after this point, a second crack appeared at a time $t^2 = 668 \text{ s}$, coinciding with a load of 99.46% of the maximum load. The crack developed from the midspan of the first block at row 10 and started to propagate vertically along the head joint and units midspan. This crack formation and propagation pattern is commonly observed in standard masonry panel subjected to vertical loads. The observation that this failure mechanism only manifested at a later stage in the loading

process can indicate that the brick geometry is a contributing factor to an overall reduction of mechanical performances the unreinforced masonry structure. Both cracks fully developed at $t^3 = 700s$, at 98.6% of the maximum load. The experiment was concluded at $t^f = 712 s$. The panel exhibited asymmetric cracking predominantly on its left side (see Figure 5.29). This cracking pattern may be attributed to construction irregularities. Minor cracks and localized crushing were observed in the boundary elements, indicative of stress concentrations at the panel edges. This can be attributed to high stress concentrations, likely due to boundary effects at the panel ends, where constraints are greater than in the middle sections, or due to construction imperfections. The second crack propagated vertically up to the 5th course and then horizontally, resulting in a detachment of the cracked portion. The failure typology and mode of failure indicate brittle material behaviour.

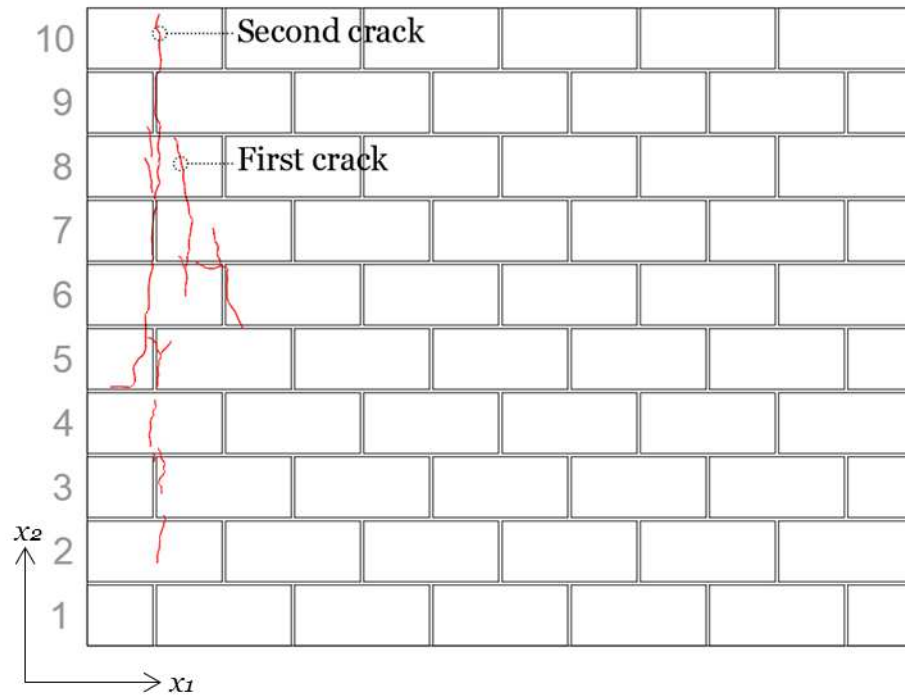


Figure 5.29: Crack pattern of UM1 specimen.

A stress-strain plot was used to investigate the global vertical deformation of the wall. Transducer data and DIC data were also compared. In order to evaluate deformation along the panel surface deformations, strain data were extrapolated from each marker column, defined from row 2 to row 9. Columns were identified with the code vn , where n number of the column from left to right (see Figure 5.31). A total of 10 columns were investigated, because the two central ones were impossible to read because of visual obstacle of the instrumentation. First and last row were excluded from calculation because of the micro cracking and crushing localised in this area which can lead to altered

result. Moreover, by doing so is possible to get a more direct correspondence with transducer measurement. A certain level of uniformity of deformation can be observed, exception made for columns v1 and v2 which experienced the highest deformation ($\varepsilon_2^{v1} = \varepsilon_2^{v2} = 0.40\%$), as expected since they define the area where majority of cracks developed. Moving from left to right minor deformation occurred, $\langle \varepsilon_2^{vn} \rangle = 0.26\%$ (see Figure 5.30).

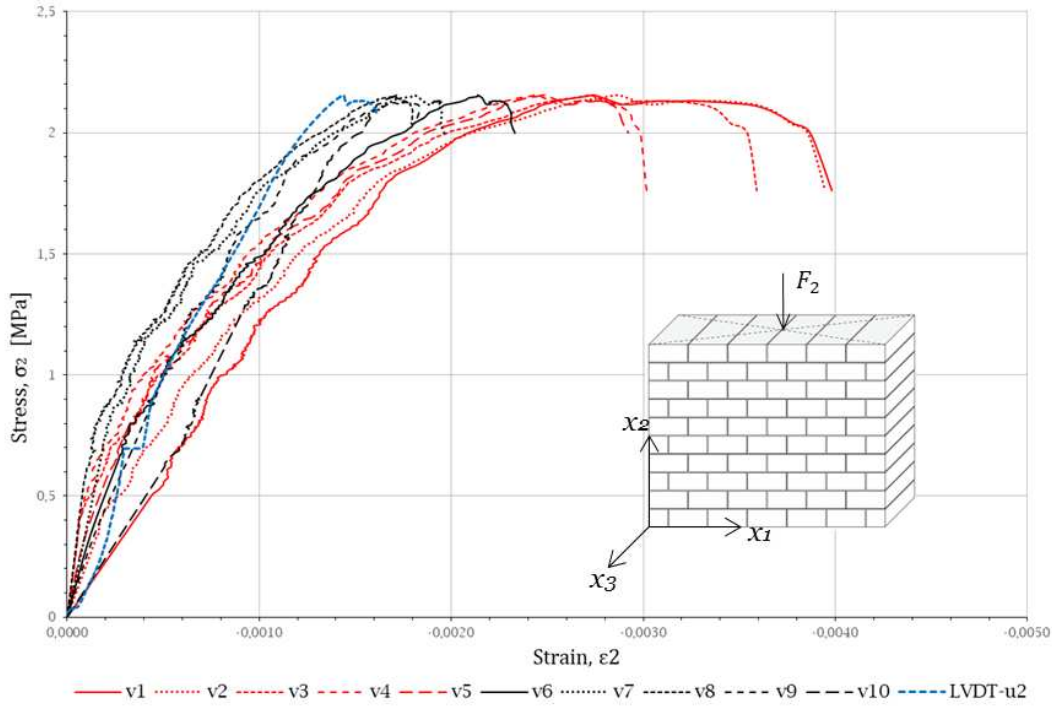


Figure 5.30: Stress-strain plot for different vertical columns from DIC and LVDT in UM1 specimen tested in compression.

If vertical deformation data are evaluated with columns extended from the 1st to the 10th row higher strain data are registered, as showed in the plot comparing v5 with v5-1 and v6 with v6-1 (see Figure 5.32), where v_{n-1} is the vertical strain registered from rows 1 to 10. As previously mentioned, this is attributable to local damages due to higher stresses concentration. Because the data from the transducer are aligned with the trend of the selected portion of the panel the selection of the columns defined between marker of 2nd and 9th rows is to be considered accurate to describe overall masonry panel deformation.

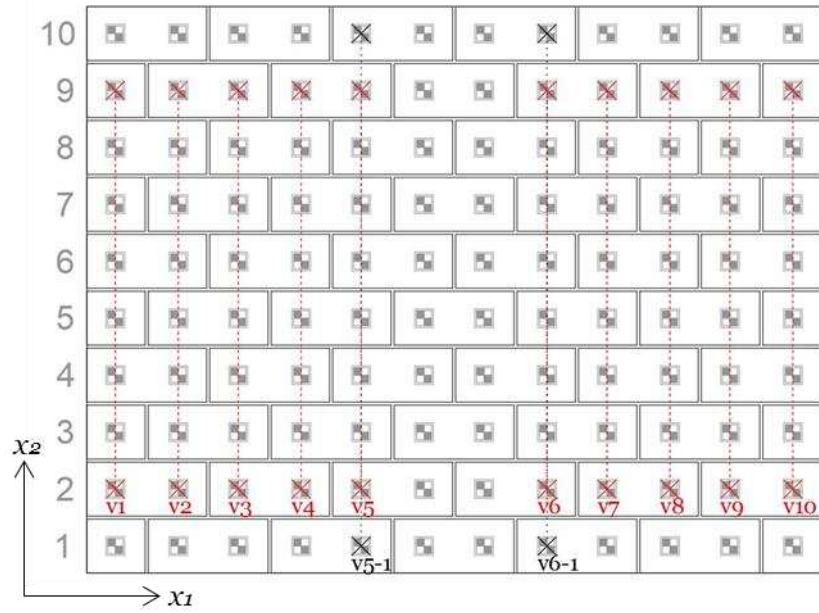


Figure 5.31: Vertical columns to evaluate vertical deformation ε_2^v using DIC technique.

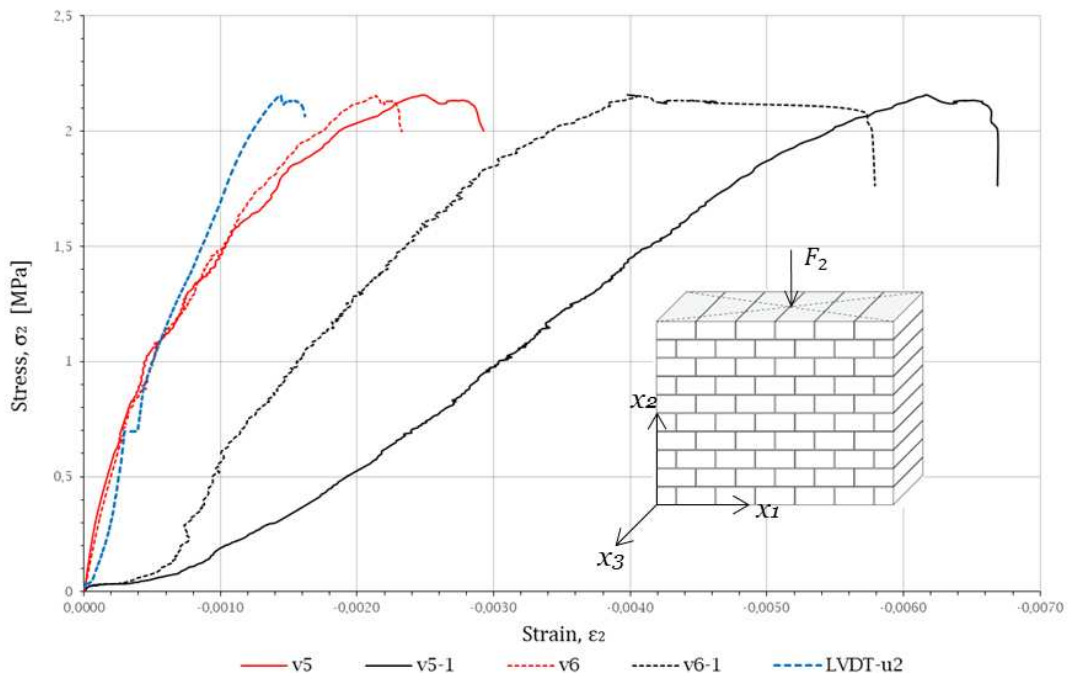


Figure 5.32: Comparison of stress-strain in ε_2^v extrapolation in range 2nd to 9th and 1st to 10th row.

In order to better understand the mechanical behaviour under compression of the block arranged in a masonry system the side of the panel was monitored as well. First sign of failure of the masonry system was visible at second $t^{1-s} = 658$ s before reaching peak load. The vertical crack was shown at the unit defying the third row. Half unit of Block-2 was used to “close” the row, the vertical crack position corresponds to a change in geometry which characterise Block-2 typology.

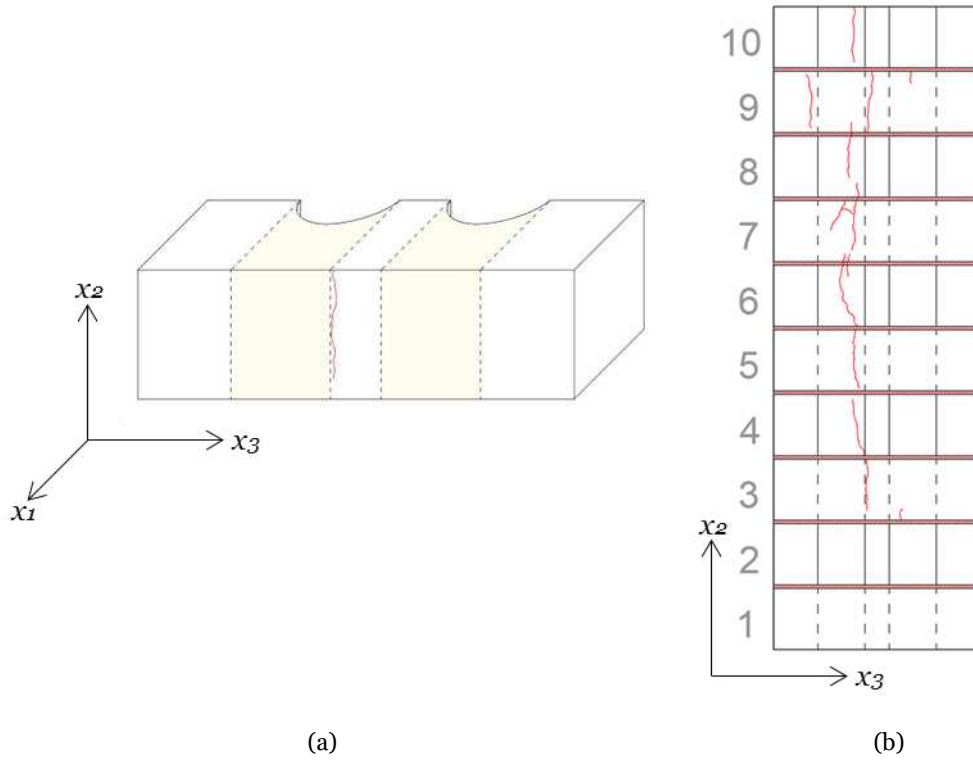


Figure 5.33: UM1 first visible crack on the specimen width, localised on the isolated block (a), crack pattern of UM1 specimen, side view (b).

After reaching peak load at time $t^{peak} = 666$ s, comparable crack pattern was later visible at multiple units. These three localised vertical cracks formed in an interval between 99.9% and 98.14% of the maximum force. At this stage, differently from the frontal side, only localised cracks were visible, developed at the weak points of the blocks. At $t^{2-s} = 702$ s first cracks on block-1 typology units were noticeable at rows four and six. They started to develop at the external part of the semi-cylindrical voids, along the same axis of the localised cracks already formed (see Figure 5.33 b). The crack pattern development helps to confirm the weak areas given by the shape of the blocks in use. Since no significant insight on the reaction of the construction system to external vertical loads was identified from the recording of the crack pattern at the side of the panel, it was no longer investigated for the other specimens.

The elastic modulus E_2^{UM1} of the unreinforced earth masonry panel was investigated by analysing directly the initial tract, which shows a linear behaviour. Usually, to define the elastic modulus of a masonry panel is indicated by the standard to investigate the behaviour at 1/3 of the ultimate compressive strength [154]. The typical stress-strain curve of a masonry material is commonly composed of an initial non-linear behaviour due to micro-cracking and settling of the elements, a linear portion followed by a non-linear phase. The tangent modulus is used in the standards to avoid non-linearity at very

low stresses and the plastic behaviour at higher stresses. In this case, the setup used allowed to use the initial part of the stress-strain curve: the recording of the strains in compression excluded the first and last brick row, so that localised cracking and crushing was not affecting the measure.

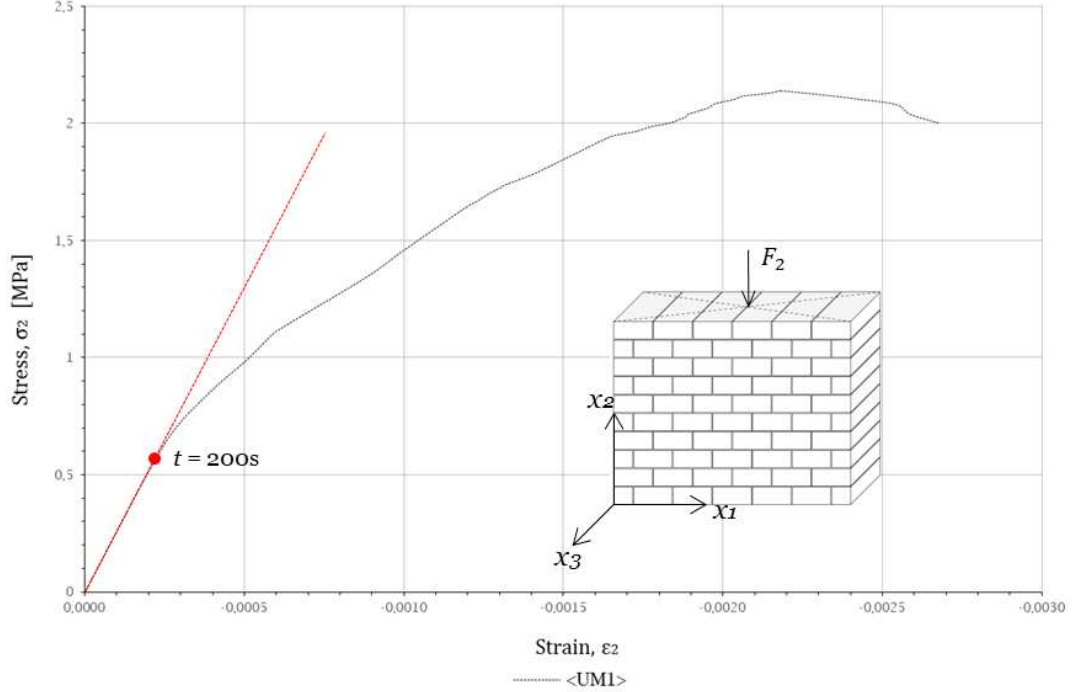


Figure 5.34: Mean compressive behaviour of UM1

Furthermore, the change in slope, marking the end of the linear phase of the panel behaviour, was correlated with the initiation of fracture. The loss of compactness within the masonry is directly linked to a decrease in stiffness, which is recorded as a variation in slope on the graph. To identify the initial fracture, horizontal strain between two points was analysed (as depicted Figure 5.36). By using DIC technique, horizontal strains within the interior points of the specimen were examined, allowing identification of the exact location of the initial break. Unlike the first visible crack (as indicated above), superficial micro-cracks are dispersed across the wall surface, caused by minor layers of mortar that crumble under load. These are not significant and may appear inconsistently during testing, making single visual analysis unreliable. Thus, the increase in horizontal deformation was adopted as an indicator of the onset of failure. By comparing the time at which this increase is recorded with the test times, this point was accurately located on the stress-strain curve. A mean elastic modulus of $E_2^{UM1} = 2550 \text{ MPa}$ was obtained.

The horizontal strain exhibits a linear increase from $t = 200 \text{ s}$, showing a similar trend for both the mortar joints in the eighth and sixth ($\varepsilon_1^8, \varepsilon_1^7$) courses as well as for the brick in the seventh course, ε_1^6 (see Figure 5.35). This pattern indicates a progressive failure

rather than an instantaneous fracture. To confirm that the failure originates in this area, a comparison was made with the deformations of the brick aligned with the ninth course, ε_1^9 . In this case, the deformation remains nearly unaltered up until $t = 600$ s.

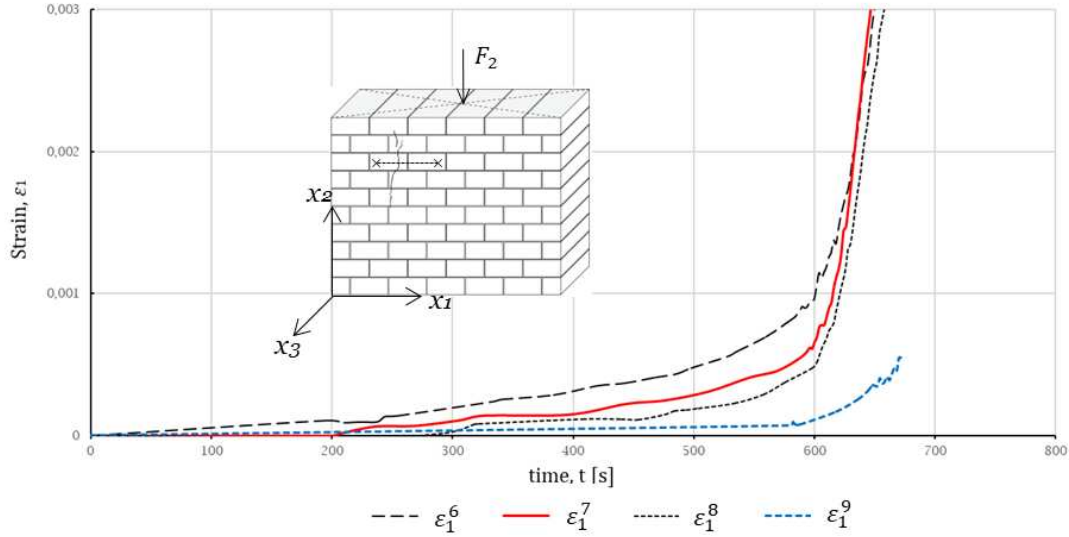


Figure 5.35: Time-strain plots for the identification of cracks initiation in UM1.

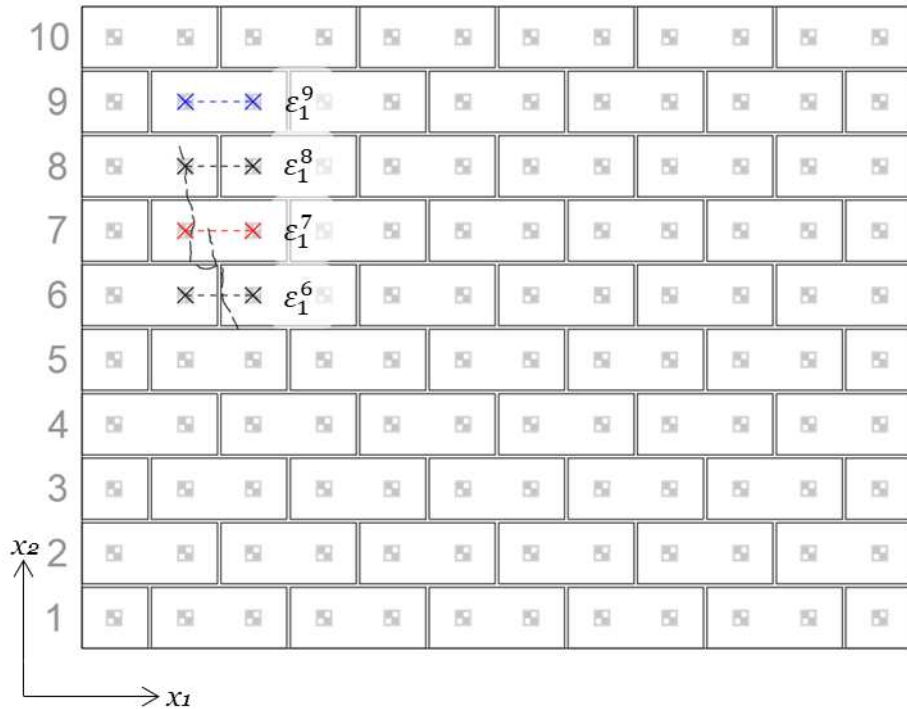


Figure 5.36: Scheme for horizontal deformation of UM1 specimen.

The second masonry panel was tested in compression as well. In this case the side of the specimen was also monitored to better investigate weakness element in the block and/or structure. In panel UM2 first visible crack occurred at time $t^1 = 340$ s when the 91.54% of the peak load was reached. The sixth block of row 7 failed showing vertical two vertical

cracks, one along the midspan and one on the side where the “hollow section” of Block-2 concludes, giving insight of the weak core of this unit typology. The second crack was originated in the mid-section of the fifth block in row 6, apparently independently from the first crack observed. The block failed at $t^2 = 380 \text{ s}$ at 96.16% of the maximum load reached. The unit was a Block-1 typology, and the development of the crack was aligned with average failing of masonry walls in compression, characterised by vertical cracking along the midspan of the unit and head joints. Just before reaching the peak load ($F^3 = 99.98\%$ of F^{max}) a third localised crack formed in the same row at block six at $t^3 = 440 \text{ s}$ (see Figure 5.37). In this case the unit failed vertically where the semi-cylindrical voids of Block-1 typology origin. Moreover, the crack was axially aligned with the first crack in the upper row. This suggests that failure was not solely due to the weakness of the resisting section but was also influenced by the pre-existing damage, which acted as a stress concentrator. The formation of localized cracks within individual masonry units, particularly in areas where changes in geometry and mass (and thus, resisting section) were observed prior to reaching the critical load, suggests an inherent weakness in the block type.

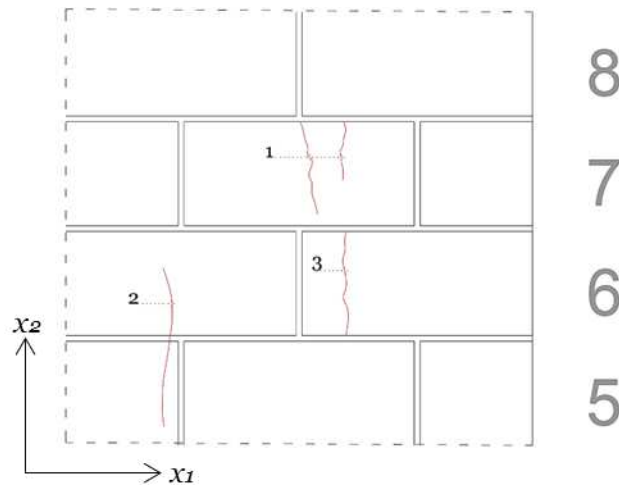


Figure 5.37: Crack formation in UM2 specimen under compressive load.

After reaching peak load the masonry panel exhibit a formation of several cracks developing along a principal vertical axis. At $t^4 = 448 \text{ s}$, with 99.8% of the maximum load, a cracking point can be observed at the midspan of block six in row 10 and at midspan of block six in row four as well and developed vertically along the underneath blocks. Right after, along the same axis, at $t^5 = 460 \text{ s}$ among fifth and sixth rows another crack developed following the midsection of the block and also the reduce-size section of the masonry units. Even though these crack remains singularly identifiable, the damage axis led to a later expulsion of the section (from row 10 to row one) at $t^6 = 482 \text{ s}$ with the corner detachment of the shaped block at the first row (see Figure 5.38). The majority of

cracks is developed on the left side of the specimen, this can be imputable to construction imperfections. Additionally, the initiation of localized cracking prior to the ultimate load suggests the presence of pre-existing damages within the blocks, which may have acted as stress concentrators, reducing the local load-bearing capacity. Comparison with specimen UM1, which exhibited a longer test duration (712 seconds versus 340 seconds), highlights the reduced mechanical strength of the specimen under investigation. The earlier onset of cracking at only 340 seconds, despite similar construction quality of the masonry, suggests the presence of potential pre-existing damage in the blocks, which reduced the load-bearing capacity and accelerated the material degradation process.

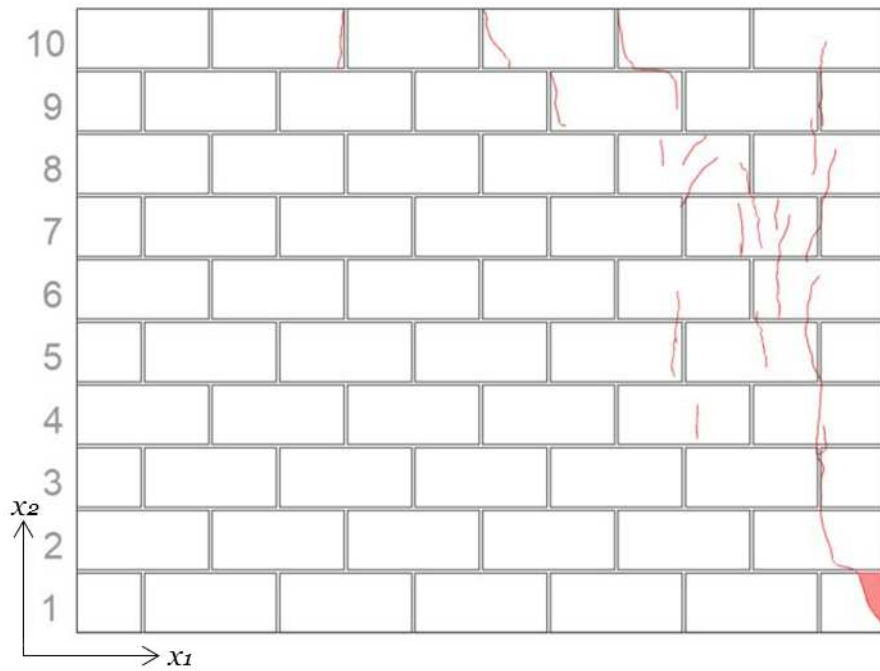


Figure 5.38: Crack pattern in UM2 specimen under compressive load.

Strain data of the panel face were collected using DIC technique. The same extraction scheme of UM1 panel was adopted to avoid higher deformation values given to local crushing at the first and tenth row. The mechanical behaviour of the panel was then investigated with a stress-strain plot using machine data from the actuator to define the stresses.

The deformation of the columns defining the left side of the panel - where no visible lesions are present - shows a similar deformation trend: an initial resistance to deformation, with almost no appreciable plastic deformation. While the data are comparable, it can be noted that as the column under study is closer to the cracked area, the deformation state becomes more pronounced (from column v1, with smaller deformations, to column v5 with larger deformations). The cracked zone, although with

increasing deformations, always shows the same mechanical behaviour, comparable to brittle materials. Interestingly, the expelled part, described by column v10, shows a greater stiffness than the two previous columns, v8 and v9 (see Figure 5.39). This indicates that the expulsion of the masonry portion occurred similarly to a rigid translation, while columns v8 and v9, where most of the deformation state is concentrated, show greater displacements compared to the panel average ($\varepsilon_1^{<v8-v9>} = 0.52\%$ against an average of the remaining columns of $\varepsilon_1^{<v1-v7,v10>} = 0.23\%$).

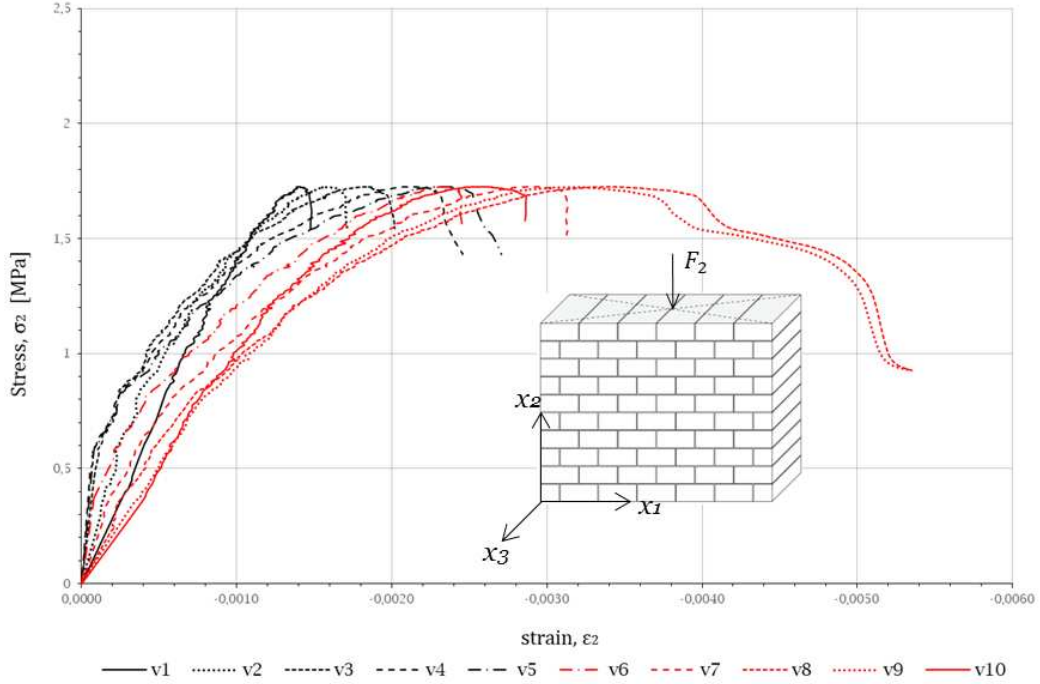


Figure 5.39: Stress-strain plot of different vertical columns in UM2 specimen tested in compression.

As per UM1 specimen, UM2 elastic modulus was evaluated by analysing the initial linear portion of the stress-strain graphic, which describe the representative global behaviour of the masonry panel. Using the markers grid, horizontal strains between internal points have been investigated to identify the first crack development, which can be matched with a loss of stiffness of the panel and therefore with the end of the linear phase behaviour. A mean elastic modulus of $E_2^{UM2} = 2440 \text{ MPa}$ was obtained. The masonry panel exits the elastic phase at a compressive stress of roughly $\sigma_2 = 0.45 \text{ MPa}$, which was reached at time $t = 100 \text{ s}$ (see Figure 5.40) By comparing the time data, a match was observed with the horizontal deformations between the two point that describe the last head joint of the eight row ε_1^8 and at sixth block of the seventh row ε_1^7 . Similar to the UM1 specimen, an initial static phase was observed, during which strain remained nearly constant, followed by a linear increase in strain between 100 and 102 seconds, marking

the initiation of the fracture. This suggests a gradual process of damage rather than an instantaneous fracture (see Figure 5.41). The identification of the crack formation in the seventh row correspond to the first visible observed (see Figure 5.37). On the other hand, the corresponding block at the ninth row (above it) ε_1^9 did not show any alteration in strain until the time $t = 476$ s.

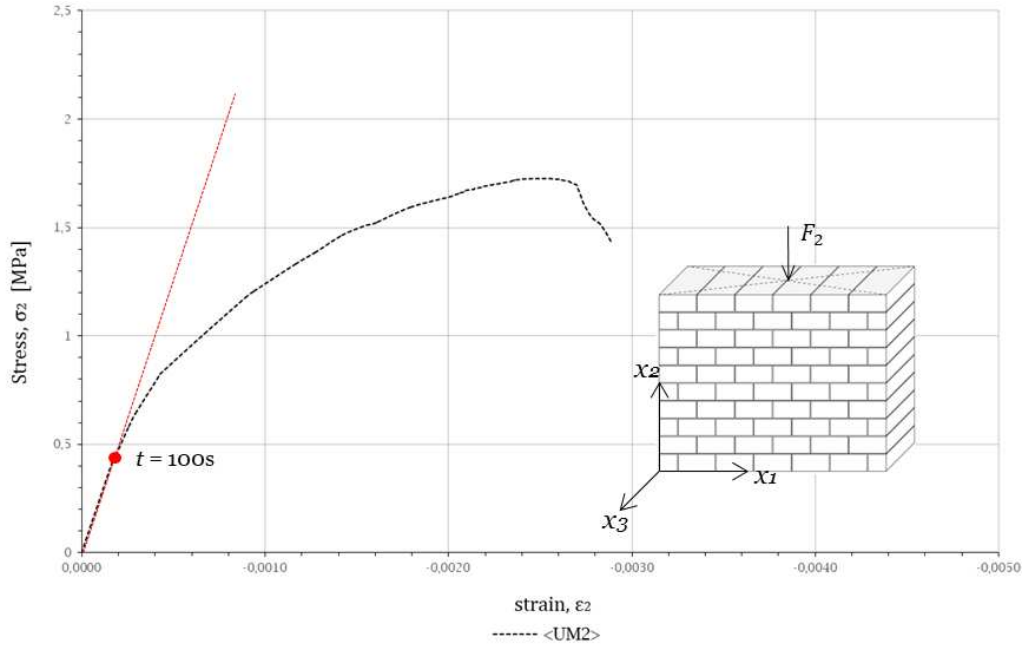


Figure 5.40: Mean compressive behaviour of UM2.

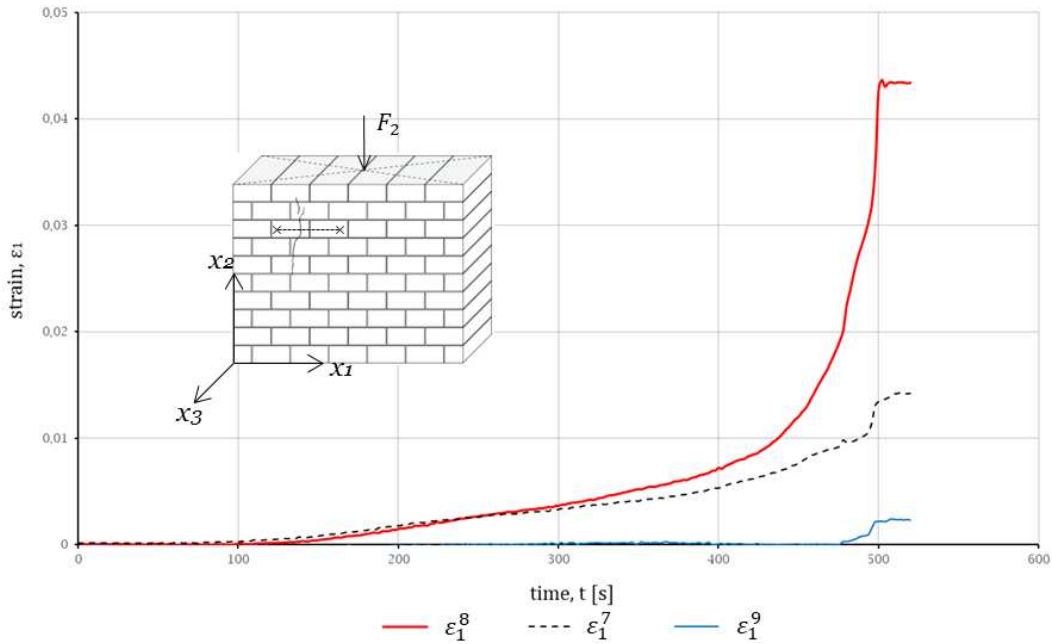


Figure 5.41: Time-strain plots for the identification of cracks initiation in UM2.

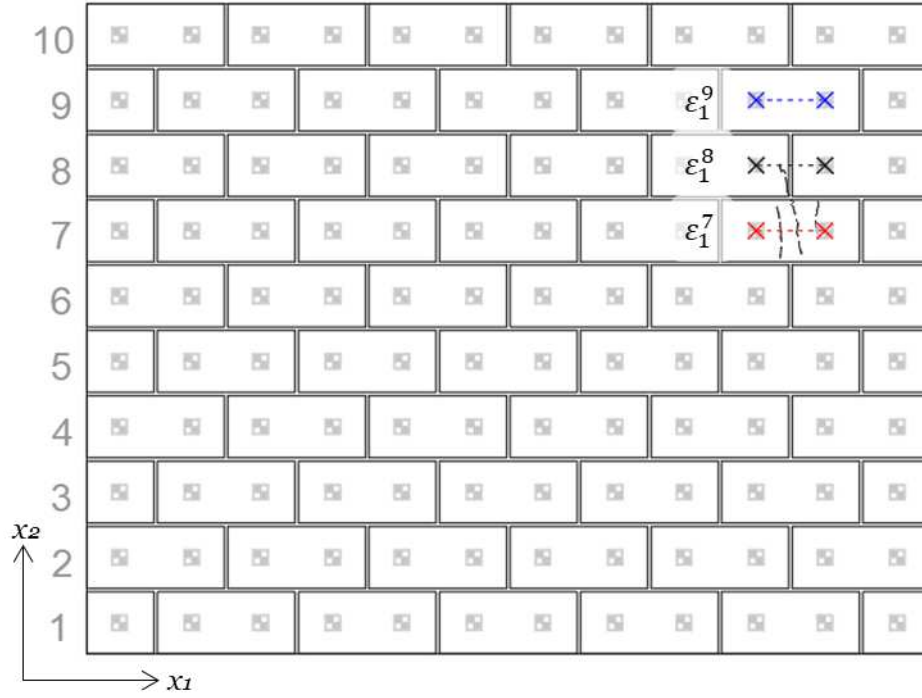


Figure 5.42: Scheme for horizontal deformation of UM2 specimen.

5.2.2 Concrete frame reinforced earth-masonry panels

In concrete reinforced panels, masonry is designed to act as permanent formwork for the frame. Two panels were built CRM1 and CMR2

In CRM1 specimen the first crack was visible at the midspan of the sixth block row nine at time $t^1 = 224$ s, with 91.10% of maximum load was reached. It is in line with the vertical reinforcement of the frame. It could be due to reduced resisting section of the block in that point (Block-2 typology) but a concentration of stress could be attributable to a change in stiffness between the concrete and the SCEB. At time $t^2 = 228$ s the crack propagated on the lateral side of the unit five at row eight. It followed the axis of the vertical reinforcement, along the reduced section of the block (Block-1 typology), compromising the head mortar joint as well. In this case, the reinforced concrete column is not only contained by the block but also by the mortar joint. Fracture formed at the interface between materials of different stiffness, where stress concentrations are likely to occur due to the disparity in material properties. In the same manner, it extended towards row 10 (see Figure 5.43). Right after the crack expanded also through row seven, which is composed by Block-3, typology at time $t^3 = 232$ s when the load reached 93.32% of its maximum. The same unit (block six of row seven) cracked vertically as well at $t^4 = 258$ s with 99.28% of peak load.

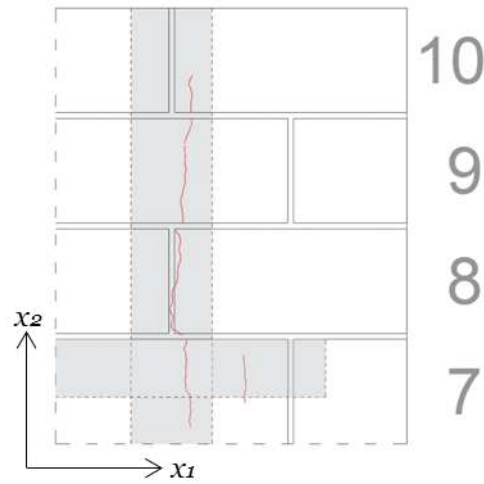


Figure 5.43: Crack formation in CRM1 specimen under compressive load.

At time $t^5 = 264$ s with 99.91% of maximum load reached, crack formation was visible at block six row six, two vertical cracks can be defined, indicating that the block is failing for tensile stress, as expected in masonry unit subjected to vertical load. From this time onwards, similar fracture development can be identified along this new identified axis in rows from seven to ten and five and four as well (see Figure 5.45). When a head joint is present the detachment of the external block is visible. At time $t^6 = 272$ s the first sign of horizontal crack started to be seen along the sixth block of row seven, which host one of the two beams of the horizontal reinforcement of the structure. At time t^6 , a decrease in the applied load compared to t^5 was observed, suggesting that the peak load had been reached. Following the formation of the horizontal crack, a slight reduction in the applied load (to 99.8% of the maximum load) was recorded, followed by a stabilization phase and a subsequent increase until the ultimate peak load was achieved at time $t^7 = 280$ s. This could be attributed to the confinement effect give by the concrete frame (see Figure 5.44).

A significant reduction in resistance was observed towards the end of the test at time $t^f = 288$ s. At this point, the right portion of the masonry panel, located between the two concrete beams, detached from the main structure, exposing the beam in row seven.

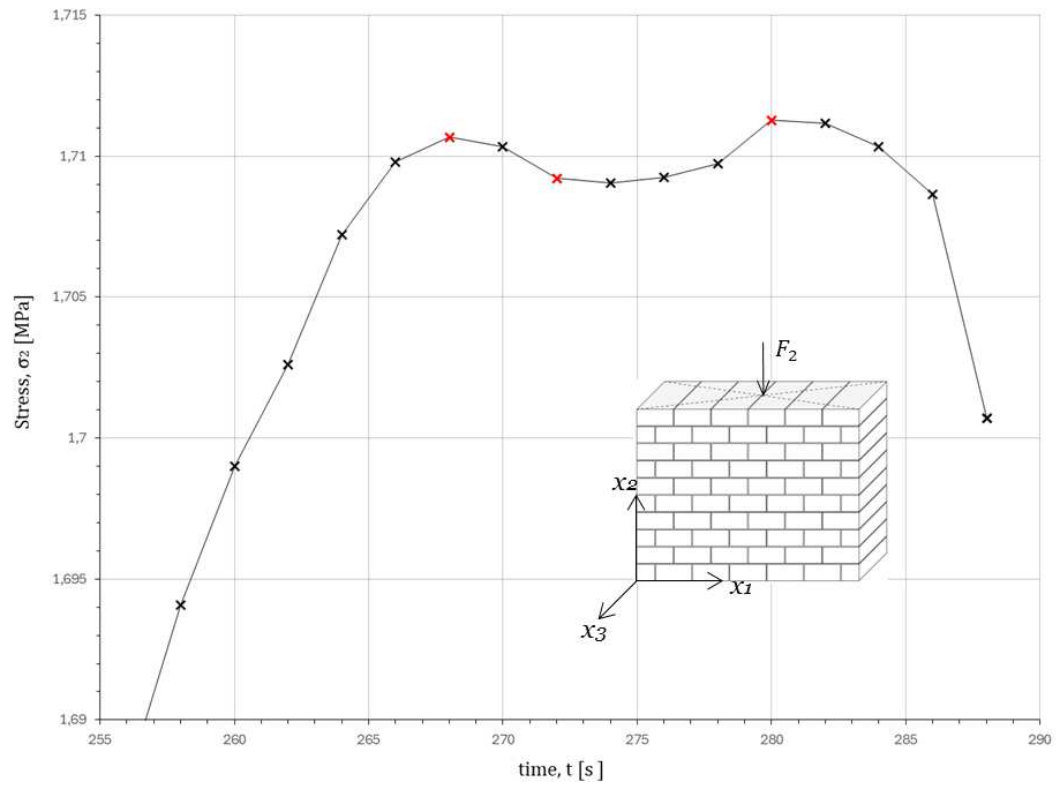


Figure 5.44: Time-compressive stress correlation plot for CRM1 specimen.

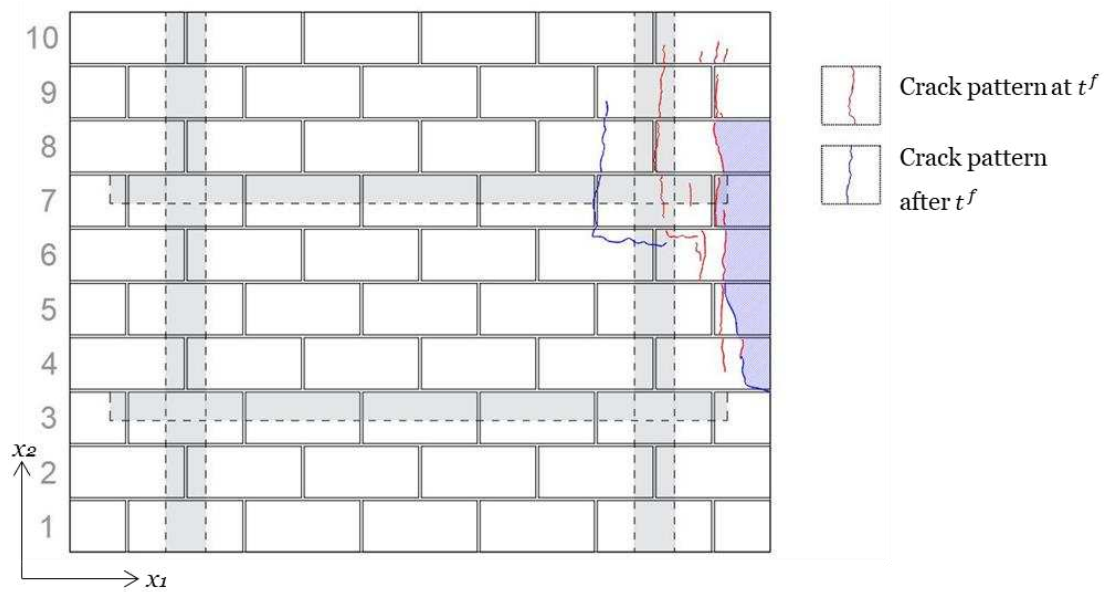


Figure 5.45: Crack pattern in CRM1 specimen under compressive load.

Stress – strain distribution was analysed to provide insights into the mechanical behaviour of the reinforced earth masonry wall. The observed nonlinearity and brittle response are consistent with the characteristics of masonry materials. The variations in the curves suggest that the material properties and construction quality may not be

uniform throughout the wall, and the presence of cracks can significantly influence the deformation behaviour of the surrounding elements. Most of the curves show a relatively brittle response, with a sharp drop in stress after reaching the peak. This suggests that the masonry material has a limited capacity for plastic deformation. Variations in the curves for different columns (from v6 to v9) can be observed, indicating variations in their stress-strain response. This could be attributed their proximity to the cracked area (see Figure 5.46). v5, v6, v7, and v8 generally show a more pronounced deformation response compared to columns further away (v1, v2, v3, and v4). This suggests that the presence of the crack may have influenced the stress distribution. On the other hand, column v10 exhibits a significantly different stress-strain response compared to the other columns, this can be ascribable to a rigid translation of the masonry portion since crack development interested the contiguous area without directly affecting it.

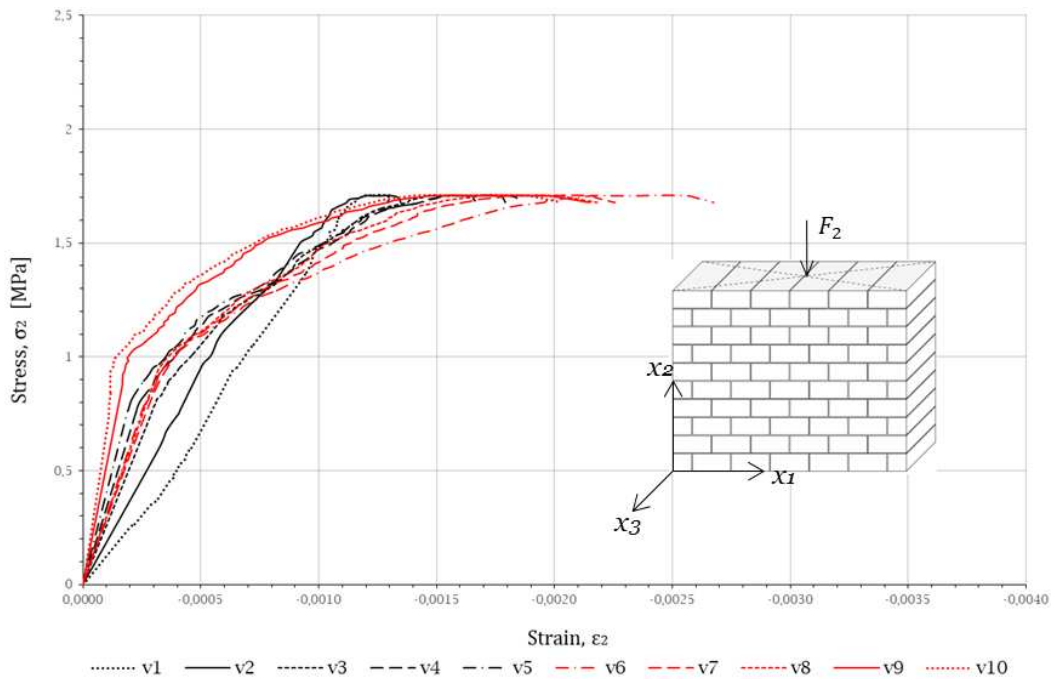


Figure 5.46: Stress-strain plot of different vertical columns in UM2 specimen tested in compression.

The elastic modulus of the panel was evaluated as the tangent of the initial linear segment of the stress-strain plot which describes the behaviour of the reinforced masonry under compressive loads. Again, the end on the elastic phase was determined by comparing the loss of stiffness with the localisation of micro-cracking, identified through the horizontal strain between internal points of the panel. An elastic modulus of $E_2^{CRM1} = 4230 \text{ MPa}$ was obtained. The end of the plastic phase was observed when the compressive strength reached $\sigma_2 = 0.34 \text{ MPa}$, at a time $t = 52 \text{ s}$ (see Figure 5.47). To verify that the end of the plastic phase in the plots correspond to a loss of stiffness given by the beginning of the

cracking generation the horizontal strains between the internal points of the specimen were investigated.

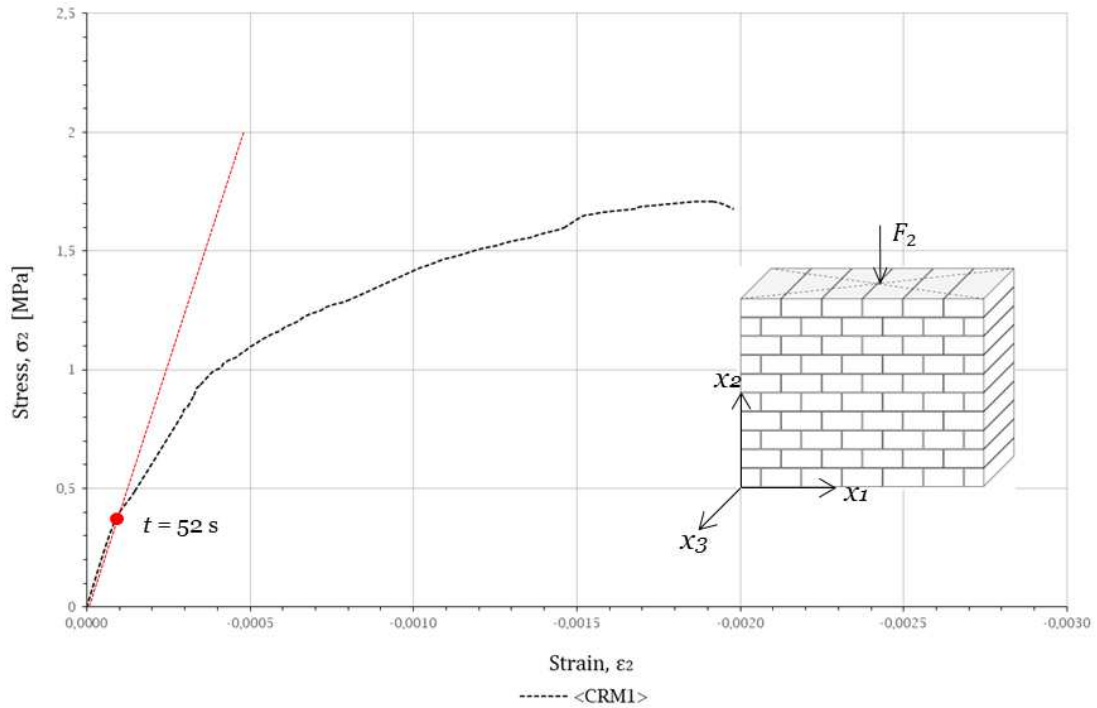


Figure 5.47: Mean compressive behaviour of CRM1.

A micro-cracking was detected at a time $t = 42 \text{ s}$ with the strain ε_1^9 on the sixth block of the ninth row, through which develops the vertical concrete reinforcement. The crack propagated through the joint underneath at the eight, where an increase of horizontal strain is detected at $t = 50 \text{ s}$ (see Figure 5.49). The presence of the vertical reinforcement might have contributed to slow down the propagation of the crack, which stopped at the head joint. To confirm that the strain ε_1^7 was investigated, which describe the mortar joint on the seventh row, aligned with the block. No significant alteration in terms of deformations was visible until the time $t = 152 \text{ s}$. This can be attributable to the concrete reinforcement which alters internal the stress distribution of the masonry. The strain on the joint ε_1^{10} at the tenth row, however, begins to show deformation from 60 seconds onward, confirming that the origin of the crack is in the block of the ninth row (see Figure 5.48).

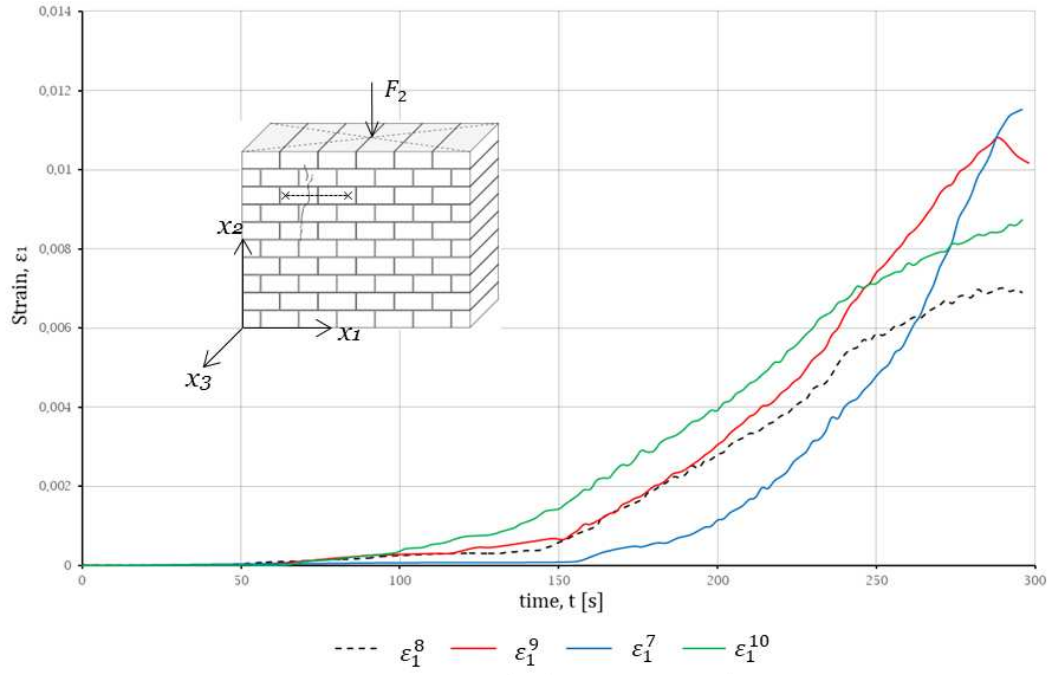


Figure 5.48: Time-strain plots for the identification of cracks initiation in CRM1.

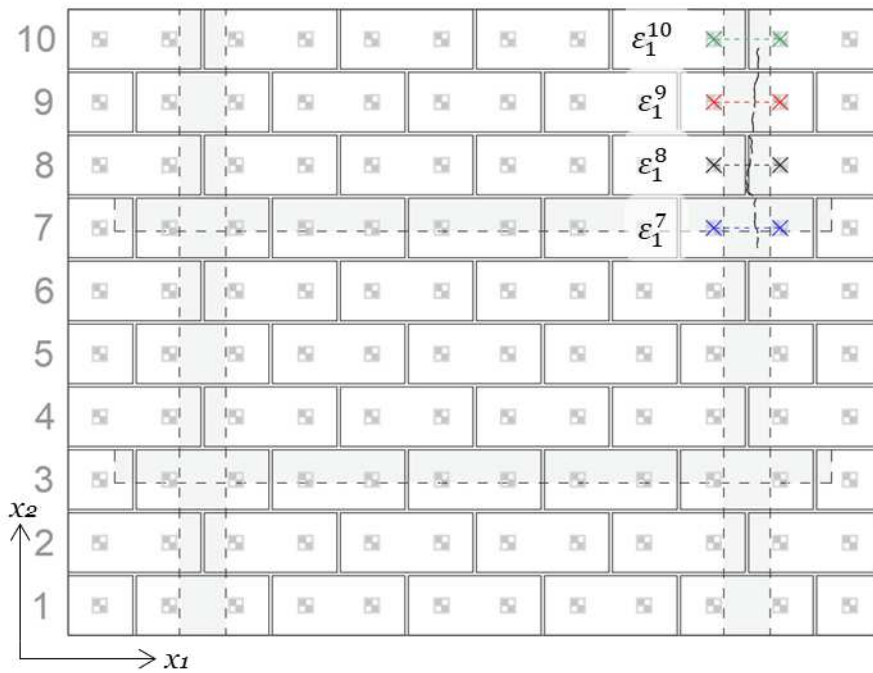


Figure 5.49: Scheme for horizontal deformation of CRM1 specimen.

The second reinforced panel CRM2 exhibited initial visible signs of failure at time $t^1 = 100$ s, upon reaching 48% of the maximum load. A crack was observed in the middle of the second block of the ninth row, corresponding to the left vertical concrete reinforcement. Almost simultaneously, within the following 10 seconds, a second crack became visible in the last vertical joint of the tenth row, corresponding to the right

vertical reinforcement, propagating towards the underlying brick. Compared to the previously observed specimens, the initial cracks appeared at a lower load percentage and symmetrically.

These damages can be attributed to the difference in stiffness between the earthen masonry and the concrete, and to the reduction in brick thickness necessary to accommodate the reinforcement. Despite this, they appeared to be of limited extent and did not propagate into the underlying masonry until time $t^2 = 200$ s, with 91% of peak load reached, where the onset of cracking in the second brick of the seventh row, where the concrete beam of the reinforcement frame develops, can be observed (see Figure 5.50). At the 230th second, the generation of a second lesion on the sixth block of the ninth row (already damaged) can be noted at about one-third of its extension, corresponding to the reduction in section given by the semicircles that characterize the Block-1 typology. This crack extends to the underlying vertical joint and damages the sixth brick of the seventh row. The load reached is now 98% of the maximum load, but the cracks still exhibit minimal width.

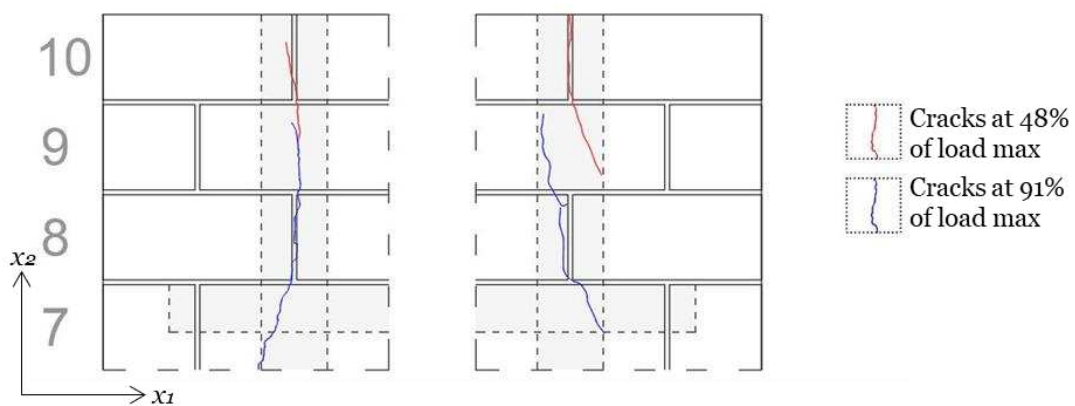


Figure 5.50: Cracks formation in CRM2 specimen under compressive load.

The vertical cracks stop at this height; below the seventh row of blocks, from the 243rd second, horizontal cracks are visible, apparently in the mortar-brick interface between the seventh and sixth rows. This type of lesion can be attributed to the presence of the horizontal concrete reinforcement, which distributes the internal stresses differently, limiting the transmission of stresses to the lower levels. This therefore leads to a greater horizontal deformation in the reinforced block row and the not reinforced one, which results in the lesion of the bed joint that unites them. The maximum load is reached at time $t^3 = 257$ s. After peak load was reached the crack pattern continue to develop above the seventh block row: more vertical cracks are noticeable at the intersection of the vertical and horizontal reinforcement on the left side of the panel, as well as another

vertical crack from the first block of the tenth row. Once $t^4 = 295$ s is reached, with 89% of the maximum load, vertical fracture started to develop under the seventh row, along the vertical from the second block of the second row (node element of the frame) downward and on the vertical mortar joint, up to the fourth row. At the 391st second a delamination process of the blocks of the sixth row occurred (see Figure 5.51). This can be attributed to a high stress concentration in the area, due to the presence of the concrete beam. The localised stress can lead to layered fracturing as the tensile strength of the block is exceeded.

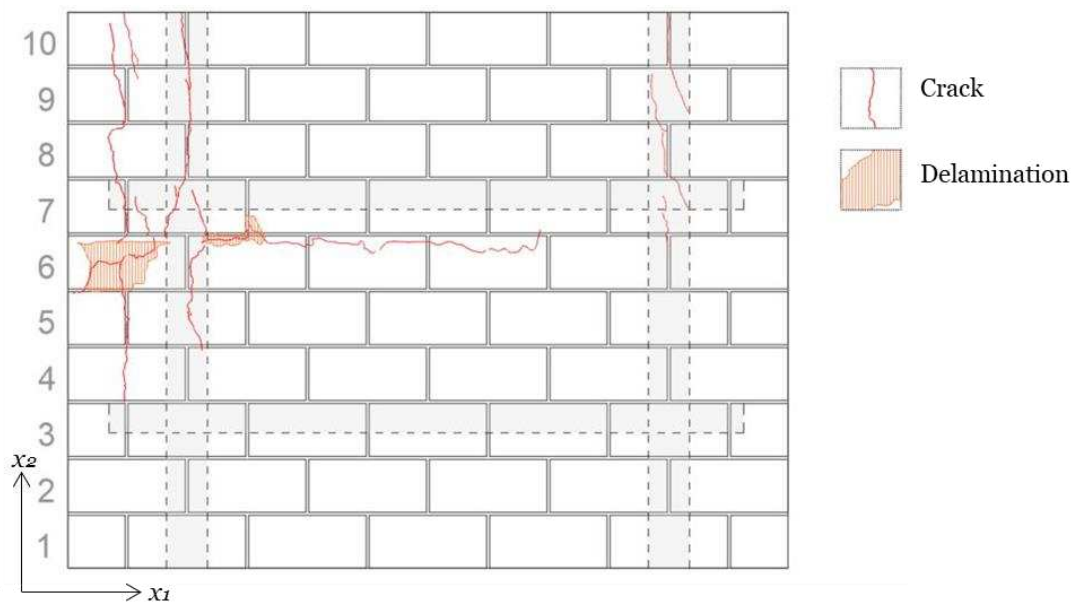


Figure 5.51: Crack pattern in CRM2 specimen under compressive load.

Using DIC technique it was possible to map vertical strain along the specimen. The evolution of the deformation was then registered through stress-strain plots. Generally, the plots showed a clear nonlinear behaviour, which is typical of masonry materials. After an initial linear phase a consistent loss in stiffness is registered, eventually reaching a strain softening phase with peak stress reached at $\sigma_2 = 1.5$ MPa. The post peak softening is visible only on the left side of the specimen (from v1 to v5) which undergoes progressive failure on a larger damaged area with both crack propagation and delamination. Columns from v6 to v10, which represent the least damaged areas, exhibited a stiffer response to the applied load even during the elastic phase. This suggests that this region may have a slightly higher rigidity compared to others, likely due to variations in the construction quality of the mortar joints. Despite different deformation level in compression, the presence of the concrete frame is not appreciable on vertical deformations.

Even though the CRM2 showed a lower resistance in compression, compared to the CRM1 specimen, higher deformations were reached, up to $\varepsilon_2^{v1} = 0.0065$ (measured in v1 column) generally higher from column v1 to v4, with a mean deformation of $\langle \varepsilon_2^{vn} \rangle = 0.003$, (see Figure 5.52).

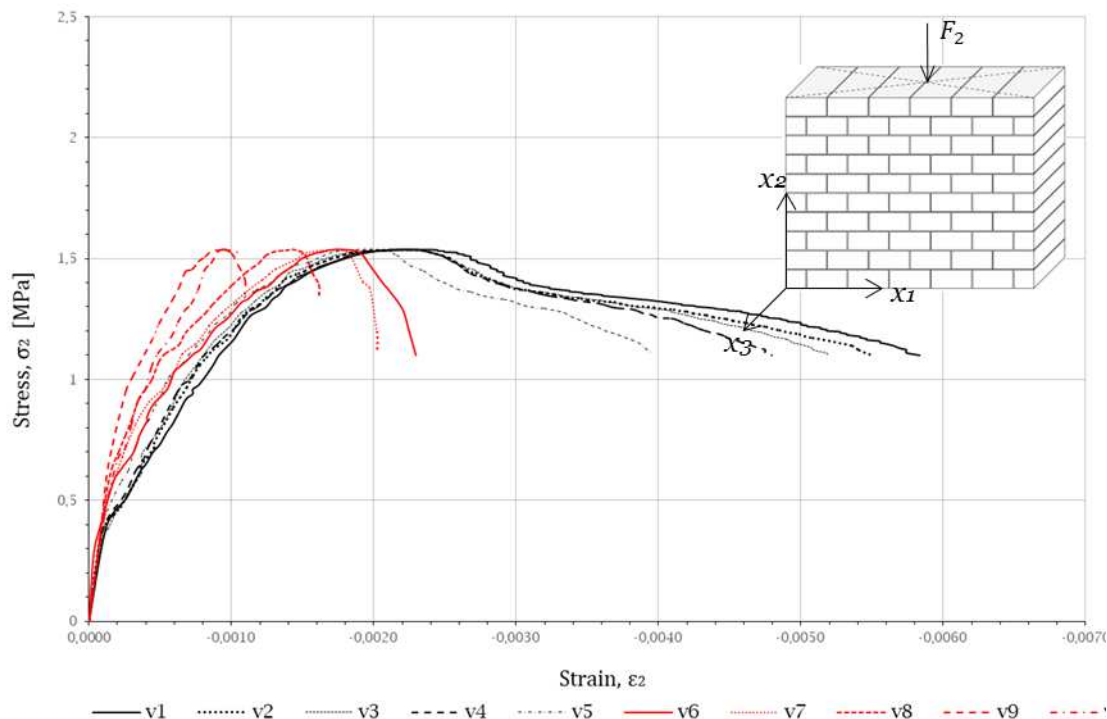


Figure 5.52: Stress-strain plot of different vertical columns in CRM2 specimen tested in compression.

The elastic modulus of CRM2 was determined as the initial linear portion of the mean stress-strain curve (see Figure 5.53), which describe the general behaviour of the reinforced masonry panel subjected to vertical loads only. The elastic modulus was registered at the end of the elastic phase, which is signed by a stiffness reduction led to micro-cracking within the specimen. To confirm that, horizontal deformations between internal points were investigated to identify the time at which the cracks start to develop. The elastic modulus was evaluated as the tangent of the stress strain curve until $\sigma_2 = 0.4 \text{ MPa}$ stress was reached, which correspond at the 62nd second, with a value of $E_2^{CRM2} = 4260 \text{ MPa}$.

For specimen CRM2 at time $t^1 = 60 \text{ s}$ and $t^2 = 60 \text{ s}$ two micro-cracks were detected in correspondence of the vertical reinforcement. The first horizontal strain ε_1^{10a} was registered on the last head joint of the tenth row, the crack later developed on the corresponding block of the ninth row at the time $t^3 = 79 \text{ s}$. The second horizontal strain

ε_1^{10b} was registered on the first joint of the tenth row and after 5 seconds on the second block of the ninth row, ε_1^{9b} .

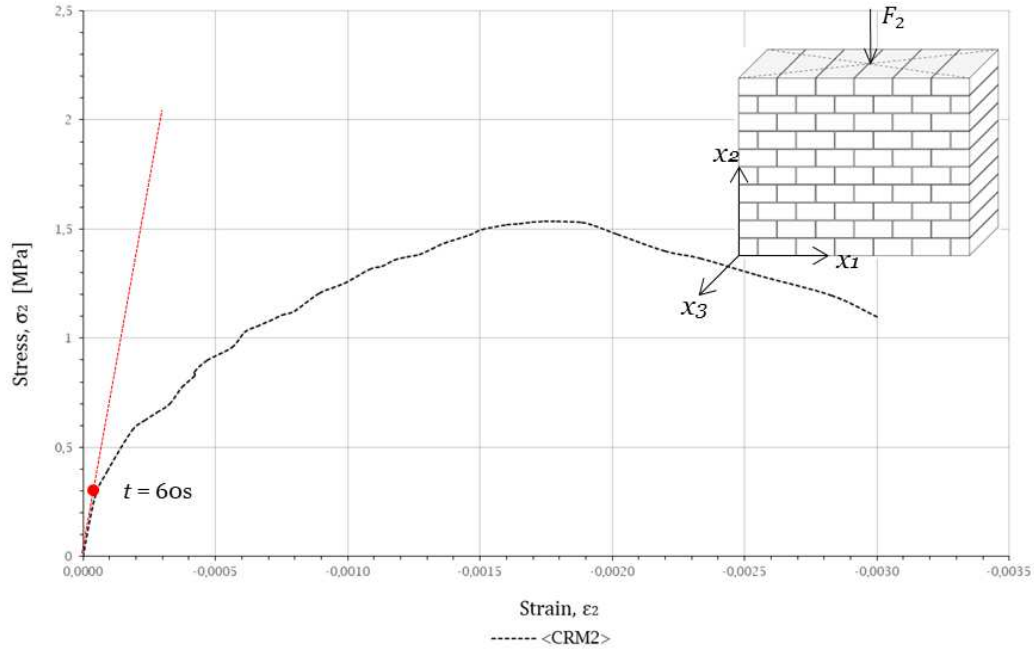


Figure 5.53: Mean compressive behaviour of CRM2.

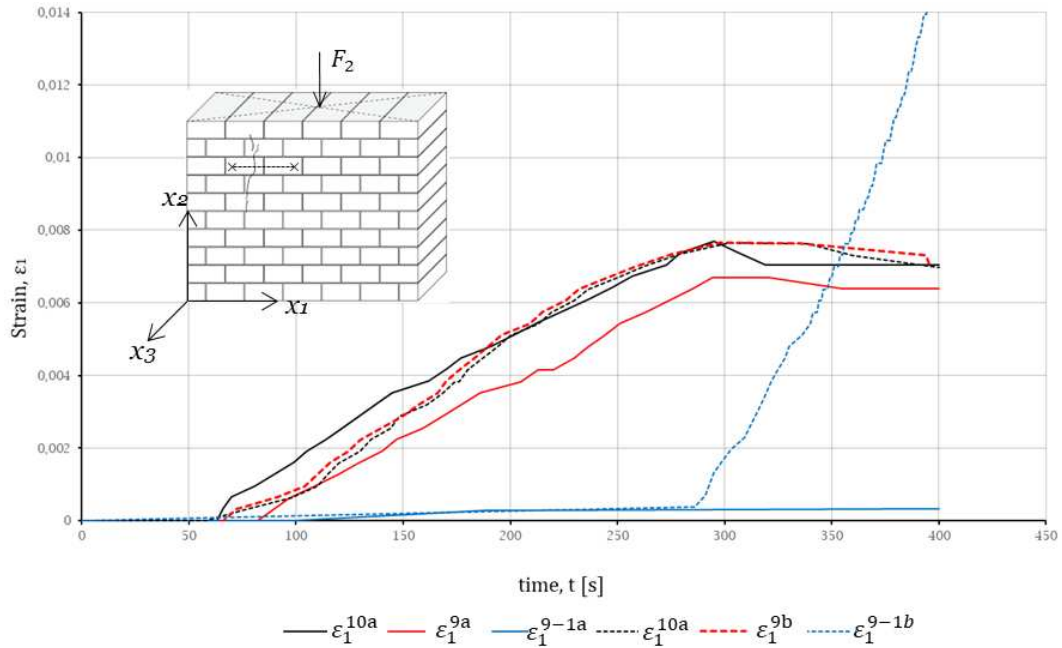


Figure 5.54: Time-strain plots for the identification of cracks initiation in CRM2.

The horizontal deformations of vertical joint on the external side of block of the ninth row were analysed to verify that the crack was limited to the reinforcement area and not due to failure of the upper corner for the specimen. Horizontal strain ε_1^{9-1b} does not show any deformation until the second 266, while ε_1^{9-1a} does not record any significant

deformation throughout the entire time of the test (see Figure 5.54), confirming that the failure mechanism corresponds to the development of cracks observed at second 100 s.

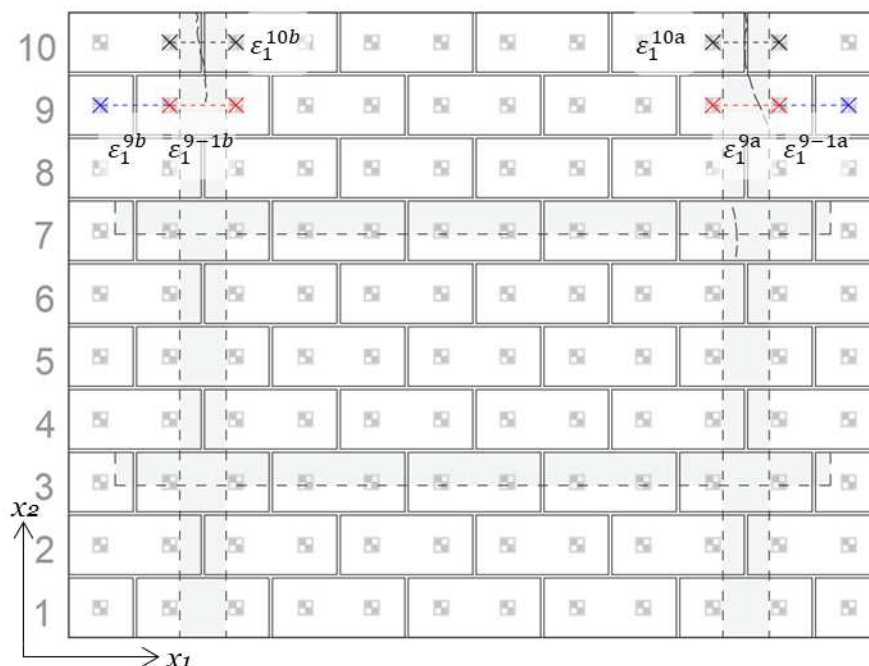


Figure 5.55: Scheme for horizontal deformation of CRM2 specimen.

5.2.3 BFRP-concrete (mixed) frame reinforced earth-masonry panels

In the mixed reinforcement frame panel (MRM), the vertical reinforcement is constructed similarly to that of the concrete frame masonry panels. However, the horizontal reinforcement beam is replaced with a basalt-fibre reinforced polymer (BFRP) net insert in the bed joint between the third and fourth rows and between the seventh and eighth rows. This configuration replicates the geometry of the concrete frame reinforcement. Two masonry panels, MRM1 and MRM2, were subjected to compression testing, with the same setting as the other earth masonry panels. The resistance of the specimens and their mechanical response to the load were subsequently analysed in order to understand if the earth masonry panel can benefit from a BFRP reinforcement.

In the MRM1 specimen, the first visible signs of damage were recorded at around time $t^1 = 70s$, upon reaching 66% of the maximum load, corresponding to the last mortar joint of the tenth course of blocks. The fracture, as in the case of panels with concrete frames, can be attributed to the different stiffnesses of the materials concentrated in areas where the masonry accommodates the reinforcement. Other localized damages can also be identified on the fourth brick of the eighth course, in the outer third of the brick, and on the fifth block of the seventh course, in its central third. As in the case of

unreinforced masonry, the cracks can be attributed to the geometry of the blocks, which imposes variable resistant sections of material within the same block: the eighth course consists of the Block-1 type, with the external semicircles, while the seventh consists of Block-2 type blocks, with central holes (see Figure 5.56). Additionally, the BFRP net is positioned in the mortar joint between the two courses of bricks. The redistribution of stresses can lead to different deformation states with the damage of the weakest elements.

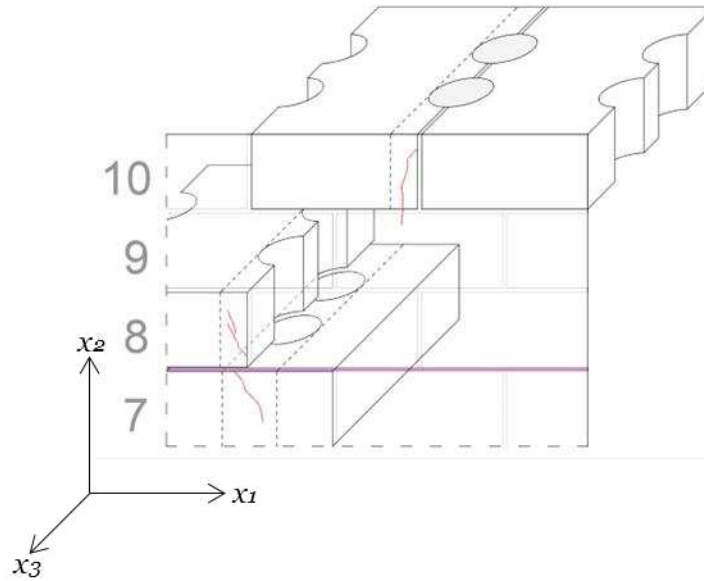


Figure 5.56: Cracks formation in MRM1 specimen under compressive load.

Although already visible at 66% of the maximum load, the damages do not appear to be extensive and are mostly associated with localized, rather than global, damage. At the 81st second, a localized damage forms on the second block of the tenth course. Also in this case, the damage is of limited extent, probably due to local failures that typically occur in the first and last courses of bricks in masonry. At time $t^2 = 97s$, it can be observed that, while the damage corresponding to the mortar joint of the tenth course has extended to the underlying block, the others present in the masonry panel have remained virtually unaltered. At the 127th second, having reached 96% of the maximum load, other localized damages in the fifth block of the seventh course led to the partial lamination of the damaged block. At time $t^3 = 144 s$, having reached the maximum load, the cracking state between the eighth and seventh courses of blocks remains unchanged, but other vertical cracks are observable in the external right portion of the vertical reinforcement (see Figure 5.57), aligned with the reduction of section of the blocks. At time $t^4 = 157 s$, the cracking pattern appears more pronounced, particularly with damages in the already damaged blocks across the mortar joint reinforced with the BFRP

net, leading to delamination and local crushing of the weakest elements. Unlike the previously tested specimens, the cracking pattern appears to be of reduced extent and distributed more extensively in the upper third of the masonry panel, above the reinforced mortar bed.

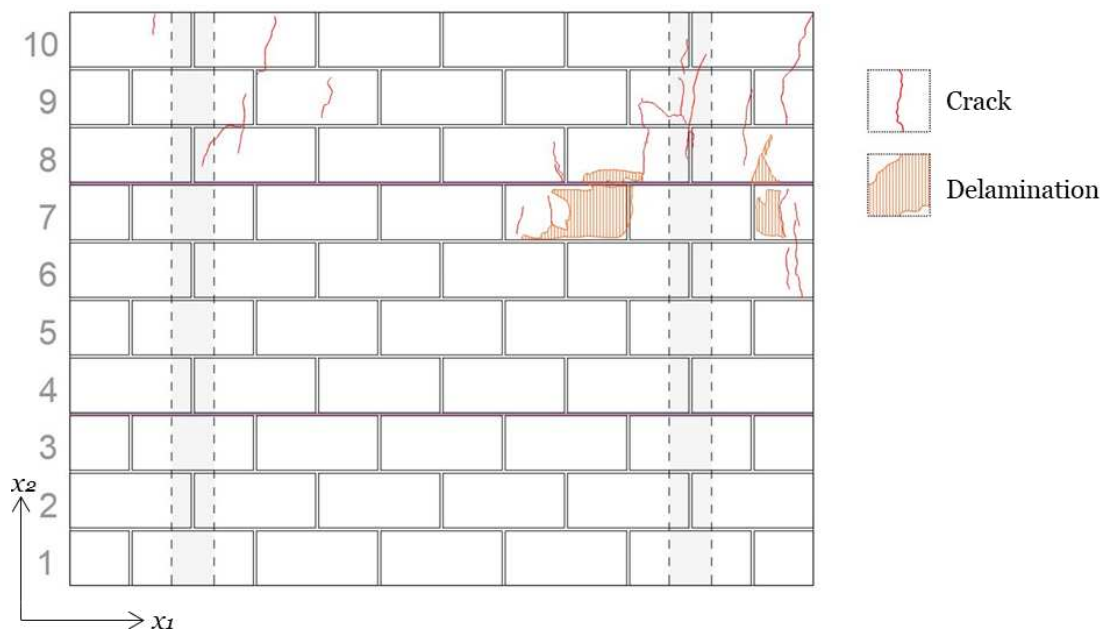


Figure 5.57: Crack pattern in CRM2 specimen under compressive load.

To understand the mechanical response of the MRM1 specimen subjected to vertical load, a mapping of the vertical strain of the masonry panel surface has been executed through the DIC method. Stress-strain curves were plotted. A linear phase can be observed in almost all the “columns” investigated, which extends from 0.7 to 1 MPa compressive strength. While from v1 to v3 there is only a slight change in stiffness which and almost no post peak phase, after reaching peak stress, v4 to v10 curves exhibit a gradual decline in stress, showing an extended softening phase. Progressively, from v4 to v10, there is a noticeable reduction in stiffness after the elastic phase, with a less abrupt softening phase. This effect is particularly evident in columns v6, v7, and v10, where a higher level of post-peak deformation is registered, with peak stress reaching 2.4 MPa (see Figure 5.58). Maximum vertical deformation is reached at $\epsilon_{v7} = 0.0114$, more than double of the higher strain registered on the unreinforced and concrete frame reinforced panels. This can be attributable to the BFRP net which helps to propagate cracks and to avoid instant failure, which is reflected on the almost ductile response, rather than the typical brittle one of the masonry systems.

Mean vertical strain was determined as $\langle \epsilon_2^{vn} \rangle = 0.0065$ with an elastic modulus of $E_2^{MRM1} = 4125 \text{ MPa}$. The elastic modulus of the MRM1 specimen was determined from

the initial linear portion of the mean stress-strain curve. This elastic phase, representative of the undamaged material behaviour, was identified up to a compressive stress of $\sigma_2 = 0.9 \text{ MPa}$, reached at the 36th second (see Figure 5.59).

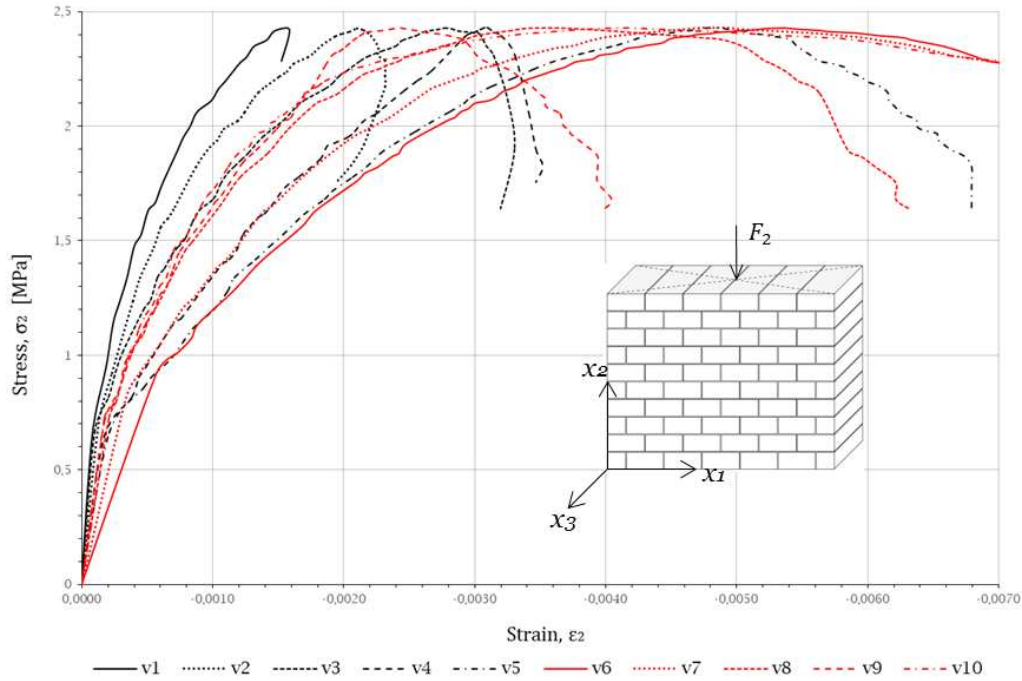


Figure 5.58: Stress-strain plot of different vertical columns in MRM1 specimen tested in compression.

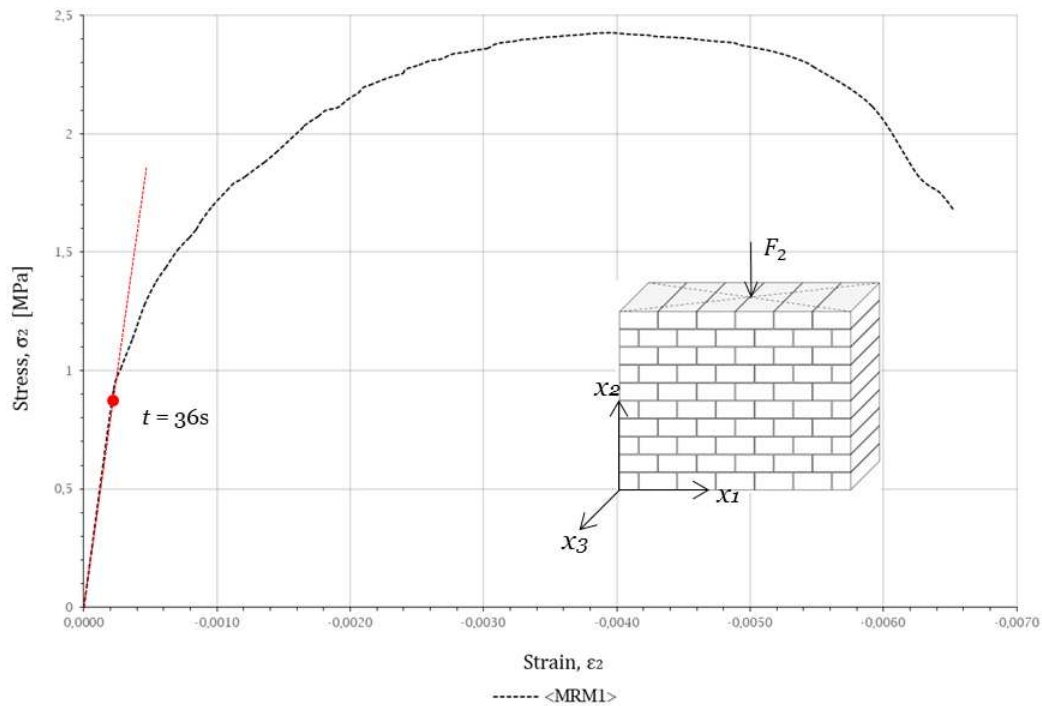


Figure 5.59: Mean compressive behaviour of MRM1.

Subsequently, a nonlinear deformation phase follows, characterized by micro-crack formation and crack propagation, leading to a reduction in the masonry system stiffness. To investigate the origin of these micro-cracks, horizontal strain measurements between internal points were analysed. A comparison was made between the time-strain development and the time at which the slope of the stress-strain curve changes.

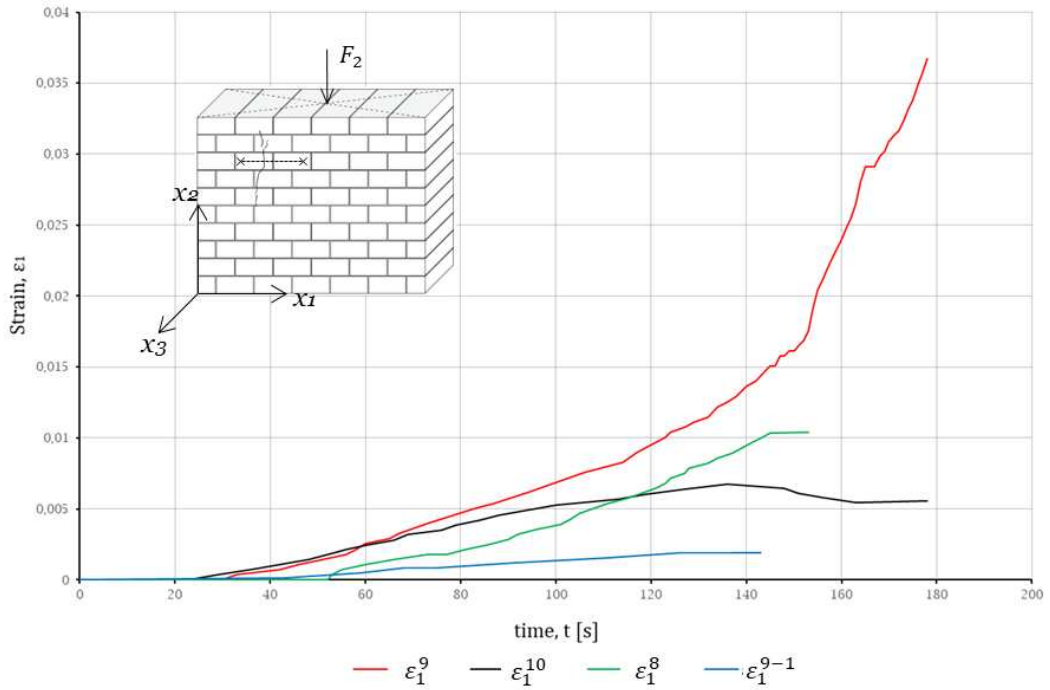


Figure 5.60: Time-strain plots for the identification of cracks initiation in MRM1.

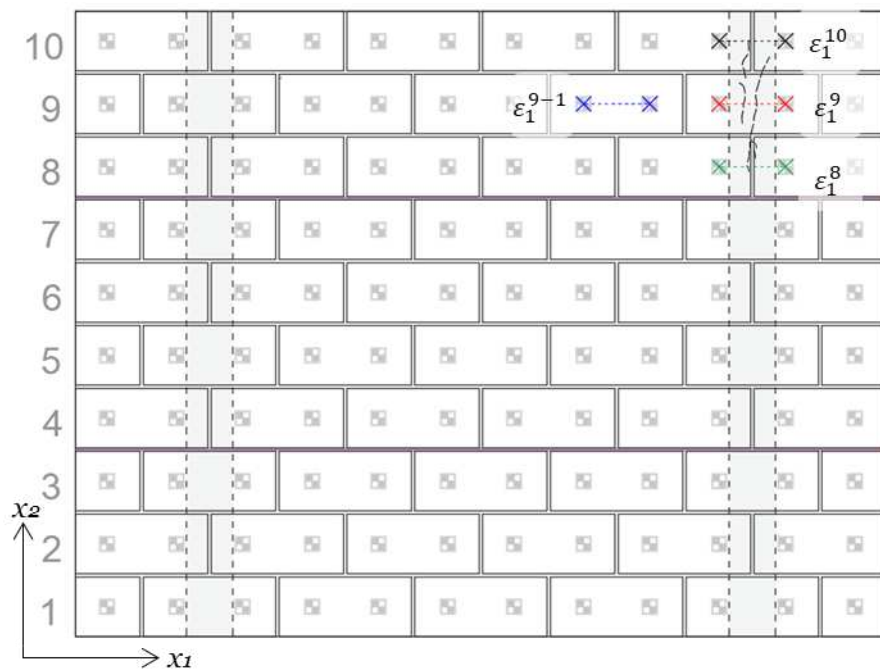


Figure 5.61: Scheme for horizontal deformation of MRM1 specimen.

The MRM1 specimen exhibited a loss of stiffness upon reaching a compressive stress of $\sigma_2 = 0.9 \text{ MPa}$. From time $t = 31 \text{ s}$, an exponential increase in horizontal strain was observed in the sixth brick of the ninth row (ε_1^9), indicating localized fracture. Analysing the horizontal deformations of the head joints corresponding to the tenth and eighth courses, it can be noted that the fracture did not originate within the block but in the joint at the tenth course of bricks, at time $t = 23 \text{ s}$, with a linear increase. Conversely, the joint of the eighth course did not show any deformation until the 52nd second (see Figure 5.60). To confirm the localization of damage to this area, the horizontal deformation of the adjacent block (ε_1^{9-1}) was investigated. Minimal increments were detected throughout the test, providing no significant information to indicate any form of failure.

A second masonry panel (MRM2) with a mixed frame was subjected to a compressive load to evaluate its mechanical behaviour. At time $t^1 = 240 \text{ s}$, when 51% of the load was reached, an initial crack was observed on the first brick of the tenth course (see Figure 5.62). The fracture appeared to develop approximately one-third of the length of the brick head (Block-1 type) at the outer semicircles. The adjacent mortar joint showed no apparent signs of damage, despite the presence of the vertical concrete reinforcement. At 270 seconds, the initiation of a second crack was recorded, again on the tenth course, on the fifth block. In this case as well, the brick serving as the formwork for the vertical reinforcement appeared damaged in the external portion corresponding to the section portion. As before, the mortar joint did not appear damaged. In both cases, these were localized, limited-extent lesions, likely attributable to the block geometry and the different stiffnesses of the materials used (SCEB and concrete).

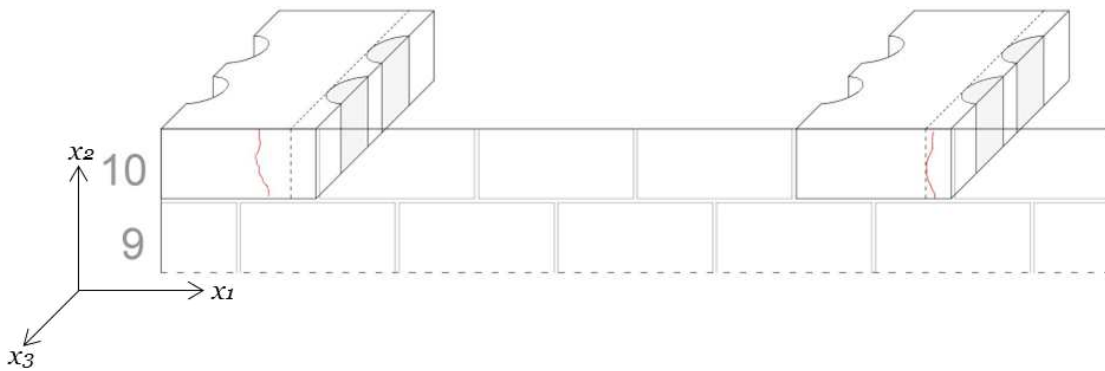


Figure 5.62: Cracks formation in MRM2 specimen under compressive load.

Up to time $t^2 = 450 \text{ s}$, with the achievement of 90% of the maximum load, the crack pattern still appeared limited to the two blocks of the tenth course, with the only difference being the formation of a crack at the vertical mortar joint of the first block.

From time t^2 onwards, the second block of the ninth course appeared cracked at the central holes (Block-2 type), which house the vertical reinforcement. When time $t^3 = 592\text{ s}$ was reached – corresponding to the maximum load supported by specimen MRM2 – the crack pattern appeared unaltered, with the lesions remaining of limited extent and size. Micro-cracks present on the brick course where the load is applied are a common phenomenon due to higher stress concentration given by the direct contact with the loading plate. However, in this reinforcement configuration, the BFRP can help to redistribute the stress across the panel, enhancing the tensile capacity of the earth masonry system, thus leading to the formation of a limited crack pattern.

At 615 seconds, the second brick on the tenth row appeared cracked in the middle of its lower half. The crack appeared to propagate along the underlying mortar joint aligned with the second brick of the eighth row, also in the middle. At the same time, the crack from the second block of the ninth row continued to expand towards the first block of the eighth row. Between 650 and 700 seconds, with the load reduced to the 80% of its maximum, vertical cracks appeared below the seventh brick course (above which the BFRP reinforcement is positioned). The cracking appeared on the first vertical mortar joint of the brick course and propagated onto the first block of the sixth course, damaged at one-third and two-thirds of its extent (see Figure 5.63). At the end of the test – with the maximum load supported reduced to 70% of the maximum load – the crack pattern remained unaltered in its expansion on the surface of the specimen.

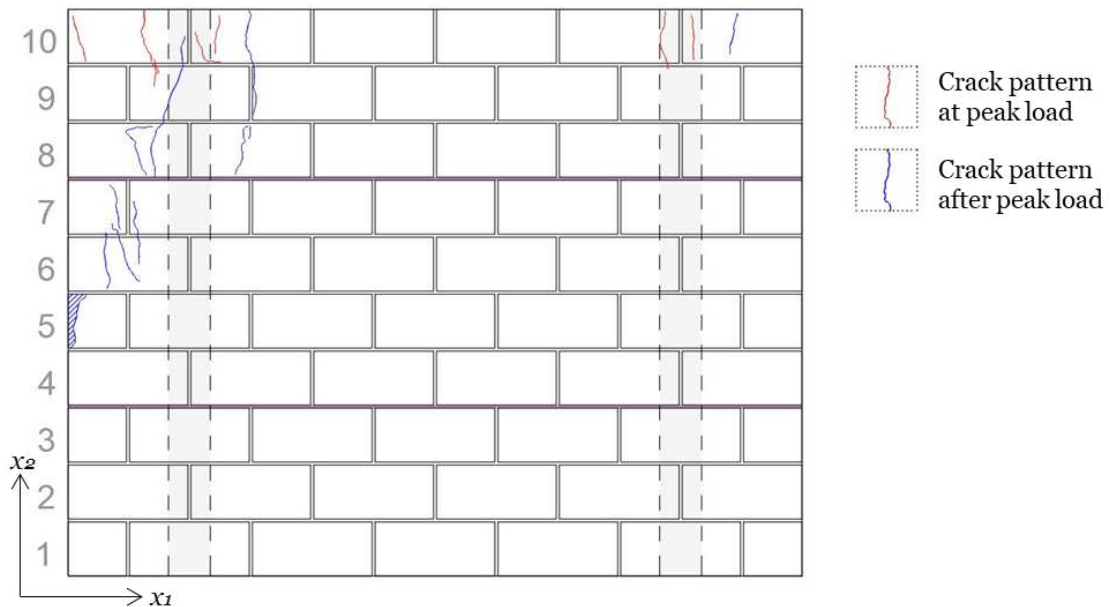


Figure 5.63: Crack pattern in CRM2 specimen under compressive load.

To analyse the mechanical behaviour of MRM2 under vertical loading, vertical deformations were studied using Digital Image Correlation (DIC). Deformations were

measured on 'columns' identified between the second and ninth brick courses (see Figure 5.31) to assess deformations generated by stress concentrations due to boundary conditions, and to extract stress-strain curves. A linear phase common to all curves is observed in the range between 0.7 and 1 MPa of compressive stress σ_2 . The maximum compressive strength was reached at 2.44 MPa, consistent with specimen MRM1 (see Figure 5.64). Columns v8 to v10 (describing the extreme right portion of the specimen) show limited post-peak deformability; however, after the linear phase, the mechanical response indicates a loss of stiffness comparable to other areas of the masonry panel. Although relatively constant, the largest maximum deformations are recorded by columns v1 to v5, which describe the left portion of the specimen where the crack pattern developed. The post-peak phase suggests a stress redistribution due to reinforcement.

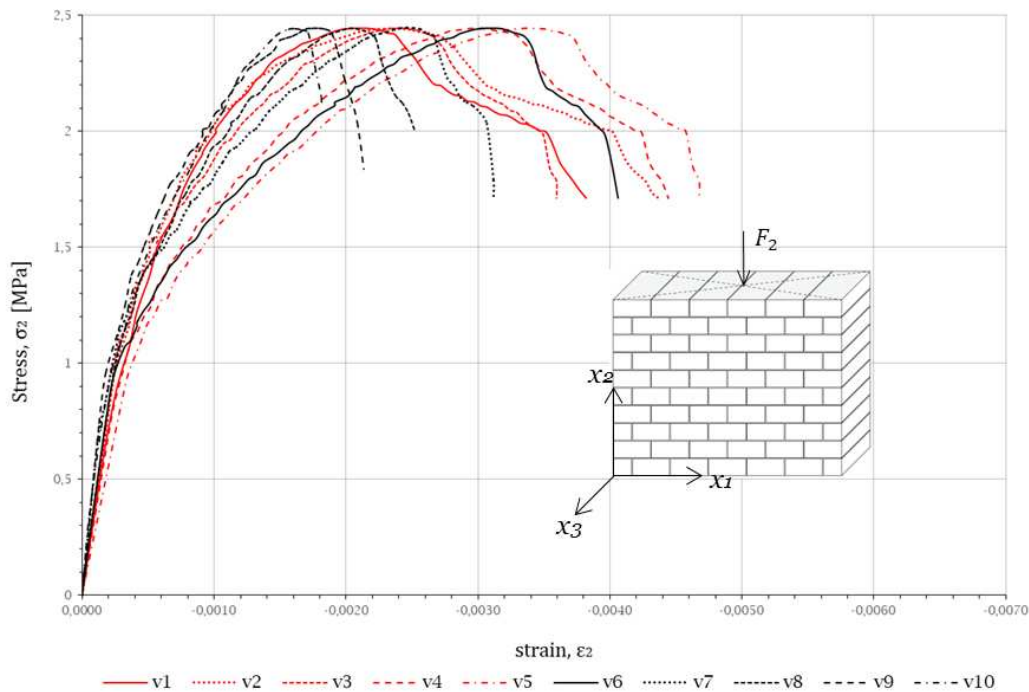


Figure 5.64: Stress-strain plot of different vertical columns in MRM2 specimen tested in compression.

The average deformation is $\langle \epsilon_2^{vn} \rangle = 0.0045$, lower than specimen MRM1, but with a more uniform distribution of the maximum vertical deformation along the specimen surface when compared to the distribution in MRM1. This indicates a greater deformability compared to unreinforced masonry specimens and those reinforced with an all-concrete frame.

The elastic modulus was determined as the tangent of the initial linear segment of the averaged stress-strain curve. This linear segment characterizes the elastic response of the material, preceding the non-linear deformation stage associated with crack initiation and

propagation. Such a non-linear behaviour results in a progressive reduction of the structural system's stiffness. The calculated elastic modulus is $E_2^{MRM2} = 3950 \text{ MPa}$. The onset of the nonlinear regime was observed approximately at stress $\sigma_2 = 1 \text{ MPa}$ (see Figure 5.65). To corroborate this transition from linear to non-linear behaviour, internal horizontal deformations within the panel were monitored. A time-strain correlation with the applied load was conducted to facilitate this verification.

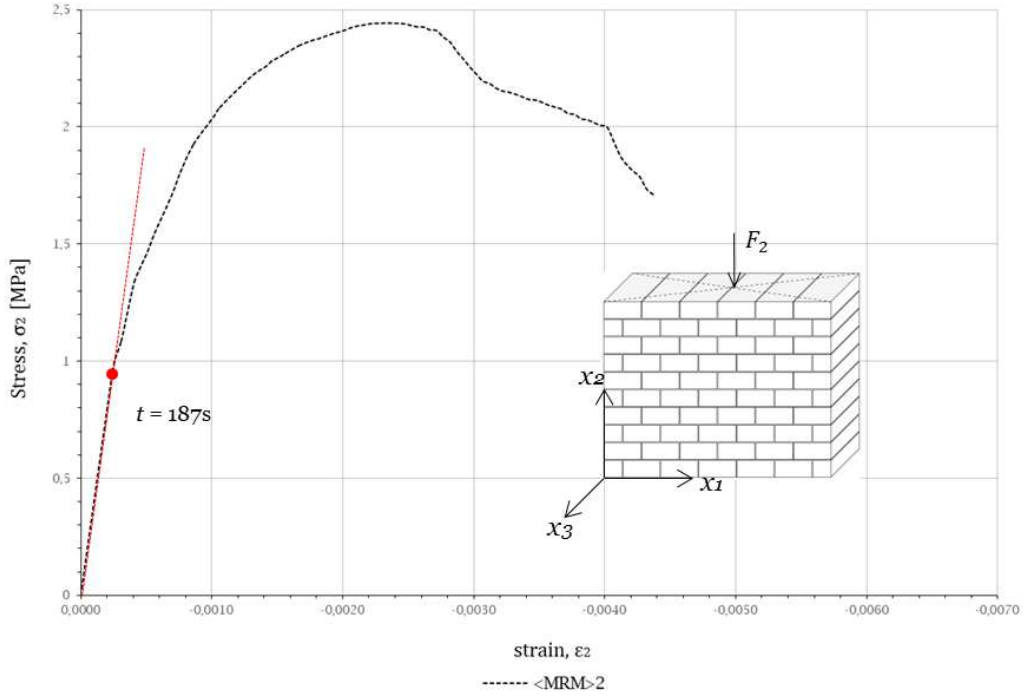


Figure 5.65: Mean compressive behaviour of MRM2.

Specimen MRM2 exhibited a loss of stiffness at a compressive stress $\sigma_2 = 0.95 \text{ MPa}$, reached at time $t = 187 \text{ s}$. The measurement of horizontal deformations, ε_1^{10b} and ε_1^{10a} , indicate that at 100 seconds, the first and fifth blocks of the tenth row were damaged (as observed in Figure 5.62). Despite this, the panel did not show a loss of stiffness at stress $\sigma_2 = 0.45 \text{ MPa}$, stress reached at 100 seconds. This confirms that the observed and recorded horizontal deformations are due to localized crushing caused by the loading plate in contact with the brick course. The hypothesis of crack extension limit only to the tenth course is supported by the horizontal deformation measurements of the mortar joints in the ninth course vertically aligned to the damaged blocks, which show no significant increase in strain around 100 seconds (ε_1^{9b} and ε_1^{9a}). However, the sixth block in the ninth row does exhibit an increase in strain at around 100 seconds (ε_1^{9-1a}) (see Figure 5.66). It can be hypothesized that the loss of stiffness is due to localized damage in the masonry upper portion of the panel, included between the seventh course (where the BFRP mesh is located) and the ninth course.

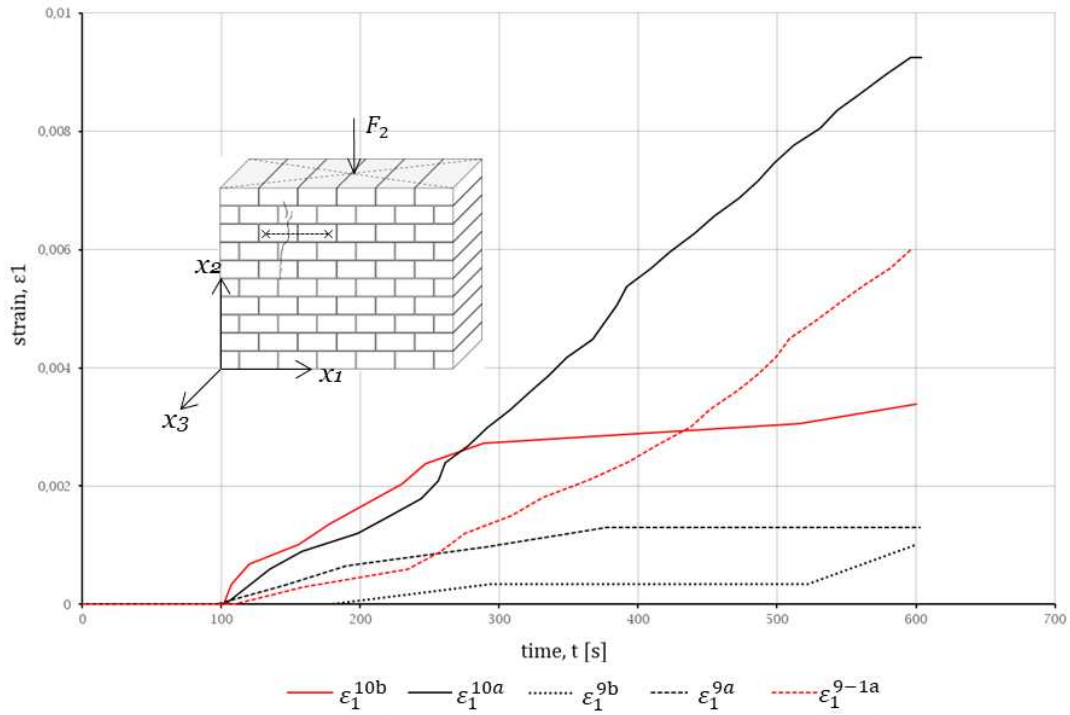


Figure 5.66: Time-strain plots for the identification of cracks initiation in MRM2.

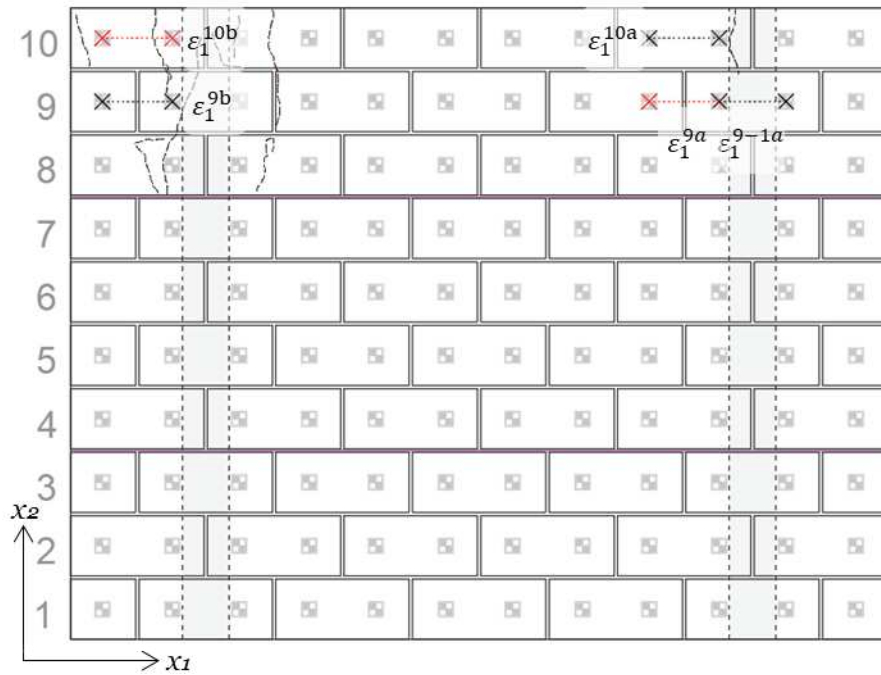


Figure 5.67: Scheme for horizontal deformation of MRM2 specimen.

A comparison between masonry panel typologies was conducted. The plot illustrates the mean compressive behaviour of earth masonry panels under various reinforcement conditions, as indicated by the stress-strain curves (see Figure 5.68). The reinforcement typologies are:

- UM1 and UM2 (Red curves): Unreinforced masonry.
- CRM1 and CRM2 (Gray curves): Concrete frame-reinforced masonry.
- MRM1 and MRM2 (Blue curves): Masonry reinforced with BFRP (Basalt Fibre Reinforced Polymer) nets and concrete frames.

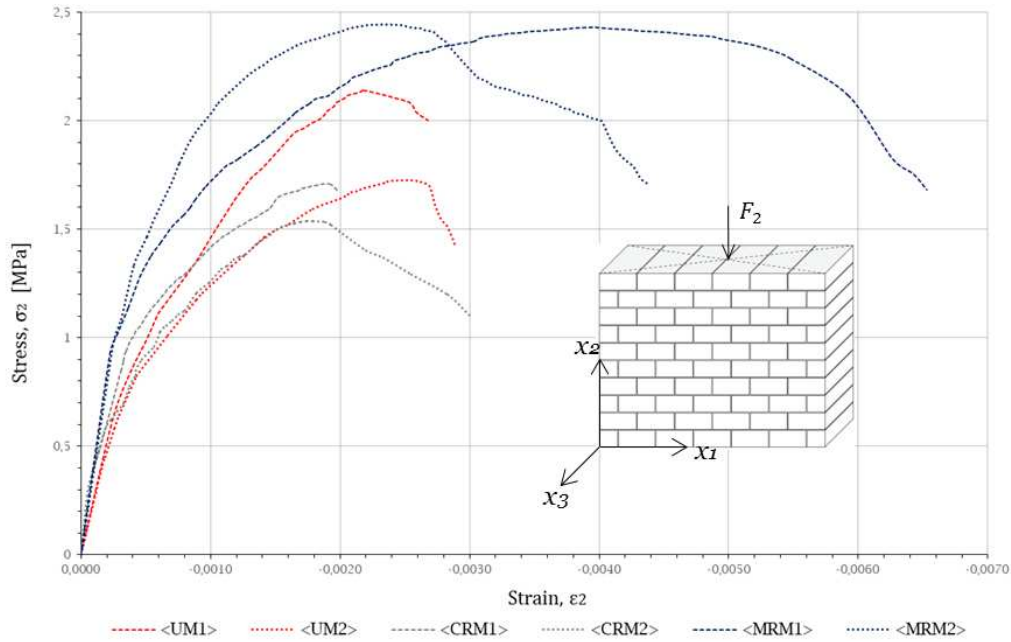


Figure 5.68: Comparison of compressive behaviour of earth masonry panels.

<i>Specimen</i>	<i>Compressive strength</i>	<i>Elastic modulus</i>
	[N/mm ²]	[N/mm ²]
UM1	2.14	2540
UM2	1.73	2240
CRM1	1.71	4230
CRM2	1.54	4270
MRM1	2.43	3950
MRM2	2.44	4125

Table 5.8: Compressive strength and Elastic modulus earth masonry panels.

Panels incorporating BFRP nets as horizontal reinforcement demonstrated superior resistance to vertical loads compared to both unreinforced and concrete frame-reinforced masonry. Conversely, concrete reinforcement appeared to reduce the mechanical resistance to compressive loads of unreinforced earthen masonry, possibly due to differences in material stiffness.

The elastic modulus indicated that CRM and MRM specimens were stiffer than unreinforced masonry. Furthermore, the comparable moduli between CRM and MRM suggest that the increased stiffness in the linear phase is primarily attributable to the vertical reinforcement, a common element in both reinforcement typologies (see Table 5.8).

MRM specimens exhibited linear behaviour up to approximately $\sigma_2 = 1 \text{ MPa}$, specifically at 0.34 and 0.38 of the maximum loads for MRM1 and MRM2, respectively. This linear range was higher than that observed in UM and CRM specimens. Unreinforced masonry demonstrated an elastic phase up to 0.28 and 0.26 of the maximum loads for UM1 and UM2, respectively. Concrete frame-reinforced specimens exhibited a lower elastic range, comparable to unreinforced masonry, with 0.21 and 0.26 of the maximum loads for CRM1 and CRM2, respectively.

The comparable percentage of peak load at which UM and CRM specimens exited the linear phase suggests that while the frame increased stiffness, it did not directly enhance the compressive strength of the wall. In contrast, the extended linear range of MRM specimens, particularly at higher stresses, indicates that the fibre net, as the distinguishing element between the two reinforcement typologies, may contribute to stress redistribution within the masonry, delaying the onset of localised damage and associated stiffness loss. Up to the peak load, all specimens exhibited a gradual loss of stiffness due to crack development and progressive damage.

The non-linear phase in UM1 and UM2 confirmed the brittle nature of the material. In concrete frame-reinforced specimens CRM1 and CRM2, the transition to non-linear behaviour occurred earlier than in other panels, suggesting potential incompatibility between the reinforcement and earthen masonry. This slight drop in stiffness, contrary to the compressive strength of the individual materials, indicates that despite concrete's compressive strength, it may not be optimal for earthen masonry due to stiffness disparities that can lead to overall structural damage.

MRM specimens exhibited higher resistance to compressive loads (MRM1) and deformation capacity ε_2 (MRM2) up to peak load. The smoother and longer non-linear phase suggests improved stress redistribution and crack formation/distribution within the panel, likely attributable to the fibre net, the unique element differentiating the two reinforcement typologies.

The crack patterns observed at peak load in each specimen corroborate the findings from the stress-strain plots. CRM1 and CRM2 exhibited larger crack patterns, concentrated in areas where the concrete frame was constructed, which corresponds to the less steep

non-linear portion of the stress-strain curves. UM1 showed a limited crack pattern compared to UM2, primarily confined to layers 9 to 6. This aligns with the more rigid non-linear behaviour of UM1. UM2, on the other hand, exhibited cracks extending from layers 9 and 10 to 4 over a larger area. As previously described, these cracks appeared along head mortar joints and reduced sections of the blocks, depending on their typology (Block-1 and Block-2).

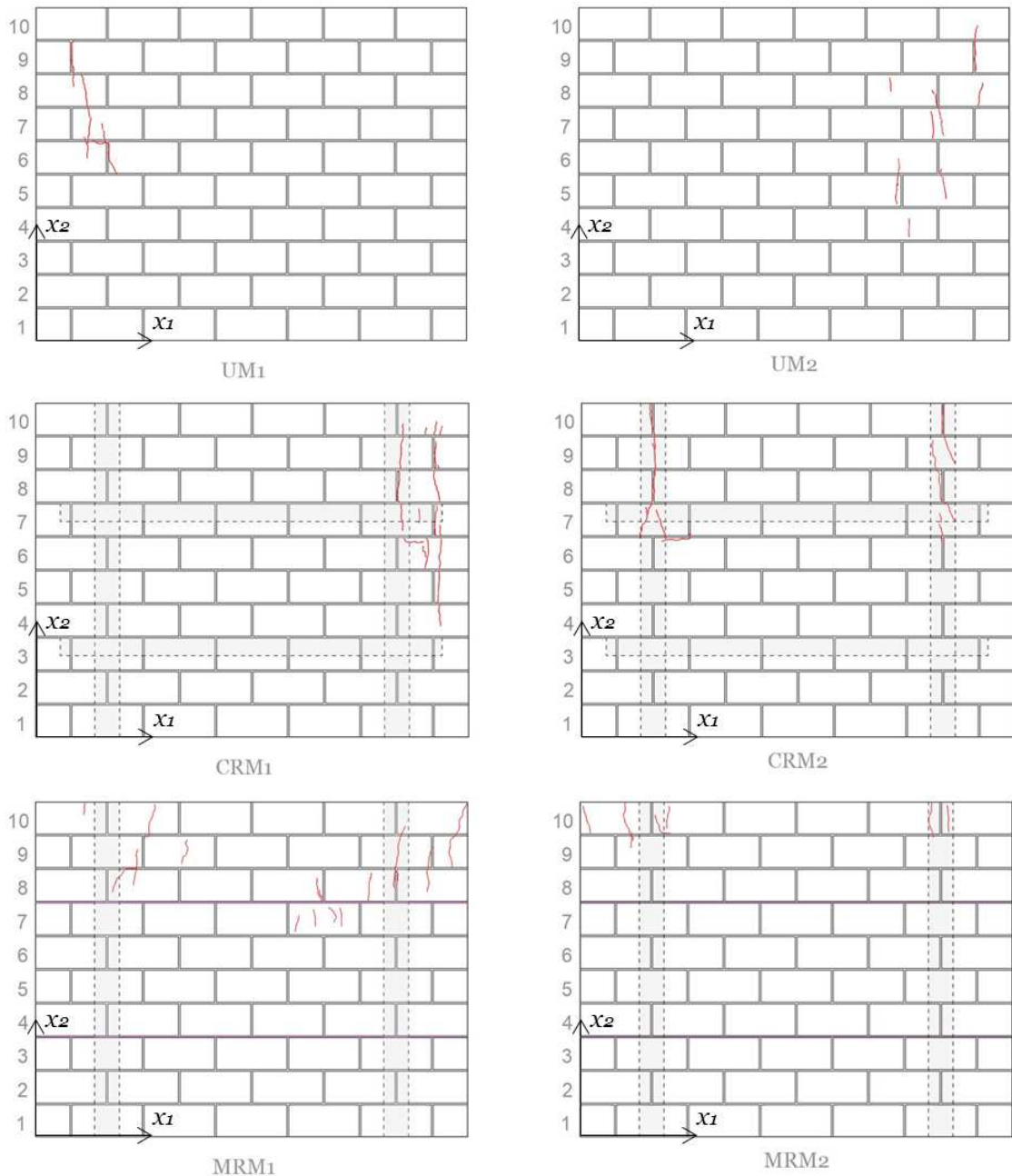


Figure 5.69: Crack pattern at peak load of earth masonry panels, under compressive load.

In mixed frame-reinforced masonry panels, crack development varied in terms of typology and localisation. MRM1 displayed diffuse localised cracks, more evenly

distributed from courses 10 to 7, indicating local block failures due to tension. In MRM2, localised cracks were limited to the upper layer, potentially influenced by contact with the loading bars. The distinct crack patterns were reflected in the differing non-linear behaviour of the specimens (see Figure 5.69).

The post-peak phase confirmed the distinct mechanical response of the MRM specimens (see Figure 5.70). Unreinforced masonry panels experienced a rapid drop in stress resistance, confirming the brittle nature of the SCEB masonry. When reinforced with a concrete frame, the post-peak drop was less abrupt in CRM2, indicating that the frame contributed to improved stress redistribution within the panel. However, in CRM1, the reinforcement's contribution appeared less significant, possibly due to inaccuracies during specimen construction.

In MRM specimens, the post-peak phase exhibited a gradual reduction in stress. This can be attributed to a more controlled failure mechanism facilitated by the fibre net used as horizontal reinforcement, resulting in minimal load-bearing capacity beyond the maximum load.

The final crack pattern of the unreinforced masonry revealed diffuse damage along the entire panel height, primarily consisting of vertical cracks and localised damage due to block failures. Conversely, despite being less extensive in height, the crack pattern of CRM specimens appeared more widespread and pronounced. Additionally, horizontal fractures were observed along the first reinforcement beam, likely resulting from differential stiffness and stress concentration. Furthermore, the direct contact between the concrete beam and the masonry, acting as permanent formwork (Block-3 block typology), may have induced compressive stresses leading to crushing of the supporting masonry.

When mixed reinforcement (BFRP net/vertical concrete reinforcement) was employed, cracks developed over a more distributed area up to the seventh block layer, not solely influenced by reinforcement geometry. Minor crack development was also observed in the sixth course. Overall, the cracks appeared less pronounced, indicating that the BFRP net contributed to crack propagation control and post-peak response stabilisation, enhancing the ductility of the masonry system.

To investigate the contribution of the reinforcement to the masonry and to verify the redistribution of internal stresses, horizontal deformations were measured at different heights of the panels. Specifically, the third and seventh courses were considered to analyse the effects of the different types of horizontal reinforcement, while the second, fifth, and ninth layers of blocks were selected to measure the deformations across the

reinforcements. The last course was finally considered to evaluate the impact of localized damage.

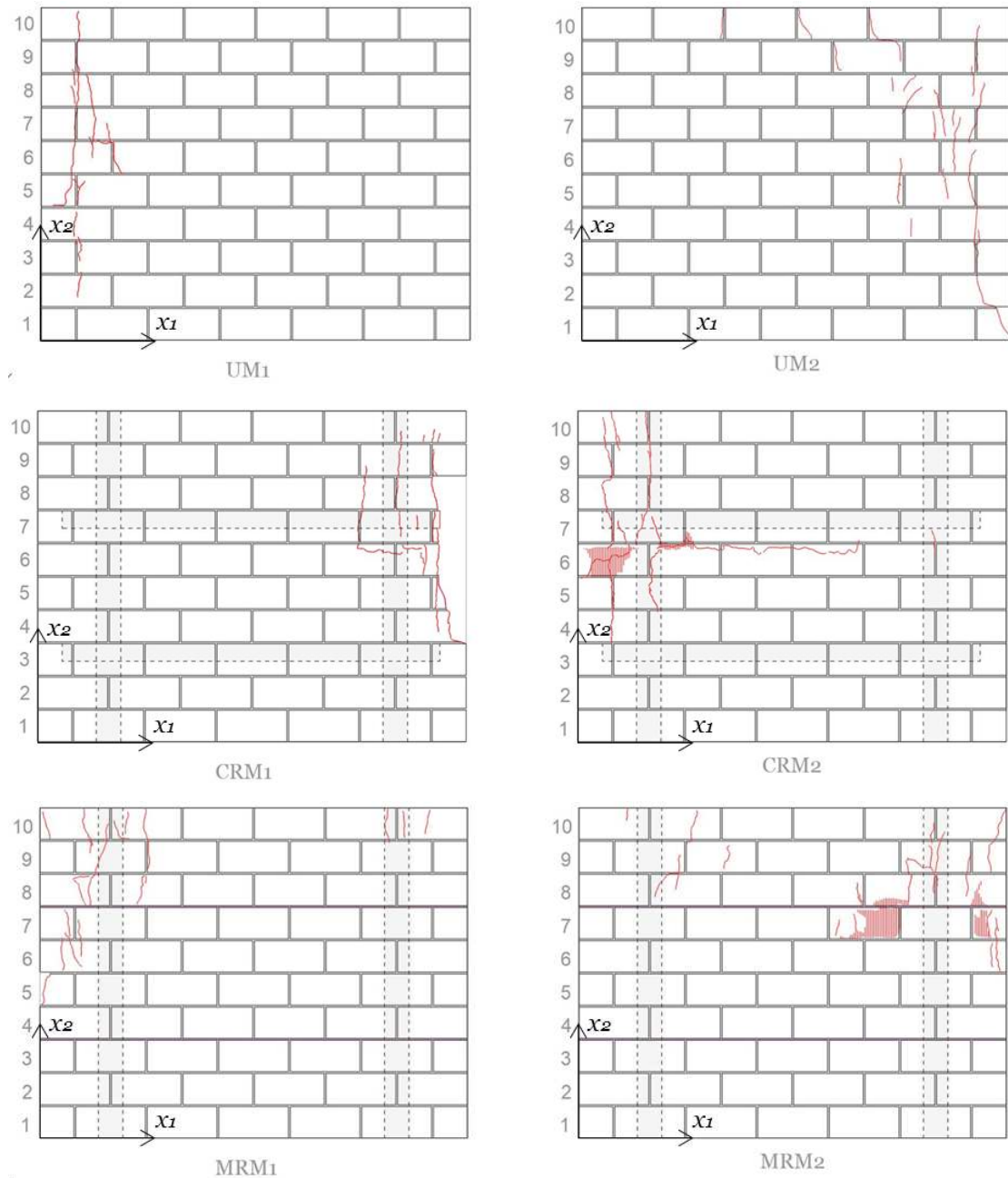


Figure 5.70: Crack pattern at end of test of earth masonry panels, under compressive load.

Considering the different behaviour shown by the different types of panels in the non-linear and post-peak phases, the horizontal deformations were analysed both at the maximum load and at the end of the test.

The UM panels exhibited a nearly uniform deformation distribution up to the 5th course, followed by a gradual decrease between the second and third joints, with a maximum strain of $\varepsilon_1 = 0.002$ (see Figure 5.71).

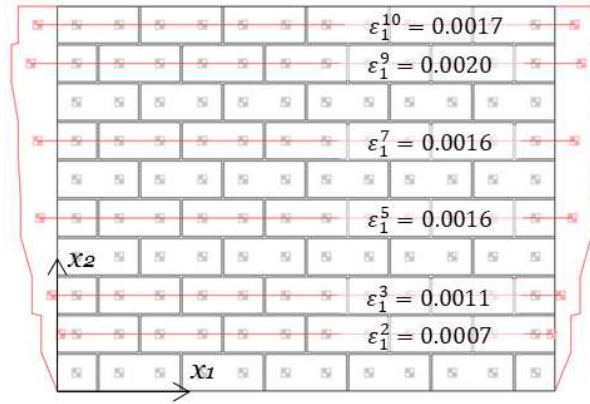
CRM panels demonstrated a wider range of horizontal deformation and a distribution distinct from unreinforced panels. This reflects the brittle behaviour typical of unreinforced masonry, where horizontal deformation rapidly increases as the material approaches its peak stress. Deformations from the tenth to seventh course were more pronounced, while from the fifth to the second, they decreased consistently. In CRM1, the strain reduction from the seventh to the fifth course was 37% (maximum strain of $\varepsilon_1^7 = 0.0029$ at the seventh course), while in CRM2, it was 60% (maximum strain of $\varepsilon_1^9 = 0.0017$ at the ninth course). Deformations at the third course, where the second reinforcement beam is located, were insignificant and comparable to those of the second and fifth block layers. This indicates that the horizontal reinforcement at the seventh level redistributed most of the stresses at maximum load, altering the masonry deformation.

In MRM panels, upon reaching the maximum force, the deformation pattern differed. Maximum deformations were not achieved at the seventh course, where the first BFRP net was located, but at the ninth layer of blocks, while maintaining comparable deformation values (in MRM1 $\varepsilon_1^7 = 0.0037$ and $\varepsilon_1^9 = 0.004$; in MRM2 $\varepsilon_1^7 = 0.0023$ and $\varepsilon_1^9 = 0.0027$). Generally, a more ductile response was exhibited. As also observed in the crack pattern, this indicates a relatively constant deformation for the upper portion of the panel, represented by courses 7 to 9. At the fifth course, a deformation reduction of 35% in MRM1 and 22% in MR2 was observed. In both panels, a linear reduction in deformations was observed in the lower courses.

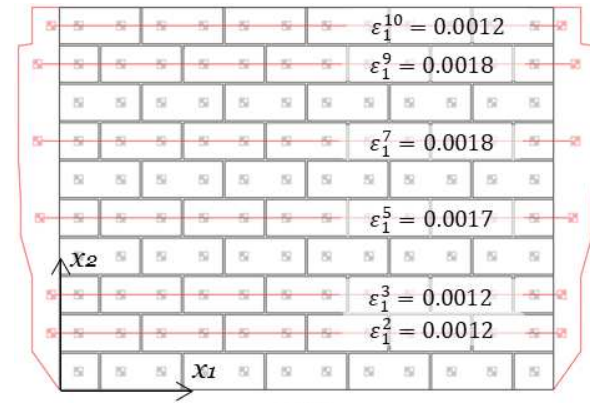
The different deformation distribution at maximum load provides insights into how the horizontal reinforcement aids in redistributing internal stresses. The redistribution appears more localized in the case of reinforcement with concrete beams, concentrated in the reinforcement zone and limiting deformations at lower levels. Conversely, in the case of the BFRP net, although stress transmission appears higher (larger deformations), the deformations were more significant and more widely distributed. The following representation shows horizontal deformation scale of 100 times).

Observations of the maximum deformation profile revealed that unreinforced panels (UM1 and UM2) exhibited their greatest deformations at the fifth and second courses, respectively. These differences can be attributed to variations in the crack patterns. Both specimens demonstrated limited post-peak deformation capacity, characteristic of brittle masonry behaviour. For panels with concrete frames, maximum deformation occurred at the seventh course (CRM1 $\varepsilon_1^7 = 0.0037$, CRM2 $\varepsilon_1^7 = 0.006$). The third course, where the second reinforcement beam was located, showed no significant increase in deformation between the maximum load and maximum deformation points. Moreover,

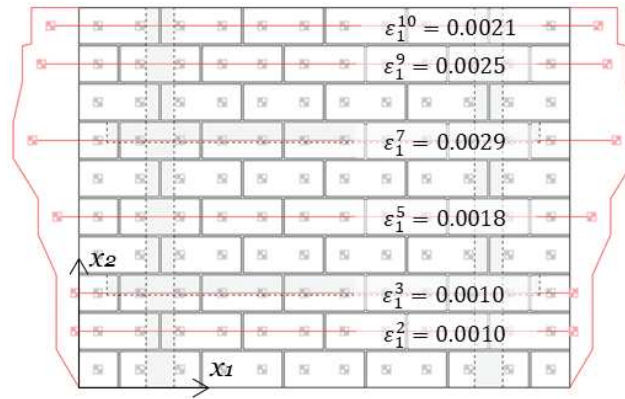
the deformation in the lower course remained relatively unchanged. This suggests that the second level of reinforcement may not have been fully activated under the applied vertical loads.



UM1



UM2



CRM1

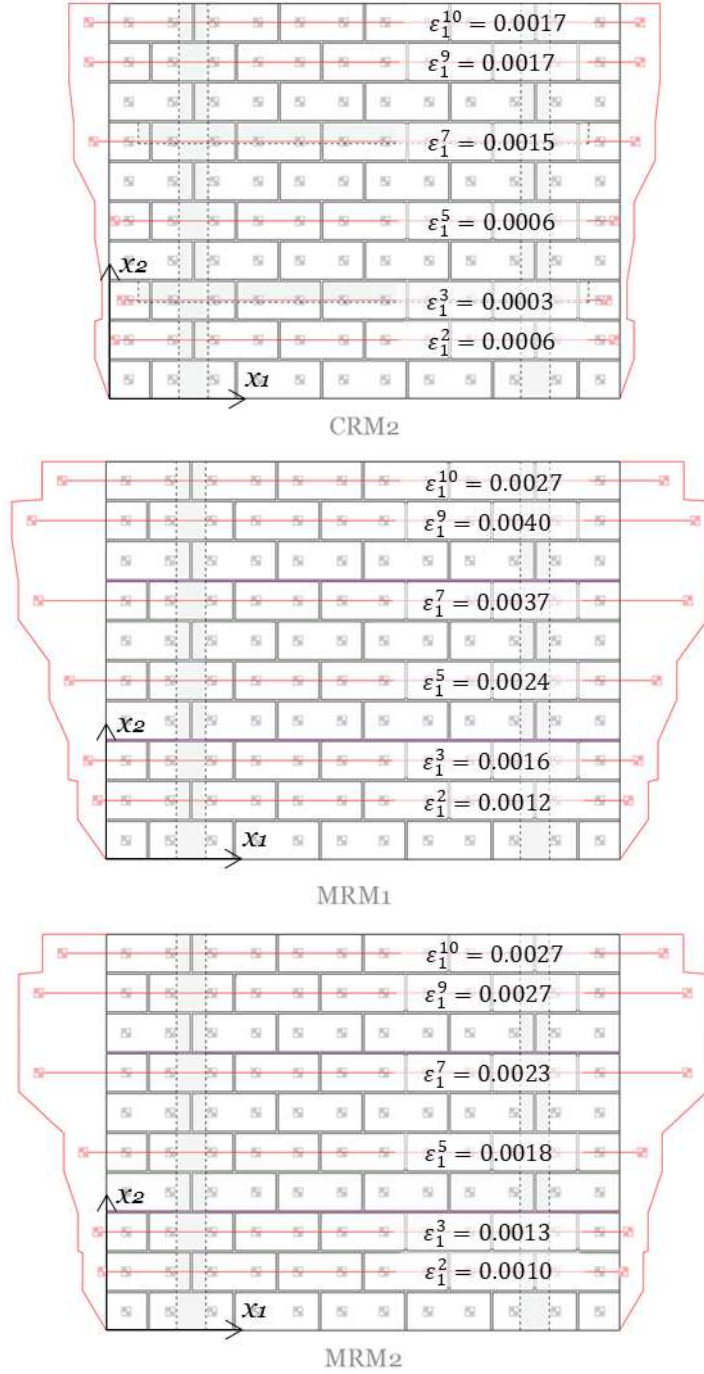
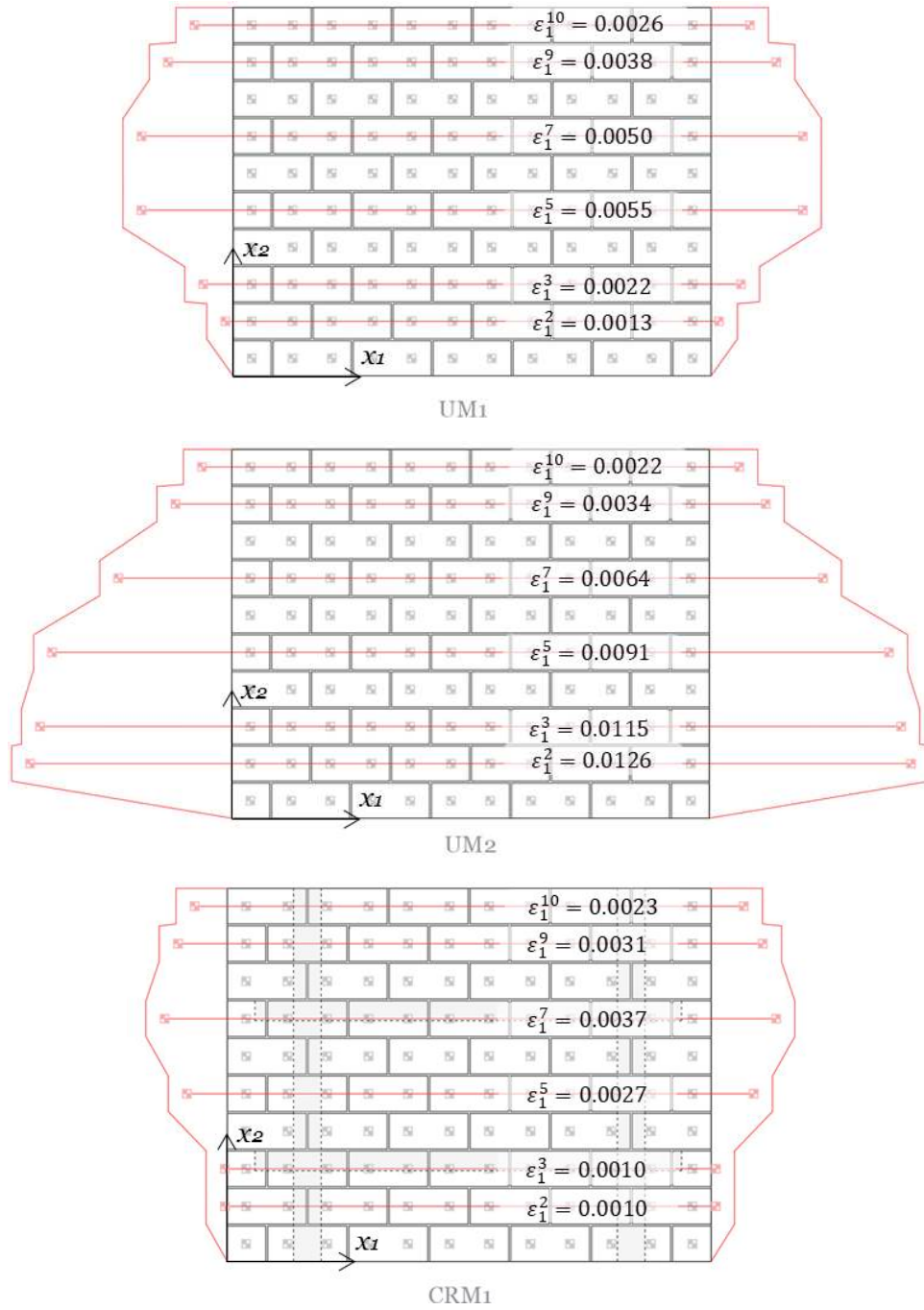


Figure 5.71: Horizontal deformation ε_1^n , registered at different block layers, for earth masonry panel at peak force under compressive load.

In panels with horizontal BFRP reinforcement, maximum deformation was also observed at the seventh layer of blocks (MRM1 $\varepsilon_1^7 = 0.0091$, CRM2 $\varepsilon_1^7 = 0.0057$). However, when comparing deformations at peak load and maximum deformation, an increase in horizontal strain was noted at the third row, while the underlying layer experienced less deformation. Since the second fibre net was positioned at the third row, this suggests that the second horizontal reinforcement was indeed activated. This observation supports the higher deformation capacity of the mixed reinforcement panel

typology. (The following representation, Figure 5.72, shows horizontal deformation scaled of 100 times).



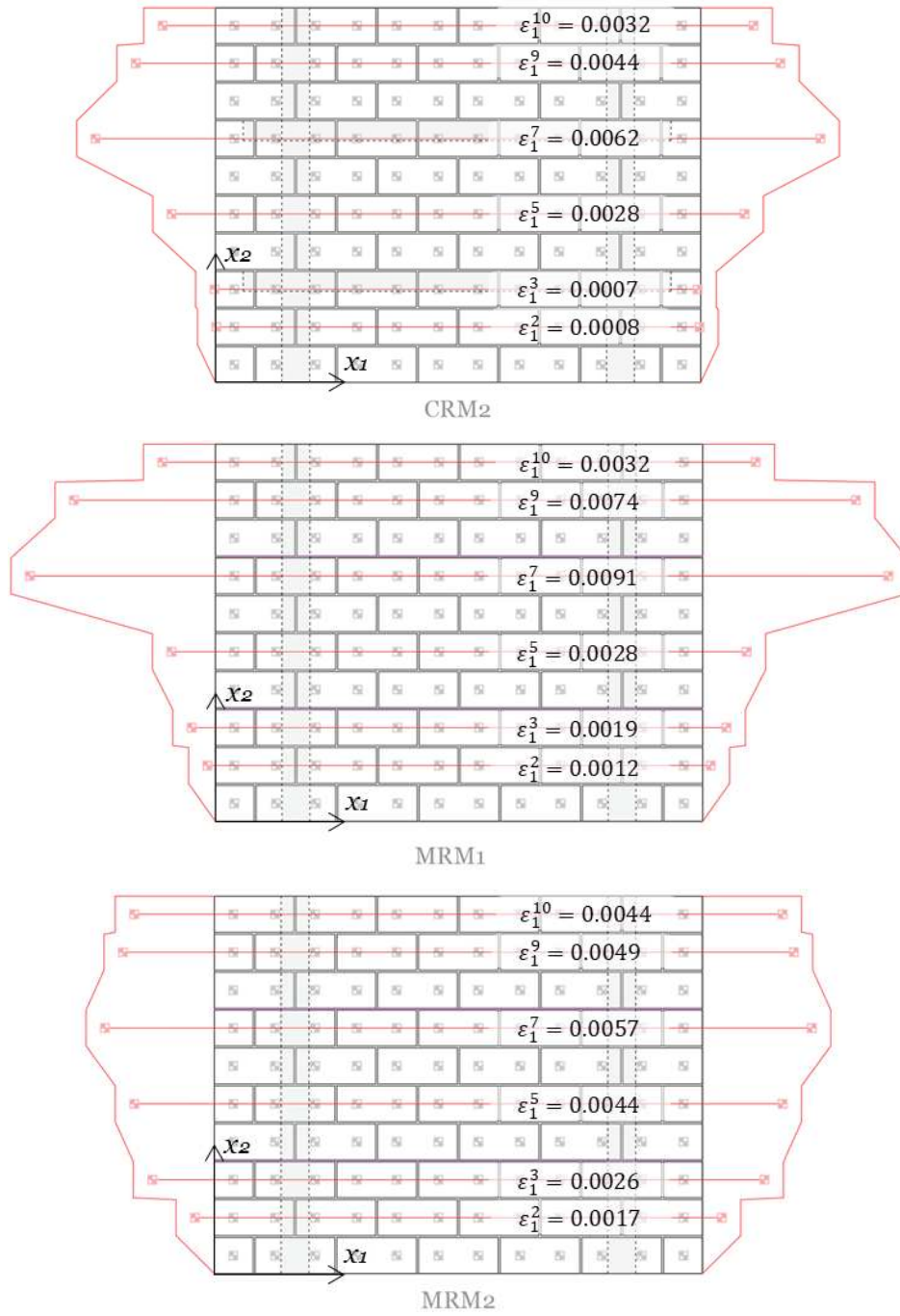


Figure 5.72: Horizontal deformation ε_1^n , registered at different block layers, for earth masonry panel at end-of-test, under compressive load.

The present research represents a first step in a broader investigation and primarily focuses on characterizing the essential mechanical behaviour of different masonry configurations under vertical loading. While the structural response to horizontal loads, particularly shear behaviour, was not addressed in the current experimental campaign, it remains a fundamental aspect for understanding the full contribution of the reinforcement frame to the masonry system. This will be the subject of future investigations.

Chapter 6 - Glass Fibre Reinforced Polymers (GFRP) Bars

This chapter investigates the use of Glass Fibre Reinforced Polymer (GFRP) bars as a substitute for steel rebars in vertical concrete elements of the reinforced masonry system presented in Chapter 4. As discussed, GFRP was selected for its improved compatibility with earthen materials, due to its lower stiffness compared to steel, aligning better with the mechanical behaviour of cement-stabilised compressed earth blocks.

However, the integration of GFRP bars in a cementitious matrix poses specific challenges related to adhesion, which is critical for ensuring effective load transfer and overall structural performance. Unlike steel, which benefits from natural mechanical interlock and chemical bonding with concrete, GFRP relies heavily on surface treatment to achieve sufficient bond strength. For this reason, a detailed study on the bond performance of GFRP bars was carried out as a necessary step prior to their integration into the masonry system.

The aim of this chapter is to evaluate the effect of surface treatments on the tensile and bond strength of GFRP bars, comparing two types: an industrially produced sand-coated bar and a prototype with a grooved surface. This investigation ensures that the selected reinforcement elements are not only materially compatible but also structurally reliable within the composite masonry framework developed in this research.

6.1 Composite Materials and Glass Fibre Reinforced Polymers (GFRP)

Steel rebars represent a conventional choice for reinforced concrete structures. Compared with Glass Fiber Reinforced Polymers (GFRP) elements, they come with a higher weight and a strong environmental impact. Despite that, steel rebars are usually adopted in construction field because of the high structural performances offered by the material.

In the context of earthen construction, specifically earthen masonry system, where the mechanical properties of the earthen materials are significantly lower than concrete elements, the reduced stiffness and tensile strength of the GFRP elements do not

represent a limiting factor for the structure. Instead, they represent a mechanically suitable alternative without overdesigning the system.

6.1.1 Composite materials

A composite material is defined as a material composed of two or more different material, combined together to produce a new material (composite) with enhanced characteristic [208].

In material science “composite” is a material consistent of mainly two material phases (see Figure 6.1): a matrix, which has the function of containing the other elements, and the reinforcement, the material which provides for strength, stiffness and load-bearing capacity to the composite [209], [210], [211], [212].

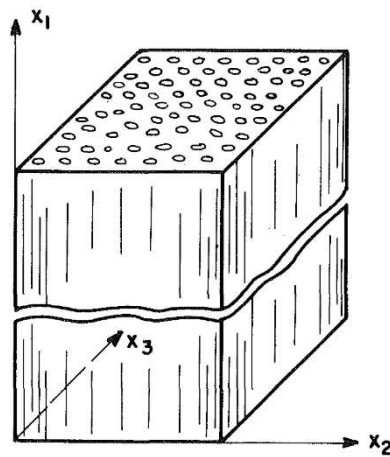


Figure 6.1: Unidirectional fibre composite [210].

Based on type, geometry, and orientation of the reinforcement (or reinforcing phase) composite material can be classified into three categories [213]:

- Particulate composite: consist of a matrix with dispersed particles that enhance mechanical properties.
 - Large particle, the reinforcing particles are relatively large, have the function of restricting the deformation (e.g., cement with aggregate).
 - Dispersion strengthened, the particle size is smaller, particles are uniformly distributed allowing for increased strength of the composite without compromising the ductility.
- Fibre composite: the reinforcement phase is composed of filamentous elements – fibre- which provide increased strength and stiffness.
 - Continuous, unbroken fibres are used through the material, offering high strength along the fibre direction, or orientation.

- Discontinuous, smaller/short fibres are used for the reinforcement phase -sparse fibres.
 - Aligned, fibres are oriented in one direction only, leading to orthotropic composite material properties (as for the continuous case).
 - Random, the fibres are randomly distributed in order to get a more homogenous mechanical response of the material - isotropic material.
- Structural Composites: engineered material for optimised strength-to-weight ratio:
 - Laminates, consist of multiple layers of fibre reinforced sheets, bonded together, with alternating fibre orientation in order to get a more homogenous response.
 - Sandwich panels, consist in coupling two mechanically stronger external sheets (skins) with a lightweight core element.

Fibre reinforced polymers (FRP) stand out in structural application as high-performance materials. They usually consist of a polymer matrix (e.g., epoxy, polyester, vinyl ester) reinforced with fibres (e.g., glass, carbon, aramid). The matrix acts as a binder, transferring the load to the resisting part, the fibres while protecting them from external degradation [214]. The mechanical properties and behaviour of FRP elements are determined by multiple factors, as fibre orientations, matrix properties, and fibre-matrix interaction. Hence, despite good structural performance exhibited, the orthotropic nature of the materials and different failure mechanism require for the FRP element to be carefully designed [210], [215], [216]. The properties of different fibre are to be considered while adopting a FRP element [217]:

- Glass Fibre Reinforced Polymer (GFRP) offers good strength and cost-effectiveness.
- Carbon Fibre Reinforced Polymer (CFRP) exhibits high stiffness, low weight with a higher cost.
- Aramid Fiber Reinforced Polymer (AFRP) offers good impact resilience.

Failure in FRPs can occur due to fibre breakage, matrix cracking, fibre-matrix debonding, delamination [218]. Because of the different nature of failure mechanisms in FRP it is crucial to understand it while designing a structure.

6.1.2 Glass Fibre Reinforced Polymers (GFRP) rebar

Glass Fibre Reinforced Polymer (GFRP) is a composite material made by combining glass fibres with a resin matrix, typically polymer based. The glass fibres provide the material with high strength and stiffness, while the resin matrix binds the fibres together and protects them from environmental factors [219], [220]

Composition of GFRP:

GFRP is a composite material made of:

- **Glass Fibres:** These are the primary load-carrying components of GFRP. They are made from finely drawn glass filaments that provide excellent tensile strength and durability. Glass fibres are non-corrosive, making GFRP a suitable alternative to steel reinforcement in concrete, especially in environments prone to corrosion [214], [221], [222].
- **Resin Matrix:** The resin, often epoxy, vinyl-ester, or polyester, surrounds and holds the glass fibres in place. The matrix transfers the load between the fibres and provides the GFRP with its shape, flexibility, and protection against external environmental stresses such as moisture, chemicals, and temperature variations [223].

General Properties of GFRP:

- **High Strength-to-Weight Ratio:** GFRP is much lighter than steel, yet it maintains high tensile strength and stiffness, making it suitable for a wide range of structural applications [224].
- **Corrosion Resistance:** Unlike steel, GFRP does not rust or corrode, making it ideal for use in aggressive environments such as marine or chemical settings [222], [225].

GFRP for Reinforcement:

GFRP bars are used as an alternative to traditional steel rebar in reinforced concrete. They offer advantages such as corrosion resistance and reduced weight [219], [220]. GFRP bars are commonly found in environments where corrosion resistance is critical, such as in marine structures, bridges, and other infrastructure exposed to aggressive conditions. They are also used for retrofitting technologies to enhance structural durability [224].

Glass Fiber Reinforced Polymer (GFRP) can be used to produce bar elements for structural reinforcement, as an alternative to steel rebars, offering a corrosion-resistant and lightweight alternative to traditional steel reinforcement.

When GFRP bars are used as reinforcement for concrete (rebar), it is important to understand the material behaviour. The mechanical properties of FRP are strongly influenced by the nature of the material. As a composite, GFRP elements exhibit pronounced orthotropic behaviour, with the principal axis aligned with the direction of the fibres, in contrast to steel, which is isotropic. Another important distinction is that the properties of FRP bars can vary depending on their composition, particularly the volume percentage of resin and fibres used, and fibres orientation.

Clarke, in [226], summarised some of the mechanical properties of the GFRP bars as follow:

- Tensile Strength:
 - GFRP rebars reach their ultimate tensile strength without yielding, higher than steel yield strength (see Figure 6.1). However, their tensile strength varies with bar diameter, as fibres at the centre of the bar are subjected to less stress than those near the surface, reducing strength and efficiency in larger diameter bars.

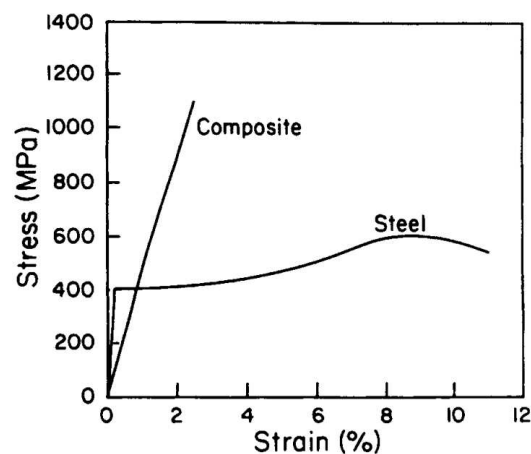


Figure 6.1: Stress-strain relationship for steel and GFRP rebar [226].

- Tensile Modulus of Elasticity:
 - The modulus of elasticity of GFRP rebars is approximately 25% that of steel. This lower value results in a reduced moment of inertia for flexural members, and deflection may control the design of flexural elements unless the rebars are prestressed.

- Compressive Strength:
 - GFRP rebars are weaker in compression than in tension. The compressive strength varies depending on whether the rebar is smooth or ribbed, and higher tensile strength correlates with higher compressive strength.
- Compressive Modulus of Elasticity:
 - The compressive modulus is lower than the tensile modulus, varying with factors such as rebar size, type, and quality control.
- Shear Strength:
 - The shear strength of GFRP is relatively low. This can be mitigated by orienting the rebars to resist loads through axial tension rather than shear.
- Factors Affecting Mechanical Properties:
 - Creep: Fibers like glass and graphite resist creep well, but resin resistance is limited. The orientation and volume of fibres influence creep performance. GFRP bars experience minimal strain under creep conditions, but prolonged loading can lead to creep rupture if sustained stress exceeds 60% of the short-term strength.
 - Fatigue: GFRP composites show good fatigue resistance, particularly with high-performance fibres. However, their fatigue strength is lower than steel.
 - Moisture: Water absorption can degrade strength and stiffness. Care must be taken to assess properties under the specific environmental conditions where GFRP is used.
 - Fire Resistance: GFRP maintains strength at elevated temperatures but experiences a significant drop at higher temperatures.
 - Corrosion Resistance: GFRP has a significant advantage in corrosion resistance compared to steel. However, it can degrade in alkaline environments, such as fresh concrete.

The main limitation of GFRP in cementitious matrices is the poor adhesion between the GFRP bar surface and concrete, which can impact the overall bond strength and structural performance.

In order to overcome the bond issue during the 1990s the issue of GFRP bonding with concrete was analysed. The first studies of the bond between FRP and cementitious were conducted by Larralde in [227]. It underlined that the adherence of the FRP bar was mainly relaying on friction and adhesion, due to the smooth surface, so mechanical anchorage systems were proposed to enhance the bond strength (see Figure 6.3).

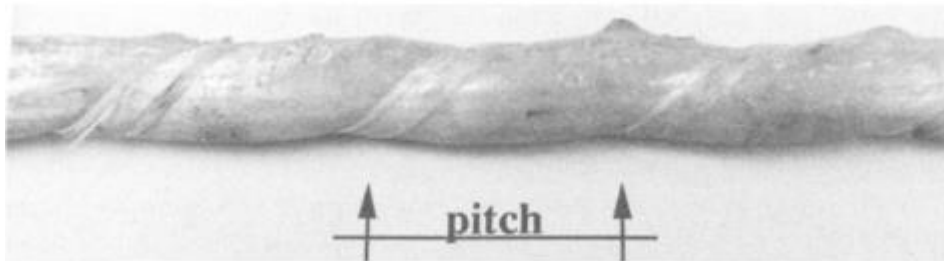


Figure 6.3: FRP Rebar with spiral indentations for enhanced bond strength [227].

In the same years, Malvar investigated the tensile strength and bond properties of GFRP rebar, providing first data on how they interact with cementitious matrix [228]. It highlighted that small surface indentations on the bar were sufficient to create bond strength between GFRP bar and cement, pointing out that some surface treatments could potentially weaken the material. Moreover, radial pressure was identified as a key factor to enhance the bond.

The rebar diameter was identified as another factor that significantly influence the bond strength of GFRP bar in cementitious matrix by Benmokrane in [229]. It also identified non-linearity in the bond stress along the embedded length of the reinforcing bar.

Tastani study [230] confirms the bar diameter being a key factor that influence the bond-strength between GFRP bar and concrete, larger diameter exhibited lower bond strength. Moreover, similar bond strength was found for bar with similar diameter but different surface treatment, indicating bar diameter as more relevant factor.

Studies of Ashour [231] investigated the use of GFRP in concrete beams, showing how surface treatment (to increase roughness) and bar diameter are crucial for assuring a good bond strength. It also highlighted that GFRP-concrete bond failure often occurs in shear region, with diagonal cracks propagating at the level of GFRP reinforcement. Failure is also reached in case of over-reinforcement, due to the poor bond and therefore inability of the concrete to hold the rebars in place.

The studies of Correia, in [232], highlighted the necessity of good shear connection system in concrete-GFRP structure, to assure composite action of the system by reducing slippage. A connection system was proposed both with epoxy adhesives and steel bolt.

Despite the epoxy adhesives experienced high initial stiffness the bolted configuration resulted more reliable.

Yan review on bond strength of GFRP and concrete [233] confirmed that the key factors influencing the bond not only depending on the rebar (e.g., diameter, embedded length, surface) but also on the properties of the concrete used. It is shown that concrete compressive strength has linear relationship with the bond strength, and an increased concrete cover improves the confinement around the bar, therefore reducing splitting failure.

Lee further investigate how the bond strength is affected by the bar diameter in high strength concrete [234]. It showed how, as the diameter increase the bond of GFRP bar decrease. The strength of the concrete improved the bond strength of the rebars, but it did not affect the trend. In contrast, the steel bar which showed decreasing of bond strength up to 19 mm diameter, reaching a constant bond strength level.

A comparison of different surface treatment was carried out by Abedini in [235]. It showed how sand-coated bars offered higher bond strength compared to the ribbed and indented bars typology.

6.2 Surface Treatment for GFRP Bars

The bond strength of GFRP bars in concrete is influenced by several factors, including the bar diameter, composite composition, surface treatment, mechanical anchorage systems, as well as the concrete compressive strength and concrete cover. In this study, the construction system investigated presented specific mechanical and physical properties, and no variations in concrete type, or confinement, were performed (see Figure 6.4). Only variations in bar-related factors were considered.

The steel rebar used in the configuration had a diameter of 16 mm, while the GFRP bars, industrially produced by FibreNet, maintained a fixed composition with no variation in resin-to-fibre ratio or resin type. The bar diameter was reduced to 12 mm for two main reasons: to improve bond strength, and because the mechanical properties required for reinforcing the generally weak masonry were not excessively high, as larger bars would not provide additional benefits. The final factor considered was surface treatment. The bars industrially produced had either a smooth surface or a sand-coated surface.

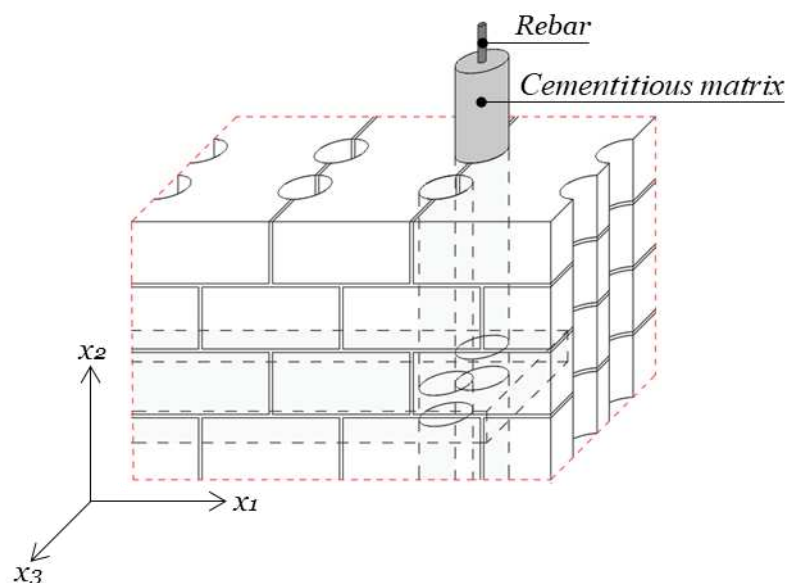


Figure 6.4: Vertical reinforcement configuration.

The study of Lee et al. [236] investigated how different surface treatment influence the bond strength of GFRP in cementitious matrix, pointing out that concrete strength has a minor effect on GFRP bars bond strength, compared to steel rebar. Helical wrapping and sand coating technique were compared (see Figure 6.5). Helically wrapped bar showed a more gradual reduction in bond stress, while sand coated type experienced a sharp drop after reaching maximum bond strength. Delamination failure mode was observed between the outer, resin-rich layer of the bar and the core. A liner proportion between

concrete strength and delaminated area was observed, affecting the interfacial bond strength.

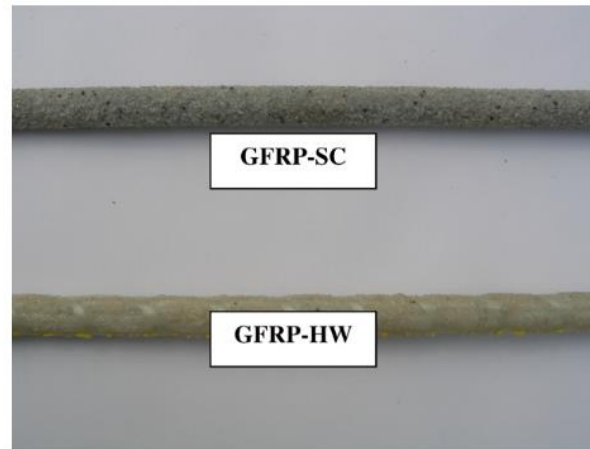


Figure 6.5: GFRP rebar with sand coated surface and helical wrapped surface [236].

Research on the bond strength of sand coated GFRP bar embedded in concrete was carried out by Robert in [237]. It revealed that bond strength remains relatively stable over time, with modest reduction when prolongedly exposed to high temperatures and moisture. Failure modes were examined in pull-out test. Slippage at the free end was observed with failure occurred at the sand coating interface rather than on the bar or cement matrix. Interface failure was registered between bar and sand coating, with the sand coated layer peeling off, limiting the bond effectiveness. No degradation of the fibre component and bar polymer matrix were observed, indicating critical role of the surface treatment.

A particular focus on the effect of surface treatment (sand coating) on strength bond was made by Arias in [238]. Bar diameter, surface treatment, and concrete strength were confirmed to be key factor influencing the bond performance. The study demonstrates that the use of a coarse sand for coated bar offered increased friction and interlocking forces, compared to finer sand. Moreover, it underscores the importance of chemical adhesion in bond strength.

This research focuses on evaluating the bond strength between industrially produced sand-coated GFRP bars and concrete, while also acknowledging that alternative technologies, such as filament winding, can be used to create ribbed systems on GFRP bars to enhance bond strength through geometry rather than surface roughness, potential downsides, such as failure at the rib/bar interface, may be observed.

The bond strength enhancement via ribs in GFRP bars was investigated by Hao in [239]. Rib geometry, and rib spacing and geometry were identified as key factors, influencing

the bond (see Figure 6.6). Optimal rib height was determined at 6% of the bar diameter and ideal spacing was set equal to the bar diameter. Other surface treatments were also considered, as sand coating and fibre strand wrapping. Study showed that sand-coated GFRP bars experienced better performance, especially of its property to reduce slippage. On the other hand, the rib system showed increased mechanical bond, but shear failure occurred with deformation and delamination of the ribs. A general better performance was registered when textured surface was used for ribbed bars.

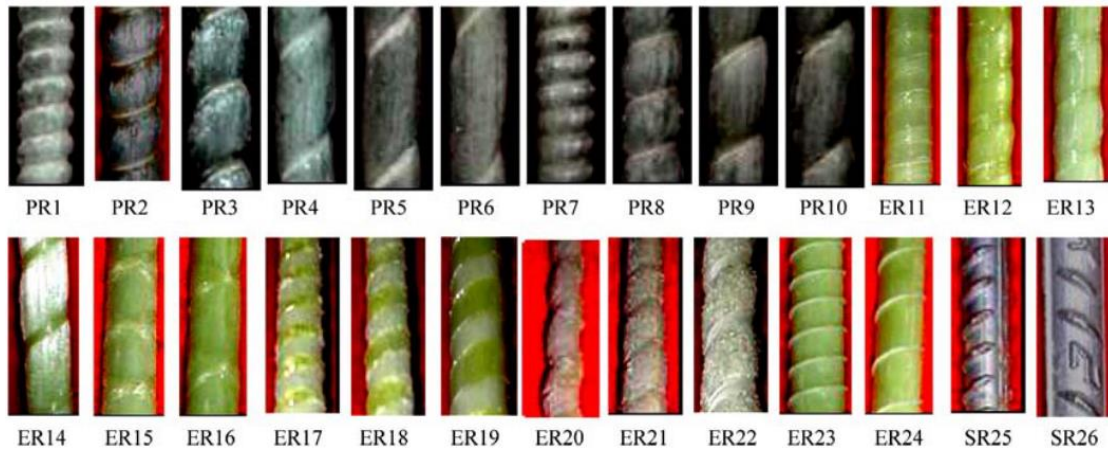


Figure 6.6: GFRP bar surface configuration from [239].

Issue of ribs detachment in GFRP bar was examined also in Ashrafi [240] (see Figure 6.7). As for steel rebar, ribbing system relies on mechanical interlocking between rib and concrete. However, GFRP bars generally showed a lower bond strength. Failure modes are observed to be conditioned by the shear strength of the surrounding concrete, highlighting that as the concrete strength increase, the ribbed surface plays a more significant role. No concrete deformation or crushing was registered, while ribs detachment was observed to be the dominant failure typology. In addition, larger ribs showed to reduce slippage.



Figure 6.7: Ribs detachment, failure at ribs/GFRP interface from [240].

A comparison between helical wrapping, sand coating and ribbed surface was performed by Gao in [241] to understand mechanical interlock of GFRP bars. Sand coating method demonstrated higher bond than helical wrapping, and ribbed bar exhibited higher bond strength due to the high interlocking effect of the ribs. Failure mode showed shearing off of the ribs (rib-bar interface failure) and concrete crushing. Higher confinement from stirrups plays a crucial role to minimize rib-bar interface failure.



Figure 6.8: Cross-section of specimen showing failure of the coated surface on GFRP bars [242].

A study of sand coated and ribbed surface was done by Devaraj in [242], to investigate the bond behaviour. In this study, ribbed GFRP bar provided an enhancement mechanical interlocking of only 3% compared to sand coated bar. However, while the sand coated bar failed due to slippage (see Figure 6.8), in the ribbed system concrete splitting was observed.

The study of You analysed the bond strength in textured surface ribbed GFRP bars, which it showed to be higher compared to smooth surfaces, especially improving slippage [243]. Investigations on the GFRP composition are also carried out. The type of resins used resulted to be not affecting bar properties or bond, while the fillers used proved to improve the tensile properties of the GFRP bar and the interfacial bond matrix-fibres. Moreover, kaolin and alumina filler alter the smoothness of the surface, creating a rougher surface with an overall improvement of the bond strength of the bar.

The composition of the GFRP bar was also studied by Moon in [244], by adding milled glass fibres to the epoxy resin, registering that significant influences on bond strength

are linked not only on the bar geometry (rib system) but also its composition. The use of milled glass fibre content in the resin surface layer (up to 50% of fibres) enhanced the bond strength, making it comparable to the steel rebars bond. However, long-term durability is affected due the milled glass fibre exposition to alkaline environment (cement).

The mechanical interlocking of GFRP bar/cement was further studied by Shakiba in [245] which proposed an alternative approach to other geometric surface modification (e.g. ribs). The study demonstrated how anchoring systems in GFRP sand coated bar can significantly improve the bond strength. The study also showed that the anchor shape, specifically designs with larger diameters at the end, allowed for better mechanical interlocking and friction, thus improving the overall bond performance (see Figure 6.9). The anchor heads contributed to reducing slippage and improving load transfer from the GFRP bar to the concrete, particularly in high-strength concrete, by increasing the friction and bearing capacity at the interface.



Figure 6.9: Anchoring system for GFRP rebars in [245].

Since the variation of the system proposed in this research uses vertical concrete elements reinforced with GFRP rebars, bond strength was analysed to assess feasibility.

Despite the potential for failure at the rib/bar interface with filament winding, this method has proven to increase bond strength. In this research, a prototype bar inspired by filament winding technology is studied, where instead of applying traditional ribs prone to detachment, indentations are used to create a rib-like surface that enhances the geometrical interlocking of the GFRP bar, improving bond strength. This approach was manually implemented in the lab by creating indentations on specimens after their production (see Figure 6.5), without requiring specialized machinery.

6.3 Experimental Investigation of GFRP Bar Strength Bond in Cementitious Matrix

In the following section an experimental campaign is presented, to investigate the bond strength of GFRP bar in cementitious matrix.

In the experimental phase, the focus was on evaluating the performance of GFRP bars as a substitute for the traditional steel reinforcement used in vertical in the concrete-frame reinforced CEB masonry configuration. In the previous system, reinforcement is achieved with a single steel rebar embedded in a low-strength concrete matrix that comprises only sand as an aggregate (see Figure 6.10a). As highlighted in the state-of-the-art review, the bond between reinforcement and cementitious matrix is often suboptimal; therefore, the investigation aimed on modifying the bar characteristics rather than altering the concrete mix. Such an approach allowed to isolate the effects of the bar on the overall bond performance without deviating significantly from the current construction practice.

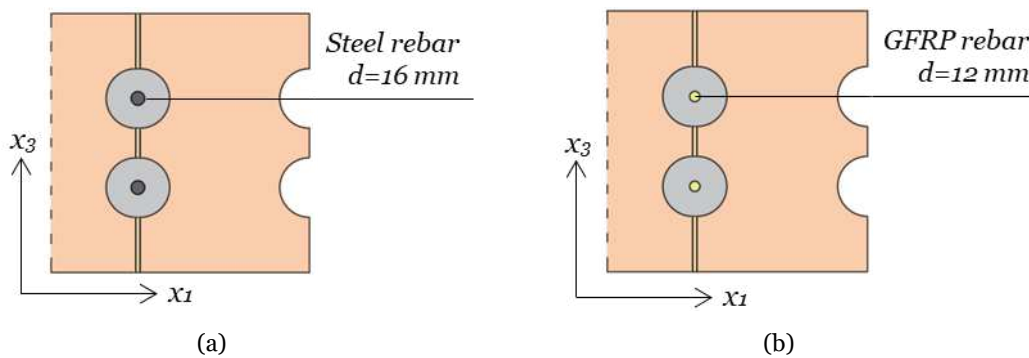


Figure 6.10: Vertical reinforcement elements, (a) steel rebar, (b) GFRP rebar.

Accordingly, the GFRP bar selected for this study was designed with a reduced diameter of 12 mm to enhance bond strength, considering that GFRP rebar diameter and bond strength are inversely proportional, and that the mechanical requirements for reinforcing weak masonry elements do not strictly necessitate larger bars. Two types of surface treatments were examined: one utilized an industrially produced sand-coated finish, and the other involved a laboratory-fabricated prototype featuring grooves (realised at Laboratorio di Scienza delle Costruzioni LabSCo of IUAV University of Venice) intended to improve the mechanical interlock with the surrounding cement matrix. For reference, smooth-finished rebars were also studied (see Figure 6.11). This experimental setup provides a focused assessment of how bar-related factors influence the bond behaviour in earthen masonry reinforced with low-strength concrete

Three surface typologies of rebar were then investigated:

- Type-1, smooth surface.
- Type-2, sand coated.
- Type-3, grooved.

While type-1 and type-2 were industrially produced, type-3 was a prototype developed for research purposes.

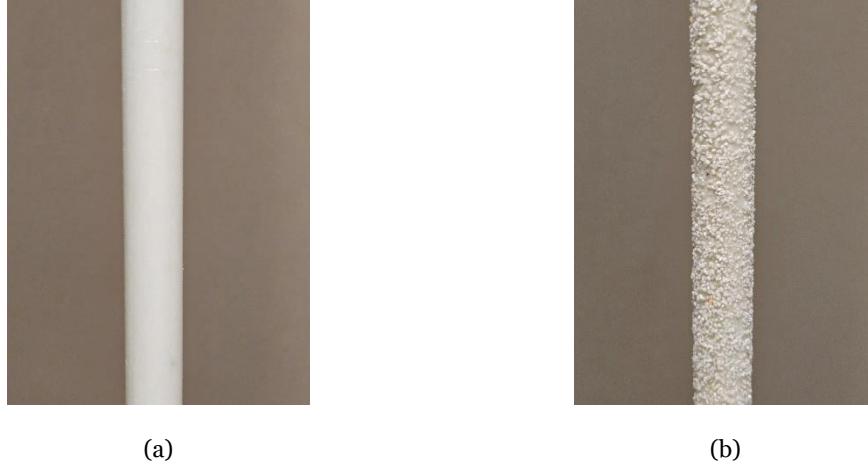


Figure 6.11: GFRP bar surface treatment (a) smooth surface, Type-1, (b) sand coated, Type-2.

In this study, the ribs are grooved, reducing the diameter of the bar. The grooves create a notch-like element that surrounds the bar. The ribbing has the following specifications: spacing is equal to the bar diameter, groove height is 1.5 mm, and groove depth is 1 mm (see Figure 6.12). As a result, the effective diameter of the bar is reduced by 2 mm.

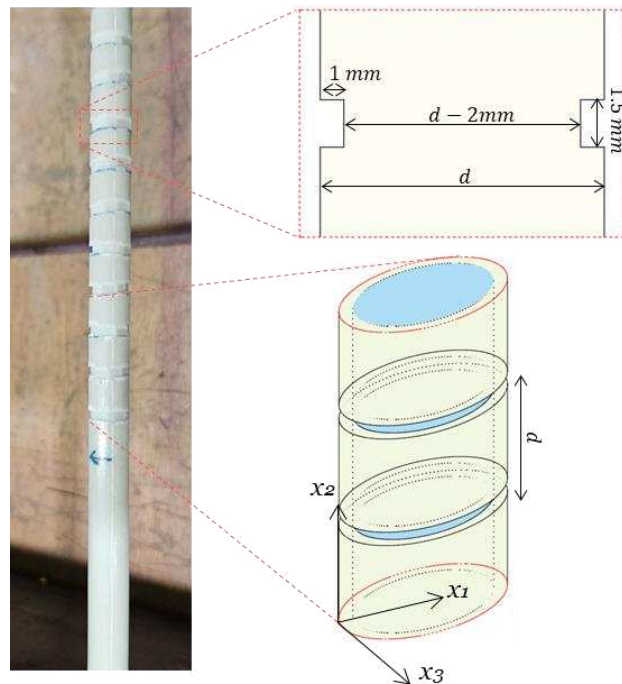


Figure 6.12: Type-3 GFRP bar configuration.

6.3.1 Tensile test

Tensile tests were conducted to evaluate the load-bearing capacity of GFRP bars and assess the impact of grooving on their mechanical resistance.

Three types of GFRP bar specimens were tested: (i) a 10-mm diameter bar with a smooth surface ($S10 - n$); (ii) a 10-mm diameter grooved bar ($G10 - n$), tested for mechanical performance comparison; and (iii) a 12-mm diameter grooved bar ($G12 - n$), selected for pull-out testing and considered as a potential alternative to steel reinforcement in the construction system. The tensile test setup (see Figure 6.13) followed the guidelines of CNR-DT 203:2006 on FRP reinforcement for concrete rebar [246].

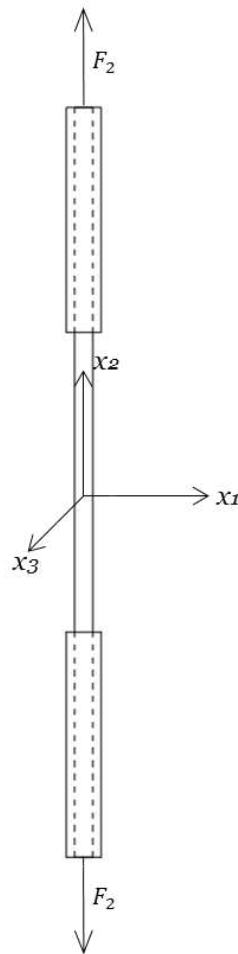


Figure 6.13: GFRP bar tensile test set up.

The G10 specimens exhibited an approximately 37% reduction in tensile strength compared to the S10 series (reference), while G12 and S10 showed comparable results (see Table 6.1). This trend indicates that the effective resisting section of grooved GFRP bars is limited to the area within the grooves, where fibres remain continuous and uncut. Consequently, the outer cylindrical ring section does not contribute to the bar

mechanical strength. This behaviour is attributed to GFRP composite nature, where glass fibres bear the load while the resin matrix primarily binds them together.

<i>Specimen</i>	<i>Tensile strength</i> [N]	<i>Mean tensile strength</i> [N]
G10-1	40637	39054
G10-2	38325	
G10-3	38754	
G10-4	38501	
G12-1	61563	61915
G12-2	63149	
G12-3	61035	
S10-1	63203	62133
S10-2	61830	
S10-3	61364	

Table 6.1: Tensile strength of GFRP bars.

The failure mode differs between smooth and grooved GFRP bars. The smooth bar (see Figure 6.14a) exhibits an explosive failure, with fibres abruptly rupturing and fraying outward, indicating a sudden loss of tensile capacity. In contrast, the grooved bar (see Figure 6.14b) fails in a more contained and progressive manner, with cracks initiating at the groove notches and propagating along the bar in a controlled way. The outer ring-like section formed by the grooving process appears to influence the failure mechanism, preventing sudden fibre rupture and redistributing stress more gradually.



(a)



(b)

Figure 6.14: Failure mode of (a) Type-1 and (b) Type-3 rebar subjected to tensile load.

6.3.2 Direct pull-out test

To evaluate the bond strength of Glass Fiber Reinforced Polymer (GFRP) bars in a cementitious matrix, a direct pull-out test was conducted using a centric bar placement configuration. The tested GFRP bars had a diameter of $d = 12 \text{ mm}$ and a total length of 500 mm . The cementitious matrix used in the specimens was the same cement employed for the masonry reinforcement frame. Three types of GFRP bars were investigated: smooth surface ($S12 - n$), sand-coated surface ($C12 - n$) (industrially produced by FibreNet company), and grooved surface ($G12 - n$), as described in Section 6.3 (see Figure 6.11). The test setup was defined following the methodologies outlined in [237], [238], [239], [240], [241], [242], [245], [247], [248], [249].

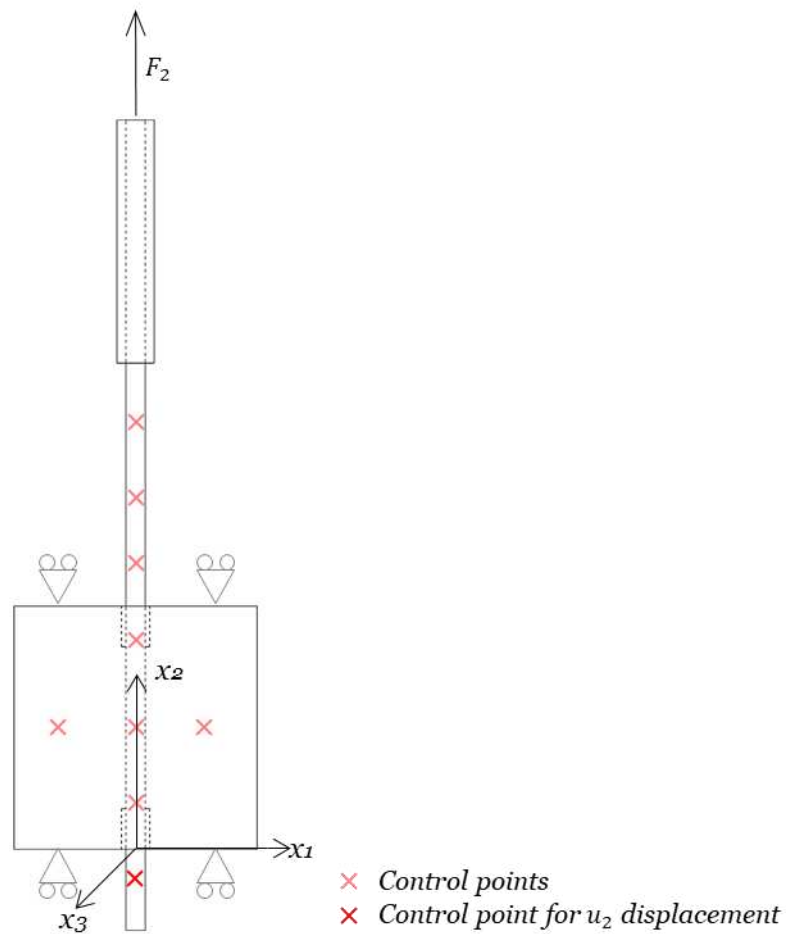


Figure 6.15: GFRP rebar pull-out test setup with indication of control point for extraction of displacement data with DIC.

The specimens consisted of a 150 mm concrete cube with a single GFRP bar embedded vertically along its central axis. To minimize stress concentration along the perimeter of the concrete cube, bonding between the bar and the concrete was prevented at both

ends using plastic insulation. The embedded length of the bar was 100 mm, while the unloaded portion extended 50 mm beyond the specimen (see Figure 6.15).

A loading frame (see Figure 6.16) was mounted within the testing machine to effectively transfer the reaction forces from the specimen to the loading system. The test was performed under displacement control, with a velocity rate of 0.008 mm/s.

To monitor the displacement of the GFRP bar (slippage), Digital Image Correlation (DIC) was employed (see Chapter 3). The unloaded portion of the bar was monitored to assess its displacement, with images captured at a time interval of 1 second throughout the test.

Bond stress was evaluated as $\tau = F_2 / \pi dl$, where F_2 is the tensile strength [N], and d and l respectively the effective diameter [mm] and embedded length [mm].

The pull-out test results highlight the influence of surface treatment on the bond strength of GFRP bars in concrete (see Table 6.2). The smooth bars ($S12 - n$) exhibited the lowest bond stress values $\langle \tau^{S12} \rangle = 2.1 \text{ MPa}$, indicating limited bond capacity. The grooved bars ($G12 - n$) showed improved bond performance, with bond stress values between $\langle \tau^{G12} \rangle = 3.3 \text{ MPa}$, ascribable to enhanced mechanical interlocking. The highest bond strength was recorded for sand-coated bars ($C12 - n$), with values between $\langle \tau^{C12} \rangle = 7.9 \text{ MPa}$, confirming the effectiveness of surface roughness in improving adhesion. The results suggest that surface treatment significantly enhances bond performance, with sand coating providing the most effective improvement. These findings align with existing literature and reinforce the importance of surface modifications in optimizing GFRP-concrete bonding.



Figure 6.16: Pull-out test frame and camera setup.

Grooved bar showed failure at the concrete indentations. During the test, incomplete adhesion between the concrete and the grooves of the GFRP bar was observed. This issue may be attributed to the fluidity of the concrete mix, as insufficient workability could prevent proper penetration into the grooves, reducing mechanical interlock. However, the grooves still contribute to the bond strength by enhancing shear resistance within the concrete matrix. The observed shear failure of the concrete suggests that bond strength was governed primarily by the concrete and shear resistance, rather than by adhesion at the interface (see Figure 6.17a). This failure mechanism is consistent with the expected behaviour of grooved bars, where mechanical interlocking dominates over chemical bonding.

The sand-coated bars exhibited the highest bond strength, with the failure mode varied along the embedded length. In the region near the free end, failure occurred at the coating/bar interface, indicating delamination or insufficient adhesion between the coating and the GFRP core [242]. Conversely, near the loaded end, failure was observed at the coating/concrete interface, leading to slippage of the bar within the concrete matrix. This suggests that the coating-concrete bond strength is critical in determining overall performance. The presence of two distinct failure zones implies that stress distribution along the embedded length is non-uniform, with higher stresses concentrated near the loaded end.

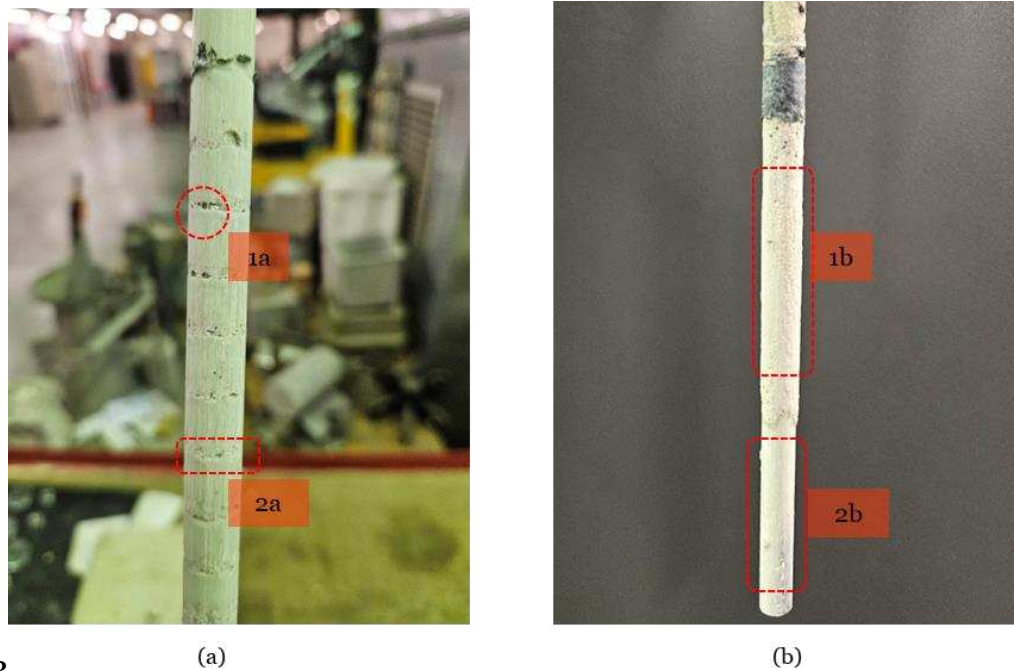
<i>Specimen</i>	<i>Bond stress</i> [MPa]	<i>Bond stress</i> [MPa]
S12-1	1.91	
S12-2	1.88	2.1
S12-3	2.54	
G12-1	3.29	
G12-2	3.19	3.27
G12-3	3.32	
C12-1	7.85	
C12-2	8.5	7.91
C12-3	7.36	

Table 6.2: Bond strength of GFRP bars.

The bond stress vs. free-end slip curves for GFRP bars embedded in a cementitious matrix highlight significant differences in bond performance based on surface treatment (see Figure 6.18). The three categories analysed - smooth bars (s12 series), grooved bars

(G12 series), and sand-coated bars (C12 series) - exhibit distinct mechanical responses under pull-out loading conditions.

The untreated bars were tested as reference. They displayed the lowest bond stress values as expected. The curves show a gradual increase in bond stress, reaching a plateau at approximately 1.5 to 2.5 MPa, followed by stable slip behavior without a sharp drop. This behaviour suggests that bond strength is primarily dependent on chemical adhesion and friction, leading to progressive slip rather than sudden failure. The relatively low slip values at failure confirm a weak interfacial bond, which is expected for smooth bars due to the absence of mechanical anchorage and purely relying on chemical adhesion.



B

Figure 6.17: (a) Grooved bar failure mechanism showing (1a) poor adhesion of concrete within the grooves and (2a) shear failure of the concrete, and (b) sand coated bar failure mechanism showing (1b) failure at coating/concrete interface and (2b) failure at coating/bar interface.

The grooved bars exhibit moderate bond stress values, with peaks around 3.0 to 3.5 MPa, significantly higher than the smooth bars. The stress-slip response shows a more pronounced peak, indicating a stronger mechanical interlock before bond degradation occurs. After reaching peak bond stress, the curves gradually decline, suggesting progressive failure due to shear stress concentration in the concrete grooves rather than an abrupt loss of adhesion. Compared to smooth bars, the higher bond strength and greater slip capacity confirm the effectiveness of surface grooves in enhancing the mechanical bond.

The sand-coated bars exhibit the highest bond stress values, reaching peaks between 7.3 MPa and 8.5 MPa, which is more than double that of grooved bars. The stress-slip curves show steep initial increases, suggesting strong adhesion between the coating and the cementitious matrix, followed by sudden drops after peak stress. This failure mode suggests coating/concrete interface failure, as previously observed. The post-peak behaviour demonstrates a rapid bond degradation, likely due to the delamination of the coating from the bar surface, leading to slip once the peak stress is exceeded.

The analysis confirms that surface treatment plays a critical role in optimizing bond behaviour and should be carefully considered when designing GFRP-reinforced concrete structures.

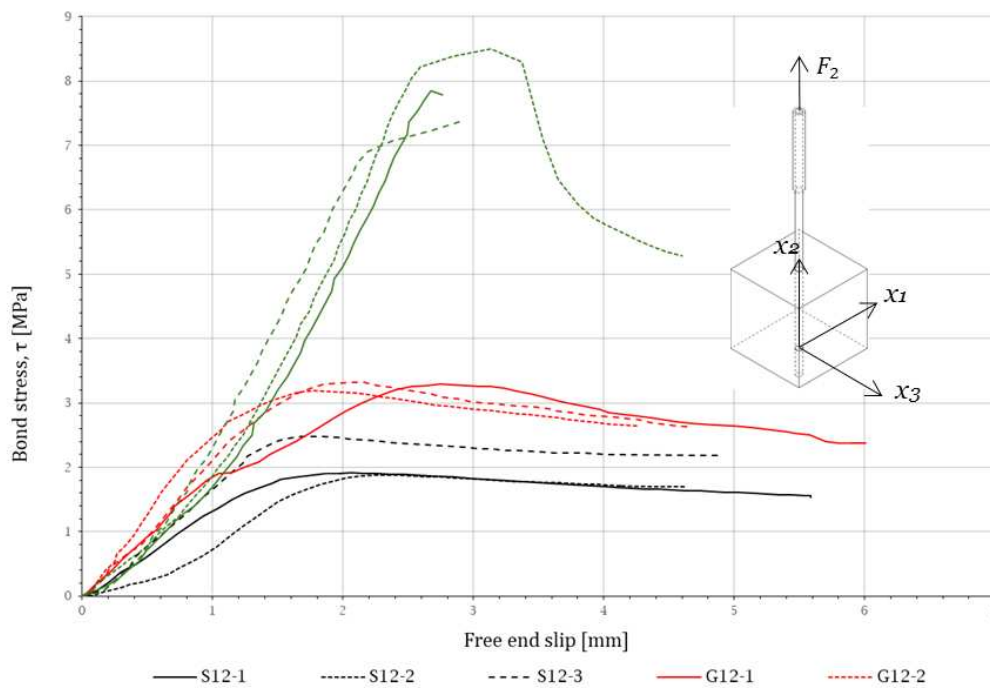


Figure 6.18: Bond stress- free end slip curves for direct pull-out test of GFRP bar in cementitious matrix.

The grooved GFRP system demonstrated a significant improvement in bond strength compared to the smooth bars, primarily due to the mechanical interlock provided by the grooves. However, the failure mechanism observed - shear failure of small concrete elements within the grooves - suggests that an optimal balance between groove geometry and concrete indentation should be further investigated.

A key aspect to consider is the proportionality between the concrete indentation and the cylindrical ring formed by the grooving pattern. If the grooves are too deep or too widely spaced, the system may behave similarly to a ribbed bar, where failure occurs at the GFRP-concrete interface rather than through shear failure in the concrete. This

behaviour is commonly observed in filament winding technology, where failure typically occurs at the interface of rib-like elements rather than within the concrete itself.

To optimize bond performance, an ideal geometric proportion between groove depth, spacing, and indentation size should be sought. If the groove spacing and depth are carefully designed, it may be possible to enhance mechanical interlocking without shifting the failure mode toward premature interface failure at the GFRP ribs. Future research should focus on experimentally determining the optimal groove-to-spacing ratio that maximizes bond strength while preventing premature detachment at the GFRP-concrete interface. Furthermore, a combination of grooving and sand coating should be investigated as a potential approach to further enhance bond performance.

Chapter 7 – Numerical Analysis

This chapter discusses about numerical models and analyses to investigate the mechanical behaviour of cement-stabilised compressed earth blocks (SCEB). In the first section the numerical modelling approach is presented. Given the high number of variables concerning stabilised earth as a construction material, a macro-modelling approach was selected to optimise computational effort. In the first section, multiple Representative Element Volumes (REV) are identified and analysed to describe the micro-scale behaviour of the masonry. Following, a finite element model, based on experimental data, is used to perform a linear analysis of the masonry behaviour under vertical loads in the three different configurations: unreinforced (UM), reinforced with a concrete frame (CRM), and reinforced with fibre reinforced elements and concrete (MRM). This aim to validate the numerical procedure adopted. The second section of the chapter delves into the non-linear behaviour of the masonry. The non-linear characteristics are described through a multilinear constitutive law, based on experimental data, and a calibration of the mortar-brick interfaces to achieve a finite element model capable of describing the behaviour of masonry panels. Once the numerical-experimental correspondence is established, shear simulations of the masonry are conducted to compare the effectiveness of the different masonry configurations.

7.1 Homogenisation Process

A macro-modelling approach was selected for modelling CEB masonry. The homogenization approach in numerical modelling is a method used to derive the effective macroscopic properties of heterogeneous materials by considering their microstructural details, moving from a heterogeneous masonry structure into an equivalent homogeneous orthotropic continuous system. This technique is particularly useful for materials like masonry, which consist of discrete units (e.g., bricks) and joints (e.g., mortar). By doing so, equivalent elastic properties are derived through a numerical simulation of a representative element volume (REV), or elementary cell, of the heterogeneous material investigated.

In the chapter that follows, two Euclidian 3D system are used where x_1 is the horizontal direction, x_2 is the vertical direction and x_3 is the depth at a macro level, whereas y_1 is

the horizontal direction, y_2 is the vertical direction and y_3 is the depth at a micro level, in accordance with the homogenisation procedure (see Figure 7.1).

7.1.1 Definition and identification of REVs

The Representative Element Volume REV is a fundamental concept in the homogenisation approach, serving as a small, statistically representative sample of the heterogeneous material. It needs to describe all the main characteristics at micro level, as geometry and physical and mechanical material properties of the constituent elements (as brick and mortar for masonry). It is the smallest volume over which effective measurement can be performed to describe the macroscopic properties. In order to correctly define a REV, different material phases must be included without alter the geometry. When highlighted the periodicity of the heterogeneous material, the REV can be identified as a pattern. In the masonry case this result in bricks shape, their arrangement, and mortar joints thickness. Proper boundary conditions must be applied during numerical simulation to guarantee accurate mechanical macroscopic response. In the masonry case, periodic boundary conditions can be used to simulate an infinite array of the masonry texture.

Whitin this framework, the main objective is to estimate the effective properties of periodic media, or a continuum, by considering its microstructural features, as well as the geometric and mechanical characteristics of its constituent materials. This results in determining the overall average constitutive properties of the Representative Elementary Volume (REV) based on the properties, arrangement, and interactions of its micro-elements.

Periodicity, in terms of stress and strain-periodic displacements, is extensively discussed in [186], [189], [196], [198]. For a given media, the stress state is considered macroscopically, or globally, homogeneous when all the cells within the medium experience identical loading conditions. This aspect is critical because cells positioned near the boundaries of the media and cells positioned at the centre of the specimen do not undergo the same stress conditions.

To maintain structural integrity and accurate modelling, it is critical that adjacent cells remain in contact even in their deformed. This requirement implies that the stress vector is continuous across the cell boundary, and strain are compatible (meaning no separation or overlapping can occur). Given the nomenclature in [189], [196]

$\sigma \cdot \mathbf{n}$ is anti-periodic on ∂S

u^{per} is antiperiodic on ∂S

where σ is the stress field, $\mathbf{n}$ the normal, u^{per} describes the strain field, and δS define the boundary condition of the cell.

The stress vectors $\sigma \cdot \mathbf{n}$ are opposite on opposite sides of the cell boundary as the stress field is periodic and the external normal opposite, this assure strains on δS to be compatible.

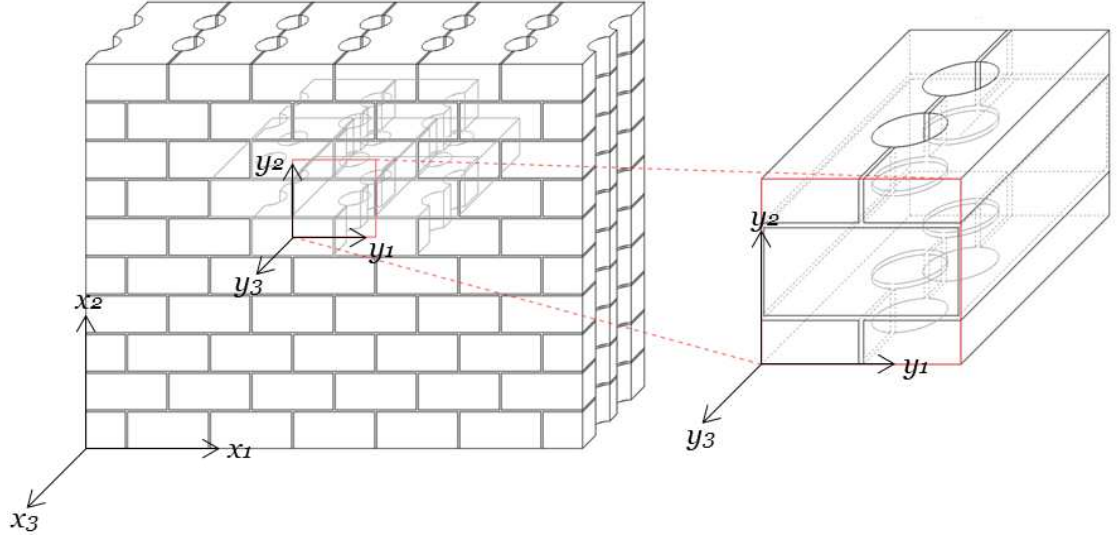


Figure 7.1: Stabilised earth block masonry panel (macro scale) and identification of a Representative Elementary Volume (REV) at the micro scale with highlight of the microstructural geometric features.

When masonry is studied, it is common to use a two-dimensional REV to obtain a homogenised media, this is due to the uniformity of the pattern which can be found in its extension along the third dimension.

Three-dimensional model, and REV, can be found also when the specimen studied undergoes forces which acts within the plane of the wall as well as perpendicular to it. In this case, as seen in Cecchi and Sab [199], a 3D model is developed to better understand the behaviour of the masonry subjected to in-plane forces and out-of-plane forces. Specifically for masonry materials, actions like bending or twisting are complex because of the nature of masonry itself (arrangement of blocks, with possible different directions and mortar joints). In the case of Cecchi and Sab study, the masonry panel considered was periodic in the third dimension, so an asymptotic model was later used to move towards a 2D Love-Kirkoff plate to describe the three-dimensional behaviour of the specimen, in order to reduce the complexity. In this specific case study, the aim was to understand the behaviour of a 3D element subjected to seismic, wind actions and actions due to connections with orthogonal walls, but no physical changes in the third dimension

were registered. The use of a 3D model allowed for the evaluation of the strain perpendicular to the middle plane and bending stiffness.

In the case study here presented, because of the singular shape of the earth-blocks (see chapter 4) the third dimension must be considered during the analysis due to geometric discontinuities, the third dimension cannot be described as a plate with a certain thickness because of its irregularities. The voids present, not only alter the geometry but, as a consequence, the mass distribution and the boundaries conditions. Given the periodicity of the voids a REV is still easily identifiable. Homogenisation process of media with voids inside the matrix can be found in modelling of cellular materials or lattice structures. In the work presented by dos Santos [250] voids are included in the homogenisation process to analyse specific structures or materials. Both voids within the matrix and along the contour of the cell were considered. To account for these voids, especially those at the boundaries of the RVE, certain mathematical adjustments are made to maintain consistency in the model. Specifically, the kinematic quantities—such as displacements—are defined only in the solid parts of the REV. Special boundary conditions and adjustments are also studies (as applied constraints on the movement of the material close to) ensure that the presence of voids, particularly on the edges, is accurately reflected in the behaviour of the material, so that more complex microstructures can be investigated.

Furthermore, when the internal reinforcement frame is present, the REV identified to describe the unit-mortar interactions is no longer effective, due to the variations in geometry and materials entailed (see Figure 7.2). The introduction of different materials within the third dimension (the thickness of the masonry panel), arranged vertically and horizontally with different geometry (reinforcement frame) imposed changes on the geometry of the REV and also on stiffness levels of the portions analysed. Moreover, in order to accommodate the frame different typology of block (Block-3, see Figure 4.1) has to be used along the masonry pattern, leading to significant changes regarding the geometry of the unit element. For these reasons, different REVs are identified and analysed to describe the specific portions of the reinforced masonry panel (unreinforced masonry, vertical reinforcement, horizontal reinforcement, reinforcement node element) so that the overall macroscopic behaviour of the reinforced masonry panels can be described. Hence, different homogenised materials are defined.

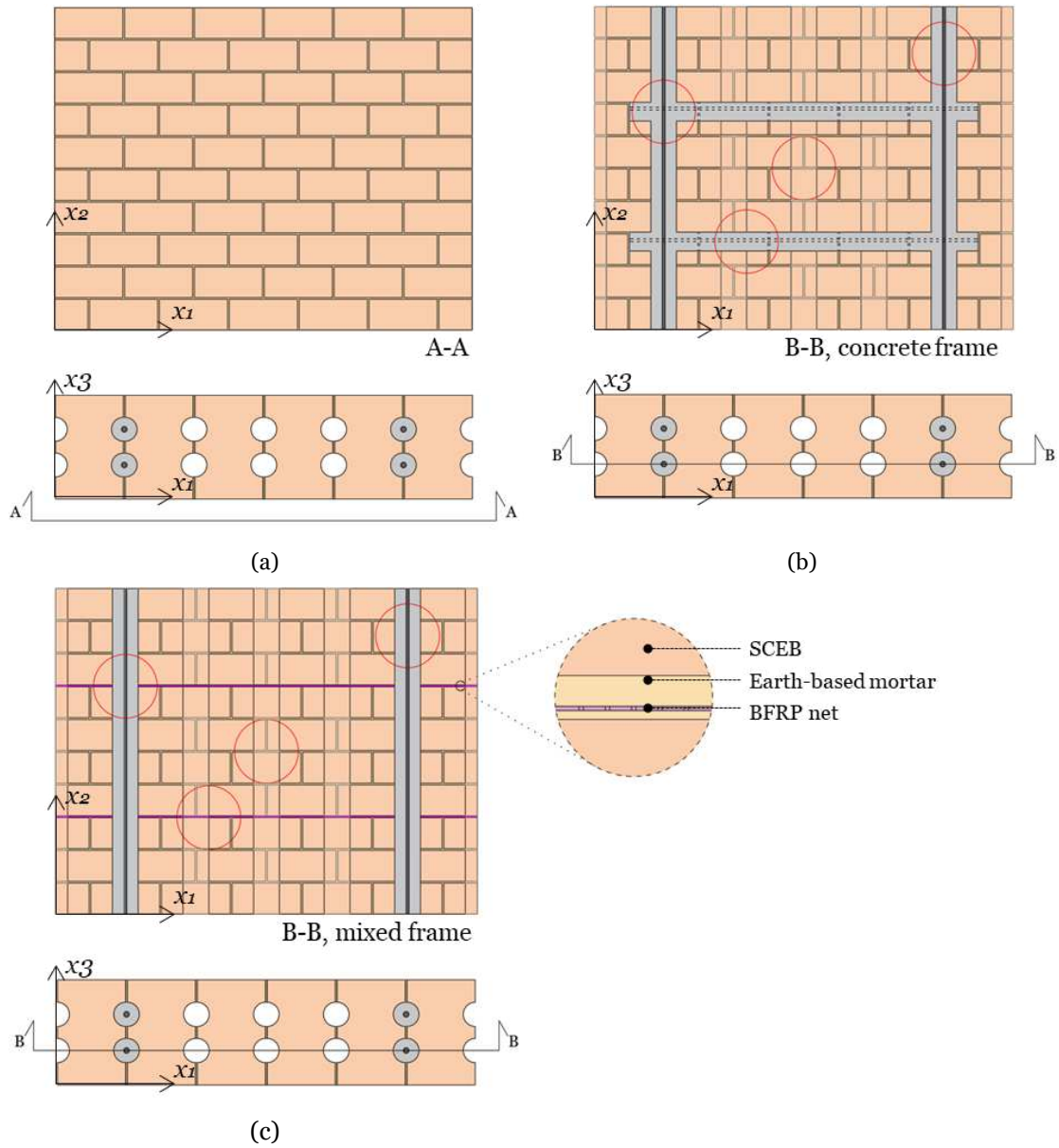


Figure 7.2: Earth masonry panel: reinforced configuration, surface (a) internal section for concrete frame (b) internal section for mixed frame (c).

In the following section, a detailed description of the different Representative Elementary Volumes (REVs) is provided. The term Material- n refers to the new homogenised material resulting from the homogenisation process. This nomenclature emphasizes the material's effective mechanical properties as derived from the REV. On the other hand, the code cell- m is used to describe only the geometric characteristics of the REV, without implying any material characteristic. This distinction is made to clarify that Material- n represents the homogenised outcome in terms of mechanical response, which is linked to the arrangement of the representative volume, while cell- m strictly pertains to the geometric aspects of the REV. This distinction is necessary because, given

the structure analysed, even with the same geometric arrangement of the masonry within the cell, different materials can be assigned to the structure. These different material assignments lead to variations in stiffness, and consequently, result in different homogenised materials, despite geometry remaining unchanged.

REV identification:

- Unreinforced masonry panel:
 - Material-0 / cell-0 (Figure 7.3), the interaction between the earth-based mortar and the cement-stabilized CEB is described. The selected geometry is determined by the head bond masonry pattern. Despite the geometric differences between Block-1 and Block-2, no additional REV's are required to describe the masonry panel in the unreinforced masonry case.

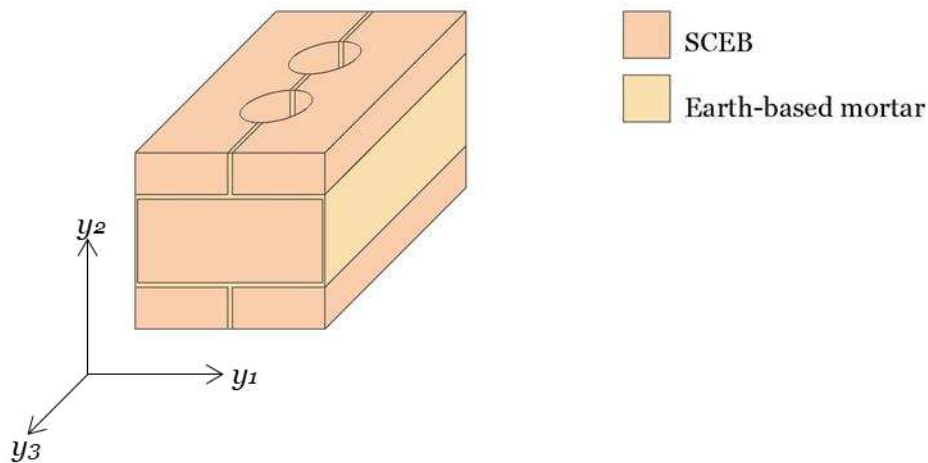


Figure 7.3: Representative Elementary Volume (REV) of the unreinforced masonry.

- Concrete-frame reinforced masonry panel:
 - Material-1 / cell-0 (Figure 7.4), the earth-based mortar and cement-stabilised CEB behaviour is described when interacting with the vertical reinforcement of the frame. The introduction of the concrete-steel rebar columns implicates changes in the materials constituent phases, but no changes in geometric configuration are recorded. The reinforcement is casted within the voids of Block-1 and Block-2 units (which arrangement is already described in cell-0) resulting in different material stiffness (and so mechanical responses) of the REV.

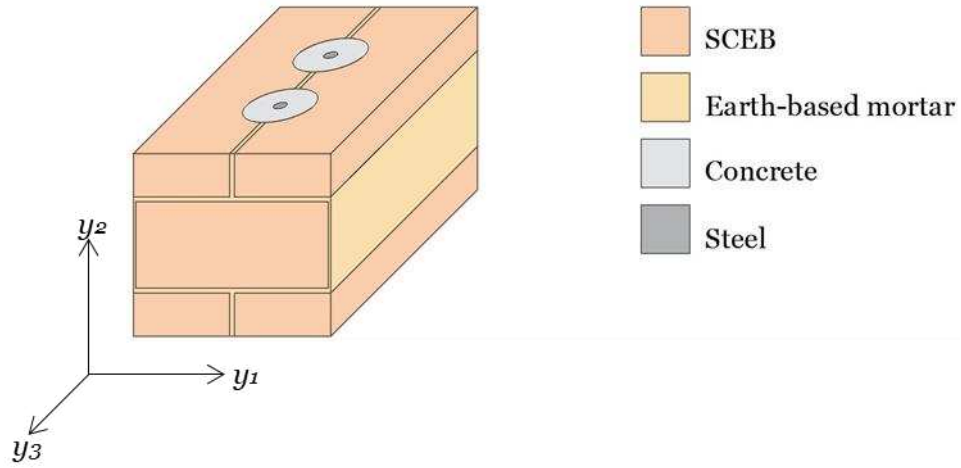


Figure 7.4: Representative Element Volume (REV) of the vertical reinforcement element.

- Material-2 / cell-1 (Figure 7.5): The interaction between the earth-based mortar and cement-stabilized CEB is analysed when the concrete beam, serving as the horizontal reinforcement of the frame, is cast into the masonry system. In this case, the third block typology is introduced (Block-3), resulting in significant alteration in terms of geometry and materials composition. Due to the reinforcement development along the horizontal axis, x_1 , the cell-1 has been designed to be periodical in just y_1 direction, making it not suitable for application in other directions. This REV specifically represents linear reinforcement along the x_1 axis, optimizing computational effort without compromising accuracy.

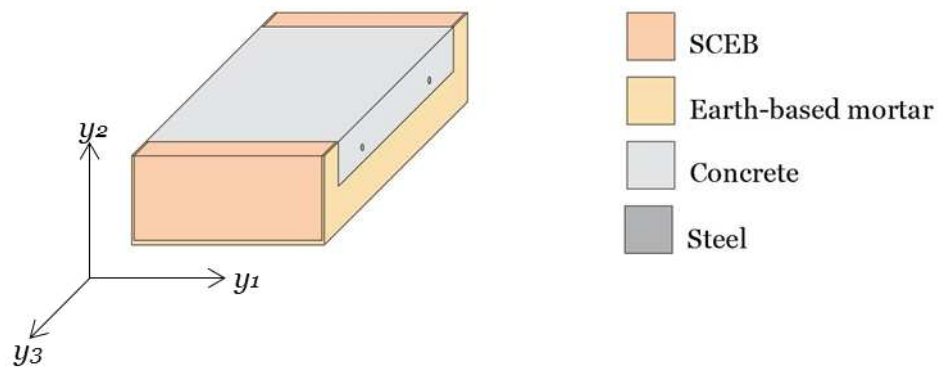


Figure 7.5: Representative Elementary Volume (REV) of the horizontal reinforcement element in concrete frame configuration.

- Material-3 / cell-1 (Figure 7.6): The earth-based masonry system is described at the intersection of the vertical reinforcement and horizontal reinforcement. To account for the horizontal reinforcement, cell-1

configuration is selected, resulting in periodicity limited to the y_1 axes. However, since the cell is used to describe a node, the application of Material-3 is limited to the nodal areas only, overcoming the periodicity issue. Due to the vertical steel bar passing through the reinforcement node, a steel rebar is added to this configuration compared to Material-2, ensuring an accurate representation of Material-3 stiffness.

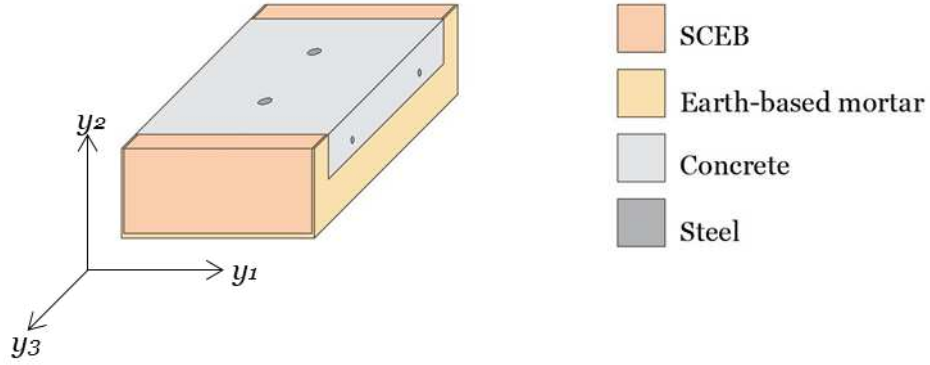


Figure 7.6: Representative Elementary Volume (REV) of the nodal reinforcement element in concrete frame configuration.

- BFRP-concrete-frame reinforced masonry panel:
 - Material-4 / cell-2 (see Figure 7.7): The behaviour of the mortar bed joint reinforced with a BFRP (Basalt Fiber Reinforced Polymer) grid is described. Cell-3 is a three-dimensional REV with the reinforcement net arranged in the $x_1 - x_3$ plane of the macrostructure. Despite the REV focus being on the enhanced mechanical performance of the mortar layer, with alterations in the x_2 direction being not strictly relevant, the three-dimensional aspect of it was studied so that the homogenised material properties could later be applied in 3D model to define the reinforced mortar layer. The net insert in the bed joint consists of two perpendicular fibre orientations: one aligned with the x_1 axis and the other with the x_3 axis. BFRP is an orthotropic material, meaning its mechanical properties depend on the fibre orientation. Orthotropic materials typically have two different elastic moduli: a higher modulus parallel to the fibre direction and a lower modulus perpendicular to it. In order to get a valid representation of the BFRP net mechanical response, two orthotropic materials were modelled to capture the fibre arrangement in the net: one using the higher elastic modulus parallel to the y_1 direction, and another using the parallel elastic modulus in the y_3 direction. To

represent the node elements in the net -where both fibre orientations are present- a third orthotropic material was introduced, combining the parallel elastic moduli along both the y_1 and y_3 directions.

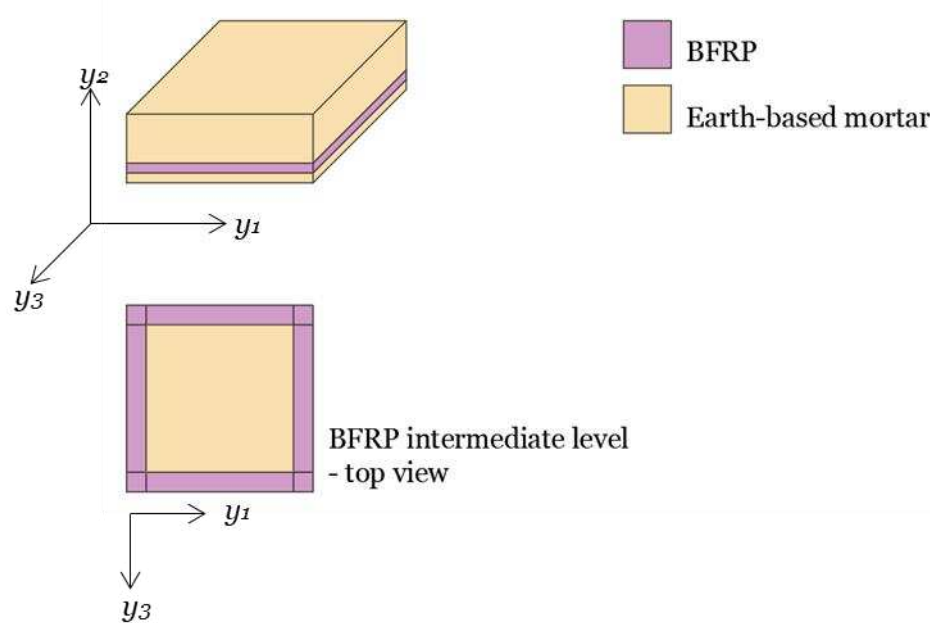


Figure 7.7: Representative Elementary Volume (REV) of the mortar bed joint element in mixed frame configuration.

- Material-5 / cell-o (see Figure 7.8): This cell is defined to describe the behaviour of the horizontal reinforcement element when a BFRP (Basalt Fiber Reinforced Polymer) grid is applied within the earthen masonry bed joint. The inclusion of the grid eliminates the need for Block-3, resulting in no geometric differences in the horizontal reinforcement area compared to Material-o/cell-o, as in Material-2. The only required modification is the adjustment of the material properties of the mortar bed joint, where the mechanical and physical properties of Material-4 are used to characterise the mortar.

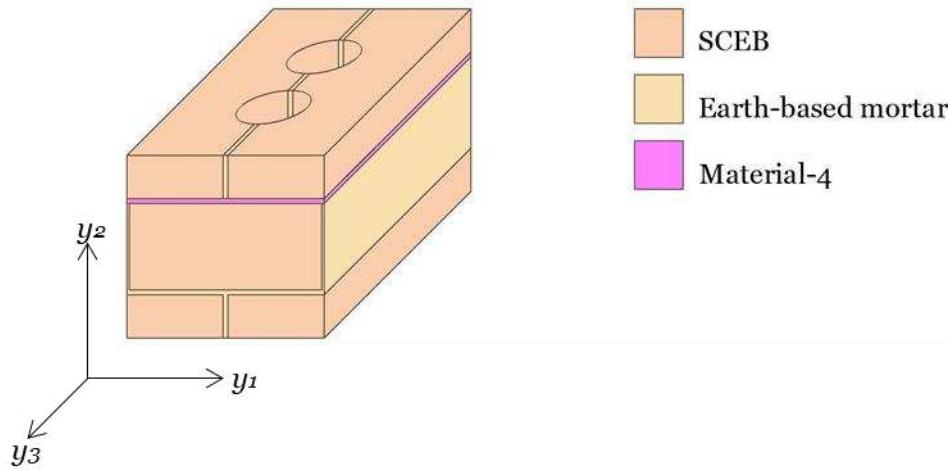


Figure 7.8: Representative Elementary Volume (REV) of the horizontal reinforcement element in mixed frame configuration.

- Material-6 / cell-o (see Figure 7.9): This configuration represents the intersection between the reinforced earth-mortar (Material-4) and the vertical reinforcement within the CEB at the node, described in Material-6. Cell-1 is unnecessary, as there are no geometric differences compared to Material-1/cell-o. Unlike Material-2, where the upper bed joint is characterized by earth-based mortar, here it is defined by the mechanical and physical properties of Material-4.

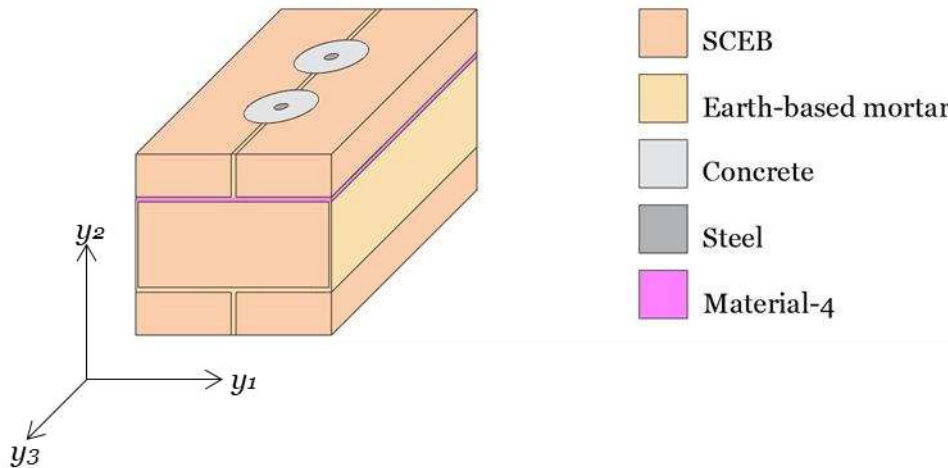


Figure 7.9: Representative Elementary Volume (REV) of the nodal reinforcement element in mixed frame configuration.

The homogenization problem was solved using the displacement method implemented in the Straus7 finite element (FE) code. After identifying the different Representative Elementary Volumes (REVs), for three-dimensional REVs only one-eighth of the volume

was studied due to the symmetry of the structure (see Figure 7.10). By doing so, computational effort is reduced. To maintain symmetry along the $y_2 - y_3$ plane in Material-5 and Material-6 both of the bed mortar joints define in the REV were considered as reinforced with BFRP net (therefore, Material-4 was assigned instead of the earth mortar properties).

The mesh size was defined to ensure at least two finite elements across the thickness of the mortar joints, allowing for accurate representation. Because of the mortar joints dimension is fixed at 5 mm height, solid 8-node elements with a side length of $l = 2.5\text{ mm}$ were adopted to design the REV mesh (see Figure 7.11a). Regular dimensions of the mesh were kept in all the cells. This mesh configuration was selected to optimize accuracy and computational efficiency, ensuring the mesh appropriately represents the REV geometry. The same approach was used to define cell-2 FE model, because reinforcement net described in the REV is 2 mm wide, a mesh size of 1 mm was set so that deformation within the fibre elements could be satisfactorily represented (see Figure 7.11b).

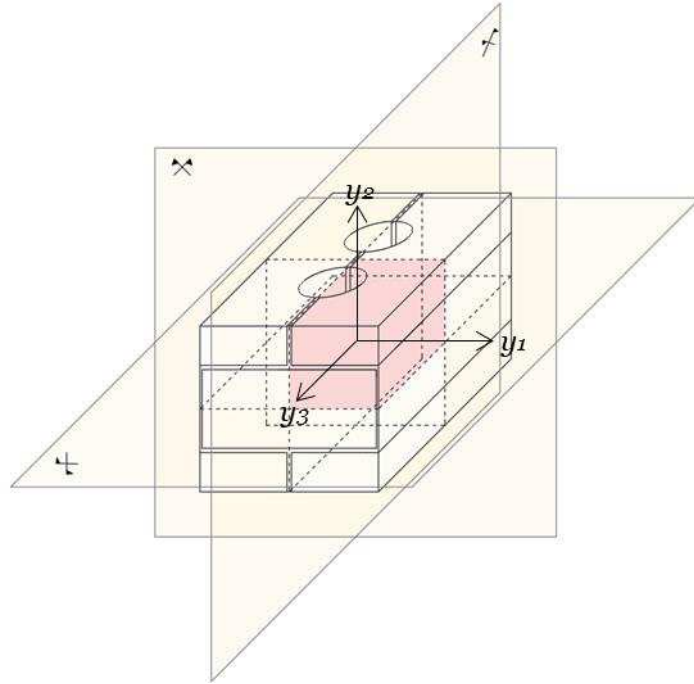


Figure 7.10: Definition of planes of symmetry for the elementary cell.

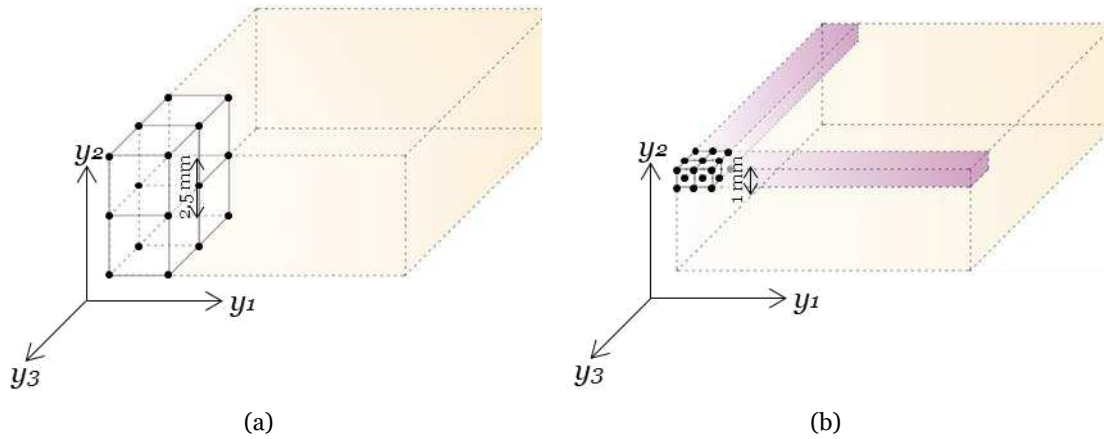


Figure 7.11: Mesh size definition for cell-0, cell-1 (a), and cell-2 (b) based on the smaller element dimension within the identified REV.

The displacement method is a computational approach used to define macroscopic properties of a heterogeneous material by analysing a small portion a microscopic level (REV). With this approach, displacements are applied at the boundary of the REV with the aim of reproducing the heterogenous macro-material stresses and deformations and strains distribution under different load conditions. Generally, the displacement is applied to one face of the REV at the time, while the other boundaries must be constrained to avoid any expansion or contraction in the other two directions. This allows for an isolated deformation along the axes in the simulation, thereby preventing unwanted movements. Due to the geometry of the elementary cell investigated which includes voids on the external face, specific considerations were made regarding the boundary conditions. Periodic boundary conditions are necessary at a micro level to assure realistic deformation and stress distribution. The perimeter of the voids, that appear along the boundary of the cell, was not strictly considered 'bounded' because of the periodicity assumption of the REV: the voids will replicate in a similar manner across adjacent cells. Therefore, from a macro-structural perspective, the voids can be considered as always being part of the internal structure of the material. For this reason, they were kept free from restrictions to capture the heterogenous material natural behaviour accurately. Unrestricted voids were essential because they allowed for a true representation of the material microstructure, ensuring that the simulation reflects the masonry behaviour under load while maintaining the integrity of the REV model.

The homogenization process was then extended to the vertical reinforcement, using GFRP bars immersed in a cementitious matrix. The adherence of this system was investigated through an experimental campaign: pull-out tests were performed to evaluate the bond strength of different typology of FRP bars and the cement mix (see section 6.3.2). Although this specific vertical reinforcement typology was not directly

tested together with the overall construction system, the homogenization process provides valuable insights into the mechanical properties of the vertical reinforcement section when different materials for the bars are used.

- GFRP-frame reinforced masonry panel
 - Material-7 / cell-o (see Figure 7.12), the earth-based mortar and cement-stabilised CEB behaviour is described when interacting with the vertical reinforcement of the frame. In this case a GFRP bars is selected, altering the properties of Material-1. No changes in the geometry of perimeter of the cell are recorded. Because of GFRP orthotropic properties, elastic (E) and shear moduli (G) relation for isotropic materials cannot be used to describe the GFRP material. Shear elastic moduli G_{ij} were defined in proportion to the elastic modulus E_{11} adopting the proportion found in [251] for GFRP composites elastic properties, while the orthotropic Poisson coefficient values ν_{ij} were taken from [252].

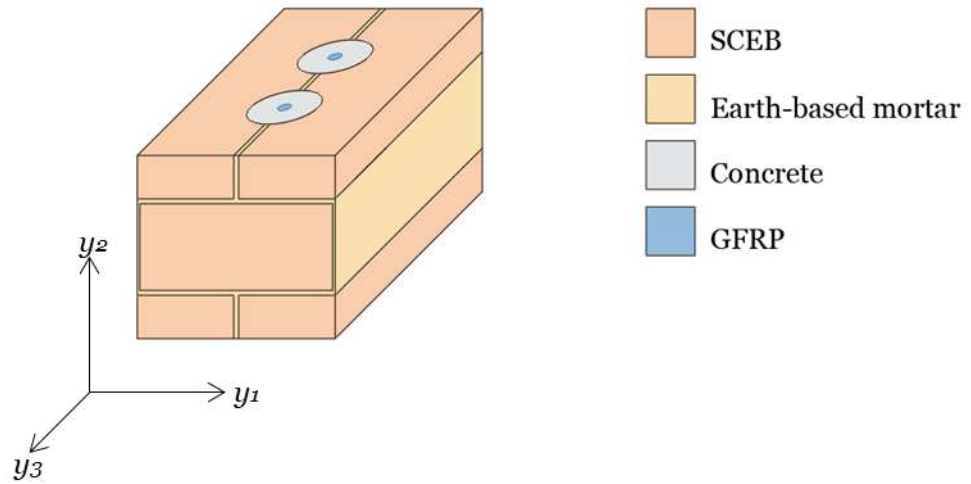


Figure 7.12: Representative Elementary Volume (REV) of the vertical reinforcement element in fibre frame configuration.

- Material-8 / cell-o (see Figure 7.13) describe the interaction of the new nodal area, consisting of concrete-GFRP bars, reinforced bed mortar joint and CEBs.

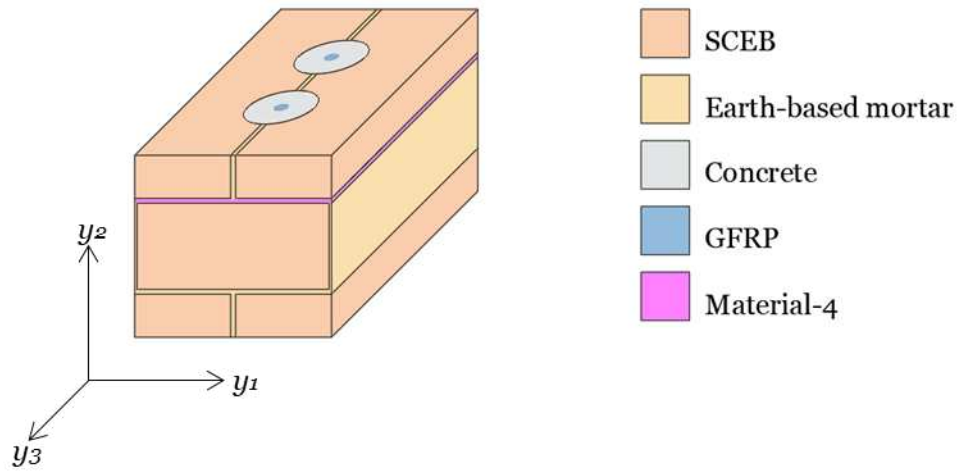


Figure 7.13: Representative Elementary Volume (REV) of the nodal reinforcement element in fibre frame configuration.

The resulting properties are listed in the table below.

	M-0	M-1	M-2	M-3	M-4	M-5	M-6	M-7	M-8
E_1 [MPa]	3180	5799	14420	14810	2300	3377	4331	5516	5720
E_2 [MPa]	2733	7098	3272	7857	250	2871	7230	6055	6153
E_3 [MPa]	5479	7994	12232	12888	2278	5574	5777	7587	7524
ν_{12}	0.204	0.13	0.192	0.128	0.24	0.196	0.104	0.137	0.14
ν_{21}	0.175	0.16	0.043	0.068	0.026	0.166	0.174	0.15	0.151
ν_{23}	0.122	0.143	0.085	0.109	0.028	0.249	0.168	0.14	0.149
ν_{32}	0.245	0.247	0.316	0.18	0.253	0.128	0.134	0.176	0.182
ν_{13}	0.16	0.254	0.251	0.254	0.044	0.17	0.261	0.251	0.262
ν_{31}	0.275	0.265	0.213	0.211	0.043	0.28	0.348	0.346	0.344
G_{12} [MPa]	742	1222	1140	1458	226	1040	1641	1220	1534
G_{23} [MPa]	1150	1805	1480	2378	127	1393	2074	1800	2066
G_{13} [MPa]	1160	2365	4799	4893	226	1190	2397	2355	2387

Table 7.1: Homogenised mechanical properties.

7.1.2 Linear analysis of earthen masonry subjected to vertical loads

The validation of the strategy approach was achieved by reproducing the unconfined uniaxial compression tests on the three typology of earth-based masonry panels: unreinforced masonry panel (UM), concrete-frame reinforced masonry panel (CRM), and mixed-frame reinforced masonry panel (MRM). A finite element (FE) model was

adopted to investigate the mechanical response of the numerical simulation, ensuring that the FE results closely matched the experimental data. The orthotropic models were characterized using the mechanical properties of the homogenised materials, which included properties such as Elastic modulus (E_i), Poisson's ratio (ν_{ij}), and Shear modulus (G_{ij}) and density (ρ). Because the models were meant to be tested in compression, corresponding elastic moduli and Poisson ratio were selected. First, a linear analysis was performed to verify the effectiveness of this approach, providing initial insights into the elastic behaviour of the system before delving into non-linear and numerical simulations.

A two-dimensional FE model was built for the linear analysis. Boundary conditions were first defined to represent the experimental set-up. The model was fixed at the base ($u_1 = 0, u_2 = 0, \omega_3 = 0$), to simulate the support provided by the concrete beam on which the panels were built. A vertical displacement was applied on top to reproduce the loads of the tests, which were performed on displacement control. The horizontal translation at top u_1 was not constrained since a certain level of displacement was allowed on the top layer block of the specimens. To verify that, maximum horizontal deformation ε_1^n was investigated at the first and tenth row, showing that the horizontal strain increases of one order of magnitude (see Table 7.2). This confirms the boundary conditions scheme of the FE model.

	<i>Maximum horizontal strain ε_2^{max}</i>					
	UM1	UM2	CRM1	CRM2	MRM1	MRM2
1 st row	0.00096	0.00048	0.00061	0.00059	0.00066	0.00088
10 th row	0.00256	0.00215	0.00234	0.00320	0.00324	0.00440

Table 7.2: Experimental horizontal strain recorded at first and last block layer within the panels.

The vertical displacement u_2 applied was defined in accordance with the maximum displacement achieved during the experimental campaign. Then an appropriate mesh size was defined for this analysis. 2D plane elements were used with dimensions b^a, h^a determined as follow: b^a set equal to half of the brick length plus half of the head mortar joint thickness, and h^a defined as the brick height plus two halves of the mortar joint bed thickness (see Figure 7.14a). The choice of the plate dimensions was dictated by the organization of the internal reinforcement. The reinforcement frame is consistently defined by a single block or a pair of half blocks. Therefore, the distinction of two half

blocks in the masonry façade is necessary to effectively and realistically assign the homogenized mechanical properties. When the horizontal reinforcement defined by the BFRP net has to be described, the definition of the plate as half of the block is no longer efficient. Due to definition of the REV for the horizontal reinforcement during the homogenisation process, the use of the 2D plate element as previously defined would imply the use of two reinforced mortar bed joint, instead of one. For this reason, rectangular plate with dimensions set as b^a , $h^b = h^a/2$ was used to define the plate elements across the reinforced horizontal mortar joint.

Since the simulation involved a linear compression test, no further refinements of the mesh were adopted in this case (see Figure 7.14b). The dimensions and arrangement of the plate elements were strategically defined as a base for more detailed analysis (e.g., non-linear compression behaviour) to allow for the calibration of the interfaces, while keeping the model set-up consistent.

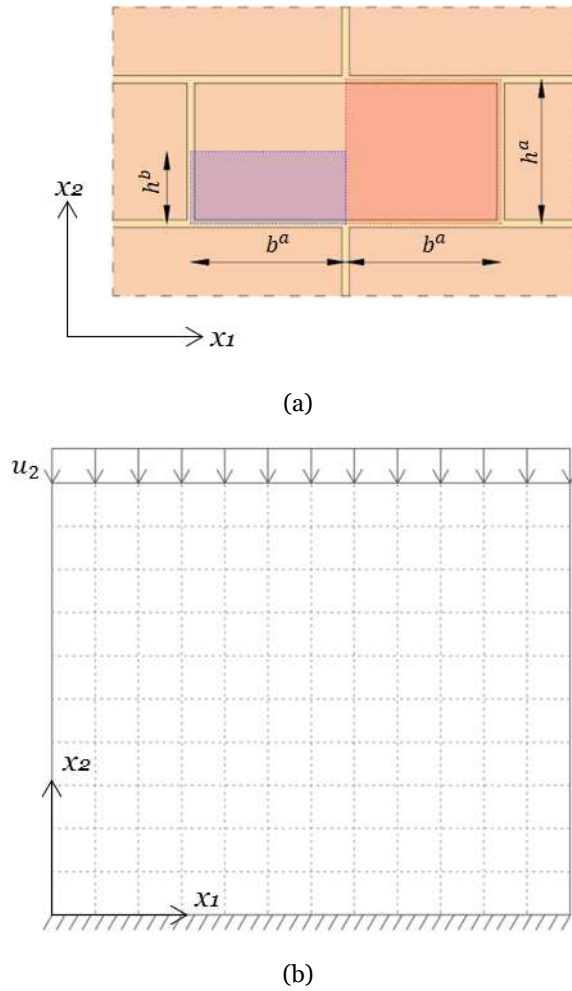
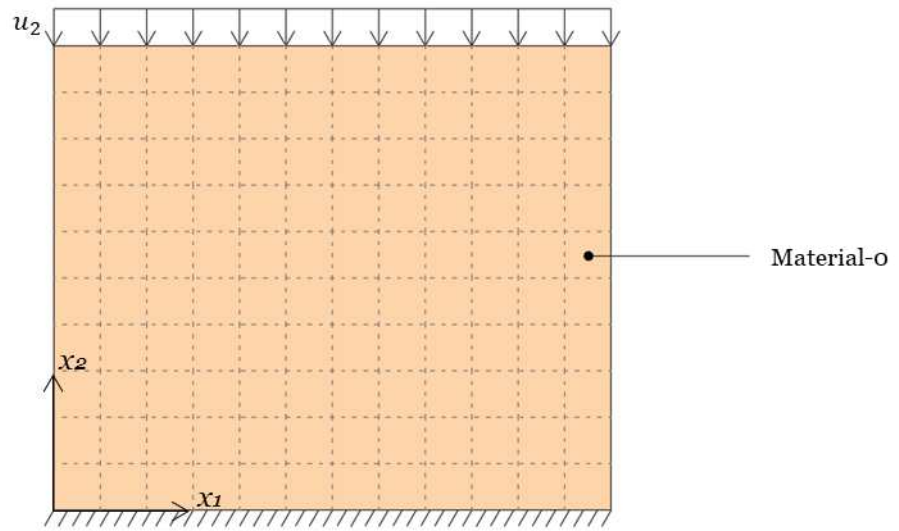
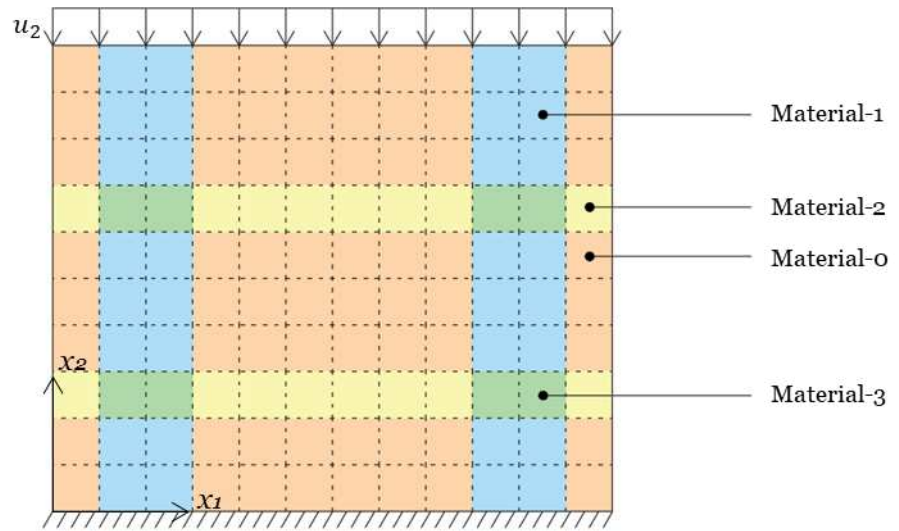


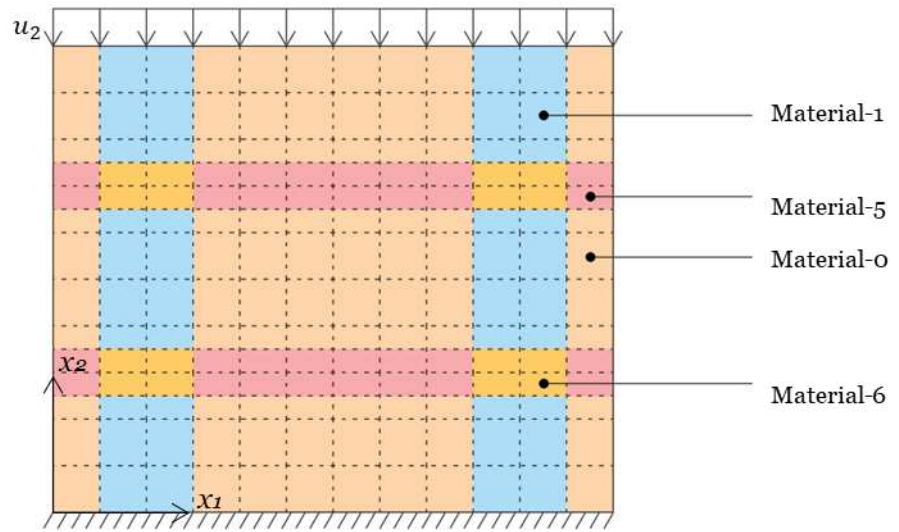
Figure 7.14: Definition mesh size (a), and mesh (b) adopted for FE analysis.



(a)



(b)

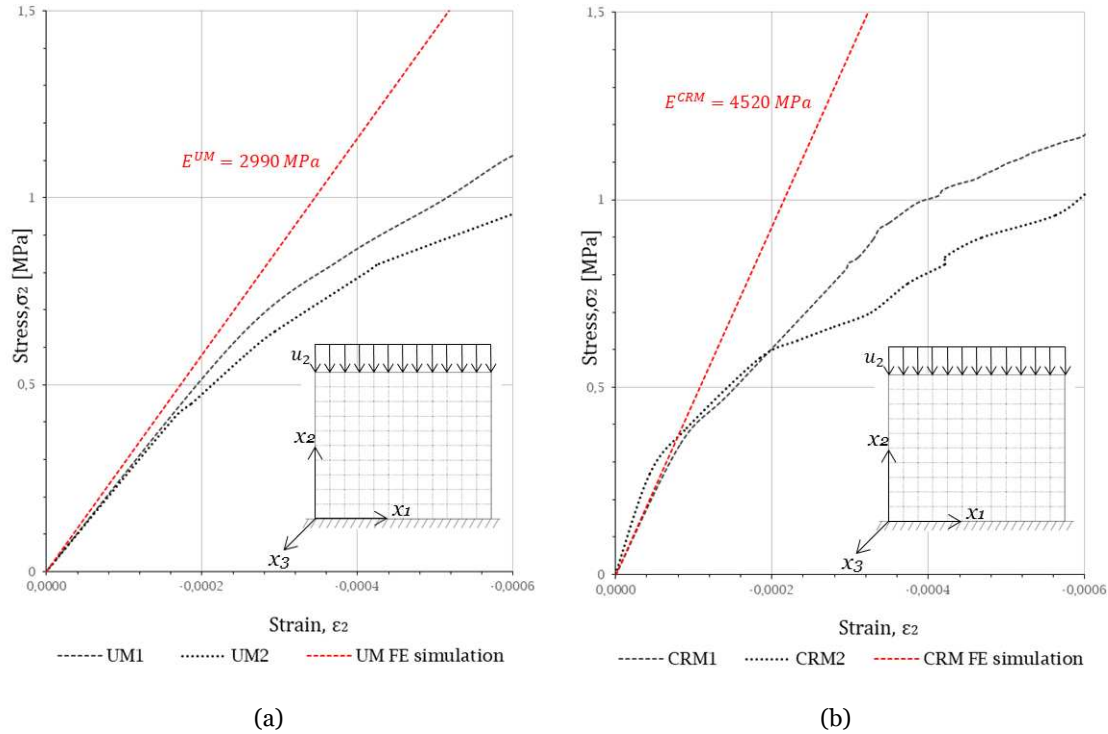


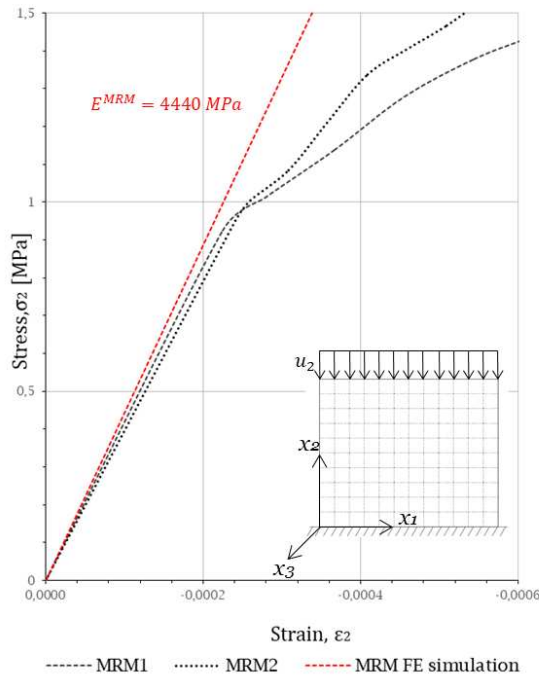
(c)

Figure 7.15: Material assignment in unreinforced configuration UM (a), concrete frame reinforced configuration CRM (b), and mixed frame configuration MRM (c).

To simulate the unreinforced panel, the homogenised material properties of stabilized compressed earth blocks (SCEB) and mortar, referred to as Material-0, were applied uniformly across all plates. In contrast, different homogenised materials were distributed throughout the plates to replicate the reinforcement systems of the concrete frame (Materials-1, -2, and -3) and the mixed frame (Materials-1, -5, and -6). These materials vary in their mechanical and physical properties across vertical elements, horizontal elements, and node connections (see Figure 7.15).

The numerical simulation results were then compared with the linear part of the experimental stress-strain behaviour of the masonry panels. A good agreement has been reached between numerical mean numerical analysis and the experimental behaviour also in the reinforced panels configuration (see Figure 7.16). This confirms the reliability of the numerical approach. As observed in the experimental tests, no difference is observed in the elastic modulus of the panels with the reinforcement frame typologies.

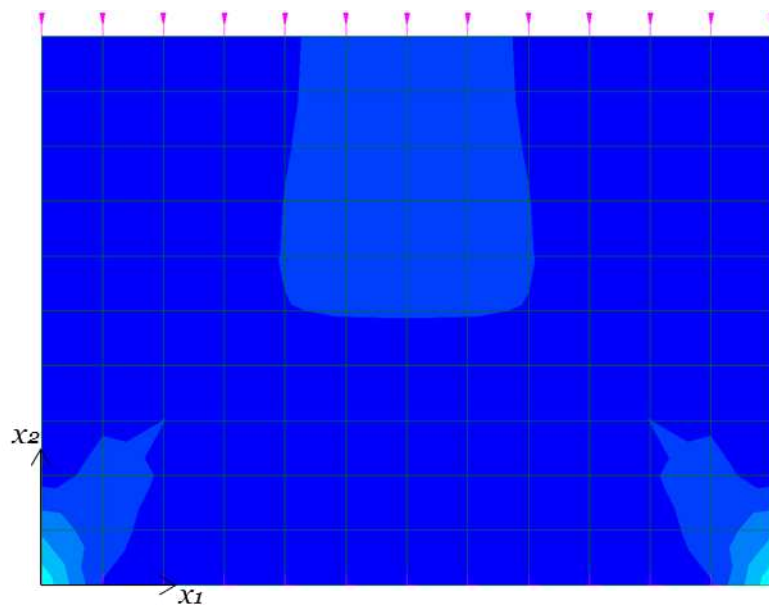
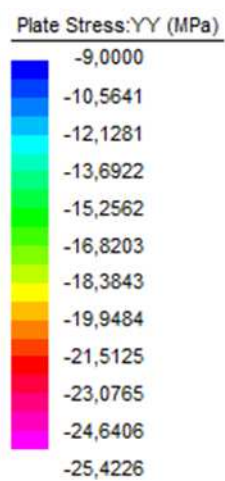




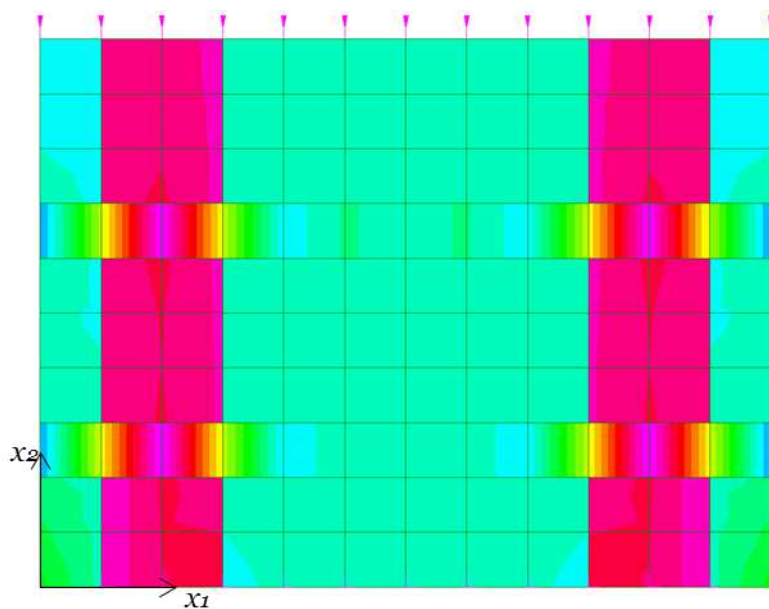
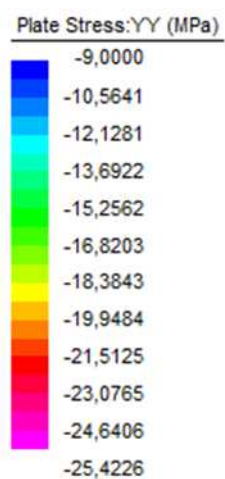
(c)

Figure 7.16: Linear numerical simulation of compressive behaviour for earth masonry panel configuration unreinforced UM (a), concrete frame reinforced CRM (b), and mixed frame reinforced MRM (c), plotted over experimental data.

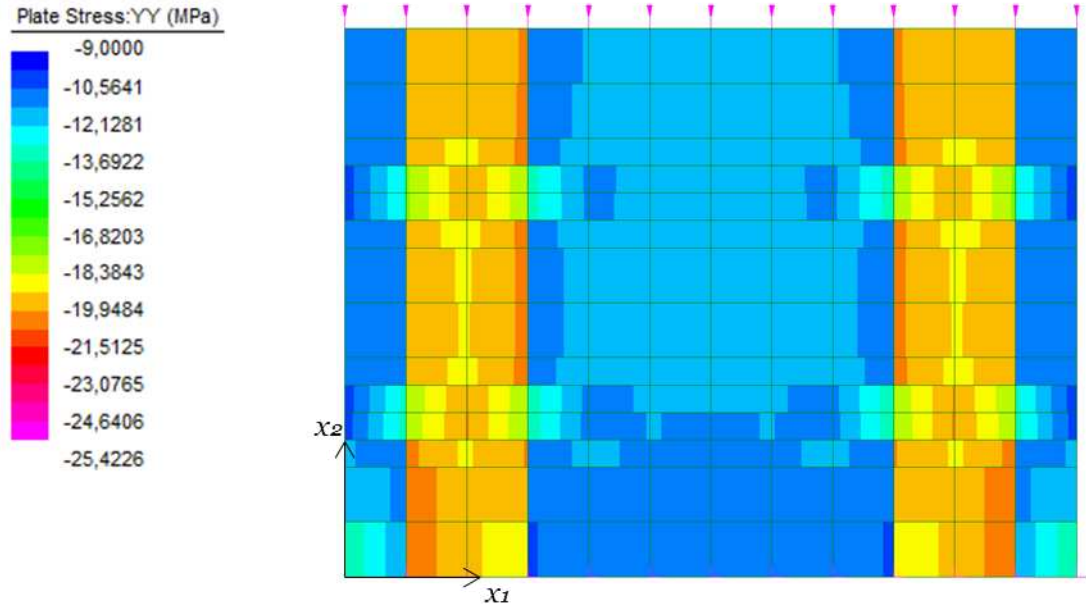
An analysis of the stress in the vertical direction was executed to understand how the internal stress is redistributed by the different reinforcement configurations (see Figure 7.17). The same scale of stress was set for each model to allow comparison. The unreinforced masonry model exhibits significant stress concentration at the bottom corners, in accordance with the boundary conditions. The model with concrete reinforcement frame (CRM) exhibits high stress concentration on the vertical concrete elements, which bear most of the compressive load, creating a new load path for the compressive stress. Moreover, in the remaining areas, only minimal stress redistribution is observed due to the lateral effect of the horizontal reinforcement. The model with vertical concrete and horizontal BFRP net reinforcement (MRM) shows the most uniform stress distribution. Despite having the same vertical reinforcement as CRM panels, the MRM configuration reduces the high stress concentration observed in the vertical elements of CRM, as the BFRP net facilitates more effective lateral stress redistribution.



(a)



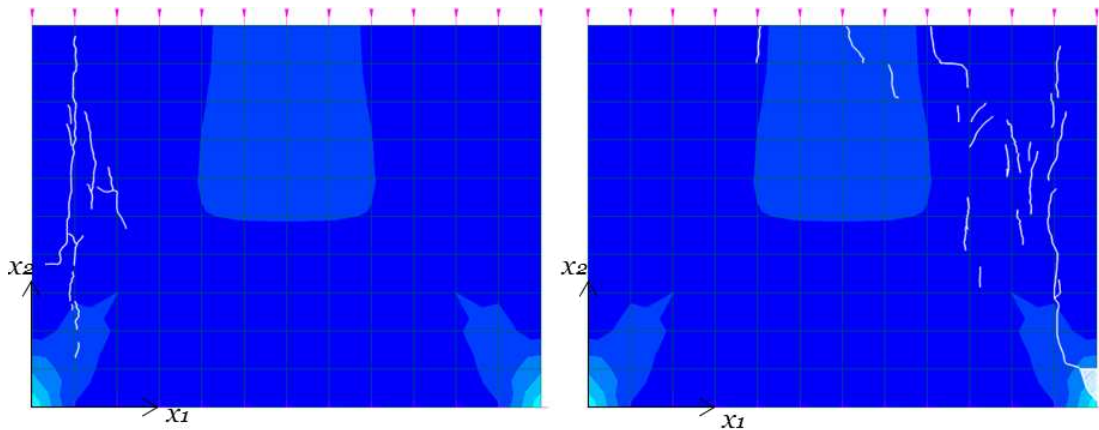
(b)



(c)

Figure 7.17: Internal stress σ_2 distribution in linear numerical simulation for earth masonry panel configuration unreinforced UM (a), concrete frame reinforced CRM (b), and mixed frame reinforced MRM (c), subjected to vertical load.

The stress distribution of the FE models reflects the different crack patterns among the specimens. While in the unreinforced masonry the cracks are developed along the height of the specimens in the CRM configuration the cracks developed along the axes of the vertical reinforcement is confirmed by the high stress concentration in these areas, while the horizontal reinforcement seems to play a minor role in the redistribution of the stress (see Figure 7.18c). However, the simulation of the MRM configuration confirms the effectiveness of the BFRP net inside the mortar bed joint on the stress redistribution. The vertical reinforcements are subjected to a lower stress which appear distributed along the specimen surface, in agreement with the crack patterns exhibited.



(a)

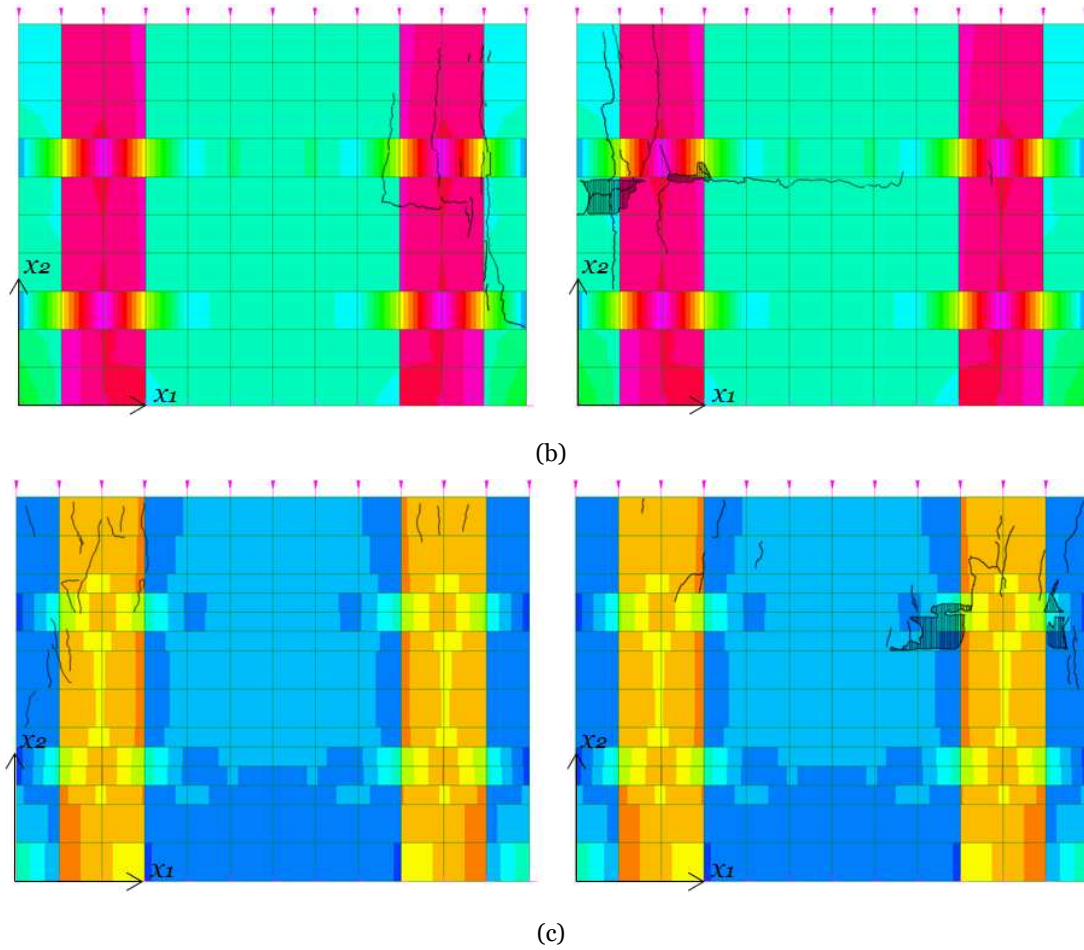


Figure 7.18: Comparison between internal stress σ_2 distribution in linear numerical simulation and crack pattern for earth masonry panel configuration unreinforced UM (a), concrete frame reinforced CRM (b), and mixed frame reinforced MRM (c), subjected to vertical load.

The simulation of the panel subjected to vertical load was conducted for the three models, and the numerical results showed good agreement with the linear phase of the experimental tests on the walls. This alignment demonstrates that the numerical model effectively captures the elastic behaviour of the masonry systems, effectively representing the stress-strain response during the initial loading phase.

Furthermore, it supports the methodology of adopting different REV's within the heterogeneous masonry medium. By using this approach, the simulations successfully captured the stress concentrations observed experimentally.

Given the reliability of the UM, CRM, and MRM numerical models, a numerical simulation of a masonry panel reinforced with a mixed frame was conducted. The steel reinforcement bars in the vertical elements were replaced with glass fibre-reinforced

polymer (GFRP) bars, resulting in a fibre-reinforced masonry (FRM) panel (see Figure 7.19). The panel was subjected to compressive loads under the same boundary conditions as the previous numerical models. The elastic modulus obtained from the numerical simulation, $E_2^{FRM} = 4360 \text{ MPa}$, indicates that the use of GFRP bars does not significantly alter the elastic response of the masonry panel in the x_2 direction (see Figure 7.20).

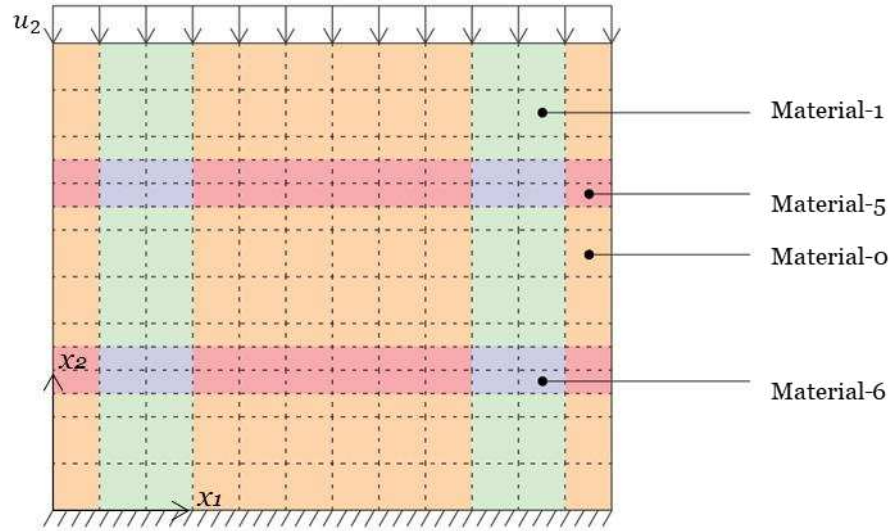


Figure 7.19: Material assignment in fibre frame reinforced configuration FRM.

A stress distribution analysis in the x_2 direction was conducted to investigate the redistribution of internal stresses, σ_2 , within the FRM system. The distribution pattern appears similar to that of the MRM system (see Figure 7.21), suggesting that the BFRP mesh plays a significant role in in-plane stress redistribution ($x_1 - x_2$).

Despite the similar stress distribution patterns, some differences can be observed between the MRM and FRM cases. The stress in the vertical columns varies along the length of the vertical reinforcement. This can be attributed to the lower stiffness of GFRP compared to steel, which may lead to a non-uniform stress distribution. This seems to have a minor influence on the redistribution of stresses in the masonry as well. A broader stress pattern is clearly visible in the central area of the panel, indicating a reduced confinement effect from the reinforcement while allowing for stress redistribution across the panel surface.

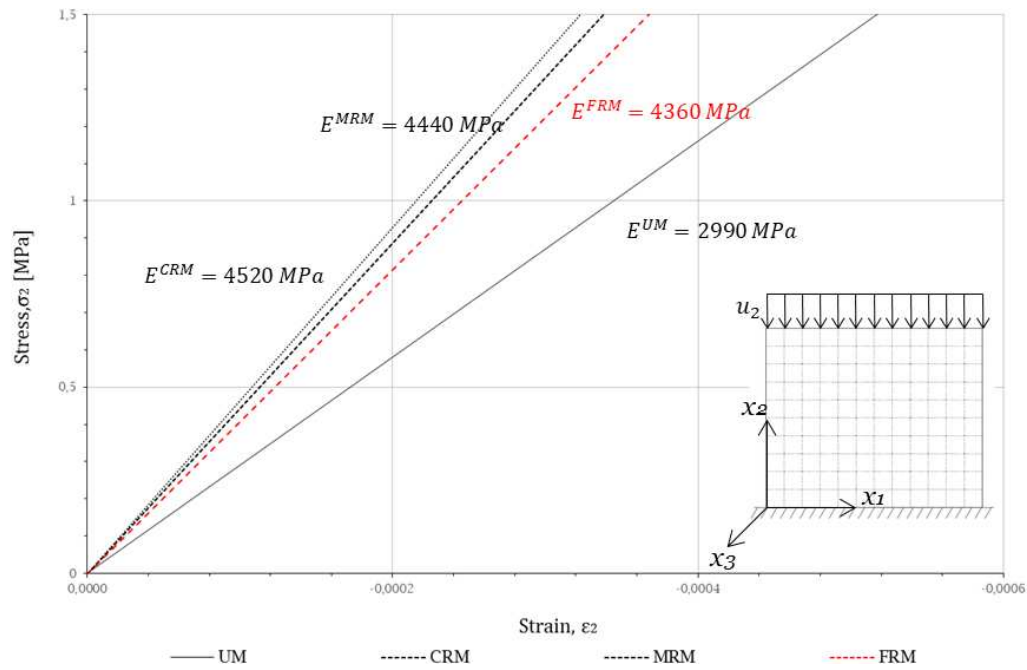


Figure 7.20: Linear numerical simulation for earth masonry panel configuration unreinforced UM, concrete frame reinforced CRM, mixed frame reinforced MRM, and fibre frame reinforced FRM.

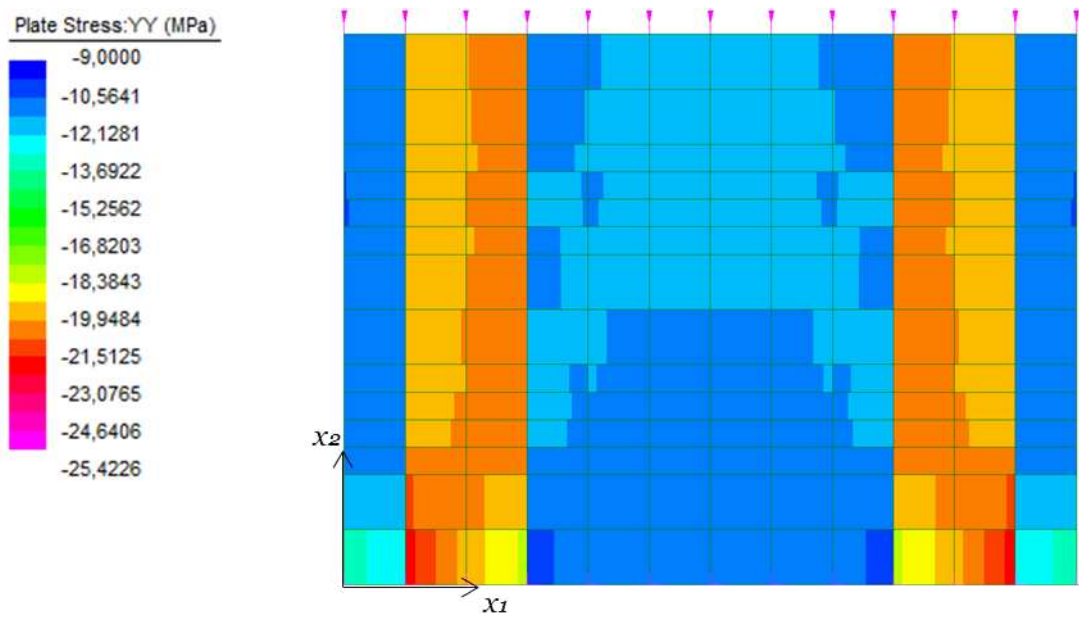


Figure 7.21: Internal stress σ_2 distribution in linear numerical simulation for fibre frame reinforced earth masonry panel configuration unreinforced FRM.

7.2 Non-linear Analysis of Earthen Masonry

Linear numerical analysis (see paragraph 7.1.2) validated the use of specific REVs in modelling to describe variations in geometry and material properties in the third dimension (depth) of the masonry system, so not directly detectable from a surface texture analysis. This validation enabled further investigation into the behaviour of masonry panels with earth-based mortar and SCEB under unconfined uniaxial compression using non-linear numerical analysis. The study of the non-linear behaviour of masonry systems is crucial for understanding their complex response once the elastic limit is reached. As a heterogeneous material composed of units and mortar joints with different properties, masonry behaves less predictably under load compared to other materials used in construction (e.g., steel). When earth is used as the base material for both units and mortar, this complexity increases due to the variability in soil mixes, which further complicates the description of the system behaviour under stress. To accurately represent the nonlinear behaviour of masonry systems, non-linear mechanical parameters are usually adopted, such as cohesion and friction angle, as in for Mohr-Coulomb numerical models. Due to the unique nature of the construction system and the use of a non-standardized material, data from the literature were not considered reliable for non-linear numerical modelling, as they could lead to altered results. For this reason, specific tests are required to determine non-linear parameters. The experimental campaign focused on testing the panels in compression, with different construction system, and did not include tests that could be used to directly acquire friction angle and cohesion – as with triplets tested to directly assess initial shear behaviour. Adopting the commercial software Ansys Workbench it was possible to define non-linear behaviour by applying a multi-linear constitutive law. A multi-linear law allows for the representation of different linear segment that correspond to different stages of the mechanical behaviour of the material at varying loads and strains, each defining different stiffness stages. The constitutive law was formulated based on experimental data from panel compression tests, without requiring additional testing.

7.2.1 Multi-linear constitutive law

The FE model was design in the same manner as for the linear one. Elastic properties obtained from the homogenisation process were used to build an orthotropic model. The orthotropic properties E_1 , E_2 , E_3 , ν_{12} , ν_{23} , ν_{13} , G_{12} , G_{23} , G_{13} were used to characterise the plates elements. In the linear model, two plate elements were used to represent each block (see Figure 7.14) in order to properly assign the homogenized properties while optimizing the construction of the numerical model. Non-linear models typically account for failure modes, crack propagation, and load redistribution; therefore, the failure

typology of the masonry panels under compression was considered. As described in paragraph 5.1, cracks were vertically passing through the x_2 direction of the brick and the head mortar joint. To simulate potential failure in the unit, the plate configuration selected in the linear model was left unaltered, with only a refined mesh applied for data accuracy.

The boundary conditions were kept unchanged to simulate the behaviour of the panel under vertical loads: a fixed base ($u_1 = 0, u_2 = 0, \omega_3 = 0$), with a displacement of $u_2 = 3 \text{ mm}$ applied at the top, as determined from the experimental data (see Figure 7.22).

A distinct multilinear constitutive law was defined for each the numerical model typology, representing the unreinforced masonry panel UM, the masonry panel reinforced with a concrete frame CRM, and the masonry panel reinforced with a mixed frame MRM. The stress-strain behaviour was obtained by averaging the stress-strain curves obtained experimentally (see Figure 7.23).

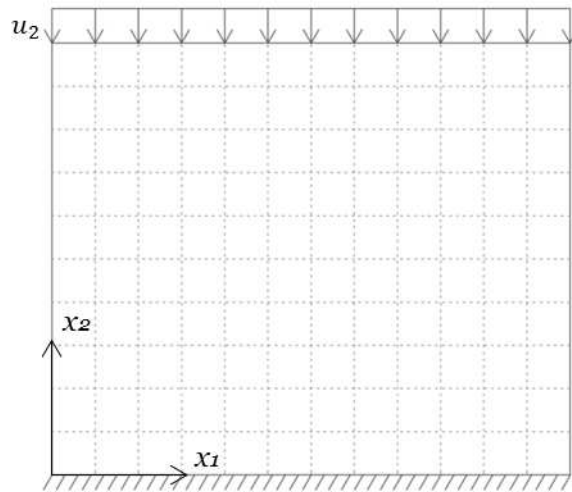


Figure 7.22: Boundary condition for FE non-linear analysis.

To describe the non-linear behaviours the multi-linear constitutive laws were built as follow. The first point of the constitutive law was defined by identifying the beginning of the non-linear (or plastic) phase, defying the point $(0; \sigma_2^{(1)})$. An adequate number of points was selected for each curve in order to accurately describe the change in stiffness of the panels during different loading stages, until peak load is reached. Despite being an efficient method to describe non-linear behaviour it cannot represent the post-peak phase. Since masonry do not usually exhibit post peak behaviour the constitutive law was used, even if it represents a limitation for the MRM panel typology (see Figure 7.24). To obtain only plastic strain data, the maximum elastic strain value was subtracted from the experimental strain at each point. By doing so, the points define the plastic strain at the

corresponding compressive strength ($\varepsilon_2^{p(n)}; \sigma_2^{(n)}$), as showed in Figure 7.23. The data for the multilinear constitutive law are presented below (see Table 7.3). A multilinear law was defined for each specimen typology based on the experimental data.

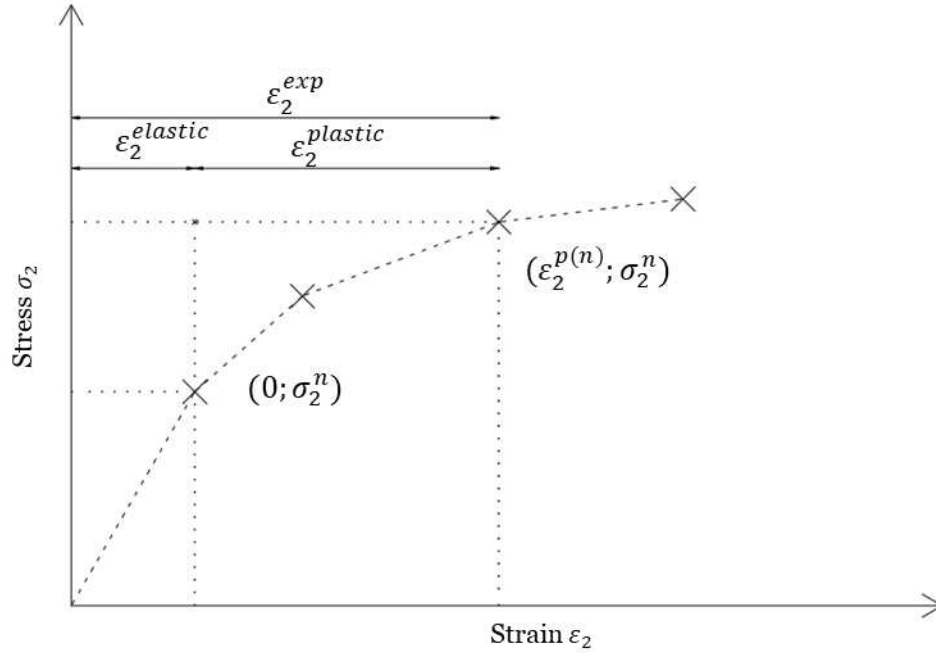
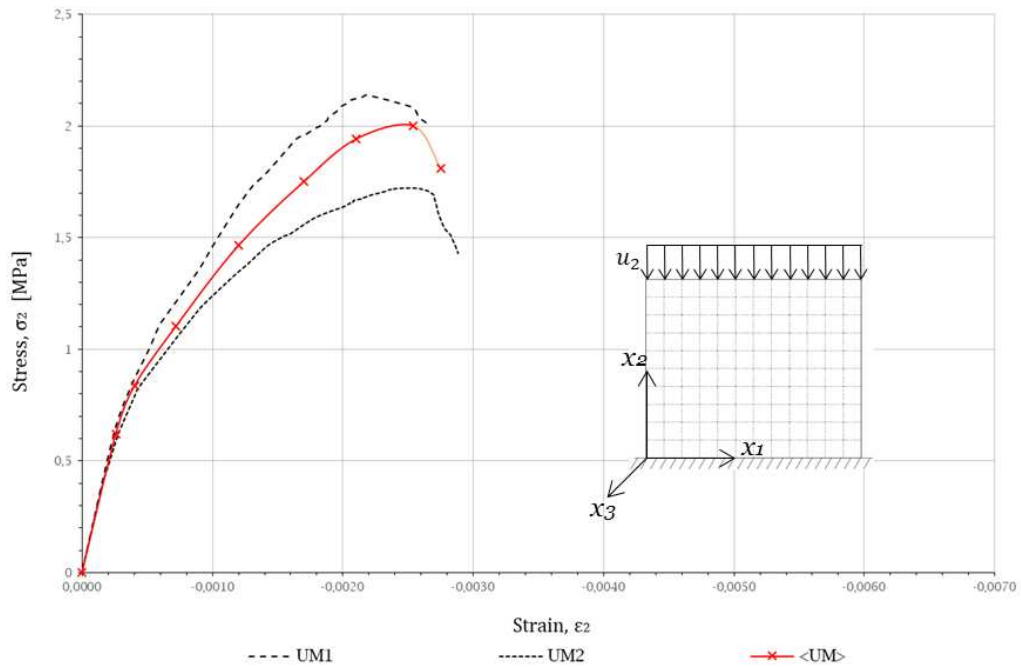
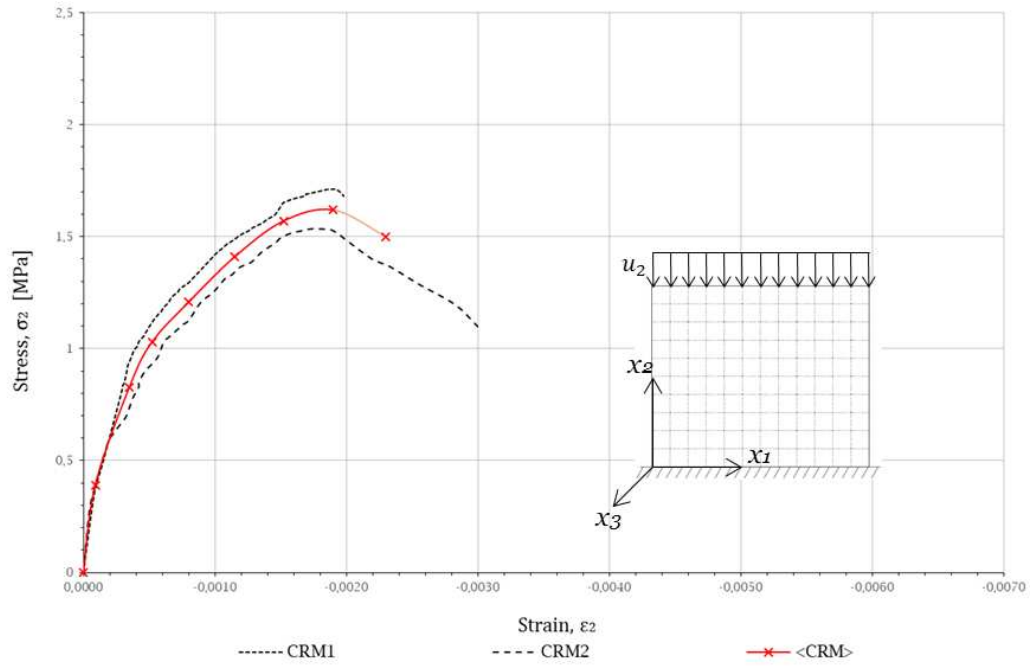


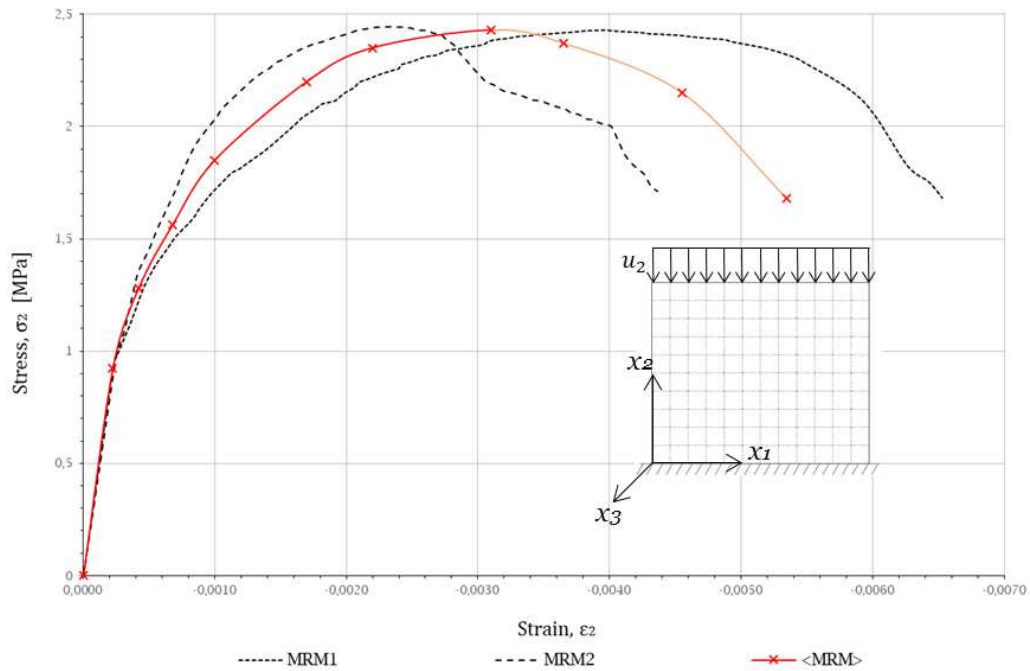
Figure 7.23: Definition of points for multilinear constitutive law in Ansys Mechanic.



(a)



(b)



(c)

Figure 7.24: Mean compressive behaviour of CEB masonry panel with identification of key point for multilinear constitutive law for (a) unreinforced configuration, (b) concrete frame reinforced configuration, and (c) mixed frame reinforced configuration.

Point	UM		CRM		MRM	
	ε_2^p	σ_2 [MPa]	ε_2^p	σ_2 [MPa]	ε_2^p	σ_2 [MPa]
Pt 0	0	0.62	0	0.39	0	0.92
Pt 1	0.000144	0.84	0.00026	0.83	0.000196	1.28
Pt 2	0.00046	1.10	0.00043	1.03	0.000456	1.56
Pt 3	0.00094	1.47	0.00071	1.21	0.000776	1.85
Pt 5	0.00144	1.75	0.00106	1.41	0.001476	2.20
Pt 6	0.00184	1.94	0.00143	1.57	0.001976	2.35
Pt 7	0.002281	2.00	0.00181	1.62	0.002876	2.43

Table 7.3: Identification of points for multi-linear constitutive law.

7.2.2 Numerical model calibrated on experimental data

The finite element (FE) model was subsequently characterized; however, the failure mode was not adequately controlled by the initial input parameters. To address this issue and better capture non-linearity, the contact interfaces were calibrated. Due to the model geometry, each block unit is represented by two 2D plate elements, requiring the definition of contacts at both the mortar/unit interface and the central block/block interface.

In Ansys Workbench, contact elements are classified as either "body" or "target." The body contact corresponds to the more deformable surface that interacts with another element, while the target surface is part of a stiffer element that remains relatively stationary. Given the symmetrical masonry configuration, the uniform boundary conditions, and the comparable stiffness levels of the interacting components, no distinction was made between the contact and target surfaces; accordingly, both the mortar/unit interfaces and block/block interfaces were modelled symmetrically.

To describe the interaction between contact pairs, different contact typologies can be assigned (see Figure 7.25):

- Bonded: Prevents any relative motion or separation between the contact pair.
- No Separation: Allows sliding without friction but prohibits any separation.
- Rough Contact: Prevents relative sliding while permitting separation.
- Frictionless Contact: Permits both sliding without resistance and separation.

- Frictional: Permits sliding only when the applied force exceeds the frictional resistance, and separation is allowed.

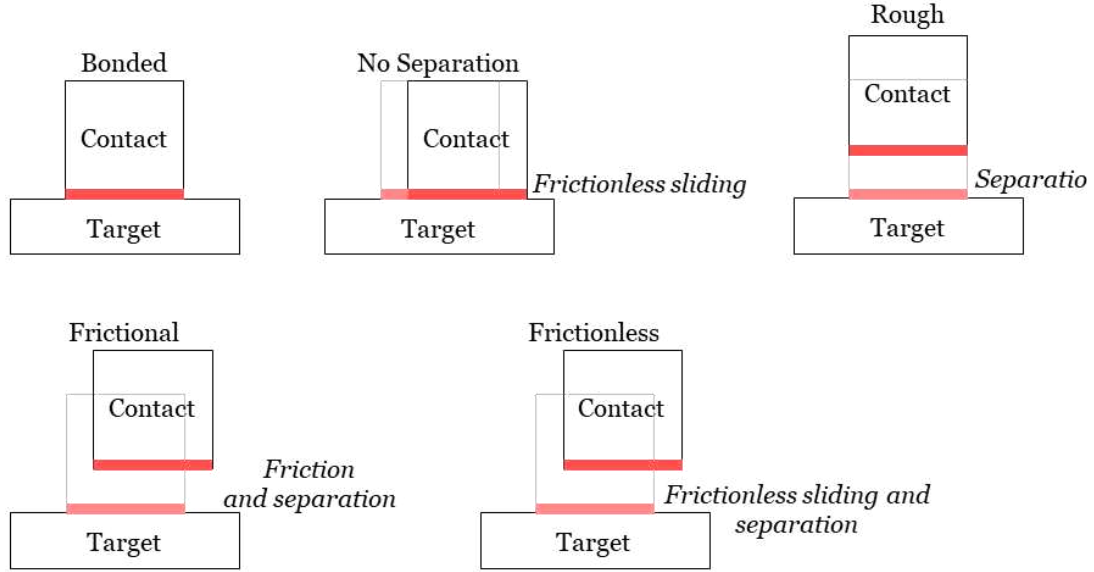


Figure 7.25: Ansys Workbench contact typologies.

A pure penalty formulation was selected to model the interfaces. This numerical technique introduces contact stiffness to avoid penetration between the bodies. It is a computationally efficient formulation that allows for quicker analysis. It is sensitive to the stiffness value of the contact that prevents the body to compenetrates—once the stiffness value is exceeded body compenetrations happens.

The unreinforced panel was first addressed, because of a reduced number of parameters to calibrate. The block-mortar contact was defined as frictional, different coefficients of friction were defined between vertical contact – head mortar joint – and horizontal contact – bed mortar joint. The coefficient of friction was set as $\mu^{hj} = 0.6$ for bed joint. Since in a masonry system the head joint is usually defined as weaker than the bed joint, the coefficient of friction μ^{vj} was reduced of 1/3. Even if the contact stiffness represents the capacity of resist penetration, the pure penalty formulation acts like a spring model, so the normal contact stiffness k_n was defined as in [180], where E_2^u and E_2^m are the elastic moduli in x_2 direction of the unit and mortar, respectively, and e the thickness of the mortar joint.

$$k_n = \frac{E_2^u E_2^m}{e(E_2^u - E_2^m)} \quad (6)$$

Since tensile splitting of the blocks was observed during the experimental campaign, the block/block contact was defined as rough typology so that separation can occur without relative movement between the two plate elements (see Figure 7.26). In this case there was only one material typology and a dimensionless contact so k_n cannot be defined as

per the mortar/block interface. Contact stiffness can also be controlled by a factor k . This factor scales the contact stiffness automatically set by the program. For the block/block interface a contact stiffness factor was set as $k = 1.2$ to simulate a rigid contact behaviour and prevent compenetration.

The model was calibrated through the experimental data. Since it is an orthotropic model, the stress-strain behaviour in the x_2 direction was monitored, along with a comparison between numerical and experimental horizontal strain (ϵ_1^n) to assess the deformation status at the 10th and 5th block layers, ensuring proper model calibration.

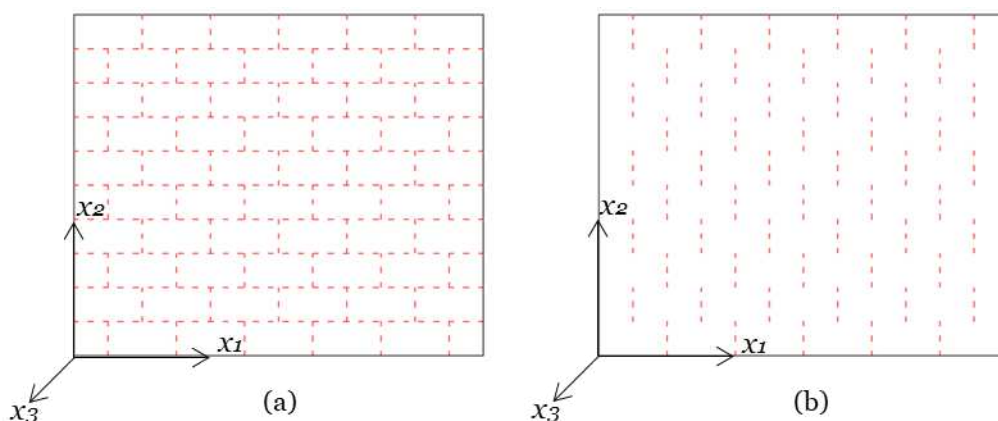


Figure 7.26: Contact identification in unreinforced masonry panel (a) mortar-block interface, and (b) block-block interface.

The tenth block course was selected for detailed inspection to verify the absence of excessive lateral deformation, while the fifth course served as a general reference. At mid-height, the unreinforced UM specimens exhibited greater deformations compared to the 7th course (see Table 7.4), whereas the CRM and MRM specimens showed higher deformations. Consequently, the fifth course was chosen to study the stress distribution and deformation patterns within the masonry panels. A good agreement of the numerical data was obtained for both the horizontal strain and the stress-strain behaviour in x_2 direction (see Figure 7.27). The absence of post-peak behaviour in the numerical simulation does not significantly alter the mechanical response of the model. finding a good agreement with the drop in resistance after peak load of UM1 and UM2 specimen.

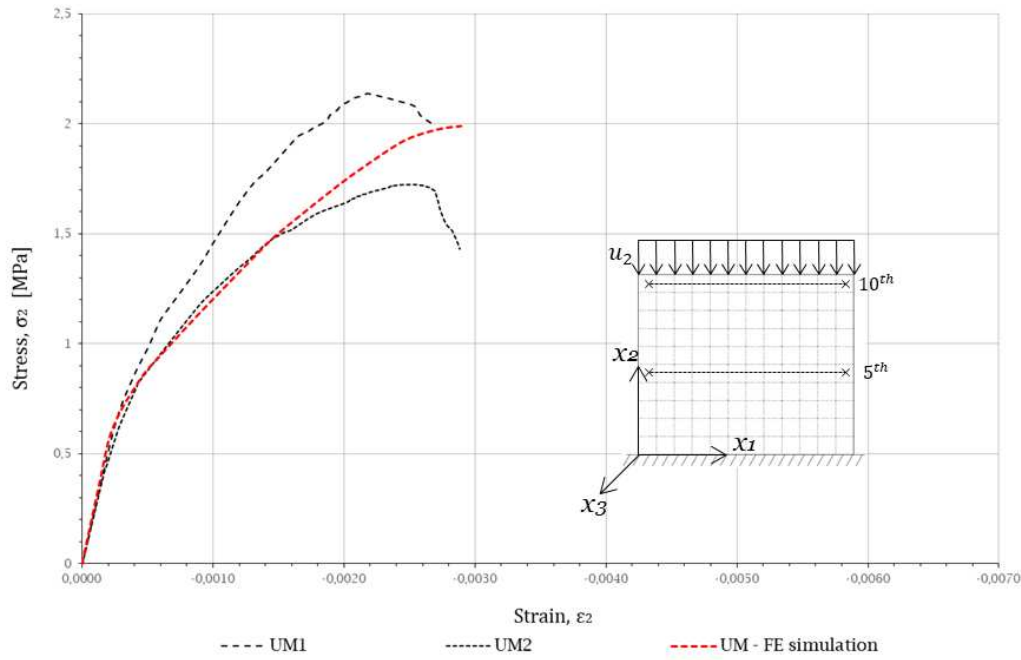


Figure 7.27: Stress-strain plot of numerical simulation of unreinforced earth masonry panel under compressive loading, plotted against experimental data.

Block layer	UM	
	Experimental $\langle \varepsilon_1 \rangle$	Numerical ε_1
10th	0.0016	0.00144
5th	0.00158	0.00146

Table 7.4: Experimental and numerical comparison of deformation in x_1 direction in UM panel.

With the mortar-brick and block-block contacts defined the numerical model of the wall reinforced with the concrete frame CRM was developed. The contacts in the areas where the reinforcement frame extends were carefully calibrated.

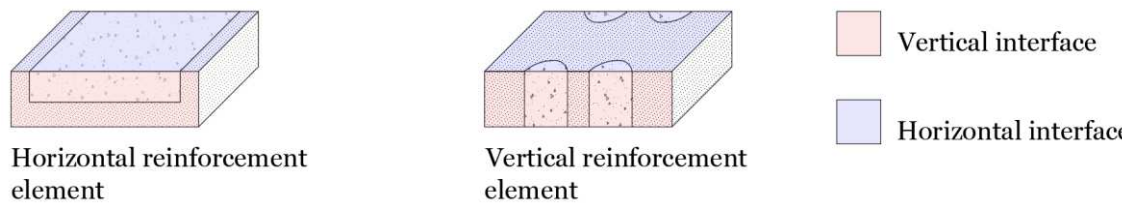


Figure 7.28: Vertical and horizontal interfaces of the concrete reinforcement frame.

To calibrate the contact between plate elements representing the reinforced concrete frame, the experimental behaviour of the CRM specimens and the propagation of cracks were observed.

- Vertical reinforcement: In both CRM1 and CRM2, cracking initiated at the vertical reinforcements, likely due to the material stiffness difference. Hence, horizontal and vertical interfaces of the vertical reinforcement elements were calibrated as follow:
 - Vertical interface:
For this reason, internal vertical contacts were modelled using the same properties as the mortar-brick interface (contact type, frictional, coefficient of friction $\mu^{vr1} = 0.2$ and $k_n = 45 \text{ N/mm}^3$), creating a weaker line within the masonry system.
 - Horizontal interface:
For the horizontal contacts of the vertical reinforcement plates, a frictional contact type was always selected, with the coefficient of friction increased to $\mu^{vr2} = 1$. Although unrealistic, a coefficient of friction of 1 was calibrated to simulate the resistance provided by the continuous reinforcement system.
- Horizontal reinforcement: Regarding the horizontal reinforcement, the experimental results showed no direct fracture at the reinforcement beam element, if not for localised cracks.
 - Vertical interface:
The vertical contact elements within the beam were therefore modelled as no-separation, allowing translation in the x_2 direction but preventing detachment (see Figure 7.25), to represent the continuity of the beam element.
 - Horizontal interface:
The horizontal contact elements between respectively, the 7th and 8th course and the 3rd and 4th, (which host the beams) were modified compared to the other bed joints (see Figure 7.29d): a higher friction was assumed due to the construction method applied, which involved spreading mortar onto fresh concrete, creating a high level of adhesion. Through parametric analysis, a coefficient of friction of $\mu^{vr2} = 1.2$ was defined.
- Nodal element: No horizontal cracks were registered at the nodal elements of the frame. The connection of the plate elements of the vertical and horizontal

reinforcement (node) were modelled as rough contacts thus, preventing translation in the x_1 direction (see Figure 7.29c), assuring good connection between the frame elements.

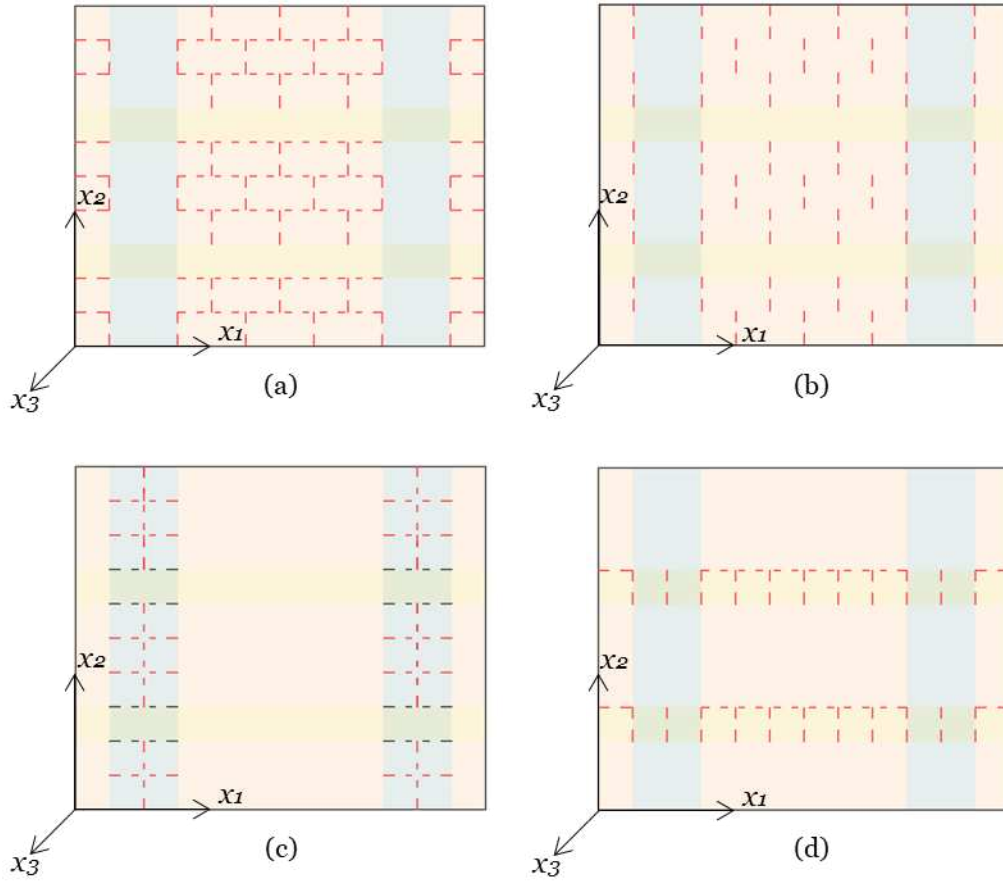


Figure 7.29: Contact identification in concrete frame reinforced earth masonry panel (a) mortar-block interface, (b) block-block interface, (c) vertical reinforcement, and (d) horizontal reinforcement.

The numerical model (CRM-FE) exhibits slightly greater deformation at peak stress compared to the experimental results, particularly for CRM1 (see Figure 7.30). Despite this discrepancy, the model remains representative, accurately capturing the general pre-peak behaviour and providing a reliable approximation of the compressive response. The absence of post-peak behaviour in the numerical model - due to the limitation of the multilinear constitutive law - is acknowledged but does not compromise its overall predictive validity for practical applications.

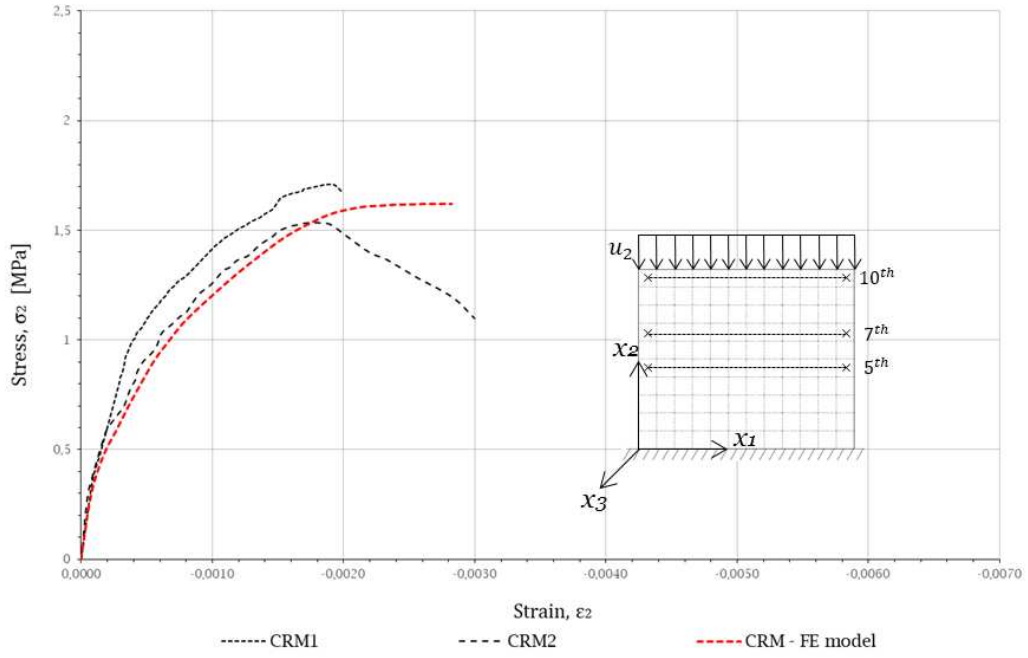


Figure 7.30: Stress-strain plot of numerical simulation of concrete frame reinforced earth masonry panel under compressive loading, plotted against experimental data.

The numerical model exhibits a strong match with the experimental horizontal deformation $\langle \varepsilon_1 \rangle$ particularly for the 7th block layer (see Table 7.5). Slight deviations are observed in the 10th and 5th layers, but they remain within acceptable margins. This reinforces the reliability of the numerical model in capturing the mechanical behaviour.

Block layer	CRM	
	Experimental $\langle \varepsilon_1 \rangle$	Numerical ε_1
10 th	0.0018	0.0015
7 th	0.00217	0.00215
5 th	0.00122	0.0014

Table 7.5: Experimental and numerical comparison of deformation in x_1 direction in CRM panel.

For the mixed reinforced frame panel (MRM), the plates of the third, fourth, seventh, and eighth courses were divided in half along the x_1 -direction to assign the material properties of different components while preserving the original geometry (see Figure 7.15c). To ensure that the coupled half-plates function as a single element, a bonded contact type was applied (see Figure 7.31c).

The experimental behaviour of the MRM specimens and their propagation of cracks were considered to assign the contacts properties within the FE model.

- Vertical reinforcement:
 - Vertical interface:

The observation of a different crack pattern in MRM1 and MRM2 specimens negates the hypothesis of weakness in the vertical contacts between the plates of the vertical reinforcement as in CRM configuration. The redistribution of stress lead to a more uniform stress state (see paragraph 7.1.2), therefore a different typology of contact would have altered the results.
 - Horizontal interface:

Due to the similar crack propagation to the unreinforced configuration and the presence of BFRP net as horizontal reinforcement which allows for a more uniform internal stress redistribution, the horizontal contacts were left unaltered compared to the UM model.

These contacts were left unaltered compared to the unreinforced configuration (UM), as no significant differences were observed in the initiation of cracks between the MRM and UM panels.

- Horizontal reinforcement: Regarding the horizontal reinforcement, the experimental results showed a very limited crack pattern along the first horizontal reinforcement mortar bed joint.
 - Horizontal interface:

The plates representing the bed joints reinforced with BFRP mesh were also assigned the same properties as the other bed joints, as the mesh is embedded within the joint thickness ($x_1 - x_3$ plane) and does not alter its behaviour.
 - Vertical interface:

The vertical contacts of the half-plates representing the horizontal reinforcement were modelled as bonded. Experimentally, the presence of the BFRP mesh allows for a high level of deformation without tensile failure of the elements. The bonded contact applied to the halves of the courses better represents this behaviour.

The compressive behaviour is well represented by the numerical model until the peak load is reached (see Figure 7.32). Differently from the other specimen. MRM present a developed post-peak phase which cannot be represented with a multi-linear constitutive law.

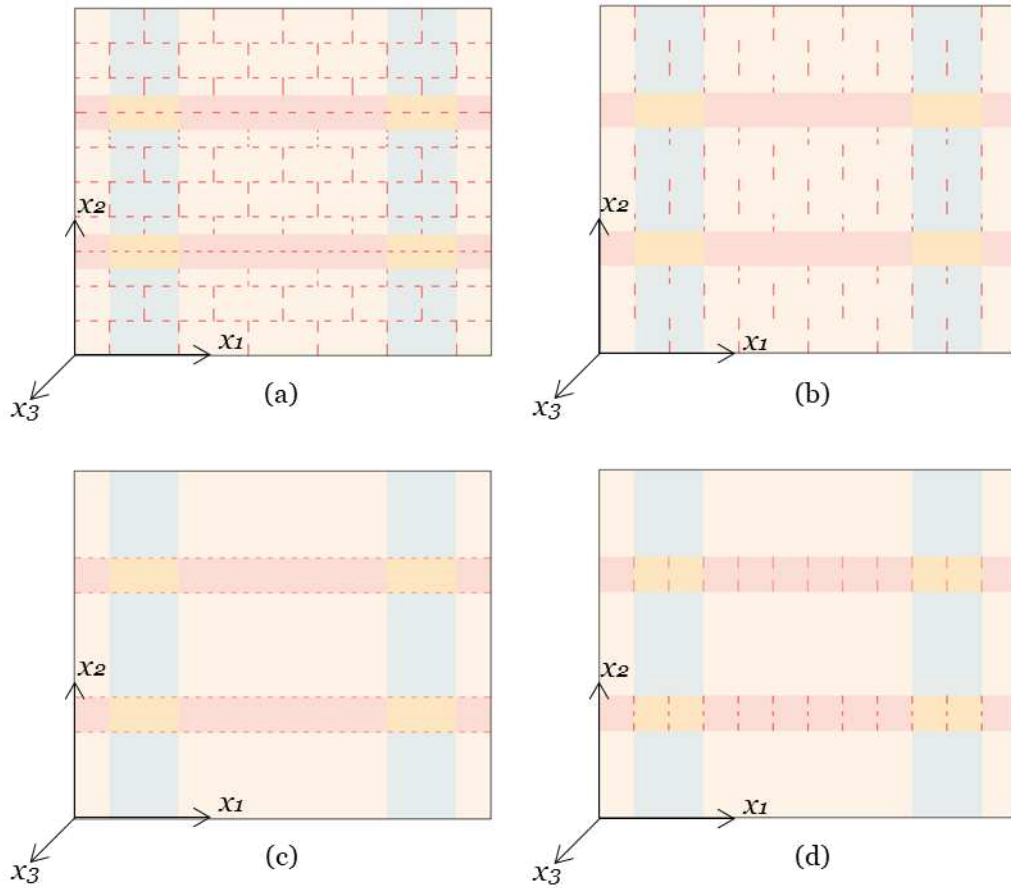


Figure 7.31: Contact identification in concrete frame reinforced earth masonry panel (a) mortar-block interface, (b) block-block interface, (c) Block-block reconnection, and (d) horizontal reinforcement.

The comparison of the horizontal deformations shows that the numerical model achieves good agreement with the experimental results, particularly at the 7th and 5th block layers, where the differences are minimal. However, for the 10th block layer, the numerical deformation is lower than the experimental value, highlighting potential limitations of the model at this block layer (see Table 7.6). Despite this discrepancy, the overall trend of deformation is well-captured, providing a representative approximation of the structural response to vertical loads.

It is important to note that, while the model captures the global compressive response with good accuracy, certain effects observed experimentally — such as the enhanced energy dissipation in BFRP-reinforced panels due to crack-bridging action — are not fully reproduced. Additionally, the assumption of perfect bond for GFRP bars may overestimate their contribution in localized zones. These aspects highlight areas for further refinement of the numerical model to more accurately simulate the post-peak and bond-related behaviour of reinforced earthen masonry.

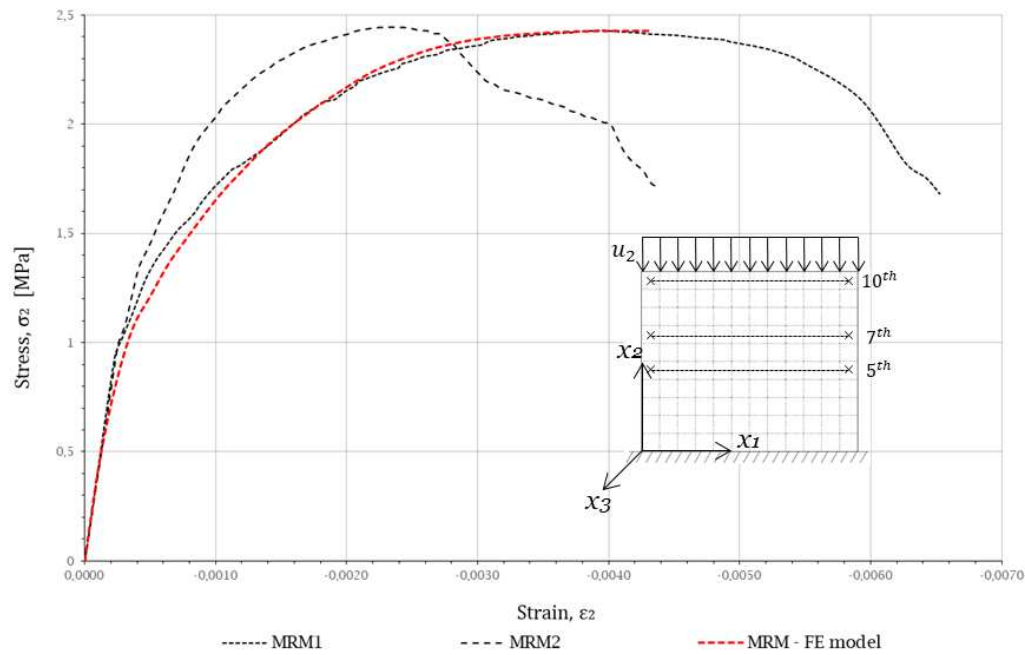


Figure 7.32: Stress-strain plot of numerical simulation of mixed frame reinforced earth masonry panel under compressive loading, plotted against experimental data.

Block layer	MRM	
	Experimental $\langle \varepsilon_1 \rangle$	Numerical ε_1
10 th	0.0027	0.0017
7 th	0.003	0.0027
5 th	0.0021	0.002

Table 7.6: Experimental and numerical comparison of deformation in x_1 direction in MRM panel.

7.2.3 Numerical simulation of shear behaviour

The numerical model can be considered calibrated for all three configurations analysed: the unreinforced masonry (UM), the concrete frame reinforced masonry (CRM), and the mixed reinforced masonry (MRM) which combines BFRP nets and vertical concrete elements. The calibrated models can be used for the simulation of shear behaviour for the three configurations, allowing for a detailed comparison of how each typology reacts to horizontal loads. This approach provides insights into the contribution of the different reinforcement systems to the structural response of CEB masonry panels to shear stresses.

To further analyse the mechanical behaviour of the construction system, an orthotropic material model was used to simulate an in-plane shear test. This step was crucial in validating the numerical approach before calibrating the model for different configurations. The non-linear constitutive laws defined earlier were applied for consistency with the numerical simulations.

Boundary conditions included a fixed base ($u_1 = 0, u_2 = 0, \omega_3 = 0$) for the model and a uniformly applied (F) vertical force to simulate compressive loads from the structure weight or external forces. This force was applied uniformly in the initial phase and maintained constant throughout the analysis (see Figure 7.33). The force was set as $F = 18900\text{ N}$ to simulate the weight of a masonry wall segment corresponding to the height of one storey of a building. At the second step, the displacement (u_1) was applied at the top corner. To constrain the model correctly, a steel support was introduced on the opposite side of the load, with height equal to the first two brick courses. The contact between the bricks and the steel support was modelled as a roller, allowing for vertical displacement but preventing penetration with a frictional contact condition. Coefficient of friction $\mu^{support} = 0.2$ to simulation friction between earth block and steel surface. The support was anchored to the ground and fixed laterally to ensure stability. First the vertical load was applied, followed by the horizontal load simulating the shear behaviour of the system.

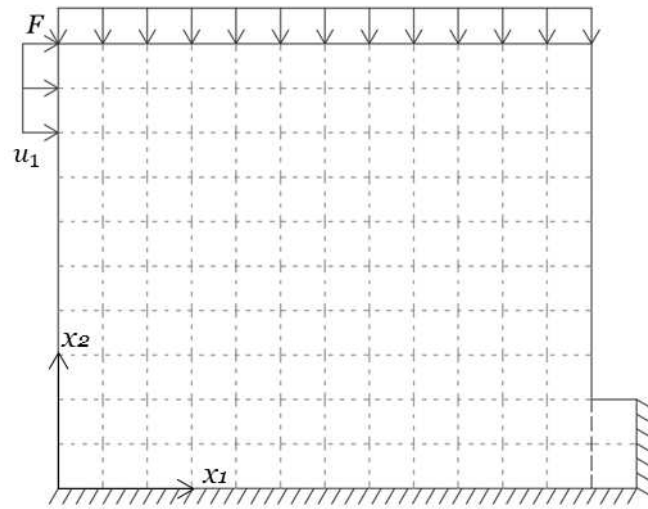


Figure 7.33: Boundary conditions for numerical simulation of shear behaviour.

The numerical simulation revealed distinct responses among the three panel typologies in the linear phase. The UM typology exhibited a shear modulus of $G_{12}^{UM} = 740\text{ MPa}$, while the CRM model experienced a higher shear modulus $G_{12}^{UM} = 900\text{ MPa}$. The UM panel (unreinforced) has the lowest shear strength, as expected, due to the absence of

reinforcement. Despite the increased stiffness provided by the concrete frame, the CRM panel demonstrated limited deformation capacity, with a maximum strain γ_{12}^{CRM} that was half of that observed in the unreinforced configuration.

The MRM shows a shear modulus of $G_{12}^{MRM} = 940 \text{ MPa}$, slightly higher than that of the CRM panel. Unlike the CRM configuration, the MRM panel achieved the highest ultimate shear strength, comparable to the other configurations.

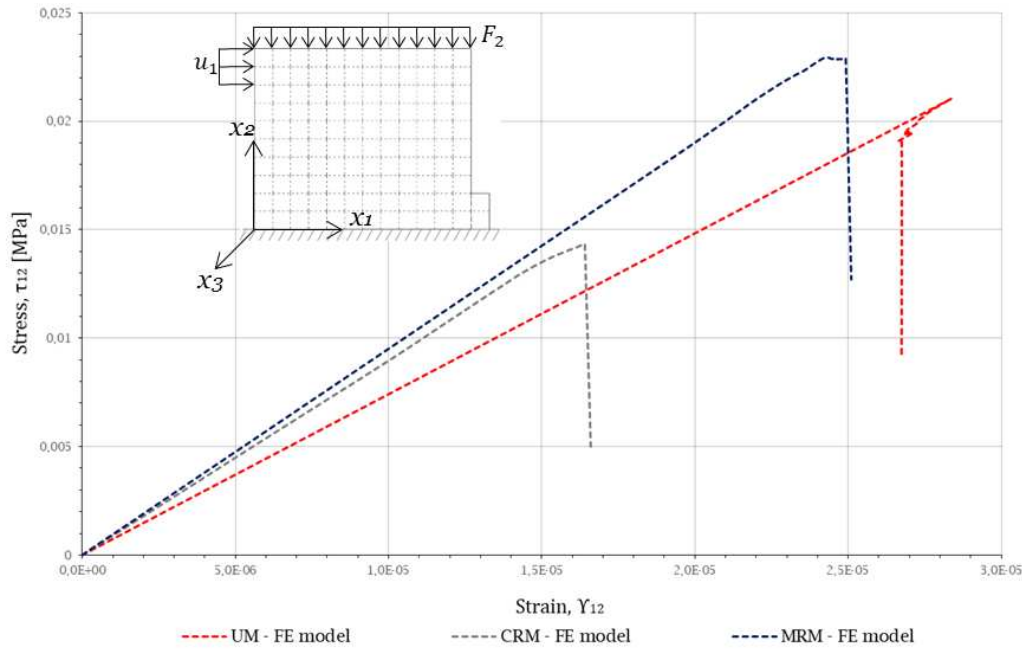


Figure 7.34: CEB masonry panels numerical simulation of shear behaviour, shear stress-strain plots.

Shear strength is consistently a small fraction of compressive strength for all configurations, with ratios ranging from 1/130 (UM) to 1/95 (MRM). The mixed reinforced panels achieved highest improvement in both shear and compressive strength. The shear-to-compressive strength ratio improves, highlighting the effectiveness of the mixed reinforcement strategy in providing a more balanced performance.

The observation of the shear stress of the different panels shows that for the unreinforced configuration UM the distribution evidenced the weakness of the head mortar joints and of the interface within the block, with stress concentrations and a diagonal pattern. Both sliding and rotation of the individual blocks can be observed (see Figure 7.35a).

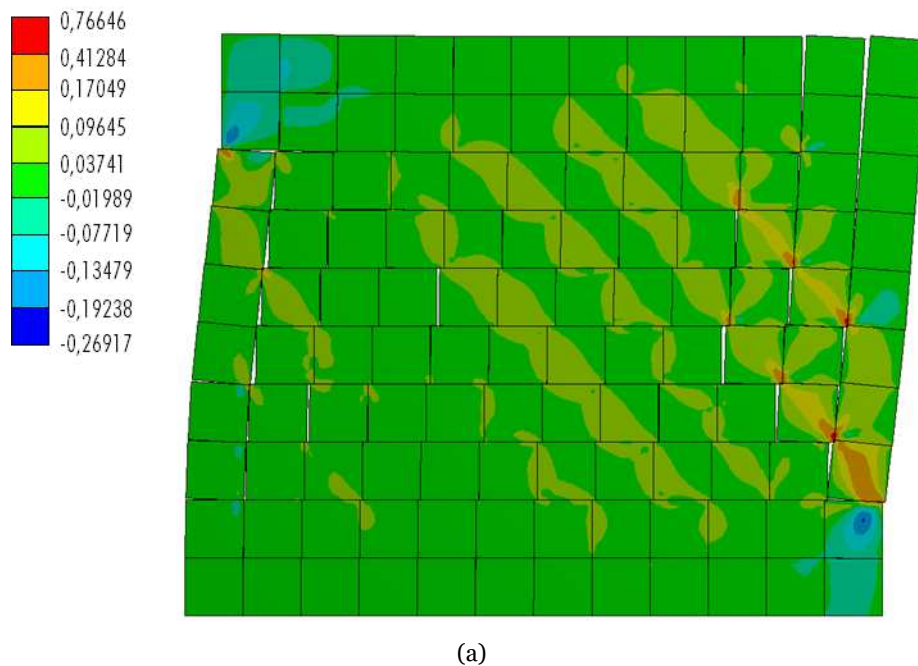
For the CRM panel the distribution of tension τ_{12} shows a different distribution, compared to the unreinforced masonry. The stress concentrates near the corners frame. The reinforcement frame causes the panel to behave not as a single unit but as three

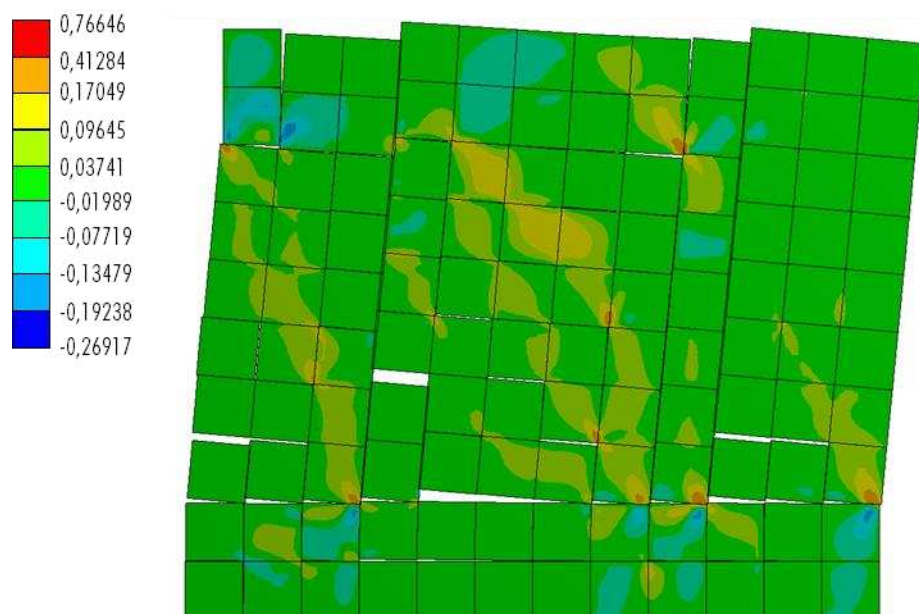
adjacent masonry segments (masonry piers), as evidenced by the deformations and the formation of multiple diagonal line of tensions (see Figure 7.35b).

Panel configuration	Shear strength	Shear modulus
	[N/mm ²]	[N/mm ²]
UM	0.021	740
CRM	0.014	900
MRM	0.023	940

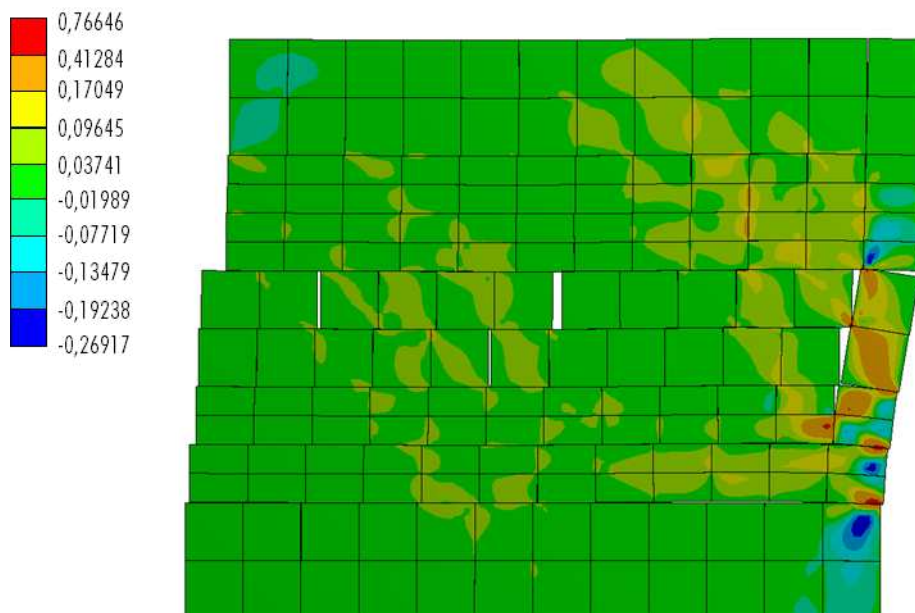
Table 7.7: Shear strength and Shear elastic modulus for FE simulation of earth masonry panels.

The MRM shows a more uniform shear stress distribution across the masonry panel. The adoption of BFRP net as horizontal reinforcement element suggests an enhanced ability to transfer stresses, indicating a good collaboration between the frame and the masonry, and mitigating stress localisation. Deformation is moderate, and rotation of the single elements is not observed within the panel.





(b)



(c)

Figure 7.35: Shear stress distribution of CEB masonry panel for (a) unreinforced configuration at $\gamma_{12}^{max} = 0,000028$, (b) concrete frame reinforced configuration at $\gamma_{12}^{max} = 0,000015$, and (c) mixed frame reinforced configuration at $\gamma_{12}^{max} = 0,000025$.

The shear model was designed with the same dimensions and masonry-to-reinforcement proportion as the FE model in compression calibrated on experimental data. The choice was made to ensure continuity in the numerical approach.

However, this design resulted in an overly reinforced configuration, where the reinforcement was disproportionately small relative to the panel. To address this, an alternative model with adjusted reinforcement was developed.

For this reason, a new model with adjusted frame-to-masonry proportions was developed to prevent results biased by disproportionate frame-to-masonry ratios and to ensure a response not overly dominated by the frame. The panel height was set to represent the height of a single-story masonry pier. In order to avoid slender proportion of the pier, the base was doubled compared to the base dimension of the previous model (see Figure 7.36). The revised panel dimensions were defined accordingly.

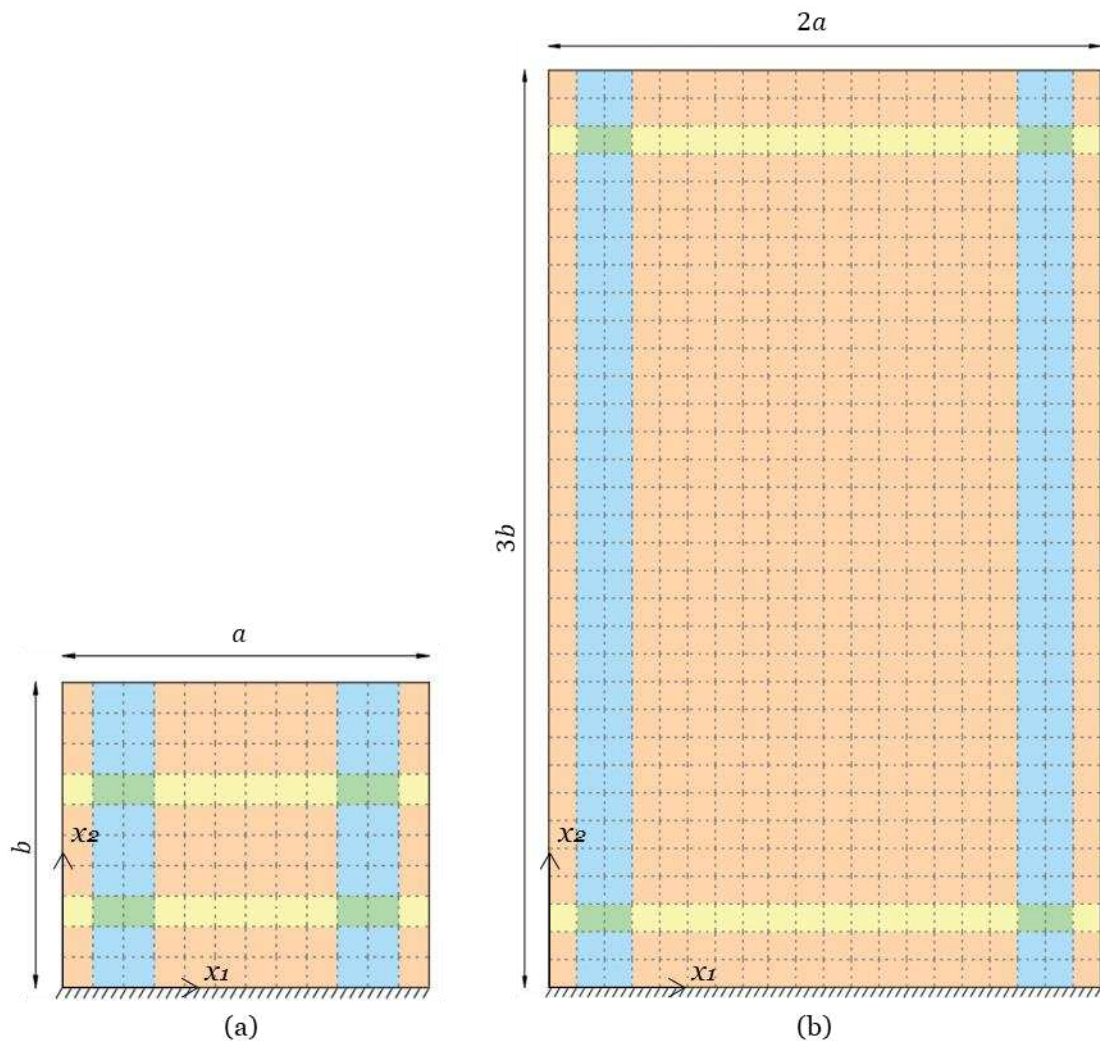


Figure 7.36: Model proportion: (a) Configuration-0, based on the panels test in the experimental campaign, and (b) Configuration-1, masonry-to-frame adjusted proportions.

To simulate the shear behaviour of the Configuration-1 panels, the boundary conditions were left unaltered compared to the previous model (see Figure 7.37).

Since the analysis relied on a multilinear constitutive law derived from experimental compression data of masonry panels and given the significantly reduced shear resistance exhibited by the masonry panels, the elastic range of the unreinforced masonry material was reduced to 10% to enter the nonlinear phase, only for comparison purposes. This approach allows for the isolation of the response of the different frames, enabling a direct comparison of their nonlinear behaviour. The panels were tested with the concrete frame reinforcement (CMR) and mixed frame reinforcement (MRM). A simulation of the unreinforced panel configuration (UM) was run as a reference.

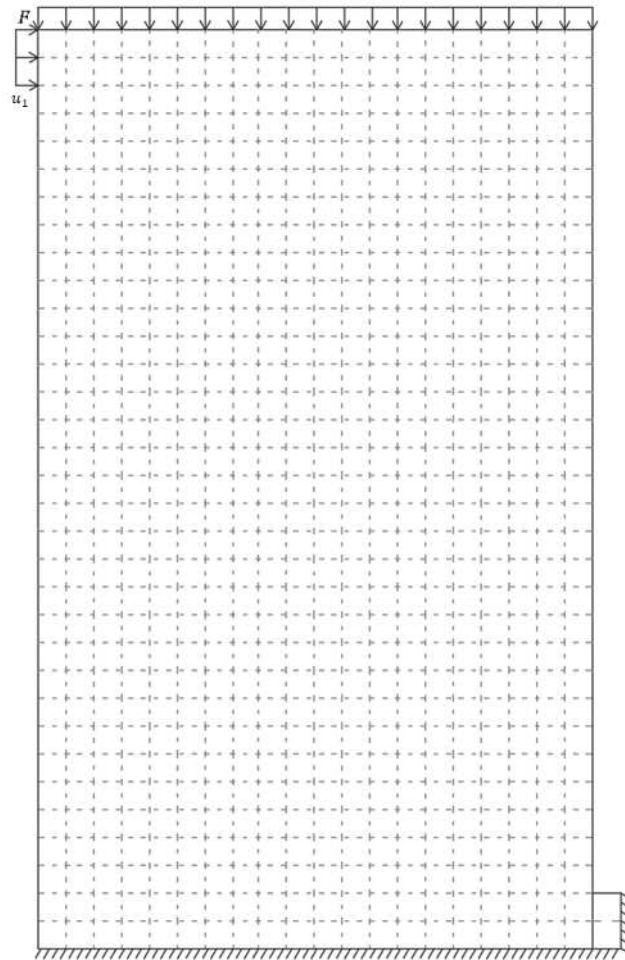


Figure 7.37: Boundary condition for Configuration-1, masonry-to-frame adjusted proportions in shear simulation.

The plot illustrates the shear behaviour of three panel configurations: unreinforced masonry (UM), concrete frame-reinforced masonry (CRM), and mixed reinforcement masonry with a BFRP net (MRM).

The unreinforced configuration exhibits limited shear resistance, as expected for an unreinforced masonry. As expected for unreinforced masonry, there is no post-peak stress recover.

The concrete frame reinforced panel experienced an increase of shear resistance compared to the UM configuration. A distinct drop in stress is registered, possibly due to a failure of the masonry. However, the presence of the reinforcement frame enables to regain shear resistance and deformation capacity (see Figure 7.38).

The MRM panel configuration shows the highest shear resistance among the three configurations. This can be attributed to the presence of the BFRP fibres as horizontal reinforcements. Similar to the CRM typology, a drop in resistance is observed when the masonry begins to fail, as seen in the UM panel (red curve). The presence of the frame significantly enhances the mechanical properties, enabling stress recovery and improved overall performance.

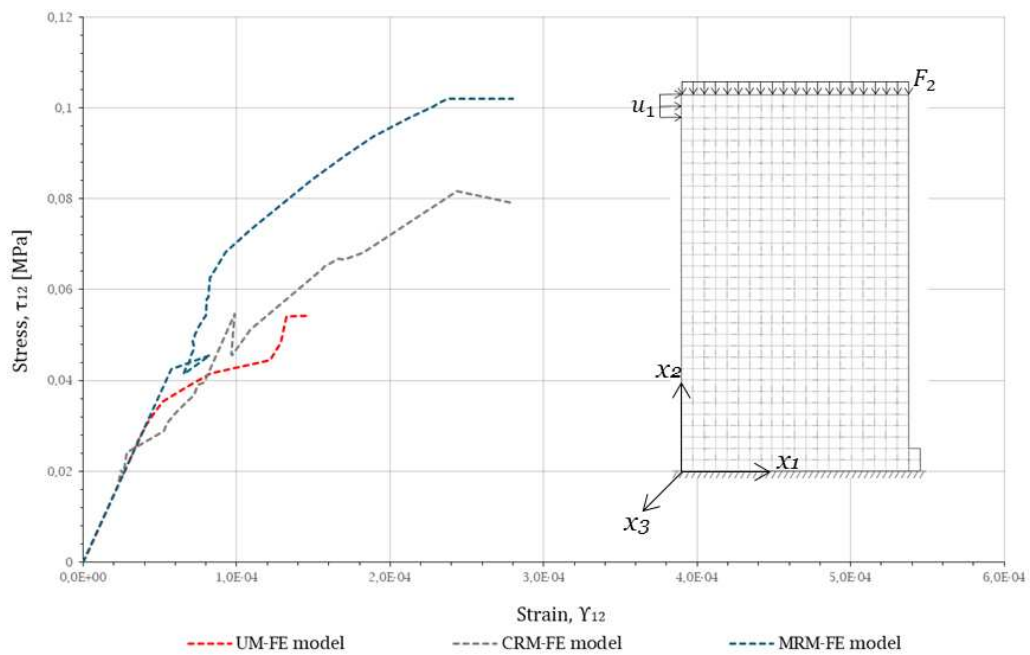


Figure 7.38: Numerical simulation of shear behaviour of Configuration-1 CEB masonry panel, shear stress-strain plots.

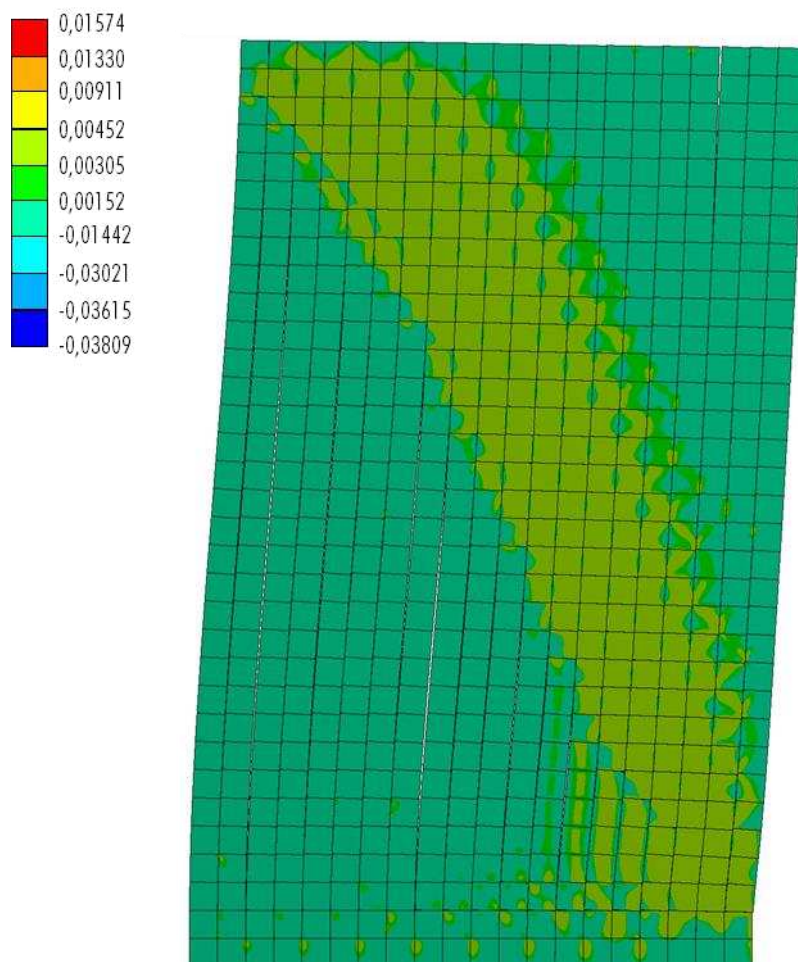
To further investigate the influence of the different reinforced frame typologies for shear behaviour, an analysis of the shear stress distributions of the three panel configurations was performed.

In the unreinforced masonry panel (UM), stress is concentrated along a single, well-defined diagonal crack, indicating a brittle failure mode with limited capacity for load redistribution (see Figure 7.39a).

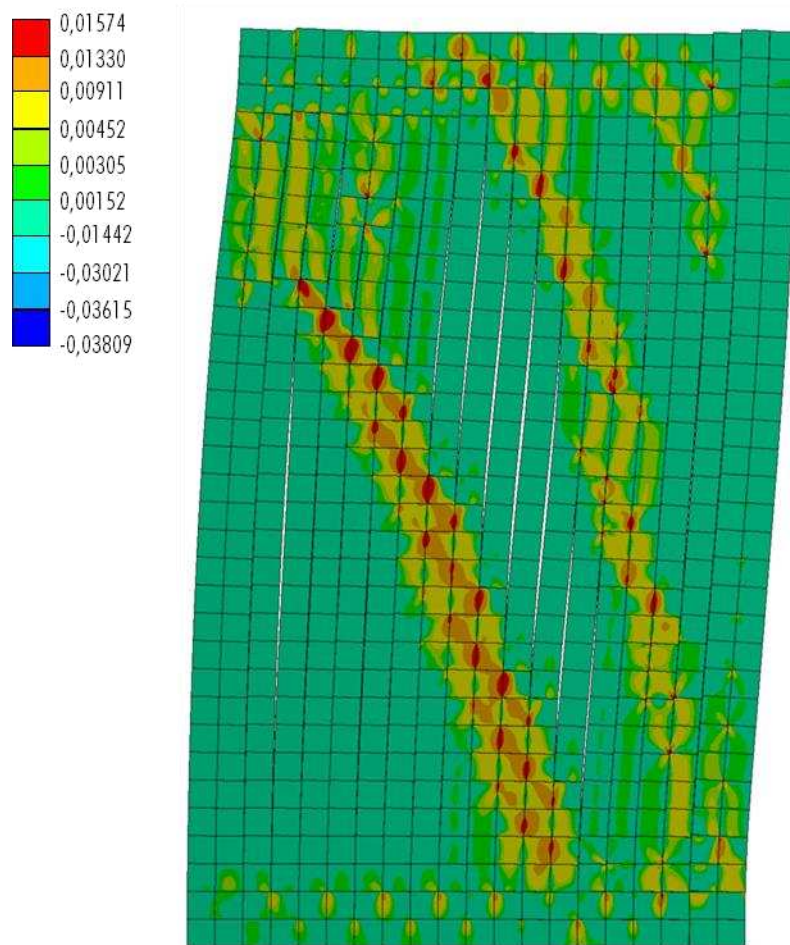
The concrete frame reinforced masonry panel (CRM) shows a more distributed stress pattern, with multiple shear planes forming near the frame, demonstrating enhanced

load-bearing capacity and reduced localized stress concentrations. However, stress remains relatively focused along the primary cracks (see Figure 7.39b).

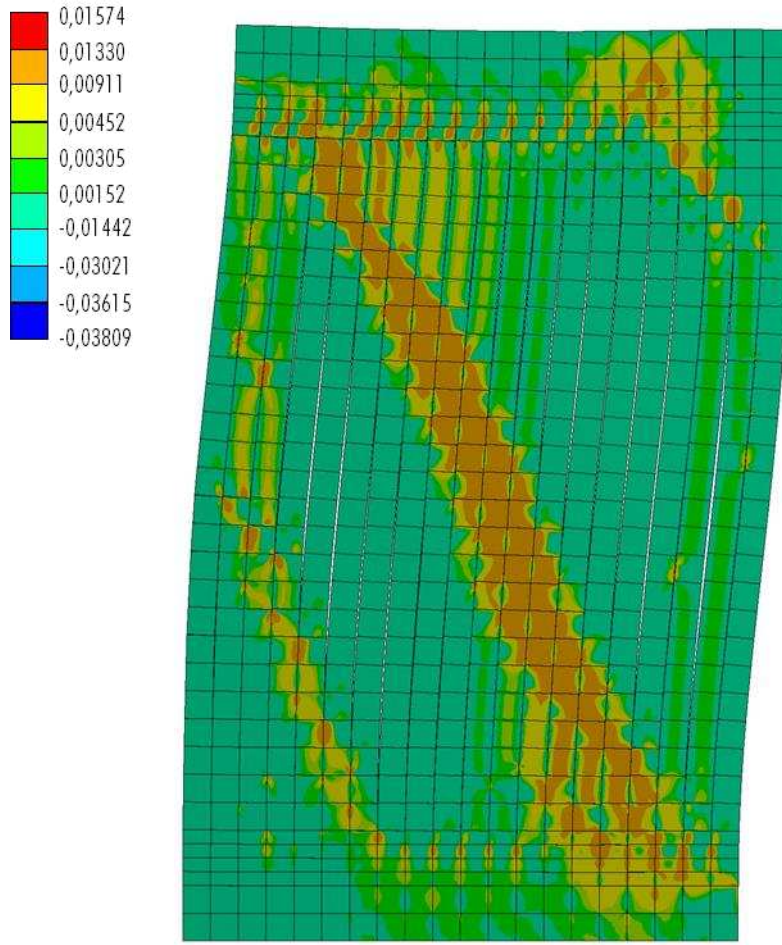
The mixed reinforced masonry panel (MRM), which includes a BFRP net, exhibits the most diffuse stress distribution, with stress spread not only along the diagonal cracks but also across the reinforced horizontal regions. This configuration effectively reduces peak stress concentrations and improves ductility, showcasing the benefits of the combined reinforcement system (see Figure 7.39c). Overall, the CRM and MRM panels mitigate the brittle behaviour of the UM panel, with the MRM configuration achieving superior stress redistribution and shear performance.



(a)



(b)



(c)

Figure 7.39: Shear stress distribution of CEB masonry panel Configuration-1 for (a) unreinforced model at $\gamma_{12}^{max} = 0,00012$, (b) concrete frame reinforced model at $\gamma_{12}^{max} = 0,00025$, and (c) mixed frame reinforced model at $\gamma_{12}^{max} = 0,00026$.

Conclusions

This research investigated the mechanical behaviour of cement-stabilized compressed earth block (CEB) masonry, reinforced with a frame, with the goal of optimizing the construction system. The study was developed within the context of sustainable construction in Senegal and focused on reducing the amount of cement used, improving block typologies, and assessing an alternative reinforcement strategy. The main agenda was to investigate the mechanical properties of the materials to optimize the construction system, minimizing cement content and refining block production. Additionally, a numerical analysis was proposed based on limited mechanical data to further evaluate the structural response of the system.

Earth is not a standardized material, as its composition varies significantly based on soil type, water content, and stabilization processes. To address this variability, an extensive experimental campaign was conducted to extract key mechanical properties of the constituent materials. Given the material specificity, existing regulations on standardised material were referenced and adapted to best suite the earth-based elements. However, while adapting existing standards provides a valid approach, earthen construction would benefit from the development of general regulations tailored to different construction techniques, ensuring a more standardised and widely applicable framework for design and implementation.

The mechanical properties of cement-stabilized CEB and earth-based mortar were evaluated, providing essential data for understanding the material behaviours. The results confirmed the suitability of CEB for structural applications, with an elastic modulus of 6970 MPa and compressive strength of 6 MPa. While the mortar exhibited a significantly lower elastic modulus (214 MPa).

Future research should focus on increasing compaction pressure to minimize cement content in block production. The difference in elastic modulus between blocks and mortar, suggest that the earth-based mortar can deform and dissipate stress effectively within the masonry system. However, incorporating natural fibres could further enhance mortar elasticity, improving overall masonry performance.

Three masonry configurations were analysed: i) Unreinforced Masonry (UM), ii) Concrete-Reinforced Masonry (CRM), iii) Mixed-Reinforced Masonry (MRM) using BFRP fibre nets and vertical concrete elements

Experimental results highlighted that MRM panels demonstrated superior resistance to vertical loads compared to both unreinforced and concrete-reinforced masonry. The CRM system exhibited lowest mechanical performance, suggesting a mechanical incompatibility between the stiff concrete frame and the more deformable earthen masonry. The presence of the concrete beam in CRM likely introduced localized stress concentrations, leading to premature failure. Moreover, an early transition to non-linear behaviour, confirms mechanical incompatibility between concrete frame and the earthen masonry. In contrast, the MRM system provided a more uniform stress distribution, improved ductility, and enhanced crack redistribution, indicating that the fibre net plays a crucial role in the reinforcement mechanism. Furthermore, MRM panels demonstrated a distinct post-peak response, absent in both UM and CRM panels. This suggests that BFRP reinforcement helps maintain structural integrity after peak load, delaying sudden failure and improving the residual load-carrying capacity of the system.

The vertical reinforcement element was then investigated, by replacing traditional steel rebars in the concrete frame with glass fibre-reinforced polymer (GFRP) bars, with a focus on the effect on strength bond of different surface treatments. Bond strength tests demonstrated that sand-coated bars provided the highest bond strength, while grooved bars contributed through enhanced shear resistance. These findings suggest that further optimization of groove depth and spacing should be refined to enhance bond performance and improve mechanical interlocking.

A homogenization approach was employed to model the masonry system, with different representative elementary volumes (REV) used to account for material and geometric variations. Linear analysis confirmed the validity of this modelling strategy. Stress distribution analysis showed that the MRM typology enhances stress transfer between the masonry system and the reinforcement frame elements.

A non-linear analysis was conducted with a limited set of non-linear parameters. A multi-linear constitutive law derived from experimental data was adopted, successfully capturing the non-linear behaviour of the masonry panels up to peak stress.

Shear simulations demonstrated that the CRM frame altered the structural response, creating stress concentrations near the frame corners and causing the panel to behave as separate masonry piers. In contrast, the MRM system exhibited a more diffuse stress distribution, reducing peak stress concentrations and improving ductility, further reinforcing the benefits of the composite reinforcement approach.

While the model accurately captured the global response of the system under vertical loading, some limitations were noted. The energy dissipation and post-peak behaviour

observed experimentally in MRM panels — due to the crack-bridging effect of the BFRP net — were not fully reproduced numerically. Additionally, the model assumed perfect bond for GFRP bars, which may lead to overestimation of their contribution at local interfaces. To improve predictive capability, future numerical development should explore the implementation of softening and damage-based constitutive models to simulate progressive failure, and the refinement of nonlinear interface laws to better represent the interaction between concrete, BFRP, and CEB. Moreover, dynamic simulations could help assess seismic performance, especially in light of the ductility improvements introduced by BFRP reinforcement.

From an experimental standpoint, additional testing should be conducted to address long-term and environmental performance. Cycling load tests would allow the evaluation of fatigue behaviour, while shear tests are essential to complete the mechanical characterisation of the system. Furthermore, tests under varying humidity conditions would provide valuable insight into the vulnerability of cement-stabilised masonry to environmental exposure.

In a broader perspective, a critical comparison with traditional construction methods is essential to assess the scalability and real-world applicability of the proposed system. While cement-stabilised CEB offers low environmental impact and social advantages — particularly in local, low-income contexts — several challenges remain. These include the lack of standardisation and regulatory frameworks, the use of additional reinforcement to meet structural requirements, and material sensitivity to humidity and weathering, which may demand protective treatments. Additionally, although raw materials are low-cost, the use of stabilisation additives and the need for skilled labour may increase the overall cost and complexity of implementation. A cost–benefit analysis, incorporating not only mechanical and economic factors but also social and environmental impact, would be essential to evaluate the true competitiveness of this construction method.

This study contributes to the understanding of cement-stabilized CEB masonry by providing quantitative data on mechanical performance to guide material optimization. It demonstrates that BFRP net reinforcement enhances structural behaviour, reducing reliance on traditional concrete elements. Further, it establishes a homogenization approach for modelling earthen masonry with composite reinforcement, by adopting limited elastic mechanical parameters

Future development should focus on exploring the reduction of cement content while maintaining mechanical performance. Moreover, a dedicated experimental campaign should be conducted to evaluate shear response and determine key non-linear

parameters. Additionally, the geometric proportion between the masonry and the reinforcement frame should be investigated to optimize the construction system and enhance its overall mechanical response. Special attention should also be given to eliminating the concrete matrix in favour of more sustainable materials

The results of this study reinforce the potential of cement-stabilized CEB masonry as a sustainable and structurally viable construction system. The proposed BFRP reinforcement method offers an effective alternative to traditional concrete beams, reducing material use while improving mechanical performance. Future research should continue optimizing material composition, reinforcement strategies, and numerical modelling techniques to further advance the understanding and application of earthen masonry systems in structural engineering.

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