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Analytical and Experimental Investigation
of Digitally Fabricated Concrete Structures:
A Design-Oriented Framework and Case
Study Validation

by

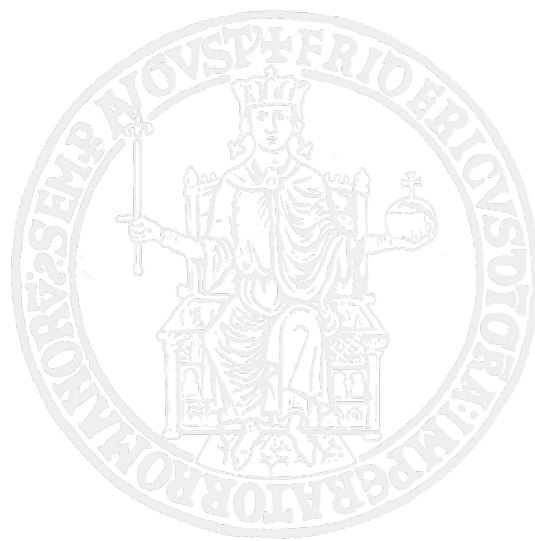
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for the fulfillment of the Degree of Doctor of Philosophy
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Napoli, 05/11/2025

GIULIVO MARCO

Abstract

This doctoral work develops and validates an innovative analytical and design framework for the structural use of digitally fabricated concrete wall structures, with particular focus on 3D-printed concrete (3DPC) fabrication additive manufacturing technology.

The main objective is to provide a mechanical and structural interpretation of the behaviour of these novel printed elements, by analysing the close relationship between digital fabrication process parameters—such as interlayer time (ILT), printing path geometry, and deposition strategy—and the resulting mechanical response across multiple scales of investigation: material, intermediate, and structural.

To this end, an extensive multi-scale experimental campaign was carried out to characterise the key compressive, tensile, and shear strength parameters, following RILEM recommendations and drawing analogies with masonry testing standards. At the structural scale, diagonal compression and in-plane cyclic tests on 3D-printed walls were performed to investigate their seismic behaviour. The results showed that, when tensile strength is neglected, the walls exhibit a behaviour comparable to that of traditional masonry; however, when tensile resistance is considered at the Ultimate Limit State (ULS), the in-plane shear strength can reach values up to three times higher than those predicted by conventional masonry formulations.

These findings motivated the development of in-plane resistance domains specifically tailored for 3DPC walls, highlighting the differences between analyses that include or exclude tensile strength, and comparing the outcomes with reference masonry typologies available in the literature. Since tensile resistance is strongly affected by printing parameters, a parametric analysis was conducted by varying the interlayer time (ILT) while keeping the main geometric and mechanical properties constant. This study enabled the creation of preliminary design framework, which will be further validated through future large-scale experimental campaigns.

In parallel, the experimental and analytical results were employed for the numerical calibration of a simplified finite element (FEM) model implemented in SAP2000, based on a linear interpretation of the structural response. This model allows for a rapid and code-consistent structural assessment, bridging advanced experimental research with traditional engineering design tools and promoting the practical adoption of 3DPC in real structural applications.

The proposed methodology was finally applied to the Montalto di Castro Control Room, the first structural 3D-printed concrete building constructed in a seismic zone, verified according to the Italian Building Code (NTC 2018). In this application, the wall was analysed following a conservative design approach, assuming brittle behaviour and neglecting tensile strength to ensure structural safety within the current regulatory framework.

Overall, this PhD work demonstrates that 3D-printed concrete—despite its intrinsic anisotropy, interlayer bonding mechanisms, and process-dependent geometric features—can be effectively interpreted and governed through the principles of classical structural mechanics. By integrating experimental investigation, numerical modelling, and code-based verification, the structural behaviour of 3DPC walls can be described through a set of main parameters: layer orientation, tensile and shear interface properties, wall slenderness, void ratio, and printing path geometry.

Keywords: Digital Fabrication, 3D-Printed Concrete (3DPC), In-plane Behaviour and Resistance Domains, Analytical and Numerical Modelling (Timoshenko / SAP2000), Design Framework and Code Integration.

Sintesi in lingua italiana

Il presente lavoro di dottorato sviluppa e valida un framework di analisi e progetto innovativo per la valutazione del comportamento strutturale di strutture in calcestruzzo fabbricati digitalmente, con particolare attenzione alle pareti in calcestruzzo stampato in 3D (3DPC).

L'obiettivo principale è fornire una chiave di lettura meccanica e strutturale del comportamento di questi nuovi materiali stampati, analizzando la stretta connessione tra i parametri di processo della fabbricazione digitale — quali il tempo interstrato (Interlayer Time, ILT), la geometria del percorso di stampa e la strategia di deposizione — e la conseguente risposta meccanica alle diverse scale di indagine: materiale, intermedia e strutturale.

A tal fine, è stata condotta una vasta campagna sperimentale multi-scala, finalizzata a caratterizzare i principali parametri di resistenza a compressione, trazione e taglio, seguendo le linee guida RILEM e traendo analogie con gli standard della muratura. Alla scala strutturale, prove cicliche nel piano su pareti stampate in 3D hanno permesso di analizzare il comportamento sismico di tali elementi. I risultati hanno mostrato che, se la resistenza a trazione viene trascurata, le pareti manifestano un comportamento analogo a quello delle murature tradizionali; tuttavia, considerando la trazione allo Stato Limite Ultimo (SLU), la resistenza a taglio nel piano può risultare fino a tre volte superiore rispetto a quanto previsto dalle formulazioni analitiche della muratura.

Queste evidenze hanno portato allo sviluppo di domini di resistenza nel piano specifici per le pareti in 3DPC, evidenziando le differenze tra le analisi che includono o escludono la trazione e confrontandole con le curve di riferimento murarie presenti in letteratura. Poiché la resistenza a trazione risulta fortemente influenzata dai parametri di stampa, è stata condotta un'analisi parametrica variando il tempo interstrato (ILT) e mantenendo costanti le principali proprietà geometriche e meccaniche.

Tale analisi ha permesso di elaborare abachi di progetto preliminari, da validare attraverso future campagne sperimentali su scala reale.

Parallelamente, i risultati sperimentali e analitici sono stati impiegati per la calibrazione numerica di un modello semplificato agli elementi finiti (FEM) in SAP2000, basato su un'interpretazione lineare del comportamento strutturale. Tale modello consente una valutazione progettuale rapida e coerente con gli strumenti di calcolo tradizionali, aprendo la strada a un uso ingegneristico diffuso del 3DPC nelle analisi di edifici reali.

Il metodo sviluppato è stato infine applicato al caso studio della Control Room di Montalto di Castro, il primo edificio strutturale in calcestruzzo stampato in 3D realizzato in zona sismica, verificato secondo le NTC 2018. In tale applicazione, la parete è stata analizzata con un approccio conservativo, assumendo un comportamento fragile e trascurando la trazione, al fine di garantire la sicurezza strutturale nel quadro normativo vigente.

Nel complesso, la ricerca dimostra come la stampa 3D del calcestruzzo, inserita nel più ampio ambito della fabbricazione digitale, possa essere interpretata e governata attraverso gli strumenti della meccanica strutturale classica, integrando sperimentazione, modellazione numerica e verifica normativa. Il lavoro getta così le basi per la definizione di nuovi standard di progetto e criteri prestazionali dedicati alle strutture fabbricate digitalmente, in grado di coniugare innovazione tecnologica e affidabilità strutturale.

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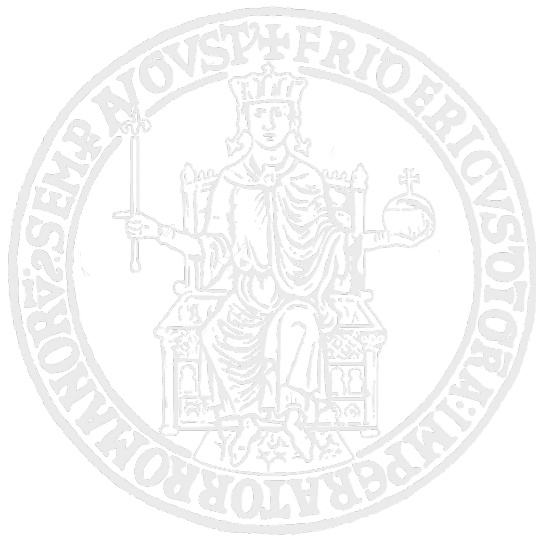
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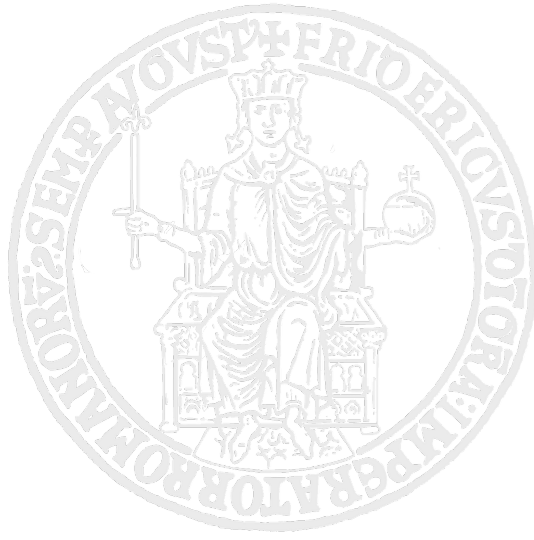
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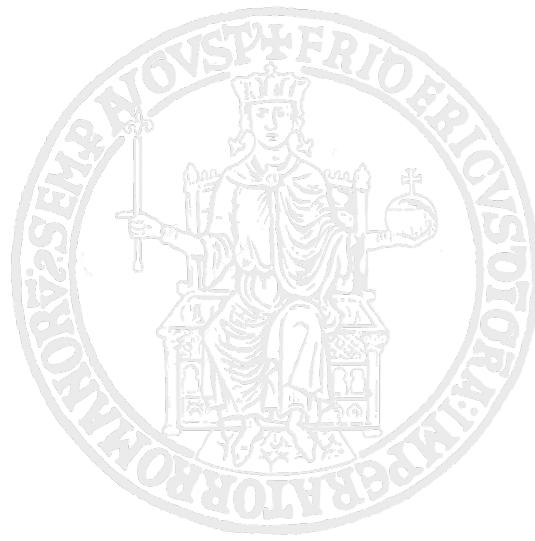


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1 Introduction

Additive manufacturing has emerged as a credible pathway to automate construction, reduce formwork and waste, and unlock geometric freedom for both non-structural and structural elements. Within this landscape, extrusion-based 3D-printed concrete (3DPC) plays a leading role; however, its responsible structural use still depends on understanding how process parameters affect the hardened material properties and on developing design-oriented frameworks. The research roadmap by Buswell and co-authors [1] crystallized the process–structure–property chain linking fresh rheology (buildability, thixotropy, open time) to the anisotropic response of hardened elements, and called for test methods that capture layered interfaces rather than treating printed concrete as simply a quasi-2D cast material. Parallel efforts by RILEM TC 276-DFC consolidated nomenclature, process classification and state-of-the-art testing— establishing the basis for the future standardization of fresh and hardened properties specifically for digital fabrication with concrete [1, 2]. A core scientific challenge is the *process–performance trade-off* introduced by interlayer time (ILT): short ILT can benefit interfacial bonding through chemical and mechanical adhesion, whereas longer ILT often improves green stability and stack height but degrades interlayer bond due to surface drying and early hydration at the substrate. Experiments across different mixes and printing setups

show that increasing ILT (or exposing filaments to drying) reduces interlayer tensile/bond strength and can *switch* the failure plane from the bulk matrix to the interface—thereby altering peak strengths, stiffness and failure mode at the element scale. These trends have been reported consistently, from foundational interlayer-adhesion studies to more recent parametric campaigns and comprehensive assessments of operational/material factors.[3,4,5,6,7]. Beyond ILT, hardened 3DPC exhibits *anisotropy* arising from filament orientation, cold joints and pore alignment. Dedicated investigations on printed beams and panels report direction-dependent stiffness and tensile capacity [8], and highlight the deleterious effect of cold joints on flexural response and crack localization [9]—evidence that motivates interface-explicit constitutive descriptions instead of treating the material as isotropic. Recent studies further synthesize how printing direction, pore morphology and flowability shape anisotropic properties, reinforcing the need for protocols that control (and report) process descriptors in both material and structural tests. [10,11,12].

In recent years, extrusion-based 3D printing with concrete has progressed from material-scale demonstrations to the construction of full-scale structural systems, including low-rise residential buildings and prototype houses [1, 13]. These structures are typically composed of 3D-printed wall elements, which act as both load-bearing and enclosing components, thus defining the global stability of the building. Understanding the structural response of such printed walls is therefore essential for their safe and efficient design. Recent experimental campaigns on 3D-printed panels and wall systems [14]

have shown that the governing in-plane mechanisms depend on geometry and loading: slender walls tend to exhibit flexural–compression failure, whereas stockier panels are subjected to shear-dominated responses (diagonal cracking or sliding). These studies also emphasize the key role of interlayer interfaces, whose activation can govern the overall deformation and strength of the wall, particularly under cyclic loading [14]. In parallel, the international community has been developing shared procedures and performance criteria for printable materials and printed components. In particular, The RILEM state-of-the-art report *TC 276-DFC* [15] and the subsequent initiative *TC 303-PFC* [16] provide harmonized testing protocols and performance requirements tailored to digital concrete fabrication, paving the way for future design recommendations.

1.1 Context and Motivations

The recent technological innovations brought to the contemporary construction sector have revealed the potential of the combination of additive manufacturing (AM) and concrete as a transformative frontier able to redefine traditional building fabrication methodologies [17]. Among the different additive AM techniques using concrete that have seen application across demonstrator projects [18,19], 3D Concrete Printing (3DCP) has attracted significant interest and prevalence not only for the well-established understanding of the physical and mechanical characteristics of the printable materials achieved so far

[20] but also for the flexibility of the equipment utilized (e.g., gantry, robotic arm etc. [18]). This technique facilitates the automatic creation of both non-structural and structural elements having various functions within the construction application possibilities, thus envisioning sustainability benefits based on different methodological assessment approaches [21, 22, 23, 24]. However, the large-scale implementation demands rigorous attention due to the distinct mechanical peculiarities that emerge from layer-by-layer fabrication, including elastic anisotropy, weak interlayer interfaces [25], different elastic properties and compressive strengths in the parallel and perpendicular direction of the layer deposition [26,27], dependency of the constitutive compressive behaviour on printing path [28]. This circumstance poses the issue on how take into account these peculiarities in the structural design [29], showing key differences in comparison with homogeneous isotropic mechanical behaviours (such as traditional concrete) or heterogeneous behaviours typical of masonry (having horizontal and vertical joints as well as lower mechanical properties at the material scale).

Despite the extensive characterization of printed materials as a function of different technological conditions [28,30], real-scale testing remains paramount to ascertain how the 3DCP fabricated elements compare to their traditional counterparts (e.g., masonry panels, reinforced and/or unreinforced concrete panels). Moreover, in most of available cases, large scale 3D printed elements are relegated to serve as permanent formwork with printed concrete not used into the calculations [31], thereby imposing limitations and raising questions

on how to design or exploit the benefits of such elements. Despite its relevance, this type of testing presents practical challenges - such as load application and boundary conditions - and is further complicated by the variability of 3DCP systems. Literature on load-bearing unreinforced walls remains limited, particularly regarding the influence of internal geometry and print quality on failure modes and structural modelling.

Axial compression is a key focus in 3DCP wall research, particularly for gravity load resistance. Studies have explored how printing-induced defects and internal geometries affect compressive strength. Aramburu et al. [32] showed that printing significantly alters mechanical properties, requiring adjusted design models. Wang et al. [33] pointed out the influence of hollow geometries, with truss-like forms showing improved performance. However, accurately predicting the ultimate compressive capacity of 3DCP walls remains a challenge, underscoring the need for refined models tailored to printed materials.

Axial compression tests alone are not sufficient to assess structural panels subjected to lateral actions, particularly in seismic conditions. In these cases, it is essential to understand failure modes involving shear, sliding, and flexural-axial interaction, which reflect the combined demands on columns and load-bearing walls. For 3DCP panels, this is further complicated by orthotropic behaviour and interlayer weaknesses, requiring more attention on large-scale investigations beyond small-scale tests. So far, numerical studies on this topic are dominant in comparison to real scale experimental

works. In [34], the potential of strain-hardening concrete to enhance the strength and seismic performance of 3D-printed lattice walls was examined. A numerical model was developed to simulate the behaviour of 3DCP walls under quasi-static cyclic in-plane loading, accurately incorporating stiffness coefficients and interlayer properties derived from experimentation. The findings highlighted how composite micro-modelling, using the cohesive zone model at the interfaces and concrete damage plasticity model, effectively captures load-bearing mechanisms and the pinching effect in hysteretic curves. The use of FEA was also useful in the case of shear wall containing steel reinforcement [35] for seismic force-resisting systems in low-rise buildings. Strength capacity equations for in-plane failure mechanisms were formulated, validated through finite element analysis (FEA) and compared with concrete block masonry (CBM) walls, finding several similarities between the two different typologies. At the design level, a recent work by Sharma et al. [36] also contributed to this field by proposing a holistic structural design methodology for low-rise 3D-printed concrete buildings, including analytical models and finite element validation under gravity and wind loads, which can serve as a reference framework for extending design procedures to seismic applications.

From the analysis above, it appears that a more holistic testing strategy - complemented by numerical and analytical models - is essential, as 3DCP introduces distinct failure mechanisms that traditional methods do not fully capture. Understanding these behaviours, especially in seismic scenarios, is crucial to developing

advanced testing protocols that ensure structural safety, reliability, and resilience. Indeed, despite promising advances in numerical modelling and small-scale testing, the current state of the art lacks consistent experimental data of full-scale unreinforced 3D-printed walls subjected to seismic-type cyclic loads. Furthermore, current design approaches lack standardization and often fail to capture the unique failure mechanisms introduced by layered fabrication. To address these gaps, this study presents a real-scale mechanical characterization of unreinforced partially hollow (with 40% voids) 3D-printed walls, providing insights into their structural behaviour under seismic loads and considering the influence of base constraint typology. A novel contribution of this study is the direct comparison between a prefabricated anchorage system - embedded during the printing process - and a conventional cast-in-place solution with protruding bars. This comparison explores the geometric adaptability of 3D printing, which potentially allows for the integration of complex anchorage details without additional formwork. The analysis aims to evaluate how these two approaches affect structural performance under seismic loads, while also addressing implications for construction efficiency and design flexibility in 3D-printed systems. Finally, the results are used to define a design-oriented analytical framework that links experimental evidence, analytical and numerical modelling, and structural check procedures within current standard codes (e.g., NTC 2018, Eurocode 8). This framework is implemented in a numerical model and demonstrated through a real case study—

the Control Room of the Montalto di Castro substation—thus providing an end-to-end pathway from laboratory characterisation to design validation within the Italian regulatory context.

1.2 Research Objectives

The overarching goal of this work is to develop and validate an integrated analytical framework for the design and verification of 3D-printed concrete wall elements, consistent with the current regulatory context. The thesis begins by identifying and measuring the material quantities that are truly useful for design—covering fresh-state, hardened-state properties,—so that robust, reproducible parameters can be established. It then moves to the element scale through a full-scale experimental campaign under in-plane cyclic loading, designed to observe stiffness evolution, damage development, and the governing failure modes. A central aim is to formulate resistance domains for 3D-printed wall panels, grounded in experimental evidence and expressed through three principal mechanisms: flexural-compression, diagonal cracking, and shear-sliding. By constructing analytical domains and validating them against tests, the thesis establishes a design-oriented tool to capture the interplay between geometry, axial load, tensile contribution, and interlayer effects such as ILT. Building on these results, the thesis proposes a concise analytical framework that summarises the in-plane response in terms suitable for design, capturing the influence of process-dependent features together with essential geometric and loading effects. The outcomes are translated into verification formats consistent with NTC

2018 and Eurocode provisions, adopting conservative criteria wherever fragility and interlayer discontinuities are significant. To ensure both interpretability and applicability, the analytical framework is implemented in a simplified finite-element environment, where Timoshenko beam formulations in SAP2000 replicate the stiffness and strength of printed panels with reduced computational demand. These models enable direct validation against experimental results and analytical predictions, while also providing a practical tool for design. Where interface parameters are required, the methodology leverages the modified slant-shear procedure to derive cohesion–friction envelopes suitable for design use [3]. Finally, the thesis demonstrates the complete workflow through a real case study—the Control Room at the Montalto di Castro electrical substation—where 3DPC structural walls are assessed within the Italian regulatory framework. This case validates the proposed methodology from material characterisation to structural checks, and highlights both the opportunities and the challenges associated with the adoption of 3DPC in practice. A schematic summary of the overall research framework is presented in Figure 1.1, which illustrates how the work connects material characterisation, element-scale experiments, analytical formulations, and numerical modelling into a unified framework consistent with the regulatory context.

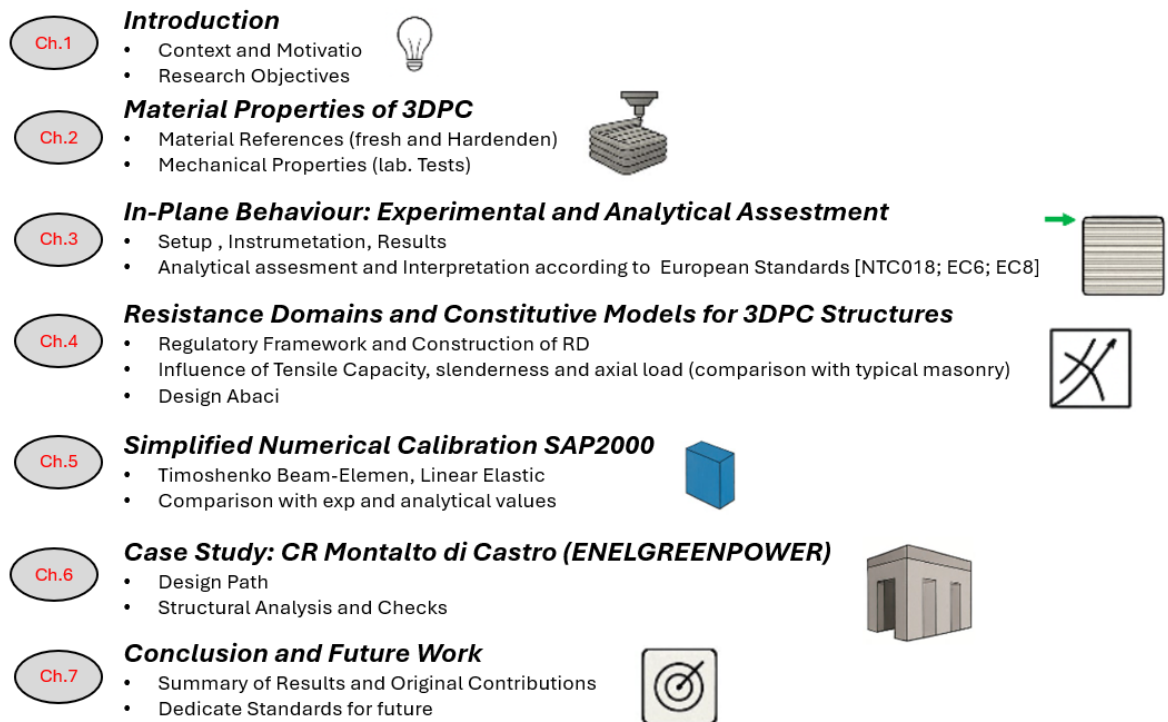


Figure 1.1. Thesis roadmap.

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2 Mechanical Properties of 3d Printed Concrete

3DPC represents a key expression of digital fabrication, replacing formwork with layer-by-layer extrusion directly driven by the digital model. The process, however, places the material at the centre, linking design and construction through controlled deposition and automation. A “printable” mortar must flow reliably through hoses and nozzle while, immediately after deposition, developing sufficient green strength to support its own weight and the weight of subsequent layers. Achieving this balance requires a carefully tuned rheological window. Yield stress and viscosity must be set so that the material shears and extrudes in a stable manner, yet rebuilds structure quickly at rest. Thixotropic behaviour is typically exploited to reconcile these needs, while admixtures—high-range water reducers, viscosity modifiers, accelerators—and a well-graded fine aggregate skeleton shape the fresh response. The kinetics of hydration must also be managed so that the material does not set too fast within the line, nor remain too soft on the substrate. In practice, the printable window is defined not only by composition but by process variables: nozzle speed and stand-off, volumetric flow, interlayer time, ambient temperature and humidity. The interplay between mix design and process control governs the quality of each deposited layer, the accuracy of the toolpath, and ultimately the repeatability of the printed geometry. [1,2]. Layered deposition creates a stratified microstructure. Each pass of the nozzle

leaves behind an interface that may behave as a plane of enhanced or reduced cohesion depending on the timing, surface condition, and local pressure during contact. When interlayer time is short and surfaces remain receptive, bonding tends to be strong; as time grows or surfaces dry, the interface can weaken and act as a preferred path for crack initiation and slip. This reality introduces anisotropy at the scale relevant to structural behaviour: properties that involve tension and shear may differ along and across the layers, and their values can show sensitivity to print orientation and path. The structural consequence is that load transfer within a printed wall depends not only on the “bulk” properties of the mortar but also on the quality of the interfaces generated by the process. A faithful material description therefore needs to acknowledge both contributions [3,4,5].

The next sections move from the fresh state to the hardened state and then to laboratory testing. The discussion of mixes and rheology clarifies how extrudability and buildability are achieved and how process variables shape the material response. The treatment of hardened behaviour outlines the mechanical properties that are most relevant to structural use and explains why interface quality matters. Finally, the overview of laboratory procedures and reference standards frames the experimental evidence on which the subsequent analytical, numerical, and design considerations will rely. This chapter presents the material framework underpinning the thesis. Section §2.1 introduces mixes and fresh-state behaviour of 3D-printed concrete (3DPC), outlining the types of printable binders and admixtures, the rheological descriptors that govern extrudability and buildability, and

the influence of process variables—such as print orientation and interlayer time—reported in the literature [1]. The specific printable mortar adopted in this work is briefly introduced in the same section so that fresh-state parameters and process conditions can be read together. Section §2.2 turns to the hardened state. It reviews, from the literature, the principal mechanical properties relevant to structural use—compression, tension, flexure and shear—and clarifies the role of printing-induced anisotropy. For later modelling purposes, the elastic behaviour is framed within an orthotropic description, and its stiffness matrix is recalled at a conceptual level. Particular attention is paid to the layer-to-layer interface, discussing cohesion, friction and their sensitivity to process and curing. In parallel, the material actually used in this research is described in its hardened state, so that the measured properties can be directly connected to the subsequent analytical and numerical work [3, 4, 1]. Finally, §2.3 summarises the standard laboratory tests typically adopted for hardened 3D printed concrete according to recognised UNI EN and RILEM procedures, including pull-off and durability-related assessments. Together, these sections establish the dataset—and its expected variability—that will be used in the experimental campaign of Chapter 3, in the development of the analytical framework and resistance domains in Chapter 4, and in the verification procedures adopted throughout the workflow [1].

2.1 3DPC Material References

The characterisation of 3D printed concretes combines traditional standards for cementitious materials with adaptations proposed by RILEM TC 304-ADC [13]. In particular, for, Compression Tests: UNI EN 12390-3 [6] prescribes compression tests on cubes and cylinders. For 3DPC, RILEM recommends sawing or coring specimens directly from printed elements in orientations u(I), v(II), w(III). Flexural Strength: UNI EN 196-1 [7] defines flexural tests on 40×40×160 mm prisms. For 3DPC, the same geometry is used, but extracted from printed filaments (single-path coupons). Tensile Strength: EN 12390-6 [6] defines splitting tensile tests, but RILEM emphasises direct tensile and pull-off tests across interlayers as more representative of weak directions. Shear Strength: ASTM C1531 [8] (triplet test for masonry) and CNR-DT 200/2013 (shear in masonry joints) provide analogies. RILEM developed specific protocols for direct shear and shear-compression on printed coupons and panels. Fresh State: EN 1015 [9] (bulk density, consistency, workability time) and EN 12350 [10] (slump flow) are used, supplemented by rheometry. RILEM [13] adds buildability and pumpability tests. Thus, while UNI/EN/ASTM standards provide the baseline, RILEM has created a dedicated framework for 3DPC, defining specimen extraction, orientations, and interlayer conditions.

2.2 Fresh-state behaviour

The performance of 3D printed concrete (3DPC) in the fresh state is a decisive factor for the success of the printing process. Unlike conventional concretes, 3DPC mixtures must meet requirements not only in terms of workability and consistency, but also in terms of printability and buildability, ensuring that the material can be extruded in continuous filaments and retain its geometry without collapse. The key fresh-state properties include:

- **Pumpability and Extrudability:** The material must be pumpable through hoses and nozzles without segregation or blockage, and must produce filaments with stable geometry upon extrusion. Roussel (2018) defines pumpability as a balance between yield stress and plastic viscosity, ensuring continuous flow under applied pressure [11].
- **Buildability:** Each printed layer must support subsequent layers without excessive deformation. This requires a structural build-up of yield stress over time, driven by thixotropy and early hydration. Wolfs et al. (2019) showed that insufficient buildability leads to buckling or collapse of the structure, while excessive thixotropy compromises interlayer bonding [12].
- **Open Time and Interlayer Bonding:** The “interlayer time” (ILT), i.e., the interval between successive passes of the printhead, directly governs the bonding between filaments. Short ILTs favour chemical adhesion (wet-on-wet deposition), while prolonged ILTs produce weak “cold joints.” RILEM TC 304-ADC

recommends explicitly recording ILT in all experimental programmes, since it is one of the main variables affecting strength anisotropy [13].

Standard test methods for the fresh state include slump flow (EN 12350-8 [14]), rheometry tests for yield stress and viscosity (ASTM C1749 [15], EN 12350-8 adapted[14]), and buildability tests where successive layers are deposited until failure. These tests, adapted from rheology and shotcrete practice, provide quantitative parameters (yield stress, viscosity, structural build-up rate) that are now considered essential for printability evaluation [13]. Table 2.1 summarizes the main fresh properties of 3D printed mortar, highlighting the relevant tests to be performed, the parameters considered, the reference standards, the typical units of measurement, the expected outputs, and some additional notes.

Category	Test / Method	Parameter(s) (symbol)	Standard / Reference	Typical Output / Units	Notes for 3DPC
Workability / Consistency	Slump-flow (flow table)	Spread diameter	EN 12350-8	mm	Often adapted for mortar-like printable mixes
Fresh density	Bulk density of fresh mortar	Fresh density (ρ)	EN 1015-6	kg/m ³	QC metric; relates to mix window and stability
Rheology	Rotational rheometry (vane / concentric cylinder)	Yield stress (τ_0); Plastic Viscosity (μ_p)	ASTM C1749; RILEM practice	Pa; Pa·s	Key descriptors for pumpability & extrudability
Buildability	Layer stacking (height-to-collapse; rate of build-up)	Structural build-up rate	RILEM TC recommendations	Pa/min (or Pa/s)	Correlates with early-age stability of printed layers

Pumpability / Extrudability (QC)	Pressure–flow monitoring; nozzle sensing; bead metrology	Pressure–flow (Δp –Q); bead width/height	RILEM TC recommendations	kPa vs L/min; mm	Tracks process window and toolpath fidelity
Process timing	Open time; Interlayer time	ILT	RILEM TC recommendations	min	Record per print segment; strongly affects bonding

Table 2.1.: Fresh State Properties.

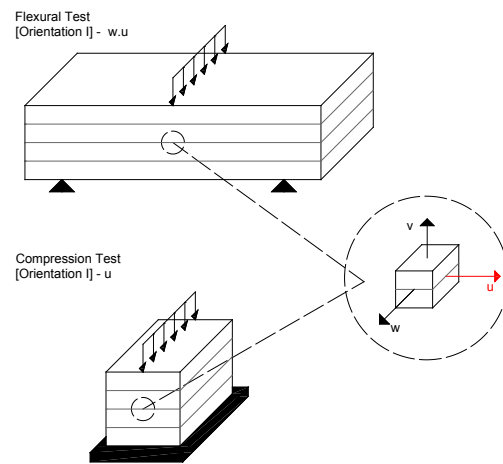
2.3 Hardened State behaviour

Once hardened, 3DPC exhibits typical concrete-like behaviour in compression but with significant anisotropy due to its layered structure. The hardened properties must therefore be described as a function of the orientation of loading relative to the printing directions. Mechanical properties of 3D printed concrete exhibit a strong directional dependence due to the layer-by-layer deposition. In accordance with the terminology adopted by RILEM TC 304-ADC [13], three principal directions are defined: u (parallel to the filament extrusion path), v (transverse within the printing plane), and w (vertical, orthogonal to the layers). These orientations (Orientation I = u, Orientation II = v, Orientation III = w), which are typically adopted in experimental testing, correspond to the principal printing and loading directions [13]. The mechanical behaviour of 3D printed concrete in the hardened state can be discussed by distinguishing its principal strength and stiffness parameters, each of which reveals specific features induced by the layered deposition process. In particular, compressive strength, flexural and tensile resistance, shear capacity,

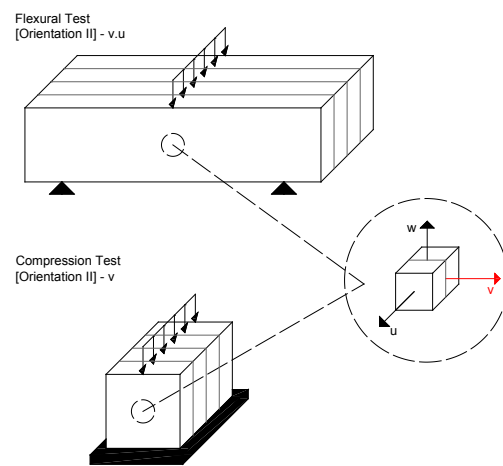
as well as elastic modulus and fracture energy, have all been investigated and are summarised as follows:

- *Compressive Strength*: In general, the compressive strength of printed specimens is comparable to cast reference values, but reductions of 10–30% have been systematically reported in the w-direction (loading perpendicular to layers), reflecting the weaker interfacial bonding [13].
- *Flexural and Tensile Strength*: Flexural and tensile strengths are strongly orientation-dependent. Tests in the weak direction (tension across interlayers) may return values 40–60% lower than cast samples, with large scatter due to voids and bonding variability [13]. Direct tensile strength is often measured using pull-off tests, which provide a reliable estimate of interlayer bonding capacity [16].
- *Shear Strength*: Interface shear behaviour, at material scale, follows a Mohr–Coulomb law, with a cohesion parameter, c , obtained from direct shear tests without precompression, and a friction angle ϕ derived from shear-compression tests under various normal stresses. Cohesion is highly sensitive to interlayer time, ILT, and other printing parameters, whereas ϕ is more stable [13].
- *Elastic Modulus*: The elastic modulus E_c is generally similar to that of conventional concretes of equivalent strength (15–20 GPa).

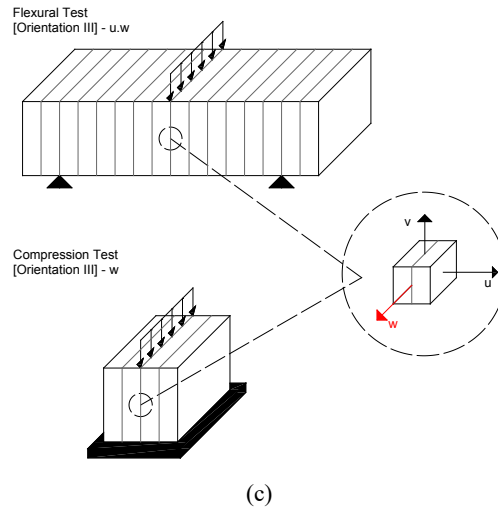
Altogether, the hardened state behaviour of 3DPC demonstrates that mechanical properties cannot be treated as isotropic quantities, but must be characterised as functions of direction and printing process parameters. Figure 2.1. provide a schematic representation of the three principal testing orientations for 3D printed concrete specimens, highlighting the anisotropy induced by the layer-by-layer deposition process (adapted from RILEM [13]).



(a)



(b)



(c)
Figure 2.1. The three principal testing orientations for 3D printed concrete specimens: (a) Orientation I(u); (b) Orientation I(v); (c) Orientation III(w);

The anisotropic behaviour of 3D printed concrete is strongly affected by the printing direction and by the interlayer time (ILT), i.e., the time lapse between the deposition of successive filaments.

Several studies (e.g.[12];[13] and [17]) have shown that increasing interlayer time (ILT) reduces the interlayer bond strength because the previously deposited layer cools/dries and a cold joint develops. More specifically, there is a work in literature that [17] explicitly link the mechanical properties of 3D-printed concrete to the time gap between successive passes. Figure 2.2 compiles results from multiple studies and compares how the uniaxial tensile strength varies with ILT, Δt .

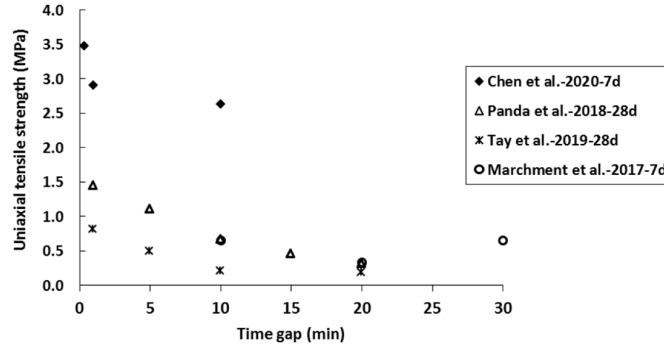


Figure 2.2. Impact of ILT in uniaxial tensile strength of 3D printed objects (reproduced from [17]).

Building on these data (Figure 2.2), in Figure 2.3 are illustrated the empirical trends obtained by least-squares polynomial fits for each dataset; the fitted equations and fit order are reported on the plot. For a physically consistent description that is monotone and saturates to a cold-joint plateau, we also report fits using the stretched-exponential (a.k.a Kohlrausch–Williams–Watts [18, 19],) decay, expressed in Eq. 2.1 , a standard model for relaxation processes [18,19].

$$f(\Delta t) = f_{\infty} + (f_0 - f_{\infty}) \exp [-(\Delta t/\tau)^{\alpha}] \quad [\text{Kohlrausch – Williams – Watts}] \quad \text{Eq. 2.1}$$

This formulation is suitable because it describes a monotonic and bounded decay of the interlayer bond strength with increasing interlayer time (Δt = ILT), tending towards a residual plateau f_{∞} that represents the strength of a cold joint. The parameters have clear physical meaning: f_0 is the fresh-on-fresh strength (at $\Delta t \approx 0$); f_{∞} is the residual strength at large Δt ; τ is the characteristic decay time; and α defines the decay kinetics—where $\alpha = 1$ yields a simple exponential decay, and $\alpha < 1$ a stretched (slower) decay. The Kohlrausch–Williams–Watts (KWW) law [18, 19], originally proposed by Kohlrausch (1854) and later extended by Williams & Watts (1970), is

a standard model for time-dependent relaxation in complex materials and has been widely applied to describe structural evolution processes of this kind [18, 19].

From a materials point of view, fresh cementitious systems mortar are thixotropic and undergo structural build-up at rest; when a new layer is placed after a delay, the interface has partially consolidated and moisture has decreased, reducing bond. This process-driven loss of adhesion naturally produces a decaying trend with a residual plateau, which the KWW law captures without nonphysical re-increase typical of high-order polynomials.

The calibration of τ and α was performed via nonlinear regression (least-squares fitting) on the experimental points, ensuring that the resulting curve not only provided the best R^2 fit but also maintained physical consistency with observed drying kinetics.

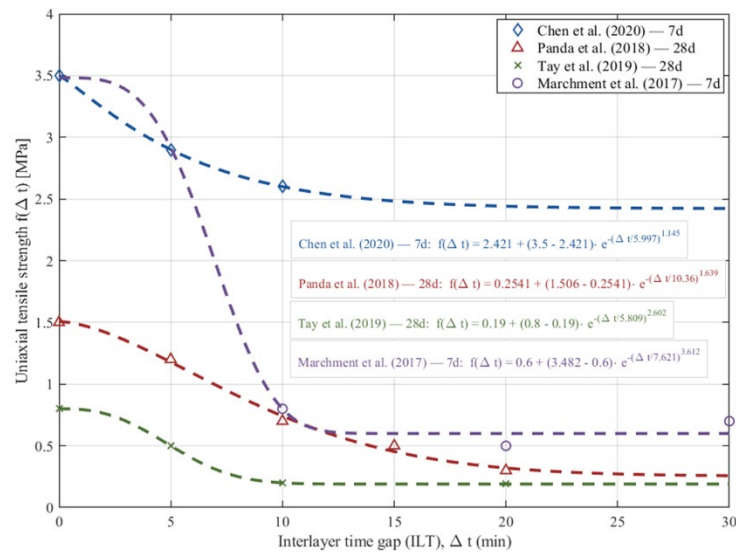


Figure 2.3. ILT-dependent empirical trends for interlayer tensile strength.

The fitted relations (Δt in minutes, f in MPa) are expressed in Eq. 2.2, Eq. 2.3, Eq. 2.4 and Eq. 2.5 from the different studies analyzed.

$$f(\Delta t) = 2.4 + (3.5 - 2.4)\exp[-(\Delta t/5.99)^{1.145}] \quad [\text{Chen et al (2020)}] \quad \text{Eq. 2.2}$$

$$f(\Delta t) = 0.25 + (1.5 - 0.25)\exp[-(\Delta t/10.36)^{1.639}] \quad [\text{Panda et al (2018)}] \quad \text{Eq. 2.3}$$

$$f(\Delta t) = 0.19 + (0.8 - 0.19)\exp[-(\Delta t/5.81)^{2.602}] \quad [\text{Tay et al (2019)}] \quad \text{Eq. 2.4}$$

$$f(\Delta t) = 0.6 + (3.5 - 0.6)\exp[-(\Delta t/7.62)^{3.61}] \quad [\text{Marchment et al (2017)}] \quad \text{Eq. 2.5}$$

Here, $f(\Delta t)$ denotes the measured uniaxial tensile strength for a given interlayer time gap Δt . As Δt increases, the tensile strength can decrease by up to 30–70% compared to the fresh-on-fresh condition, highlighting the critical influence of interlayer time on crack initiation, fracture propagation, and the overall structural integrity of 3D printed elements. For realistic and conservative structural assessment, ILT (Δt) should therefore be included explicitly in material models and design checks.

In conclusion, within the general framework of the hardened properties of 3D-printed concrete, Table 2.2 summarizes the principal hardened-state properties of 3DPC, identifies the applicable reference standards and 3DPC-specific adaptations, and reports the typical test outputs together with their units of measurement.

Property	Test / Specimen	Orientation(s) (u, v, w)	Standard / Reference	3DPC-specific adaptations	Typical Output / Units
Compression	Cubes/cylinders; cut/extracted coupons from printed elements	u, v, w (per test plan)	RILEM TC 304- ADC	Extract/cut coupons directly from printed walls/blocks	Compressive strength (f_c) [MPa]
Flexural strength	Prisms 40×40×160 mm (3- or 4-point bending)	Load direction vs layers (weak across- layer tension)	RILEM TC 304- ADC	Single-path coupons sawn from filaments/panels	Flexural strength (f_f) [MPa]

Splitting tensile (Brazilian)	Cylinders/cubes	—	RILEM TC 304-ADC	Less representative across interlayers than direct tension	Splitting tensile ($f_{ct,sp}$) [MPa]
Direct tensile / Pull-off (interlayer)	Dog-bone / pull-off across layers	Across layers (weak direction)	RILEM TC 304-ADC/ EN 1542-1999	Preferred for interlayer bond assessment	Direct tensile ($f_{ct,dir}$) [MPa]
Shear (interface)	Direct shear; shear-compression on coupons/panels	Across layers (interface plane)	Analogy ASTM C1531	Measure cohesion and friction angle (Mohr–Coulomb)	Cohesion (c) [MPa]; Friction angle (ϕ) [$^{\circ}$]
Elastic modulus	Static modulus with strain gauges/LVDT	u, v, w (optional)	RILEM TC 304-ADC	Account for anisotropy if extracted coupons are oriented	Elastic modulus (E_c) [GPa]

Table 2.2.: Hardened State Properties.

The testing protocols, described in Table 2.2, for hardened state properties of 3D-printed concrete largely rely on conventional EN/ASTM standards, but these have to be adapted to account for the layered, anisotropic, and time-dependent nature of additive manufacturing.

2.4 Mechanical Properties of the 3D-Printed Concrete Used in This Work – From Material to Intermediate Scale

2.4.1 Mechanical Properties of material tested (Heidelberg)

A ready to use dry printable mortar (i.tech 3D 2MM supplied by Heidelberg Materials) was adopted to print the structural walls, containing Portland cement and siliceous aggregates with a maximum particle size of 2 mm, with a final dry bulk density of 2000 kg/m³. The mix design ensured buildability properties during the printing process as well as the original shape of the extruded elements expected by the 3D model. The main mechanical characteristics of the printable mortar

in the hardened state after 28 days were determined according to UNI EN 196-1 [20] for both cast and printed configuration. The resulting compressive and the flexural strengths of the cast samples were 45 MPa and 9 MPa, respectively, on the average of 3 samples.

Samples for basic mechanical characterization of the 3D printed walls were also considered. Specifically, printed samples were retrieved from square printed paths made of a single concrete filament in width, then cut and sawn into 4x4x16 cm samples as done in previous studies [20, 21, 22]. The directions and testing methods for printed material were defined according to RILEM interlaboratory study plan [23-24, 25], relative to the orientation of the printed layers and results interpretation. In this study, the mechanical properties were specifically evaluated in the direction orthogonal to the deposited layers. For compressive behaviour, the W-direction was tested to assess the compressive strength. For flexural strength, the U-W plane was considered, focusing on the response of the material and interlayer interfaces under bending. The resulting average strengths after 28 days of curing in open air conditions were 40 MPa and 7 MPa for the compressive and flexural strength, respectively. Figure 2.4(a) shows representative images of the extraction/sawing phases for the printed specimens used for mechanical characterization in compression and flexure (Figure 2.4(b)). In addition to the basic mechanical characterization of the printable material in the hardened state, material samples were also extracted post-mortem from the wall specimens for further investigations. Specifically, pull-off tests (according to EN 1542 [26]) were performed orthogonally to the

filament deposition plane, yielding an interlayer tensile strength ranging between 1.5 and 2.5 MPa. Direct shear tests campaign, currently ongoing and adapted from the masonry triplet test standard [27], is being carried out on sawn specimens to determine the interlayer shear strength (or cohesion, c , at the material scale), which is found to range between 3.5 and 4.5 MPa.

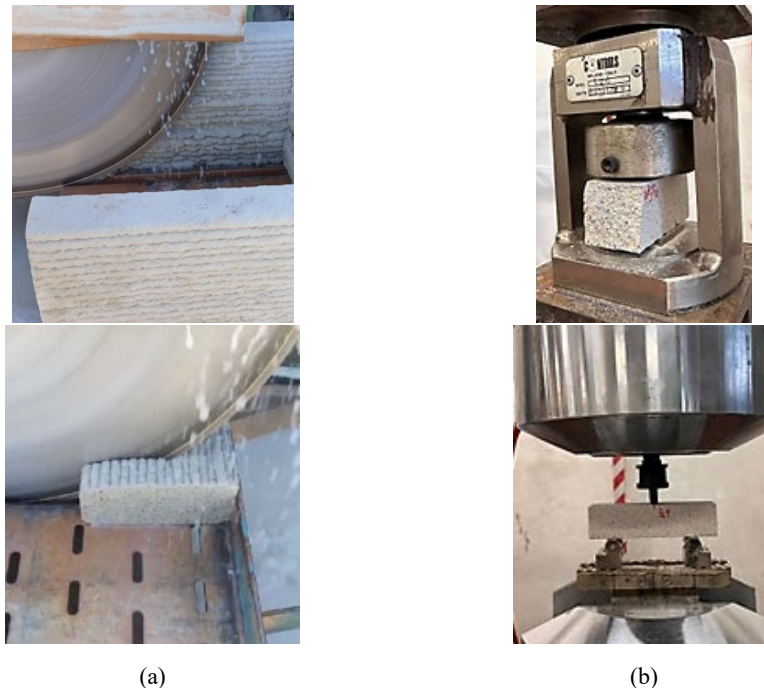
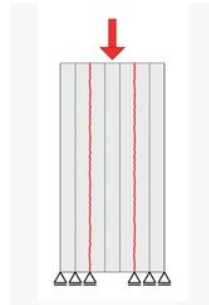


Figure 2.4.: Phases of sample extractions and sawing (a); compression and flexural testing after sample preparation from 3D printed elements (b).

More specifically, direct shear tests on specimens (7×6 cm shear loading surface and 14 cm specimen height), performed analogously to the triplet test commonly used for masonry [27] and as illustrated in Figure 2.5, the ongoing tests indicated a cohesion of approximately 3.5 MPa under zero pre-compression, while complementary shear–

compression tests resulted in a friction angle of about 35° (in the starting stage of the experimental campaign).

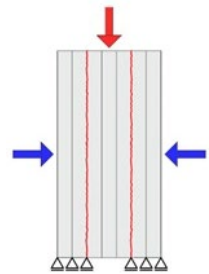
Although higher shear strength values are expected under pre-compression, those tests are part of ongoing investigations and will be discussed in future studies. For the purpose of this PhD work, only the cohesion value under zero pre-compression is reported, to provide a consistent reference within the adopted framework. For the sake of clarity, Figure 2.5 show direct shear test schematic representation without normal stress (a); test photo without normal stress (b); schematic representation with applied normal stress (c); test photo with applied normal stress (d).



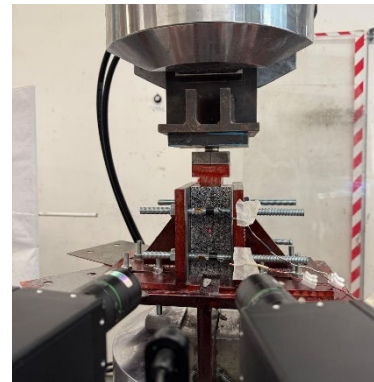
(a)



(b)



(c)



(d)

Figure 2.5.: Direct shear test: schematic representation without normal stress (a); test photo without normal stress (b); schematic representation with applied normal stress (c); test photo with applied normal stress (d).

In summary, Table 2.3 reports all the main mechanical properties of the mortar used in this work, specifying the test element, the reference standard followed, the property considered, and the corresponding reference value.

Category	Property	Value / Range	Standard / Note
Dry mix	Maximum aggregate size	≈ 2.0 mm	Declared for the dry compound.
Dry mix	Bulk density (dry mix)	$\approx 1,230 \pm 50$ kg/m ³	DIN EN 1097-3.
Dry mix	Fire classification	A1 — non-combustible	DIN 4102-1.
Fresh (lab mix)	Mixing water (by dry mass)	≈ 12.0 – 13.0 %	Lab conditions 20 ± 2 °C; RH 65 ± 5 %.
Fresh (lab mix)	Fresh density	$\approx 2,210 \pm 50$ kg/m ³	DIN EN 1015-6.
Fresh (lab mix)	Flow table spread	$\approx 140 \pm 15$ mm	DIN EN 1015-3.
Processing	Recommended fresh mortar temperature	20 ± 10 °C	Processing guidance.
Mechanical (lab prisms 40×40×160 mm, water-cured at 20 ± 1 °C)	Compressive strength	≥ 10 MPa (1 d); ≥ 30 MPa (7 d); ≥ 45 MPa (28 d)	DIN EN 196-1.
Mechanical (lab prisms 40×40×160 mm, water-cured at 20 ± 1 °C)	Flexural strength	≥ 2 MPa (1 d); ≥ 5 MPa (7 d); ≥ 7 MPa (28 d)	DIN EN 196-1.

Table 2.3.: Main Properties, Orientation I, of Material Tested (Heidelberg).

In addition to the reference mechanical properties obtained from cast samples, a dedicated experimental campaign (ongoing) is performed on specimens extracted from the printed walls to evaluate the material behaviour in the printed configuration. Compressive and flexural strengths were determined according to the same procedures as for the cast material, while interfacial tensile and shear strengths were evaluated through pull-off and direct shear tests, respectively. Additional shear–compression (triplet-type) tests are performed to derive the cohesion and friction angle of the interlayer under combined normal and tangential actions.

Category	Property	Direction / Condition	Value / Range	Note
Printed (hardened, 28 d, air-cured)	Compressive strength	Orientation I (u)	38.29 MPa	Material Scale
Printed (hardened, 28 d, air-cured)	Flexural strength	Orientation 3 (u.w)	≈ 7 MPa	Material Scale
Interlayer (interface)	Tensile bond strength	to layer plane (u)	1.3 – 2.7 MPa	Material Scale
Interface (triplet-type test)	Cohesion, Shear Strenght	—	3.5 - 4.5 MPa	Material Scale (Ongoing)
Interface (triplet-type test)	Friction angle	—	$\approx 35^\circ$	Material Scale (Ongoing)

Table 2.4.: Main mechanical properties of the 3D-printed material (Heidelberg).

2.4.2 The Shear Behavior Of 3d Printed Concrete Panels Under Diagonal Compression Test

In order to assess the shear strength also at the panel scale, diagonal compression (shear) tests were carried out in accordance with ASTM standards [28], similarly to what is typically done for masonry. This choice stems from the need to capture potential scale effects introduced by the printing process and by the formation of the interlayer interface, whose characteristics are strongly influenced by the interlayer time (ILT).

Indeed, while in conventional concrete structures the material strength (e.g., compressive, tensile and shear strength) directly represents the design parameter, this assumption does not hold for masonry. In masonry, a clear distinction exists between the unit strength (brick or block) and the wall strength, which emerges from the combined behaviour of units and mortar joints. Similarly to masonry, the mechanical characterization of 3D-printed concrete is performed at the wall-panel scale, rather than on the constituent material alone.

Following this way, diagonal shear tests on 3D-printed concrete panels were conducted to investigate their structural-scale behaviour and to determine the corresponding initial diagonal shear strength, f_{vk0} , representative of the global shear capacity of printed walls.

2.4.2.1 Print Process And Sample Preparation

Square panels of about 800x800x300 mm were printed by using a 25 mm diameter nozzle, 10 mm layer height and 100 mm/s linear printing speed. Due to the printing process, which does not involve the

use of formworks, the thickness of the panel walls cannot be considered constant along their height. Out of the four different patterns shown in Figure 2.6, Pattern (a) includes no infill and is commonly used for walls with low strength requirements, while Patterns (b), (c), and (d) are more commonly observed when infill is required.

In Figure 2.6 it was reported the cross-section together with the print direction (black arrow), the transversal longitudinal section in which the compressive load acts and a lateral view to evidence the infill role on the stress distribution along the whole panel.

Two printing methods were adopted. The hollowed pattern (a), composed solely of the outer shell, was easier to design and print, as each successive layer directly replicates the printing trajectory of the previous one. The path was optimized to ensure continuous extrusion and motion. Conversely, the honeycomb infill patterns (b), (c), and (d) were produced using a different printing method: the nozzle first follows the contour, then creates a zigzag infill, and finally retraces the previous printing path in the next layer. This results in a repeating cycle every two layers, so that two layers are effectively treated as one continuous cycle.

In this case, crosses on the path are created by the contact between external contour wall and infill, at the turns of the printing paths.

More specifically, Pattern (b) is based on a quadratic wave infill with a side length of 222 mm and a total of 6 units. The connection with the external sides is continuous over 154 mm.

Patterns (c) and (d) have a triangular infill geometry (triangular weaves), with a distance of 190 mm between each triangle side and a total of 6 wave units. In this case, the connection with the external edges of the panel occurs at a single point (the vertex and the base of the triangle).

For the sake of clarity, the panels' label corresponds to their respective pattern shapes. The letters 'QS' and 'TS' in the specimen labels represent Quadratic Shape and Triangular Shape, respectively. 'TS' and 'TS1' indicate two distinct triangular shapes adopted in the study. 'REF' refers to the reference specimen with no infill pattern.

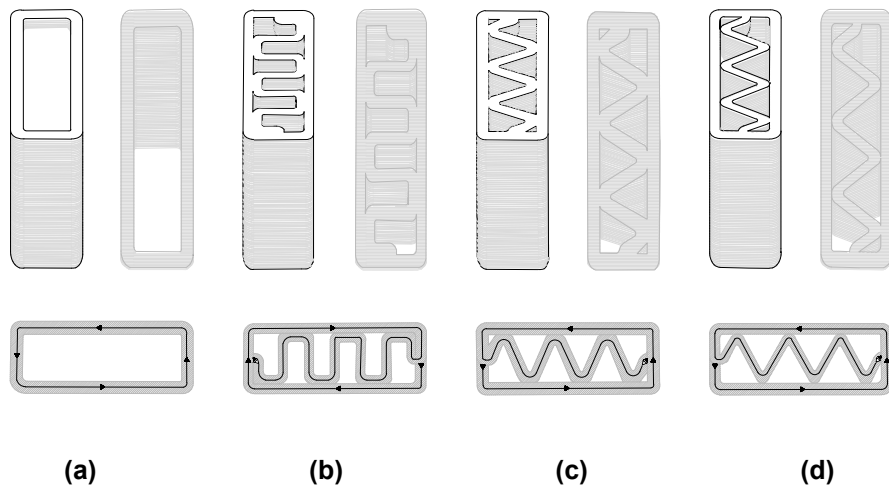


Figure 2.6.: Different cross-sectional patterns of the selected 3DPC panels: PANEL 1 (REF) with no infill (a), PANEL 2_QS (b), PANEL 3_TS (c), and PANEL 4_TS1 (d)

Thus, panels were characterized from different percentage of voids (ranging from 32% and 57%) with the same external geometry (i.e. 800x800x300 mm square panel).

2.4.2.2 Test Methods

The shear strength of the 3D printed concrete panel specimens was determined, analogously to masonry, according to ASTM E509-02 [28] by diagonal compression tests on 800x800 mm square panels (Figure 2.7). Four specimens were tested. The test panels were built with layer parallel to the laboratory basis and then turned according to the test standard. Specimens were nebulized with water and covered with a plastic sheet for a curing-time of seven days to reduce risk of cracking. After at least 28 days from fabrication, the panels were clamped by an external rigid steel frame and then carefully rotated 45° to be positioned on a steel loading shoe. The adopted test set-up is described in Figure 2.7. The diagonal displacement was applied using a large-scale steel testing frame equipped with a servo controlled hydraulic actuator having a capacity of 3000 kN. The load was measured using an additional 1000 kN load cell with an accuracy of 0.1 kN.

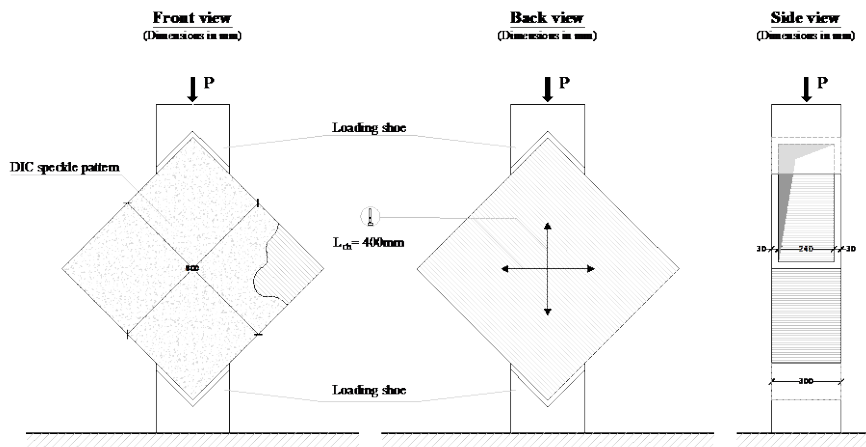
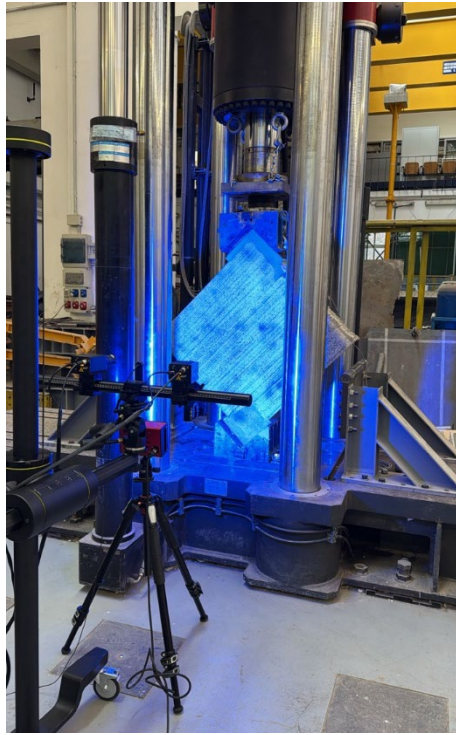


Figure 2.7: Diagonal compression test: panel geometry and typical test set-up

Concerning the measuring apparatus arrangement, two different techniques were used to monitor the deformations of the panels. On the front side of the sample the displacements were measured by using the 3D Digital Image Correlation (DIC) technique through the ZEISS ARAMIS Adjustable 24M. Two 24-megapixel cameras were used to capture 3D displacements and surface strain, both for point-based and full-field measurements, during the test. The system captured pictures every 15 s. The data analysis was performed through the commercial software GOM Correlate. The speckle pattern was prepared by spraying black paint on the panel surface previously painted white (Figure 2.8). In order to validate the results provided by the DIC, traditional transducers were mounted on the other side of each tested beam. Two potentiometric transducers were placed on the back side of the sample and aligned parallel to the panel diagonals (Figure 2.7).



(a)



(b)

Figure 2.8: Diagonal compression test: DIC methodology arrangement (a), Traditional transducers (b)

2.4.2.3 Results and Discussions

Figure 2.9 reports load vs. applied vertical displacement experimental curve of the four panel specimens (a). Concerning the global behavior, the panels were characterized by different stiffness and resistance: panels have comparable elastic modulus whereas TS1 exhibits a higher stiffness, until to reach an unexpected failure mode, which led to initial premature cracking and a change in stiffness before reaching the actual shear failure. Overall, these different behaviors seem to confirm the influence of the infill pattern on the global structural behavior of the panel.

Figure 2.9 (b) shows a correlation between the maximum applied load and the actual vertical net area (A_n) measured on the specimen. It is worth noting that, with reference to the TS1 panel and in light of the previous considerations on the failure mode, the value taken into account refers to the diagonal shear failure, whereas for the remaining panels, the values used in the plot correspond to the onset of first cracking. Assuming a linear correlation with a zero intercept, a strong correlation ($R^2 = 0.95$) was found, confirming the dependence of the specimen's resistance and the net area even though the REF panel would seem to be underestimated. Nevertheless, evaluating the net area without considering its spatial distribution may be inadequate, especially when analyzing stiffness differences among specimens; hence the shear response of the four tested panels was investigated as well. The shear strain (γ) was calculated as $\gamma = (\Delta x + \Delta y)/g$, where: γ = shearing strain, mm/mm; Δx = shortening in the direction parallel to loading, mm; Δy = extension in the direction perpendicular to loading, mm and g = gage length in the direction parallel to loading, mm. Δy must be based on the same gage length as for Δx .

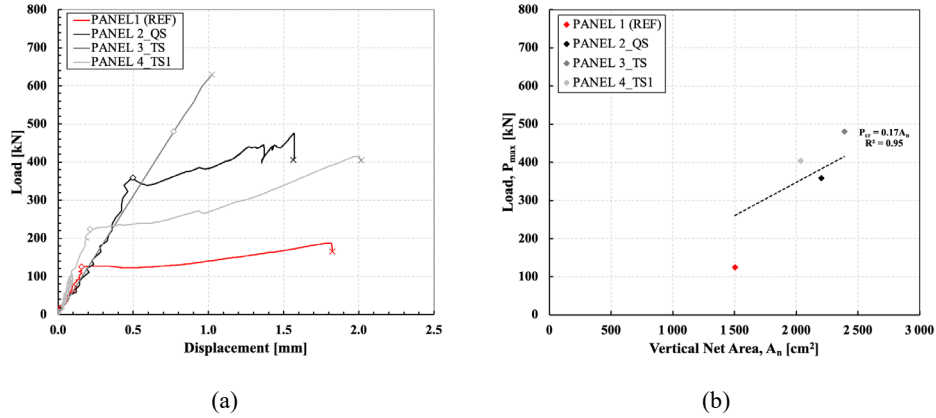


Figure 2.9: Load Vs Deflection for the four tested panels(a), Correlation between Load and Vertical net area (b).

Concerning the shear stress on traditional masonry, the ASTM assumes that the diagonal compression test causes a uniform shear stress $\tau = \frac{0.707 \cdot V}{A_n}$, where A_n is the net area of the specimen, thus the vertical section which acts to the shear (Figure 2.6). The latter would result from the equation $A_n = \left(\frac{w+h}{2}\right) t \cdot n$, where: w = width of specimen, mm; h = height of specimen, mm; t = total thickness of specimen, mm; n = percent of the gross area of the unit that is solid, expressed as a decimal. The modulus of rigidity (modulus of elasticity in shear) is calculated as $G = \tau/\gamma$, where G = Modulus of rigidity in MPa. Figure 2.10 (a) plots the shear stress vs. strain response for the four panel according to ASTM; the dashed lines represent the shear stress computed following ASTM code, but using the actual net areas retrieved from the specimens instead of the nominal values calculated according to the standard. As expected, it should be noted that the shear modulus, as derived from the ASTM provisions, is predominantly governed by the nominal net area (A_n). Notably, for

identical specimens, the use of actual rather than nominal net area A_n results in substantial discrepancies in the evaluated stiffness, thereby raising critical questions regarding the adoption of the code.

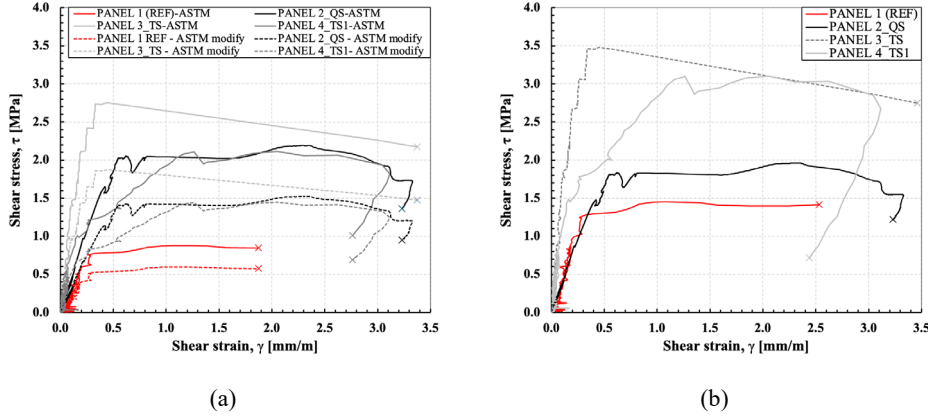


Figure 2.10: Shear stress Vs strain responses of the four panel specimens according to ASTM (a) and Jourawsky theory (b).

Moreover, ASTM formulation is not suitable for this research because of the non-uniform and asymmetrical panels sections; hence the more accurate Jourawsky's theory was adopted which allows for a more accurate and spatially aware analysis of shear stresses (Figure 2.10 (b)). The formula derived by Jourawski for the shear stress at a given point in a cross-section is $\tau_{zy} = \frac{V S_n^*}{b_n I_x}$, where V is the shear force on the section, S_n^* is the first moment of inertia about the strong axis, b_n is the width of the section (at location n , perpendicular to the shear stresses) and I_x is the second moment of inertia about the strong axis. Thus, the maximum shear stress will be at the middle of the vertical cross section due to the higher first moment of inertia. Figure 2.10 (b) plots the highest shear stress value (at the middle of the vertical cross section) vs. the shear strain experimentally retrieved. Beyond the differences in shear strength among the four panels, it could be

observed that the stiffness of the REF panel is nearly identical to that of the panel with quadratic infill (PANEL 2_QS). On the contrary, the two panels featuring triangular infill exhibit a significantly higher shear stiffness. Furthermore, regarding the overall structural response, it is worth noting that the two panels with triangular infill reached comparable maximum shear strengths, although panel TS1 achieved this peak with a stiffness reduction beyond a shear strain of approximately 0.25 mm/m.

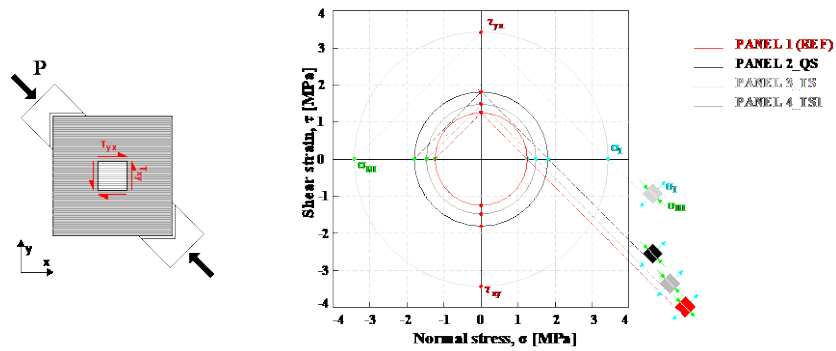


Figure 2.11: Interpretation of the diagonal compression test according to ASTM by the Mohr's circle representation.

By assuming a pure shear stress state at the center of the panel, the principal compressive and tensile stresses can be determined using Mohr's circle, along directions inclined at 45° to the principal axes (Figure 2.11).

Accordingly, based on the maximum applied load, the tensile strength of 3DPC panels can be obtained indirectly from diagonal compressive test. However, the definition of the internal stress field within the panel under diagonal loading remains a matter of debate within the scientific community. The ASTM standard assumes a pure shear stress state, whereas the RILEM guidelines suggest that the stress field in a square

panel subjected to diagonal compression is non-uniform and characterized by a combination of normal compressive and shear stresses.

	REF	QS	TS	TS1
Cross section properties				
Cross section, A_c [cm ²]	1068	1560 (+46%)	1692 (+58%)	1442 (+35%)
Vertical net area effective, A_n [cm ²]	1503	2205 (+47%)	2389 (+59%)	2037 (+36%)
Vertical net area by ASTM, A_n^{ASTM} [cm ²]	1023	1533 (+50%)	1625 (+59%)	1395 (+36%)
Percent of the gross area of the unit that is solid, expressed as a decimal, n [-]	0.43	0.64	0.68	0.58
Shear parameters				
Maximum diagonal compression load, P_{max} [kN]	104	434	480	404 (*)
Shearing strain, γ [mm/m]	0.27	0.55	0.34	1.263 (*)
Shear stress				
a) according to Jourawsky's Theory, $\tau_{zy, Jourawsky}$ [MPa]	1.25	1.81	3.43	1.48
b) according to ASTM code, $\tau_{zy, ASTM}$ [MPa]	0.76	2.02	2.7	2.11
c) according to ASTM code with effective A_n , $\tau_{zy, ASTM}^*$ [MPa]	0.46	1.40	1.86	1.44
Modulus of rigidity				
a) by using $\tau_{zy, Jourawsky}$, $G(\tau_{zy, Jourawsky})$ [MPa]	4630	3291	10088	n.a.
b) by using $\tau_{zy, ASTM}$, $G(\tau_{zy, ASTM})$ [MPa]	2815	3673	7941	n.a.
c) by using $\tau_{zy, ASTM}^*$, $G(\tau_{zy, ASTM}^*)$ [MPa]	1704	2545	5471	n.a.
Failure Mode				
Description	Diagonal shear	Diagonal-shear & Splitting	Diagonal-shear	Sliding & Diagonal-shear & Splitting

(*) Value corresponding at the diagonal-shear failure

Table 2.5.: Main geometrical panel's characteristics and properties obtained from diagonal compression tests.

These results offer valuable insights into the role of infill pattern geometry on the overall structural performance of the panels, suggesting that variations in stress distributions mechanisms are influenced by the infill pattern. Specifically, regarding shear behavior,

the quadratic infill, not lead to a substantial increase in stiffness or shear strength compared to the hollow panel, despite an increase in net area (+46%). Conversely, the triangular infill appears to enable a more efficient stress distribution resulting in a better mechanical performance. More specifically, Table 2.5 reports the main geometrical panel's characteristics and the main results of the tests, including the maximum diagonal compression load (P_{max}), the corresponding shear strength obtained through the different methods above reported, the shear strain at the crack load (γ) and the modulus of rigidity (G). It is worth remarking that the mechanical properties obtained from 3DPC was considerably higher than typical strength values suggested by the literature as well as by structural codes for unreinforced masonry.

Masonry Type	Cat.	f_m [N/mm ²]	τ_0 [N/mm ²]	f_{v0} [N/mm ²]	E [N/mm ²]	G [N/mm ²]
Disorganized irregular stone	I	1.0	0.018	-	690	230
		2.0	0.032		1050	350
Barely cut stone with leaves of uneven thickness	II	2.0	0.035	-	1020	340
		2.6	0.051		1440	480
Roughly cut stone with good texture	III	2.6	0.056	-	1500	500
		3.8	0.074		1980	660
Irregular soft stone (tuff, limestone, etc.)	IVa	1.4	0.028	-	900	300
		2.2	0.042		1260	420
Ashlars of soft stone (tuff, limestone, etc.)	IVb	2.0	0.04	0.10	1200	400
		3.2	0.08	0.19	1620	500
Stone square blocks	V	5.8	0.09	0.18	2400	800
		8.2	0.12	0.28	3300	1100
Bricks and lime mortar	VI	2.6	0.05	0.13	1200	400
		4.3	0.13	0.27	1800	600
Hollow bricks with cementitious mortar (e.g., holes $\leq 40\%$)	VII	5.0	0.08	0.20	3500	875
		8.0	0.17	0.36	5600	1400

Table 2.6.: Mechanical parameters of existing Italian masonry categories.

For instance, as regard to existing unreinforced brick and block masonry, the Italian code [29, 30, 31] provides both τ_0 and f_{v0} values i.e the value of the shear strength at zero normal stress at failed. The former is defined as a general parameter valid for all masonry types, while the latter is introduced specifically for regular masonry and is applied in cases where failure occurs through the mortar joints, typically resulting in a diagonal stepped crack along the bed and head

joints. Regarding to existing unreinforced brick and block masonry, the code reports values of the shear strength at zero normal stress ranging from 0.05 MPa to 0.17 MPa (Table 2.6).

Failure of all panels was governed by a classical combination of sliding-shear and diagonal-shear mechanisms that caused a sudden drop of the capacity right after the achievement of the peak load. In this regard, in TS panel, when the load is applied on the top loading steel plate, stresses spread more homogeneously through the whole panels by means of transversal inclined concrete ribs by following a preferential path. A better stress distribution on the cross section of TS panel results in a higher maximum shear stress, as well. DIC methodology have permitted to accurately assess the crack pattern formation over the tests.

Figure 2.12 illustrates the ultimate crack patterns exhibited by four wall panels subjected to diagonal compression tests, highlighting the influence of internal panel configuration and boundary conditions on failure mechanisms. The TS panel (Figure 2.12 (c)) presents a nearly ideal crack trajectory, characterized by a single, well-defined vertical crack aligned with the direction of the applied compressive load. This behavior reflects a uniform stress distribution and minimal interference from internal structural anomalies or support effects.

In contrast, the REF (Figure 2.12(a)) and QS (Figure 2.12(b)) panels exhibit more complex ultimate crack pattern, deviating from the expected vertical alignment. First cracking, initially occurred in proximity of the end of the loading shoe, orthogonally at one side of the panel and tends to follow that direction until to reach the mid height;

thereafter it deviates and became vertical. Simultaneously a second diagonal crack occurs. Moreover, additional secondary cracks arisen, as well. These irregular patterns suggest significant interaction with internal pattern, pre-existing weaknesses within the matrix, as well as possible influence from eccentric loading.

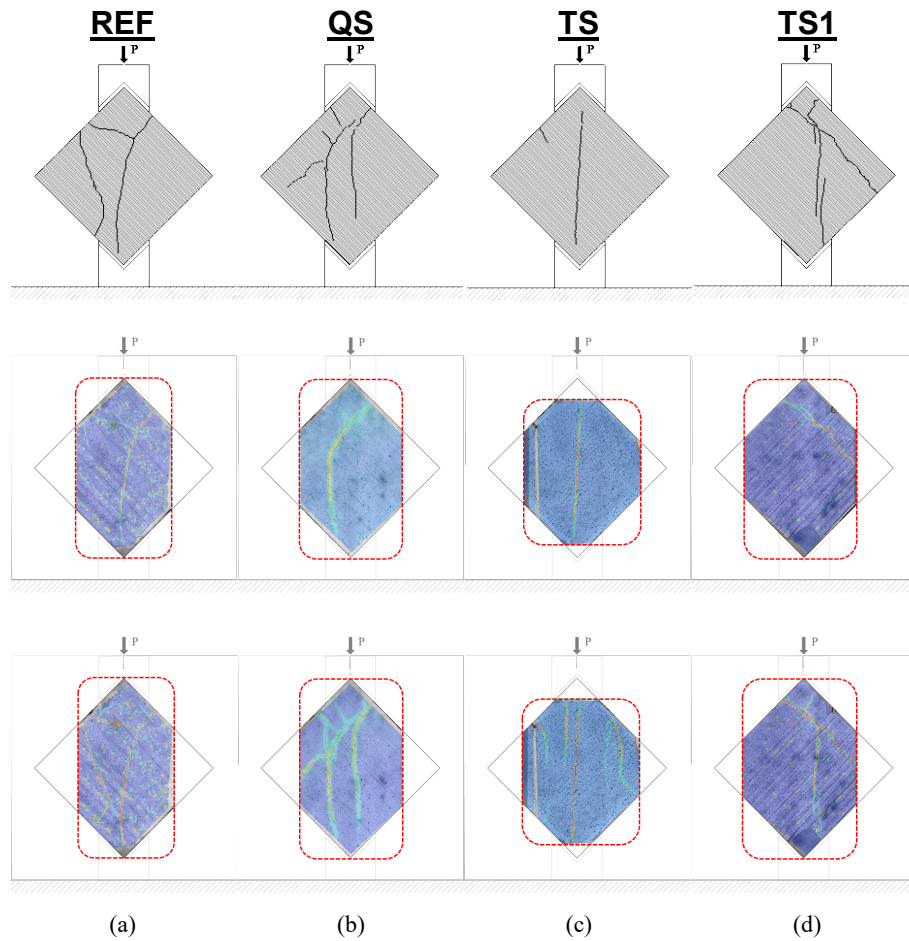


Figure 2.12.: Ultimate crack patterns exhibited from PANEL 1 (REF) with no infill (a), PANEL 2_QS (b), PANEL 3_TS (c), and PANEL 4_TS1 (d). First failure and ultimate crack pattern.

The TS1 panel (Figure 2.12(d)) shows a pronounced diagonal cracking starting from the loaded corner, along one of the horizontal

interface of the internal partition and tends to follow its direction until to reach the opposite edge; thereafter, a second diagonal crack occurs. This pattern may be attributed to the influence of the loading shoe at the loading point, which likely introduced localized stress concentrations and altered the primary stress path. Overall, the deviation from the ideal crack path in REF, QS, and TS1 highlights the combined effects of panel heterogeneity and boundary conditions on the structural response under diagonal compression. In addition, specimens QS and TS1 exhibited a splitting failure across the thickness of the panels, occurring in the longitudinal direction (i.e. along the height) (Figure 2.13).

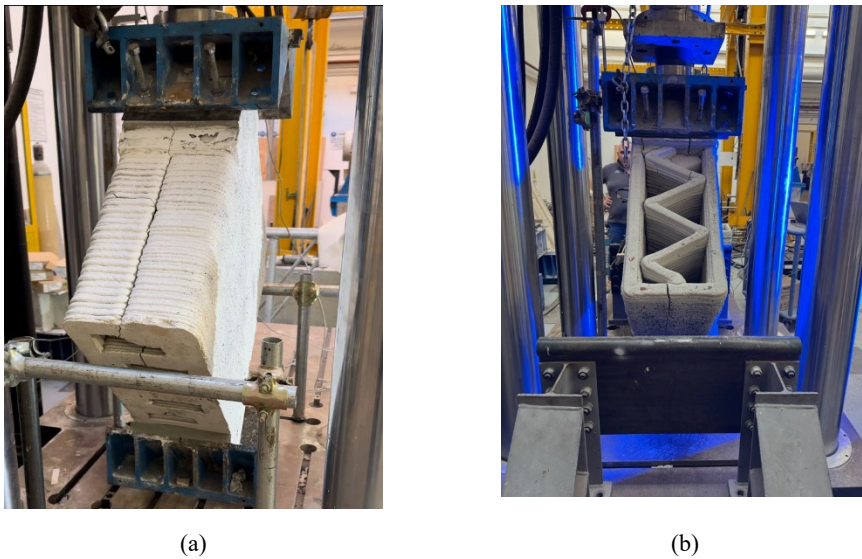


Figure 2.13.: Splitting failure across the thickness of the panels QS (a) and TS1 (b).

2.4.2.4 Conclusions

This experimental campaign has demonstrated the fundamental role of the internal printing path configuration on the shear response of 3D-printed concrete (3DPC) panels. Diagonal compression tests revealed that, while the quadratic infill did not significantly enhance stiffness or shear capacity compared to the hollow reference panel, triangular infills promoted a more homogeneous stress distribution and markedly improved the mechanical performance. In particular, the inclined ribs of the triangular pattern acted as diagonal struts, effectively mobilizing a larger portion of the cross-section and leading to higher shear stiffness, greater load-bearing capacity, and reduced out-of-plane displacements.

Importantly, despite the larger nominal net area of the quadratic infill, its structural contribution proved inefficient—highlighting that infill geometry, rather than the mere amount of solid material, is the critical factor governing the shear performance of printed panels. Moreover, the shear strengths observed in 3DPC panels were significantly higher than those typically prescribed for unreinforced masonry in current design codes. This finding clearly indicates that assimilating 3D-printed concrete to masonry is conceptually inappropriate. Instead, the printing path configuration should be regarded as a genuine structural parameter, capable of governing stiffness, strength, and failure mechanisms.

These results also clarify that the parameter previously identified as cohesion exhibits scale-dependent behaviour: higher values are observed at the material scale, while lower and geometry-dependent values emerge at the panel scale, reflecting the influence of interlayer

properties and infill configuration. Overall, the results highlight that printing path design directly influences stress distribution, crack propagation, and failure modes. By demonstrating that geometric and process-related parameters can fundamentally alter structural behaviour, this study reinforces the need to integrate printing path design into performance-based frameworks, and ultimately, into structural design codes for digital concrete construction.

In conclusion, it is worth noting that, for the square configurations analysed in this section, the results of the direct shear tests (triplet-type) provided a shear strength of about 3.5 MPa, whereas the diagonal compression (shear) tests, performed according to ASTM E519, yielded a diagonal shear strength of approximately 2 MPa. This difference highlights the presence of scale effects, the possible influence of interlayer time (ILT), and, more generally, the strong dependence on the printing path configuration. Drawing an analogy with masonry, it is worth recalling that, in the absence of a large experimental database at structural scale—which would allow a thorough characterization of shear strength as a function of parameters such as interlayer time and printing process conditions—several empirical formulations have been proposed to estimate the initial shear strength of walls based on the material tensile capacity. Among these, the well-known Turnšek and Čačović (1971) [32] approach provides a conservative empirical relationship between the diagonal shear strength and the tensile strength of masonry materials, expressed as $f_{vk0} \approx \frac{f_{ctk}}{1.5}$.

This formulation is typically used when the shear parameters are not experimentally determined. Adopting this approach for 3D-printed concrete, and considering a tensile strength of about 1.3 MPa, the corresponding reference shear strength would be approximately 0.87 MPa. This value is substantially lower than those obtained experimentally—about 3.5 MPa from the triplet test and 2 MPa from the diagonal shear test—thus confirming that a direct application of masonry-based correlations would lead to overly conservative estimates for 3D-printed concrete. Nevertheless, at the current stage of research, and for the sake of a conservative and consistent design assumption, it is reasonable to adopt the shear strength inferred from tensile strength, following the same rationale used for masonry, until a sufficient database of dedicated tests allows the calibration of specific relationships for 3D-printed concrete.

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3 In-Plane Behaviour of 3D Printed Concrete Walls Subjected to Cyclic Loads

Within this PhD work, the focus is placed on understanding the structural behaviour of 3D printed concrete elements to establish reliable design parameters and modelling strategies. In this context, full-scale 3D printed walls were tested under in-plane cyclic loading to investigate their seismic response and the influence of connection details. Two wall configurations were examined: one with a prefabricated steel base anchorage and the other with a traditional cast-in-place base using steel rebars. Both configurations reproduced fixed boundary conditions, achieved respectively through integrated steel inserts or rebars anchored into the foundation. The printing processes also differed, with prefabricated components integrated during the printing path in the first configuration and anchorage bars inserted during assembly in the second. The experimental results revealed distinct force–displacement behaviours for the two configurations, characterised by three stages: elastic, yielding up to the peak load, and ultimate failure. Crack pattern observations showed different damage evolutions influenced by the layered nature of the printed material and the anchorage system. This experimental campaign also allow to develop an analytical interpretation,

considering an equivalent bilinear capacity curve and classic failure mechanisms of masonry such as flexural-compression, shear, and diagonal tension.

3.1 Introduction

Recent developments in additive manufacturing (AM) have positioned 3D Concrete Printing (3DCP) as a promising technique capable of transforming conventional construction methods [1]. Among the various AM approaches using cement-based materials [2,3], 3DCP has gained attention for the maturity of printable mixtures [4] and the flexibility of the available printing systems, such as gantry and robotic arm setups [2]. This process allows for the automated production of structural and non-structural components with complex geometries, potentially reducing formwork, waste, and labour [5-8].

Nevertheless, the large-scale structural use of 3D printed concrete requires careful evaluation of the material's peculiarities, such as interlayer weakness, anisotropy, and the dependency of mechanical properties on printing direction and process parameters [9–12]. These characteristics distinguish 3D printed concrete from both traditional homogeneous concrete and masonry-like heterogeneous materials, raising questions about how to represent its behaviour in design.

While several studies have explored material-scale properties and printing parameters [12, 14], experimental evidence at full scale remains limited. In particular, there is scarce information on the in-plane cyclic response of unreinforced 3D printed walls, where failure

mechanisms such as flexural–compression, shear–sliding, and diagonal cracking may coexist. Understanding these mechanisms is essential to identify design-relevant parameters—such as shear strength and deformation capacity—especially in seismic applications.

To address these gaps, the present work investigates the in-plane cyclic behaviour of full-scale 3D printed concrete walls with 40% internal voids. Two base connection systems are compared: a prefabricated steel anchorage integrated during printing, and a cast-in-place solution with embedded rebars. Both configurations reproduce fixed boundary conditions but differ in the way the load is transferred to the foundation. The cyclic tests allow for assessing stiffness degradation, energy dissipation, and failure progression, with particular attention to the interaction between the layered structure and the base constraint. The outcomes of this work provide fundamental experimental evidence for interpreting the global response and local failure mechanisms of printed walls, laying the foundation for the analytical modelling and parameter calibration developed in the subsequent stages of the PhD research.

3.2 Materials and Methods

3.2.1 3D Printable mortar

The structural walls are printed using the dry printable mortar previously described in Chapter 2, characterized by high buildability and extrusion stability. The full material composition and mechanical

properties, including compressive, flexural, and interfacial strengths, are presented in that chapter.

3.2.2 Fabrication of the 3D printed wall panels

Two unreinforced 3D-printed concrete walls of total height of 210 cm, 131.5 cm length and 35 or 41 cm (gross) depth were fabricated through 3DCP system consisting of a continuous pumping machine (M-tech duomix) connected to an ABB robotic arm (available from Etesias srl company). Each wall was composed of 210 printed layers (i.e., concrete filaments), each with a nominal height of 10 mm and an average width of approximately 35 mm. The panels were fabricated using a predefined hollow printing path (see Figure 3.1(e),(f)), with a nozzle travel speed of 60 mm/s and a continuous deposition sequence - without pauses between layers. The material was extruded at a nominal output rate of 9 L/min, resulting in an average interlayer time of approximately 1.5 minutes. To minimize moisture loss and promote adequate interlayer bonding, the specimens were kept under controlled laboratory conditions (25 ± 2 °C, $60 \pm 5\%$ RH) during and after printing. Additionally, the printed walls were covered with plastic film for 24 hours following the completion of the printing process. It is important to note that the walls were printed to a height of 2.10 meters using solely 3D printed concrete, hence without internal reinforcements.

The study concentrated on analysing two base anchorage systems, characterized by two specific geometries: (i) 3D-printed Concrete Wall with Prefabricated Base Anchorage (P-BA3DW); (ii) 3D-printed

Concrete Wall with Conventional Base Anchorage (C-BA3DW). In both cases, the base anchorage was used to reproduce the fixed constraint of the wall at the foundation. Figure 3.1 (a) and (b) illustrate the two types of base anchorage, and the printing process associated to both configurations. The two anchorage configurations were specifically selected to explore different design solutions for connecting 3D-printed concrete walls to foundations: (i) the prefabricated anchorage system was chosen to investigate an innovative hybrid solution leveraging prefabricated steel components (anchor bolts and column shoes) which can be integrated during the printing process; (ii) the conventional anchorage system with cast-in-place rebar dowels was adopted as a reference configuration more consistent with traditional practice. This comparative approach aimed to assess how different connection strategies influence the structural response and failure mechanisms, providing insights relevant both for innovative and conventional construction scenarios.

The wall cross-sectional design was initiated with a fundamental geometry and then modified to ensure compatibility with two distinct types of base anchorages. In the first case (P-BA3DW), the anchorages were made of prefabricated connections (i.e., six anchor bolts and column shoes supplied by Leviat) incorporated during the printing process; whereas in the second scenario (C-BA3DW), the connection was composed of traditional reinforcing bars protruding from the foundation's rigid plate with an anchorage length designed according to the regulations outlined in the Italian Structural Code [30]. It should be pointed out that this approach inevitably led to slight

modification in geometry of the wall thicknesses (350 mm for P-BA3DW and 410 mm for C-BA3DW, see Table 3.1), as the conventional anchorage system (C-BA3DW) required additional depth to guarantee the proper arrangement of the protruding rebars in accordance with Italian Structural Code provisions [30]. Conversely, the prefabricated anchoring system (P-BA3DW) could be integrated during printing, thereby enabling a more compact cross-section. While this difference introduces some minor variability in the comparison of mechanical performance, all resisting force values are normalized with respect to the cross-sectional area to mitigate its influence, and this limitation is explicitly discussed in Section 3.4.

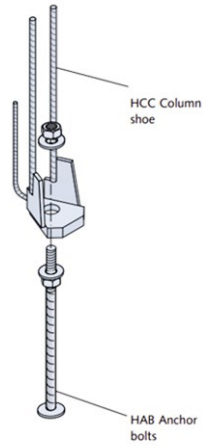
Figure 3.1(e) and (f) show the final cross-sectional dimensions, including the printing path layout for wall P-BA3DW and wall C-BA3DW, respectively. Both designs featured an optimized internal geometry, forming hollow prismatic patterns that reduced the total volume by approximately 40% compared to a fully solid wall. The Figure 3.1 also defines the axes used for calculating in-plane (y – y) and out-of-plane (x – x) moments of inertia. Table 3.1 provides a comprehensive overview of the main geometric characteristics of the two tested 3D-printed concrete wall panels. It includes detailed specifications of the unreinforced wall cross-sectional area, i.e. the wall depth (hereafter indicted as thickness, t_{w}), wall length (L_w), the cross-sectional area consisting of the solid layers ($A_{\text{Cross-section}}$), as well as the inertia calculated *i*) at the centroid of the cross-section in the longitudinal direction, denoted as in-plane ($I_{\text{in\_plane}}$), and *ii*) the inertia

calculated at the centroid of the cross-section in the transverse direction, known as out-of-plane ($I_{out\_of\_plane}$).

Specimen	L_w	th_w	$A_{Cross-section}$	I_{in_plane}	$I_{out\_of\_plane}$
-	[mm]	[mm]	[m ²]	[m ⁴]	[m ⁴]
P-BA3DW	1315	350	0.288	0.0453	0.0038
C-BA3DW	1315	410	0.314	0.0498	0.0058

Table 3.1.: Geometrical characteristics of the two 3D wall types investigated.

In both wall types, after the positioning on the foundation plate, a fully fixed base constraint was achieved by pouring a small amount of low-viscosity grout after curing (approximately 400 mm in height), to ensure proper bonding between the anchoring bars and the printed material, i.e., to create a monolithic connection between the wall and the foundation plate. To enable this operation, small openings were integrated into the design, as shown in red in Figure 3.1(c) and (d). The filling height was selected to ensure sufficient coverage of the rebars while minimizing interference with the hollow wall geometry above. The volume and height of the filling were controlled through reference lines inside the cavity and monitoring the volume of concrete introduced during the operation. This post-pouring procedure created a geometric discontinuity along the wall height—between the solid anchored base and the hollow upper section—referred to as the D-Section (Discontinuity Section), which will be further discussed in subsequent analyses.



(a)



(b)

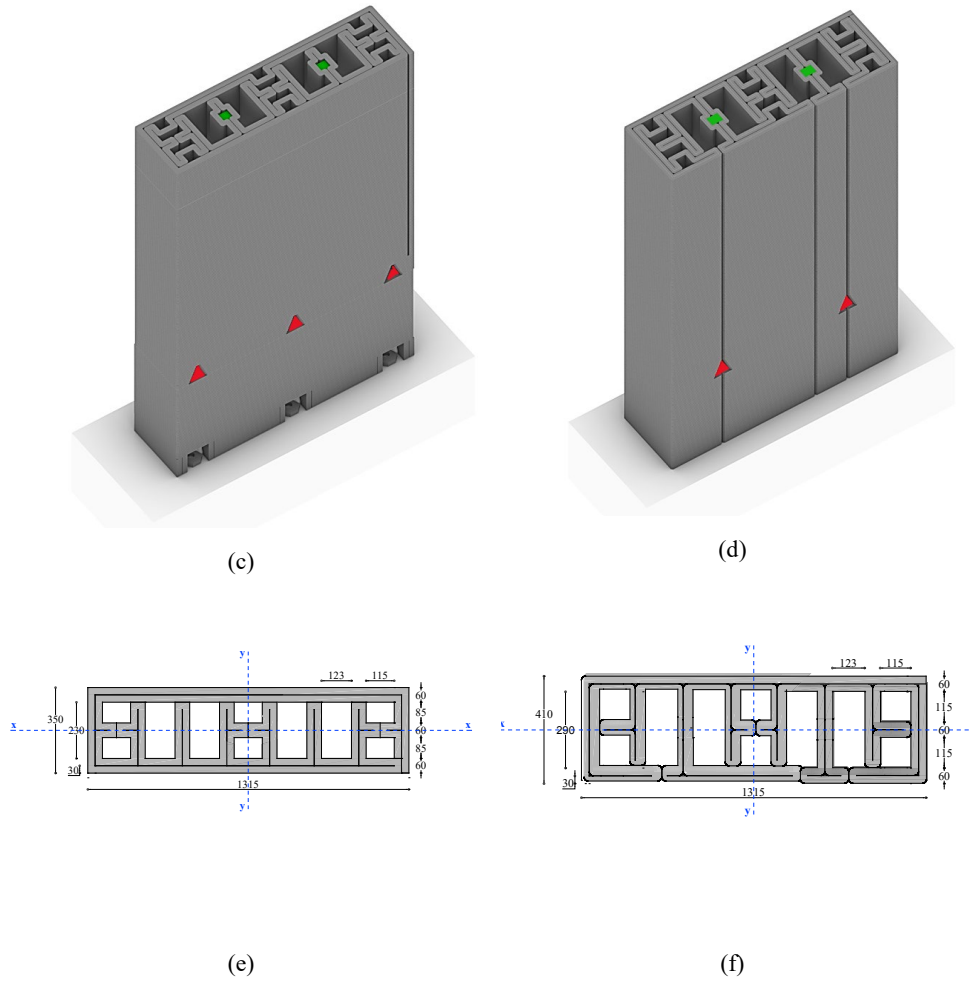


Figure 3.1.: Details of base anchorage for (a) P-BA3DW and (b) C-BA3DW wall samples including the placement phase, when the printed wall is suspended by lifting devices and being aligned onto the protruding rebars prior to final positioning; rendering of (c) P-BA3DW and (d) C-BA3DW walls - red marks indicate openings to facilitate pouring operations whereas green marks indicated the position of lifting devices; cross section details and dimensions (in millimetres) for (e) P-BA3DW and (f) C-BA3DW wall samples.

3.2.3 Experimental test setup and instrumentation

The experimental tests were carried out under displacement control on both 3DCP wall types to assess their response to cyclically acting

in-plane forces. A uniformly distributed vertical load was applied by means of two steel beams constrained to the base foundation and monitored through two load cells. The vertical load was aimed to replicate the potential weight of an overhead structure (roof); more specifically, the vertical load was set to 58 kN, corresponding to an average axial compressive stress acting on the wall of 0.20 MPa (also considering the self-weight of the steel beams of the experimental setup). A cyclic horizontal displacement was applied to the top side of the walls by means of a servo-hydraulic actuator (500 kN, Figure 3.2(a)) to reproduce the response of a lateral pseudo-static force acting in the plane of the 3D-printed concrete wall panels and centred at a height of 1.95 m. The cyclic lateral displacement was set at a rate of 0.01 mm/sec and grouped into three cycles for each predefined drift value (defined with letter D as the ratio between lateral displacement and height of wall, H_{wall}) with increasing increments between 0.1 and 1 mm. The cyclic protocol is showed in Figure 3.2(c). The final test setup and instrumentation are shown in Figure 3.2(a) and (b). A total of seven LVDTs, two linear potentiometers, and one laser device were used to monitor displacements. LVDTs were positioned based on expected crack patterns: two at the wall base to measure potential rocking, two across the D-Section to capture tension/compression, and three along the wall height to monitor in-plane deformation and potential sliding. The linear potentiometers tracked diagonal displacements, while the laser, aligned with the top LVDT, provided a redundant measurement of in-plane drift.

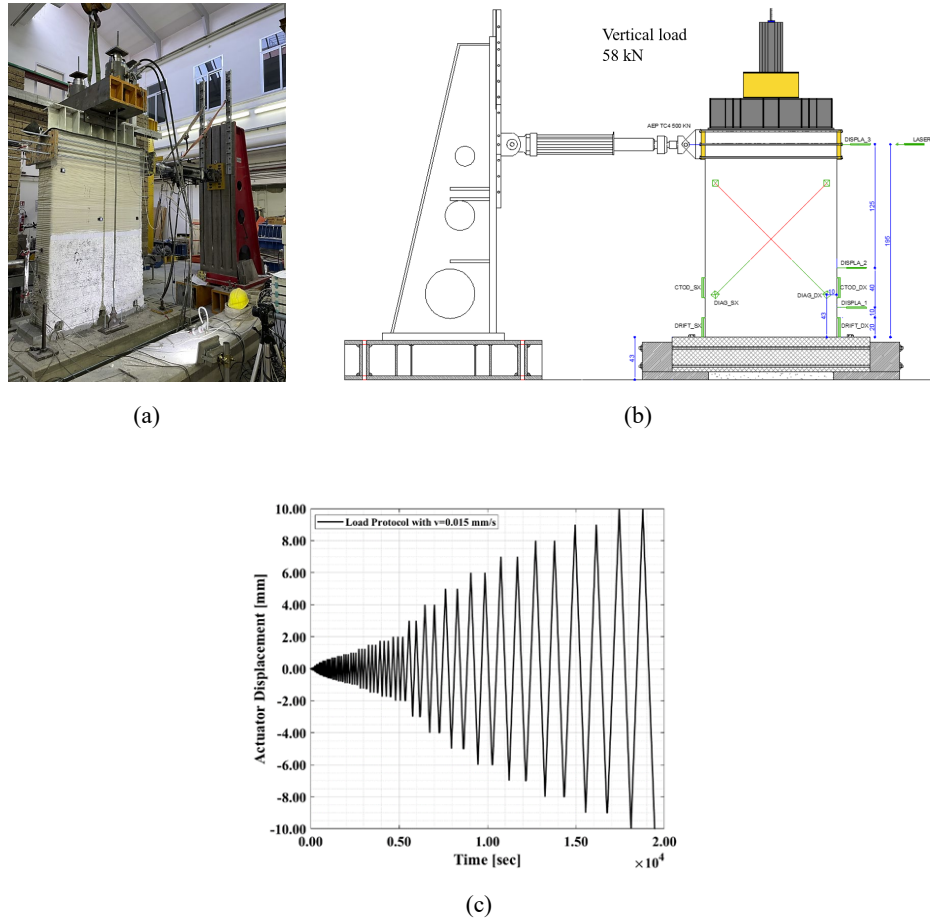


Figure 3.2.: Test setup (a); details of instrumentation (b); cyclic displacement testing protocol (c).

3.3 Experimental Results

Figure 3.3 presents the main experimental results from the in-plane cyclic tests, showing applied lateral displacement, corresponding drift and lateral force response for both wall types. The response of the P-BA3DW wall is shown in Figure 3.3(a), and that of the C-BA3DW wall Figure 3.3(b). Grey lines indicate individual cycles, while the black line

represents the envelope curve. Drift was calculated assuming a wall height of 1.95 m, corresponding to the actuator load point. The plots in Figure 3.3 were obtained by comparing the top displacement (LVDT DISPLA_3) and the applied force (from the actuator load cell, AEP TC4 500 kN), as shown in Figure 3.3(b). A sign convention is included to distinguish push (positive) and pull (negative) stages for both force (V) and displacement (δ_{TOP}). The curves reported in Figure 3.3 revealed that both wall panels exhibited a linear behaviour until the tensile cracking occurred at a specific height of the specimen. Subsequently, a limited post-cracking phase was observed, with a slight increase in load up to the attainment of a moderate sharp peak. This occurred in correspondence of lateral displacements less than 1.00 mm, as described in detail in the following sections. This post-cracking phase was followed by a softening behaviour, characterized by an increase of damage mainly consisting in the propagation of cracks along the weak interfaces of the printed walls. The lateral loading drop was almost rapid, indicating a brittle failure (especially for the prefabricated anchorage type) with no evidence of nonlinear/plastic softening as typically showed in masonry wall typologies. The differences within a specific set of cycles (i.e. with same applied drift) were negligible in the elastic regime. On the contrary, after the peak of load, remarkable losses of lateral capacity were recorded in the same range of applied lateral cyclic displacement. The tests were stopped for a load reduction of approximately of 40% compared to the peak value of the lateral force.

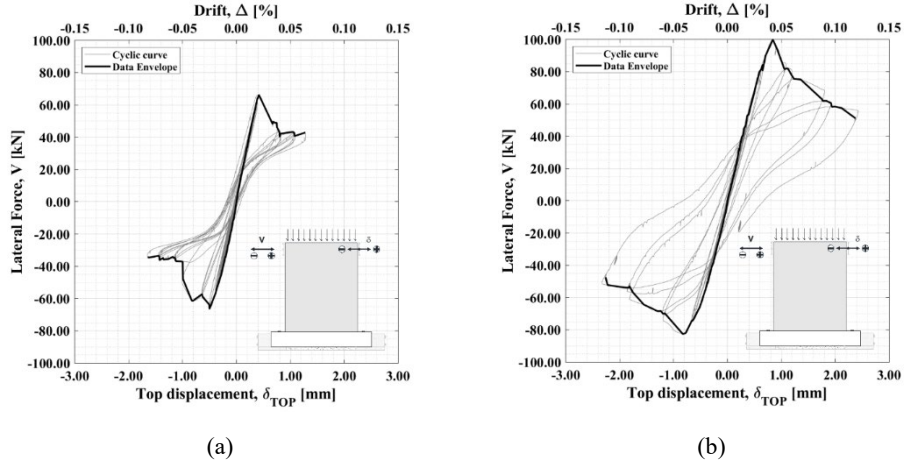


Figure 3.3.: Lateral Force vs Displacement (and drift) diagram for P-BA3DW (a) and C-BA3DW (b).

The failure behaviour was similar in both the pushing and pulling stages and for both wall types; however, the peak of lateral force (i.e., corresponding to the shear resistance of the walls) and corresponding displacement values were found to be different (as summarized in Table 2), likely due to variations in the cross-sectional area dimension and the type of anchorage at the base, as it will be further discussed in Section 3.4.1. A summary of the key data obtained from the cyclic tests carried out on each wall panel is provided in Table 3.2, including: the reference cross-sectional area across the discontinuity section (D-Section as shown in Figure 3.1), labelled as $Area_{D-Section}$; the values of maximum and minimum shear force, determined during the push and pull stages at the absolute value of the peak of the cyclic curves shown in Figure 3.3, denoted as $V_{y,PullStage}$ and $V_{y,PushStage}$; the top displacements values in pull and push stages at the yield limit (following the onset of damage and corresponding to the peak), labelled as $d_{y,PullStage}$ and $d_{y,PushStage}$, respectively; the secant stiffness

values in pull and push stages at the yield limit, labelled as $K_{sec, PullStage}$ and $K_{sec, PushStage}$, respectively; and finally, the drift (expressed as a percentage considering a wall height equal to 1.95 mt.), calculated at the yield limit recorded at the top during the pull and push stages, labelled as $Drift_{PullStage}$ and $Drift_{PushStage}$, respectively. Overall, it can be observed that the structural response of both panels was significantly rigid and stiff, with recorded values of lateral displacement at onset of damage and corresponding drift values which ranged in the order of magnitude of 1 mm and 0.03%, respectively.

Sp.	Area _{Section}	$V_{y, PullStage}$	$V_{y, PushStage}$	$d_{y, PullStage}$	$d_{y, PushStage}$	$K_{sec, PullStage}$	$K_{sec, PushStage}$	$Drift_{y, PullStage}$	$Drift_{y, PushStage}$
-	[m ²]	[kN]	[kN]	[mm]	[mm]	[kN/mm]	[kN/mm]	[%]	[%]
P-BA3DW	0.288	66.25	66.35	0.50	0.40	132.50	165.63	0.03	0.02
C-BA3DW	0.314	82.56	99.83	0.83	0.84	99.47	118.85	0.04	0.04

Table 3.2.: Main experimental outputs.

Considering classical failure mechanisms of panel structures under in plane loading, several measurements were analysed. To assess potential rocking effects, vertical displacements at the wall base were monitored using LVDTs (DRIFT_SX and DRIFT_DX, see Figure 3.2(b)). Rocking, typically associated with panel uplift under lateral loads, was not significant in either wall. As shown in Figure 3.4, base displacements remained below 0.05 mm up to peak load for both walls - an order of magnitude lower than the tensile/compressive displacements recorded at the D-section (Figure 3.5(a), (b)). This confirms that the fixed base constraint was effectively maintained for both anchorage types during testing.

To better understand rupture modes and reconstruct the crack pattern, displacements were monitored across the D-Section, the discontinuity section between the full and hollow portions of the wall—also the location of maximum bending moment under cantilever conditions. Both vertical and sliding displacements were recorded and plotted against lateral force. Vertical displacements supported the estimation of peak compression and tension levels across load cycles, while sliding displacements assessed potential shear movements along the interlayer plane near the anchorage zone.

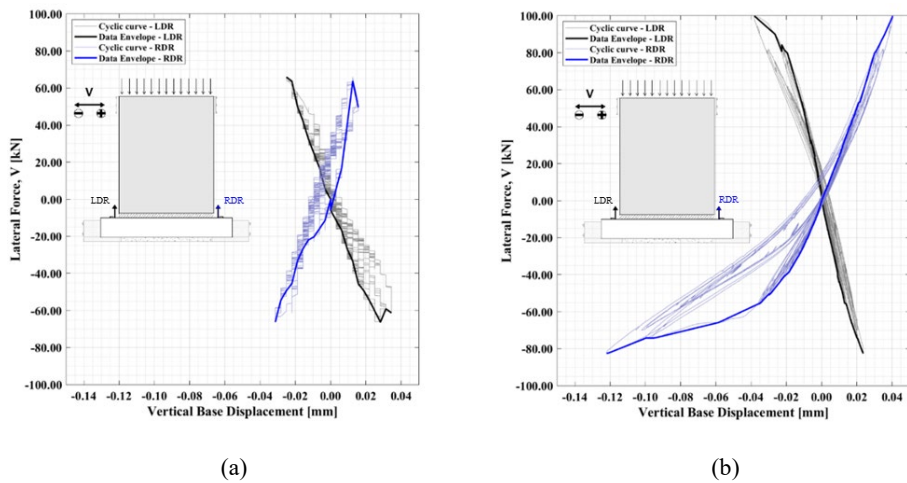
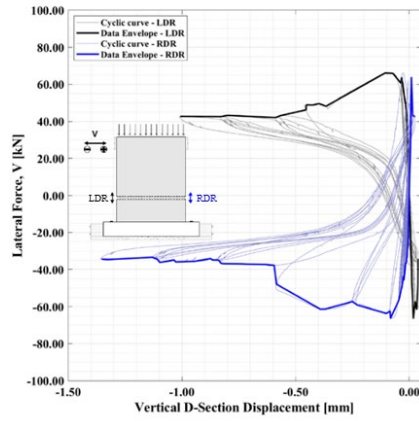


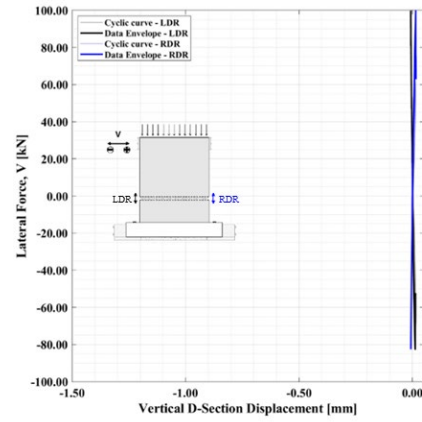
Figure 3.4: Force vs Vertical Base Displacement plot for P-BA3DW (a) and C-BA3DW (b).

In Figure 3.5(a) and (b), vertical displacements are plotted for both cyclic and as envelope actions (i.e., from CTOD_SX and CTOD_DX LVDTs as labelled in Figure 3.2(b)) for P-BA3DW and C-BA3DW wall types, where crack openings are recorded with a negative sign in the graphs. The relative sliding recorded by LVDTs (i.e., DISPLA_2 and DISPLA_3 in Figure 3.2 (b)) is also displayed for both P-BA3DW (Figure 3.5(c)) and C-BA3DW (Figure 3.5(d)) wall types. The analysis

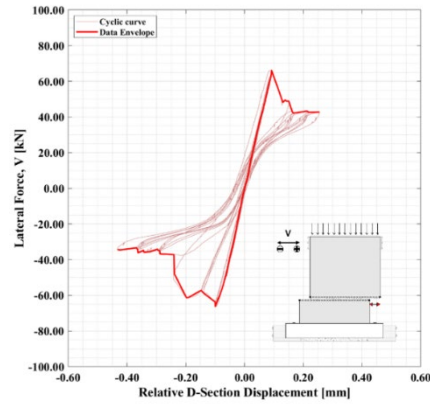
of Figure 3.5 highlights two kinematic mechanisms at the D-Section during cyclic loading: (i) vertical displacements from alternating tension/compression, and (ii) shear deformation within the section plane (i.e. corresponding to the interlayer plane of D-Section). In the P-BA3DW case, compression displacements remained minimal, while tensile displacements increased markedly at peak load (exceeding – 1 mm), indicating vertical crack opening at the most stressed edge. For C-BA3DW, both tensile and compressive displacements remained below 0.05 mm, likely because the main cracking zone developed outside the D-section, beyond the instrumented area—an issue discussed further in the next sections. Before major failure, sliding increased linearly and remained low (~0.2 mm). After peak load, it doubled in P-BA3DW, yet stayed below the corresponding vertical crack opening values. In C-BA3DW, sliding remained negligible due to the failure crack forming outside the instrumented region. The observed interaction between tensile, compressive, and shear deformations requires further analysis to better define failure modes and shear capacity, as discussed in the next sections.



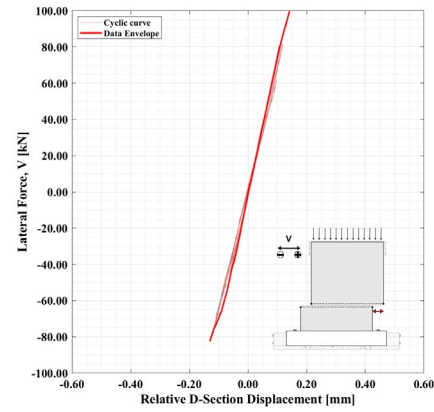
(a)



(b)



(c)



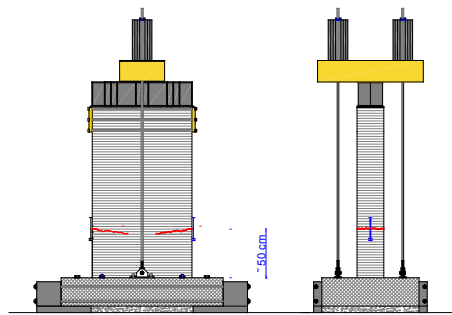
(d)

Figure 3.5: Recorded values in D-Section: Force vs Vertical D-Section Displacement plot for P-BA3DW (a); Force vs Vertical D-Section Displacement plot for C-BA3DW (b); Force vs Relative D-Section Displacement plot for P-BA3DW (c); Force vs Relative D-Section Displacement plot for C-BA3DW (d).

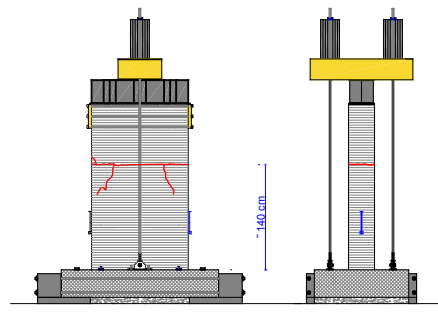
3.3.1 Failure Modes

Upon reaching ultimate failure, both walls exhibited complex cracking patterns, influenced by the 3D printing process and the type of base connection. Figure 3.6 highlights the final crack lines (in red) overlaid

on the printed layer structure. Specifically, Figure 3.6 (a) and (c) refer to the wall with prefabricated connections (P-BA3DW), while Figure 3.6 (b) Figure 3.5 (d) show the cracks in the wall with conventional anchorage (C-BA3DW). In terms of overall damage, regardless of the type of connection, the major failure crack was singular and occurred at a specific height along the wall, with an orientation predominantly parallel to the deposition layer plane (see also after the post-mortem removal of the upper part, Figure 3.6 (e) and (f)). Regarding crack propagation, in both cases, the initial cracks appeared in the outermost zones - where tensile stress was highest due to the bending moment - and subsequently propagated rapidly towards the centre of the wall specimens as damage progressed.



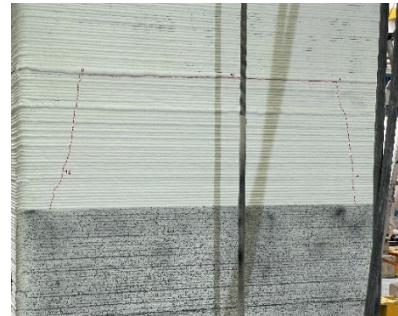
(a)



(b)



(c)



(d)



(e)



(f)

Figure 3.6.: Crack patterns from tests - Sketch of the crack lines drawn in red for P-BA3DW (a) and C-BA3DW (b) walls; photographic recording of the cracks on the wall and after upper part removal for P-BA3DW (c)-(e) and C-BA3DW (d)-(f) walls.

In one case (wall labelled P-BA3DW), the initial cracks in the maximum tensile zone were observed at an elevation of approximately 50 cm from the base, near the change in cross-section (D-section), triggered by high tensile forces at that height. In the second case (wall labelled C-BA3DW), the initial cracks appeared at an elevation of approximately 135 cm from the base. These cracks were also caused by high tensile forces at that height but exhibited a more rapid propagation along the deposition plane of the corresponding interface layer (i.e., approximately where internal shear distribution is maximum). To assess crack development, LVDT data from the D-section were analysed and plotted in Figure 3.7 for both push and pull stages. Due to the positioning of the instruments (Figure 3.2(b)), only the P-BA3DW wall allowed for detailed tracking of damage propagation, as the main crack passed through key sensors (CTOD_DX, CTOD_SX, DISPLA_1, DISPLA_2). Figure 3.7 (a) shows the relationship between relative lateral displacement (slippage) and vertical crack opening on the left side during the push stage, while Figure 3.7 (b) shows the same for the right side during the pull stage.

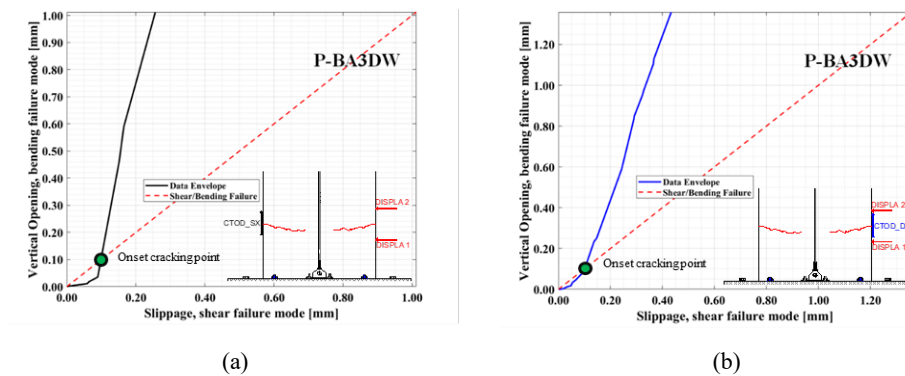


Figure 3.7.: Damage Description - Vertical Opening vs Slippage in the D-Section during the Push (a) and Pull (b) stages.

The onset of cracking occurred during both the push and pull stages at approximately 0.1 mm, as indicated by the values for vertical crack opening and sliding movement (i.e., slippage measured at the layer interface plane of the D-cross section), highlighted by the green dot in Figure 3.7 (a) and (b). This behaviour is confirmed by the displacement pattern prior to crack initiation, where the displacements exhibit a nearly linear relationship. At approximately 0.1 mm, the vertical crack opening became significantly larger than the horizontal relative displacement of the wall at the D-cross section, and this trend persisted throughout the cyclic test. From these observations, it can be assumed that the onset of damage was primarily induced by bending mode, driven by tensile failure at the most stressed edge of the wall. This is further exemplified in the same Figure 3.7 with a dashed red line representing the quadrant middle line, which plausibly indicates an intermediate (balanced) bending-shear failure mode, from which vertical displacement diverges after the initiation of damage. Data points closer to the vertical axis in the graphs of Figure 3.7 suggest a failure dominated by bending (traction side). On the other hand, data points closer to the horizontal axis indicate a failure primarily influenced by shear forces. Based on this consideration, both in the push and pull phases, the mechanism that triggered and propagated the cracks in the P-BA3DW wall was traction-bending driven failure mode.

Following the onset of damage caused by bending, the crack propagation observed in the subsequent cycles in both wall types showed no evidence of significant compression strain/damage

concentration on the opposite side of the walls (as typically happens in unreinforced masonry walls failing under in plane bending). Instead, a rapid propagation of the crack occurred in a direction parallel to the deposition plane of the 3D-printed layers. However, a key difference was observed between the two wall configurations. For the wall with prefabricated connections, the crack did not cross the entire plane of the wall. Instead, it created a hinge-like mechanism at the centre, where one of the three the metallic connection elements were located. Conversely, for the wall with conventional connections using rebar dowels, the crack extended across the entire wall, remaining nearly parallel to the deposition plane of the layers. This resulted in a very rapid loss of load-bearing capacity. This behaviour is notably different from that of traditional unreinforced masonry walls. In such walls, after the initial bending cracks, compressive forces typically concentrate on the opposite side, or a rocking mechanism is activated. In contrast, the rapid crack propagation along the printed layer interface in the 3D-printed walls highlights a distinct failure mechanism influenced by the layered fabrication process. Very ultimate compression failure was plausibly responsible for vertical cracks showed in Figure 3.6 (b), after that interface disconnection (horizontal cracks) reduced the area of compression during final cycles.

Compared to conventional reinforced concrete (RC) shear walls, the observed cracking behaviour in the 3D-printed concrete walls shows notable differences. In RC walls, cyclic lateral loading typically results in distributed flexural and inclined shear cracking along the entire height of the wall, as the tensile strength of the concrete is exceeded

and the longitudinal and transverse reinforcement progressively yields. This distributed cracking pattern enables energy dissipation and a gradual reduction of stiffness and strength, resulting in a ductile failure mode [31]. In contrast, the unreinforced 3D-printed walls tested here exhibited a single localized tensile crack, propagating along the interlayer plane, without the confining and bridging effect of reinforcement.

This behaviour draws the attention to the inherently brittle nature of unreinforced 3D-printed concrete and highlights the critical role of process parameters (interlayer time, layer orientation, internal voids) in defining failure mechanisms. Moreover, it should be noted that the different locations of shear slippage and crack initiation (i.e., the major crack) observed in the two specimens are not solely attributable to the type of base anchorage. Although the anchorage configuration plays a key role in governing stiffness distribution and stress concentrations along the wall height, the layered nature of the 3D printing process introduces scale-dependent effects and local imperfections that can significantly affect performance. Indeed, slight variations in interlayer time, temperature, and surface moisture during printing can reduce local cohesion and friction between layers, generating planes of weakness that increase the variability of slippage initiation compared to conventional monolithic systems. Additional weaknesses may arise from unintended printing interruptions, insufficient surface reactivation, or partially integrated embedded elements (e.g., connectors or sensors) disrupting the continuity of the printed matrix. These imperfections can act as stress concentrators by locally reducing

tensile/shear strengths, promoting early crack initiation and modifying stress redistribution, thus influencing the transition from flexural to mixed-mode - and eventually to shear-sliding failure mechanisms. In this regard, while detailed microstructural analysis was beyond the scope of this study, future research employing advanced techniques such as X-ray computed tomography, digital image correlation, or scanning electron microscopy is encouraged to quantify the influence of interlayer defects, especially at structural scale, and to support the refinement of predictive models for 3D-printed components.

3.4 Discussion

3.4.1 Effect of base anchorage typology

Figure 3.3(a) and (b) illustrated the cyclic force-displacement behaviour of the 3D-printed walls, revealing differences in stiffness and load capacity linked to wall geometry and base connection type. Figure 3.8 further compares the in-plane lateral stiffness, calculated from the elastic slope of the force-displacement curves (push: red points, pull: blue points) and normalized by the net cross-sectional area. The normalization was applied to enable a direct comparison between wall specimens, emphasizing the relative influence of anchorage. While the second moment of area would be relevant for stiffness assessment (see detailed analysis in Sections 3.4.3 and 3.4.4), the chosen approach offers a more intuitive representation of different global behaviour. Notably, in Figure 3.8(a), the corresponding experimental stiffness points for both the push and pull stages lie to

the right of the quadrant's midpoint line (indicated in green), demonstrating that the P-BA3DW wall exhibits higher overall stiffness compared to the C-BA3DW wall. This observation is further supported by the normalized cyclic force-displacement comparison graph, shown in Figure 3.8(b), which shows the higher stiffness of the P-BA3DW wall when adjusted for the net cross-sectional area and before reaching the onset of damage.

The type of base anchorage reasonably influenced the structural behaviour of the 3D printed walls, as it locally modifies the degree of constraint at the base, independent of the net cross-sectional area. In the case of prefabricated connections with relatively large, embedded steel elements (e.g., P-BA3DW), the connectors enhanced the local stiffness but also introduced stress concentrations in the discontinuity section between the solid base and the hollow printed wall. This stiffer base restraint plausibly altered the stress distribution along the wall height, limiting flexural deformations and promoting early tensile stress concentration near the D-section. Such localized effects contributed to premature cracking at the wall extremities, particularly around the embedded connectors (Figure 3.6(a)). Conversely, when conventional base anchorage with uniformly distributed bars is used, as in C-BA3DW, stress distribution is more uniform, reducing localized stress peaks and resulting in a more consistent shear stiffness. However, this configuration can make the damage location more arbitrary (and wider) along the height of the wall, as it may depend on imperfections such as weak interfaces between layers. In this case, stress concentration shifted to a higher elevation, closer to the layer

interfaces (Figure 3.6(b)). Moreover, during propagation, the presence of prefabricated anchorage in the middle of the wall, reduced the risk of instantaneous shear crack along the entire cross section, towards a more “hinge” like behaviour around it.

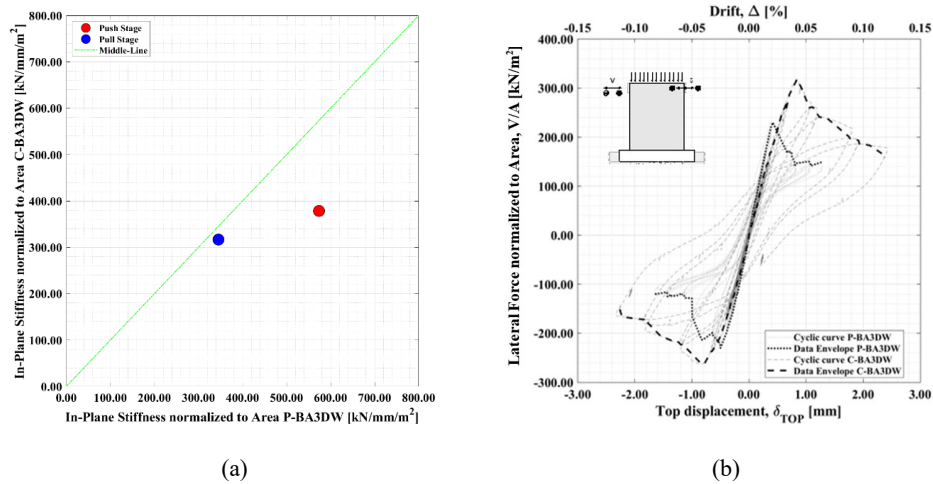
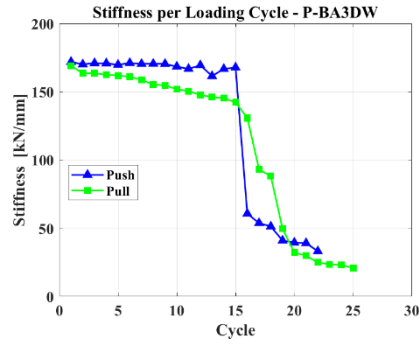


Figure 3.8: Comparison of the normalized in-plane stiffness (a) including experimental stiffness data – blue and red point for pull and push stages, respectively –; Comparison of normalized Lateral Force vs. displacement curves (b).

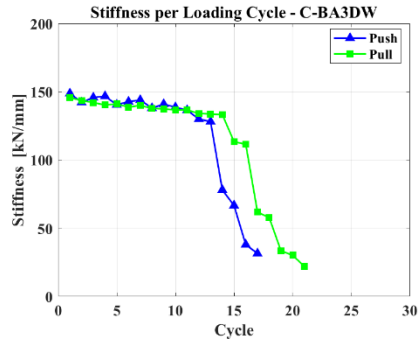
Finally, the cyclic degradation of secant stiffness and the evolution of dissipated energy per cycle for the two wall specimens are illustrated in Figure 3.9. The Figure 3.9 comprises four subplots: (a) and (b) show the secant stiffness per cycle for P-BA3DW and C-BA3DW, respectively, while Figure 3.9 (c) and (d) display the corresponding dissipated energy (area computed underneath each cycle of the Force-displacement curve). Each plot includes both the push (blue) and pull (green) loading phases, enabling a direct comparison of the cyclic response in both directions. The trends observed in the two directions are highly consistent, indicating a symmetric and repeatable mechanical behaviour under reversed cyclic loading. In the early

cycles, both wall specimens exhibit a predominantly elastic response, characterized by an almost constant stiffness for increasing cycles (Figure 3.9 (a) and (b)) and minimal energy dissipation for cycle (Figure 3.9 (c) and (d)). With increasing displacement amplitude, the response shifts into a brittle failure regime, clearly marked by a sudden degradation in stiffness and a sharp rise in dissipated energy during the final cycles. This transition occurs almost simultaneously in both push and pull directions, confirming the prevalence of a brittle failure mechanism with limited ductility and poor post-peak dissipation capacity.

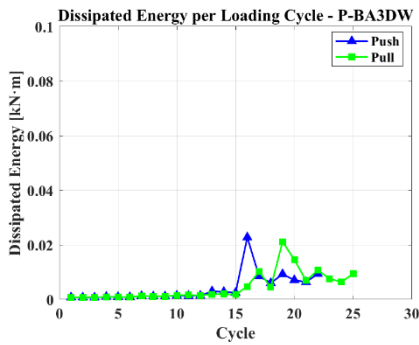
While both walls show a rapid post-peak reduction in stiffness (between cycles 15 and 25), the C-BA3DW specimen appears to undergo a slightly less brittle degradation in stiffness (Figure 3.9(a) and (b)) and higher dissipated energy. This behaviour may be related to the role of frictional effects between adjacent layers after onset of damage which involved a larger area compared to the P-BA3DW wall, which locally redistribute stresses in the discontinuity section (D-section) and around the prefabricated steel anchorage elements. Overall, these results support the interpretation derived from hysteretic curves and crack propagation patterns: both wall systems are governed by interlayer-driven brittle failure modes, combining tensile rupture and interface sliding.



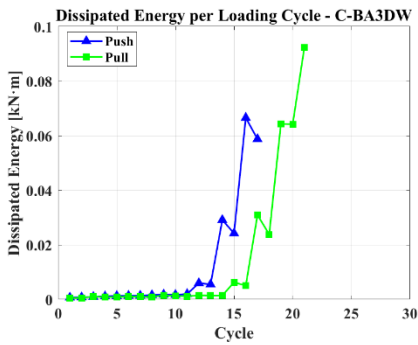
(a)



(b)



(c)



(d)

Figure 3.9: Stiffness per Cycle for P-BA3DW (a); Stiffness per Cycle for C-BA3DW (b); Energy Dissipation per Cycle for P-BA3DW (c); Energy Dissipation per Cycle for C-BA3DW (d).

3.4.2 Analytical Interpretation

Based on the cyclic test results, both wall types exhibited failure initiated by reaching the tensile strength limit at their most stressed edges. The force–displacement curves (Figure 3.3(a) and (b)) revealed a three-stage cyclic response during both push and pull phases, with the first two stages occurring at similar load levels. This behaviour is schematically illustrated in the trilinear curve of Figure 3.10(a), and can be summarized as follows:

- The first stage (blue line) represents the undamaged linear elastic state until the onset of damage, primarily characterized by pure flexural/tensile cracking (i.e., vertical opening) occurring at a single layer interface plane.
- The second stage (orange line) represents the propagation of the damage occurred at the onset, which can be dominated by shear/sliding at the layer interface due to the mostly planar direction observed during the experiments. The propagation is characterized by the loss of linearity and reasonably ends when a large portion of the layer interface suddenly breaks under shear stress component, indicating the final of yielding stage. In this stage and for determining the design peak load, the interface properties play a major role and deserve careful attention in the experimental evaluation at the material scale.
- The third stage (yellow-red line) illustrates the ultimate state, characterized by softening and dominated by shear frictional behaviour at the interfaces, potentially leading to ultimate local compressive failure in certain slender wall elements/parts of the cross-section, thus representing the very ultimate state (as it can be observed in the final failure pattern of C-BA3DW in correspondence of the vertical cracks at the wall edges where compressive stress localized).

Figure 3.10(a) is based on the evidence of failure mechanisms of the tested 3D-printed concrete walls and can be interpreted using fracture mechanics and bonding theory. Interlayer bonding—critical for load

transfer between printed layers—is strongly influenced by the interlayer time during deposition [32]. Adopting a fracture mechanics framework [33], the adhesive interfaces can be modelled to reflect key fracture modes:

- *Elastic Stage* (Blue): Failure in Mode I (opening), with tensile stresses causing delamination perpendicular to the printed layers.
- *Yielding Stage* (Orange): Mixed Mode failure (Modes I and II), driven by combined tensile and shear stresses, resulting in both opening and sliding between layers.
- *Ultimate Stage* (Red): Dominated by Mode II (sliding), where relative displacement occurs along the layer plane, potentially followed by localized crushing.

Recognizing these rupture modes is essential to improve interlayer bonding and inform design strategies for 3D-printed concrete structures. Histograms in Figure 3.10 (b) and (c) summarize the experimental outcomes in terms of evolution of shear force (V) and top displacement (δ) across the three stages—elastic (blue), yielding (orange), and ultimate (red)—for both push and pull actions. Overall, the response in terms of push and pull forces shows slight differences between the two walls across the three stages of cyclic behavior. The elastic phase ends at shear force values between 54 kN and 83 kN for both wall types, with slightly higher values observed for the conventional anchorage (C-BA3DW). The peak force recorded at the end of Stage 2 (yielding phase) is higher for the distributed anchorage

(C-BA3DW), with values reaching approximately 83 kN for P-BA3DW and 99 kN for C-BA3DW. In the ultimate phase, the multiple load cycles oscillate between 30 kN and 45 kN, reflecting a significant reduction in load-bearing capacity due to progressive damage and softening. The global displacement response (Figure 3.10(c)) reveals a very low elastic limit, below 1 mm, with the prefabricated anchorage (P-BA3DW) exhibiting slightly smaller displacements (~0.4 mm) likely due to localized stiffening effects from the metallic connectors. The displacement at peak force is only marginally higher than the elastic limit, indicating limited deformation capacity during the yielding phase and sudden fracture at the layer interface. The ultimate displacement capacity is slightly above 2 mm, marking the point at which the structural behavior transitions to complete failure due to accumulated damage and loss of stiffness.

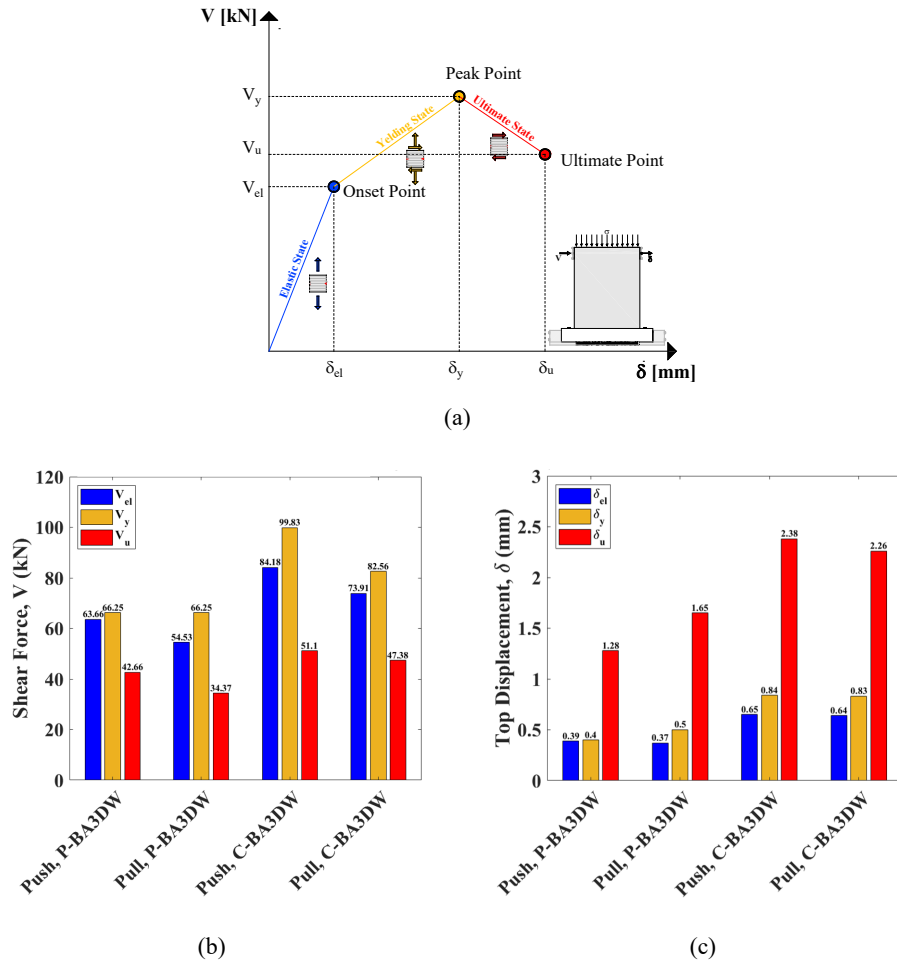


Figure 3.10: Schematic representation of the Lateral Force-displacement curve (a); histogram comparing the recorded shear forces in different stages (b); histogram comparing the lateral displacements in different stages (c).

3.4.3 Resistant mechanisms and analytical interpretation

Since damage initiation in both walls (P-BA3DW and C-BA3DW) was primarily driven by flexural behavior and Mode I tensile rupture at the edges, a detailed stress analysis was carried out at the most critical section—the D-section—at the end of the elastic phase. Using LVDT data capturing tensile and compressive displacements at the wall

edges, internal stress levels were reconstructed to estimate the elastic limit, i.e., the stress state at crack onset. This analysis offered valuable insights into the link between external loads and internal stress distributions responsible for initial damage under cyclic loading.

To this scope, the central core of inertia - kern area - [34] was calculated and represented for the P-BA3DW wall, as shown in Figure 3.11(a), and for the C-BA3DW wall, as shown in Figure 3.11(b). Then, the eccentricity of the applied external loads was computed in both the push and pull phases for both walls, by considering the recorded experimental lateral shear force at the elastic limit (V_{el} – see Table 3.3), the applied axial (constant) stress including the self-weight of the wall and setup (N_{ext}), and the critical height corresponding to the distance between the point of force application and the D-section ($h_{D-section}$), used for calculating the induced bending moment. The eccentricities, e_{push} and e_{pull} , were calculated as the ratio between the external applied moment, M_{el} (equal to V_{el} multiplied by $h_{D-section}$ in each case), and the applied external normal stress, N_{ext} . Table 3.3 summarizes the values of eccentricities calculated for both tested walls in both the push and pull phases. It can be observed that the eccentricity exceeded the edges of the central core of inertia, highlighted in Figure 3.11, in both wall type cases, making the D-section reactive both in traction and compression.

Specimen	N_{ext}	$V_{el,push}$	$V_{el,pull}$	$h_{D-Section}$	e_{push}	e_{pull}
-	[kN]	[mm]	[mm]	[mm]	[mm]	[mm]
P-BA3DW	58.00	63.66	54.53	1600.00	1756.16	1504.33
C-BA3DW	58.00	84.18	73.91	1600.00	2322.23	2038.95

Table 3.3.: Loading actions at the elastic limit and characteristic values employed for the computation of eccentricity.

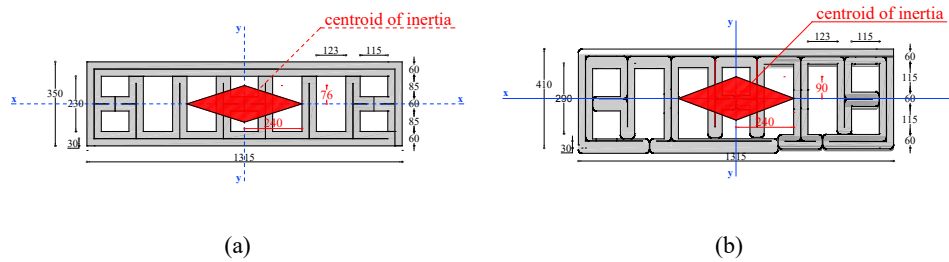


Figure 3.11: Kern of area (in red) at the location of the D-section for the P-BA3DW and C-BA3DW wall types (b).

With a classical Navier approach [35] and within elastic assumptions, it was possible to calculate the maximum compression, σ_c , and tension stresses, σ_t , acting at the edges of the D-section for both wall types at the onset of damage, as reported in the Eq. 3.1 and Eq. 3.2 – Where I_G represents the moment of inertia calculated with respect to the centroidal axis, M_{el} is the product of the elastic limit shear force, V_{el} , and the height of the section relative to the D-section ($h_{D-section}$), as specified in Table 3.3. In addition, L_w denotes the base of the section, corresponding to the longer side, as highlighted in Table 3.1. Lastly, $A_{D-section}$ represents the cross-sectional area, as reported in Table 3.2. The neutral axis in the elastic stage is the one where the equilibrium between the applied external normal stress, N_{ext} , and the forces

resulting from the linear distribution of the compression and traction stress diagrams, N_t and N_c , respectively, is maintained.

$$\sigma_c = \frac{N_{ext}}{A_{d-section}} + \frac{M_{el}}{I_G} \cdot \frac{L_w}{2} \quad \text{Eq. 3.1}$$

$$-\sigma_t = \frac{N_{ext}}{A_{d-section}} - \frac{M_{el}}{I_G} \cdot \frac{L_w}{2} \quad \text{Eq. 3.2}$$

In light of this, it was possible to determine the neutral axis. In Figure 3.12, the positions and values of the neutral axis, y_c , at the end of the elastic stage are represented for each type of wall and for both pull and push stages of the applied cyclic load, with the compressed zone indicated with a red line. Figure 3.13 shows the computed stress diagrams at the equilibrium for the elastic limit, including the values and positions of the resultant normal forces in the compressed zone (N_c) and the zone under traction stresses (N_t). Specifically, Figure 3.13 (a) and (b) depict the balanced stress diagrams and the resultant compression and tension forces during the push and pull stages, respectively, for the P-BA3DW wall. Figure 3.13 (c) and (d) depict the same information but for the C-BA3DW wall type. It is worth noting that in the Figure 3.13, the compressed zone is represented in red, while the tensile zone is represented in blue.

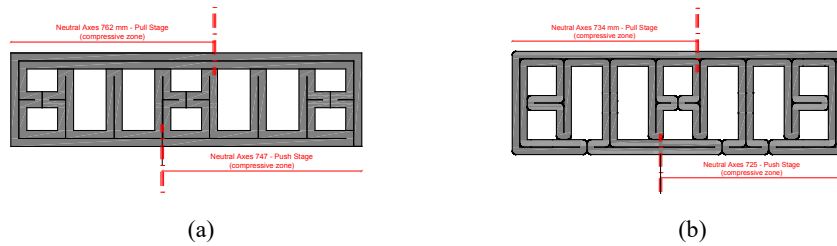


Figure 3.12: Neutral axes location for P-BA3DW (a) and C-BA3DW (b) wall panels at elastic limit.

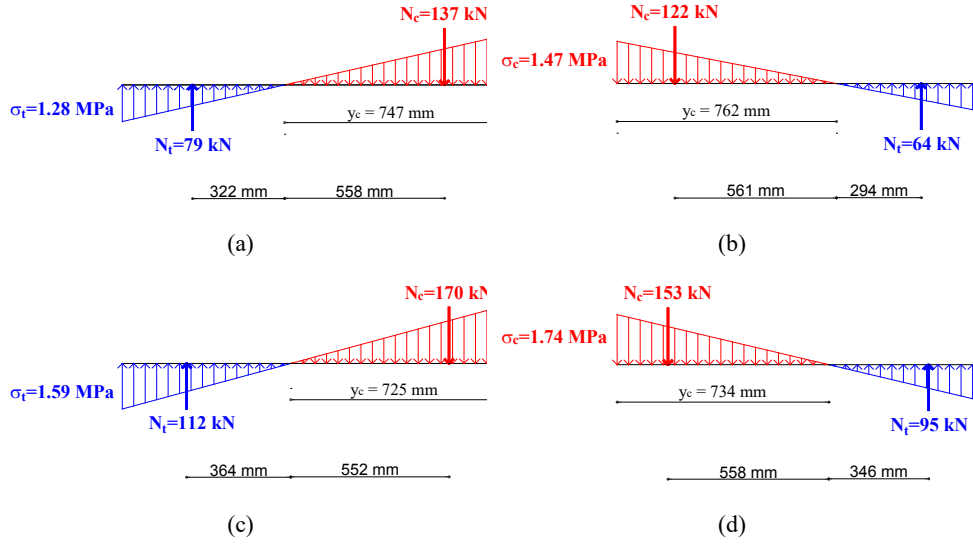


Figure 3.13: Balanced stress diagram in the push (a) and pull stage (b) for P-BA3DW; balanced stress diagram in the push (c) and pull stage (d) for C-BA3DW.

Based on the elastic stress distributions at the critical cross-sections of the two wall types at the elastic limit, the tensile stress levels causing the initial cracking appeared compatible with the values obtained with direct tensile (pull-off, rather than flexural strengths) tests carried out on the 3d printed concrete herein investigated, ranging between 1.5 and 2 MPa, even if a scale effect was possible ([36]). Indeed, this aligns with values reported in other experimental studies on direct tensile strength of 3D-printed concrete [37]. Similarly, at the same elastic limit, compressive stress levels are relatively low, around 1.5 MPa, as shown in Figure 3.13. After exceeding the elastic limit, the rupture following yielding (i.e., at the force peak in the force-displacement/drift diagrams) is primarily driven by rapid crack propagation along adjacent layers in the same fracture plane. Given the very close elastic and peak displacements summarized in Figure 3.10(b) and (c), it is reasonable to conclude that compressive failure

of the 3d printed material is never reached as primary failure mode, as compressive strength values are significantly higher (~40 MPa) than the observed stress levels. Indeed, even during failure, the compressive stress remains low (i.e., no increase until crushing), contrasting with traditional unreinforced masonry, where failure often occurs under combined bending and compression (e.g., compressive stresses typically reach maximum 3–15 MPa in masonry walls before failure in flexural-compression mechanisms). This distinction highlights the different failure modes in the in-plane behavior of 3D-printed concrete walls, governed predominantly by tensile and shear behavior at the layer interfaces rather than compressive crushing (being used in traditional calculations for unreinforced masonry). However, different applied load levels (e.g., higher axial load) or stronger/weaker layer interface at the interlayers, could improve/worsen this condition, respectively.

By considering the experimental displacement values at the elastic limit for both walls (shear force V_{el}) according to the graphs in Figure 3.5, it was possible to determine the strains and validate the analytically calculated values. From those measurements and for P-BA3DW, the strain values retrieved were approximately 0.011% and 0.008%, in compression and tension, respectively. For the C-BA3DW specimen, the recorded deformations were 0.013% in compression and 0.011% in tension. These values are consistent with those obtained analytically. In fact, considering an elastic modulus of approximately 15 GPa, a reverse analysis confirms that the calculated stresses are substantially consistent with the experimental ones.

The peak response shown in Figure 3.10(a) reflects a complex mixed-mode fracture which arises from interactions between tensile, shear, and interfacial adhesion mechanisms at the layer interfaces. This failure mode is strongly influenced by the geometry and mechanical properties at the sub-assembly scale, requiring further experimental validation. At this stage, tensile strength (between adjacent layers) is key for crack initiation (Mode I), while shear strength governs sliding failures (Mode II). The quality of interfacial bonding between layers also plays a critical role, particularly when cracks propagate along or across the printed interfaces.

Based on the discussion above and to analyse the ultimate shear strength of the tested (unreinforced) 3D-printed concrete walls, it was reasonable to draw parallels with the failure mechanisms observed in unreinforced masonry walls subjected to in-plane actions, as described in Eurocode 6 [38] and the Italian Structural Code [30]. In unreinforced masonry, the primary failure mechanisms under in-plane loads are well-documented in the literature, and they include:

- *Flexural Compression Failure* (Figure 3.14(a)): This mechanism occurs when compressive stresses reach the material compressive strength, typically at the most compressed edge of the wall. In 3D-printed concrete walls, this failure mode is less likely due to the relatively low compressive stress levels observed and very high strength of the material, as discussed earlier.
- *Diagonal Cracking* (Figure 3.14(b)): Diagonal cracks arise when tensile stresses exceed the material tensile strength along the

principal tensile direction. In 3D-printed walls, this mechanism may also involve interfacial delamination in the direction perpendicular to the weak layers due to weak adhesion or imperfections between layers but with a different propagation pattern (i.e., not following principal tensile direction inclined at 45 degrees).

- *Shear Sliding Failure* (Figure 3.14(c)): This mechanism is characterized by sliding along a defined plane, governed by the material shear strength and influenced by the cohesion and frictional properties at the interfaces of the masonry or brick/mortar elements. In 3D-printed walls, shear sliding is particularly relevant due to the layered nature of the material, where shear stresses concentrate at the interfaces, but may involve the interaction with direct tensile strength at the interlayer for damage initiation as observed during the described experimental results.

The comparison to masonry failure mechanisms provides a valuable framework for interpreting the behaviour of 3D-printed walls. However, the unique characteristics of 3D printing - such as anisotropy, weak interlayer bonding, and geometric flexibility - necessitate further tailored criteria to fully capture the mixed-mode failure processes and their dependency on both geometric and material properties at different scales. For comparison purposes, it is worth noting that experimental tests on unreinforced brick masonry walls of comparable geometry (i.e., approximately 1–1.5 m in length and 2–3 m in height) conducted by Magenes and Calvi [39] reported initial secant stiffness

values generally ranging between 50 and 70 kN/mm under similar levels of applied axial load. In contrast, the 3D-printed concrete walls investigated in this work exhibited higher initial stiffness values in the order of 100–160 kN/mm. This difference can be attributed both to the higher elastic modulus of the printed material (approximately 15–20 GPa) and to the monolithic nature of the printed assembly, which—despite potential interlayer imperfections—provides higher overall cohesion compared to discrete brick–mortar (horizontal and vertical) interfaces typical of traditional masonry. Nevertheless, it is important to emphasize that stiffness is strongly dependent on specimen geometry, slenderness ratio, and boundary conditions (e.g., fixed-fixed vs. cantilever support), and the comparison provided herein is intended as a reference benchmark rather than a direct equivalence.

In terms of deformation capacity, unreinforced masonry walls of similar dimensions tested by Magenes and Calvi [39] typically exhibited ultimate drift capacities on the order of 0.5–0.6%, reflecting the combined contributions of bed joint sliding and distributed cracking mechanisms. By contrast, the 3D-printed walls in this study demonstrated markedly lower ultimate drift values, generally below 0.02 – 0.04%. This confirms that unreinforced 3D-printed concrete exhibits a distinctly brittle post-peak behaviour. Unlike traditional brick–mortar masonry, which can accommodate deformations through progressive joint sliding, frictional energy dissipation, and gradual microcracking, the printed interfaces tend to fail suddenly with limited warning and little capacity for energy absorption after peak load.

This distinction is particularly evident in the failure mechanisms observed under flexural loading. Unlike traditional unreinforced masonry, where flexural failure often involves stable rocking and localized/progressive toe crushing under vertical loads [39], the 3D-printed walls in this study exhibited brittle tensile rupture once stresses perpendicular to the layer plane exceeded the interfacial strength. This led to sudden crack propagation rather than progressive compression or ductile rocking. The resulting hysteresis loops show pronounced pinching and sudden stiffness loss, distinct from the stable cyclic response typical of masonry. Moreover, in the absence of reinforcement, no post-cracking bridging action was possible, further confirming that failure was governed by brittle tension along the printed interfaces rather than rocking or ductile flexural yielding.

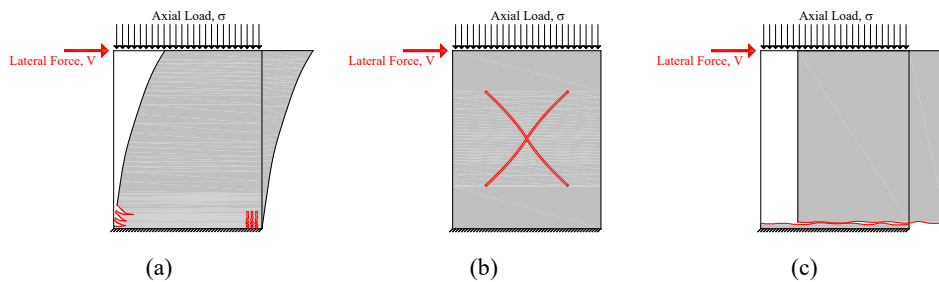


Figure 3.14.: In-Plane failure mechanism in unreinforced masonry due to flexural compression (a); diagonal cracking (b); shear sliding (c).

3.4.4 Interpretation according to available codes

Based on the considerations above, we provided also a numerical interpretation of the observed failure modes in relation to the corresponding peak of lateral forces recorded. In accordance with the Italian Structural Code [30] and Eurocode [38], in-plane flexural

compression failure of unreinforced masonry occurs upon reaching (i) the design compressive strength of the masonry wall, f_{cd} ; (ii) diagonal cracking failure which is triggered upon reaching the design tensile strength in the panel, f_{td} ; and (iii) the shear sliding failure which is initiated upon reaching the design shear strength in the panel, f_{vd0} [21,30] – where subscript d indicates design values. Therefore, applying those design failure criteria to the wall cases investigated and considering the mechanical characteristics at the 3d printed material scale - in the direction orthogonal to the layers, as specified in the Materials and Methods section - it was possible to predict the ultimate in-plane resistance of the wall in the various mechanisms described earlier. Specifically, the strength values for compression (f_{ck}), tension (f_{tk}), and shear (f_{vk0}) were obtained from experimental tests conducted by the authors (according to UNI EN 196-I [21] and ASTM E519-07 [41]) and reported in a different work [42]. It is worth noting that the tensile strength was indirectly determined through a three-point bending test that generates tensile stresses in the direction perpendicular to the layer deposition (U.W direction) [21], representing the same flexural stress condition of the wall; an additional comparison for the interlayer traction properties was also conducted with respect to the values obtained through pull-off tests carried out post mortem of the walls. On the other hand, the diagonal compression strength (f_{vk0}), is derived from another study conducted by the authors, which focused specifically on shear mechanisms in 3D-printed concrete structures; the authors relied on ASTM E519 [41, 42], which is commonly adopted for masonry testing, ensuring a robust and standardized approach to evaluate shear resistance under controlled conditions. The calculation

of the ultimate shear, V_u (i.e., in plane wall strength), in different failure conditions, i.e., whether the failure is caused by the activation of the diagonal cracking mechanism, $V_{u,D.C.}$, or shear sliding, $V_{u,S.S.}$, or flexural-compression, $V_{u,P.F.}$, is formulated as follows:

$$V_{u,D.C.} = L_w \cdot th_{eq,central} \cdot \frac{f_{ctm}}{1.5} \sqrt{1 + \frac{\sigma}{f_{ctm}}} \quad \text{Eq. 3.3}$$

$$V_{u,S.S.} = f_{vko} \cdot A_c + \mu \cdot N_{ext} \quad \text{Eq. 3.4}$$

$$V_{u,P.F.} = \left(L_w^2 \cdot th_{eq} \cdot \frac{\sigma}{2} \right) \cdot \left(1 - \frac{\sigma}{0.85 \cdot f_{ck}} \right) \cdot \frac{1}{h_{D-Section}} \quad \text{Eq. 3.5}$$

Where the external compressive stress, σ , is calculated as the ratio of the external normal forcestress, N_{ext} , to the actual area of the section, $A_{D-section}$. In addition, due to the non-uniform (i.e., not fully solid) of the wall cross section, th_{eq} is the equivalent thickness of the section as if it were completely solid; it is calculated as the ratio between the net area of the section, $A_{D-section}$, and the length of the wall, L_w . A_c represents the area of the compressed zone (equal to the product of the neutral axis, y_c , and the equivalent thickness of the section, th_{eq}); finally, $h_{D-Section}$ is the height from the point of load application to the D-section.

The results of Eq. 3.3, Eq. 3.4 and Eq. 3.5 are summarized in Table 3.4 and were obtained by considering a characteristic compression strength of the printed material in the W direction, f_{ck} , of 38.29 MPa; an average tensile strength value, f_{ctm} , of 4.10 MPa (see fig. 20(b)); a characteristic shear strength value in the absence of normal stress, f_{vk0} , of 1.31 MPa [42]; and a coefficient of friction value (derived, in a

conservative manner, from literature studies on shear tests on conventional concrete [43]), μ , equal to 0.5. In addition, an average value of the tensile strength equal to 1.5 MPa was also included in the calculation of Eq. 3.5 based on the experimental evaluation (postmortem through pull-off tests) of the material tensile stresses acting at the wall scale, $f_{ctm-exp}$, as described in Section 3.2. The difference between $V_{u,D.C.}$ and $V^*_{u,D.C.}$ of Table 3.4 lies in the tensile strength parameter adopted in Eq. 3.3. Specifically, $V_{u,D.C.}$ was calculated by using the average tensile strength f_{ctm} obtained from three-point bending tests at the material scale, whereas $V^*_{u,D.C.}$ was calculated by replacing f_{ctm} with the characteristic tensile strength derived from pull-off tests performed directly on the printed wall surfaces ($f_{ctm-exp}$). This approach was introduced to provide both a reference estimation based on intrinsic material properties and a conservative estimation reflecting possible reductions in tensile strength due to scale effects and interlayer bonding.

Specimen	$V_{u,D.C.}$	$V^*_{u,D.C.}$	$V_{u,S.S.}$	$V_{u,P.F.}$
-	[kN]	[kN]	[kN]	[kN]
P-BA3DW	660.65	251.32	240.34	23.67
C-BA3DW	652.37	247.43	257.88	23.66

Table 3.4.: Values of ultimate shear strength for both 3D Printed Concrete walls tested calculated according to Italian Structural Code and Eurocode for unreinforced masonry.

It is important to note that Eq. 3.3 describes the ultimate shear capacity in the case of diagonal cracking failure ($V_{u,D.C.}$), which occurs when the average tensile strength (f_{ctm}) equals the maximum tensile stress (σ_t), as determined using Mohr's circle criteria [44]. In this equation, the shear force is derived from the Jourawski distribution [34]. To

determine the principal tensile stress σ_t , an infinitesimal element at the midsection is considered, subjected to both the normal stress σ and the shear stress computed using Jourawski's distribution. By equating it to the average tensile strength of the material, f_{ctm} , Eq. 3.3 is obtained. This leads to the definition of an equivalent thickness for the central zone, based on the midsection of the wall (where the shear stress reaches its maximum according to Jourawski's distribution).

Eq. 3.4 represents the ultimate shear in the case of a shear sliding failure ($V_{u,S.S.}$), which occurs when the mean shear stress (τ_m) acting in the compressed zone at the discontinuity section (D-section) - defined as the ratio of the ultimate shear force to the compressed sectional area (A_c) - equals the characteristic shear strength, f_{vk0} , as determined using a Mohr-Coulomb approach [44].

Finally, Eq. 3.5 describes ultimate shear in the case of flexural compression failure ($V_{u,P.F.}$), resulting from the equality between the characteristic compression strength (f_{ck}) of the wall/material and the compressive stress acting on the cross-section of the wall (s_c) in the analysed cases [44].

From the comparison with the experimental results, it appears that the ultimate shear strength of the tested walls is overestimated compared to the values in Table 3.2, except for the case of failure due to compression under flexure, which is underestimated. From the side of diagonal compression and shear sliding, the overestimation can be attributed to the higher strengths - both tensile and from diagonal compression - compared to masonry [45]).

On the other side, the underestimation arises from several simplifications. The wall is assumed to behave like unreinforced masonry, neglecting tensile strength and considering a non-linear compressive stress distribution. Based on these assumptions, Eurocode 6 [38] adopts a simplified rectangular stress diagram with a peak compression stress value equal to 85% of the characteristic compressive strength ($0.85 \cdot f_{ck}$). Similarly, the design calculation in the Italian Structural Code [30] for ultimate shear strength under combined axial and bending stress does not align with experimental evidence, both in terms of ultimate shear force values (see Table 3.2 and 4) and crack propagation. As a result, for this failure model there is underestimation with respect to the actual ultimate shear strength (by approximately 40% - see Table 3.2 and Table 3.4).

For the axial force value analysed (N_{ext} , equal to 58 kN, comparable to the weight of a roof), the neutral axis position, y_c , can be derived from equilibrium, relating the applied normal force to the 85% of f_{ck} acting on the compressed zone with equivalent thickness, t_{eq} . Under ultimate conditions, the design approach of unreinforced masonry yields neutral axis positions of 8.13 mm for P-BA3DW and 7.45 mm for C-BA3DW - significantly lower than the experimental values (Figure 3.11). This discrepancy suggests that failure did not occur according to this mechanism, i.e. characterized by compression crushing, as also indicated by analytical results in Table 3.4. The underlying assumptions should therefore be revised to better reflect the behaviour of 3D-printed concrete material.

Additionally, the characteristic compressive strength of 3D-printed concrete is much higher than that of conventional masonry (typically between 1–10 MPa, so at least 4 times higher) [30,38]. This reduces the neutral axis depth even under high normal stresses, reasonably shifting the failure mechanism from compression-governed to tensile-governed. Therefore, a more appropriate model should account for a linear stress distribution and the tensile strength of the material and/or between adjacent layers, which plays a key role in failure under flexural compression. It is worth noting that, unlike traditional masonry - where tensile strength is negligible and the compressed area A_c is defined based on contact length (typically $3u$) - 3D-printed concrete walls may exhibit significant tensile capacity across the resisting area. This tensile strength governs crack initiation and leads to brittle failure, rather than the progressive sliding and toe crushing observed in masonry. Therefore, determining A_c through force equilibrium, while accounting for tensile resistance, better reflects the actual stress distribution and failure modes observed experimentally.

Neglecting to introduce correction factors or dedicated models for 3D-printed elements - while relying solely on nominal material strengths - can result in overly conservative designs and oversizing of structural sections. A masonry-based approach often underestimates actual flexural capacity but may significantly overestimate resistance in diagonal cracking and sliding shear modes, highlighting the need for targeted experimental validation. Overall, the findings underline the necessity of developing design values that incorporate process-related factors such as printing defects and geometric imperfections. For

instance, two factors could be considered for the 3D printed material: γ_c , traditionally set to 1.5 [30,38], and a different coefficient, γ_{3DCP} , which accounts for the 3D-printing process (e.g., longer delays between layer depositions tend to reduce the cohesion and adhesion at the interfaces, leading to decreased tensile and shear strength), layer orientation, and printed geometry/path, based on experimental evidence. It's important to note that for 3D-printed concrete walls, both of these coefficients are essential to properly define the design strengths.

3.4.4.1 Bi-linearization of the lateral force-deformation curves

In line with Eurocode 8 [46], the capacity curves for the in-plane response of the 3D-printed walls were derived using the N2 method, based on an energy-equivalent bilinearization of the experimental results. This phenomenological, macroscopic approach - commonly applied to unreinforced masonry - was primarily considered over more complex fracture mechanics or cohesive zone modelling strategies. Indeed, while cohesive zone models can capture the interfacial behaviour in greater detail, their implementation would have required additional assumptions and a larger number of interface parameters characterized by extensive calibration, limiting the suitability for design-oriented applications at the structural scale.

This approach simplifies the cyclic response into a bilinear shear–displacement diagram (Figure 3.15(a)), constructed from the trilinear response observed experimentally (Figure 3.15(b) and (c)). The bilinear system consists of an initial elastic segment and a post-yield plateau. The stiffness (K^*) of the elastic branch is calculated at the

intersection with 70% of the experimental peak shear force (V_y), while the ultimate shear (V_u) is obtained by matching energy areas. The maximum displacement (d_{max}) corresponds to a 20% strength drop from V_y , as per Italian Structural Code. Figure 3.13 shows the comparison between the experimental and bilinear capacity curves for both walls and loading directions: blue lines for P-BA3DW (push/pull in Figure 3.13(b)) and black lines for C-BA3DW (Figure 3.15(c)). To simplify calculations, the energy equivalence was performed using the trilinear approximation from Figure 3.15(a).

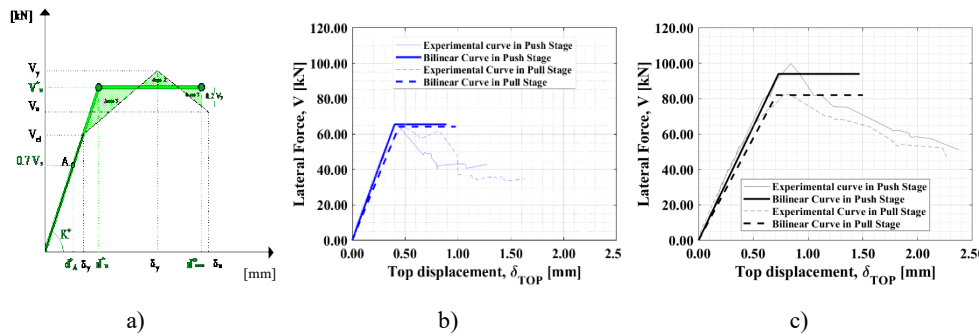


Figure 3.15.: Bilinear equivalent curves based on Eurocode (a); experimentally based capacity curves for P-BA3DW (a) and C-BA3DW in push and (c).

3.5 Conclusion

In this chapter is evaluated the structural performance of full-scale 3D-printed concrete walls under in-plane cyclic (seismic) loads. Two anchorage configurations were tested: prefabricated with steel shoes (P-BA3DW) and conventional (C-BA3DW) with steel rebars. The walls showed elastic behaviour up to the onset of flexural cracking, followed by failure governed by mixed-mode and shear-sliding mechanisms.

Peak lateral forces were approximately 66 kN (P-BA3DW) and 90 kN (C-BA3DW), with corresponding top displacements of 0.40 mm and 0.50 mm. The minimum secant stiffness was 132.50 kN/mm, confirming a stiffer response than traditional masonry, especially in the P-BA3DW case, which exhibited localized failure due to stress concentration around the steel anchorages. In contrast, C-BA3DW showed more distributed damage progression. Overall, failure was characterized in three phases:

- Elastic: flexural cracking along weak interfaces.
- Yielding: combined shear and tensile interaction.
- Ultimate: shear-sliding potentially followed by localized crushing.

Tensile strength (mostly across layers) between 1–2 MPa governed the crack initiation, while compressive crushing was not critical due to the high concrete strength (~45 MPa). Classical models adopted from unreinforced masonry tended to overestimate shear strength and underestimate flexural compression (~40%). These observations - and within the axial load range investigated - confirm that traditional masonry-based failure models are limited for 3DCP and should be revised to include tensile strength contributions in flexural behaviour. Indeed, the higher compressive strength of 3D-printed concrete (~45 MPa) significantly reduces the risk of failure due to compressive crushing (especially in the case of moderately low combined axial load), shifting the critical failure mode toward flexural and interface-driven mechanisms. Additionally, the interlayer time (affecting tensile

strengths) could represent as a key variable affecting structural performance, confirming the importance of including process-related parameters in mechanical predictions and design criteria. Overall, phenomenological capacity curves - developed via the N2 method – were proved to be effective for seismic assessment and represent a possible tool for implementing resistant models of 3D printed walls.

The results highlighted the need for updated analytical models that incorporate tensile contributions and printing-induced particularities. Interlayer time and additional printing parameters could be explicitly considered via correction factors for material strengths. As a preliminary framework, the correction factor for 3dpc could be formulated as a function of key process parameters, including interlayer time, print orientation, and layer height, which are known to affect interface strength and anisotropy. Future studies should aim to define and calibrate this factor through targeted experimental campaigns (e.g., with different vertical load magnitudes to analyse possible shift of the observed failure modes) and parametric analyses to support reliable seismic assessment of 3D-printed walls. Notably, scale effects may influence the effective tensile and shear strength of printed structures due to increased interfacial variability over larger areas. Indeed, future research should also address scalability aspects - such as the performance of multi-storey wall systems and connections - and investigate environmental effects including creep, shrinkage, and long-term durability, which are likely to further impact structural reliability over time.

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4 Resistance Domains and Constitutive Models for the structural design of 3DPC walls

The advancement of 3D-printed concrete (3DPC) introduces new opportunities for optimized, automated, and material-efficient structural solutions. However, the mechanical behaviour and failure mechanisms of these structures differ significantly from conventional reinforced concrete and masonry walls, requiring an in-depth investigation of their resistance domains and constitutive models. In this chapter aims to develop resistance domains for 3D-printed concrete walls.. A fundamental aspect of structural assessment is the regulatory framework, which currently lacks specific design provisions for 3DPC elements. The study references Eurocode 2 (EN 1992-1-1) and NTC 2018, adapting their principles to define ultimate limit states (ULS) and derive failure envelopes for walls under combined axial and bending actions. Key failure modes, including flexural compression, diagonal cracking, and shear sliding, are analysed using experimental data and theoretical modelling. A crucial part of this research is the constitutive modelling of 3D-printed concrete, which exhibits layer-dependent strength variations. The material behaviour is analysed using a three-phase constitutive approach: elastic response, post-

cracking compression regime, and non-linear up to failure.. Additionally, the effect of interlayer time on mechanical performance is assessed, demonstrating how increased printing intervals weaken bonding and reduce overall strength. Comparative analyses with traditional masonry highlight the unique failure patterns observed in 3DPC walls.

4.1 Introduction: Regulatory Framework

The introduction of 3D-printed concrete (3DPC) structures into the construction industry represents a significant technological advancement, offering new opportunities for optimized and automated building processes. However, their innovative nature necessitates a clear regulatory framework to define structural behaviour, safety assessments, and ultimate limit states (ULS). In this regard, it is crucial to integrate 3DPC elements into existing design codes to ensure their reliability and structural performance. Current regulations, such as the Italian Technical Standards for Construction (NTC 2018, Ministerial Decree 17/01/2018), establish a limit state-based approach for structural design, distinguishing between ultimate limit states (ULS) and serviceability limit states (SLS). The ULS ensures that a structure can withstand extreme loading conditions without failure, while the SLS governs deformations and durability under operational loads.

In traditional reinforced concrete (RC) and masonry structures, the verification of structural safety at ULS follows the inequality $R_d \geq E_d$, where R_d is the design resistance and E_d is the design action, both

derived using partial safety factors [1]. For any wall structural element, resistance domains are typically constructed by considering the axial force N_{Ed} and bending moment M_{Ed} acting on the resisting section. The resistance domain is thus a graphical representation that describes the evolution of M_{Rd} (or shear resistance, V_{Rd} , equal to M_{Rd} divided by the height of the wall, corresponding to the point of application of the lateral load) as a function of N_{Ed} , allowing engineers to assess the structural capacity under combined loading conditions [2]. One of the key challenges in 3D-printed concrete design is the presence of interlayer interfaces, which introduce anisotropic mechanical behaviour and reduce the homogeneity of the material [3]. Unlike cast-in-place concrete, which can be assumed as isotropic at the macro scale, 3DPC exhibits layer-dependent strength variations, affecting both compressive and tensile resistance [4]. Consequently, the construction of resistance domains for 3DPC walls requires specific considerations regarding:

- Bonding quality between layers and its impact on flexural strength [5].
- Layer orientation and print direction, influencing the overall stress distribution [6].
- Material properties under different curing conditions, affecting mechanical performance [7].

Previous studies have shown that the flexural resistance of 3DPC walls strongly depends on interlayer bonding and curing conditions, with significant reductions in strength as interlayer time increases [8]. Despite its growing adoption, 3D-printed concrete is not yet explicitly

covered in major international design codes, such as Eurocode 2 (EN 1992-1-1). However, research efforts are ongoing to define appropriate design methodologies, integrating existing mechanical models from masonry [9]. In this work, we aim to construct resistance domains for 3D-printed concrete walls, analysing their failure mechanisms and deriving constitutive models that capture the effects of interlayer bonding. In this context, the definition of appropriate constitutive models becomes essential, as they represent the physical link between stress and strain within the printed material creating the analytical starting point to construct the resistance domains.

4.2 Material Constitutive Models for 3d Printed Concrete

To accurately capture the behaviour of 3D-printed concrete, it is essential to identify and describe the constitutive models that govern its mechanical response. The literature consistently reports a pronounced structural anisotropy, with strengths that vary significantly with the direction of loading [10; 11]. Reference should also be made to the tested wall presented in the previous chapter, since its results clearly illustrate the three main stages of material response. Furthermore, in the present analytical evaluation, characteristic strength values are adopted rather than design values. This choice is motivated by the high variability and directional dependence of the mechanical properties observed in 3D-printed concrete, which introduce significant uncertainty in defining suitable partial safety

factors for each printing orientation. As discussed in Chapter 3, the use of characteristic strengths therefore provides a consistent and transparent basis for comparing analytical and experimental results.

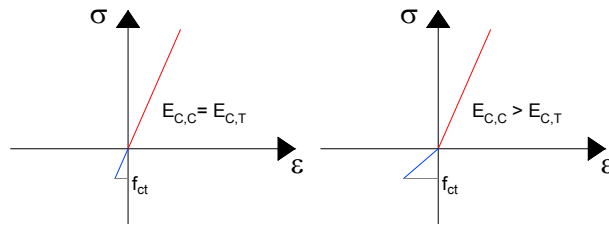
Based on experimental studies on printed concrete walls [12] (described in previous chapter 3), the material response can be organised into three main stages. In the first stage, concrete carries both tension and compression; in the second stage, it carries compression only (after reaching its tensile capacity); and in the third stage it reaches the ultimate condition, with the ultimate failure. The transition thus proceeds from an elastic regime with tensile resistance (Stage 1), to an elastic regime with compression-only resistance after cracking (Stage 2), and finally to a nonlinear regime up to collapse (Stage 3). Examining these stages in more detail, the behaviour of a 3D-printed concrete wall section can be interpreted in terms of material-level and section-level. In particular, Examining these stages in more detail, the behaviour of a 3D-printed concrete section, at material scale, can be evaluated in terms of tension–compression resultants (Figure 4.1) as well as shear actions (Figure 4.2). In the first stage (linear-elastic), the section resists both tension and compression and no cracking has developed yet. The material behaves in a linear-elastic manner, so stresses are proportional to strains (Hooke's law). Failure may occur if the tensile strength of concrete is exceeded (Figure 4.1a) or if the shear strength limit is reached (Figure 4.2a) according to the Mohr–Coulomb criterion, which describes shear failure by the relation $\tau = c + \sigma \tan (\phi')$ where c is cohesion, ϕ' is the internal friction angle, and σ is the acting normal

stress. The manifestation of either tensile or shear failure marks the end of the first, elastic stage. Still at the material scale, the second stage (linear-elastic with compression-only resistance), once the tensile strength has been exceeded, the concrete cracks and can no longer sustain tensile stresses. In this stage, The section, at panel scale, therefore works in a linear-elastic manner in compression only (tensile load-carrying capacity is zero beyond cracking). In this intermediate stage, failure of the element may occur either by compressive crushing (Figure 4.1b) or by shear (Figure 4.2b) along the printed layers, depending on which mechanism activates first. The response remains approximately linear until one of the ultimate resistances (in compression or in shear) is reached.

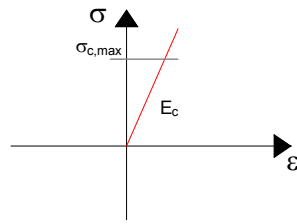
Finally, in the third stage (nonlinear response up to failure at panel scale), the material has reached the ultimate limit state and enters a nonlinear regime, with plasticity, strain localisation, and the development of macro-cracks. If the dominant collapse mechanism is compression, the response can be described by established nonlinear compression models (Figure 4.1c)—for example, the Eurocode parabola–rectangle diagram or the classical stress-block used in reinforced concrete design [13]; these are the design formulations typically adopted to describe the linear and nonlinear behaviour of concrete, providing a consistent representation of its response under both tension and compression. If failure of the section is governed by shear, the collapse is brittle and can be represented by fracture-mechanics models (Figure 4.2c). In particular, a progressive fracture approach such as that proposed by Bažant and Planas [14] is

applicable, accounting for the nonlinear evolution of cracking in concrete. Energy-based cohesive-crack and crack-band models describe the gradual degradation in resistance beyond the peak, in agreement with experimental evidence of failures that are not instantaneously brittle but preceded by crack propagation.

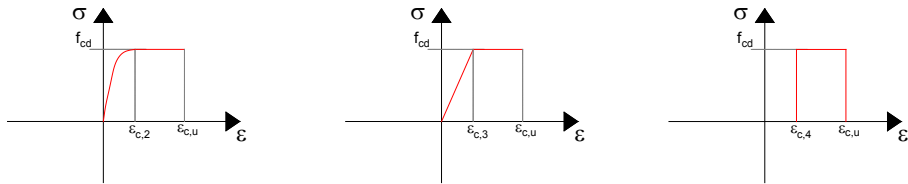
In summary, the constitutive behaviour of 3D-printed concrete can be partitioned into three characteristic stages, each requiring appropriate constitutive laws. The first stage follows a linear-elastic model with active resistance in both tension and compression (uncracked material). The second stage remains linear-elastic but with resistance only in compression and shear (cracked material with no tensile capacity). The third stage is governed by a nonlinear model—plastic or brittle depending on the prevailing failure mechanism (compression vs. shear)—with residual capacity essentially confined to compression. In the linear-elastic range (Stages 1 and 2), normal stresses σ are linked to strains ϵ by Hooke's law, $\sigma = E \epsilon$, where E is the tangent elastic modulus; shear stress follows the Mohr–Coulomb relation $\tau = c + \sigma \tan(\phi')$ with material cohesion and internal friction parameters as defined above at material scale. These simple linear relations hold until cracking or yielding occurs. In the nonlinear range (Stage 3), the material exhibits progressive softening if shear fracture governs, or plastic behaviour followed by an abrupt strength drop if governed by compressive crushing.



(a)



(b)



(c)

Figure 4.1. Constitutive models of 3DPC in tension and/or compression: (a) initial linear-elastic stage; (b) post-cracking stage with loss of tensile capacity and linear-elastic response in compression; (c) final nonlinear stage up to collapse.

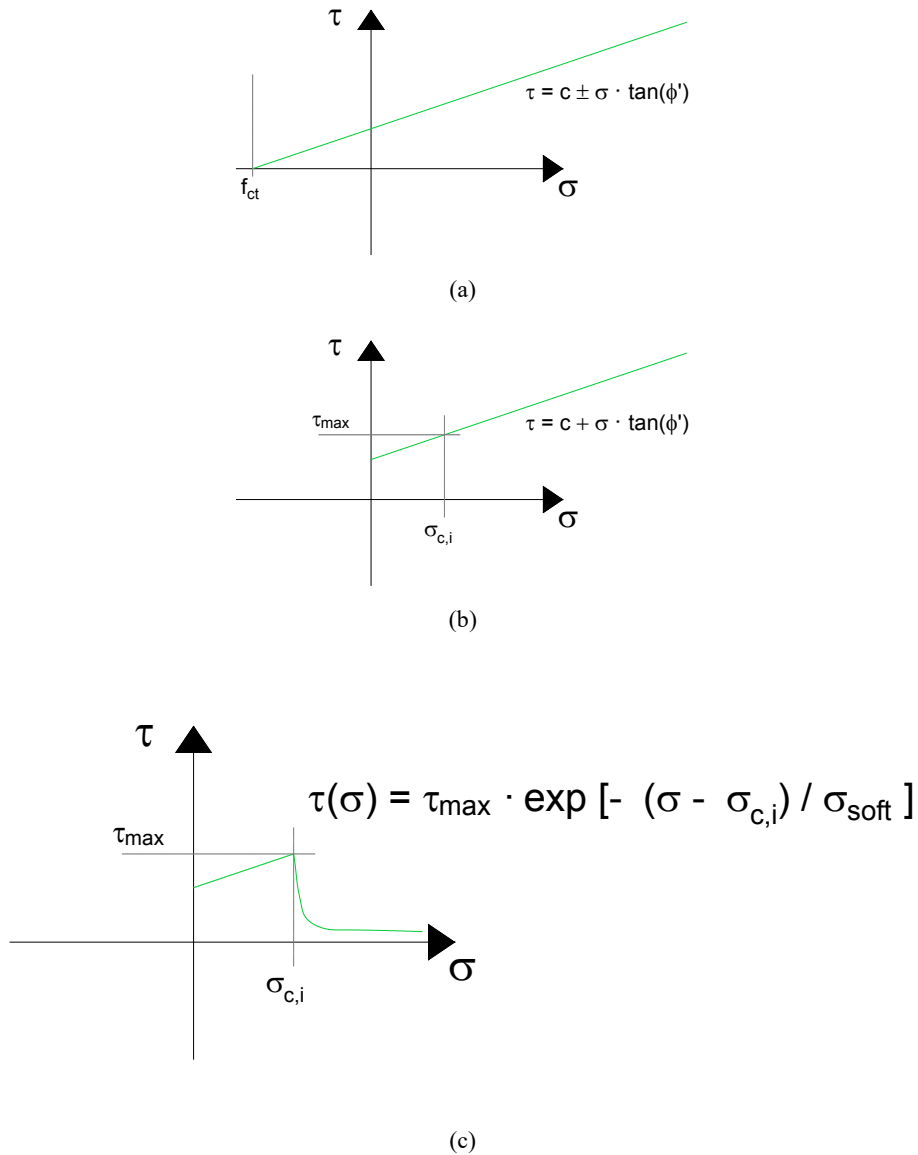


Figure 4.2. Constitutive models of 3DPC in shear: (a) initial linear-elastic stage; (b) post-cracking stage with loss of tensile capacity and linear-elastic response in shear; (c) final nonlinear stage up to collapse.

It is worth noting that the constitutive modelling of 3D-printed concrete (3DPC) must explicitly account for its directional anisotropy, as the mechanical properties vary with the orientation of the printed layers

[10, 11]. In particular, tensile and shear strengths are typically lower along the interlayer interfaces, while compressive stiffness and strength are less affected in the printing direction. Consequently, the constitutive relationships—linking stresses and strains—should be defined with respect to the local printing reference system ($u-v-w$), where w denotes the deposition direction. If the orientation of the printed element changes relative to the global loading direction, the constitutive parameters must be transformed accordingly, as the material response depends on the relative angle between load and layer orientation. This directional dependency is fundamental for accurately representing the structural behaviour of 3D-printed components and highlights the need for anisotropic constitutive formulations that remain consistent with the printed architecture.

Furthermore, in conventional solid materials, such as cast concrete, the shear strength τ can be directly associated with the intrinsic cohesion of the material, derived from laboratory tests on homogeneous specimens. However, in the case of 3D printed concrete, the situation is more complex due to its layered architecture (Figure 2.1). The mechanical response is not governed solely by the cohesion of the bulk mortar but is also strongly conditioned by the weaker interfacial planes formed between successive layers during the printing process. As a result, the effective shear strength that should be adopted in design cannot be taken as the cohesion of the material alone. Instead, it must be conservatively defined as the minimum value between the intrinsic cohesion of the mortar, c (at material scale), and the shear resistance measured at the panel scale,

f_{vk0} , where interlayer bonding, anisotropy, and process-induced defects become decisive. This approach ensures that the design parameter reflects the weakest link of the system, thereby embedding into the verification procedure the specific vulnerabilities of printed elements and the variability associated with the additive manufacturing process.

4.3 Analysis of 3D-Printed Concrete Walls Under In-Plane Actions

In the global seismic response of buildings composed of 3D-printed concrete panel systems, walls subjected to in-plane actions play a decisive role. If the building has rigid diaphragms that are well connected to the wall system, the walls oriented in the direction of the earthquake will attract most of the seismic action, which will be distributed among them in proportion to their stiffness. Conversely, if the building lacks rigid, well-connected diaphragms, the individual walls oriented in the earthquake direction will be subjected to horizontal actions associated with the masses associated to them. The response of each wall is, in turn, governed by the flexural and shear behaviour of its constituent elements.

4.3.1 Behavior Analysis of 3d-Printed-Concrete Walls

4.3.1.1 Main Parameters to Analyse Behaviour Analysis 3d-Printed-Concrete Walls

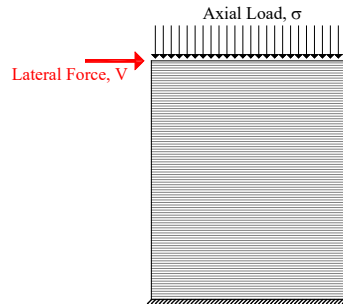


Figure 4.3. Internal force demands on a typical 3D-printed concrete element under in-plane loading.

If an experimental test is performed on this printed panel — subjected to an axial load σ and a lateral action V (as reported in Figure 4.3) — one can obtain a diagram featuring an experimental curve (shown in black in Figure 4.4) that relates the displacement, δ , to the shear, V [12].

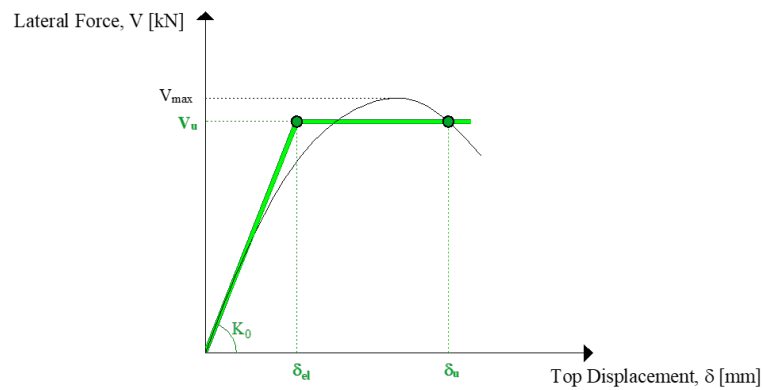


Figure 4.4.. Simplified constitutive relationship

As expected, the experimental curve, represent in the chapter 3, is nonlinear and reaches a maximum shear $V_{\max}$ at its peak. For structural design purposes, a simpler—often linearized—law is adopted so that it can be characterised by a few parameters that are

easy to determine experimentally or analytically. To this end, an elastic–perfectly plastic bilinear law is used. This relationship is characterised by an ultimate shear V_u equal to 90 percent of V_{max} , by an elastic-limit displacement δ_{el} , by an ultimate-limit displacement δ_u , and by an initial stiffness K_0 [15]. The ratio between δ_{el} and δ_u defines the ductility, μ . In summary, the parameters that allow us to model the behaviour of the 3D-printed concrete panel are the initial stiffness K_0 , the resistance V_u , and the ductility μ .

4.3.1.2 Stiffness, K_0 , of 3D-Printed Concrete Panels Under Shear Actions

Turning to the initial stiffness K_0 , its value depends on the structural static scheme adopted. Two reference configurations are typically considered: a panel that is well restrained at the top by infinitely rigid cross-beams (see **Figure 4.5a**), and a panel that is not adequately restrained at the top (see **Figure 4.5b**), which correspond, respectively, to the cases of panels with full top boundary constraint and panels with free or flexible top restraint..



Figure 4.5. Strcutural Patterns.

When the panel is well restrained at the top, the element can be idealised as a fixed base with a double-pendulum restraint at the top (**Figure 4.5a**). Conversely, if the structure is not adequately restrained at the top, the scheme reduces to a simple fixed-base cantilever (**Figure 4.5b**). Accordingly, the element's flexural stiffness, K_M , changes: in the first case (Figure 4.5a) it is given by Eq. (Eq. 4.1), whereas in the second case it is given by Eq. (Eq. 4.2).

$$K_M = \frac{12EI}{h^3}$$

Eq. 4.1

$$K_M = \frac{3EI}{h^3}$$

Eq. 4.2

where E is the elastic modulus of the wall, I is the in-plane second moment of area of the wall cross-section, and h is the wall height. It is worth noting that the adopted elastic modulus (15 GPa) refers to the bulk material and is used here as a simplifying assumption. In reality, the printed wall behaves as a quasi-monolithic structure, unlike the masonry-like configuration with horizontal and vertical joints, for which a specific wall-scale characterization would be required. The shear stiffness is evaluated using Eq. 4.3.

$$K_T = \frac{GA}{\chi h} \quad \text{Eq. 4.3}$$

where χ is the shear factor and A is the cross-sectional area. Recalling that deformability (compliance) is the inverse of stiffness, the total deformability is then defined by Eq. 4.4.

$$\frac{1}{K_0} = \frac{1}{K_M} + \frac{1}{K_T} \quad \text{Eq. 4.4}$$

from which the initial stiffness K_0 can be readily obtained.

4.3.1.3 Strength of 3d Printed Concrete Panels Under In-Plane Actions

To determine the maximum shear action that a 3D-printed concrete wall panel can sustain, it is necessary to assess the different failure mechanisms that may develop as the shear demand increases. The failure modes of a panel depend on both its geometry and the applied loading. In particular, the main collapse mechanisms are [12]:

- Flexure Compression (FC)
- Diagonal cracking (DC)
- Shear-sliding (SS)

4.3.1.3.1 Failure Mechanism in 3D Printed Concrete Walls

Failure mechanisms in 3D-printed concrete (3DPC) walls can be grouped into three main categories—flexure with axial load, diagonal cracking, and shear-sliding—each associated with a distinct stress state and collapse mode. Figure 4.6 illustrates these three mechanisms.

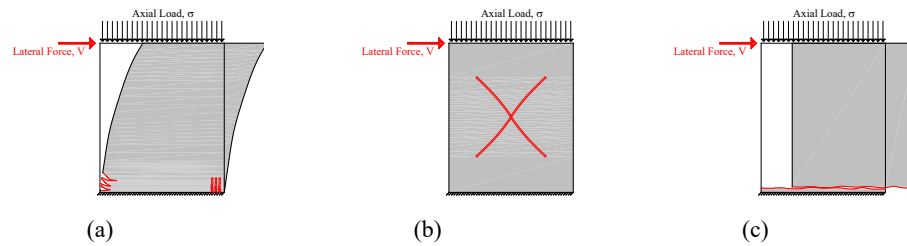


Figure 4.6. Representation of the main failure mechanisms in 3D-printed concrete (3DPC) wall panels.

In flexure with axial load, the response is governed primarily by bending moments acting together with a normal force. Tensile cracks initiate at the tension edge, while concrete on the compression side progressively crushes as demand increases. The likelihood of a flexural collapse grows with wall slenderness and unfavourable boundary conditions, and it is strongly influenced in 3DPC by layer orientation and interlayer tensile capacity, which affect the neutral-axis position and post-cracking stiffness. Diagonal cracking corresponds to a shear–compression mechanism in which inclined cracks form across the web when the principal tensile stress exceeds the material’s tensile strength. This mode is sensitive to the shear-span ratio and the level of axial load (which can provide confinement but also raise compression demands). In 3DPC, weaker or poorly bonded interlayers can lower the diagonal and flexural tension capacity and inclination of crack trajectories toward print-induced planes. Shear-sliding occurs when sliding localizes along weak interfaces—typically layer-to-layer contacts—once the available cohesion and friction are exceeded. It is controlled by the interfacial Mohr–Coulomb parameters (cohesion and friction angle) and is highly sensitive to interlayer time, surface condition, and curing. The resulting failure is usually brittle, with limited

ductility and pronounced pinching under cyclic loading, as also observed in the experimental tests on full-scale 3D-printed walls, illustrated in chapter 3, where a sudden loss of stiffness and strength occurred after the peak load. The hysteretic curves exhibited narrow loops and a rapid degradation of secant stiffness, confirming the prevalence of interlayer-driven brittle failure mechanisms. In practice these mechanisms may interact, but one will govern depending on geometry, axial load level, boundary conditions, and interface quality. A robust assessment therefore characterizes stiffness, strength, and ductility for each mode and identifies the governing one through analytical domains and targeted testing.

More specifically, flexure with axial load occurs when the maximum compressive stress σ_c and/or tensile stress σ_t reach the corresponding material strengths—respectively the compressive strength f_{ck} and the tensile strength f_{tk} —of the material. The activation of this mechanism depends on load eccentricity: with small eccentricity the crisis is compression-controlled, whereas with large eccentricity it becomes tension-controlled. The typical collapse pattern for this mode is shown in Figure 4.6 (a). The diagonal-cracking mechanism is associated with the formation of inclined cracks across the panel, see Figure 4.6 (b). It arises from a field with one tension diagonal and one compression diagonal: cracking develops along the tension diagonal when the principal tensile stress (or equivalently the tensile stress σ_t in the critical direction) reaches the tensile strength of the material, f_{tk} . In this case, our aim is to determine the ultimate shear V_u corresponding to the attainment of the critical tensile stress in the panel.

Finally, shear-sliding is triggered by relative slip concentrated along a weak layer—typically near the support—see Figure 4.6(c). Sliding starts when the shear stress τ_u along the prospective slip plane equals the shear resistance. In particular, the shear resistance of the system, which can be expressed either as the material cohesion c or as the panel shear strength f_{vk0} . Since possible scale effects may occur, and while the exact value of cohesion (c) is still under experimental investigation, f_{vk0} was determined through diagonal compression tests, yielding a value of approximately 2 MPa. The shear resistance can be expressed, in Mohr–Coulomb terms, as the sum of the cohesion, c , and the product of the tangent of the friction angle, $\tan \phi'$, and the axial stress, σ . However, to ensure a conservative estimation of the shear strength, the lower value between the experimentally derived f_{vk0} and the cohesion c was adopted. In addition, in the absence of extensive large-scale experimental data, the Turnšek and Čačović (1971) [16] empirical formulation — $f_{vk0} \approx \frac{f_{ctk}}{1.5}$ — was also considered as a reference conservative criterion, relating the diagonal shear strength to the material tensile strength. This provides a lower-bound estimate of cohesion consistent with traditional masonry-based approaches, ensuring safety while acknowledging the ongoing refinement of experimental parameters for 3D-printed concrete.

4.4 Assumptions of the Calculation Model

The resistance domains presented in this chapter are evaluated at the Ultimate Limit State (ULS), adopting the characteristic material strengths defined in Chapter 2 (Materials) and recalled in this following section. The choice to perform the analysis using characteristic values—rather than design values—was made intentionally to avoid introducing additional uncertainties related to partial safety factors, which are not yet calibrated for 3DPC structures. In fact, for this material, the transition from laboratory-based material properties to design parameters is not straightforward, as it must inherently account for the printing-process variables. Among these, the interlayer time (ILT), printing path geometry, and direction of deposition have been shown to significantly affect the mechanical response and anisotropy of the printed elements, as discussed in Chapters 3 and 4.

It is clear that, given the many possible geometries of 3D-printed concrete elements, the mechanical and geometric parameters that describe the behaviour at the structural scale must be defined with care. In line with the guidance reported by RILEM [17], the mechanical properties adopted here were established through laboratory testing carried out at the Department of Structures for Engineering and Architecture, University of Naples Federico II. More specifically, the characteristic compressive strength of the printed material in the W direction (f_{ck}) was 38.29 MPa, while the average tensile strength (f_{ctk}) was 1.3 MPa, derived by experimental test described in chapter 3. The characteristic shear strength in the absence of normal stress (f_{vk0})

was 0.87 MPa (in a conservative manner, as described in chapter 2), and a coefficient of friction (μ) equal to 0.45 was adopted.

The ultimate in-plane shear strength corresponding to the adopted loading path is derived by combining Navier's stress equations with equilibrium conditions, under the assumption of a linear strain distribution across the section. This assumption is justified by the experimental evidence presented in Chapter 3, where the recorded in-plane drift ratios did not exceed 0.10%, indicating that the panels exhibited small displacement and rotation fields throughout the tests.

Within this small-deformation regime, the kinematic assumptions of Navier's beam theory remain valid, implying that (i) plane sections can be reasonably assumed to remain plane after deformation, and (ii) the normal strains vary linearly along the section depth. Consequently, the stress distribution and resulting internal force equilibrium can be consistently derived using Navier's linear elasticity framework, without requiring geometric nonlinearities or higher-order deformation terms. This simplification not only aligns with the observed experimental behaviour but also provides a consistent basis for evaluating the ultimate in-plane strength under combined axial and bending actions in 3D-printed concrete walls (seismic loads). Furthermore, unlike traditional masonry, where tensile resistance is neglected in Ultimate Limit State (ULS) conditions, the present formulation explicitly includes the tensile contribution of 3D-printed concrete up to the onset of brittle cracking. This choice reflects the experimental evidence of a measurable tensile response prior to failure. Hence, the proposed model accounts for tension stiffening effects up to cracking, ensuring

a more realistic representation of the pre-peak structural behaviour observed in the tests.

4.4.1 Scenario of the Analytical Model

The analytical framework developed in this work is based on a set of assumptions that allow the construction of resistance domains in a design-oriented framework. A fundamental assumption concerns the layered nature of 3D-printed concrete. Although the analytical formulation adopts an isotropic constitutive law at panel scale, the directional weakness associated with horizontal layer deposition is indirectly embedded through the mechanical parameters used as inputs. In this work, in particular in this chapter, tensile and interlayer shear resistance are assigned according with the weak direction across layers, Orientation I, i.e. the configuration in which all layers are horizontally aligned and the printing direction is perpendicular to the extrusion path that forms the horizontal layers, as described in Chapter 2. Hence, the resistance domains incorporate the effects of layer-induced weakness through the assigned mechanical parameters, while the analytical formulation itself remains isotropic. This approach is admissible because the walls considered in this work are printed with horizontally deposited layers and exhibit no geometric variation along their height, ensuring that the direction of weakness and the cross-sectional configuration remain constant over the full elevation.

It is therefore important to clearly separate the phase of material characterisation, where construction-related effects are quantified, from the phase of structural modelling, where such effects are

embedded through the assigned parameters. The analytical model does not explicitly simulate printing-induced phenomena; rather, it is based on mechanical inputs that already incorporate them in a consistent and conservative manner. Once these properties are defined, the global response of the walls follows the classical mechanics of planar elements.

A distinctive aspect of 3D-printed concrete is the geometric and mechanical variability introduced by the printing process, which must be rigorously identified. In this work, two forms of geometric variability are of primary relevance:

- Cross-sectional variability (internal property) – modifications in void layout or in the internal printed path;
- Layer-deposition variability (external property) – deviations in layer alignment or height along the wall elevation, affecting interlayer mechanical behaviour.

A third geometric aspect is the external shape of the element, which influences the external distribution of actions but not the material response (consistent with what occurs in conventional structural systems). The configuration of wall analysed in this work represents a controlled baseline case: layers keep horizontal along the full height, and both the void ratio and the printing path are constant over the entire wall height. Within this setting, a dedicated parametric study is carried out by varying (i) the interlayer tensile strength through different ILT conditions and (ii) the slenderness λ through geometric stretching of the benchmark section while keeping the void ratio

constant, in order to assess the influence of both the mechanical parameter (tensile strength) and the geometric parameter (slenderness) on the resulting resistance domains.

For more complex printed components—where internal geometry, layer orientation, or mechanical properties change along the height—extended formulations may be required, potentially involving resistance domains that vary with elevation, supported by updated of mechanical inputs. Additionally, the present analytical model describes the wall in-plane response up to the peak shear strength (as detailed in Chapter 3), considering both the cases with and without tensile capacity.

Finally, it is important to clarify that, a key conceptual insight emerging from this work is that 3DPC should not be regarded as a single material, but rather as a family of mechanically different typologies, shaped by printing parameters such as ILT, layer orientation, and internal geometry. Analogously to the classification of masonry units (e.g., solid brick, tuff, hollow blocks), it could be idealize the development of printed “families”, each characterised by a coherent set of tensile, shear, and compressive properties associated with its printing configuration. Validating such families through targeted testing—at both material and wall scale—would ultimately support the calibration and generalisation of analytical and numerical models across different 3DPC typologies.

4.4.2 Geometric and Mechanical Properties of the case study wall

The geometric and mechanical characteristics of the wall panel investigated are summarized in the following. The analysed panel is the same specimen tested by the authors in [12] and described in chapter 3. Accordingly, the wall has a plan footprint of 1.31×0.41 m and a height of 2.10 m with section of the type shown in Figure 4.7.

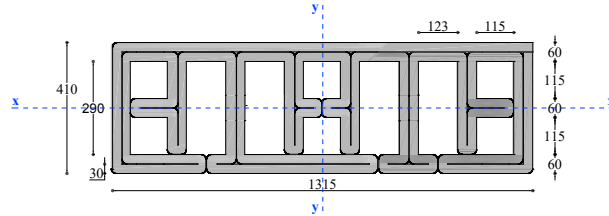


Figure 4.7. Cross Section of Panel Analytically studied.

It is possible to determine the geometric properties , such as the cross-sectional area, moment of inertia, and first moment of area, of the printed cross-section using a rigorous approach (e.g., through the integral form of the 1st and 2nd moment of area); however, given the flexibility of 3DCP in generating printing paths and related cross-sections we propose an alternative computation. In detail, a simplified equivalent-thickness model may be adopted. In this approach, the section is idealized as a solid rectangle of length L_w and equivalent thickness th_{eq} , defined as the ratio between the product of the net (real) sectional area A and the external wall thickness th_{ext} , and the gross (external) area A_{ext} . Accordingly, Eq. 4.5 gives:

$$th_{eq} = \frac{A \cdot th_{ext}}{A_{ext}} \quad \text{Eq. 4.5}$$

With th_{eq} , the main in-plane section properties can be obtained by using the solid-rectangle formulas for the centroidal position of the area with respect to the y-axis (as sketched in Figure 4.7). The second moment of area is described by Eq. 4.6 below:

$$I_{y,G} = \frac{th_{eq} L_w^3}{12} \quad \text{Eq. 4.6}$$

and the first moment of area of the portion of section below the centroidal chord with respect to the y-axis, $S_{max,y,G}$, is described by Eq. 4.7.

$$S_{max,y,G} = \frac{th_{eq} L_w^2}{8} \quad \text{Eq. 4.7}$$

where L_w is the wall length. In any case, Table 4.1 summarises the principal geometric properties of the analysed section, computed both by rigorous calculation and using a simplified approach.

Geometric Properties	A	L_w	th_{ext}	th_{eq}	$I_{y,G}$	$S_{max,y,G}$
	[m ²]	[mm]	[mm]	[mm]	[m ⁴]	[m ³]
Real cross-section	0.314	1315	410	-	0.049	0.05
With Equivalent thickness Th_{eq}	0.314	1315	-	238.8	0.045	0.05

Table 4.1. Main Geometric Properties: Difference between real and equivalent approach.

In addition, from the histogram in Figure 4.8, the values of the cross-sectional area A , the in-plane centroidal second moment of area about the y-axis $I_{y,G}$ (as defined in Figure 4.7), and the maximum first moment of area of the portion of section below the centroidal chord $S_{max,y,G}$ are essentially the same whether they are computed on the real section or on the equivalent section.

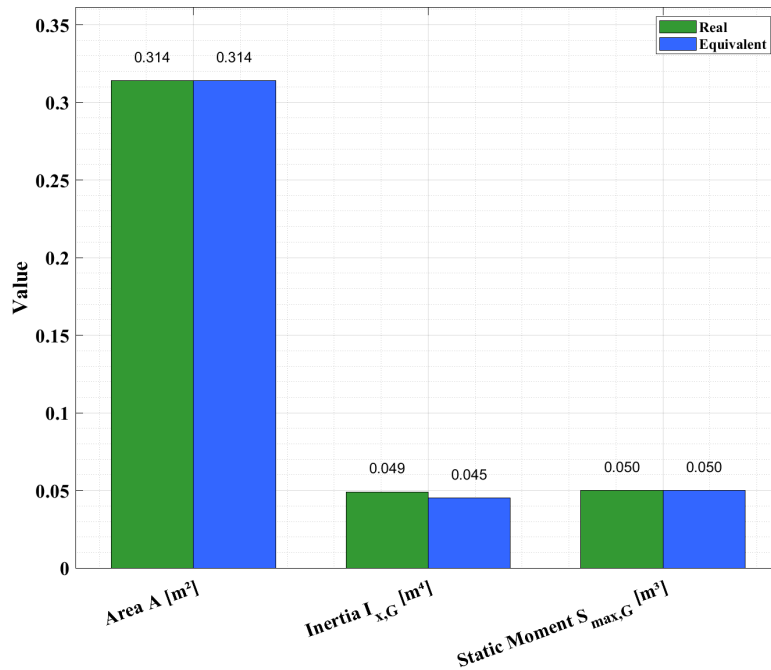


Figure 4.8. Bar chart comparing the real and equivalent geometric properties of the case-study wall.

For this case, the simplification is therefore justified, as the compared quantities show negligible differences for global in-plane assessment. Note that, in this case study, the first moment is evaluated with respect to the y-axis and to a chord passing through the mid-depth of the section. With reference to the equivalent thickness, an illustrative representation is provided in Figure 4.9. The equivalent thickness corresponds to the thickness that allows the cross-section to be treated as solid for the purpose of structural calculations. In Figure 4.9, for the case under investigation, the red-highlighted area represents the equivalent thickness and thus the corresponding full section, which can be used as a reference for the mechanical evaluation of the wall.

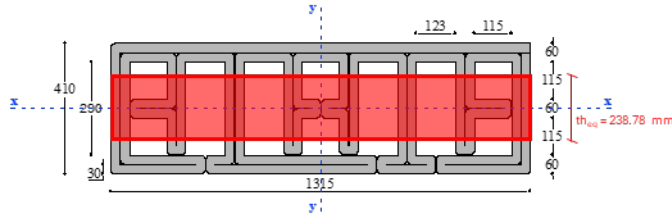


Figure 4.9. Illustrative representation of the equivalent thickness and the corresponding full section.

As for the material properties, we adopt the same values used in the previous sections and in the authors' earlier study [12]; they are summarised in Table 4.2. These are the parameters used consistently in the analytical derivations and in the subsequent modelling (e.g., construction of resistance domains).

f_{ck}	f_{ctk}	f_{vk}	μ	E	G
[MPa]	[MPa]	[MPa]	[-]	[MPa]	[MPa]
38.29	1.3	0.87	0.45	15000	6250

Table 4.2. Main Mechanical Properties

4.4.3 Small and Large Eccentricity

In developing the resistance domains for the 3D-printed wall subjected to in-plane loading, it is essential to distinguish between small and large load eccentricities (e), as they govern the transition from shear/compression-dominated to flexure-dominated failure mechanisms. This distinction is particularly relevant in the present case study, which aims to evaluate the ultimate in-plane shear capacity of the wall as a function of the applied vertical (axial) load, treated as the main in-plane action variable.. For a section that is

doubly symmetric about the x- and y-axes (Figure 4.7), the centroid of inertia can be determined. Specifically, for the section in Figure 4.7 the limits of the centroid of inertia along the x- and y directions—denoted n_{ox} and n_{oy} —are given by Eq. 4.8 and Eq. 4.9. For completeness, Eq. 4.10 and Eq. 4.11 define the radii of gyration ρ_x and ρ_y .

$$n_{ox} = \frac{\rho_x^2}{\frac{th_{ext}}{2}} \quad \text{Eq. 4.8}$$

$$n_{oy} = \frac{\rho_y^2}{\frac{L_w}{2}} \quad \text{Eq. 4.9}$$

$$\rho_x = \sqrt{\frac{I_{x,G}}{A}} \quad \text{Eq. 4.10}$$

$$\rho_y = \sqrt{\frac{I_{y,G}}{A}} \quad \text{Eq. 4.11}$$

where n_{ox} and n_{oy} are the maximum centroid of inertia (kern) limits with respect to the x- and y-axes (Figure 4.10); th_{ext} and L_w are, respectively, the external wall thickness (along y) and the wall length (along x); ρ_x and ρ_y are the radii of gyration about the x and y axes; $I_{x,G}$ and $I_{y,G}$ are the moments of inertia about x and y; and A is the real sectional area.

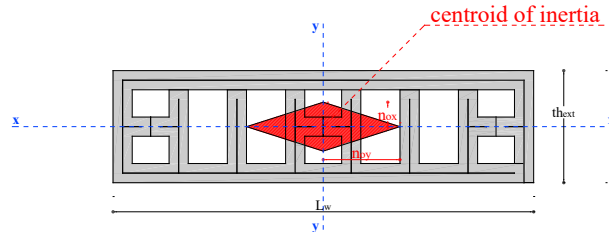


Figure 4.10. Representation of the central kern (area).

In practice, the transition between small and large eccentricity is assessed by comparing the applied eccentricities e_x and e_y with the kern limits. Figure 4.10 highlights the kern limits n_{ox} and n_{oy} in red. Accordingly, the section is in small eccentricity when the location of

the pressure centre (eccentricity e), defined as the ratio between the external bending moment, M , and the external axial force, N ($e=M/N$), lies within the central kern (no tension); otherwise, it is in large eccentricity (tension develops). More specifically, for the present case study (in order to calculate the ultimate in-plane shear capacity as a function of the applied vertical load (in-plane action variable) with a centred axial force and bending about the y-axis (M_y), the section is in small eccentricity if e is minus than n_{oy} , and in large eccentricity if e is major than n_{oy} .

4.5 Calculation of Resistance Domains

With this principle established, the goal is to determine—for each wall panel failure mode—the ultimate in-plane shear capacity V_u as a function of a generic applied axial force N , in order to construct resistance domains representative of 3D-printed concrete panels under in-plane actions. To this end, we introduce the average shear stress τ (Eq. 4.12) and the average normal stress σ (Eq. 4.13), defined as:

$$\tau = \frac{V}{L_w \cdot th_{eq}} \quad \text{Eq. 4.12}$$

$$\sigma = \frac{N}{L_w \cdot th_{eq}} \quad \text{Eq. 4.13}$$

where th_{eq} denotes the panel's equivalent thickness and L_w its length, so that their product yields the net transverse area of an equivalent solid panel.

4.5.1 Construction of the Resistance Domain for Flexural Compression (FC)

In this failure mode, collapse is triggered when either the maximum compressive stress, σ_c , reaches the compressive strength, f_{ck} , of the material, or when the maximum tensile stress, σ_t , attains the tensile strength, f_{ctk} . It is therefore necessary to distinguish between small and large eccentricities.

In the small-eccentricity regime, the entire section remains effective (no tensile cut-off), and failure occurs when the extreme compressive stress reaches f_{ck} . It is worth noting that, for 3D-printed concrete (3DPC) panels, the compressive strength f_{ck} is typically much higher than that of masonry, often by one order of magnitude compared to the characteristic masonry strength f_{wk} reported in Eurocode 6 [18]. Consequently, compression-controlled collapse may govern only at relatively high levels of axial load.

Under large eccentricity, the tensile strength f_{ctk} contributes up to the onset of flexural cracking. Beyond this point, the tensile capacity rapidly decreases due to the interaction with shear mechanisms, as discussed in detail in Chapter 3.

Overall, the flexure-with-axial mechanism is governed by the simultaneous attainment of ultimate tensile and compressive stresses at the cross-section edges under combined axial and bending actions. The ultimate in-plane shear strength corresponding to the adopted loading path is derived by combining Navier's stress equations with

equilibrium conditions, assuming a linear strain distribution and including, for 3D-printed concrete, the tensile contribution up to cracking within the resistance domain—accounting for the actual panel geometry, as previously described. Two cases are considered: (i) a fully effective (compressed) section, corresponding to small eccentricity; and (ii) a partially effective section, corresponding to large eccentricity with tensile contribution up to cracking.

For the small-eccentricity case, the problem is described by the equations (Eq. 4.14, Eq. 4.15 and Eq. 4.16) reported below.

$$\sigma_c = f_{ck} \quad \text{Eq. 4.14}$$

$$\sigma_c = \frac{N}{A} + \frac{M}{I_{y,G}} \cdot \frac{L_w}{2} \quad \text{Eq. 4.15}$$

$$M = V_u \cdot H_{effective} \quad \text{Eq. 4.16}$$

It is worth noting that Eq. 4.16 expresses the bending moment at the section in terms of the in-plane shear. In the case where the wall behaves as a cantilever with deformable diaphragms, the moment is evaluated using the ultimate shear at the critical section height $H_{effective}$. If, instead, the floor diaphragms are infinitely rigid, the global mechanism corresponds to a fixed base with a double-pendulum restraint at the top; in that case, the moment is computed using the same shear but with the critical height halved, providing a more conservative estimate that was therefore adopted in the resistance domain construction. To determine the flexure-controlled ultimate shear for the small-eccentricity case, Eq. 4.14, Eq. 4.15 and Eq. 4.16 are combined to obtain an expression for V_u as a function of the wall's

mechanical properties, geometric parameters, and the applied axial force N . In summary, the governing relation for flexural failure (as a function of the level of axial load N and applied lateral force) at small eccentricity is given in Eq. 4.17.

$$V_u = \left(f_{ck} - \frac{N}{A} \right) \cdot \frac{2 \cdot I_{Y,G}}{L_w \cdot H_{effective}} \quad \text{Eq. 4.17}$$

Instead, In the large-eccentricity situation, the section is partially effective. Unlike conventional masonry—see [12;19]—the tension zone must be retained, as it is essential both for a correct tracing of the resistance domain and for enforcing equilibrium due to possible interlayer shear mechanisms potentially acting, as explained in chapter 3. The failure scheme considered is shown in Figure 4.11. Under these assumptions — namely small displacements, as confirmed by the experimental drift measurements (below 0.10%), linear strain distribution, and elastic material behaviour up to failure — the equilibrium and compatibility equations can be applied within a linear-elastic framework. Although this may appear as a simplification for an ultimate limit state (ULS) analysis, it is consistent with the brittle response typically observed in unreinforced 3D-printed concrete (see chapter 3), whose stress–strain relationship remains nearly linear until fracture, as reported in recent studies on printed mortars and walls, For these reasons, the relationship adopted in the derivation stems from the similarity of triangles and the plane-sections hypothesis, as illustrated in Eq. 4.18. .

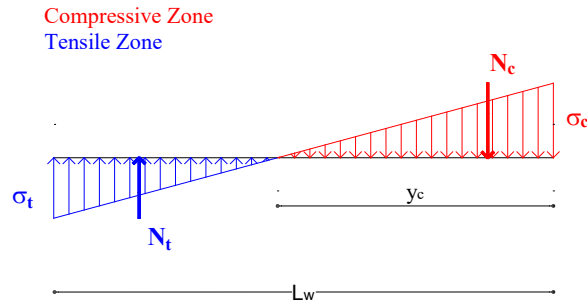


Figure 4.11. Stress Diagram in case of Large Eccentricity.

Under the assumptions of small displacements and linear elastic behaviour of the printable concrete, the position of the neutral axis can be uniquely determined using Eq. 4.18.

$$y_c = \frac{\sigma_c \cdot L_w}{\sigma_c + \sigma_t} \quad \text{Eq. 4.18}$$

Failure in this case is governed by the equilibrium equation (Eq. 4.19) and occurs when either the maximum compressive stress σ_c equals the panel's compressive strength f_{ck} , or when the maximum tensile stress σ_t equals the panel's tensile strength f_{ctk} .

$$\frac{\sigma_c \cdot A_c}{2} - \frac{\sigma_t \cdot A_t}{2} = N \quad \text{Eq. 4.19}$$

where the compression area A_c and the tension area A_t are functions of the neutral-axis position y_c measured from the compressed edge. The compressive stress σ_c and the tensile stress σ_t are given by the Navier bending formulas (Eq. 4.20 and Eq. 4.21) **[Errore. Il segnalibro non è definito.]**. Depending on the case under examination, σ_c and σ_t are then set equal to the corresponding characteristic strengths, f_{ck} and f_{ctk} ; the bending moment is evaluated as described in Eq. 4.16.

$$\left\{ \begin{array}{l} \sigma_c = \frac{N}{A} + \frac{M}{I_{y,G}} \cdot \frac{L_w}{2} \\ -\sigma_t = \frac{N}{A} - \frac{M}{I_{y,G}} \cdot \frac{L_w}{2} \end{array} \right. \quad \text{Eq. 4.20}$$

$$\left\{ \begin{array}{l} \sigma_c = \frac{N}{A} + \frac{M}{I_{y,G}} \cdot \frac{L_w}{2} \\ -\sigma_t = \frac{N}{A} - \frac{M}{I_{y,G}} \cdot \frac{L_w}{2} \end{array} \right. \quad \text{Eq. 4.21}$$

where σ_c denotes the compressive stress evaluated at the governing limit state, while σ_t denotes the tensile stress under the same condition. N is the externally applied axial compressive force. M is the external bending moment, taken as the product of the acting shear V and the cracking height $H_{\text{effective}}$ (or H_{crack}); the latter is the net height over which cracking develops, equal to the difference between the total wall height H_{tot} and the anchorage (fixing) height at the base H_{fix} [12]. $I_{y,G}$ is the centroidal second moment of area of the cross-section about the axis orthogonal to the direction of the applied shear on the panel. Finally, L_w denotes the overall wall length in the direction parallel to the applied shear. The two relations expressed in Eq. 4.20 and Eq. 4.21— are derived from the Navier formulas [**Errore. Il segnalibro non è definito.**] and combined appropriately—allow the neutral axis to be determined for the various configurations. Infact, for each pair of external actions (M, N) , a unique neutral-axis position y_c is obtained by imposing the limit tensile and compressive stresses and enforcing translational equilibrium (Eq. 4.19) between internal and external forces. (Eq. 4.19). Under this condition, the difference between the compressive resultant N_c and the tensile resultant N_t must equal the applied axial load N . Consequently, the compressive (or tensile) resultant can be computed by multiplying the area of the compressed (or tension) zone by the corresponding extreme stress σ_c (or σ_t) and dividing by two (consistent with the plane-sections

assumption and a linear stress block). What is also important is how to quantify the compressed area A_c and the tension area A_t of the real section. For clarity, these areas are illustrated in Figure 4.12.

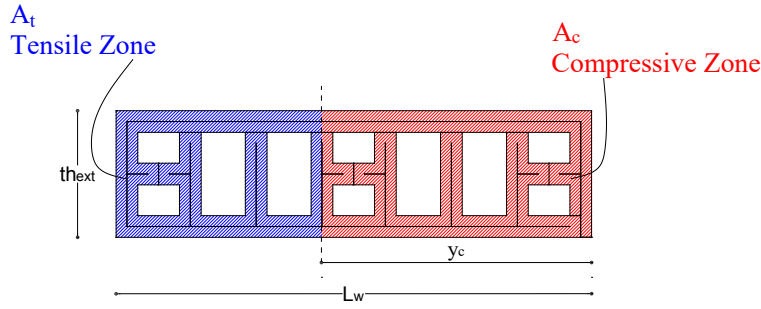


Figure 4.12. Representation of the compression and tension areas for a generic neutral-axis position.

In Figure 4.12, starting from a generic position of the neutral axis y_c —measured from the most compressed edge—the compressed area A_c and the tension area A_t are shown. These quantities are functions of y_c and are governed by the equilibrium expressed in Eq. 4.19. More simply, as discussed in the previous paragraph, the problem can be simplified by expressing the compressed area as the product of y_c and an equivalent thickness th_{eq} , treating the section as if it is solid. Owing to the linearity and symmetry of the problem, a good approximation for the equivalent thickness of the 3D-printed concrete section, th_{eq} , can be obtained by applying the relation given in Eq. 4.5. In light of this, Eq. 4.22 and Eq. 4.23 are written to determine the portions of area in compression A_c and in tension A_t .

$$A_c = y_c \cdot th_{eq} \quad \text{Eq. 4.22}$$

$$A_t = A - A_c \quad \text{Eq. 4.23}$$

Accordingly, by suitably combining Eq. 4.18–Eq. 4.23, the governing relation depends on which limit controls the response: when failure is triggered by attainment of the tensile capacity f_{ctk} (that is, σ_t equal to f_{ctk}), the problem is governed by Eq. 4.24; when it is triggered by attainment of the compressive capacity f_{ck} (that is, σ_c equal to f_{ck}), Eq. 4.25 applies. In the limiting situation where both limits are reached simultaneously, the appropriate governing relation is given by Eq. 4.26.

$$V_{u,FC,tension} = \left(f_{ctk} + \frac{N}{A} \right) \cdot \frac{2 \cdot I_{y,G}}{H_{crack} \cdot L_w} \quad \text{Eq. 4.24}$$

$$V_{u,FC,compression} = \left(f_{cd} - \frac{N}{A} \right) \cdot \frac{2 \cdot I_{y,G}}{H_{crack} \cdot L_w} \quad \text{Eq. 4.25}$$

$$V_{u,FC,lim} = (f_{ck} + f_{ctk}) \cdot \frac{I_{y,G}}{H_{crack} \cdot L_w} \quad \text{Eq. 4.26}$$

By applying Eq. 4.24–Eq. 4.26 to the case-study section and varying the axial force N , the flexure–axial resistance domain of the analysed 3D-printed concrete wall can be obtained. Accordingly, Figure 4.13 shows the resistance domain for the section, with the characteristic points highlighted that uniquely delineate the governing limit conditions.

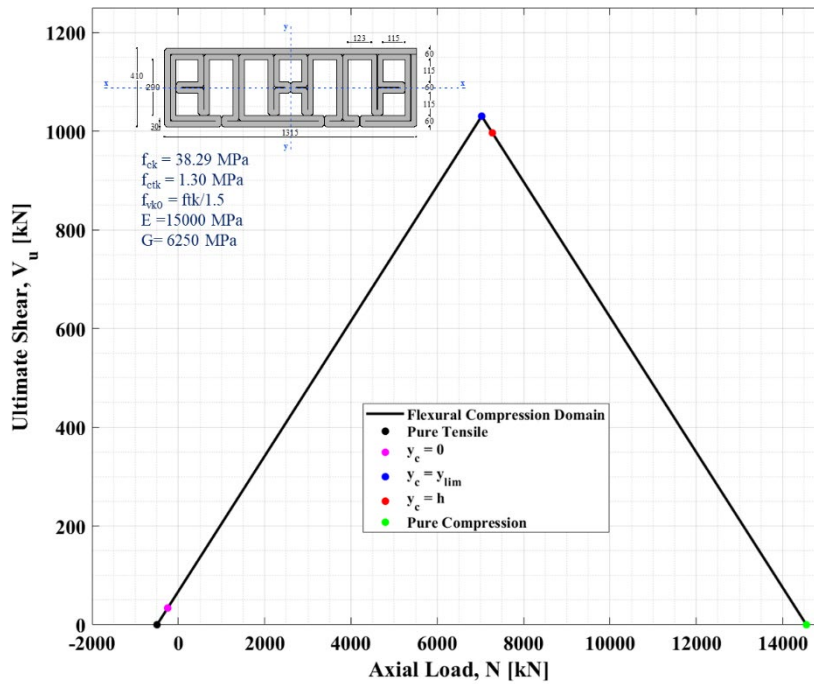


Figure 4.13. Resistance domain for flexural compression failure (FC) of the analysed section, with the domain's limit points marked.

To describe better the behaviour of the 3D-printed concrete panel under combined axial load and bending more transparently, it is convenient to construct a pointwise resistance domain. This representation clarifies the flexural–compression failure mechanism by marking a finite set of five representative points, each corresponding to a distinct limiting configuration of the section. In Figure 4.13, these five points are identified on the domain; in particular, the black point—representing a small-eccentricity section in pure tension—corresponds to the loading state illustrated in Figure 4.14.

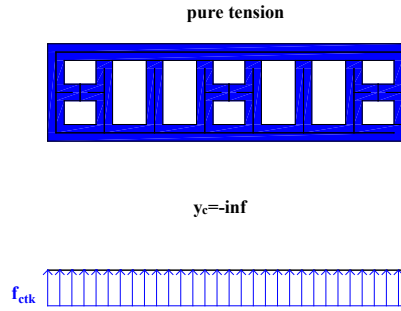


Figure 4.14. Loading configuration corresponding to the black point on the FC domain.

In particular, this corresponds to the case where the neutral axis is formally at $y_c \rightarrow -\infty$; the section is not reduced and the entire cross-section is effective in tension, reaching the tensile limit f_{ctk} . The black point on the domain is identified by N_I and V_I . The axial force equals the resultant of the tensile stress distribution, while the bending moment M —and hence the shear V —are zero under pure tension. The relations defining this point are given in Eq. 4.27 and Eq. 4.28.

$$N_I = A \cdot f_{ctk} \quad \text{Eq. 4.27}$$

$$V_I = 0 \quad \text{Eq. 4.28}$$

where A is the actual (net) area of the section and f_{ctk} is the tensile strength of the 3D-printed concrete material.

The stress state (still under traction stress) at the *magenta point*—the boundary between small and large eccentricity, with the section fully effective and maximum tensile stress at a single edge—is represented by the loading condition shown in Figure 4.15.

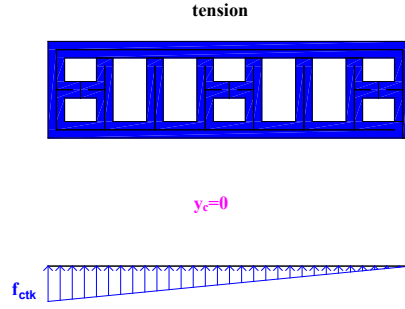


Figure 4.15. Loading configuration corresponding to the magenta point on the FC domain.

In this case, the neutral axis measured from the compressed edge, y_c , is zero.

The section therefore remains fully effective in tension, operating within a bending-dominated tensile regime — an intermediate condition between pure tension and flexure. The tensile limit f_{ctk} is reached only at the lower edge, while the remainder of the section continues to carry tensile stresses.

The magenta point on the domain is identified by N_{II} and V_{II} . The axial force equals the resultant of the triangular tensile stress distribution, whereas the shear follows from the Navier relations. Specifically, by appropriately combining Eq. 4.21 and recalling that the bending moment is taken as Eq. 4.16, one obtains the expressions reported in Eq. 4.29 and Eq. 4.30.

$$N_{II} = \frac{A \cdot f_{ctk}}{2} \quad \text{Eq. 4.29}$$

$$V_{II} = \frac{f_{ctk} \cdot I_{y,G}}{L_w \cdot H_{effective}} \quad \text{Eq. 4.30}$$

where A is the actual (net) area of the section; f_{ctk} is the tensile strength of the 3D printed concrete; $I_{y,G}$ is the centroidal second moment of area

about the axis orthogonal to the direction of the applied shear; L_w is the overall wall length, measured parallel to the direction of the applied shear; and $H_{\text{effective}}$ is the net height over which cracking develops in the anchorage region, equal to the difference between the total wall height H_{tot} and the anchorage height at the base H_{fix} .

The stress state at the blue point in Figure 4.13 corresponds to a partialized section under large eccentricity, engaging both compression and tension at the ultimate failure condition. It is represented by the loading configuration shown in Figure 4.16.

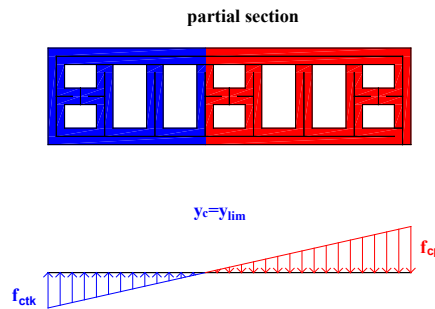


Figure 4.16. Loading configuration corresponding to the blue point on the FC domain.

In this case, the first step is to identify the limiting neutral-axis position measured from the compressed edge. By invoking the plane-sections (Navier–Bernoulli) assumption [**Errore. Il segnalibro non è definito.**], this value follows from triangle similarity and is given by Eq. 4.31. The corresponding stress distribution for the blue point on the domain is shown in Figure 4.16, whereas the neutral axis dept is provided by the following equation:

$$y_{lim} = \frac{f_{ck} \cdot L_w}{f_{ck} + f_{ckt}} \quad \text{Eq. 4.31}$$

In this case, the neutral axis measured from the compressed edge, y_c , is at its limiting position, so that the extreme stresses reach (simultaneously) the tensile and compressive strengths, f_{ctk} and f_{ck} , respectively. The section is therefore under combined tension and compression under large eccentricity. The blue point on the domain is identified by N_{III} and V_{III} . The equilibrium axial force is obtained as the difference between the compressive and tensile resultants, i.e., between the areas of the corresponding triangular stress distributions; the corresponding shear strength follows from the Navier relations through the bending moment. In particular, by combining (Eq. 4.20, Eq. 4.21) and recalling that the bending moment is evaluated as Eq. 4.16, subtracting Eq. 4.20 from Eq. 4.21 yields the expressions reported in Eq. 4.32 and Eq. 4.33.

$$N_{III} = \frac{A_{c,ylim} \cdot f_{ck}}{2} - \frac{A_{t,ylim} \cdot f_{ctk}}{2} \quad \text{Eq. 4.32}$$

$$V_{III} = \frac{f_{ck} + f_{ctk}}{L_w \cdot H_{effective}} I_{y,g} \quad \text{Eq. 4.33}$$

where $A_{c,ylim}$ is the area of the portion in compression, $A_{t,ylim}$ is the area of the portion in tension determined through the neutral axis depth at the equilibrium, L_w is the overall wall length measured parallel to the direction of the applied shear, and $H_{effective}$ is the net height over which cracking develops, equal to the difference between the total wall height H_{tot} and the anchorage height at the base H_{fix} . The stress state at the red point corresponds to the entire cross section subjected to compression with a triangular stress distribution as represented by the loading configuration shown in Figure 4.17.

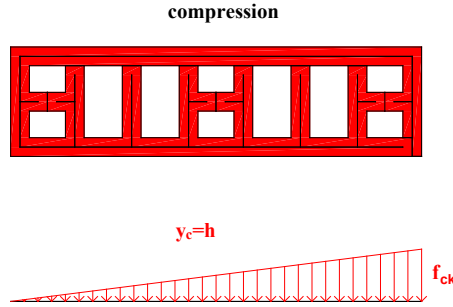


Figure 4.17. Loading configuration corresponding to the red point on the FC domain.

In this case, the neutral axis measured from the compressed edge, y_c , is located at the full section large L_w .

The section therefore operates within a bending-dominated compressive regime — an intermediate condition between pure compression and flexure. The entire cross-section remains active in compression, with the compressive limit f_{ck} being reached only at the upper edge.

The red point on the domain is identified by N_{IV} and V_{IV} . More specifically, the axial force corresponds to the resultant of the triangular compressive stress block, while the shear follows from the Navier relations. By appropriately combining Eq. 4.20 and recalling that the bending moment is evaluated as Eq. 4.16, one obtains the expressions reported in Eq. 4.34 and Eq. 4.35.

$$N_{IV} = \frac{A \cdot f_{ck}}{2} \quad \text{Eq. 4.34}$$

$$V_{IV} = \frac{f_{ck} \cdot I_{y,G}}{L_w \cdot H_{crack}} \quad \text{Eq. 4.35}$$

Finally, the stress state of the section at the green point—small eccentricity, fully effective section, pure compression—is represented by the loading condition shown in Eq. 4.16.

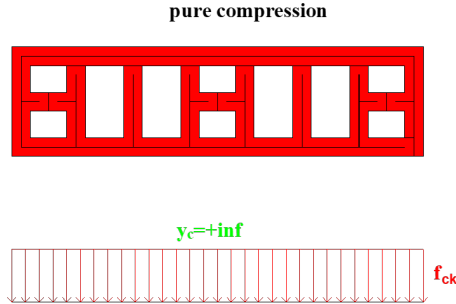


Figure 4.18. Loading configuration corresponding to the green point on the FC domain.

In this case, the neutral axis is at $+\infty$; the entire cross-section is effective in compression, reaching the compressive limit f_{ck} at both edges. The green point on the domain is identified by N_V and N_V . Specifically, the axial force equals the resultant of the compressive stress distribution, while the bending moment M —and hence the shear V —are zero under pure compression; accordingly, the relations defining the green point are given in Eq. 4.36 and Eq. 4.37.

$$N_V = A \cdot f_{ck} \quad \text{Eq. 4.36}$$

$$V_V = 0 \quad \text{Eq. 4.37}$$

4.5.2 Construction of the Resistance Domain for Diagonal Cracking (DC)

In the Turnšek–Čačovič model [20], when a structural wall is subjected to in-plane shear (as during an earthquake), inclined (diagonal) cracks develop across the panel. These cracks initiate when the tensile stress

induced by shear, σ_η , exceeds the material tensile strength f_{ctk} . Accordingly, the wall's shear behaviour can be described in terms of its tensile resistance along inclined fracture planes. Failure is deemed to occur when the maximum tensile stress in the section reaches the wall's tensile capacity—conceptually consistent with a Mohr–Coulomb-type interpretation for the onset expressed in tensile terms—i.e., when σ_η is equal to f_{ctk} . The tensile stress σ_η is obtained from Mohr's circle construction for the in-plane stress state. Considering a typical loading configuration of a wall under lateral actions, the shear force V applied at the top induces the stress components on an infinitesimal element as illustrated in Figure 4.19.

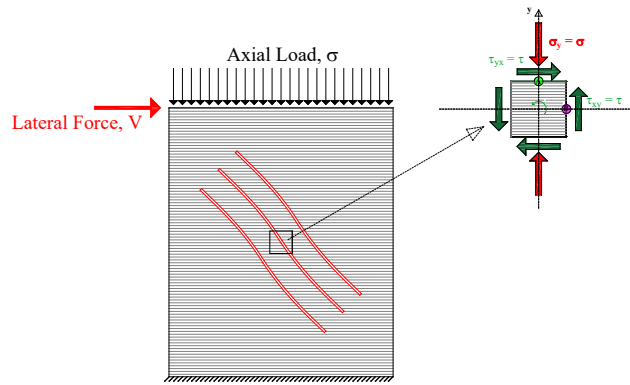


Figure 4.19. Stresses in an infinitesimal element of a 3D-printed concrete panel.

In the present case, it is assumed that one diagonal is in tension and the other in compression [20]. Consequently, the tensile stresses developing along the diagonal under traction lead to the formation of a crack orthogonal to the direction of tension. The crack initiates when the tensile stress σ_η equals the material's characteristic tensile strength f_{ctk} . For this failure mode, our goal is to determine the value of V_u such that σ_η is equal to f_{tk} . To this end, we consider an

infinitesimal element at the centre of the panel and denote the stresses acting on each face (with τ taken as positive in the clockwise sense), as shown in Figure 4.20. In this formulation, several assumptions are made: (i) the material is treated as monolithic, neglecting the presence of weak interlayer interfaces that may arise from large interlayer times; (ii) the wall thickness is represented by an equivalent solid section, disregarding the geometric effects of the internal void pattern; (iii) this assumption is supported by previous experimental observations on 3D-printed panels that exhibited monolithic-like shear failure despite the layered deposition process (see, e.g., [21]); and (iv) the proposed shear stress distribution is valid as long as the section remains uncracked in flexure — once tensile cracking occurs, the shear transfer mechanism and its distribution across the depth should be revised accordingly.

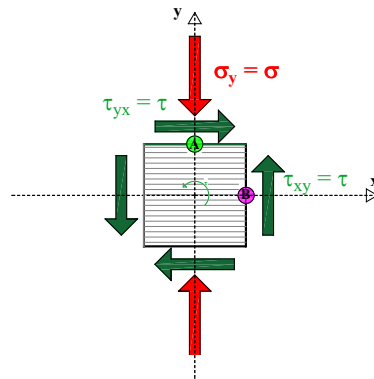


Figure 4.20. Stresses acting on an infinitesimal element at the centre of the 3DPC panel.

On the generic infinitesimal element, the normal stress σ is given by Eq. 4.38, while the shear stress τ is evaluated according to the Jourawski formulation Eq. 4.39.

$$\sigma = \frac{N}{A} \quad \text{Eq. 4.38}$$

$$\tau_{xy} = \frac{V_u S_y}{th_{eq} I_{y,G}} \Rightarrow \tau_{max} = \frac{V_{u,D.C.} S_{y,max}}{th_{eq} I_{y,G}} \quad \text{Eq. 4.39}$$

where th_{eq} denotes the panel equivalent thickness (see Eq. 4.5) , A is net (real) sectional area, $I_{y,G}$ is the centroidal second moment of area about the axis orthogonal to the direction of the applied shear, and $S_{y,max}$ is the maximum first moment of area of the portion of section above the generic fibre (chord). By constructing Mohr's circle, the principal tensile direction, σ_η , can be obtained. Figure 4.21 shows the generic stress state of the infinitesimal element at mid-depth ($L_w/2$) for the loading configuration depicted in Figure 4.20.

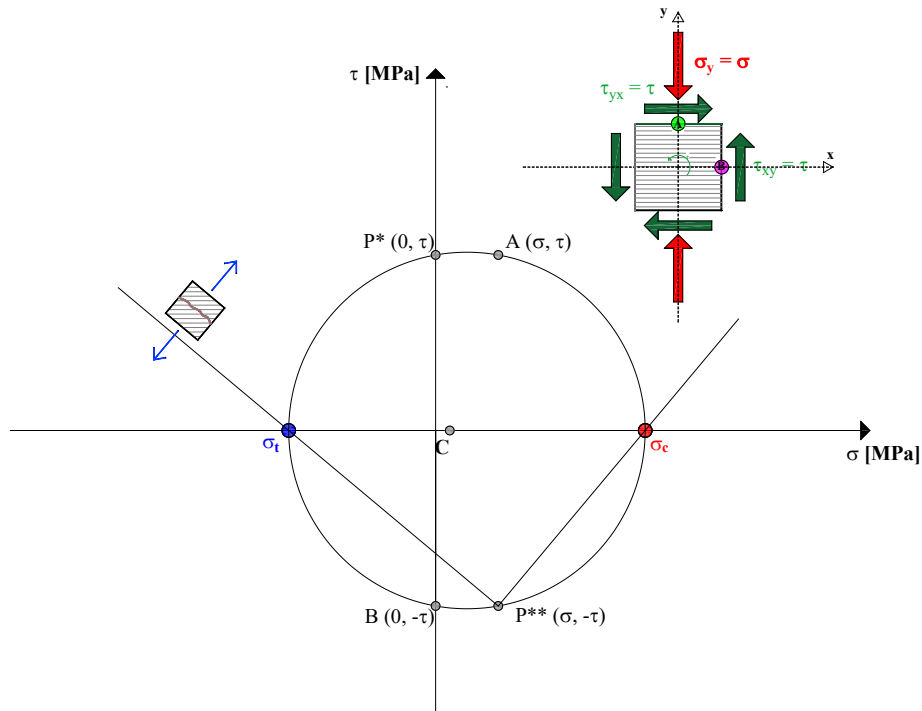


Figure 4.21. Representation of the stress state on Mohr's circle.

Once the principal tension and compression directions have been identified, Mohr's circle can be used—via straightforward geometric–mathematical constructions—to determine the circle center C (Eq. 4.40), the shear stress τ (Eq. 4.41), the tensile stress σ_η , or σ_t to simplify, (Eq. 4.42), and the circle radius r (Eq. 4.43), as well as the pole associated with the set of planes P^* and the pole of the normals P^{**} [**Errore. Il segnalibro non è definito.**].

$$C = \left(\frac{\sigma_x + \sigma_y}{2}, 0 \right) = \left(\frac{\sigma}{2}, 0 \right) \quad \text{Eq. 4.40}$$

$$\tau = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2} = \sqrt{\left(\frac{\sigma}{2} \right)^2 + \tau_{xy}^2} \quad \text{Eq. 4.41}$$

$$\sigma_t = \frac{\sigma}{2} + r \quad \text{Eq. 4.42}$$

$$r = \sqrt{\left(\frac{\sigma}{2} \right)^2 + \tau_{xy}^2} \quad \text{Eq. 4.43}$$

Accordingly, by setting the maximum principal tensile stress σ_t equal to the panel's tensile strength f_{ctk} , and after straightforward algebraic steps, one obtains Eq. 4.44, which represents the diagonal-cracking failure condition.

$$V_{U,D.C.} = \frac{I_{y,G} \cdot t h_{eq}}{S_{max,y,G}} \sqrt{\left(f_{ctk}^2 + \frac{N \cdot f_{ctk}}{A} \right)} \quad \text{Eq. 4.44}$$

where all quantities have already been discussed in the previous paragraph. Accordingly, by varying the axial force N while keeping the geometric and mechanical properties of the analysed section fixed, the resistance domain for diagonal-cracking failure is obtained, as shown in Figure 4.22.

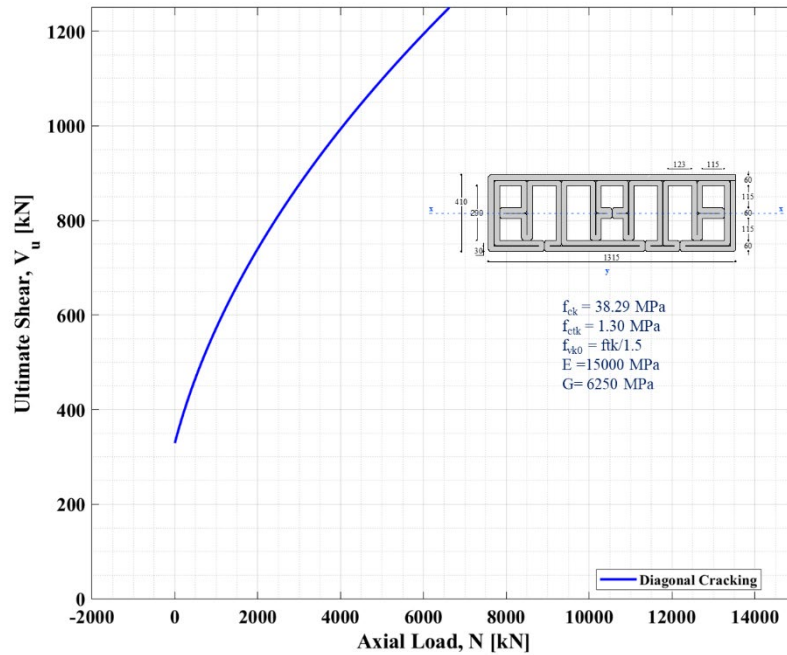


Figure 4.22. Resistance domain for diagonal-cracking (DC) failure.

4.5.3 Construction of the Resistance Domain for Shear Sliding (SS)

Failure is assumed to initiate when the shear sliding capacity of the panel is reached. Specifically, sliding is taken to occur along the interface between two printed layers across of the base restraint. In this case, sliding develops when the shear stress τ acting on the prospective slip plane equals the panel's sliding resistance, taken as $f_{vk0} + \mu \sigma$. However, this formulation refers to a global sliding mechanism, analogous to that typically adopted for masonry walls. In the case of 3D-printed concrete, a local shear resistance also exists at the material scale, governed by the bond and cohesion between

adjacent printed layers. Therefore, it would be more appropriate to consider the minimum between the global sliding resistance, f_{vk0} , and the interlayer shear strength, c . For this failure mode, the objective is to determine the ultimate shear V_u such that τ_u , in accordance with Eq. 4.45, where f_{vk0} denotes the cohesive term, μ is the friction coefficient, and σ is the normal stress on the interface.

$$\tau_u = c + \mu\sigma = f_{vk0} + \mu\sigma \quad \text{Eq. 4.45}$$

The pure shear capacity of the panel, f_{vk0} , is obtained from shear tests conducted in accordance with ASTM [22] procedures or from other studies in the literature [23]. As before, it is necessary to distinguish between small and large eccentricity. In the small-eccentricity case, the entire section is effective, and the ultimate shear for shear sliding, $V_{u,SS}$, is obtained directly by applying Eq. 4.46.

$$\tau_u = \frac{V_{u,SS}}{A} = f_{vk0} + \mu \cdot \frac{N_{ext}}{A} \quad \text{Eq. 4.46}$$

where A is the actual (net) cross-sectional area; f_{vk0} is the panel's pure shear strength in the absence of axial stress (i.e., the cohesive term, c); μ is the internal friction coefficient (equal to $\tan$ of ϕ' , with ϕ' the internal friction angle—typically 30° – 45° for unreinforced concrete according to Eurocode 2 [1], and possibly 20° – 30° for weaker cementitious materials [24;25]). σ is the applied normal stress, given by the ratio of the external axial load N_{ext} to the reacting area; for fully effective cross section this corresponds with the total actual area of the panel. Furthermore, τ_u is the ultimate shear stress, given by the ratio of the ultimate lateral shear force V_u to the reacting area; again,

for a fully effective (in compression) cross-section this equals with the panel's total actual area.

It is worth noting that, for 3D-printed concrete (3DPC) walls, the value of f_{vk0} can vary appreciably with the test method used to determine it. In particular, diagonal compression/shear tests on printed panels often yield an apparent cohesion different from that obtained from material-level tests. This is because diagonal tests are frequently governed by “solid” material tension rather than by sliding along interlayer interfaces [23]. By contrast, in traditional masonry, diagonal cracking typically follows the mortar joint due to interface activation — as documented by Lourenço (1996) and Page (1978) [26, 27] — so failure develops along predefined weakness planes (the joints), where low cohesion and limited friction dominate. In 3DPC, however, the interlayer shear strength is often greater than the matrix tensile strength, especially when the interlayer time is very short (1.5-2.5). Cracks therefore cut through the printed material rather than propagating along the interface, indicating a tension-governed rather than a sliding-governed failure. As a result, the printed block behaves more monolithically (again, in the presence of a strong interlayer bond), and the f_{vk0} measured in this context reflects an effective cohesion that should not be conflated with values obtained on weak joints or activated interfaces. This peculiarity has also been highlighted in recent studies on diagonally loaded 3D-printed panels, where interface activation is observed only when the matrix tensile strength is high or interlaminar cohesion is weak [24;28].

In the large-eccentricity case—i.e., with a combined tension and compression—only the compressed zone contributes to the shear resistance. Accordingly, the relation to use is Eq. 4.47.

$$\tau_u = \frac{V_u}{A_{compressive}} = f_{vk0} + \mu \cdot \frac{N}{A_{compressive}} \quad \text{Eq. 4.47}$$

where $A_{compressive}$ is the area of the compressed portion of the section and thus a function of the neutral-axis position y_c for each pair of applied external actions. Consequently—just as in the flexure–axial checks, where the material tensile strength plays a key role—it is essential to determine the position of the neutral axis y_c . Under the assumptions of small displacements and plane sections remaining plane (Bernoulli hypothesis [**Errore. Il segnalibro non è definito.**]), y_c can be evaluated as a function of the external loading state defined by the bending moment M and the axial force N . In particular, by adopting a Navier-type stress distribution (Eq. 4.20 and Eq. 4.21) and using triangle similarity as illustrated in Figure 4.11, one obtains Eq. 4.48.

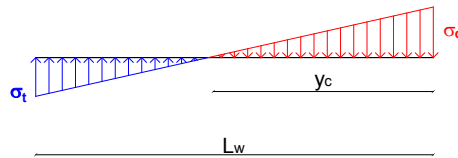


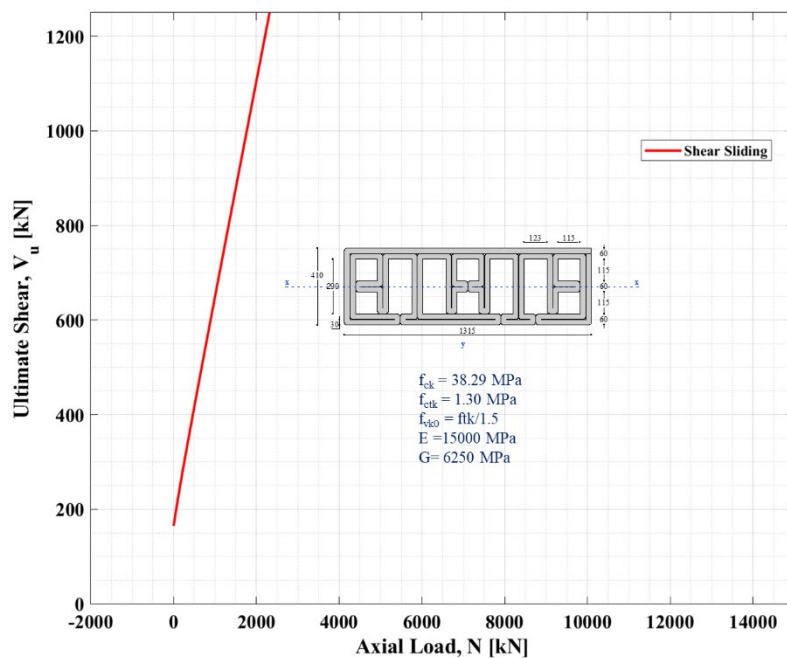
Figure 4.23. Stress distribution in the generic large-eccentricity configuration.

$$y_c = \frac{\sigma_c \cdot L_w}{\sigma_c + \sigma_t} \quad \text{Eq. 4.48}$$

By applying the Navier relations and noting that the compression area A_c equals the product of the wall's equivalent mean thickness t_{eq} and the neutral-axis depth y_c , substitution into Eq. 4.48 yields the explicit form reported in Eq. 4.49, and thus the expression for the ultimate shear under shear-sliding failure, $V_{u,SS}$, as a function of the axial force N .

$$V_{u,SS} = \frac{\left(\frac{f_{vk0} \cdot t_{eq} \cdot L_w \cdot H_{eff}}{2} + \mu \cdot N \cdot H_{crack} \right) + \sqrt{\left(\frac{f_{vk0} \cdot t_{eq} \cdot L_w \cdot H_{eff}}{2} + \mu \cdot N \cdot H_{eff} \right)^2 + 4 \cdot H_{crack} \cdot \frac{f_{vk0} \cdot h_{eq} \cdot N \cdot I_{yc}}{A}}}{2 \cdot H_{eff}} \quad \text{Eq. 4.49}$$

where A is the total actual cross-sectional area; f_{vk0} is the panel's pure shear strength in the absence of axial stress (i.e., the cohesive term c [23]); μ is the internal friction coefficient, to be obtained from direct shear tests (it is equal to $\tan \phi'$, with ϕ' typically in the range 30–45° for unreinforced cementitious materials [25]); σ is the applied normal stress, given by the ratio of the external axial load N to the reacting area. By applying 4.47 and Eq. 4.49, the shear-sliding resistance domain shown in Figure 4.24 is obtained.



4.6 Experimental Comparison and Discussion: Influence of slenderness and Axial Load

To compare the three failure modes examined—flexure compression, shear-sliding, and diagonal cracking—for the element type under study, a Matlab-based script was developed to create a resistance domain. The overall results for a wall panel type with cross section described in section 4.5 are summarised in the plot of Figure 4.25. In this context, the shear resistance, f_{vk0} , was computed starting from the reference tensile strength f_{tk} , reduced by a factor of 1.5, consistently with the approach adopted for masonry-like materials [19]. It is worth noting that this formulation represents a global sliding mechanism,

whereas the local shear strength between printed layers may be governed by the intrinsic material properties (cohesion and friction) rather than the overall compressive state. Therefore, in defining the effective shear resistance, the minimum value between the interlayer shear strength and the diagonal compression strength was considered, ensuring a conservative and physically consistent representation of the failure envelope.

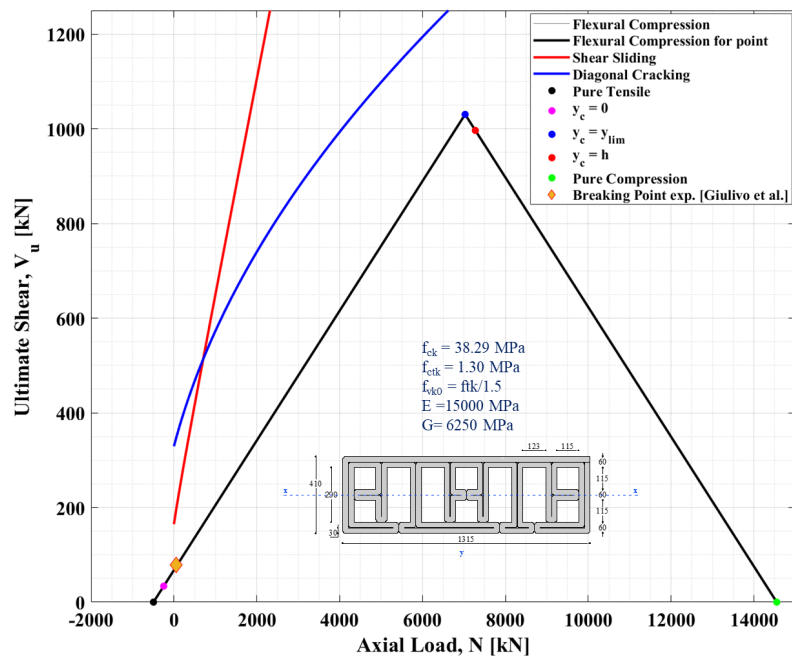


Figure 4.25. Failure domains for unreinforced 3D-printed concrete (3DPC) panels.

From the plot in Figure 4.25, it can be seen that, for the section type analysed, the governing failure mechanism is mostly flexure compression (FC) (black curve), essentially independent of the applied axial force N . This indicates that the distribution of internal stresses—driven by the wall's geometric configuration—systematically favours a

flexure-controlled collapse, probably due to a major role played by of traction/interlayer strengths. Comparing these results with the elastic-limit maximum shear obtained experimentally in the chapter 3, an excellent match is observed: the experimental point (shown in ochre/yellow in Figure 4.25) coincides with the analytically predicted value, thereby validating the adopted model.

To better understand how the failure mode and the ultimate lateral capacity V_u vary with geometry and axial force, a parametric analysis is performed by varying the slenderness λ — defined as the ratio between the total wall height H_{tot} and the wall length L_w — and the axial force N , while keeping the wall thickness t_{ext} constant. The plot in Figure 4.26 reports the results: as λ and N change, so does the ultimate shear V_u , computed for each failure mechanism considered — flexure compression (black markers), diagonal cracking (blue markers), and shear-sliding (red markers). Each curve in the figure traces the evolution of response for a different applied value of N .

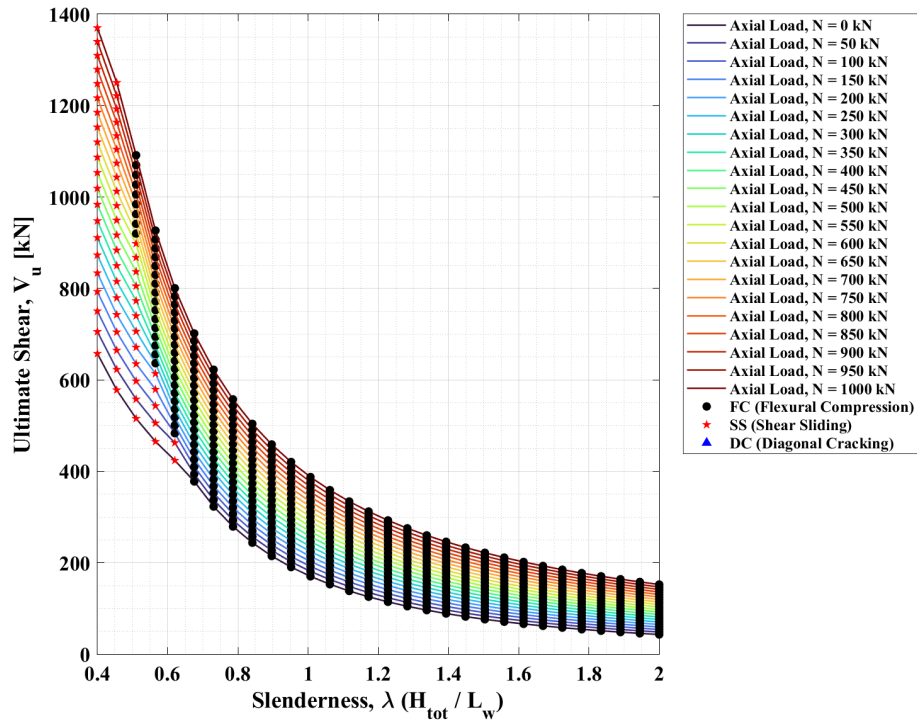


Figure 4.26. Variation of ultimate shear and failure mechanisms with slenderness λ and axial force N , with wall thickness held constant.

From the plot in Figure 4.26, it is observed that for walls with slenderness λ equal to 1.6, and for varying axial force N , failure is consistently governed by flexure (tensile cracking in the outer fibre) compression with axial load. This holds for larger slenderness provided that the internal infill is fixed—i.e., the void ratio is kept constant at 40% of the gross cross-section—and apart from minor variations in the ultimate shear value. For this section type, the transition from flexure compression failure to shear-sliding occurs predominantly at slenderness values below about 0.62, especially for lower axial loads, as also highlighted in Figure 4.26. On this basis, an

operational distinction for 3D-printed concrete panels of the present type (rectangular infill, 40% voids) can be proposed: a panel is slender if λ is major than 0.7 and stocky if λ is minor than 0.7. Moreover, as shown in Figure 4.27, a zoomed-out view of the same plot—including a larger range of axial load combinations up to 10000 kN (not typical for ordinary applications but plausible under vertical precompression)—shows that the ultimate shear capacity tends to decrease once the axial load exceeds about 7000 kN. This threshold corresponds to the peak of the flexure–axial resistance domain already identified in Figure 4.25. The effect is particularly evident for walls with slenderness greater than 1.3, suggesting a significant influence of geometry on the transition from a flexure compression dominated to a shear-sliding dominated regime.

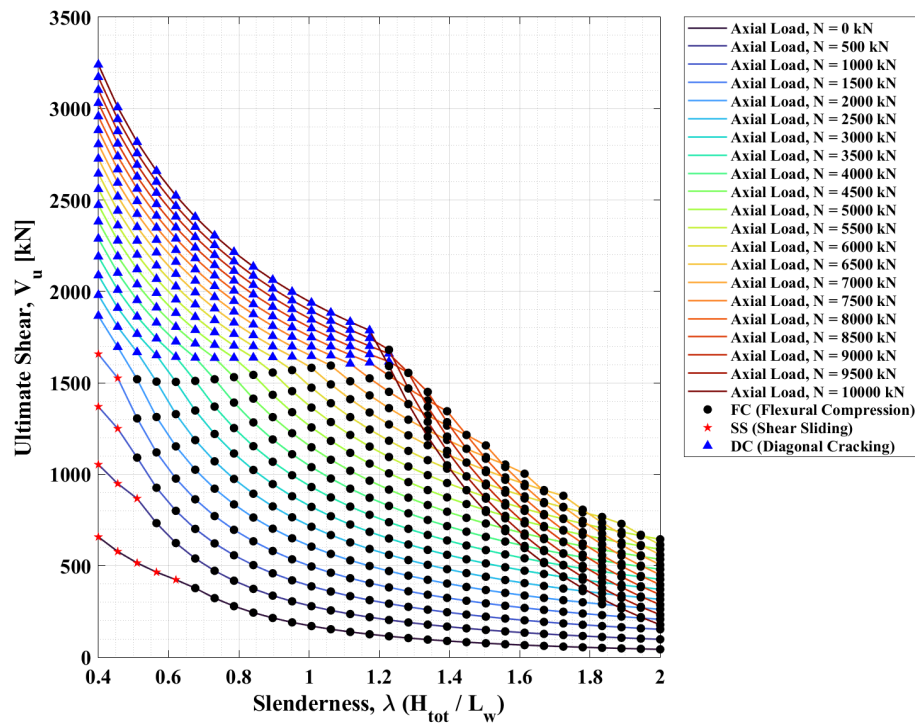


Figure 4.27. Zoomed-out view of the variation in ultimate shear and failure mechanisms as slenderness λ and axial force N vary, with wall thickness held constant.

It is worth emphasizing that slenderness values below about 0.6-7.7 can be representative of on-site 3D-printed concrete (3DPC) walls, i.e., elements produced directly in place by continuous, non-modular printing. In such configurations, wall lengths/widths are generally large relative to height, which reduces the slenderness ratio λ . These stocky, joint-free walls tend to exhibit failure mechanisms governed primarily by shear-sliding or diagonal cracking, especially under high axial load. This behaviour is consistent with the discussion in Paulay and Priestley (1992) [29], who note that squat walls are generally prone to

brittle, shear-dominated failures rather than flexure. Similar trends are reported in other study [30] on the shear capacity of reinforced concrete masonry walls, which confirms the reduction in flexural effectiveness as slenderness decreases. In the context of 3D printing, Wolfs et al. (2019) [31] on the early-age mechanical behaviour of 3D-printed concrete show how process-induced features (e.g., interlayer effects, early-age strength development) can favour more brittle response modes under combined tension and shear. Likewise, Le et al. (2012) [32]—focusing on mix design and fresh properties for printable concretes—highlight process-driven anisotropy, which, when carried to the structural scale, may accentuate the propensity for sliding or diagonal-tension failure in continuous, on-site prints. Finally, a recent contribution that aligns closely with the present work is the experimental study by Mercimek et al. (2025) [33] on unreinforced 3D-printed concrete (3DPC) walls tested in-plane under varying process and loading conditions. Their campaign [33] isolates three variables that are central to our framework: the presence or absence of cold joints introduced during printing, the level of axial load, and the use of cyclic versus monotonic lateral loading. Across nominally identical wall geometries, the authors report that removing cold joints systematically enhances peak lateral capacity and stabilises the response, whereas intentionally introduced cold joints shift the behaviour toward more brittle mechanisms concentrated near the base.

It should be noted that all parametric plots reported in this section are constructed assuming a constant void ratio for the reference cross-section, equal to 40% of the cross-sectional area. This choice ensures

consistency with the section analysed in the previous domain-based framework, where the same level of porosity was adopted. By fixing the percentage of voids, the influence of slenderness, axial force, can be isolated without the additional variability introduced by changing compactness, or geometry shape, of the section..

4.7 Influence of Tensile Behavior of 3dpc and Comparison with Traditional Masonry

The tensile strength used in the analytical models derives from the experimental characterisation performed on printed specimens, are described in chapter 2 and 3. Some sources of uncertainty inevitably affect the estimation of tensile strength in 3D-printed concrete, including: (i) variability in interlayer time (ILT), which directly influences the quality of interlayer bonding; (ii) fresh-state defects or pores aligned along filament boundaries; (iii) sensitivity of flexural and splitting tests to specimen geometry; and (iv) environmental variations during printing and curing. For these reasons, the tensile strength incorporated into the analytical models corresponds to a conservative characteristic value, f_{ctk} , equal to almost 1.5 MPa. This ensures that the predicted cracking and failure mechanisms remain realistic and safely representative of the printed walls analysed in this work, and—crucially—consistent with the experimental evidence reported in Chapter 3. Infact, in the in-plane testing campaign, the walls exhibited an elastic tensile capacity (onset of cracking) of approximately 1.5 MPa, confirming that the characteristic tensile value adopted in the

analytical models is not only conservative but also fully aligned with the tensile behaviour observed experimentally at panel scale.

In order to compare unreinforced 3D-printed concrete (3DPC) walls with conventional unreinforced masonry, the central role of tensile participation in representing the resisting mechanisms becomes clear. In traditional masonry, both NTC 2018 [34] and Eurocode 6[**Errore. Il segnalibro non è definito.**] treat the material as unable to carry tension; accordingly, in flexure–compression checks the resistance domain starts at the origin (0, 0), implicitly assuming no tensile contribution (see also [19]). For 3DPC walls, by contrast, neglecting tensile strength leads to a marked underestimation of the true capacity, as shown by both analytical calculations and tests reported in the previous chapters. When the tensile term is omitted in constructing the flexure–axial resistance domain, the domain shifts downward (Figure 4.28), so failure is predicted prematurely for any level of applied axial load N . This underestimation (approximately three times lower) can, in turn, drive unnecessary overdesign of the section [12], also in the perspective of topology optimization of the element. The difference between the two approaches is particularly evident in subfigures (a) and (b). In Figure 4.28(a) the two flexure–axial domains are compared: the blue curve represents the model including tensile strength (f_{ctk}), whereas the green curve reproduces the masonry-style assumption in which tension is ignored. As shown, the tension-inclusive domain captures more faithfully the mechanical response of printed walls, which—thanks to material continuity across layers and adequate interlayer bond—exhibit a non-negligible tensile

contribution. Figure 4.28(b) magnifies the critical region, making the divergence between the two models immediately apparent and underscoring the need to model tensile effects explicitly when assessing 3DPC wall panels. Infact, for an axial load of almost 250 kN, the predicted ultimate shear is approximately 120 kN when tensile behaviour is included, compared to only 40 kN when it is neglected. This clearly highlights the divergence between the two models, demonstrating that neglecting tensile effects can lead to an underestimation of the shear capacity by roughly a factor of two. Such evidence underscores the importance of explicitly modelling tensile behaviour when assessing the load-bearing capacity of 3D-printed concrete wall panels.

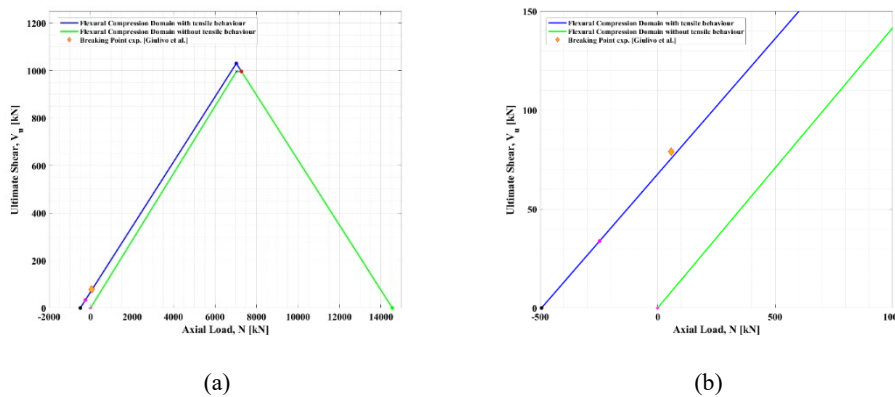


Figure 4.28. (a) Comparison of flexure-controlled failure with and without tensile contribution; (b) zoomed view.

In Figure 4.29, a comparison is presented between all masonry typologies covered by NTC 2018[34] and Eurocode 6[**Errore. Il segnalibro non è definito.**] and 3D-printed concrete (3DPC) panels herein developed, with and without the tensile contribution included in

the calculation of the ultimate shear. For all masonry cases, an axial load, N , equal to 58 kN is applied, on the same way with the discussion in the previous chapter (experimental test, chapter 3). The comparison is performed at equal plan footprint, i.e., keeping the external geometric properties of the cross-section constant, and using the code-specified mechanical properties for traditional masonry, as calculated in chapter 3 without tension strength as masonry, together with the material parameters adopted in this study for the 3D-printed walls.

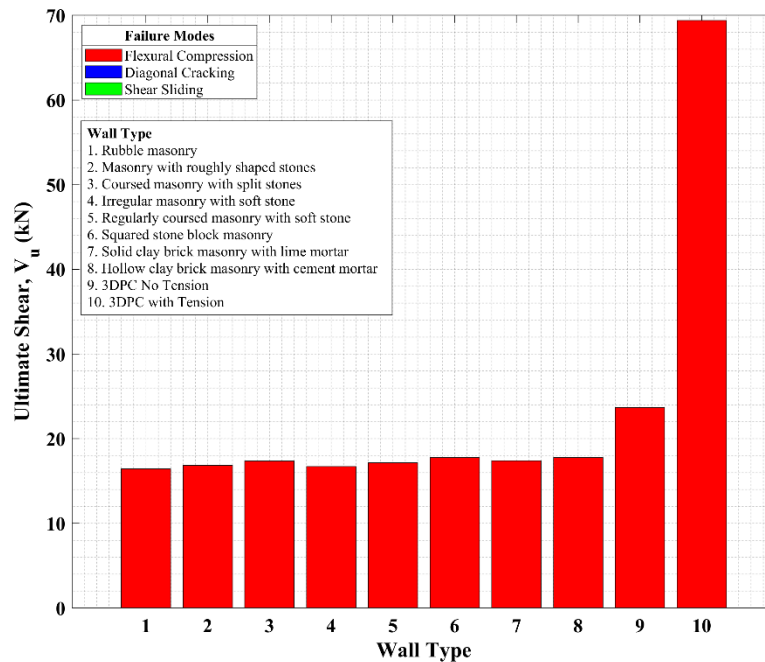


Figure 4.29. (a) Comparison of flexure-controlled failure with and without tensile contribution; (b) zoomed view.

From the bar chart in Figure 4.29, it emerges that for all conventional masonry typologies considered (Wall Types 1–8), the governing failure

mechanism is flexure compression, as indicated by the red color (FC). The corresponding ultimate shear values are fairly similar across these masonry walls, a condition that can be attributed to the dominant role of section geometry and slenderness—parameters that strongly influence capacity regardless of the specific masonry type adopted [29]. By contrast, Wall Types 9 and 10, representing 3D-printed concrete (3DPC) panels without and with tensile contribution respectively, display a distinctly different response. In particular, Wall Type 10—which includes tensile strength in the model—exhibits a significantly higher capacity, capturing more realistically the ultimate shear developed by the wall, as specified in chapter 3[12]. This difference confirms that including tensile contribution in analytical models for 3DPC elements is essential to avoid substantial underestimation and to represent the actual failure mechanisms correctly.

4.8 Influence of Interlayer time (ILT): Variation of tensile strength and Ultimate shear from slenderness

At this stage, with the aim of generalizing the structural behaviour of the investigated elements as much as possible, the analysis was extended by introducing the effect of a variable interlayer time (ILT). As already discussed in the section dedicated to material characterization, the ILT exerts a significant influence on the mechanical properties of 3D printed elements, as it directly affects the

bonding quality between adjacent layers and, consequently, the global response of the structure (see Eq. 2.2, Eq. 2.3, Eq. 2.4 and Eq. 2.5). In order to evaluate the variation of tensile strength with increasing Interlayer Time (ILT), reference values of tensile strength equal to 3.0, 2.5, 2.0, 1.5, 1.0 and 0.5 MPa were selected. Each of these values corresponds to a deposition interval Δt estimated through the exponential decay law expressed in Eq. 2.1 and Eq. 2.2 (for example Chen et al data [35]).

These assumptions are consistent with results reported in the literature [35]. Previous studies have shown that the bond strength between layers decreases significantly after only a few minutes of delay and stabilizes at a lower plateau within a few tens of minutes [35]. Systematic investigations on printable mortars at ambient temperature reported time constants in the range of approximately ten to thirty minutes. In the plotted curve in Figure 4.30, the selected tensile strength targets were used to determine the corresponding interlayer times. This approach provides a clear representation of the effect of interlayer time on tensile strength.

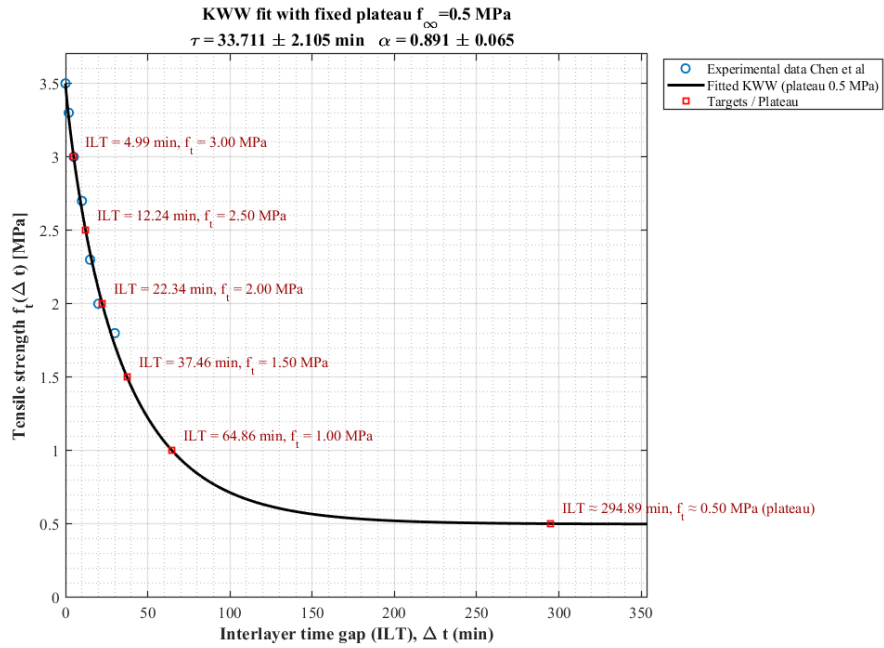


Figure 4.30. Tensile strength decay as a function of interlayer time (ILT). Reference values of tensile strength from 3.0 to 0.5 MPa were selected to determine the corresponding deposition intervals Δt according to the exponential decay law. The adopted parameters are α almost equal to 0.9, τ almost equal to 33 min.

In the specific configuration considered, it was found that the first mechanism to be activated is flexural compression, which therefore governs the failure process. Building on this evidence, and in order to move beyond the sole variation of the element slenderness already presented in Figure 4.31, a parametric analysis was conducted in which the tensile strength of the material was varied within a fixed interval ranging from 0.5 to 3 MPa, while the axial force was kept constant at 50 kN. This choice makes it possible to highlight the direct correlation between the tensile properties of the material and the shear capacity of the wall, isolating the influence of ILT-related changes in tensile resistance from other variables. The results of this extended

analysis are reported in Figure 4.26, where the ultimate shear strength, V_u , is represented as a function of both the tensile strength and the slenderness of the element, for the fixed axial load condition. The graph shows that, as expected, higher tensile strength values translate into higher ultimate shear capacities, confirming the strong dependency of the overall behaviour on the material's tensile resistance. However, even with this increase in strength, the dominant failure mode remains flexural compression, which consistently governs the collapse mechanism across the entire investigated range. For this reason, Figure 4.31 provides additional insight into this aspect, since it does not merely represent the resistance domains but also identifies, through markers placed along the curves, the specific failure mechanism activated at each combination of tensile strength and slenderness. In this way, the interpretation of the results becomes more immediate: the regions associated with flexural compression, shear sliding, and diagonal cracking are clearly distinguished, and it is possible to appreciate that the first of these mechanisms always prevails under the examined conditions. This graphical evidence further reinforces the conclusion that, although an increase in tensile strength enhances the absolute resistance of the walls, the critical mechanism governing their safety remains flexural compression, a behaviour consistent with the inherently brittle nature of the material.

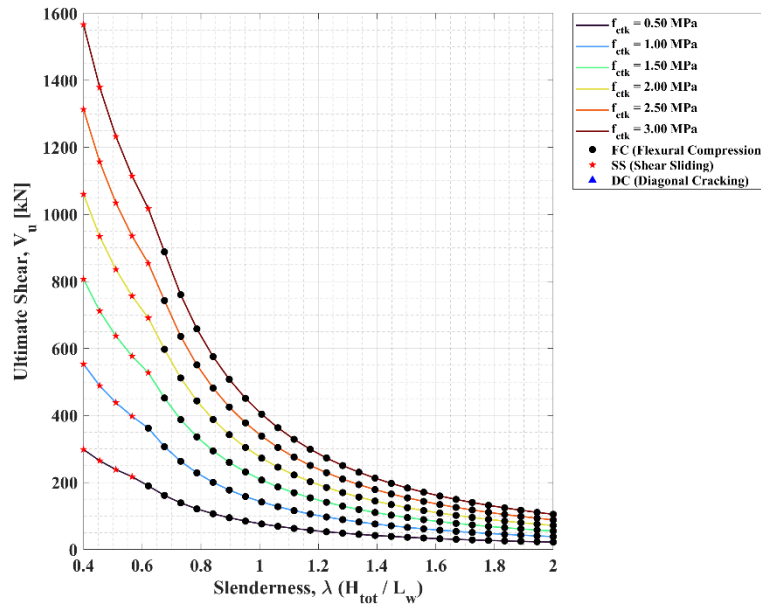


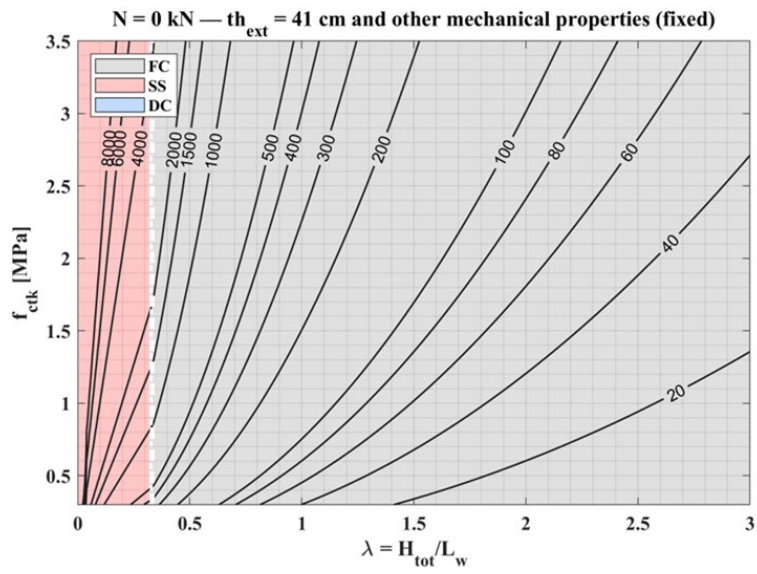
Figure 4.31. Parametric analysis of the ultimate shear capacity V_u as a function of the wall slenderness, λ , for a fixed axial load equal to 50kN and different tensile strength values ranging from 0.5 to 3.0 MPa. The curves highlight the progressive increase in ultimate shear with higher tensile strength, while the markers indicate the governing failure mechanisms for each configuration: flexural compression (FC, black circles), shear sliding (SS, red stars), and diagonal cracking (DC, blue triangles).

Therefore, the analyses presented in this section provide clear evidence of the decisive role played by the interlayer time in governing the structural response of 3D printed concrete elements. By modelling its effect through the variation of tensile strength, it was shown that the ultimate shear capacity of the wall increases consistently with higher tensile resistance values. Nevertheless, despite this beneficial influence, the flexural compression mechanism systematically emerges as the dominant failure mode across the entire investigated domain of slenderness and axial load. This outcome highlights two complementary aspects. On the one hand, it confirms the sensitivity of

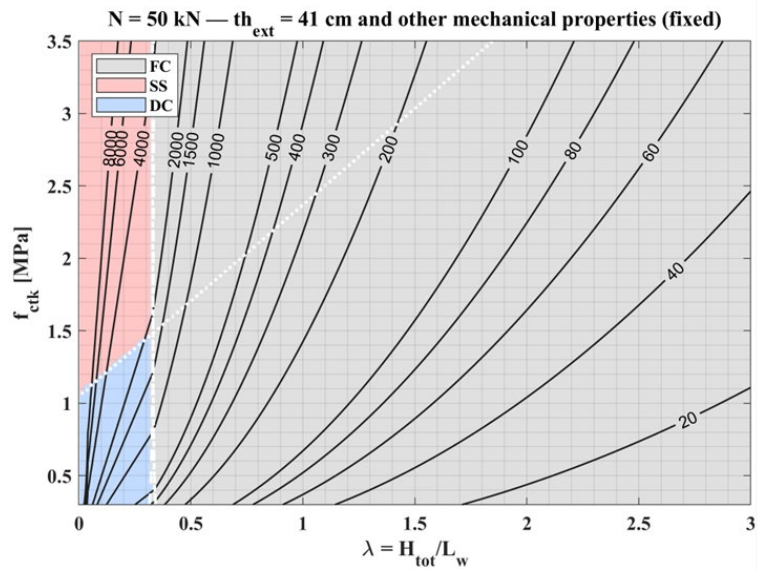
3D printed structures to the tensile properties imparted by the printing process and the interlayer bond quality, thereby underlining the importance of carefully controlling the interlayer time during fabrication. On the other hand, it stresses that, even in the presence of enhanced tensile strength, the ultimate safety of the element remains governed by flexural compression, reflecting the inherent fragility of plain 3D printed concrete.

The combined representation of slenderness, tensile resistance and ultimate shear capacity therefore provides a general framework to assess the structural reliability of printed walls under different interlayer conditions. This framework not only supports the interpretation of experimental outcomes, but also offers a rational basis for design-oriented modelling where the variability induced by the printing process must be explicitly taken into account. To further extend the scope of the investigation, the analysis was subsequently broadened by allowing the axial force to vary, in addition to the tensile resistance and slenderness already considered. This step makes it possible to reproduce a wider spectrum of boundary conditions, thereby reflecting more realistic design scenarios in which the vertical load acting on the wall is not constant but can vary significantly depending on the structural configuration. The resulting design abacus, reported in Figure 4.32, provides a comprehensive overview of the governing failure modes across the three-dimensional parameter space defined by axial load, tensile strength, and slenderness. In these maps, constructed for representative values of the axial force (N equal to 0 see Figure 4.32(a), N equal to 50 kN see

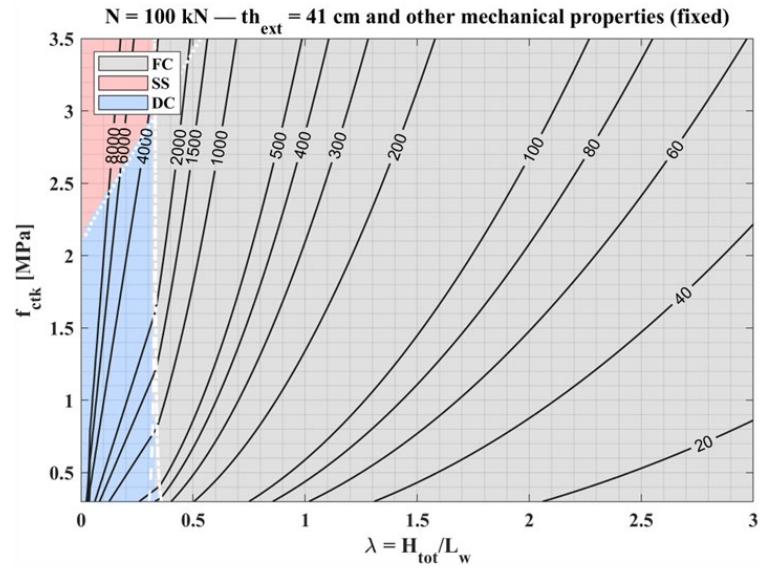
Figure 4.32(b), N equal to 100 kN see Figure 4.32(c), N equal to 150 kN see Figure 4.32(d)), the background shading identifies the governing failure mechanism at each point: flexural compression (FC, grey), shear sliding (SS, light red), and diagonal cracking (DC, light blue). The iso- V_u curves, plotted with progressive spacing, allow the ultimate shear capacity to be quantified within each region, making the abacus directly applicable for design-oriented assessments. The comparison between the four level of axial load clearly shows the stabilising effect of the axial load. As N increases, the region associated with shear sliding progressively shrinks due to the enhanced contribution of friction at the interfaces, while the diagonal cracking domain remains confined to low values of tensile strength and slenderness. Flexural compression, on the other hand, systematically governs the response over the vast majority of the parameter space, particularly at medium-to-high slenderness ratios. This consistent dominance of flexural compression confirms once again the intrinsic fragility of plain 3D printed concrete elements, whose structural safety is critically limited by the onset of crushing at the compressed edge, regardless of improvements in tensile resistance or confinement provided by the axial load.



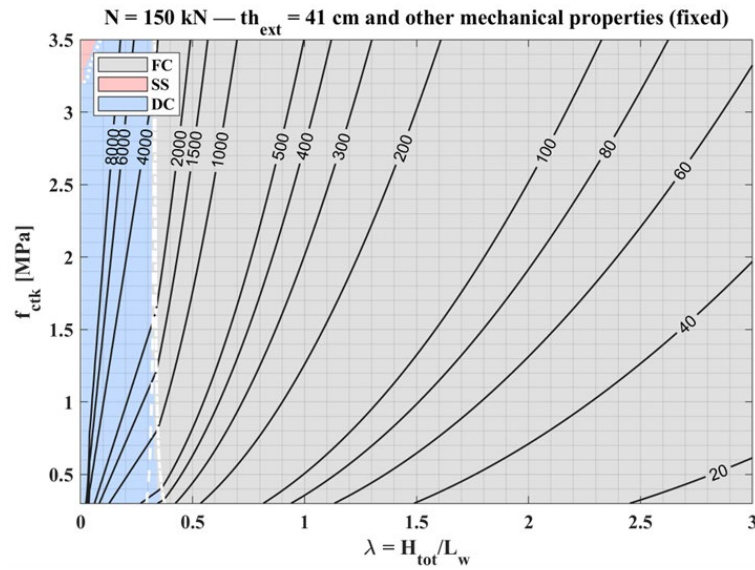
(a)



(b)



(c)



(d)

Figure 4.32. Design abacus of the governing failure modes in the $\lambda - f_{ctk}$ domain for walls with fixed external thickness $th_{ext}=41$ cm and constant mechanical parameters (μ, f_{vk0}, E, G). The shaded regions indicate the governing mechanism—flexural compression (FC, grey), shear sliding (SS, light red), and diagonal cracking (DC, light blue)—while the iso- V_u curves (kN) illustrate the ultimate shear

capacity correspond to different axial load levels: (a) $N=0$ kN, (b) $N=50$ kN, (c) $N=100$ kN, and (d) $N=150$ kN.

In all the abaci presented, the wall section was assumed with a constant porosity, corresponding to a void ratio of 40%. This assumption ensures full consistency with the resistance domains previously discussed, where the same reference section was analysed. By keeping the void percentage fixed, the influence of the interlayer time and of the other mechanical parameters can be isolated, avoiding that variations in the geometric compactness of the section affect the interpretation of the results. In this way, the derived abaci provide a general framework that is directly comparable to the resistance domains of this section, while remaining representative of a realistic 3D printed configuration with a constant level of porosity.

4.9 Conclusions

The analysis has enabled a detailed definition of resistance domains for 3D-printed concrete (3DPC) walls subjected to in-plane actions. It was developed analytical models that represent the three principal collapse modes—flexural compression, shear-sliding, and diagonal cracking—by consistently combining the material's mechanical properties with the section geometry. Expressed as functions of the applied axial force, these domains were validated against available experimental data and proved effective in capturing transitions between governing mechanisms. In particular, for the axial load level investigated experimentally (≈ 58 kN), the printed walls exhibited a flexural–tensile failure, with an elastic cracking onset of approximately

1.5 MPa, followed by post-peak degradation governed by tensile–shear separation. This behaviour corresponds to the central region of the analytical domains, confirming that this portion is experimentally validated and mechanically consistent.

By contrast, the boundaries of the domains—where shear-sliding or diagonal cracking govern—have not yet been experimentally reached. According to the analytical framework, reducing the axial load N decreases frictional resistance and shifts the wall response towards shear-sliding, especially for walls with low slenderness, where sliding mechanisms are more easily activated. Conversely, increasing the axial load N enhances confinement and tends to promote diagonal cracking, as the shear-compression interaction mobilises inclined compressive struts that may eventually fail in diagonal tension. These predictions remain plausible from a mechanical standpoint, but require experimental confirmation.

For this reason, a natural extension of this work is the execution of multi-level axial load tests on walls with the same and different geometry, to verify whether lower N triggers the onset of shear-sliding and whether higher N leads to compression–shear diagonal cracking, thereby validating the predicted transitions in governing mechanisms and refining the realism of the numerical boundaries.

A key outcome is the governing role of wall slenderness, λ . For slender walls greater than 0.7, the prevailing mechanism is flexure compression, whereas lower values—typical of on-site, wide-footprint prints—favour more brittle modes such as shear-sliding and diagonal

tension. This transition, clearly visible in the computed domains, provides a practical geometric threshold for design decisions. When compared with standard masonry at equal plan area and axial load, the discriminating factor is not compressive strength but the presence of tensile capacity in the material. Including tensile resistance—commonly neglected in conventional masonry models—can increase the predicted ultimate shear by up to a factor of three, yielding a more realistic description of 3DPC behaviour.

Although anisotropy leads to directional variation of strengths, the observed failure modes remain coherent with, and well captured by, the proposed models. Overall collapse topology is driven primarily by geometry and stress distribution rather than by print direction or the absolute value of axial force. That said, print orientation can influence the response when tension acts parallel to the layers, where higher tensile capacity is often observed and may shift the governing mechanism. Within this framework, a generalised constitutive model is adequate provided that mechanical parameters are properly calibrated to reflect material anisotropy. The proposed models describe the full evolution up to failure.

Finally, looking ahead, targeted experimental characterisation of interlayer interfaces is essential to quantify cohesion and friction parameters with greater accuracy, thereby refining the shear-sliding checks.

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5 Simplified Calibration of the Numerical Model in SAP2000

To develop a rapid, design-oriented tool consistent with the specimens tested and described above, the as-built geometry and mechanical properties of the tested elements were implemented in SAP2000. In particular, the wall with conventional base connections—designated C-BA3DW in Sections 3 and 4—was imported and modeled. The numerical model was then benchmarked against the experimental response in the elastic range. For a like-for-like comparison, Figure 5.1 reports the geometry of the tested wall (height 2.10 m) alongside the measured numerical values from the experimental campaign. This framework is used to assess whether a Timoshenko (shear-flexible) beam element provides an adequate simplified representation of the specimen at the elastic limit, suitable for preliminary design purposes. The tested C-BA3DW wall exhibited a mean elastic-limit shear capacity of 79 kN, with a corresponding top displacement of 0.65 mm at the elastic limit. These experimental data were therefore used to compare with the elastic response obtained from the calibrated SAP2000 model, in order to validate the simplified numerical representation.

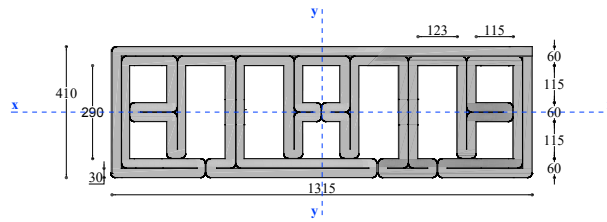


Figure 5.1. Cross Section of Panel Numerically studied.

5.1 Element-Scale simplified model (Timoshenko Beam)

Timoshenko first-order shear-deformation beam is adopted to represent the tested 3D-printed concrete (3DPC) wall panels. Because these members have non-negligible transverse dimensions—as in masonry walls or RC shear walls—shear deformation contributes materially to the initial stiffness and must be included in a design-oriented analysis [1,2 and 3]. The numerical analysis is limited to the elastic range,, with attention to initial stiffness and the elastic-limit response measured on the reference specimen. Assumptions are non-dissipative behavior for combinations (q equal to 1) and an infinitely rigid diaphragm where justified, so that global drift reflects wall shear and flexure rather than diaphragm flexibility. Input definition begins with geometry (clear height, length, thickness, openings, and print-path void ratio) and materials (Young's modulus and Poisson's ratio—and thus the shear modulus by isotropic elasticity—and density for masses), noting any directional effects due to printing. Bending properties, expressed by the cross-sectional area (A) and second

moment of area (I), are derived from the real wall geometry. They are taken as real section values, in particular for flexure and shear the area is treated as effective, reduced to reflect voids and interlayer weakness. The shear-correction factor χ starts from the classical rectangular value $5/6$; if higher fidelity is warranted by the adopted Poisson's ratio, Cowper's ν -dependent refinement is preferred [2]. In SAP2000, frame elements are used with shear deformations enabled, assigning E and ν (thereby G) together with the effective bending and shear properties A and I . Boundary conditions reproduce the conventional base connections of the test, described in chapter 3; diaphragm behaviour is enforced at the roof where appropriate, with tie-beam and panel connectivity modelled consistently with the superstructure. Calibration is carried out against specimen C-BA3DW using the elastic stiffness as the primary target and the mode-shape consistency as a supporting check. The model response is compared to the experimental elastic-limit point—mean shear ≈ 79 kN with top displacement ≈ 0.65 mm—and the effective shear areas (and, where justified, κ within a code-consistent range) are tuned until the elastic slope matches the test within a practical tolerance, for example $\pm 10\%$. Young's modulus is retained at the measured value; κ refinement is used only if supported by Cowper's guidance for the specific ν [2]. The final A_2/A_3 and κ values are recorded and serve as design defaults for panels with comparable print parameters.

The adequacy of the proposed model was verified by direct comparison with the experimental results obtained on the reference specimen. Since the model relies on a single Timoshenko beam

element, no mesh dependency issues arise. The real sectional properties of the wall were imported into SAP2000, and the same elastic-limit load measured during the test was applied at the top of the element. The resulting displacements and stiffness values closely matched the experimental ones, confirming that the internal calculation path implemented in SAP2000 correctly reproduces the global stiffness and external load–displacement response.

A more detailed assessment of the internal stress distribution would require advanced numerical modelling (e.g., solid elements with explicit interface representation). However, for the purposes of this Ph.D. work, the elastic calibration was deemed sufficient to validate the simplified model within the elastic range, ensuring its reliability for preliminary design and parametric analyses—fully consistent with established beam theory and finite-element practice [1,2 and 3].

5.1.1 Implementation in Analysis Software (SAP2000)

The element selected for calibration is the previously introduced specimen labelled C-BA3DW. In order to reproduce its response, the exact same geometry was imported into SAP2000 and modelled as a Timoshenko beam element [1], thus accounting for shear deformations in addition to flexural behaviour. This modelling choice ensures that the numerical representation is consistent with the physical test setup and is able to capture the influence of shear flexibility, which is particularly relevant for relatively deep wall elements such as the one under consideration. The beam element was assigned the same overall height of 2.10 m as the tested

specimen, thereby preserving geometric fidelity between the experimental and numerical models. Figure 5.2 provides a visual representation of the implemented model. In particular, Figure 5.2 (a) shows the idealized beam element within SAP2000, while Figure 5.2 (b) presents an extruded view of the element with the local reference axes explicitly indicated. This graphical representation clarifies the orientation of the local axes and the way in which the numerical formulation interprets the geometry of the wall. Such a description is important because the local axes govern how loads and boundary conditions are applied within the software environment. By displaying both the beam abstraction and its extruded view, Figure 5.2 highlights the correspondence between the simplified numerical model and the actual physical specimen, ensuring that the calibration process is grounded on a faithful geometric and mechanical representation.

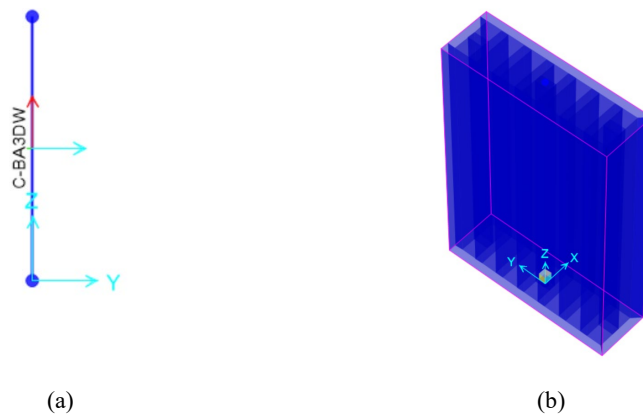


Figure 5.2. Beam Element in SAP2000 (a); Extrude View of the numerical Element (b).

5.1.1.1 Define Materials in Numerical Model (SAP2000)

For the sake of computational simplicity, the material was modeled as isotropic, with its mechanical properties defined along the direction of interest (y – y , see Figure 5.1). This choice allows the model to reproduce the elastic response of the element without introducing additional complexity associated with anisotropy or nonlinear constitutive laws. In particular, the parameters implemented in SAP2000 included the Young's modulus E , the Poisson's ratio ν , the shear modulus G , and the specific weight γ . By adopting these properties, the model intentionally neglects nonlinear behavior, focusing instead on a simplified representation aimed at reproducing the overall stiffness of the system. In this way, the elastic branch of the wall response can be effectively captured by considering only the elastic moduli in conjunction with the geometry of the section. Such an approach, although simplified, is consistent with the purpose of the analysis, namely to provide a reliable evaluation of the element's stiffness in the elastic range and to compare it with the experimental evidence. The definition of material properties in this form allows for a clear and transparent link between the input parameters and the corresponding elastic response of the structural model. For completeness, the mechanical properties adopted in SAP2000 are summarized in Table 5.1, which reproduces the values already introduced in the previous sections.

ν	E	G	γ
[-]	[MPa]	[MPa]	[kN/m ³]
0.20	15000	6250	22

Table 5.1. Main Mechanical Properties used in SAP2000.

5.1.1.2 Loads and Constrains

The boundary conditions were idealized by assigning a fixed support at the base, reproducing a cantilever scheme. At the top of the element, a lateral load corresponding to the elastic stage recorded during the experimental campaign was applied. This value, obtained as the average between the push and pull phases, amounted to 79 kN for the wall specimen labeled C-BA3DW. An additional axial load of 58 kN was imposed at the wall top, representing the vertical load equivalent to the weight of a roof slab, consistent with the test setup described in Section 3. A schematic representation of the applied loading configuration is provided in Figure 5.3. Specifically, Figure 5.3 (a) illustrates the vertical load acting along the global z -axis (longitudinal to the wall), while Figure 5.3 (b) shows the lateral force applied along the y -axis.



Figure 5.3. Vertical load (a); Lateral Load (b).

5.2 Comparison with experimental results: Elastic Response

The outputs obtained from SAP2000, with an applied load of 79 kN at the top of the beam element, reported a corresponding top displacement of 0.635 mm. This value is remarkably close to the displacement recorded during the experimental tests (0.65 mm), confirming the consistency between numerical prediction and experimental observation. For completeness, the graphical outputs are shown in Figure 5.4.

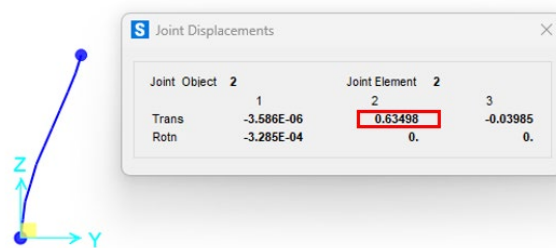


Figure 5.4. Output sap2000

This close agreement indicates that the Timoshenko beam model [1] provides an accurate representation of the element's stiffness within the elastic range. The model is therefore validated as a reliable tool for capturing the elastic response of the system, ensuring that the fundamental stiffness properties are correctly reproduced and aligned with experimental evidence.

5.3 Comparison with experimental and analytical results: Elastic Response

To provide a comprehensive understanding of the element's behavior, the results obtained through analytical, numerical, and experimental approaches are superimposed and summarized in Figure 5.5. This figure allows for a direct comparison of the different methods, emphasizing the degree of consistency among them and highlighting the specific role of each modeling strategy.

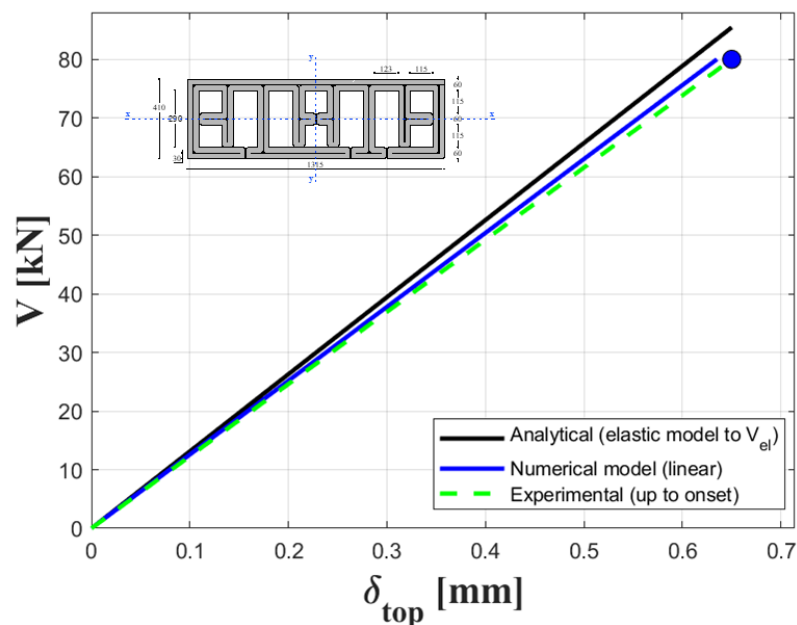


Figure 5.5. Analytical vs Numerical vs Experimental (focus on elastic branch)

As shown in Figure 5.5, the elastic branch is consistently reproduced by both the analytical model and the experimental results, with the numerical simulation falling in line with these predictions. This

agreement demonstrates that the analytical formulation, despite its simplicity, is capable of capturing the initial stiffness of the element with remarkable accuracy. The elastic range represents the fundamental domain for stiffness and strength verification in structural design, as design checks are conventionally referenced to elastic stress distributions. Focusing on this regime (elastic) allows the evaluation of key stiffness parameters—such as global shear and flexural stiffness—and the assessment of the structural resistance up to crack initiation. These quantities are essential for serviceability verification, preliminary sizing, and for ensuring that deformations remain within acceptable operational limits. The analytical formulation, described in chapter 4, provides an important link to relate material properties and section geometry to the overall structural response. The good agreement observed between analytical predictions and the initial stage of the experimental curve, showed in chapter 3, confirms the validity of the adopted assumptions—namely linear-elastic material behaviour and the equivalent-section stiffness representation.

The numerical model, though more detailed, consistently reproduces both the analytical and experimental responses, further reinforcing the reliability of the proposed interpretation. As summarized in Figure 5.5, the integrated use of analytical, numerical, and experimental approaches yields a comprehensive framework for understanding the structural behaviour: the experimental curve provides the reference response, the analytical model predicts the stiffness and elastic capacity, and the numerical simulation extends the analysis to more complex loading conditions.

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6 Case Study: Control Room

Montalto di Castro, Italy

6.1 General Description of the case study

The case study concerns the application of 3D Concrete Printing (3DCP) technology for the construction of the control room of the electrical substation in Montalto di Castro (Viterbo), owned by ENEL Green Power. The project builds upon recent advances in the structural use of 3DCP, integrating insights from the results of an experimental campaign carried out at the Structural Laboratory of the Department of Engineering, University of Naples Federico II and illustrates in the chapter 3 of this work. The building is designed as a single-storey structure, composed of a single room with two openings, without a basement and completed with a flat roof. The adoption of a numerical model consistent with the experimental evidence provided a realistic representation of the wall behaviour, thereby supporting the technical feasibility of the proposed solution. Figure 6.1 and Figure 6.2, present the architectural layout of the project. Specifically, Figure 6.1 shows the render view of the control room, while Figure 6.2 provide a plan view that highlight the geometry of the 3d printed concrete wall.



Figure 6.1. Architectonical view of the 3dpc structure studied

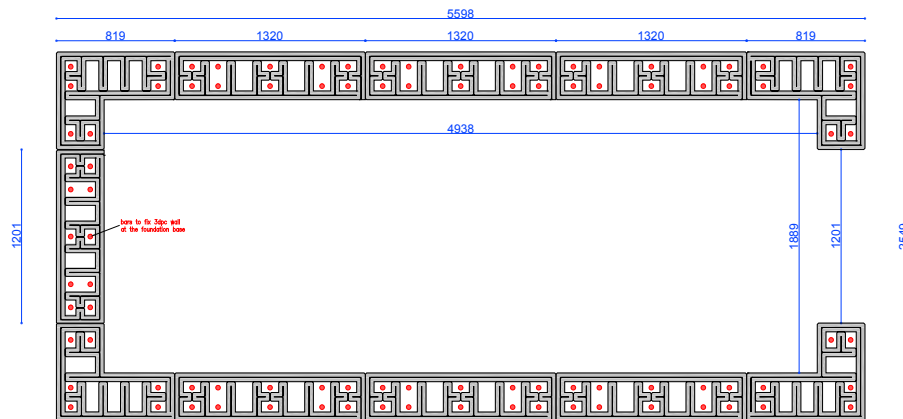


Figure 6.2. Plan view of the 3dpc structure studied.

The building footprint measures 5.60 m × 2.55 m, with a total height of 3.55 m. The vertical structure is composed of ten wall panels

manufactured using 3D Concrete Printing (3DCP) technology, which together form the above-ground enclosure of the control room.

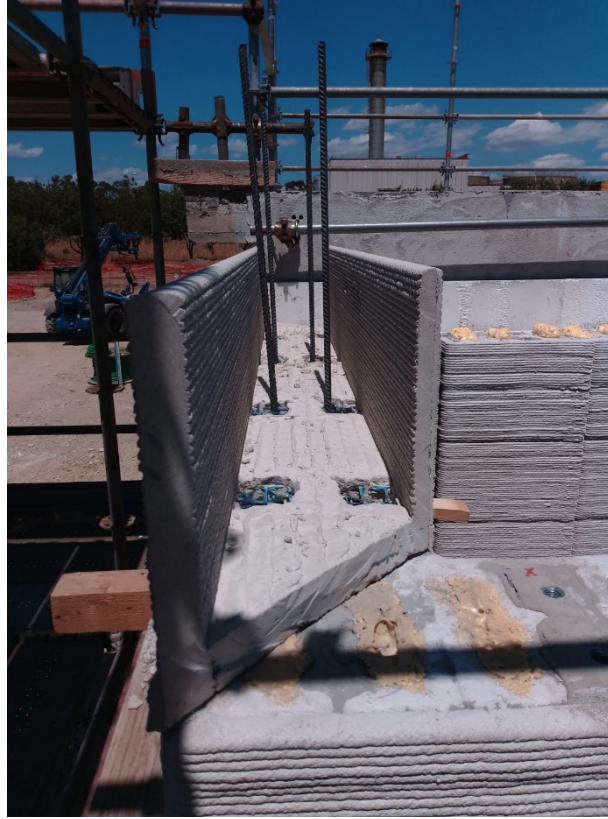
For greater clarity, Figure 6.3 shows the main phases of wall production and installation. In particular, Figure 6.3 (a) illustrates the printing process carried out in the production facility; Figure 6.3 (b) shows the detail of the base connection and wall installation; Figure 6.3 (c) depicts the placement of the top concrete ring beam connecting the walls; and Figure 6.3 (d) presents the assembled building after completion of the installation process.



(a)



(b)



(c)



(d)

Figure 6.3. Main phases of the 3D printed wall production and installation:
(a) printing phase; (b) base connection and installation; (c) top ring-beam placement; (d)
assembled building.

In addition to these panels, the design also includes ten permanent U-shaped formworks, to be printed and subsequently filled with cast-in-place concrete during the construction of the slab. This solution ensures that the resulting elements act as continuous edge beams, providing an effective stiffening system that can be regarded, from a structural standpoint, as infinitely rigid. Figure 6.4 shows an exploded axonometric view of the structural system. From bottom to top, the figure illustrates the 3D printed wall panels, the perimeter stiffening beam cast in place within U-shaped permanent formworks, and the reinforced concrete roof slab. This representation clarifies the assembly of the main structural components and highlights the integration between the 3D printed and cast-in-place elements.

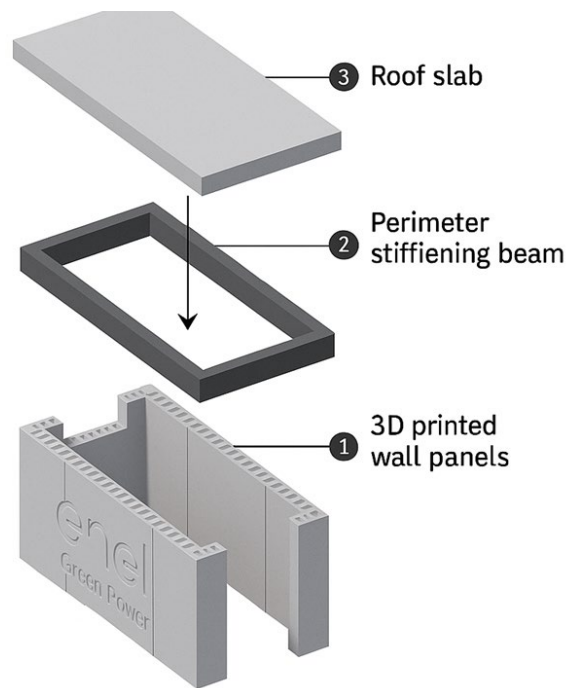


Figure 6.4. Exploded axonometric view of the 3dpc structure studied.

6.2 Design Path: From Material Properties to the Regulatory Framework

The structural system under investigation is composed of three main components: the vertical load-bearing walls manufactured through 3D Concrete Printing (3DCP), which form the resistant core of the building; a perimeter edge beam realised by means of permanent formworks, also 3D printed and subsequently filled with cast-in-place concrete; and finally a prefabricated predalles floor slab, completed with an additional topping layer of concrete of class C20/25. This configuration ensures the collaboration between the printed and the

traditional components, with the aim of achieving a box-like structural behaviour and an adequate overall stiffness of the system.

From a design perspective, the mechanical analysis carried out in the previous sections is sufficient to frame the problem within a performance-based model, as it allows the structural response to be evaluated without introducing additional constitutive complexity. In particular, since the design in this case study is conducted with reference to the Serviceability Limit State (SLS), it is appropriate to consider only the elastic response of the structure, without the need for nonlinear constitutive models. This approach is further justified by the current regulatory framework: given that 3D Concrete Printing is an innovative technology not yet fully codified, existing standards allow the panels to be treated as weakly reinforced elements, and therefore as components prone to brittle failure. In fact, In this case study, the design is conducted within the elastic domain, consistently with the assumptions made for weakly reinforced or unreinforced concrete elements, whose behaviour is inherently brittle. According to NTC 2018 (§4.1.2.1.3), such elements must be designed and verified assuming a non-dissipative response, with behaviour factor $q = 1$ and ultimate checks performed in terms of sectional resistance.

Within this context, elastic verifications assume a central role, constituting the main reference for validating the safety and reliability of 3D printed elements [1][2][3]. This interpretation is also aligned with the recommendations of the *fib Model Code 2010*, where for innovative materials and non-traditional structural systems a conservative approach is advised, based on simplified models and

elastic verifications. As additional experimental and numerical data become available, future codes will be able to incorporate more advanced constitutive laws to represent post-cracking behaviour, residual ductility, and progressive collapse mechanisms. For the time being, however, the adoption of an elastic, performance-based framework provides a solid methodological foundation, consistent with both Italian and international standards, and adequate for the design of innovative 3D printed concrete structures.

The focus of this chapter is therefore on the printing material that constitutes the vertical panels. The following sections will illustrate its fundamental mechanical properties, the current regulatory framework concerning its identification, classification, and approval, and finally the modelling of its constitutive behaviour for design purposes. This integrated approach makes it possible to frame both the experimental and the design aspects, providing a robust basis for the validation and practical application of 3D Concrete Printing in the structural field.

6.2.1 Regulatory References and Material Framework

The Italian regulatory system for structural design is governed by the *Norme Tecniche per le Costruzioni* (NTC 2018) [1], supported by the Explanatory Circular of 2019 [4]. These documents, aligned in philosophy with the Eurocodes, define how materials and structures must be verified in order to ensure safety under both serviceability and ultimate limit states. A specific chapter on 3D printed concrete (3DPC) does not yet exist. However, the NTC provide clear guidance for materials and structures that fall outside traditional categories. Section

4.1.2.1.3 of the NTC states that *“in the case of unreinforced concrete or structures with only modest reinforcement, the behaviour shall be considered brittle and the verification shall be carried out in terms of ultimate resistance of the section.”* [1]. This prescription is fundamental because it acknowledges that certain materials cannot rely on ductility, redistribution, or post-cracking capacity.

The Explanatory Circular [4] reinforces this interpretation (§C4.1.2.1.3), stressing that elements with little or no reinforcement must be checked using cautious resistance models capable of capturing brittle failure [4]. Similarly, §12.1 of the NTC explicitly opens the door to innovative materials, provided that their adoption is supported by adequate experimental documentation demonstrating their compatibility with safety criteria. This means that 3DPC can be formally employed in structural projects as long as its strengths and stiffness parameters are properly characterised.

The analogy with unreinforced masonry and plain concrete is particularly strong. The *CNR-DT 200/2013* recommendations [5], regulating masonry structures, specify that for non-ductile materials design strengths must be reduced compared to experimental values, precisely to account for the lack of ductility and the impossibility of stress redistribution. A comparable caution is present in the *MiBACT Guidelines (2021)* [6], which state that for brittle materials such as plain concrete or masonry, design strengths should be taken lower than those derived from conventional reduction factors applicable to ductile materials. As a result, the framework available [1-9], and associated guidelines provides a regulatory “space” where 3DPC can be located:

a material considered brittle, to be designed using elastic, non-dissipative models, with reduced strengths and verification against ultimate capacity without assuming ductility reserves.

6.2.2 Placement of 3DPC Structural Elements in the Italian Regulatory Framework

When the general principles outlined above are transferred to 3D printed concrete structural walls, the consequences are direct and significant. Since 3DPC walls are constructed layer by layer and lack conventional reinforcement, they must be assimilated to weakly reinforced or unreinforced systems. Their verification is therefore conducted elastically, with no reliance on plastic redistribution or ductility, as required for non-dissipative structures by §7.2.2 of NTC 2018 [1]. From the material standpoint, this implies that the characteristic strengths obtained from laboratory testing (compressive strength f_{ck} , tensile strength f_{ctk} , shear strength through cohesion f_{vk0} and friction angle φ) must be converted into design strengths through a more conservative process than for conventional concrete. While Eurocode 2 [7] and NTC 2018 [1] adopt α_{cc} equal to 0.85 (is the coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied) for ordinary concrete, the value of this factor is inadequate for 3DPC due to its anisotropy, layer interfaces, and printing-related defects. Following the approach of §12.1 (innovative materials) and the caution of *CNR-DT 200/2013* [5], a lower value has been introduced, here denoted as α_{3DPC} in Eq. 6.1.

$$f_{cd,3dpc} = \alpha_{3dpc} \frac{f_{ck}}{\gamma_c} \quad \text{with } \alpha_{3dpc} \leq 0.85 \quad \text{Eq. 6.1}$$

The same reduction is extended to tensile and shear strengths and they are expressed in Eq. 6.2 and Eq. 6.3.

$$f_{ctd,3dpc} = \alpha_{3dpc} \frac{f_{ctk}}{\gamma_c} \quad \text{with } \alpha_{3dpc} \leq 0.85 \quad \text{Eq. 6.2}$$

$$\tau_{Rd,3dpc} = \alpha_{3dpc} \left(\frac{f_{vk0}}{\gamma_c} + \sigma_n \tan \varphi \right) \quad \text{with } \alpha_{3dpc} \leq 0.85 \quad \text{Eq. 6.3}$$

where σ_n is the applied normal stress and γ_c is the partial factor for concrete-type structures checks [1, 7]. For this reason, the structural design of 3D printed concrete (3DPC) elements requires the definition of a consistent set of mechanical properties, analogous to those employed for conventional concrete and masonry. Among these, the compressive strength f_{ck} , the tensile strength f_{ctk} , and the shear cohesion parameter f_{vk0} are the most critical values to be determined, as they directly govern the resistance domains of walls and panels.

The compressive strength f_{ck} is traditionally obtained from uniaxial compression tests performed on cubic or cylindrical specimens, following the procedures established for cast concrete (EN 12390 series [8], Eurocode 2[7]). However, in the case of 3DPC, the presence of interfaces and anisotropy requires dedicated protocols. The recent interlaboratory campaign conducted by RILEM TC 304-ADC [9] has highlighted the necessity of extracting specimens directly from printed elements rather than cast counterparts, in order to capture the influence of layer orientation, interlayer time, and extrusion defects. The study recommends the use of prismatic or cubic samples sawn or cored from printed walls, tested in different orientations (u, v,

w) with respect to the printing direction, since compressive strength reductions of 10–30% compared to cast specimens have been consistently observed [9]. In particular, a distinction should be made between mortar-scale tests, typically performed on 40×40×40 mm cubic specimens [9], and concrete-scale tests, generally carried out on cylindrical cores extracted from printed elements [9]. This differentiation is crucial, as the obtained strengths differ significantly depending on the scale and specimen preparation method, directly affecting the structural assessment of 3DPC walls and panels.

The tensile strength f_{ctk} of printed materials is even more sensitive to anisotropy. While conventional concrete employs splitting tensile tests (EN 12390-6) or direct uniaxial tension, in 3DPC both test types must be adapted to account for the layered microstructure. RILEM TC 304-ADC recommends performing splitting tests on specimens extracted at different heights and orientations, complemented by uniaxial tensile tests where feasible, in order to obtain reliable values of tensile resistance and fracture behaviour. These tests are fundamental to assess the weak direction corresponding to interfaces loaded in tension, which often represent the governing failure mode. Experimental results confirm that tensile strength exhibits a larger scatter than compressive strength and may reach values as low as 50% of the corresponding cast reference, depending on interlayer bonding and printing parameters [9].

Finally, the shear cohesion parameter f_{vk0} plays a pivotal role when printed walls are designed as weakly reinforced or unreinforced elements. In the Italian context, following the approach of CNR-DT

200/2013 for masonry [10], the shear resistance is expressed through a Mohr–Coulomb criterion as $\tau_{Rk} = f_{vk0} + \sigma_n \tan(\varphi)$, where f_{vk0} is the cohesion, c , at zero normal stress and φ is the friction angle. For 3DPC structures, the reference shear strength f_{vk0} is not directly obtained from small-scale interlayer shear tests, but is instead derived from the global response of the printed panel tested under diagonal compression. As discussed in the previous chapters, material-scale tests provide the interface parameters (cohesion and friction), whereas the panel-scale test captures the combined shear–flexural behaviour of the printed element, where stress redistribution and layer interaction govern the apparent shear capacity. Accordingly, the value of f_{vk0} adopted here represents an *effective shear strength* of the printed wall, consistent with the structural-scale behaviour rather than with the local interface response. In addition, friction angle, φ is derived from combined compression–shear tests under different levels of axial load [11,9], at material scale. Recent studies and the RILEM TC 304-ADC [9] interlaboratory programme confirm that these parameters are strongly influenced by interlayer time, orientation of filaments, and printing process control, underlining the necessity of project-specific calibration. Altogether, the experimental characterisation of f_{ck} , f_{ctk} , and f_{vk0} in 3DPC does not simply replicate conventional procedures, but rather adapts them to capture anisotropy and process variability. The RILEM guidelines [9] represent the current international reference for establishing these values, ensuring that they can be consistently adopted in design frameworks such as Eurocode 2 [7] and NTC 2018 [1], albeit with the introduction of additional reduction factors (e.g. α_{cc} or α_{3DPC}) to account for the peculiarities of the additive process.

The strength parameters measured experimentally on laboratory specimens (e.g. average compressive strength f_{cm} , mean tensile strength f_{ctm} from pull-off [9]) cannot be directly used in verifications. The Italian building code (NTC 2018) and Eurocode 2 [EN 1992-1-1] prescribe that the characteristic strength of a material is defined as a fractile of the statistical distribution of test results, typically the 5% fractile, i.e. the value below which only 5% of results are expected to fall. This ensures that the adopted value is conservative with respect to variability and uncertainty. For concrete, Eurocode 2 [7] provides simplified conversion rules. For instance, if only the mean compressive strength f_{cm} is available, the characteristic strength can be estimated as expressed in Eq. 6.4.

$$f_{ck} = f_{cm} - k \cdot s = f_{cm} - 8 \quad \text{Eq. 6.4}$$

where s is the standard deviation of the test series and k is the fractile factor (≈ 1.48 for a 5% fractile with 15 specimens, according to EN 1990 Annex D [12]). In practice, when the number of specimens is limited and full statistical parameters are not available, codes [1,7] often provide empirical relations (e.g. $f_{cm} \approx f_{ck} + 8$ MPa for normal concretes) that allow the characteristic value to be back-calculated. For 3D printed concrete, no dedicated conversion formula exists yet. Therefore, the recommended approach is to follow the general statistical definition of characteristic value as the 5% fractile of the distribution of test results, in accordance with EN 1990 [12] and RILEM TC 304-ADC guidelines [9]. In pull-off tensile tests, for example, the measured values of direct tensile strength are first averaged (f_{ctm}) and

their standard deviation computed. The characteristic tensile strength is then obtained as expressed in Eq. 6.4 Eq. 6.5.

$$f_{ctk} = 0.7 \cdot f_{ctm} \quad \text{Eq. 6.5}$$

The adoption of the reduction factor 0.7 in deriving the characteristic tensile strength from the mean value is not arbitrary, but is firmly rooted in the statistical basis of the Eurocodes [13]. Eurocode 2 [7] (EN 1992-1-1, §3.1.2) explicitly formalises this principle, stating that the lower characteristic tensile strength can be taken as explained in Eq. 6.5. This simplification is the outcome of extensive statistical analyses performed on large experimental databases of conventional concretes, which demonstrated that the 5% fractile typically corresponds to approximately 70% of the mean tensile strength.

In the present work, the same ratio is extended to 3D printed concrete (3DPC). Although 3DPC exhibits higher variability and anisotropy than cast concrete, the use of these coefficient in Eq. 6.4 and Eq. 6.5 ensures that the adopted characteristic tensile strength remains consistent with the conservative philosophy of Eurocode 2 [7], while aligning with the recommendations of RILEM TC 304-ADC [9], which emphasise the need to translate mean values from pull-off tests into lower-bound design parameters [9]. This approach guarantees that the safety margins adopted for conventional concretes are preserved, if not enhanced, when applied to an innovative material where uncertainties are more pronounced. A key conceptual point in the definition of cohesion for 3D printed concrete (3DPC) is the distinction between material-level cohesion and structural-level cohesion. At the material scale, cohesion reflects the intrinsic bonding capacity of the

mortar and of the interfacial transition zones between filaments, as obtained through laboratory tests such as direct shear on small printed coupons or pull-off adhesion tests. These tests tend to return higher values, since they isolate the material behaviour under controlled boundary conditions and exclude size effects and construction-related defects. At the structural scale, however, design codes and testing standards typically require the adoption of lower, assembly-based values, derived from tests on larger specimens or representative wall panels. For masonry, for example, ASTM C1531[14] defines shear bond strength through triplet tests, where the cohesion f_{vk0} is directly obtained as the shear strength at zero normal stress. These values, systematically lower than those obtained at the material scale, are explicitly intended as design-level properties, accounting for workmanship, variability of bed joints, and scale effects [14; 10].

In the case of 3DPC, the same conceptual framework applies. The material cohesion measured through laboratory direct shear tests on filaments or small coupons provides an upper-bound value that characterises the intrinsic interlayer bond [9]. For structural design, however, the relevant parameter is the characteristic cohesion of the wall system, which must be obtained either through shear- tests on printed panels (as in laboratory study 11) or through conservative statistical treatment of small-scale tests, and then reduced to a characteristic value in line with Eurocode/NTC safety philosophy. This transition—from material-level strength to element-level design parameter—is therefore essential. It explains why the values used in design are systematically lower: they embed not only the intrinsic

material properties but also anisotropy, interlayer time effects, voids, and construction tolerances. In analogy with masonry, where f_{vk0} is determined at the assemblage scale and then used in Mohr–Coulomb formulations for shear resistance of walls, 3DPC requires a similar passage: from cohesion measured at the material scale to a reduced characteristic cohesion adopted at the structural element scale.

In fact, for the sake of clarity, these expressions (Eq. 6.1, Eq. 6.2 and Eq. 6.3) are not merely mathematical adjustments: they translate the brittle, process-dependent nature of 3DPC into codified inequalities that can be used in design. They guarantee that the uncertainties introduced by anisotropy, interlayer time, or voids are reflected in the safety margin, in full coherence with the provisions of the Italian framework [1] and with international recommendations such as the *fib Model Code 2010* [3]. The effectiveness of this methodology has been concretely demonstrated in the Montalto di Castro control room project, the first full-scale 3DPC building realised in a seismic zone in Italy [15]. In that project, walls were treated as non-dissipative elements and verified elastically, seismic actions were derived from the elastic spectrum without reduction by ductility factors, and material strengths were reduced through α_{3DPC} . The combination of printed vertical walls, a cast-in-place perimeter edge beam, and a prefabricated *predalles* slab acting as a stiff diaphragm ensured that the global system behaved as a box-like structure within the elastic domain. In this case study, all checks were carried out strictly following the Italian building code format. Since the control room is classified as a Class of Use IV structure, in accordance with NTC 2018, the relevant

limit states are identified by Table 7.3.III of the code [1]. For this class of importance, the Serviceability Limit State of Operability (SLO) must be explicitly verified in terms of rigidity (RIG). This requirement ensures that the building remains functional and operational even after seismic events with high probability of occurrence, without suffering excessive deformations. Consequently, the printed walls were checked not only with respect to ultimate strength but also to guarantee adequate stiffness under serviceability conditions, as prescribed by NTC 2018 [1]. This experience demonstrates that, even in the absence of a dedicated chapter for 3DPC, the current Italian framework already provides the necessary tools to design such elements safely. By assimilating them to fragile, non-dissipative systems and applying more severe strength reductions, compliance with the safety principles of the NTC is ensured, while still enabling the integration of innovative technologies into real buildings [15].

6.3 Materials Properties used in the Case Study

The structural walls printed at Montalto di Castro were manufactured using the same mortar, mix design and printing configuration adopted in the laboratory campaign. The panels tested in Chapter 3 were printed under identical process conditions—same nozzle width, same layer height, comparable interlayer time (ILT range), the same layer orientation (Orientation I), and an equivalent layer-by-layer deposition strategy. For this reason, the mechanical properties adopted in the case study are equivalent with those experimentally determined on the

laboratory-printed specimens and presented in the previous sections. More precisely, the material parameters used for the structural checks of the Montalto di Castro Control Room derive from: (i) the laboratory-tested values reported in Chapter 2 and Chapter 3, which are fully representative of the printed material used at full scale; (ii) the manufacturer's certified strengths for the i.tech® 3D 2MM mortar, ensuring consistency with the declared performance of the mix; and (iii) the design-oriented safety framework, including the process-related coefficient α_{3DPC} , introduced to account for the inherent variability of the printing process during the transition from characteristic to design strengths.

6.3.1 Material Constitutive relations

The constitutive response of the 3D printed mortar, whose mechanical properties correspond to the design values adopted for the structural analysis, is described through three complementary relations: the stress–strain law in compression, the stress–strain law in tension, and the shear envelope at the material interfaces. These relations are formulated to be consistent with design codes and with current recommendations for 3D printed concrete, and they are calibrated on laboratory data and on the manufacturer's certified tests. In compression, the material behaviour is represented by a parabola up to the peak, followed by an exponential softening branch inspired by the *fib Model Code 2010* [3]. Denoting strains in compression as positive, the curve reads (Eq. 6.6 and Eq. 6.7)

Eq. 6.6

$$\sigma_c(\varepsilon_c) = \begin{cases} f_{cm} \left[2 \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right) - \left(\frac{\varepsilon_c}{\varepsilon_{c0}} \right)^2 \right], & 0 \leq \varepsilon_c \leq \varepsilon_{c0} \\ f_{cm} \exp \left(-\frac{\varepsilon_c - \varepsilon_{c0}}{0.0015} \right), & \varepsilon_c \geq \varepsilon_{c0} \end{cases} \quad \text{Eq. 6.7}$$

where f_{cm} is the mean compressive strength and ε_{c0} is the peak strain. In accordance with practice for ordinary concretes, the peak strain is estimated with Eq. 6.8,

$$\varepsilon_{c0} = 0.7 f_{cm}^{0.31} 10^{-3} \quad \text{Eq. 6.8}$$

with f_{cm} expressed in MPa. This choice provides a compact way to anchor the curve to the measured compressive strength while keeping the post-peak branch in a realistic softening regime for quasi-brittle materials.

In tension, the behaviour is linear elastic up to the cracking point, followed by an exponential softening that captures the rapid strength degradation once micro-cracking localises. Adopting negative strains for tension, the relation used in the model is expressed in Eq. 6.9 and Eq. 6.10.

$$\sigma_t(\varepsilon_t) = \begin{cases} -E_c |\varepsilon_t| & |\varepsilon_t| \leq \varepsilon_{t0} \\ f_{ctm} \exp \left(-\frac{|\varepsilon_t| - \varepsilon_{t0}}{5 \cdot 10^{-5}} \right) & |\varepsilon_t| \geq \varepsilon_{t0} \end{cases} \quad \begin{matrix} \text{Eq. 6.9} \\ \text{Eq. 6.10} \end{matrix}$$

where E_c is the Young's modulus, f_{ctm} is the mean tensile strength, and the tensile cracking strain is taken from Eq. 6.11.

$$\varepsilon_{t0} = \frac{f_{ctm}}{E_c} \quad \text{Eq. 6.11}$$

This construction is fully consistent with linear-elastic design in the pre-cracking regime and with a cohesive-softening idealisation for the

post-cracking stage, which is particularly relevant for 3D printed materials due to the presence of weak interlayer planes.

In addition, Shear resistance along printed interfaces is governed by a Mohr–Coulomb-type envelope, which combines an intrinsic cohesion with a frictional contribution mobilised by normal compression. Denoting by σ_n the normal stress on the shear plane (compression taken as positive) and by τ the shear stress, the characteristic shear strength is expressed in Eq. 6.12.

$$\tau_{Rk}(\sigma_n) = f_{vk0} + \sigma_n \tan \varphi \quad \text{Eq. 6.12}$$

where f_{vk0} is the cohesion (shear strength at zero normal stress) and φ is the friction angle. In design, negative portions of the envelope (if any) are disregarded because no cohesion is mobilised under net tension on the plane, which is why the numerical plotting hides τ minus than 0. This formulation mirrors the Italian guideline for brittle, weakly reinforced or unreinforced materials and is directly transferable to 3D printed concrete interfaces, where sliding failures have been repeatedly observed. The numerical implementation provided, in Figure 6.5, adopts positive strains in compression (red line) and negative strains in tension (blue line) ; compressive stresses are positive, tensile stresses negative. In addition, in Figure 6.6, is plotted the Mohr–Coulomb criteria, σ_n is varied across tension and compression; the model reports only the physically meaningful branch with $\tau \geq 0$, consistent with zero cohesion under net tension on the plane.

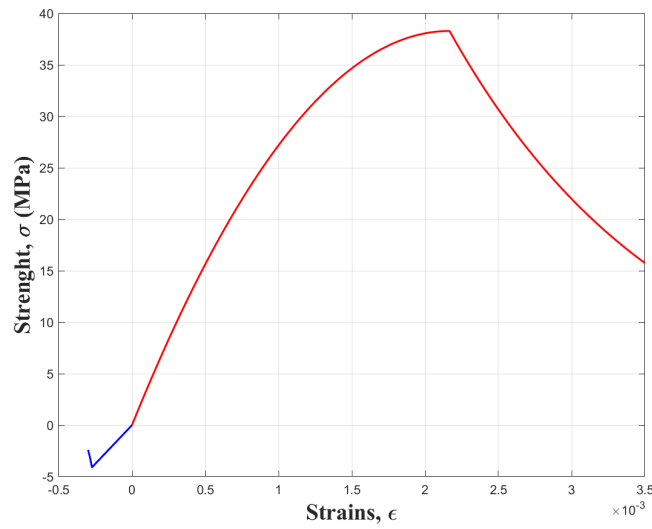


Figure 6.5. Compressive and Tensile Constitutive Law of 3dpc material.

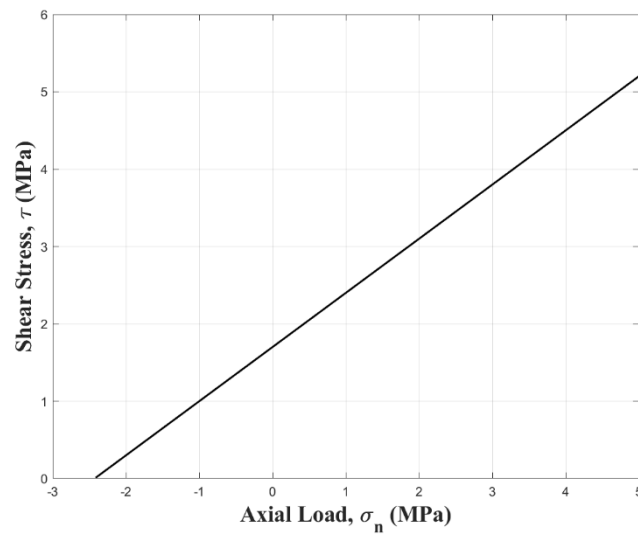


Figure 6.6. Shear Constitutive Law of 3dpc material.

6.3.2 Parameter identification

The strengths used to anchor these laws, expressed in previous section, derive from two point. The first one, the manufacturer's

datasheet for i.tech® 3D 2MM reports strengths determined in accordance with UNI EN 196-1[16] on cast specimens after 28 days: compressive strength not lower than about 48 MPa, flexural strength not lower than about 7 MPa, and a Young's modulus of at least 15 GPa. Second, laboratory tests performed on printed specimens cut from single-filament paths, following the orientations and procedures recommended by the RILEM TC 304-ADC interlaboratory programme [9], provided values representative of the actual printed. In the present work, the cast prisms yielded average strengths of 45 MPa in compression and 9 MPa in flexure after 28 days, while the printed prisms tested in the W-direction for compression and in the U–W plane for flexure yielded almost 40 MPa and 7 MPa, respectively [9]. The reduction observed in the printed configuration is consistent with anisotropy, interlayer time effects, and potential voids highlighted by RILEM [9].

6.3.3 Design Values

For the design of structural elements, the characteristic values f_{ck} , f_{ctk} , and f_{vk0} derived from tests and literature were converted into design values in accordance with NTC 2018 [1] and Eurocode 2 [7]. The adopted equations are summarised in Eq. 6.1., Eq. 6.2 and Eq. 6.3, where:

- $\gamma_c = 1.5$ is the partial safety factor for concrete;
- α_{3DPC} is a process reduction factor that extends the concept of α_{cc} (typically 0.85 for ordinary concrete) to account for printing-induced anisotropy, interlayer bonding variability, and voids.

In line with §12.1 of NTC 2018 [1] on innovative materials, $\alpha_{3\text{DPC}}$ was conservatively taken equal to 0.60–0.75.

For shear behaviour along printed interfaces, the design envelope follows the Mohr–Coulomb criterion (Eq. 6.3), in which f_{vk0} represents the shear strength without pre-compression—typically measured through direct shear tests [9, 11]—and the friction angle φ is obtained from compression–shear tests at different normal stress levels. Both procedures are consistent with CNR-DT 200/2013 [10] and the testing methodology described by RILEM TC 304-ADC [9]. However, in a conservative approach, as detailed in Chapter 2, the value derived from the empirical Tunsek–Čaković formulation [17] was adopted, by considering the characteristic tensile strength divided by 1.5 to determine f_{vk0} .

Table 6.1 summarises the mechanical parameters adopted for the design of 3DPC structural walls, distinguishing between mean values obtained from laboratory tests or empirical formula, the corresponding characteristic strengths, and the resulting design values obtained by applying both partial and process-related safety factors.

Meaning	Test / Standard	Mean value (f_m)	Characteristic value (f_k)	Design value (f_d)
Mean compressive strength	UNI EN 196-1 cast & printed (Or. I) [RILEM TC 304]	45 MPa (cast), 38.29 MPa (printed) → model input 40 MPa	38.29 MPa	$f_{d,3DPC}= \alpha_{3DPC} f_k / \gamma_c \rightarrow \approx 15\text{--}18 \text{ MPa}$ (with $\gamma_c=1.5$, $\alpha_{3DPC}=0.60\text{--}0.75$)
Mean direct tensile strength (pull-off)	EN 1542-1999	-	$\approx 1.5 \text{ MPa}$	$f_{d,3DPC}= \alpha_{3DPC} f_{tk} / \gamma_c \rightarrow \approx 0.60\text{--}0.75 \text{ MPa}$ (with $\gamma_c=1.5$, $\alpha_{3DPC}=0.60\text{--}0.75$)
Reduction of f_{tk} (conservative)	Tunsek-Cakovic formula	-	$\approx 1 \text{ MPa}$	$f_{d,3DPC}= \alpha_{3DPC} [(f_{k,0} / \gamma_c) + \sigma_{tamp}] \approx 0.40\text{--}0.60 \text{ MPa}$. (with $\gamma_c=1.5$, $\alpha_{3DPC}=0.60\text{--}0.75$)
Young's modulus	RILEM TC 304	15 GPa	—	-
Partial safety factor (concrete)	NTC 2018 / EC2	—	—	1.5
Process reduction factor (innovative materials)	NTC 2018 / EC2	—	—	0.60–0.75 (calibrated)

Table 6.1. Main Mechanical Properties used in work.

6.3.4 Justification of the adopted reduction factor α_{3DPC}

A central point in the transition from experimental mean values to design strengths for 3D printed concrete (3DPC) is the introduction of the process-related reduction factor α_{3DPC} . This coefficient was proposed in analogy with the partial factors applied in conventional codes for innovative or non-standardised materials [1,7], and is intended to account for the uncertainties specific to additive manufacturing: anisotropy due to layer orientation, defects related to the extrusion process, and variations in interlayer bonding linked to the printing time gap. In this study, the value of α_{3DPC} was calibrated to 0.60, following an iterative comparison between calculated design strengths and the actual resistances recorded during the experimental campaign. When adopting the upper bound suggested in the literature ($\alpha_{3DPC} \approx 0.85$ equal to α_{cc} for ordinary concrete [1]), the resulting design strengths were systematically higher than the failure loads observed in full-scale tests, leading to a non-conservative assessment.

The adoption of α_{3DPC} equal to 0.60 therefore emerges as a balanced choice, ensuring that the design values remain on the safe side while still being representative of the actual performance envelope of the tested walls. This calibration was performed consistently across compression, tension, and shear parameters. In particular, the observation that the onset point of failure in the elastic regime effectively represents the tensile resistance of the structural element provides a direct bridge between the material scale and the element scale. In this way, the onset response inherently captures the influence

of printing-related variability—such as interlayer bonding, filament orientation, and process defects—allowing the design curves derived from the constitutive laws to reproduce the experimental capacities within a conservative yet realistic margin.

For this reason, the use of α_{3DPC} equal to 0.60 reflects the approach of performance-based design: the safety factors are not arbitrarily assumed but tailored to the experimental evidence, thus creating a consistent bridge between the material characterisation, the structural element response, and the regulatory framework. Furthermore, this approach is in line with recent recommendations by RILEM TC 304-ADC[9], which emphasise that, for 3DPC, the transfer from mean values to design properties should include both statistical reduction and a process-related coefficient to explicitly reflect the peculiar uncertainties of additive manufacturing [9].

6.4 Structural Analysis

The structural analysis of the control room was performed through a Linear Dynamic Analysis (Response Spectrum Analysis), in compliance with the prescriptions of the Italian Building Code (NTC 2018, §7.3.3). This procedure, widely adopted in seismic design, enables the effects of earthquake loading to be directly computed by combining the elastic modal response of the structure with a prescribed design spectrum. In the present case, the analysis was conducted under the assumption of non-dissipative structural behaviour, in accordance with §7.2.2 of NTC 2018. For such

behaviour, “all members and connections remain in the elastic or substantially elastic range; the seismic demand is evaluated according to the reference limit state, independently of the structural typology and without accounting for material nonlinearities, through an elastic model. As a consequence, the elastic response spectrum was directly assigned to the model, without reduction by means of a behaviour factor. The seismic action was therefore considered with behaviour factor q equal to 1.0. This choice reflects the intrinsic fragility of the investigated elements, which are only weakly reinforced, and is consistent with the normative requirement that such members cannot benefit from ductility-related reductions in seismic demand. The resulting verifications are intentionally more restrictive, ensuring adequate safety margins in accordance with §7.3.3 and §7.8 of the Code. The design spectrum was defined on the basis of the seismic hazard parameters corresponding to the site of Montalto di Castro (VT), extracted from the official national hazard map and processed in compliance with NTC 2018 [1]. The reference structure belongs to Use Class IV (critical civil protection infrastructure), which entails the most stringent seismic performance requirements and an importance factor γ greater than unity. Accordingly, both the Ultimate Limit State (SLV) and the Serviceability Limit State (SLO) spectra were constructed as elastic acceleration spectra, taking into account the local soil category and stratigraphic conditions as determined by the geotechnical survey. Figure 6.7 shows the elastic design spectra adopted for the analysis, corresponding to SLV and SLO at Montalto di Castro. These curves

represent the seismic demand applied to the structural model and form the basis for the evaluation of the building's seismic response.

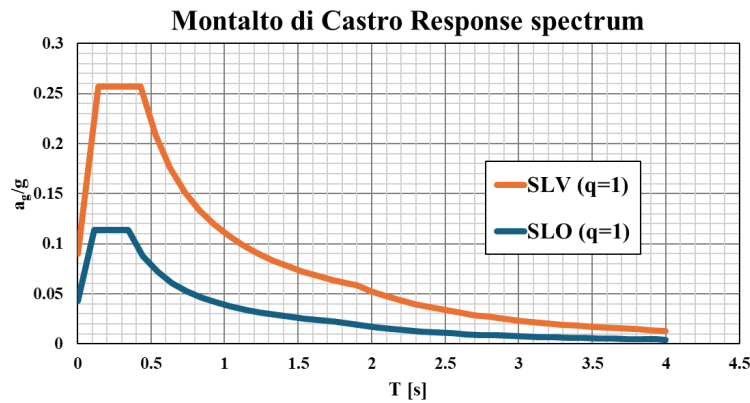


Figure 6.7. Design Response Spectrum Montalto di Castro (VT), Italy.

The numerical analysis was carried out using SAP2000, a finite element software widely used in structural engineering. The walls produced through 3D Concrete Printing (3DCP), which constitute the above-ground portion of the control room, were idealised as frame elements. In SAP2000, frame elements are formulated in three dimensions and are capable of capturing the full range of structural actions, including biaxial bending, torsion, axial deformation, and biaxial shear deformations [18]. This modelling strategy ensured a consistent representation of the experimental configuration and allowed the structural response to be evaluated with adequate accuracy.

6.4.1 Structural Elements

The structure was conceived and modelled using the equivalent frame approach. This method is based on the assumption of a simplified

scheme in which the real system is represented by a set of one-dimensional elements (either linear or nonlinear), properly oriented and interconnected to reproduce the actual seismic response of the structure. Such an assumption is considered reasonable whenever the structure under investigation is sufficiently regular, thereby meeting the fundamental requirements of the equivalent frame idealisation. Within the framework of the equivalent frame method [19], the wall is schematized as a frame, where infinitely rigid nodes are assigned at the intersections between the 3D printed concrete walls and the spandrel beams. For instance, in the case of a multi-storey wall with openings, loaded in its plane, the system can be idealised as an equivalent frame composed of vertical wall elements, horizontal spandrel elements, and rigid nodes. The wall and spandrel elements are modelled as frame members deformable in both axial and shear response, while the nodes are idealised as infinitely rigid links, located at the ends of wall panels and spandrels. In the present case study, the structural system is composed of deformable elements, namely the 3D printed wall panels, and rigid elements, represented by the floor spandrels, which include both the slab and the perimeter edge beams. The modelling scale is at the level of the structural element (i.e., the single 3D printed concrete panel). Consequently, the approach necessarily requires the explicit geometric definition of all structural members, including walls and spandrels. The equivalent frame schematization adopted in SAP2000 is illustrated in Figure 6.8. More specifically, Figure 6.8(a) shows a lateral view of the model, where the deformable beam element representing the 3D printed wall is

highlighted in red, and the rigid element (SAP2000 link element) representing the spandrel is highlighted in blue. The spandrel corresponds to the area enclosed by the green perimeter, which in this case consists of the roof slab and the perimeter beam. Similarly, Figure 6.8(b) shows the same elements represented in a frontal view of the analysed structure.

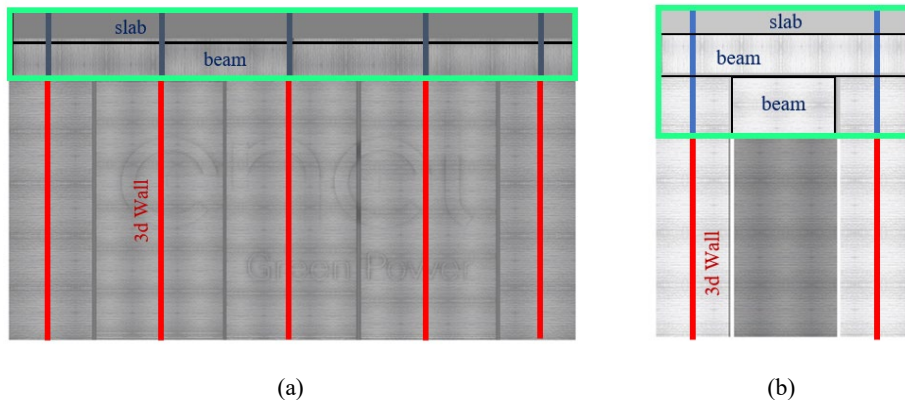


Figure 6.8. Equivalent frame schematization of the structure in SAP2000: (a) side view highlighting in red the deformable element (3D printed concrete wall) and in blue the rigid spandrel element; (b) front view with the same distinction between deformable and rigid elements.

To provide further clarity, the structure can be idealised as a shear-type frame [20], with a fixed support at the base and rotations restrained at the top, thus corresponding to a double-pendulum scheme at the roof level. For the sake of illustration, Figure 6.9 shows the simplified analytical model adopted for the case-study structure. In the Figure 6.9, the deformable elements are highlighted in red, each characterised by an equivalent stiffness K_0 , which is a function of both shear and flexural stiffness, as expressed in Eq. 4.4. The spandrel element is highlighted in green and is subjected to the equivalent

seismic shear force, V , which produces a corresponding top displacement, δ_{top} .

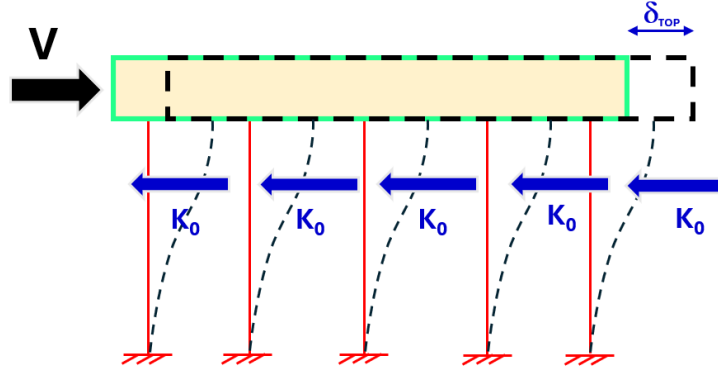


Figure 6.9. Shear-type frame idealisation of the control room structure: deformable wall elements (in red) with equivalent stiffness K_0 accounting for shear and flexural contributions, and rigid spandrel element (in green) subjected to the equivalent seismic shear force V , producing a top displacement δ_{top} .

In the linear range, and under the shear-type frame assumption, the relationship between the lateral force V and the corresponding top displacement δ_{top} is governed by the equivalent stiffness K_0 , as expressed in Eq. 6.13.

$$V = \sum_{i=1}^n K_{0,i} \cdot \delta_{top} \quad \text{Eq. 6.13}$$

With n total of deformable elements (red beams in Figure 6.9).

6.4.2 Equivalent Structural Modelling in SAP2000

On the basis of these principles, the structure was modelled in the structural analysis software SAP2000. In particular, the panels produced through 3D Concrete Printing (3DCP), which constitute the vertical structural system, were idealised as frame elements. All

beam–column members were defined using a general three-dimensional frame formulation, capable of accounting for biaxial bending, torsion, axial deformation, and biaxial shear deformations [18]. Special attention was given to the corner panels, each of which was modelled by means of two frame elements oriented along the two principal directions. Figure 6.10 shows the discretisation adopted for the deformable panels in SAP2000. For clarity and consistency, different colours were assigned to distinguish between the various geometries whenever necessary. Each deformable beam element was positioned at the centroid of the corresponding 3D printed wall panel. More specifically, the main wall panels are highlighted in green, while the corner panel is subdivided into two deformable elements, represented in purple and red, respectively. Thus, Figure 6.10(a) illustrates the areas of the panels considered in the analysis, highlighted with the aforementioned colours, whereas Figure 6.10(b) presents the corresponding SAP2000 schematization in an extruded view, preserving the same colour convention.

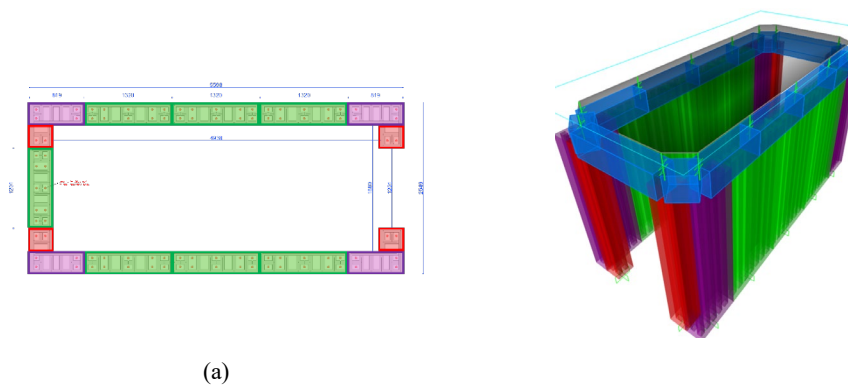


Figure 6.10. Discretisation of the deformable panels adopted in the SAP2000 model: (a) highlighted panel areas according to their geometry, with the main panels in green and the

corner panel subdivided into two elements in purple and red; (b) corresponding SAP2000 schematization in an extruded view, using the same colour convention.

More specifically, the deformable beam element was defined by assigning the exact geometry of the corresponding structural member, as previously reported in the preceding chapter. Conversely, the rigid element was modelled as an infinitely stiff link. Figure 6.11 shows a standard SAP2000 view of these elements. Finally, in order to correctly transfer the seismic load to the entire structure, the roof slab was modelled as a rigid diaphragm, connected to all the rigid link elements. A standard view of this modelling strategy, as implemented in SAP2000, is also presented in Figure 6.11.

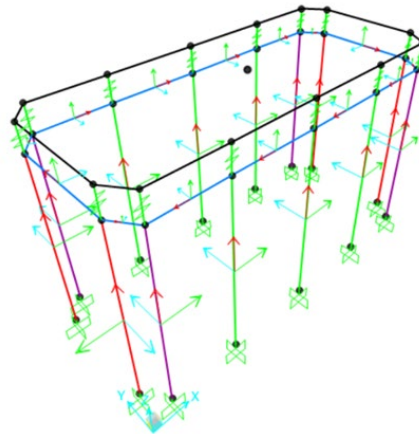


Figure 6.11. Standard SAP2000 view: deformable elements are represented with the actual panel geometry, while rigid elements are modelled as infinitely stiff links; the floor slab is idealised as a rigid diaphragm connected to all link elements.

6.4.3 Analysis of Loads, Seismic Mass Evaluation and Load Combinations

The evaluation of the loads acting on the structure and the corresponding seismic masses was carried out in accordance with EN

1991-1 (Actions on Structures) [21] and with the prescriptions of the Italian Building Code (NTC 2018) [1]. According to the code, loads are classified on the basis of their variability over time into permanent actions G , variable actions Q , accidental actions A , and seismic actions E . For the present case study, accidental actions were disregarded, as they are not relevant to the building considered. Permanent actions include both structural self-weight (G_1) and non-structural permanent loads (G_2). Variable and Accidental actions, Q and A , consist of live loads, wind, snow, and temperature effects. Seismic actions are defined through the design response spectra at the site of Montalto di Castro (VT), as detailed in §3.2 of NTC 2018 [1]. The floor system of the control room is a prefabricated joist slab with infill blocks and a topping slab, leading to a structural permanent load of approximately 3.0 kN/m^2 , while the contribution of non-structural elements was estimated at 2.5 kN/m^2 . With reference to NTC 2018 [1], category H ("accessible roofs for inspection and maintenance"), the live load was taken as 0.5 kN/m^2 . The snow load, derived according to §3.4 of NTC 2018, was evaluated based on the site altitude (a_s equal to 42 m.s.l.m.) and zone III classification, resulting in a design value of 0.48 kN/m^2 . Wind loads were determined in accordance with §3.3 of NTC 2018 [1], using the site parameters (a_s equal to 42 m.s.l.m., exposure class I, roughness class D, seismic zone 3). This led to a design wind pressure of approximately 0.456 kN/m^2 . In order to evaluate the seismic demand, the seismic mass was calculated by assigning a lumped joint mass at the centre of the rigid diaphragm representing the roof slab in SAP2000. The translational seismic mass along the horizontal degrees of freedom was computed as the sum of

the individual seismic weights of the structural components divided by the acceleration of gravity. For the building under investigation, this resulted in a total translational seismic mass of approximately 24.3 tons. The rotational seismic mass, associated with torsional effects about the vertical axis, was obtained by multiplying the translational mass by the square of the radius of gyration, evaluated from the building's plan dimensions. This led to a rotational seismic mass of about 76.5 ton·m². The design seismic action was defined according to NTC 2018 [1] for an importance class IV building, corresponding to critical civil protection infrastructure, with a nominal design life of 50 years and an importance factor, γ_I , equal to 2.0. The elastic spectra at both the Ultimate Limit State (SLV) and the Serviceability Limit State (SLO) were adopted, with no reduction due to ductility ($q=1$) in compliance with §7.2.2 of NTC 2018 [1], which prescribes non-dissipative behaviour for weakly reinforced elements. Load combinations were developed following §2.5 of NTC 2018 [1]. These include the fundamental combination for ultimate limit states, the characteristic (rare), frequent, and quasi-permanent combinations for serviceability limit states, as well as the seismic combination. For seismic checks, the three earthquake components (E_x , E_y , E_z) are considered, with the vertical component neglected in this case as permitted by §7.2.2. In addition, accidental eccentricities ($\pm 5\%$ of the plan dimensions along both axes) were introduced by shifting the joint mass in SAP2000, resulting in four distinct seismic models. From these, a total of 32 seismic combinations were generated, together with the fundamental combination at SLU and two additional wind-

dominant combinations for overturning checks. Finally, in order to ensure consistency between stiffness and mass distribution, the centre of rigidity was evaluated and compared with the geometric centroid of the structure. Given the symmetry of the structural layout, the eccentricity between the two was found to be negligible, confirming the adequacy of the shear-type modelling.

6.4.4 Dynamic Properties

Before implementing the elastic spectrum and the load combinations required for seismic demand evaluation, it is necessary to present the dynamic properties of the control room structure, as obtained from the modal analysis performed in SAP2000. From a theoretical perspective, the fundamental period of a structure is defined as the time it takes to complete one full oscillation under free vibration, following an initial disturbance. In the simplest case of a single-degree-of-freedom oscillator (SDOF), the fundamental period can be expressed, in Eq. 6.14, as a function of the system mass m and stiffness k :

$$T = \sqrt{\frac{m}{k}} \quad \text{Eq. 6.14}$$

where T is the fundamental period, m is the effective vibrating mass, and k is the lateral stiffness of the system (^{22;23}). In the literature, several empirical formulations are available that relate the fundamental period of a structure to its total height, often calibrated on experimental databases of existing buildings (^{24;25}). For regular single-storey structures in concrete, it is generally accepted that the fundamental period is of the order of 0.10 seconds. Considering the

building examined here—a rigid, single-storey control room with limited height—the modal analysis provides a fundamental period of 0.07 seconds, which is fully consistent with theoretical expectations [26].

6.4.4.1 Modal Analysis and Mass Participation

According to §7.3.3.1 of the NTC 2018 [1], a sufficient number of vibration modes must be considered so that the total participating mass ratio exceeds 85% of the total seismic mass of the system. Moreover, modes with individual participating mass greater than 5% should not be neglected. In the present case, 35 modes were extracted in the modal analysis, ensuring more than 95% of the seismic mass was captured in both horizontal directions, thus satisfying the code requirements. Each mode contributes to the overall seismic response through its own base shear, which depends on the product of the spectral acceleration corresponding to the mode's period and the modal participating mass. Neglecting higher modes with significant participation would result in underestimating the seismic action transferred to the structure. In the present case, the structure is a single-storey and very stiff enclosure. Consistently with theory and empirical ranges, the SAP2000 modal analysis yields:

- Mode 1 (X-translation, short side – least stiff direction): $T_1 \approx 0.07\text{s}$.
- Mode 2 (Y-translation, long side – stiffer direction): $T_2 \approx 0.05\text{ s}$.
- Mode 3 (torsion about Z): $T_3 \approx 0.03\text{ s}$.

Figure 6.11 of the previous section illustrates the global reference system adopted here, where X corresponds to the short side of the building, Y to the long side, and Z to the vertical axis. According to NTC 2018 §7.3.3.1 [1], a sufficient number of modes must be included so that the cumulative participating mass exceeds 85 percent of the total seismic mass in each principal direction, and individual modes with participation above five percent should not be neglected. In this analysis 35 modes were extracted; the cumulative dynamic mass captured (modal output in SAP2000 “Dynamic Percent”), demonstrating that the modal basis adequately represents the system response [23]. This confirms the adequacy of the modal solution and validates the shear-type modelling approach adopted.

The results of the modal analysis are summarised in Table 6.2. Summary of Dynamic Properties., which reports the fundamental periods associated with the first translational and torsional modes of the structure, together with the corresponding cumulative participating mass ratios. Specifically, the Table 6.2 highlights that the first mode, with a period of approximately 0.07 seconds, corresponds to translation along the X direction (the short side of the building), which is the least stiff direction. The second mode, with a shorter period of 0.05 seconds, is associated with translation along the Y direction (the long side of the building), reflecting the higher stiffness of this axis. The third mode, with a period of about 0.03 seconds, is a torsional mode about the vertical axis (Z). The cumulative participating mass ratios confirm that nearly the entire seismic mass is captured in the first few

modes: about 99.7% in the X direction, 95.6% in the Y direction, and almost 100% in torsion about Z. These values demonstrate that the modal basis is sufficient to represent the dynamic behaviour of the structure in accordance with the requirements of NTC 2018 [1].

Direction / DOF	Description	Fundamental period (s)	Cumulative participating mass (Dynamic %)
X (short side)	Translational Mode 1 (least stiff)	0.07	99.7
Y (long side)	Translational Mode 2 (stiffer)	0.05	95.6
Z (about Z)	Torsional Mode 3	0.03	99.9

Table 6.2. Summary of Dynamic Properties.

6.4.5 Structural Checks in line with NTC2018

The seismic verification of the control-room building was carried out in accordance with the Italian Building Code NTC 2018 [1]. Checks were performed both at the Ultimate Limit State (SLV) and at the Serviceability Limit State (SLO), considering the peculiar mechanical behaviour of the adopted material, namely 3D printed concrete (3DPC). Given the lack of a dedicated normative framework for 3D printed concrete, the material was conservatively assimilated to masonry, i.e. a non-dissipative and brittle system, as prescribed by §7.2.2 of NTC 2018 [1] for structural elements that cannot exploit ductility. In this framework, the seismic demand was considered with q equal to 1.0, and verifications were performed under the assumption of elastic response, consistent with the treatment of unreinforced or weakly reinforced brittle elements [26;27].

6.4.5.1 Checks at SLV (Ultimate Limit State)

At the Ultimate Limit State, the structure was verified according to Table 7.3.III of NTC 2018, which specifies checks for masonry and weakly reinforced concrete walls. In plane checks included:

- Shear sliding failure (SS), governed by cohesion and friction along potential sliding planes, verified through the Mohr–Coulomb failure criterion [28].
- Diagonal shear cracking (DC), checked against the condition of principal tensile stresses exceeding the tensile strength of the material, leading to diagonal cracking patterns [26].
- Flexural Compression (FC), verified by means of axial force–bending interaction domains, in line with §4.1 and §7.8 of NTC 2018 [1].

Furthermore, out-of-plane verifications included:

- Overturning instability, assessed through equilibrium between seismic inertia forces and stabilising vertical loads (§7.8 NTC 2018).
- Out-of-plane flexure, accounting for slenderness and limited reinforcement, following prescriptions for brittle walls [29].
- Global stability, verified to exclude local buckling phenomena.

These verifications confirmed that the structural members, considered as brittle elements, satisfied the resistance (RES) checks under the elastic seismic action at SLV.

6.4.5.2 Checks at SLO (Serviceability Limit State)

At the Serviceability Limit State, the focus is shifted from strength to rigidity (RIG), in line with §7.3.6.1 of NTC 2018. For brittle materials, such as masonry and, by analogy, 3D printed concrete, the governing parameter is the maximum interstorey drift. According to NTC 2018, for ordinary masonry structures, the allowable drift at the Damage Limit State (SLD) is set to 0.002 (2 ‰). Following consolidated practice [30; 31], the drift limit at the Serviceability Limit State (SLO), $\theta_{max,SLO}$, is assumed equal to two-thirds of the SLD value, corresponding to Eq. 6.15.

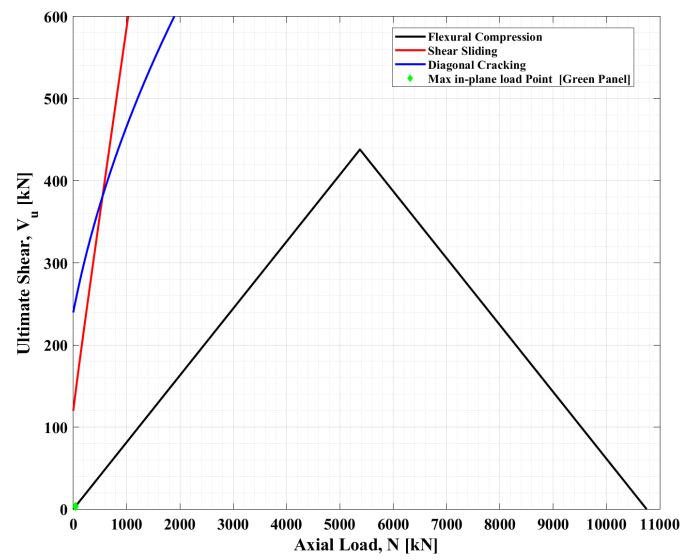
$$\theta_{max,SLO} = \frac{2}{3} 0.002 = 1.33 \text{ ‰} \quad \text{Eq. 6.15}$$

In the present case, the control-room structure in 3D printed concrete was conservatively assigned the same drift limit of 1.33 ‰ at SLO, reflecting its brittle nature and ensuring consistency with the treatment of masonry in seismic codes. Displacements and interstorey drifts obtained from the linear dynamic analysis were compared against this threshold. Results confirmed that the building maintains adequate serviceability performance, preventing excessive damage to non-structural elements and ensuring continued functionality under operational seismic events.

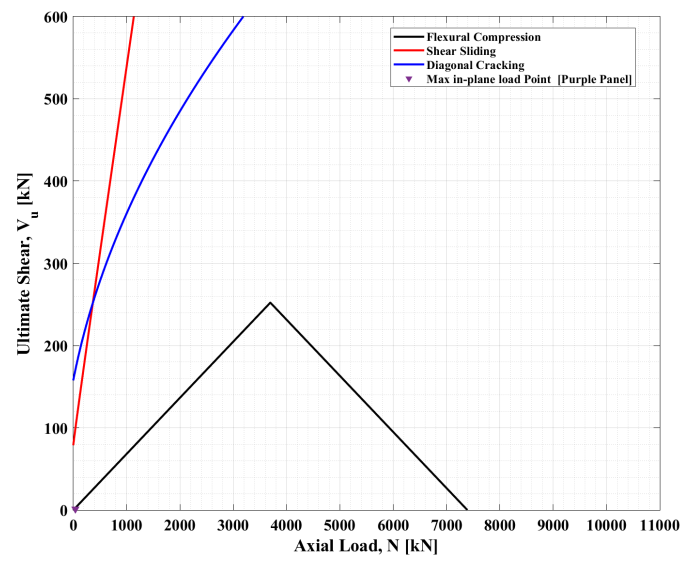
6.4.6 Summary of Seismic Structural Checks: Comparison with Resistance Domain in Plane

For completeness, and in order to provide a direct comparison with what was discussed in the chapter 4 on resistance domains, Figure 6.12 illustrates the resistance domains of the three representative wall

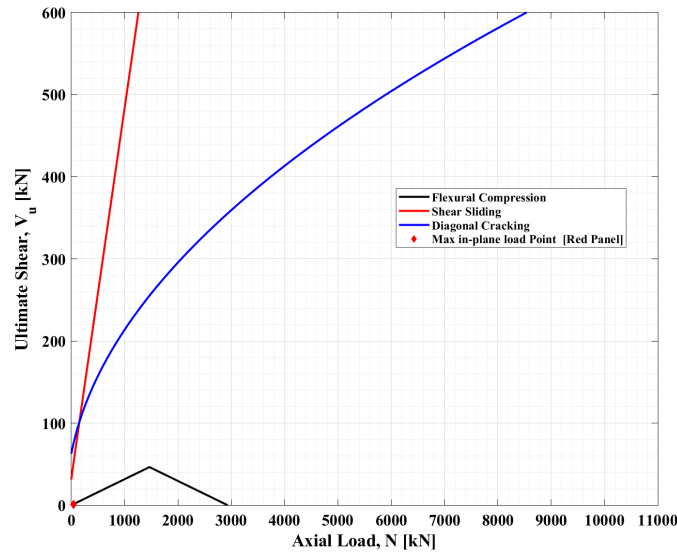
types analysed (Figure 6.10). In particular: panel (a) corresponds to the green wall, panel (b) to the purple wall, and panel (c) to the red wall.



(a)



(b)

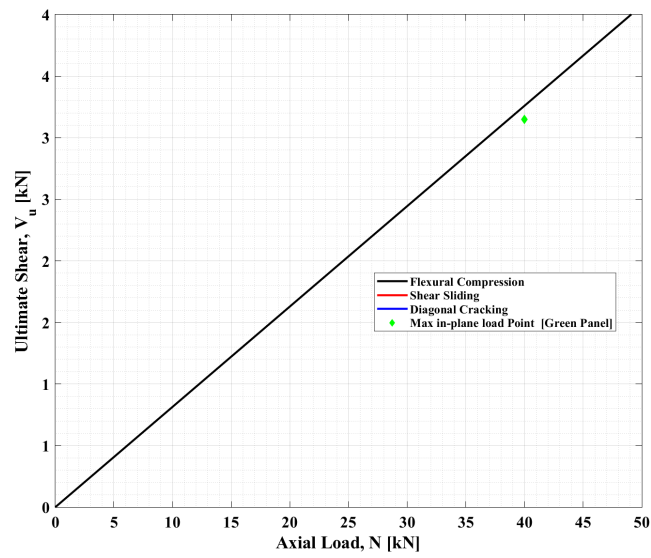


(c)

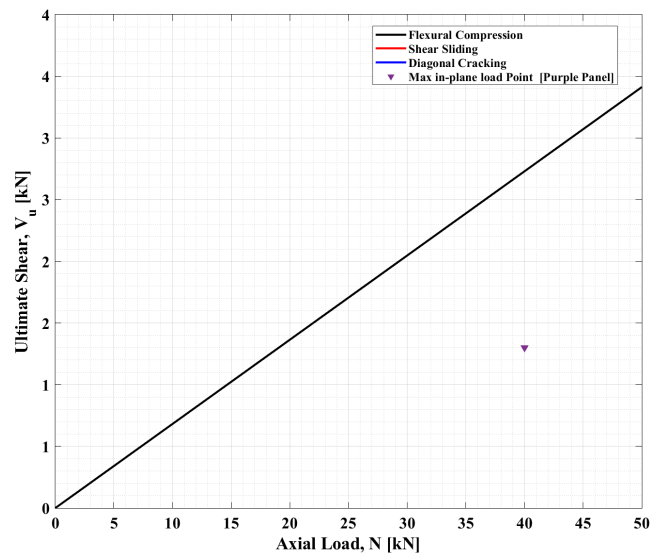
Figure 6.12. Resistance domains and checks for the three representative 3D printed wall panels: (a) green panel, (b) purple panel, and (c) red panel.

Since no specific design code currently exists for 3D printed concrete, the walls were assimilated to unreinforced and therefore brittle elements, in analogy with the provisions of NTC 2018 for plain concrete and masonry. On this basis, the tensile strength was conservatively assumed equal to zero, as prescribed for fragile systems. The verification domains were constructed by plotting the maximum shear and axial actions obtained at the ultimate limit state (SLU). Specifically, for the green panel, the design axial force, $N_{Ed,SLU,green}$, was taken equal to 40kN, and the corresponding maximum shear, $V_{Ed,SLU,green}$, equal to 3.15 kN; For the purple panel, the design axial force, $N_{Ed,SLU,purple}$, was taken equal to 40 kN, and the corresponding maximum shear, $V_{Ed,SLU,purple}$, equal to 1.30 kN; And finally, for the red panel, the design maximum axial force, $N_{Ed,SLU,red}$,

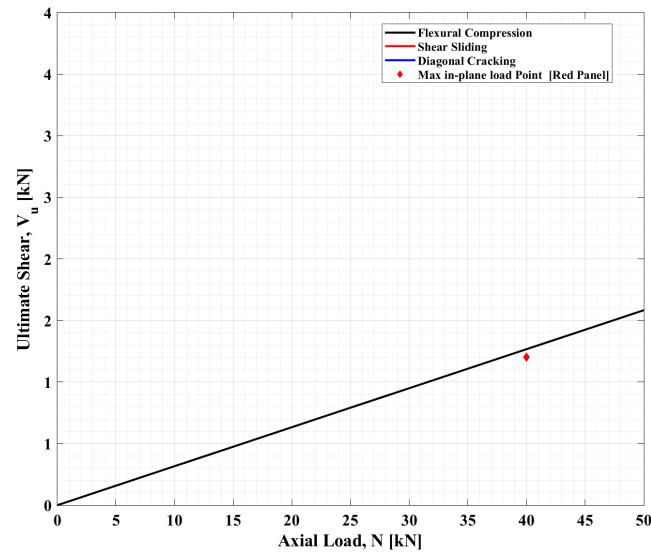
was taken equal to 40 kN ,and the corresponding maximum shear, $V_{Ed,SLU,red}$, equal to 1.20 kN; As can be observed in the figure, the points representing the design actions for each wall fall well within the corresponding resistance domains. In addition, Figure 6.13 provides a zoom-in view of the resistance domains previously shown in Figure 6.11, in order to offer a clearer visualization of the verification points. It can be clearly observed that the points of maximum design action for the green (a), purple (b), and red (c) panels all lie within their respective resistance boundaries. This confirms that all verifications are satisfied under in-plane seismic loading, further validating the conservative modelling assumptions adopted for the 3D-printed walls.



(a)



(b)



(c)

Figure 6.13.: Zoom-In of Resistance domains and checks for the three representative 3D printed wall panels: (a) green panel, (b) purple panel, and (c) red panel.

It is worth noting that, in the present case, in accordance with the Italian standard code, the tensile strength was neglected in the ultimate limit state verifications. As already discussed in previous sections, including tensile strength would further increase the resistance values; however, neglecting it ensures consistency with the code and results in a safety and more conservative design approach.

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7 Conclusions and Future work

This PhD work develops a design-oriented pathway for unreinforced 3D-printed concrete (3DPC) wall elements that begins at the material scale—where process parameters such as interlayer time (ILT) control tensile and interface properties—and culminates in structural-scale resistance domains and abaci that quantify how geometry and axial load interact with process-driven tensile capacity. The central idea is that 3DPC cannot be treated as a conventional, homogeneous continuum: the printing process imprints anisotropy and interface sensitivity that must be made explicit in design checks. To that end, the work translates process variables into structural descriptors in a way that is both analytically transparent and usable within current code logics. The derivations are tested against experimental evidence, exercised through parametric studies, and finally deployed on a real-world project, showing that a consistent way to safe design is already available—even in the absence of a dedicated standard—provided the mechanics specific to 3DPC are kept in view.

A key methodological choice underpins all comparisons: every resistance domain and abacus is generated for a wall section with a constant void ratio, aligned with the reference section used throughout the thesis. This constant-porosity assumption removes an otherwise confounding geometric variable (compactness of the printed section), so that trends can be attributed to slenderness, axial load, and tensile capacity. From this experimental and analytical investigation it turned

out that, the dominant failure mode for the examined typologies is flexural compression, while shear-sliding and diagonal-tension appear only in delimited regions of the parameter space associated with stocky geometries and/or degraded tensile performance. In addition, the calibration of a linear–elastic model in SAP2000 enabled the structural analysis of the Montalto di Castro case study. This allowed the verification of the printed structural elements by considering their brittle behaviour, thereby framing them within the current regulatory context. A schematic overview of this end-to-end pathway work is presented in Figure 1.1, which synthesises the research steps.

7.1 Summary of Results and original Contributions

The first contribution is a unified framework of resistance domains that simultaneously represents the three canonical in-plane mechanisms—flexural compression (FC), diagonal cracking (DC), and shear-sliding (SS)—and makes their competition explicit as a function of slenderness λ , axial load N , and tensile strength f_{ctk} . Rather than discussing mechanisms separately, the domains allow a direct, visual reading of which collapse mode governs for any given triplet (λ, N, f_{ctk}) , and by how much the ultimate shear V_u changes when one parameter is varied at fixed values of the others. This structure provides the logic to design 3d printed walls with different aspect ratios, boundary restraints, and vertical precompression.

A second contribution is the explicit identification of slenderness. In particular, the parametric analyses, constructed at constant porosity,

show that for the reference section the governing mode remains FC across a broad range of axial loads; the transitions toward SS or DC cluster in the stocky regime and align with reductions in tensile capacity. More specifically, increasing N initially stabilizes the element by mobilizing friction along weak planes and enhancing compression block capacity, and only beyond a large threshold the axial load pushes the section toward crushing-controlled failure, offering ductility benefits. This effect of N is also captured in the abaci through iso- V_u curves, which represent not only the ultimate shear capacities under seismic actions (in-plane) for the analysed section, but also account for variations in the tensile strength of the structural element. This approach aims to explore the widest possible range of failure mechanisms, providing a mechanical interpretation and paving the way for further validation of these abaci. Infact, a third contribution concerns process-to-design translation. ILT, a process variable often confined to material, is here a structural parameter via its impact on structural scale. Framed in this way, the abaci quantify how improving interlayer adhesion (shorter ILT, better cure overlap, controlled environment) shifts the operating point across the domains: capacities increase systematically, yet the governing mechanism for slender panels remains FC because the stress resultants and the compression block geometry dominate the collapse sequence. This result is not a limitation of the approach but a characterization of the material system: plain, unreinforced 3DPC is intrinsically brittle in flexure, and enhanced tension improves strength more than it modifies the failure hierarchy.

The fourth contribution is methodological. A lightweight Timoshenko-beam representation, calibrated on real bending properties and effective shear area, reproduces the measured elastic stiffness with excellent accuracy. This is crucial because code checks for brittle systems are anchored to the elastic branch: if stiffness and force demand are reliable there, domain-based strength verifications become meaningful and conservative without resorting to heavy non-linear models. This Ph.D. work also demonstrates end-to-end applicability on an actual printed structure, showing how resistance domains, abaci, and simplified analysis can be embedded in the workflow of NTC-style checks. The overall outcome is a compact toolset that makes 3DPC design decisions transparent, auditable, and reproducible.

7.2 Implications for the Structural adoption of 3DPC

The implications of these findings are twofold — technical/scientific and procedural/standard. From a technical standpoint, the domains and abaci provide an immediate mapping from geometry, axial load, and tensile capacity to both the governing failure mechanism and the ultimate lateral strength. Since the construction is performed at constant porosity, the observed capacity trends can be interpreted as the pure effects of λ , N , and f_{ctk} .

This implies the need for an extensive experimental campaign to verify whether, under different levels of normal stress, varying slenderness

ratios, and tensile strengths, these abaci—as currently formulated—remain valid.

From a procedural standpoint, with respect to the current regulatory framework, it is clear that such elements—currently classified under existing codes as weakly reinforced, brittle members—are treated as having negligible tensile strength. The standards therefore prescribe neglecting their tensile contribution. However, these elements exhibit material-scale characteristics that are not directly reproducible at the structural scale. Due to their layered nature, each interlayer effectively acts as a mortar joint, similar to masonry construction. Consequently, as in masonry, it becomes necessary to characterise not only the material itself but also the structural element. Furthermore, because the geometry and, consequently, the printing direction of the layers can vary significantly, another variable is introduced at the structural scale: the same printed element may exhibit different behaviours depending on the layer orientation.

While the experimental evidence and analytical formulations presented in this work provide a consistent framework of the in-plane behaviour of 3D-printed concrete (3DPC) panels at the element scale, extending these results to multi-story buildings requires careful consideration of scale effects, boundary conditions, and load distribution in real structural systems.

- A first aspect concerns the transition from isolated panels to wall assemblies. Panel-scale tests typically involve well-defined and often idealised boundary conditions—such as fixed bases

or controlled top restraints—that do not fully reproduce the interaction between adjacent structural elements, diaphragms, and vertical load paths. In real buildings, walls are part of a continuous load-bearing system where geometry, connectivity, and stiffness distribution influence the global response. The in-plane behaviour of an isolated panel may differ significantly from that of a wall embedded in a multi-story configuration, not only in terms of flexural stiffness, crack localisation, and shear transfer mechanisms, but also because the internal stress distribution varies with the type of panel-to-panel connection and the degree of global continuity. Moreover, in real buildings, out-of-plane effects and diaphragm–wall interactions may further influence the overall response, leading to behaviours that cannot be captured by isolated panel testing (in-plane and out of plane interaction).

- A second important consideration is the variation of axial load with building height, which produces differentiated failure modes along the structure. In low to mid-height 3DPC buildings, the lower stories typically higher compressive forces, which may suppress sliding-controlled shear failures and promote flexure-controlled behaviour. Conversely, upper stories, where axial load is smaller, are more sensitive to diagonal cracking or interface-governed mechanisms. The resistance domains developed in this work inherently capture the interplay between axial load and failure mode; however, their application along the height of a multi-story structure must account for the vertical

distribution of seismic forces and the cumulative effect of gravity loads.

- A third issue relates to geometric scaling. The panels tested in this thesis have thickness, void ratio, and printing patterns that are representative of real structural walls; however, modifying these geometric ratios for different design requirements may lead to variations in structural response that extend beyond the configurations examined here. Although the analytical framework developed in this work accounts for changes in slenderness, axial load, and sectional properties, a broader set of large-scale experimental tests is still required to fully validate these predictions across the wider spectrum of geometries expected in practical 3DPC construction.

Finally, global dynamic behaviour introduces additional considerations. Multi-story buildings exhibit mode shapes, torsional responses, and mass–stiffness distributions that cannot be captured in single-panel tests. Although the simplified numerical model developed in this thesis (Timoshenko beam analogy) provides a first bridge between element-scale behaviour and global seismic verification, comprehensive validation will require dynamic testing or hybrid experimental–numerical campaigns on taller, full-scale 3DPC walls. In addition, shake-table tests on both single-story and multi-story 3DPC assemblies will be essential to capture the true dynamic response of these structures, including the interaction between in-plane and out-of-plane forces, and to validate the seismic performance under realistic ground-motion excitation. In summary, panel-scale

results constitute a necessary foundation for understanding the mechanics of 3D-printed concrete, but their transposition to real buildings must account for axial load gradients, slenderness effects, diaphragm–wall interactions, and the redistribution of forces across interconnected wall systems. Additional factors such as the type and continuity of panel-to-panel connections, potential out-of-plane contributions, and geometry-dependent scaling effects may further influence the global response. These considerations highlight the need for extended large-scale experimental programmes—including full-height wall tests and shake-table dynamic testing—as well as refined numerical modelling efforts, to fully bridge the gap between element-scale behaviour and the structural performance of single-story and multi-story 3D-printed concrete buildings.

7.3 Future Work: dedicated standards, further testing, extended analytical and numerical validation

The next logical step of this work concerns the consolidation of a robust design framework and the definition of a standardisation path for 3D printed concrete structures. While this thesis has demonstrated the feasibility of process-aware and code-consistent design checks, further progress depends on expanding the experimental, analytical and regulatory foundations of the field.

At the experimental level, systematic and parametric testing campaigns are required to validate and generalise the proposed

resistance domains under a wider range of boundary conditions, slenderness ratios and stress states. Such campaigns should independently vary interlayer time (ILT), layer orientation, internal geometry and void content, enabling the identification of distinct mechanical “families” of printed components—an approach analogous to the classification already established for masonry (e.g., tuff, solid brick, hollow clay units). These characterisations should include not only tensile, shear and compressive properties, but also the evolution of stiffness and damage under cyclic and long-term loading, including interface behaviour under multiaxial stress states.

Once defined at material scale, these mechanical families must be validated at wall scale through dedicated in-plane cyclic tests and full-scale assemblies subjected to combined in-plane and out-of-plane loading (including shake-table testing). Such multi-scale datasets would enable the statistical calibration of characteristic values and process-related safety factors (α_{3DPC}) for future inclusion in NTC and Eurocode provisions.

At the procedural and normative level, the development of dedicated guidelines or annexes within the NTC and Eurocode frameworks is essential. These documents should formalise the treatment of printed anisotropy, interlayer bonding, internal void patterns, and process control parameters (e.g., ILT windows, deposition speed, filament geometry) as explicit design variables. The integration of these aspects is necessary to place 3D printed concrete on a consolidated regulatory footing comparable to traditional materials.

Further developments should also extend the resistance-domain framework to reinforced and hybrid 3DPC systems, particularly those incorporating steel bars, or polymer reinforcements. The interaction between reinforcement and the printed matrix introduces additional mechanical phenomena that require dedicated modelling and testing, especially in shear-dominated and cyclic regimes. Finally, a key direction for future research is the digital integration of design and fabrication. Embedding resistance domains, mechanical families and process–structure relationships into parametric, algorithmic and BIM-based design tools would enable real-time feedback between printing parameters and structural performance. This would close the loop between process control and design verification, fostering a new generation of performance-driven, fabrication-aware structural design methods for 3D printed concrete.

In summary, the continuation of this work should bridge the remaining gap between research-scale demonstration and engineering practice, ensuring that 3D printed concrete structures can be designed, verified and standardised with the same level of confidence and transparency as conventional structural materials.

Author's publications

Insert here the list of your scientific publications during the PhD, e.g.:

1. Giulivo Marco, et al. "Out of plane behaviour of innovative infill walls made of aerated autoclaved concrete (AAC) subjected to shake table test." *Frontiers in Built Environment* 9: 1168241.
2. Giulivo Marco et al. "Experimental Behavior of a 3D Printed Concrete Wall with Fixed Base Anchorage Subjected to In-Plane Cyclic Loads" <https://doi.org/10.24355/dbbs.084-202408190801-0>
3. Giulivo Marco et al. "The Effect of Printing Path on the Shear Behavior of 3D Printed Concrete Panels Under Diagonal Compression Test" <https://doi.org/10.24355/dbbs.084-202408190840-0>
4. Giulivo, Marco, Vittorio Capozzi, and Costantino Menna. "Experimental and analytical assessment of the in-plane behaviour of 3D printed concrete walls subjected to cyclic loads." *Engineering Structures* 345 (2025): 121516.
5. Mudadu, A., Giulivo, M., Giulivo, G., Menna, C. The effect of printing path on the shear behavior of 3d printed concrete panels under diagonal compression test (submitted)

