



Mesoscale and macroscale modelling for nonlinear analysis of masonry wall structures under cyclic loading[☆]

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ABSTRACT

The paper investigates the nonlinear response of unreinforced masonry wall components under seismic loading. Finite element models formulated according to different scales of representation for masonry are compared, focusing on the hysteretic behaviour under in-plane cyclic loading up to collapse. To this aim, brick-masonry wall structures, including two panels with different aspect ratios and a large two-storey perforated wall, as well as block-masonry wall panels subjected to various vertical loads, all previously tested under horizontal cyclic loading, are analysed. The results of macro and mesoscale simulations are compared against the experimental findings to assess the ability to represent strength and stiffness degradation and the amount of dissipated energy. Emphasis is laid upon the objectivity of the definition and calibration of the model material parameters to assess how their selection affects the response predictions. The comparison highlights differences in predictive accuracy, numerical robustness, variability of results, and computational cost. The results show that the macroscale approach adopted herein, despite its computational robustness, yields widely variable results and may significantly deviate from experimental observations. In contrast, the mesoscale results appear less scattered and accurate, but the computational burden is far more significant.

1. Introduction

Over recent years, considerable effort has been devoted to developing numerical models for simulating the seismic response of unreinforced masonry (URM) structures. Accurate models are critical for realistic assessment under earthquake loading, where the response is particularly complex due to the masonry anisotropic nature and highly nonlinear behaviour. In current practice, the seismic assessment of existing masonry buildings is primarily carried out using nonlinear static analysis under increasing lateral loading (pushover analysis). While such an analysis procedure provides a satisfactory estimate of load capacity and global ductility, it does not explicitly allow for the distinctive hysteretic behaviour under cyclic loading conditions, which ultimately determines the actual seismic response up to collapse. The energy dissipated during an earthquake is essential for refined structural design and assessment. It is considered by the Capacity Spectrum Method in the ATC-40 [1], which requires the evaluation of the equivalent damping factor comprising hysteretic and viscous contributions. The former is

obtained by equating the energy the analysed system dissipates in a single hysteretic cycle to that of an equivalent linear damped system.

Several authors investigated the accuracy of different macroscale modelling strategies for URM components and structures considering the global load-displacement characteristics under monotonic loading. Extensive comparative studies were carried out in [2], where discrepancies in the capacity curves provided by (i) the equivalent frame (EF) approach [3] with different nonlinear material constitutive relationships for masonry, (ii) 2D macroelement models and (iii) nonlinear continuum finite element (FE) models, were highlighted. Calderoni et al. [4] investigated the use of the EF approach, paying particular attention to the influence of the geometric discretisation of masonry perforated walls. It was shown how the capacity curves vary significantly depending on the adopted numerical implementation of the method. De Falco et al. [5] studied the monotonic nonlinear response of masonry façade walls by changing the material properties within a probabilistic framework and showed how different modelling hypotheses significantly affect the probability of failure. In [6], a comparison among three

[☆] This paper is dedicated to the memory of Prof. Claudio Amadio (1954-2023).

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different material models for 3D continuum macroscale simulation of masonry walls, including the double Drucker Prager model [7,8], the total strain crack model [9,10] and the concrete damage plasticity (CDP) model [11,12], was carried out. In the same study, numerical comparisons under monotonic loading were also performed to evaluate the suitability of the material property ranges provided by the Italian building code [13] for various masonry types. The obtained numerical monotonic curves were compared against experimental results in the literature, showing significant discrepancies between the numerical and experimental curves for material properties within the indicated ranges of variation. Cattari et al. [14] explored the potential of using macroscale continuum FE descriptions with the CDP model and the EF approach to predict the local and global response of masonry panels and a representative masonry building façade subjected to horizontal monotonic loading. A cross-consistent calibration procedure was also proposed, where the material and geometrical characteristics of the equivalent frame models were calibrated based on the numerical predictions provided the baseline continuum FE model. The nonlinear response of unreinforced masonry wall structures was investigated in [15] using three 1D elastoplastic models implemented in commercial software for structural assessment. It was found that the different models provide close monotonic predictions but very different responses under cyclic loading conditions.

As pointed out before, most of the previous numerical research focused on predicting the monotonic behaviour of masonry wall structures under horizontal forces, disregarding the energy and dissipation characteristics under cyclic loading, which ultimately determine the response under earthquake conditions. Importantly, it was found that different models for URM may lead to quite dissimilar predictions, and using a sophisticated model does not necessarily imply more accurate results. Furthermore, more refined models typically require several material parameters, potentially not all quantifiable through standard material tests. Thus, since numerical predictions are greatly affected by the specific selection of the material properties for URM, the sensitivity of the global response to less known material parameters should be assessed appropriately before considering any refined model as a reference.

The present work investigates the ability of different advanced modelling approaches to reproduce the hysteretic behaviour of URM wall components. Special attention is paid on the effects of the calibration of the model material parameters on the prediction of the dissipated energy under cyclic loading conditions. FE models based on two different scales of representation for masonry are compared: detailed mesoscale models [16] accounting for masonry bonds explicitly, by a separate representation of masonry units and mortar joints, and continuum macroscale models, in which masonry is assumed as a uniform and isotropic material, adopting a damage-plasticity constitutive model for masonry [17]. The accuracy guaranteed by the two alternative modelling strategies is assessed through comparisons against experimental tests from the literature, considering several cases of brick-masonry and block-masonry wall structures with varying geometry, vertical loads, and support conditions. Emphasis is given to defining the material properties required by the two modelling approaches based on the results from standard material tests. Extensive parametric studies are conducted by varying the most critical and uncertain parameters within realistic ranges to evaluate their influence on the global response.

The paper explores aspects related to the numerical modelling of masonry structures under seismic loading, examining the ability of meso- and macroscale FE models to accurately capture the hysteretic response of masonry wall structures through objective material calibration. It offers both qualitative and quantitative assessments of different modelling approaches, focusing on prediction accuracy, numerical robustness, variability of results due to changes in material calibration, and computational efficiency. Moreover, it provides suggestions for material calibration strategies, including possible connections between material calibration at the macroscale and mesoscale.

2. Modelling techniques for URM

Several factors affect the behaviour of masonry structures, including the mechanical properties of the constituents, their interaction, and their spatial arrangement. Each of these characteristics has a distinct effect on the macroscopic behaviour of the masonry material. For these reasons, achieving an accurate prediction of the response of URM components and structures under extreme loading is a particularly challenging task.

2D and 3D FE modelling of masonry can generally follow three main computational strategies according to the level of accuracy desired:

- Detailed micro-modelling (Fig. 1a): units and mortar joints are represented by nonlinear continuum elements, while unit-mortar physical interfaces can be represented with zero-thickness cohesive interface elements [18] or an additional layer of continuum elements [19].
- Simplified micro-modelling or mesoscale modelling (Fig. 1b): expanded units are modelled by elastic continuum elements. Meanwhile, mortar joints and unit-mortar interfaces are lumped together in zero-thickness nonlinear interface elements [20–22]. More accurate approaches also consider the potential occurrence of cracks within the masonry units by using additional nonlinear interfaces to model unit fracture [16,23]. A similar approach to FE mesoscale is the discrete element method (DEM) [24–26], where interfaces are not modelled as joint elements but as contact boundaries between distinct blocks.
- FE continuum macro-modelling (Fig. 1c): units, mortar, and unit-mortar interfaces are not explicitly modelled since masonry is assumed to be a homogeneous material employing isotropic [6,17, 27] or anisotropic [28,29] nonlinear continuum material models. Alternative strategies for macroscale modelling include the use of macroelements [3,30,31], which represent large portions of masonry components.

Microscale and mesoscale modelling strategies typically rely on basic material tests conducted on individual units and mortar joints. This is important when dealing with existing structures of historical and architectural value, where only low-invasive in-situ tests can be performed [32]. Typical tests for the micro- and mesoscale characterisation of mortar interfaces include shear triplet tests to determine cohesion and coefficient of friction, cross-couplet tests to obtain the bond strength, and compressive tests on small masonry wallets or flat-jack tests for estimating masonry compression strength. Furthermore, splitting or tensile tests on units are conducted to assess the unit tensile strength, which is required for the brick-brick interfaces representing the development of cracking within masonry units. In the case of microscale modelling, mortar is explicitly defined; thus, compression and bending tests on mortar specimens should also be performed to achieve mortar compressive and tensile strength, respectively.

On the other hand, mesoscale or microscale models generally involve a high computational cost and a time-consuming pre-processing stage, preventing their use for the analysis of large masonry structures. As a result, microscale masonry models are typically employed for analysing small-scale problems, such as individual assemblages of units and mortar layers up to small masonry components [19,33,34]. Mesoscale approaches can be applied to larger problems, such as building parts (e. g. large perforated walls [23]), while applications to entire buildings are still possible but require superior computational resources [35]. Thus, the analysis of large URM structures under extreme loading is typically conducted using macroscale strategies, which offer computational efficiency but are characterised by more challenging material calibration, potentially resulting in inaccurate response predictions. Since the link between the properties of individual components and those of masonry at the macroscale is not straightforward, it may be necessary to conduct more invasive tests to characterise the shear-tensile response, such as

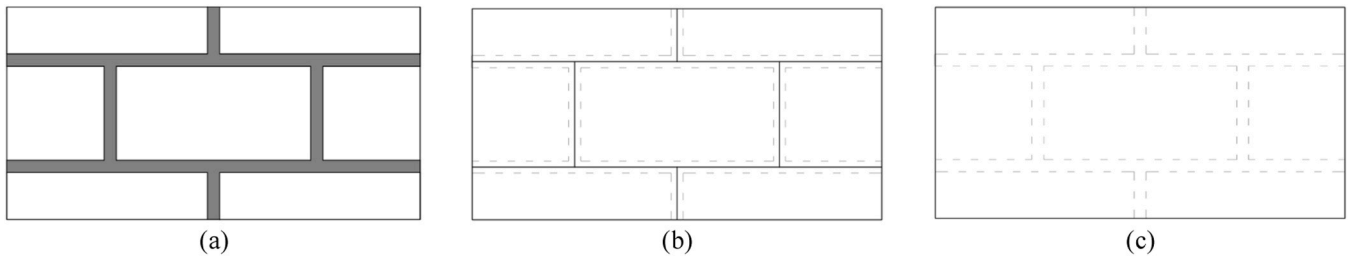


Fig. 1. Different scales of representation for FE modelling of masonry components: (a) microscale, (b) mesoscale and (c) macroscale approach.

diagonal compression tests or shear compression tests on large masonry panels [36].

In the following, the main characteristics of the mesoscale and macroscale modelling approaches for masonry used in this research and implemented in ADAPTIC [37], are presented.

2.1. FE Mesoscale approach

The 3D mesoscale description for URM structures herein adopted [16] is based on using 20-noded elastic solid elements combined with 16-noded zero-thickness nonlinear cohesive interface elements (Fig. 2a). The former represents expanded masonry units; the latter describes mortar joints and fracture interfaces inside bricks and blocks. The behaviour of the brick-mortar and brick-brick interfaces determines the global nonlinear response of masonry, and it is modelled by the refined constitutive formulation presented in [23], which combines plasticity and damage theories. A simplified multi-surface Coulomb yield criterion (Fig. 2b) in the stress domain governs the development of permanent plastic strains. The conical surface simulates the behaviour in shear, corresponding to Mode II failure (sliding mode, due to shear stress acting parallel to the plane of the crack and perpendicular to the crack front, represented by surface F2 in Fig. 2b). This surface is governed by cohesion c and friction angle φ , and is capped by two planar surfaces representing Mode I failure (opening mode, due to tensile stress acting normal to the plane of the crack; surface F1) and crushing failure (surface F3). Stiffness and strength degradation are determined by the evolution of an anisotropic damage tensor as a function of the plastic work produced. The compressive strength for the nonlinear interfaces representing mortar joints is assumed to be equal to the composite masonry compressive strength, as suggested by Lourenço and Gaetani [38]. The post-peak and cyclic response can be affected by other properties more difficult to quantify, i.e. fracture energies related to tensile, shear and crushing failure modes and a stiffness degradation parameter, which is defined as the ratio between the normal strain at which full damage in Mode I is attained under pure tension and the plastic strain at unloading [23]. These primary properties with typical values for clay brick masonry are listed in Table 1. The 3D mesoscale description allows for the representation of any regular masonry bonding pattern, defined by the in-plane and through-thickness brick arrangement; it also enables

modelling non-planar brick/block masonry components, such as arches, vaults, and domes. The main drawback of the mesoscale strategy is the high computational burden, which usually limits its use to small structures. One way to mitigate this issue is through domain partitioning, as developed in [39,40]. The model is divided into smaller subdomains (called child partitions), which are analysed in parallel by different processors. The so-called parent structure is composed of dual super-elements identified by the boundary nodes of the child partitions. It controls the two-way communication of the sub-domains, allowing the parallelisation of the analysis. Increased computational efficiency can be achieved by considering multiple levels of hierarchical partitioning.

2.2. FE continuum macroscale approach

According to the adopted FE continuum macroscale modelling, masonry material nonlinearity is represented by the CDP model [12,50], an isotropic model that combines the concepts of damage with tensile and compressive plasticity. A scalar isotropic damage, which comprises separate damage variables in tension (d_t) and compression (d_c), is assumed. The nonlinear response is defined through a uniaxial nonlinear constitutive law, with softening in tension and compression as shown in Fig. 3a, where the stiffness recovery parameters w_c and w_t for compressive and tensile behaviour, varying from 0 to 1 for zero and full stiffness recovery, are also indicated. The model uses the yield surface proposed by Lubliner et al. [50], with the modifications put forward by Lee et al. [12], to account for different evolutions of strength under tension and compression. The typical form of the yield function in the 2D space of the principal stresses is sketched in Fig. 3b. Notably, the CDP model was initially developed to model the behaviour of concrete structures and isotropic quasi-brittle materials. However, it has also been widely adopted in modelling masonry structures [6,14,17,51,52].

In the present work, the CDP model developed in [17] is considered, which is a slightly modified version of the original model proposed by [12,50]. The main difference between the two model implementations is the definition of the damage parameters d_c and d_t . In Lubliner's model, the parameters are directly defined by the user as a function of the inelastic deformation, while in Chisari's model, they are described as functions of the nominal and effective strengths, which are, in turn, functions of the state variables related to plastic strain.

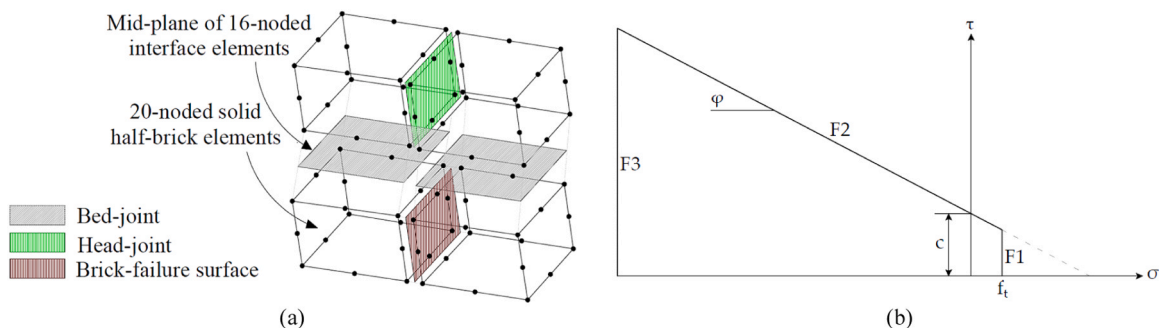


Fig. 2. (a) Mesoscale modelling of brick and block masonry; (b) multi-surface yield criterion of the plastic problem [23].

Table 1

Typical material properties for a mesoscale clay-brick and calcium-silicate block masonry model (subscripts: b: brick; j: mortar joint; n: normal direction; t: tangential direction).

Parameter			Symbol	Unit	Typical values	Source
Units elastic parameters	Elastic modulus		E _b	N/mm ²	4000–18000 (clay bricks); 7100–15000 (calcium silicate blocks)	NZSEE [41] Grabowski [42]
	Poisson's coefficient		ν _b	-	0.2–0.35	NZSEE [41]
Interfaces elastic parameters	Mortar joints	Normal stiffness	K _{n,j}	N/mm ³	10–250	Simplified estimations (Young's modulus divided by 1 cm of mortar thickness), corresponding to mortar Young's modulus values between 100 and 2500 MPa
		Tangential stiffness	K _{t,j}	N/mm ³	5–105	
	Internal bricks	Normal stiffness	K _{n,b}	N/mm ³	10 ⁴	High values simulating infinite stiffness
		Tangential stiffness	K _{t,b}	N/mm ³	10 ⁴	
Interfaces nonlinear parameters	Mortar joints	Cohesion	c _j	N/mm ²	0.13–0.38	EN 1996–1–1, 1995 [43]
		Friction coefficient	tanφ _j	-	0.3–0.8	NZSEE [44]
		Tensile bond strength	f _{t,j}	N/mm ²	0–0.1	Lourenço and Gaetani [38]
	Internal bricks	Tensile strength	f _{t,b}	N/mm ²	1.7–4.2 (clay bricks); 1.0–2.0 (calcium silicate blocks)	NZSEE [44] Lourenço and Gaetani [38]
		Compressive strength		f _c	N/mm ²	1.4–17.2 (solid and hollow artificial units masonry)
Interfaces fracture energies	Mortar joints	Mode I	G _{I,j}	N/mm	0.005–0.035	Van der Pluijm [46]
		Mode II	G _{II,j}	N/mm	0.01–0.25	Van der Pluijm [47]
	Internal bricks	Mode I	G _{I,b}	N/mm	(0.018–0.040)·f _{t,b}	Lourenço [48]
Interfaces stiffness degradation	Crushing mode		G _c	N/mm	(0.88–5.3)·f _c	Jafari et al. [49]
	Mortar joints		μ _j	-	0.1	Minga et al. [23]
	Internal bricks		μ _b	-	0.1	Minga et al. [23]

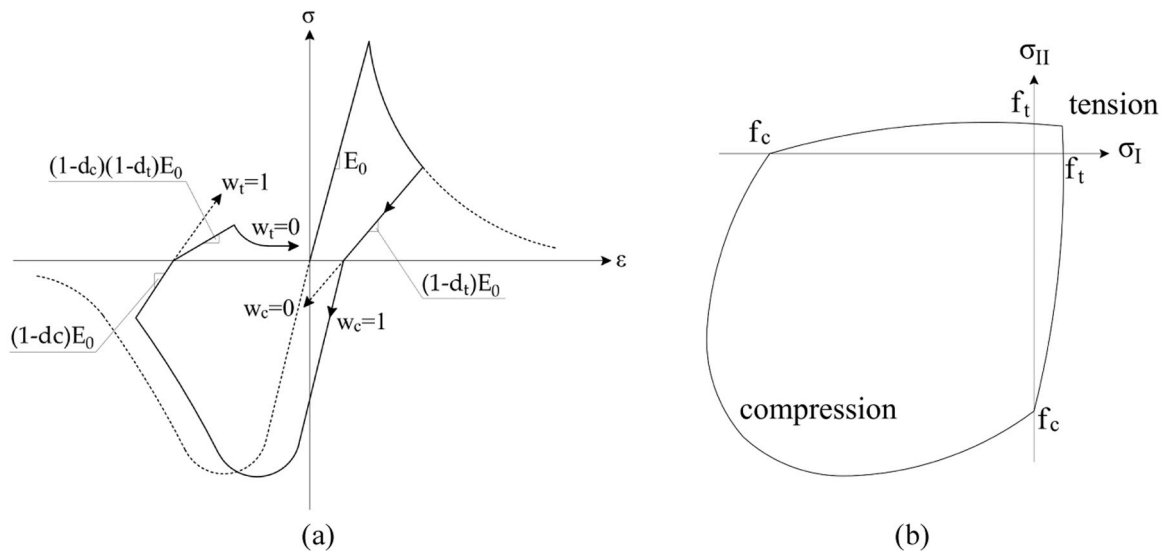


Fig. 3. (a) Uniaxial load cycle (tension-compression-tension) for damaged plasticity assuming total stiffness recovery for crack closure ($w_c=1$) and null recovery for crack reopening after crushing ($w_t=0$); (b) Elastic domain in the two-dimensional space of the principal stresses.

In general, the CDP material model must be defined through many parameters (Table 2, with typical values provided for brick and block masonry walls), characterising:

- the elastic behaviour: Young's modulus E , Poisson's coefficient ν ;
- the uniaxial post-elastic behaviour in tension: uniaxial tensile strength f_t , fracture energy G_t , stiffness degradation in tension parameter μ ;
- the uniaxial post-elastic behaviour in compression: uniaxial compressive strengths f_c , ratio between uniaxial yielding stress and maximum strength f_{yr} , plastic strain at the peak compression strength $k_{c,max}$;
- the multi-axial cyclic behaviour: dilatancy ψ , ratio between biaxial and uniaxial compressive strength f_{b0} , flow potential eccentricity e , the ratio of the second stress invariant on the tensile meridian to that

Table 2

Typical material properties for a continuum macroscale modelling approach for clay brick and calcium-silicate block masonry walls employing Concrete Damage Plasticity material model.

Parameter	Symbol	Unit	Typical values	Source
Elastic modulus	E	N/mm ²	1200–5600	MIT [13]
Poisson's coefficient	ν	-	0.2	Lourenço and Gaetani [38]
Dilatancy angle	ψ	°	0–35	Van der Pluijm [53]
Ratio between biaxial and uniaxial compressive strength	f_{b0}	-	1.16	Hoveidae et al. [54]
Flow potential eccentricity	ε	-	0.1	Hoveidae et al. [54]
Ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at an initial yield	K_c	-	0.66	Hoveidae et al. [54]
Tensile strength	f_t	N/mm ²	0.075–0.255	MIT [13]
Fracture energy (assumed as for Mode I failure)	G_t	N/mm	0.005–0.035	Van der Pluijm [46]
Parameter controlling stiffness degradation in tension	μ	-	0–1	Chisari et al. [17]
Maximum compression strength	f_c	N/mm ²	1.4–17.2 (solid and hollow artificial units masonry)	MIT [45]
Ratio between uniaxial yielding stress and maximum strength in compression	f_{yr}	-	0.01–1	Chisari et al. [17]
Plastic strain at maximum compressive strength	$k_{c,max}$	-	$1 \cdot 10^{-5}$ – $1 \cdot 10^{-2}$	Chisari et al. [17]
Tensile stiffness recovery	w_t	-	0	Lee and Fenves [12]
Compressive stiffness recovery	w_c	-	1	Lee and Fenves [12]

on the compressive meridian at initial yield K_c , stiffness recovery parameters in tension w_t and compression w_c .

The calibration of the material parameters for the macroscale model is not straightforward when only the properties of the masonry constituents are available. In addition, some parameters that may significantly affect the response cannot be explicitly and objectively quantified.

3. Analyses on brick-masonry wall structures

3.1. Experimental tests

Mesoscale and macroscale analyses have been performed to simulate the response of brick-masonry walls previously tested under in-plane horizontal loading at the Joint Research Centre in Ispra [55]. The two analysed panels, built with alternate courses of stretchers and headers (English bond), have the same cross-section of $1 \times 0.25 \text{ m}^2$ but different heights: 1.35 m for the *Short Panel* and 2.0 m for the *Tall Panel*. A detailed characterisation of the masonry material properties of the two specimens is given in [56], and the main properties of mortar, bricks and brick-mortar joints are provided in Table 3.

Table 3

Material properties of the brick-masonry wall specimens [56].

Property	Source	Mean value	
Brick tensile strength	Bending test	2.44	MPa
Brick compression strength	Simple compression test	26.9	MPa
Mortar compression strength	Simple compression test	3.31	MPa
Brick secant elastic modulus	Simple compression test	2515	MPa
Mortar tensile strength	Bending test	0.58	MPa
Joint tensile strength	Cross-tension test	0.04	MPa
Joint cohesion	Triplet shear test	0.23	MPa
Joint coeff. of friction	Triplet shear test	0.58	
Masonry compression strength	Compression test on wallets	6.2	MPa
Masonry secant elastic modulus	Compression test on wallets	1491	
Brick nominal dimensions		$55 \times 120 \times 250$	mm
Bed joint nominal thickness		10	mm
Mortar composition		Hydraulic lime/sand = 1/3 in volume	

In the physical tests, the masonry wall specimens were subjected to an initial vertical load of 150 kN, which was uniformly distributed at the top of the walls by a stiff spreader beam, resulting in a vertical compression stress of 0.6 MPa. Subsequently, horizontal forces were applied under displacement control at the top beam, which was forced to remain horizontal during the test.

The two walls exhibited distinct failure modes and cracking patterns, as reported by Anthoine et al. [55] and schematically depicted in Fig. 4. The short specimen showed predominant diagonal shear cracking and horizontal joint openings at the top and bottom of the wall and a cyclic behaviour with notable strength and stiffness degradation (Fig. 4a). In contrast, the tall wall specimen exhibited horizontal cracking due to flexural bending and rocking with typical flexural S-shaped cycles with no significant strength degradation (Fig. 4b).

Further numerical simulations have been conducted on a two-storey perforated wall. The façade wall, named Wall D, is 250 mm thick and has a regular distribution of openings. It was part of a full-scale two-storey URM building specimen which was experimentally tested by Magenes et al. [57]. The structure was built using the same masonry materials as the two wall specimens introduced before (Table 3). In the test, Wall D was disconnected from the rest of the building. The floors consisted of isolated steel beams designed to simulate a flexible diaphragm. Loads were applied through these beams, with additional gravity loads simulated using concrete blocks. Overall, uniform vertical loads of 248.4 kN and 236.8 kN were applied at the first and second-floor levels, respectively, resulting in a precompression of 0.5–0.6 MPa at the base of the piers. The structure was then subjected to in-plane cyclic loading under displacement control with equal horizontal forces applied at the two floor levels. The test adopted a pseudo-dynamic procedure, imposing quasi-static displacements at the floor levels. The horizontal forces were transmitted through the floor beams using displacement-controlled screw jacks anchored to a steel reaction frame.

Fig. 5a depicts the evolution of damage due to increasing drift levels. Cracking first developed in the spandrel walls and at the bottom of the piers. Subsequently, it extended to the top of the ground-floor piers and to the two ends of the lateral first-floor piers. For higher drift levels, significant damage in the spandrels and shear diagonal cracking in the ground-floor piers were observed, which correlates with the widening of the base shear - second-floor horizontal displacement cycle (Fig. 5b).

3.2. Mesoscale simulations

This section presents the results of nonlinear simulations on the two solid masonry panels and the two-storey façade wall using the refined mesoscale approach developed in [16]. The elastic material parameters

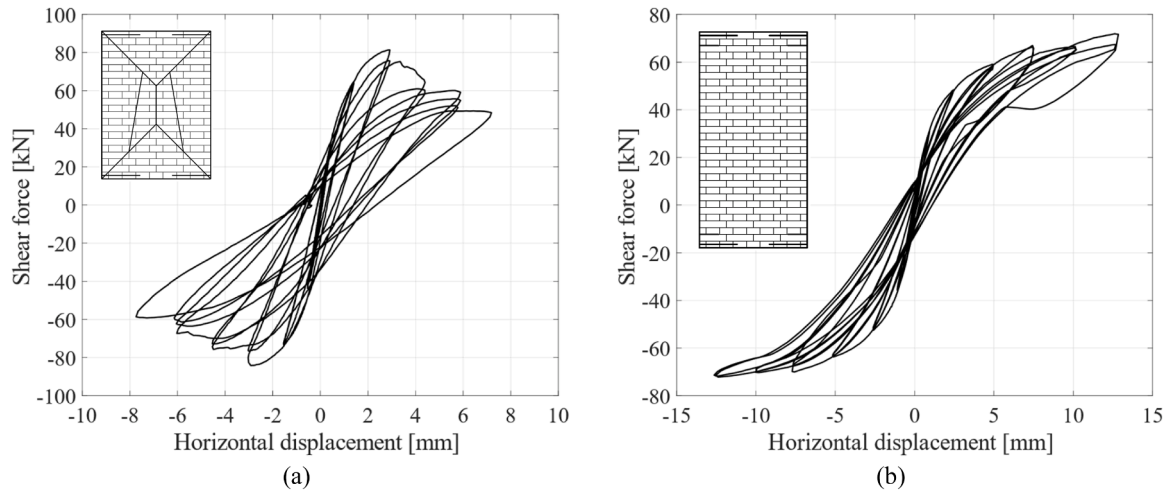


Fig. 4. Experimental cracking and base shear vs lateral displacement curves of the two brick-masonry wall specimens tested in [55]: (a) short panel; (b) tall panel.

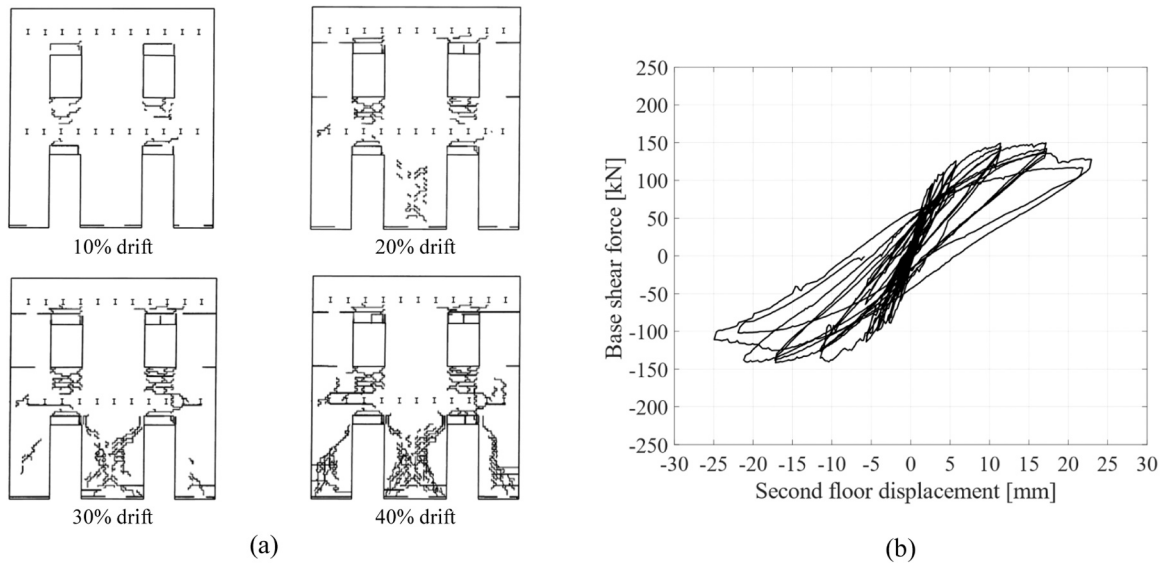


Fig. 5. Wall D experimental (a) cracking and (b) base shear vs. 2nd floor lateral displacement cycles [57].

for both interface and solid elements, along with the strength material properties for the nonlinear interfaces have been determined from material tests on masonry constituents and small components [56]. The influence of additional model material parameters, such as Mode I and II fracture energies and reduced cohesion in mortar head joints due to shrinkage (which cannot be directly obtained from material tests), has been explored in a parametric study considering the sets of material properties in Table 4. On the other hand, the large perforated façade wall has been modelled utilising a fixed set of material properties, considering the results of the parametric study on the two solid walls.

3.3. Mesoscale material characterisation

The normal and tangential stiffnesses of the joint interfaces, denoted as $K_{n,j}$ and $K_{t,j}$, have been calculated following the strategy in [38], as functions of the elastic properties of the units and the masonry material at the macroscale, using the Eqs. (1) and (2):

$$K_{n,j} = \frac{E_b E_M}{(h_b + h_m)(E_b - E_M)} \quad (1)$$

$$K_{t,j} = \frac{G_b G_M}{(h_b + h_m)(G_b - G_M)} \quad (2)$$

where E_b , G_b , and E_M , G_M , represent the normal and tangential elastic moduli for brick and masonry, h_b denotes the height of the brick and h_m is the thickness of the mortar joints; the subscript “b” refers to brick, “m” to mortar, “M” to masonry, “j” to mortar joint, “n” to normal direction, “t” to tangential direction. Assuming typical Poisson’s ratios (ν) for brick and mortar and a linear elastic isotropic behaviour, the tangential stiffness can be directly derived from the normal stiffness through (3).

$$K_{t,j} = \frac{K_{n,j}}{2(1 + \nu)} \quad (3)$$

The Young’s modulus of the masonry material E_M and the brick units E_b are assumed as the initial tangent values derived from the experimental stress-strain curves reported in [56] and are equal to 1980 MPa and 4000 MPa, respectively. Additionally, a Poisson’s coefficient of 0.2 is considered for both bricks and masonry. The elastic stiffnesses values for the joint interfaces calculated using Eqs. (1) and (3) are reported in Table 4. On the other hand, high normal and tangential stiffnesses of 10^4 N/mm³ are adopted for the brick-brick interfaces. The other mechanical

Table 4

Material characteristics for reference and variant brick-masonry mesoscale models; the variable parameters are reported in shaded cells (hj: head-joints).

Parameter			Symbol	Unit	Reference	Var. 1	Var. 2	Var. 3	Var. 4
Units elastic parameters	Elastic modulus		E_b	N/mm ²	4000				
	Poisson's coefficient		ν_b	-	0.2				
Interfaces elastic parameters	Mortar joints	Normal stiffness	$K_{n,j}$	N/mm ³	60.3				
		Tangential stiffness	$K_{t,j}$	N/mm ³	25.1				
	Internal bricks	Normal stiffness	$K_{n,b}$	N/mm ³	10 ⁴				
		Tangential stiffness	$K_{t,b}$	N/mm ³	10 ⁴				
Interfaces mechanical parameters	Mortar joints	Cohesion	c_j	N/mm ²	0.23		0.023 (only hj)		
		Friction coefficient	$\tan\phi_j$	-	0.58				
		Tensile strength	$f_{t,j}$	N/mm ²	0.04		0.004 (only hj)		
	Internal bricks	Tensile strength	$f_{t,b}$	N/mm ²	1.22				
		Compressive strength							
Interfaces fracture energies	Mortar joints	Horizontal interfaces	$f_{c,h}$	N/mm ²	6.2				
		Vertical interfaces	$f_{c,v}$	N/mm ²	3.1				
	Internal bricks	Mode I	$G_{I,j}$	N/mm	0.016		0.005	0.035	
		Mode II	$G_{II,j}$	N/mm	0.16		0.014	0.25	
	Crushing mode	Mode I	$G_{I,b}$	N/mm	0.12				
Interfaces stiffness degradation	Mortar joints	Horizontal interfaces	$G_{c,h}$	N/mm	17.5				
		Vertical interfaces	$G_{c,v}$	N/mm	16.3				
	Internal bricks								

parameters for the nonlinear interfaces representing mortar joints, including tensile strength ($f_{t,j}$), cohesion (c_j) and coefficient of friction ($\tan\phi_j$), which were obtained from material tests, are reported in Table 3. To determine the brick tensile strength ($f_{t,b}$), a corrective factor of 0.5 has been applied to the experimental value. This correction is suggested in [58] to account for the indirect evaluation of brick tensile strength through bending tests. Finally, the compression strength for the interface elements corresponds to the experimental compressive strength for the masonry at the macroscale, where different values are assumed for horizontal ($f_{c,h}$) and vertical ($f_{c,v}$) interfaces. For the horizontal interfaces, the compression strength is derived from standard tests on masonry wallets under vertical compression, while a 50 % reduction from the vertical compressive strength is assumed for the vertical interfaces to account for the orthotropic characteristics of the masonry material. Mortar joint modes I, II fracture energies, $G_{I,j}$ and $G_{II,j}$, and masonry crushing energy $G_{c,j}$ were not provided by the material tests in [56], thus they represent uncertain properties. In this research, the values obtained by Van der Pluijm [46,47,53] and recommended in [38], have been used in the parametric investigation considering a reference set of mesoscale model parameters and four variations, as indicated in Table 4). Interface stiffness degradation parameters (μ_j , μ_b), which represent the ratio between the plastic strain at a given unloading point and the normal strain at which full damage in mode I is reached under pure tension [23], control the damage in tension. These parameters are set to 0.1 for both mortar joints and internal brick joints. A higher value of 0.5, leading to reduced stiffness degradation, is also considered for the mortar joints in variant 4.

3.4. Mesoscale numerical results

The mesoscale mesh for the two panels (Fig. 6) has been created

using Gmsh [59] following the strategy described in [35]. The English bond pattern of the two walls, characterised by the alternating arrangement of header and stretcher courses, is represented by a distinct sequence of nonlinear interfaces for mortar head joints and brick-to-brick fracture planes. The two masonry panels are fully restrained at the bottom. A uniform compressive pressure of 0.6 MPa is applied at the top by an elastic stiff component, representing a spreader beam supported to remain horizontal during the test under prescribed in-plane horizontal displacements. The short panel model features 22080 degrees of freedom (DoF), while the tall panel model consists of 31680 DoF. Nonlinear dynamic simulations have been performed by applying a time history of accelerations, obtained by double differentiation and spline fitting of the experimental prescribed displacements, to the nodes of the solid elements of the top spreader beam. Rayleigh damping is used with a high mass-proportional coefficient (set to 50) to minimise inertial effects in the slow cyclic test and enhance numerical stability. Time integration is performed using the Hilber-Hughes-Taylor scheme ($\alpha = -0.05 \text{ s}^{-1}$) to introduce numerical dissipation. This approach is consistently applied throughout all analyses reported in this article.

Figs. 7 and 8 depict the results of the reference models under monotonic and cyclic top displacements. These include shear force vs. horizontal displacement curves, tensile damage on the interface elements, and deformed shapes.

In the case of the short panel subjected to monotonic loading (Fig. 7), the damage pattern aligns well with the experimental observations. The experimental initial elastic stiffness and the peak strength are predicted with high accuracy. However, the simulated post-peak behaviour does not feature the strength softening observed in the experimental cycles. The model exhibits the typical X-shaped damage from shear diagonal cracking when subjected to cyclic actions. The numerical cyclic curve closely approximates the experimental cycle, with stiffness degradation

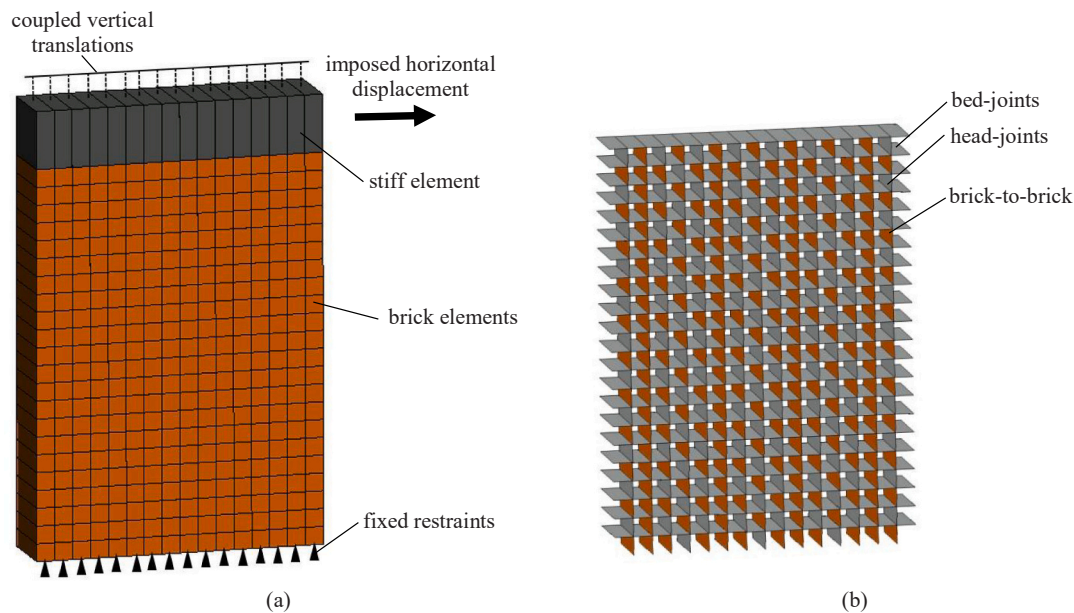


Fig. 6. Mesoscale representation of a panel subjected to in-plane loading: (a) brick elements, spreader beam, and boundary conditions; (b) mortar and brick-to-brick nonlinear interface elements.

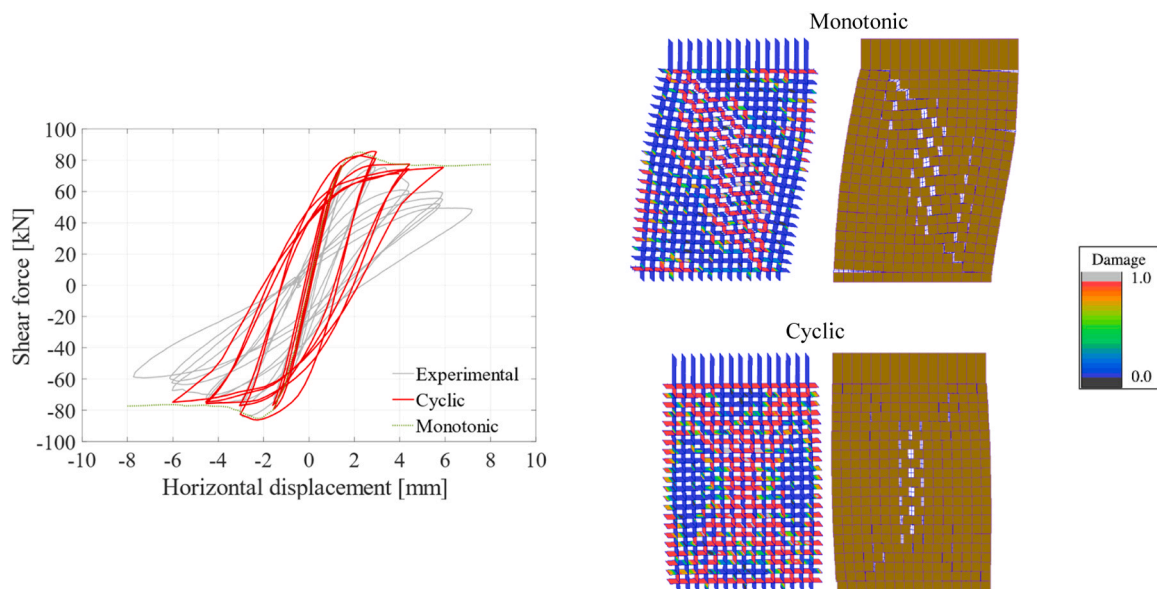


Fig. 7. Mesoscale short panel model under in-plane loading: experimental-numerical shear force vs. horizontal displacement comparison, tensile damage developed in the interfaces and deformed shape at the last step reached in the monotonic and cyclic analysis.

reasonably replicated. However, like the monotonic curve, the post-peak behaviour does not exhibit the marked strength softening observed in the experimental cycle.

In the case of the tall panel (Fig. 8) subjected to monotonic loading, the typical damage associated with in-plane rocking behaviour, characterised by cracking zones at the wall bottom and top, is observed. The monotonic shear-displacement curve closely fits the envelope of the experimental cycle. Regarding the cyclic response, the typical S-shaped cycle, which is typical of flexural damage in masonry elements, is noticeable. However, after reaching a certain displacement, the numerical model experiences a loss of symmetry and begins to shift towards one side due to accumulated shear plastic strain at the two extremities of the model. The observed experimental degradation in stiffness is not replicated. One reason for this discrepancy could be

attributed to the assumption of simplified elastic unloading-reloading in the constitutive model proposed by Minga et al. [23]. It should also be noted that during the experimental test, damage within the panel was observed for large displacements, likely leading to the widening of the force-displacement cycle.

Monotonic and cyclic pushover analyses have been subsequently conducted varying the material properties as in Table 4. Remarkably, a low variability in response predictions is observed considering capacity curves, cyclic diagrams, failure modes, and dissipated energy. The experimental-numerical comparisons in terms of shear vs. displacement curves are reported in Fig. 9. In the case of the short panel, the analysis of variant 1 (weak head joint) terminates earlier, even though its response aligns with the reference characterisation. Employing higher but realistic fracture energies (Variant 3) may facilitate the convergence

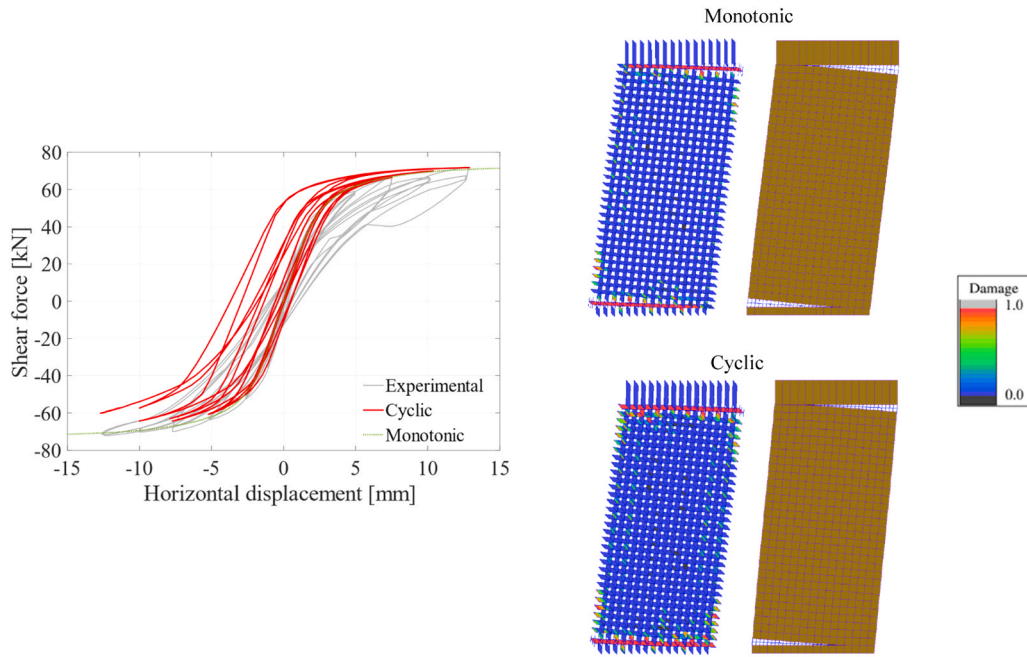


Fig. 8. Mesoscale tall panel model under in-plane loading: experimental-numerical shear force vs. horizontal displacement, tensile damage developed at the interfaces and deformed shape at the last step of the analyses under the monotonic and cyclic loading.

of the analysis. As for the tall panel, Variants 1 (weak head-joint) and 2 (low fracture energies) exhibit, at approximately 10 mm displacement, a change in the slope of the unloading stiffness, leading to an earlier termination of the analysis. This change is attributed to the spreading of shear-tensile damage on the interfaces within the panel, which is observed for large displacements in the experimental test.

Fig. 10 shows the evolution of total energy throughout the analysis for each model, highlighting the close agreement between the reference and variant models. For the short panel (Fig. 10a), a slight difference between the experimental and numerical outcomes is observed, with the numerical energies appearing overestimated. In contrast, better experimental-numerical accordance is observed for the tall panel (Fig. 10b).

Table 5 summarises the main outcomes of the cyclic analysis. It reports the failure modes predicted by the numerical models—which are always correctly captured—along with the total energy at the last step of the numerical simulations (W_{tot}). The robustness of the models is measured by the percentage of completed analyses (due to premature failures, the analyses have not been completed), with relatively high average values of 70.6 % for the low panel and 86.0 % for the tall panel. Furthermore, the variability of the responses between the variant and reference configurations is evaluated through the percentage variation in total energy (ΔW_{tot}) calculated according to Eq. (4), considering the minimum last step reached (t_i) between the reference and variant analyses. The results show a very low sensitivity of the responses to parameter variations for the short panel (maximum variation of -7.9 %), and a higher variability for the tall panel (maximum variation of $+38.7$ %).

$$\Delta W_{tot} = \frac{W_{tot,variant}(t_i) - W_{tot,reference}(t_i)}{W_{tot,reference}(t_i)} \quad (4)$$

Finally, the façade with openings is examined under horizontal cyclic actions. Considering the low variability of the responses from the models of the single panels, the material characterisation with high fracture energy (Variant 3 of Table 4) is employed to facilitate the converge of the nonlinear simulation. The vertical actions, denoted as w_1 and w_2 , are uniformly distributed at the two levels. The model is fully fixed at the base. For the horizontal action, the average of the measured

experimental displacements at the two floors, denoted as U , is applied to a master node located between the two floors, as represented in Fig. 11. Subsequently, through a spreader element, the corresponding total force (denoted as F in Fig. 13) is equally distributed to all the nodes at the two floors (n_{tot} indicates the total number of nodes), ensuring the same force magnitude is applied at the two floor levels, as in the experimental test [57]. With the aim of evaluating the influence of construction details on the reliability of the simulation, two different models are analysed: one without lintels and one with lintels above each opening represented by elastic solid elements. To improve computational efficiency, the large mesoscale model, comprising 156035 nodes and 471450 DoF, is hierarchically partitioned into 77 parts, following the strategy proposed by Joghio and Izzuddin [40]. The analyses were performed on the high-performance computing system at Imperial College London, each requiring approximately 170 h of computational time.

For the model without lintels, the second-floor displacement vs. base shear cycle and the experimental response are shown in Fig. 12a. A good prediction of the initial stiffness is observed, while the maximum strength appears to be slightly overestimated, within a 10 % variation. However, there is significant strength degradation in the first cycles, attributed to the excessively rapid failure of the masonry spandrels (Fig. 12b and c). The rapid collapse of the spandrels on the first floor leads to a cycle with pronounced pinching caused by the rocking of the piers on both sides. This prediction is consistent with the reliability of mesoscale modelling. Since it is a detailed model, when modelling large structures, it is essential to appropriately consider construction details, such as the contribution of lintels and the connections between walls and floors. In contrast, the inclusion of lintels results in hysteretic capacity (Fig. 12d) that is significantly greater and closer to the experimental response. Overall, the damage prediction (Fig. 12e and f) resembles the experimental observations, with shear and flexural cracking observed in the spandrels levels and the flexural damage at the bottom of the piers; however, diagonal cracking in the piers at the first level is only partially reproduced.

3.5. Macroscale simulations

This section investigates the application of the continuum modelling

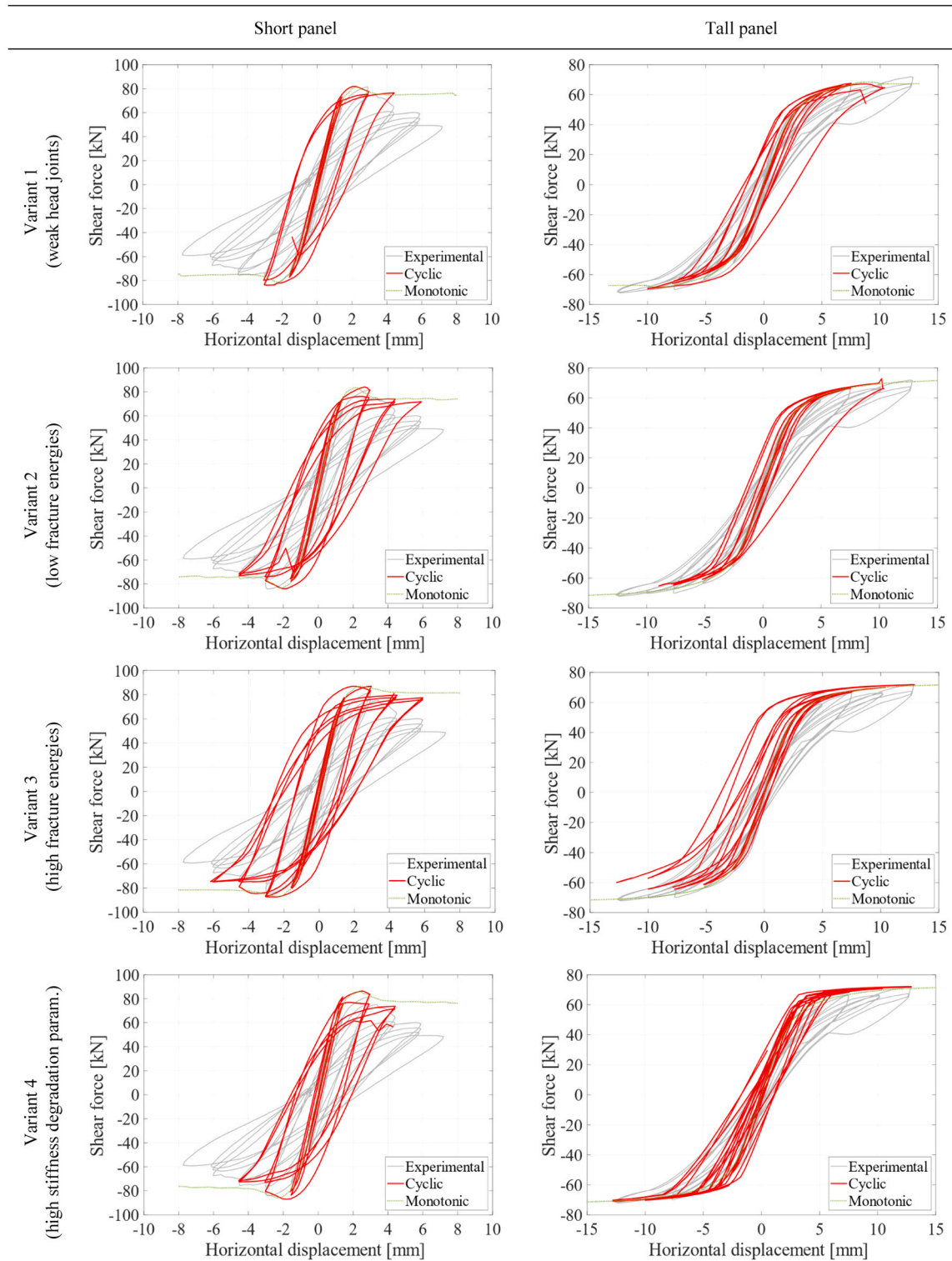


Fig. 9. Monotonic and cyclic horizontal displacement vs. shear force curves for each mesoscale variant.

approach with the CDP model representing masonry as a nonlinear isotropic material at the macroscale. The material model requires defining numerous parameters, which may not directly correlate with available experimental material properties. An extensive series of analyses on the two experimental panels under in-plane cyclic loading has been conducted to thoroughly assess the model sensitivity to the arbitrary definition of some key model parameters. This involves considering a set of reference values and 14 additional variants, each derived

by modifying several model parameters as indicated in Table 6. Furthermore, Wall D has been analysed employing a more refined material characterisation based on the results from the parametric study on the single panels.

3.6. Macroscale material characterisation

The masonry Young's modulus and compressive strength obtained

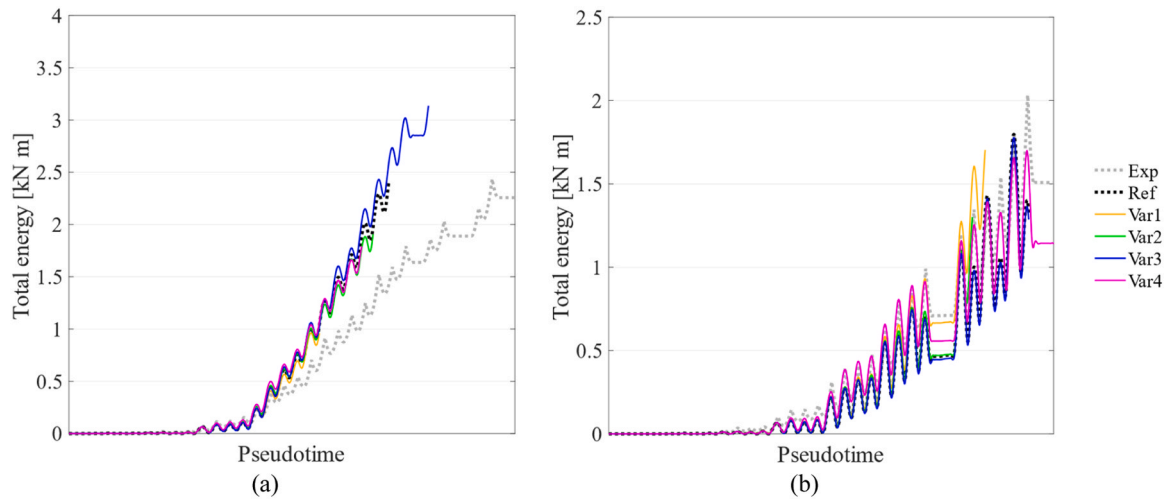


Fig. 10. Progression of total energies in each mesoscale analysis for (a) short panel and (b) tall panel.

Table 5

Summary of the results from sensitivity cyclic analyses on mesoscale models (S: shear-diagonal cracking; F: flexural).

Changing parameter	Case study	Short panel				Tall panel			
		Failure mode	W_{tot} [kN m]	% analysis completed	ΔW_{tot}	Failure mode	W_{tot} [kN m]	% analysis completed	ΔW_{tot}
Weak head joints low fracture energies high fracture energies high stiffness degradation param.	Experimental	S	2.26			F	1.78		
	Reference	S	2.40	74.0 %		F	1.32	87.7 %	
	Variant 1	S	0.96	57.9 %	−7.9 %	F	1.70	78.7 %	+ 28.4 %
	Variant 2	S	1.91	70.3 %	−3.3 %	F	1.30	76.0 %	+ 38.7 %
	Variant 3	S	3.14	83.1 %	+ 7.7 %	F	1.29	87.7 %	−2.8 %
	Variant 4	S	1.79	67.7 %	−5.5 %	F	1.25	100.0 %	+ 21.0 %

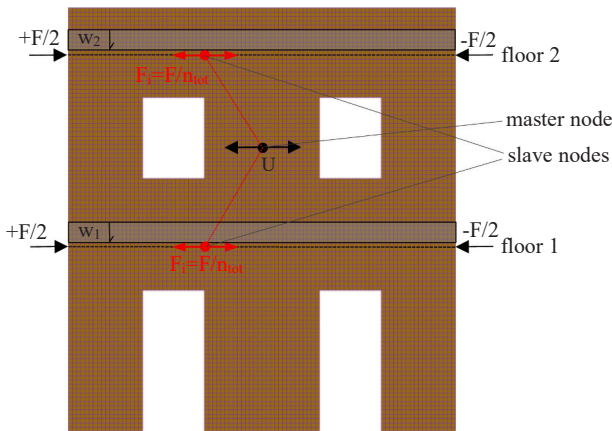


Fig. 11. Loading application and boundary conditions of the Wall D model.

from material compression tests on masonry wallets [56] are directly used for the macroscale masonry models, along with a Poisson's coefficient of 0.2 as recommended in the literature [38]. On the other hand, the masonry tensile strength for analysed masonry wall structures has not been determined in physical experiments. Furthermore, a direct correlation between the available compressive strength and the unknown tensile strength cannot be easily established, as the two strength characteristics are associated with different local interactions between mortar joints and units [38]. Thus, in the numerical simulations, to assess how the overall response of the analysed wall components varies depending on the unknown tensile strength, different tensile strength values have been assumed within a realistic range, including a reference value and three additional values. The reference value has been

determined based on the mechanical parameters suggested by the Italian code [13] and standard engineering practices. Specifically, the typology of solid brick masonry with lime mortar is considered, whose mechanical parameter ranges are provided in Table 7. Since the experimental values for the elastic modulus and compressive strength exceed the range limits, the average shear tensile strength with zero vertical stress (denoted as τ_0) is adopted as the maximum value equal to 0.13 MPa. The evaluation of the tensile strength is then conducted considering the maximum tensile stress at the centre of the panel as 1.5 times the shear strength τ_0 , resulting in a masonry tensile strength of 0.2 MPa.

The minimum value of the variation range is set by considering the experimental bond strength (Table 3), which is equal to 0.04 MPa. Another value evaluated within the assessed range is determined by considering the lower limit in Table 5 for the shear strength τ_0 , which results in a tensile strength of 0.08 MPa. As for the maximum range value, the analytical relation (5) is employed, as proposed in [38]. This relation considers the typical behaviour of masonry under tension parallel to the bed joint, characterised by failure marked by a stepped crack through the head and bed joints. The relation establishes the tensile strength as a function of the joint cohesion (c_j), overlapping length (l_0) – calculated as the average of the unit half-length (l_b) and half-width (t_b) due to the alternation of header and stretcher – and unit height (h_b), resulting in a tensile strength of 0.39 MPa.

$$f_t = \frac{c_j}{h_b} \frac{l_0}{h_b} \quad (5)$$

Regarding the fracture energy in tension G_t , a reference value of 0.02 N/mm, as recommended in [38] for macroscale modelling, is adopted. The minimum and maximum fracture energy values are 0.005 and 0.035 N/mm (as reported in Table 2), respectively. These values are derived linking masonry tensile failure to the failure of mortar joints in tension, thus using limit Mode I fracture energy values for the mortar

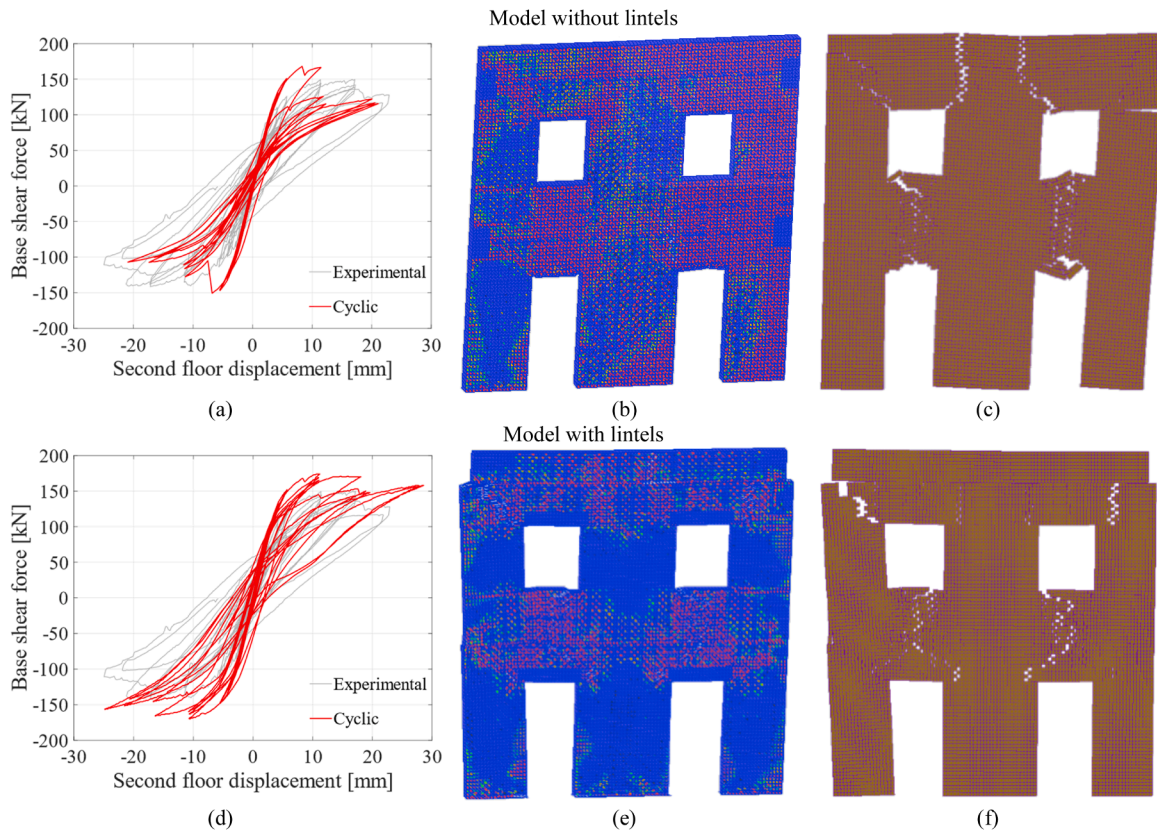


Fig. 12. Perforated wall results for the mesoscale models without and with lintels: (a, d) experimental-numerical shear-displacement curves comparison, (b, e) damage on the interfaces, (c, f) deformed shape at the end of the cyclic analysis.

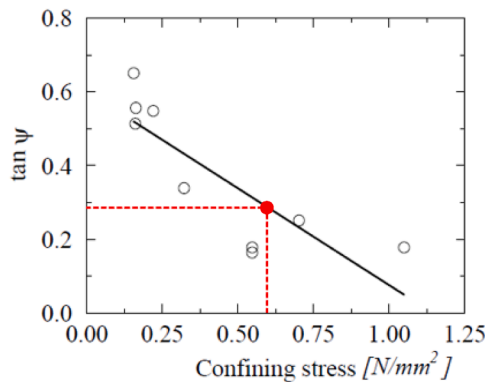


Fig. 13. Tangent of the dilatancy angle ψ as a function of the normal stress level, adapted from [53]; the marked point corresponds to the confin stress of the analysed masonry panels.

joints.

The baseline parameter governing stiffness degradation in tension, denoted as μ and defined as in the mesoscale modelling approach [17, 23], is taken as 0.2 [17]. Sensitivity analyses are conducted with assuming alternative values of 0.01 and 0.5.

The ratio between biaxial and uniaxial compressive strength $\tilde{f}_{b0}$, flow potential eccentricity ϵ , and ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield K_c are assumed as the values recommended in [11], which are equal to 1.16, 0.1, and 0.66, respectively.

The dilatancy angle, denoted as ψ , represents the tendency of certain materials to expand their volume when subjected to shear actions in its plastic range. As pointed out in [60], this phenomenon is directly linked

to the magnitude of confining stress. Specifically, lower confining stress levels are generally associated with higher material dilation. The dilatancy angle is defined at the macroscopic level so that its tangent is equivalent to the ratio between normal and shear displacement. It quantifies the uplift of one unit over the other when subjected to shear stresses. In Fig. 13 [53], the outcomes of numerous shear tests on joints in masonry with solid clay units are presented. The figure reports a linear regression, illustrating the relationship between the tangent of the dilatancy angle and the level of confining stress. Given this linear relation and the confinement level of the solid brick masonry panels under consideration (vertical stress equal to 0.6 MPa), the corresponding tangent of the dilatancy angle is approximately 0.3. For the analysis, a reference value of 20° is adopted, and the sensitivity of this parameter is subsequently examined by varying it between 10° and 35° .

Stiffness recovery parameters are crucial in simulating the cyclic response, with the distinction between compression and tension recovery arising from the different failure mechanisms involved, crushing and cracking. These parameters account for micro-crack opening and closure. Typically, the recovery is significantly higher when passing from tension to compression than vice versa. Therefore, the adopted reference values are 1 for compression stiffness recovery (w_c), indicating full recovery upon crack closure as the load changes from tension to compression, and 0 for tension stiffness recovery (w_t), denoting no recovery during the transition from compression to tension once the concrete crushing is initiated. Additionally, two further combinations are considered, with values of 0.8 and 0.2, and 0.16 and 0.87. In particular, the latter combination is derived in [17], using inverse analysis and genetic algorithms. The sensitivity assessment also involves varying the ratio between uniaxial yielding stress and maximum strength in compression (f_{yr}), ranging from a standard value of 0.3 to a minimum of 0.1 and a maximum of 0.5. Finally, the plastic strain at the compression peak ($k_{c, \text{fmax}}$), initially set at a central value of 0.002,

Table 6

Material characteristics for reference and variant brick-masonry macroscale models; the variable parameters are reported in the shaded cells.

Parameter	Symbol	U.o.m.	Reference	Variants	Case study
Young's modulus	E	N/mm ²	1980		
Poisson's coefficient	ν	-	0.2		
Dilatancy angle	ψ	°	20	10 – 35	CDP1 – CDP2
Ratio between biaxial and uniaxial compressive strength	$\widetilde{f}_{b0}$	-	1.16		
Flow potential eccentricity	ε	-	0.1		
Ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield	K_c	-	0.66		
Global tensile strength	f_t	N/mm ²	0.20	0.04 – 0.08 – 0.39	CDP3 – CDP4 – CDP5
Fracture energy	G_t	N/mm	0.02	0.005 – 0.035	CDP6 – CDP7
Stiffness degradation	μ	-	0.2	0.01 – 0.5	CDP8 – CDP9
Maximum compression strength	f_c	N/mm ²	6.2		
Ratio between uniaxial yielding stress and maximum strength in compression	f_{yr}	-	0.3	0.1 – 0.5	CDP10 – CDP11
Plastic strain at maximum compressive strength	$k_{c,max}$	-	0.002	0.001 – 0.003	CDP12 – CDP13
Tensile stiffness recovery	w_t	-	0	$w_t=0.2$ & $w_c=0.8$	CDP14
Compressive stiffness recovery	w_c	-	1	$w_t=0.87$ & $w_c=0.16$	CDP15

Table 7

Suggested material characteristics by Italian standard for solid brick masonry with lime mortar [13].

f_c [MPa]	τ_0 [MPa]	f_{v0} [MPa]	E [MPa]	G [MPa]	w [kN/m ³]
min-max 2.6–4.3	min-max 0.05–0.13	min-max 0.13–0.27	min-max 1200–1800	min-max 400–600	18

typically found in concrete-like materials, is varied from a minimum of 0.001 to a maximum of 0.003. The full set of material characterisations for the reference and variant scenarios adopted in the macroscale simulations are provided in Table 6.

3.7. Macroscale numerical results

Monotonic and cyclic pushover analyses have been conducted on the

reference model and 14 variants. The boundary and loading conditions are the same as those of the mesoscale models introduced before. The short panel comprises 2891 3-DoF nodes, while the tall panel consists of 4178 3-DoF nodes. Fig. 14 and Fig. 15 show the monotonic and cyclic predictions for the short and tall panels using the reference material characterisation. While the initial elastic stiffnesses are accurately captured, the maximum strengths are overestimated by both models. Noteworthy, the monotonic curves deviate significantly from the numerical cyclic envelopes. In the cyclic response of the short panel (Fig. 14), the initial cyclic behaviour is characterised by flexural damage, with damage concentration at the top and bottom of the panel, but with broad shear-lateral displacement hysteretic cycles. In the second phase, the spreading of cracks along the panel height leads to substantial strength and stiffness degradation, with a decrease in the rate of dissipated energy. This crack spreading does not take place under monotonic loading. The tall panel exhibits a similar overall behaviour (Fig. 15). Notably, the S-shaped cycle, which was found in the test and is typical of

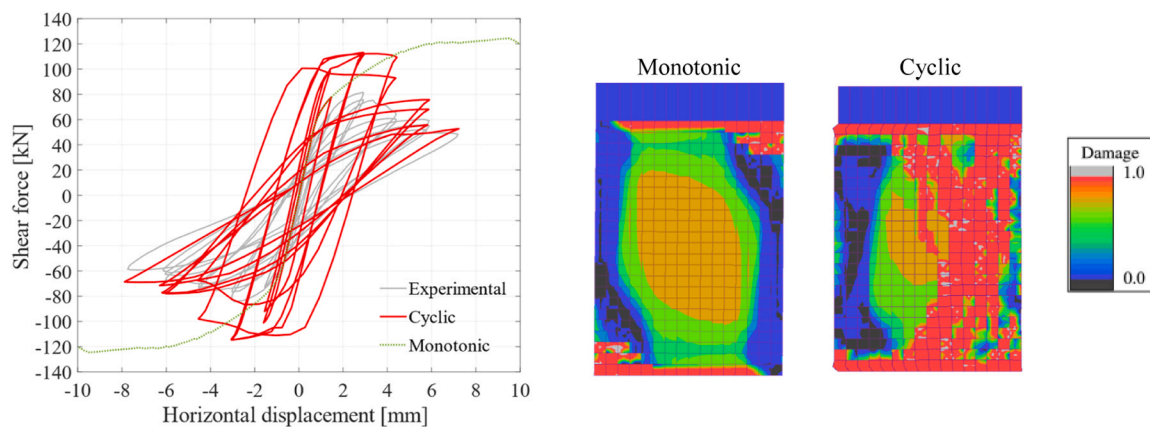


Fig. 14. Macroscale short panel model - reference characterisation: experimental-numerical comparison and tensile damage developed in the monotonic (displacement equal to 9 mm) and cyclic analysis (last step).

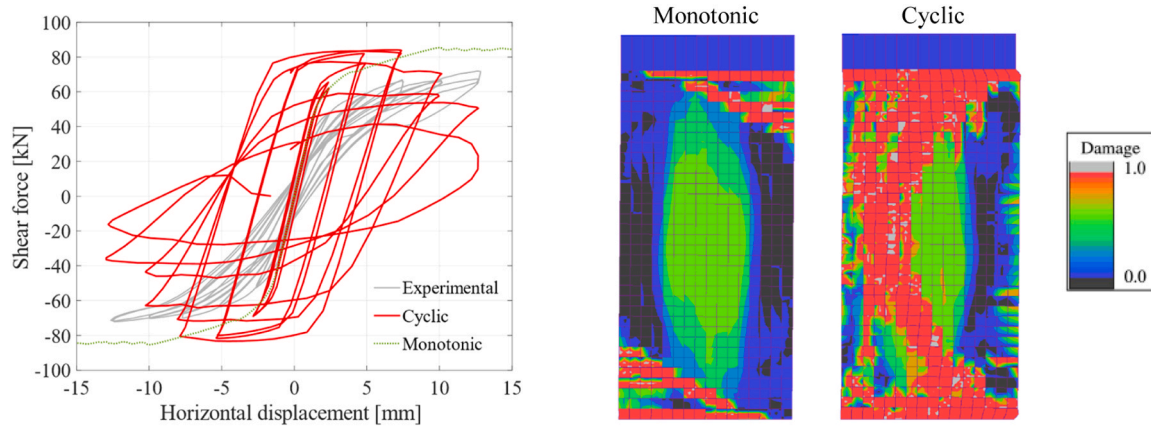


Fig. 15. Macroscale tall panel model - reference characterisation: experimental-numerical comparison and tensile damage developed in the monotonic (displacement equal to 14 mm) and cyclic analysis (last step).

a flexural response, is not reproduced. This suggests that, in the model, damage localised at the panel ends does not adequately represent the potential opening and closing of mortar joints, which is responsible for the characteristic S-shaped cyclic response. It is also clarified that the damage propagation towards the centre of the panel, which occurs only at the end of the analysis, is not the cause of the widened hysteresis cycles.

The variant models with the material characterisations of Table 6 exhibit large variability regarding capacity curves, cyclic diagrams, failure modes, and the amount of dissipated energy. Notably, significantly different numerical responses arise from varying the global tensile strength, as can be observed in Fig. 16 for the short panel and Fig. 17 for the tall panel. In the case of the short panel, for the two lower value specifications (CDP3 and CDP4 variants), a shear diagonal cracking failure mode is consistently observed throughout the cyclic analysis with no damage at the extremities. The maximum shear force from variant

CDP4 is close to the experimental responses, while it is lower in CDP3 where the adopted tensile strength is the bond strength. Moreover, the excessive strength degradation hinders accurate prediction of the total dissipated energy, thereby preventing the completion of the analyses for both models. Conversely, adopting the highest tensile strength (variant CDP5) does not lead to significant changes compared to the reference case, neither in the monotonic nor the cyclic response. For the tall panel, similar variations in responses are observed. Again, the typical flexural S-shape cycles are not reproduced in any case.

The results of all the other variant cases are depicted in Figs. 18 and 19. The main results from all the cyclic analyses are summarised in Table 8. In many cases, the predicted failure modes are not accurately captured. Compared to the mesoscale results (Table 5) the percentage of completed analyses is generally higher—often reaching 100 %—but the variability in the results is significant for both the low and tall panels. The differences in dissipation capacity between the models are also

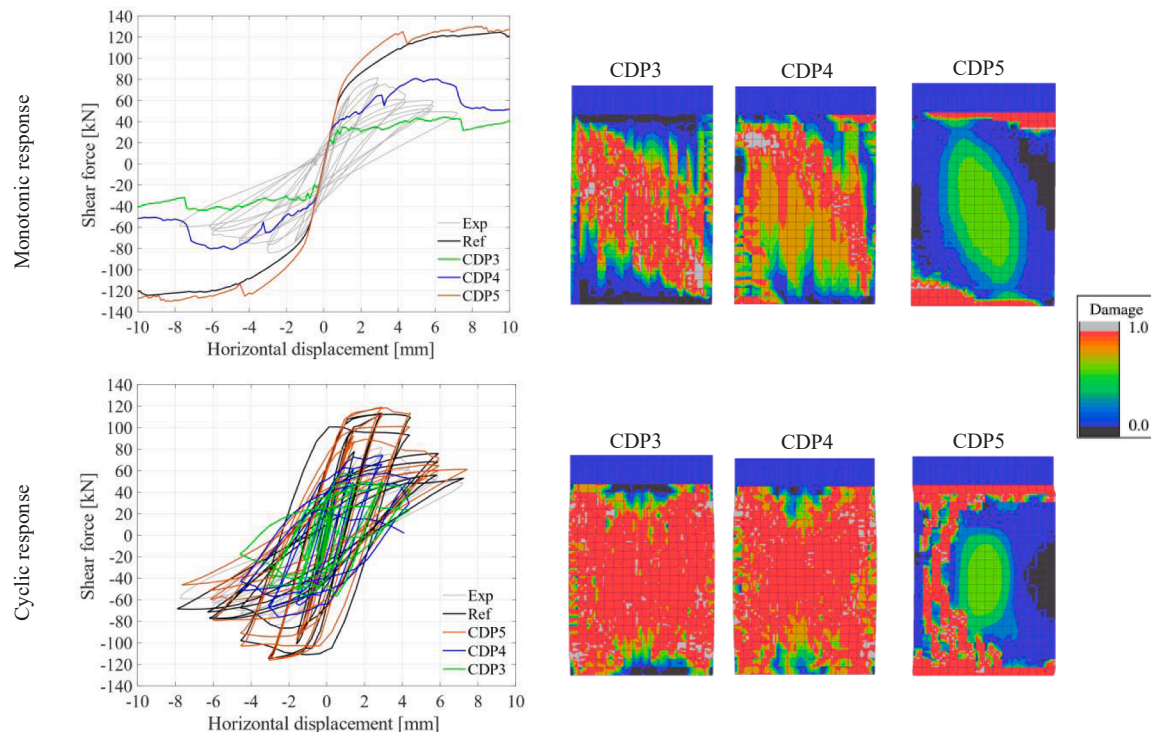


Fig. 16. Macroscale short panel model – tensile strength variations: experimental-numerical comparison and tensile damage developed in the monotonic (displacement equal to 9 mm, top) and cyclic analysis (last step, bottom).

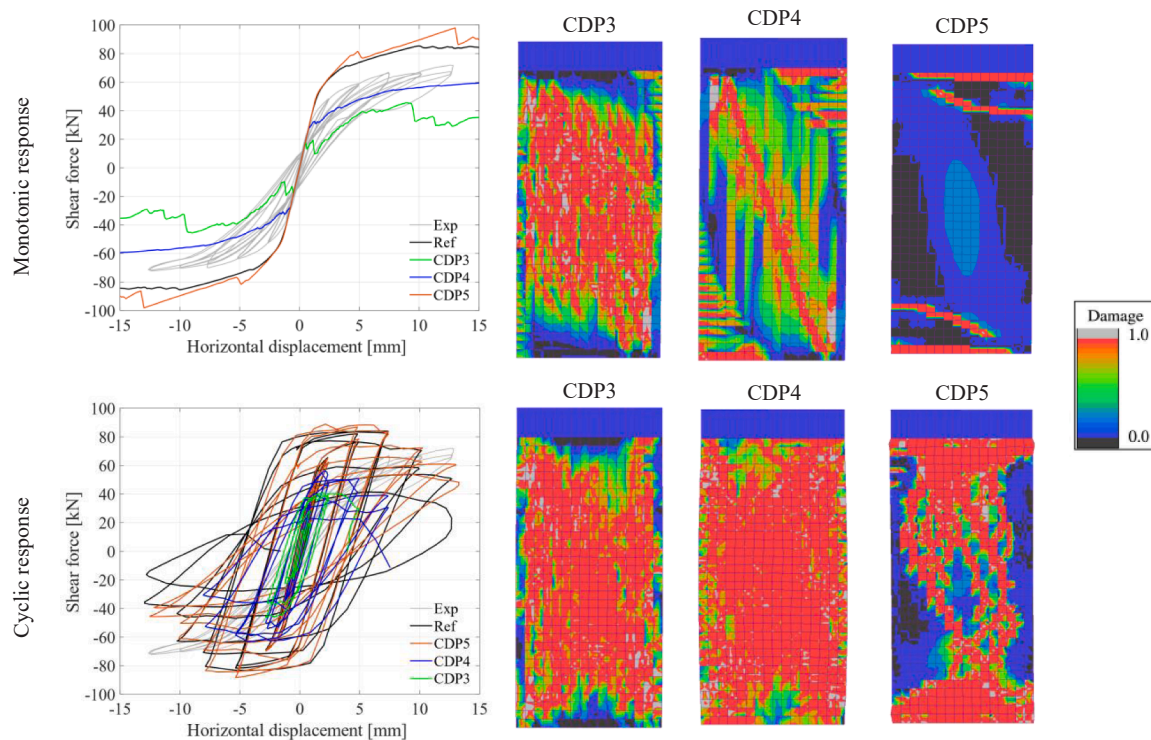


Fig. 17. Macroscale tall panel model – tensile strength variations: experimental-numerical comparison and tensile damage developed in the monotonic (displacement equal to 14 mm, top) and cyclic analysis (last step, bottom).

represented through the graphs in Fig. 20, which highlights the high sensitivity of the responses to variations in the selection of the parameters. Fig. 21 shows the evolution of total energy throughout the analysis for each model.

The difference in dissipated energies and failure modes, among experiments, reference models, and variants, is significant. Specifically, in addition to the tensile strength variation already discussed:

- Smaller variations (even if still significant) in the dissipated energy are observed when varying dilatancy ψ (CDP1 and CDP2 variants), fracture energy G_t (CDP6 and CDP7 variants), and ratio between uniaxial yielding stress and maximum strength in compression $f_{y,r}$ (CDP8 and CDP9 variants) parameters assumed within realistic ranges.
- A greater variation is observed in the cyclic responses when $k_{c,max}$ is changed from a low value of 0.001 (CDP12 variants), which leads to responses closer to the experimental curves, to the upper limit of 0.003 (CDP13 variants), despite the monotonic responses being similar.
- Both variants related to the stiffness degradation parameter μ result in lower dissipated energies compared to the reference model (μ equal to 0.2). For the upper value (μ equal to 0.5; CDP9 variants), the high degradation leads to premature termination of the analysis (only 51 % of the analysis is terminated for the short panel and 59 % for the tall panel). On the other hand, the lower value of stiffness degradation parameter (μ equal to 0.01; CDP8 variants) results in narrower cycles.
- Regarding the stiffness recovery parameters, the CDP14 variants (w_t and w_c equal to 0.2 and 0.8, respectively) does not significantly affect the analysis. However, the CDP15 variants, characterised by non-realistic stiffness recovery parameters (w_t and w_c equal to 0.87 and 0.16, respectively), results in substantial strength and stiffness decay.

Based on the results obtained for the single panels, a “refined”

material characterisation is specified for the analysis of the perforated wall and provided in Table 9. Some parameters are selected to align the numerical model with the experimental hysteretic capacity. Therefore, they are chosen by selecting values within ranges that resulted in cyclic responses closer to the experimental curve. These parameters include dilatancy, global tensile strength, stiffness degradation parameter, tensile fracture energy, and peak plastic strain in compression. Meanwhile, other parameters, such as the ratio between yielding and maximum strength and stiffness recovery parameters, are the default reference values, considered the most common. The Wall D model is discretised using a coarser mesh of 125 mm $\times$ 125 mm $\times$ 250 mm with 7371 3-DoF nodes. Further mesh sensitivity analyses have demonstrated that adopted mesh characteristics are suitable. The same spreader system for applying the external loads, introduced in the Wall D mesoscale model, is employed (Fig. 11). The model is characterized by 7371 nodes and 22113 DoF, and no partitioning strategy is applied resulting in a single partition. The analysis was completed in approximately 20 h.

The global responses are presented in terms of force-displacement graphs in Fig. 22, for reference and refined characterisations. The selection of different material parameters significantly modifies the overall hysteretic response. The monotonic pushover analyses result in capacity curves that vastly overestimate the overall load capacity. Specifically, the maximum forces exhibit an overestimation of + 46 % with the reference characterisation and + 67 % with the refined one, compared with the experimental response. The cyclic responses also show substantial differences, but the refined characterisation provides a better prediction of strength and stiffness degradation, resulting in a progression of total energy remarkably close to the experimental one, as illustrated in Fig. 23. The experimental damage pattern (Fig. 5a) is only partially replicated by the models, as shown by the progression of tensile damage depicted in the two models presented in Fig. 24. Notably, the crack pattern appears more pronounced with the refined characterisation. Widespread flexural damage at the top and bottom of the piers on the first floor is observed in both models. Moreover, the shear-diagonal cracking damage on the piers of the first floor and the spandrel of the

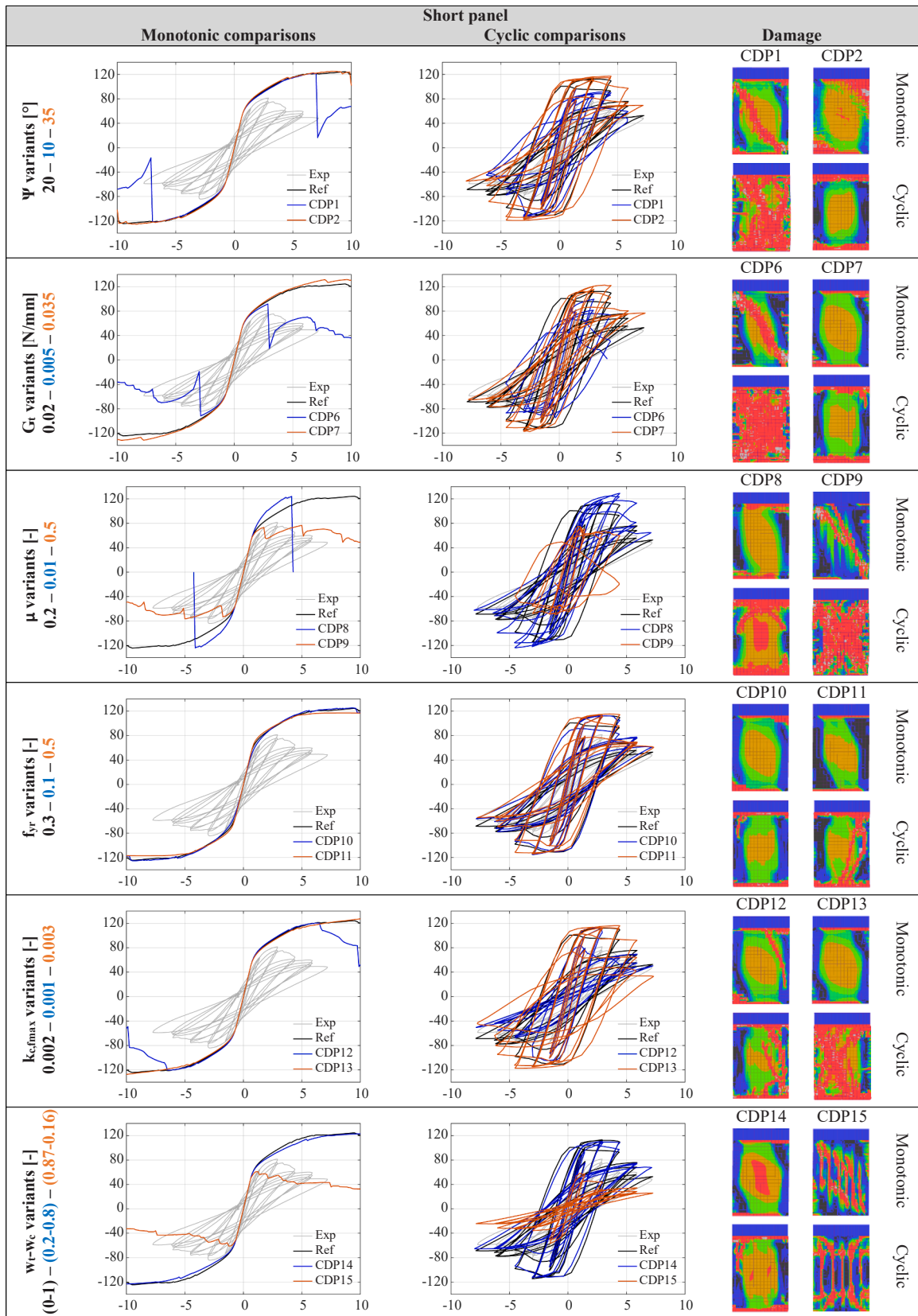


Fig. 18. Monotonic and cyclic shear force vs. horizontal displacement responses of macroscale short panel models (X-axis in [mm], Y-axis in [kN]) and damage patterns at the last step reached.

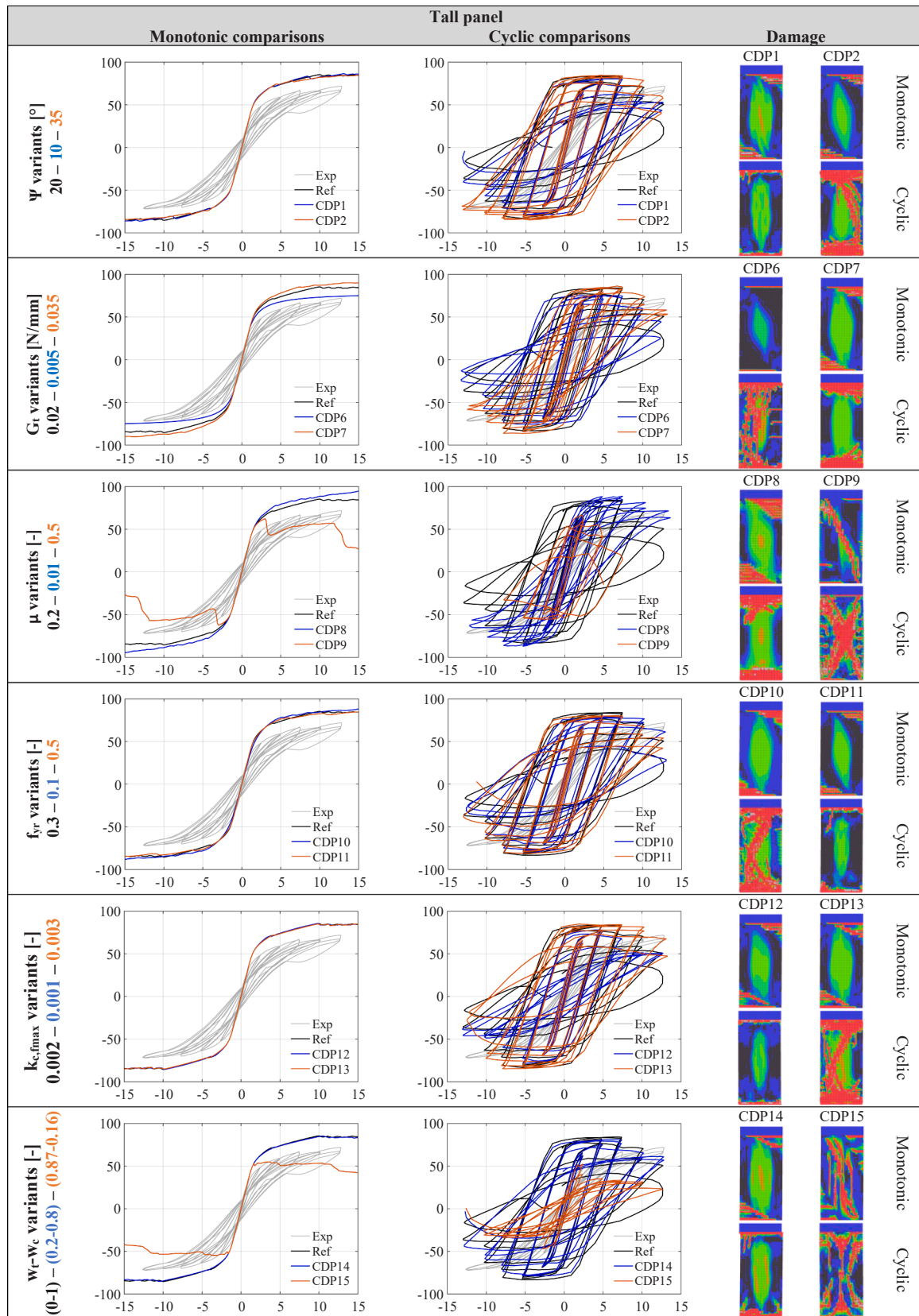


Fig. 19. Monotonic and cyclic shear force vs. horizontal displacement responses of macroscale tall panel models (X-axis in [mm], Y-axis in [kN]) and damage patterns at the last step reached.

Table 8
Summary of the results from sensitivity analyses on macroscale models (S=shear-diagonal cracking - F=flexural - F→S: flexural first, and then diagonal cracking).

Changing parameter	Case study	Short panel				Tall panel			
		Failure mode	W_{tot} [kN m]	% analysis completed	ΔW_{tot}	Failure mode	W_{tot} [kN m]	% analysis completed	ΔW_{tot}
ψ	Experimental	S	2.26			F	1.78		
	Reference	F→S	4.32	100 %		F→S	8.80	100 %	
	CDP1	F→S	3.32	65.9 %	−8.5 %	F	7.29	97.6 %	−16.9 %
f_t	CDP2	F	5.50	100 %	27.5 %	F→S	10.82	100 %	22.9 %
	CDP3	S	1.33	51.4 %	−45.3 %	S	0.52	47 %	56.2 %
	CDP4	S	1.94	54.1 %	−31.3 %	S	2.75	73.1 %	−42.3 %
G_t	CDP5	F→S	4.39	100 %	1.7 %	F→S	7.71	100 %	−12.4 %
	CDP6	S	2.57	54 %	−8.9 %	F→S	8.55	100 %	−2.8 %
	CDP7	F	3.84	100 %	−11.1 %	F	6.54	100 %	−25.7 %
μ	CDP8	F→S	3.52	100 %	−18.4 %	F	4.64	100 %	−47.2 %
	CDP9	S	1.93	51.3 %	−18.9 %	S	2.29	58.7 %	−30.4 %
	CDP10	F→S	4.30	100 %	−0.4 %	F→S	8.50	100 %	−3.3 %
$f_{y,r}$	CDP11	F→S	4.78	100 %	10.9 %	F	9.32	97 %	6.6 %
	CDP12	F→S	2.31	100 %	−46.5 %	F	4.86	100 %	−44.8 %
	CDP13	F→S	6.42	100 %	48.7 %	F→S	10.04	100 %	14.1 %
$w_t - w_c$	CDP14	F→S	4.04	100 %	−6.3 %	F	8.37	100 %	−4.8 %
	CDP15	S	0.97	100 %	−77.5 %	S	1.99	87.2 %	−74.4 %

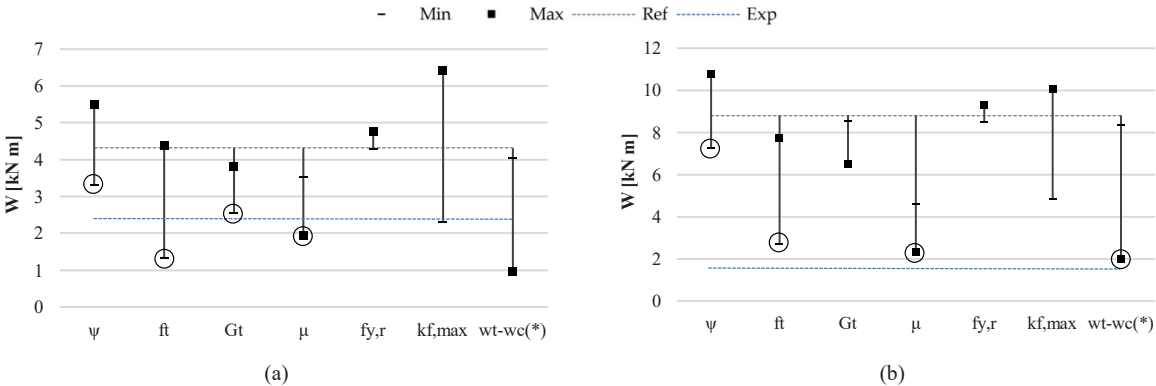


Fig. 20. Variation of final total energies (W) for (a) Short Panel and (b) Tall Panel macroscale models. 'Min' and 'Max' represent the final total energies with the minimum and maximum variants of each range, respectively, while 'Ref' denotes the total energy with the reference models. 'Exp' indicates the experimental total energy, and circled indicators denote incomplete analysis. (*) For the $w_t - w_c$ range, the minimum and maximum cases are assumed as CDP13 and CDP14, respectively.

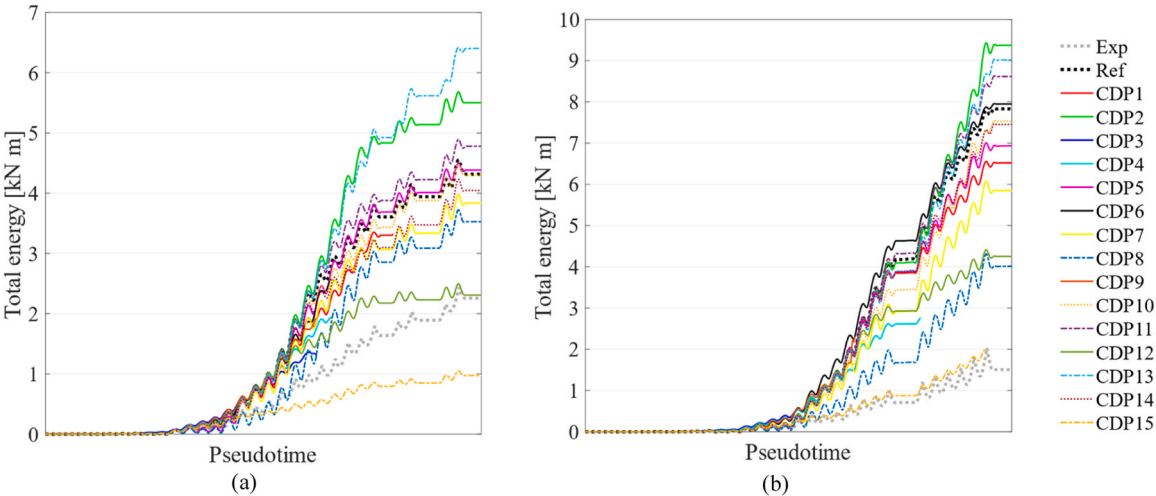


Fig. 21. Progression of total energies for each macroscale analysis for (a) short panel and (b) tall panel.

Table 9

Reference and refined characterisations for the Wall D macroscale models.

Parameter	Symbol	Unit	Reference	Refined
Young's modulus	E	MPa	1980	1980
Poisson's coefficient	ν	-	0.2	0.2
Dilatancy angle	ψ	°	20	10
Tensile strength	f_t	MPa	0.20	0.08
Fracture energy	G_f	N/mm	0.02	0.005
Stiffness degradation	μ	-	0.2	0.01
Maximum compression strength	f_c	MPa	6.2	6.2
Ratio between uniaxial yielding stress and maximum strength in compression	$f_{y,r}$	-	0.3	0.3
Plastic strain in compression	$k_{c,fmax}$	-	0.002	0.001
Tensile stiffness recovery	w_t	-	0	0
Compressive stiffness recovery	w_c	-	1	1

first level, as observed experimentally, is only partially reproduced: only the left pier at the base on the reference model collapses through a spreading of damage along the height.

4. Analyses on block-masonry wall structures

4.1. Experimental tests

In this chapter, the comparison between the modelling strategies is extended by analysing the response of block-masonry wallets. In-plane cyclic tests on masonry piers conducted at the University of Pavia within the FP6 European project “Enhanced Safety and Efficient Construction of Masonry Structures in Europe (ESECMASe)” [61], are considered. Among the various tested configurations, only specimens made of calcium silicate units are analysed. The walls were built using $250 \times 175 \times 250$ mm calcium silicate units (Type 6DF, specifically optimised for the project). The units were assembled with thin mortar bed joints, while the head joints were left unfilled. Mechanical properties were derived from compression and tensile tests on individual units and small assemblages, as summarised in Table 10. The experimental setup was designed to simulate either double-fixed or cantilever boundary conditions. Constant vertical loads were applied in conjunction with displacement-controlled horizontal cyclic loading at the top of the wall panels, with progressively increasing displacement amplitudes. Three tests have been selected for numerical simulation in the present work; their main characteristics are reported in Table 11.

4.2. Mesoscale and macroscale material characterisation

The material properties adopted for the mesoscale models are

reported in Table 12. The characterisation follows the procedure described in Section 3.2.1 and is based on the experimental values provided in Table 10. In addition to the reference material configuration, two alternative scenarios are analysed, featuring lower (Meso1) and higher (Meso2) values of Mode I and Mode II fracture energies for the mortar joints. For the head joints, reduced stiffness and strength values are considered to account for their unfilled condition.

Similarly, the material properties for the continuum macroscale models are defined following the procedure described in Section 3.3.1 and are summarised in Table 13, which includes the reference configuration and four variant case studies. Differently from the analysis on brick-masonry panels, in this case the tensile strength is derived from macroscale experimental tests, specifically from diagonal compression tests. Two variants for tensile strength are considered: the lower tensile strength is obtained from the experimentally determined cohesion value (CDP1), using Eq. (4), while the higher tensile strength corresponds to the joint tensile strength (CDP2). Additionally, variations in fracture energy are evaluated (CDP3 and CDP4), adopting the same values used in the brick-masonry panel analyses.

4.3. Numerical results

A total of nine cyclic analyses with mesoscale models and fifteen with macroscale models were carried out. The numerical-experimental comparisons of the shear-displacement cycles are shown in Fig. 25. For all mesoscale models, the initial stiffness and maximum shear strength are consistently well captured, along with the degradation of stiffness and, in general, the overall shape of the cycles, within the portion of the

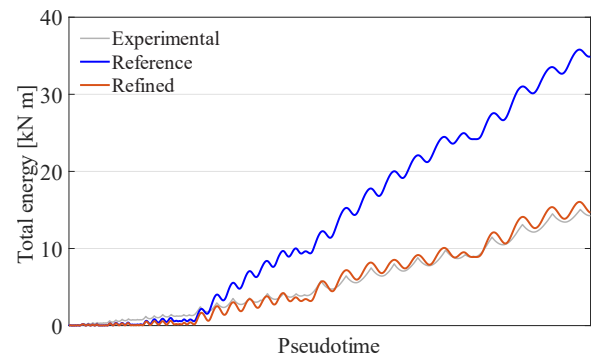


Fig. 23. Progression of total energies during the cyclic analysis for Wall D macroscale models: comparison between experimental, reference and refined characterisation.

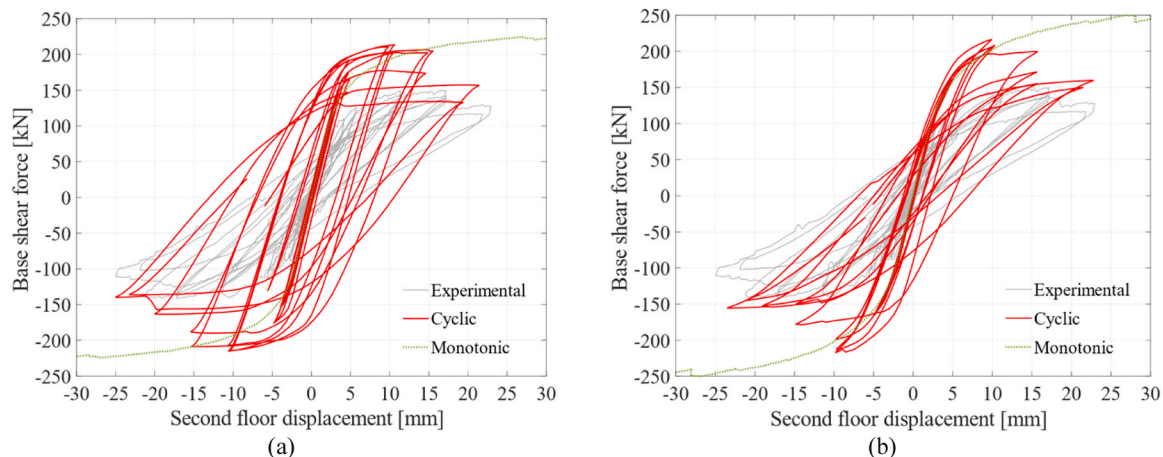


Fig. 22. Cyclic macroscale responses of Wall D with reference (a) and refined (b) characterisation.

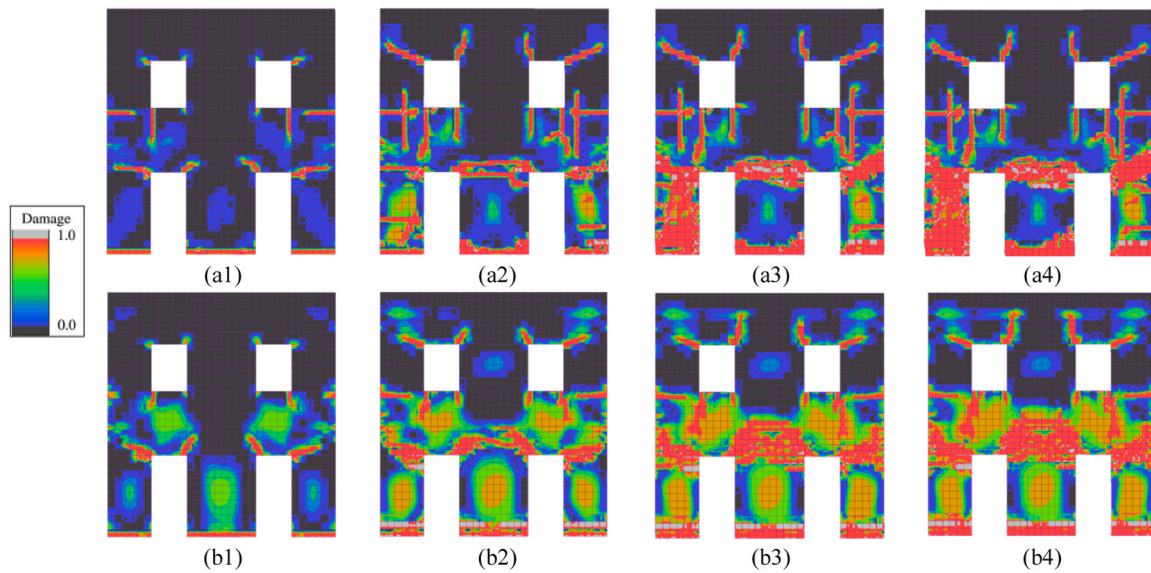


Fig. 24. Progression of crack pattern (tensile damage) during the cyclic analysis of Wall D macroscale models: (a1-a4) Reference characterisation; (b1-b4) Refined characterisation, with (1) drift 0.10 %; (2) drift 0.20 %; (3) drift 0.30 %; (4) drift 0.40 %.

Table 10

Material properties of the block-masonry wall specimens [42,61].

Property	Source	Mean value
Block tensile strength	Bending test	1.49 MPa
Block compression strength	Simple compression test	26.5 MPa
Block elastic modulus	Simple compression test	13620 MPa
Block Poisson's ratio	Simple compression test	0.253
Joint tensile strength	Cross-tension test	0.35 MPa
Joint cohesion	Triplet shear test	0.28 MPa
Joint coeff. of friction	Triplet shear test	0.55
Masonry compression strength	Compression test on wallets	23.6 MPa
Masonry elastic modulus	Compression test on wallets	7560 MPa
Masonry tensile strength	Diagonal compression tests	0.27 MPa

Table 11

Calcium-silicate masonry piers analysed.

Specimen	Vertical load [MPa]	Boundary condition
CS02	1.0	Double fixed
CS03	0.5	Double fixed
CS06	1.0	Cantilever

analysis successfully completed. In contrast, the macroscale models demonstrate greater numerical robustness, allowing the analyses to proceed further than in the mesoscale case. However, despite a reasonable match in initial stiffness, the maximum shear strength is never captured. The shear–displacement cycles significantly overestimate the total dissipated energy, without any noticeable strength or stiffness degradation.

All the main results are summarised in Table 14. Regarding the failure mode prediction, the mesoscale models consistently capture the experimentally observed failure mechanisms (see Fig. 26), specifically diagonal shear cracking in CS02 and CS03, and predominantly flexural behaviour in CS06. On the contrary, the macroscale models for CS02 and CS03 tend to localise damage only at the top and bottom of the panels. For CS06, however, the experimentally observed concentration of damage at the bottom is captured. As for the hysteretic response, the total energy at the end of the analyses for the mesoscale models is always significantly lower than the experimental value, primarily due to the premature termination of the simulations. In contrast, the total energy from the macroscale models is consistently much higher than the

Table 12

Material characteristics for reference and variant block-masonry mesoscale models.

Parameter	Unit	Reference	Variant Meso1	Variant Meso2
E_b	N/mm ²	13620		
ν_b	-	0.253		
$K_{n,j}$	N/mm ³	68.0		
$K_{t,j}$	N/mm ³	33.1		
$K_{n,b}$	N/mm ³	10 ⁴		
$K_{t,b}$	N/mm ³	10 ⁴		
c_j	N/mm ²	0.28		
$\tan\phi_j$	-	0.55		
$f_{t,j}$	N/mm ²	0.35		
$f_{t,b}$	N/mm ²	1.49		
$f_{c,h}$	N/mm ²	23.6		
$f_{c,v}$	N/mm ²	11.8		
$G_{I,j}$	N/mm	0.016	0.005	0.035
$G_{II,j}$	N/mm	0.16	0.014	0.25
$G_{I,b}$	N/mm	0.12		
$G_{c,h}$	N/mm	23.1		
$G_{c,v}$	N/mm	19.6		
μ_j	-	0.1		
μ_b	-	0.1		

Table 13

Material characteristics for reference and variant block-masonry macroscale models.

Parameter	Unit	Reference values	Variants	Case study
E	MPa	7560		
ν	-	0.253		
ψ	°	• 5 for CS02-CS06 • 20 for CS03		
$\tilde{f}_{b0}$	-	1.16		
ε	-	0.1		
K_c	-	0.66		
f_t	MPa	0.27	0.139 – 0.35	CDP1 – CDP2
G_t	N/mm	0.02	0.005 – 0.035	CDP3 – CDP4
μ	-	0.2		
f_c	MPa	23.6		
f_{yr}	-	0.3		
$k_{c,fmax}$	-	0.002		
w_t	-	0		
w_c	-	1		

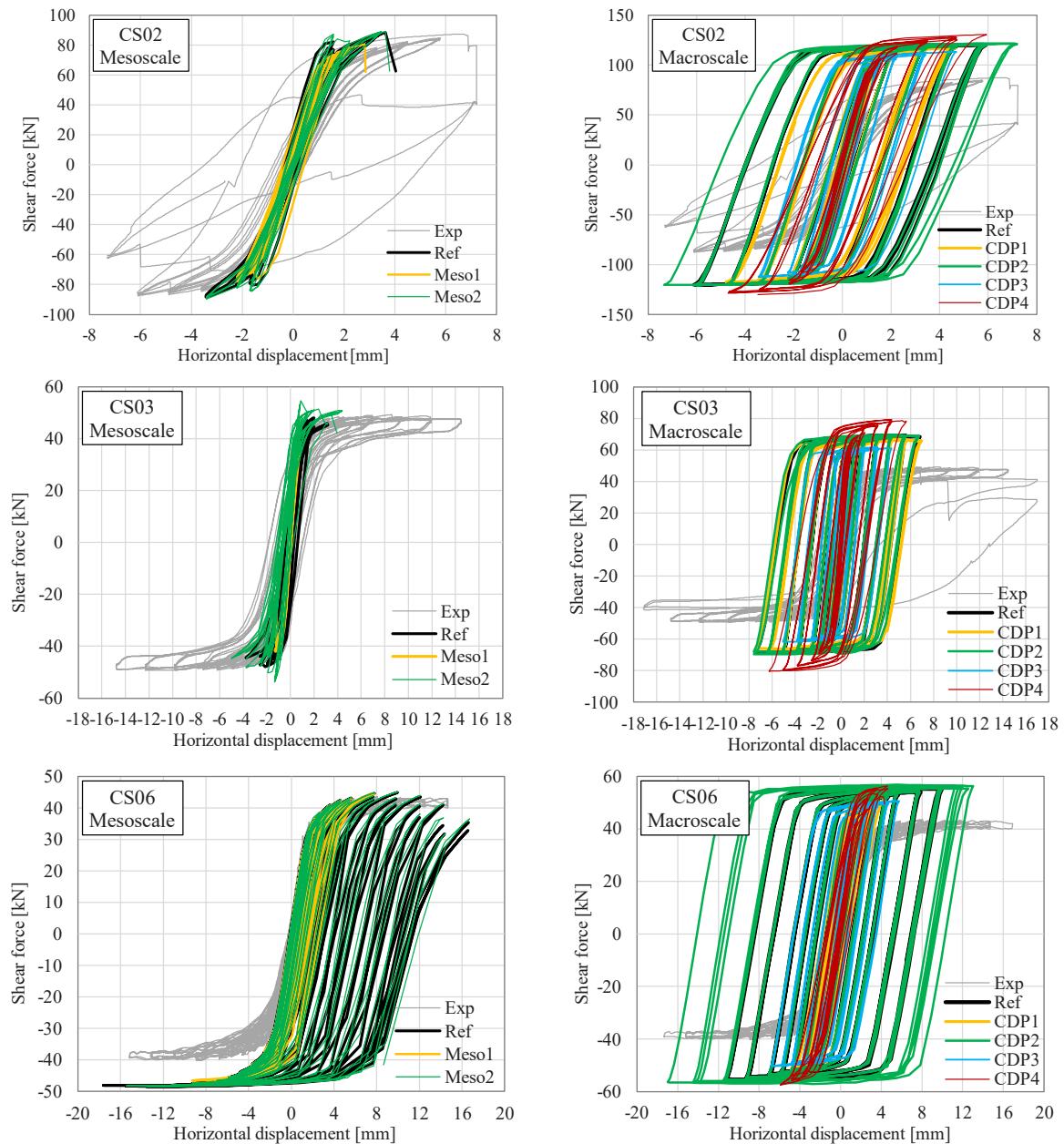


Fig. 25. Block-masonry mesoscale and macroscale models: experimental–numerical comparison of shear–displacement cycles.

experimental one. An example is shown in Fig. 27, which presents the experimental–numerical comparison for the CS02 panel. It can be seen that, for the portion of the analyses completed, the trends of the mesoscale models closely match the experimental response. In contrast, the macroscale models, although extending further into the analysis, tend to diverge from the experimental behaviour at an early stage.

5. Conclusions

This study evaluated the effectiveness of the mesoscale modelling strategy proposed in [16,23] and a continuum macroscale approach utilising the CDP material model [17] in predicting the hysteretic response of masonry wall components. The numerical results indicate significant variability in response predictions when employing the two alternative modelling strategies, despite using the same material test results for material model calibration. Given baseline sets of material parameters determined from physical experiments on masonry constituents and small wallets, the models were used to simulate the response

of brick masonry wall structures (two panels with different aspect ratios and a large perforated wall) and block masonry wall structures (three panels with varying confinement levels and boundary conditions) subjected to monotonic and cyclic loading. The main results from the mesoscale simulations indicate that the mesoscale models with a straightforward calibration of the model material parameters lead to accurate predictions of the load–displacement response and the evolution of damage and cracking in the masonry, where the numerical results are in excellent agreement with the experimental findings. Notably, the numerical curves are not significantly affected by variations within realistic ranges of some model material parameters that cannot be directly obtained from standard experiments. On the other hand, the adopted macroscale modelling strategy using a continuum damage–plasticity isotropic material model for masonry, along with standard calibration of the model material properties, does not consistently produce results that closely match the experimental responses in terms of load–displacement curves, failure modes, and dissipated energy. In this case, changes in some material properties, which are difficult to

Table 14

Summary of the results from analyses on block-masonry models (S=shear-diagonal cracking - F=flexural - F→S: flexural first, and then diagonal cracking; W_{tot} in kN m).

Mesoscale												
CS02				CS03				CS06				
Case study	Failure mode	W_{tot}	analysis completed	ΔW_{tot}	Failure mode	W_{tot}	analysis completed	ΔW_{tot}	Failure mode	W_{tot}	analysis completed	ΔW_{tot}
Experimental	S	3.41			S	5.12			F	6.98		
Reference	S	0.65	64.7 %		S	0.47	35.8 %		F	1.28	67.3 %	
Meso1	S	0.51	51.6 %	16.3 %	S	0.05	15.1 %	7.0 %	F	0.23	44.5 %	38.6 %
Meso2	S	0.61	64.6 %	−2.6 %	S	0.61	47.1 %	−4.0 %	F	0.39	64.9 %	4.0 %

Continuum macroscale												
CS02				CS03				CS06				
Case study	Failure mode	W_{tot}	analysis completed	ΔW_{tot}	Failure mode	W_{tot}	analysis completed	ΔW_{tot}	Failure mode	W_{tot}	analysis completed	ΔW_{tot}
Experimental	S	3.41			S	5.12			F	6.98		
Reference	F	11.94	91.1 %	−6.8 %	F	10.45	61.3 %	−1.3 %	F	13.12	51.1 %	−29.3 %
CDP1	F→S	6.12	77.2 %	1.8 %	F	10.85	62.6 %	0.6 %	F	1.33	29.9 %	0.6 %
CDP2	F	15.71	98.1 %	28.2 %	F	12.19	64.6 %	14.2 %	F	24.77	61.1 %	9.1 %
CDP3	F→S	4.47	66.1 %	−31.0 %	F	4.38	43.6 %	−19.4 %	F	5.28	38.7 %	−55.6 %
CDP4	F	5.57	80.4 %		F	5.68	52.8 %		F	1.27	33.1 %	

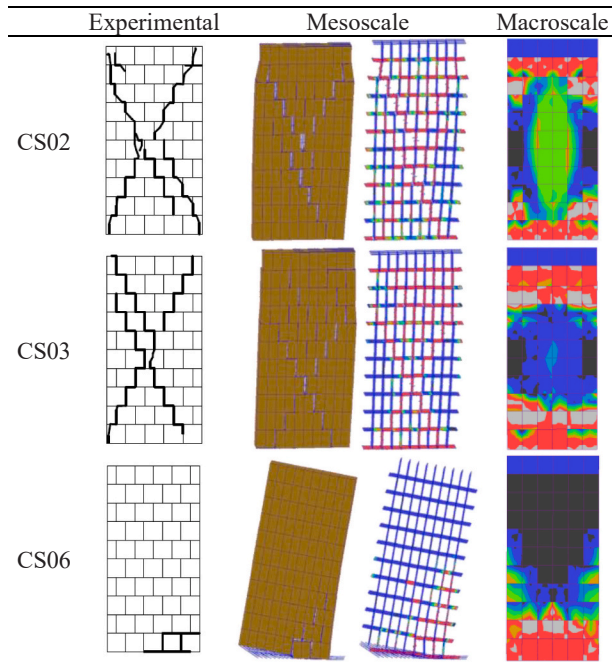


Fig. 26. Comparison between experimental damage observations and numerical predictions for the block-masonry reference case studies using mesoscale and macroscale models.

quantify objectively, lead to significant variations in the numerical predictions. Refined model calibration provides more accurate results, but some typical features of the actual load-displacement curves, such as the S-shape of the load-displacement response associated with the rocking behaviour and flexural failure of slender panels, cannot be captured.

The research guided the selection of material parameters for meso-scale and macroscale masonry models. For mesoscale models, accurate numerical predictions are ensured by defining:

- Strength parameters for nonlinear mortar interfaces (cohesion, friction coefficient, bond strength) based on standard material tests such as triplet shear and cross-couplet tension tests on mortar joints.

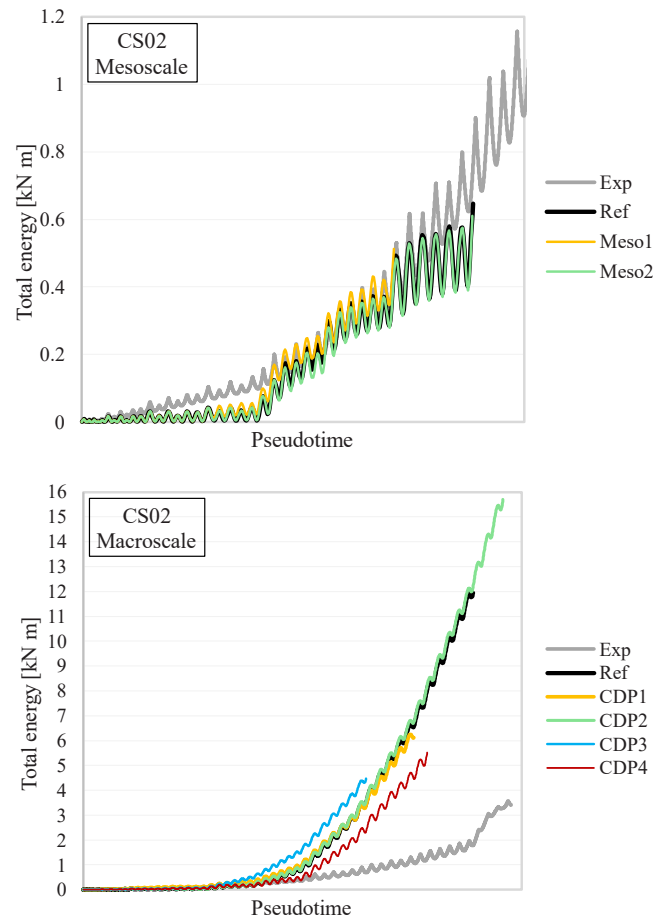


Fig. 27. Comparison of total energy between CS02 mesoscale and macroscale models.

- Strength parameters for nonlinear unit-to-unit interfaces (tensile and compressive strengths) derived from bending and compression tests on masonry units;
- Compressive strength of mortar joints obtained through compression tests on masonry wallets or, alternatively, by flat-jack tests

- Stiffness of mortar joints calculated using Eqs. (1) and (2), employing the elastic modulus of masonry (determined from compression or flat-jack tests on masonry wallets) and the elastic modulus of units (from compression tests on masonry units).
- Upper-bound but still realistic fracture energy values for mortar joints (e.g., Mode I fracture energy of 0.035 N/mm and Mode II fracture energy of 0.25 N/mm) are recommended to facilitate convergence of nonlinear simulations (these parameters, evaluated within realistic literature ranges, have been shown not to affect the response significantly).

For macroscale models, calibration based on a small mesoscale model is advisable. If this is not feasible, given that the responses obtained in all analysed cases tend to deviate considerably from experimental results, the following conservative parameter recommendations apply:

- Masonry Young's modulus can be determined from compression tests on masonry wallets, or flat-jack tests.
- Tensile strength should be taken as the minimum of bond strength, tensile strength derived from cohesion (3), and global tensile strength from macroscale tests, if available (simulations on block masonry have shown that even using tensile strength derived from diagonal compression tests does not ensure accurate prediction of base shear strength)
- Low dilatancy angles in the range of 5° to 10°
- Low fracture energy values in the range of 0.005–0.01 N/mm.

Future research directions include extending the study by analysing the response of spandrel elements and considering the effects of out-of-plane loads. Further analyses on larger structures within the macroscale framework could provide valuable insights into how variability in individual components influences their overall hysteretic response. The results of this research can support the development of numerical models for retrofitted masonry walls at different scales.

CRedit authorship contribution statement

Bassam A. Izzuddin: Writing – review & editing, Supervision, Software, Resources. **Corrado Chisari:** Writing – review & editing, Supervision, Methodology, Conceptualization. **Lorenzo Macorini:** Writing – review & editing, Validation, Supervision, Methodology, Investigation, Conceptualization. **Luca Bomben:** Writing – review & editing, Writing – original draft, Visualization, Validation, Methodology, Investigation, Conceptualization.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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