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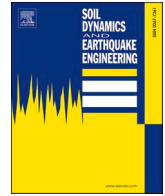
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Out-of-plane seismic behavior of non-framed unreinforced masonry walls with pre-existing seismic damage: numerical study via a block-based modeling approach

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ABSTRACT

This paper presents a numerical investigation into the influence of seismic in-plane (IP) pre-damage on the seismic out-of-plane (OOP) response of non-framed unreinforced masonry (URM) walls. These effects, referred to as IP/OOP interaction effects, are rarely investigated for non-framed URM and remain insufficiently understood. Consequently, seismic design and assessment provisions for non-framed URM neglect their potential impact on the vulnerable OOP response of URM. The paper conducts a parametric analysis using the high-fidelity modeling approach developed by the authors and previously validated against the IP and OOP responses of non-framed URM walls. The approach represents URM unit-by-unit via 3D finite-element nonlinear expanded blocks and cohesive-frictional zero-thickness joints. Four wall geometries are considered and pre-damaged by static cyclic IP loading under varying pre-compression levels and boundary conditions. Different IP pre-damage states are used as initial conditions for static and dynamic OOP analyses. Dynamic simulations employ signals representing induced and tectonic earthquakes. OOP stiffness reduction is also tracked through modal analyses. The results show that one-way spanning walls exhibit almost no sensitivity to pre-damage, whereas two-way spanning specimens experience up to 30% and 33% reductions in OOP stiffness and strength at severe pre-damage state. Moreover, under specific conditions, such as opening walls under low pre-compression, does pre-damage alter the OOP failure mechanism and cause more drastic stiffness reductions. Among all configurations, only the pre-damaged one-way spanning walls collapse under induced seismicity, and only at intensities higher than real events. Finally, good agreement is observed between static and dynamic simulations results under pre-damage.

1. Introduction

Unreinforced masonry (URM) is one of the most commonly used materials in global construction, playing a dominant role in both heritage structures and newer low-rise buildings [1]. In the Netherlands, more than 90% of houses are made with brick URM [2]. This widespread use of URM is due to its straightforward construction process, low cost, good sound and thermal insulation, and limited maintenance requirements [3]. The strong performance of URM under vertical gravity loads, due to its notable compressive strength, makes it ideal for non-framed wall systems that carry structural weight [4]. However, compared to framed URM wherein the walls are placed inside a

surrounding frame [5–8], non-framed URM does not perform well when exposed to lateral forces such as earthquakes, mainly because of its weak shear and tensile properties [9–13]. Lateral forces can be either in-plane (IP) or out-of-plane (OOP) of walls, with non-framed URM being particularly susceptible to OOP loading [14,15], which often leads to full structural failure. A large portion of existing URM buildings have not been designed with resistance to seismic action [16], and many have also suffered prior damage from earlier loading, increasing their fragility under future earthquakes [17].

An example of these challenges can be found in the Groningen province in the Netherlands. Decades of seismic events induced by gas extraction have caused damage to the non-framed URM building stock

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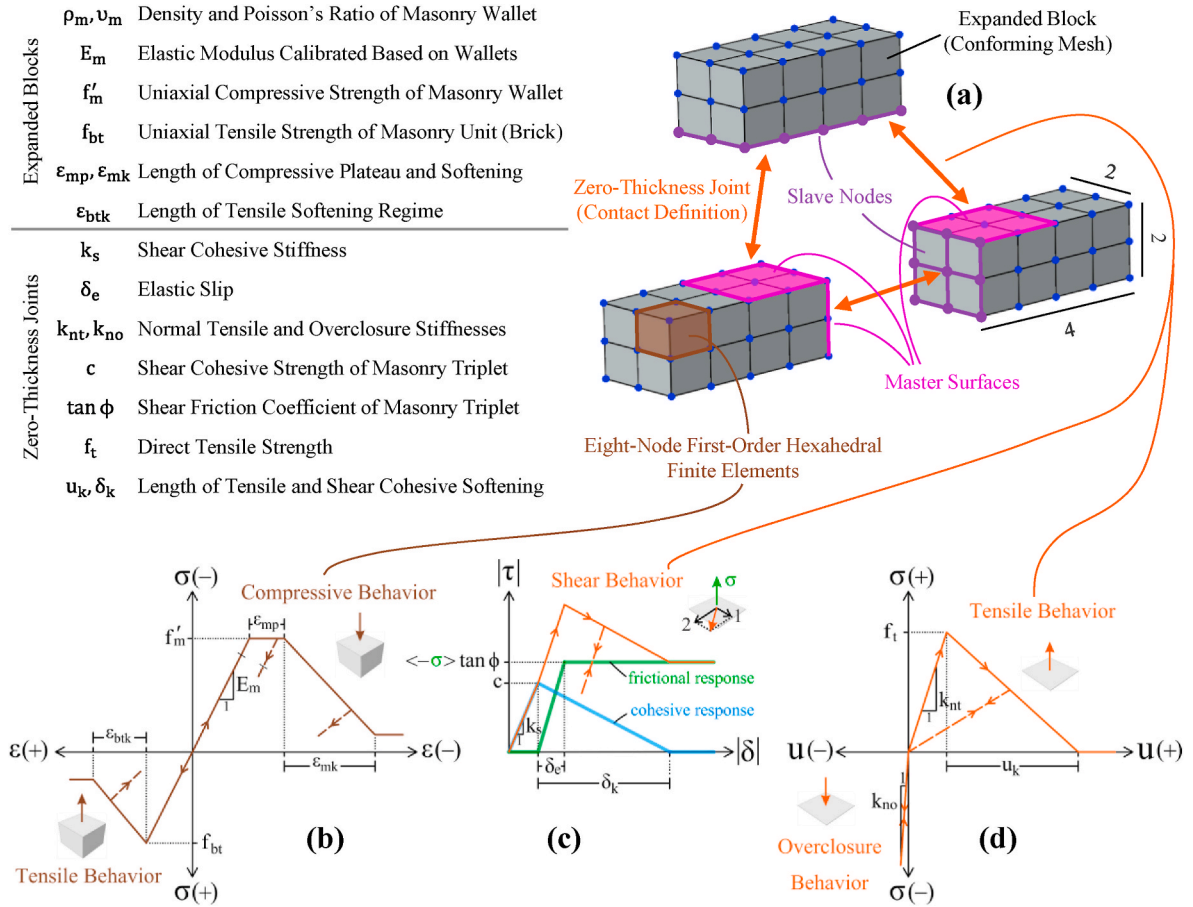


Fig. 1. The high-fidelity numerical modeling approach used in the current study: geometrical representation of URM (a), expanded blocks compressive and tensile behavior (b), and zero-thickness joints tangential shear response (c) as well as normal tensile and overclosure responses (d). The shear response of the joints is drawn for a scenario where $(-\sigma)\tan \phi$ is larger than c , and elastic slip is steeper than cohesive stiffness.

[18]. Although each event is minor in magnitude, roughly 1700 incidents have been recorded, greatly elevating seismic risk in a region historically not known for seismic activity [19]. Almost 28000 residential structures built with non-framed URM walls are impacted [18]. Two main factors worsen the situation: the highly slender wall profiles with reduced OOP capacity and repeated seismic activity that may continuously damage the buildings [20]. Consequently, these structures endure seismic forces they were not engineered to resist, often while already pre-damaged. The financial implications are substantial with €10 billion already allocated to repairs and an estimated €12 billion still needed for strengthening measures and homeowner compensation [19]. Moreover, reports show that nearly 29% of residents feel unsafe in their homes [21], revealing the deeper societal consequences of this ongoing issue.

One main limitation impeding intervention to mitigate the risk of damage in such regions is the lack of understanding regarding the effects of seismic pre-damage on the OOP response of non-framed URM walls. Given the multi-directional nature of earthquakes, seismic pre-damage can originate from IP, OOP, or combined responses to prior events. However, the effects of pre-damage caused specifically by IP actions, referred to as seismic IP pre-damage, on subsequent OOP response (i.e., IP/OOP interaction effects) are considered to be of greater investigative relevance for several reasons. First, IP/OOP interaction effects in non-framed URM walls are not sufficiently investigated. Previous studies either focus on IP/OOP interaction effects in framed URM walls (infilled or confined in surrounding frames) [22], or consider the reversed interaction scenario, OOP/IP interaction, when studying non-framed URM walls [23–36]. Meanwhile, for IP/OOP interaction effects in

non-framed URM walls, limited physical tests [37,38] and numerical studies [39–42] are available. Second, the current knowledge of the IP/OOP interaction effects in non-framed URM walls is not comprehensive. Existing tests are conducted on small wall specimens (wallets) with boundary conditions that are not representative of non-framed URM construction. Similarly, the available numerical studies predominantly focus on one-way spanning walls [39,41] and often rely on simplified static loading procedures, neglecting the inertial effects that govern the true dynamic response of buildings during earthquakes. This is while studies of IP/OOP interaction effects in framed URM walls have already covered full-scale wall specimens under dynamic loading [43]. Consequently, further validation is required to ensure the reliability of these outcomes. As a result, current design and assessment guidelines entirely disregard IP/OOP interaction effects in non-framed URM walls, which may lead to inaccurate evaluations of structural safety [44–48]. Third, while previous studies correctly focus on the IP response of non-framed URM walls which forms the primary load transfer path in buildings during seismic loads [49], the study of pre-damaged OOP responses is deemed equally important [50], especially for situations such as those of the buildings in Groningen where OOP collapse is high-risk due to their slenderness.

This study aims to contribute to a better understanding of effects of IP/OOP interaction on the OOP seismic response of non-framed URM walls. For this purpose, the simplified micro-scale (or meso-scale) block-based modeling approach developed in Ref. [51] and extended to dynamic simulation of one- and two-way spanning URM walls in Refs. [52, 53] is used. This approach simulates masonry unit-by-unit as an assembly of nonlinear expanded blocks and cohesive-frictional

zero-thickness joints. It is preferred over those previously employed in the study of IP/OOP interaction effects, as it enables simulation of dynamic OOP responses and has been extensively validated against the IP and OOP responses of non-framed URM walls [54–56]. Concerning the former, the static pushover method used in the previous studies is widely used in both practice and literature due to its significantly lower computational demand compared to dynamic analyses [25,57,58]. However, its appropriateness, particularly given that it neglects inertial effects, which govern true seismic response, requires further validation [17]. The approach is also favored over simplified continuum-based [59–69] or macro-elements-based [70,71] approaches, as it captures both local and global responses in greater detail [72]. Moreover, it allows for the use of more realistic IP pre-damaging procedures than the monotonic ones currently adopted [39,40]. Finally, the modeling approach is already validated against multi-step dynamic OOP wall tests [52,53] where the effects of progressive damage on the subsequent OOP responses are considered and simulated with high accuracy. Hence, the approach is already proven to be able to capture the effect of pre-damage on the OOP performance.

The study is presented as follows. A summary of the modeling approach, presenting its key features in terms of geometry, mechanical behaviors, solver, and damping, is provided in Section 2. The ability of the modeling approach to simulate the IP response of flanged URM walls is validated in Section 3. This step is deemed necessary to ensure the reliability of the outcomes of this study, as two-way spanning OOP boundary conditions are reproduced through the use of flanges (also called return walls) perpendicular to the main wall and connected to its lateral edges. Section 4 outlines the parametric analysis conducted on the effects of IP pre-damage on the OOP performance of non-framed URM walls. Several two- and one-way spanning specimens are subjected to cyclic IP loading under varying overburden and loading conditions, and are then undergone static and dynamic OOP loading at different pre-damage states, as well as modal analysis. Section 5 presents and discusses the outcomes of the simulations in terms of IP pre-damage patterns under different loading conditions, along with changes in OOP stiffness, static strength, dynamic collapse, and failure mechanism of the pre-damaged specimens. The section also compares the outcomes of the static and dynamic OOP simulations to investigate the validity of static analyses in the context of simulating interaction effects. The main outcomes of the study are ultimately presented in Section 6 with suggestions for future research.

2. Numerical modeling approach

The high-fidelity block-based numerical approach developed in Ref. [51] for quasi-static analysis of URM at material to building level, and extended to the dynamic regime in Ref. [52], is used in this study.

The approach is a simplified micro-scale (or meso-scale) modeling methodology, such as those in Refs. [33,73–81]: URM is simulated unit-by-unit, making the approach suitable for capturing not only global but also local responses of URM to various actions, including earthquakes. Since previous publications of the authors [52,55,82] have described the approach in detail, only a summary is presented here.

2.1. Geometrical representation

The typology of URM is represented via expanded blocks as units and zero-thickness horizontal (bed) and vertical (head) joints as mortar, as shown in Fig. 1a. The size of the expanded blocks corresponds to the size of the units plus the thickness of the mortar layers added to length and height. A $4 \times 2 \times 2$ (length $\times$ height $\times$ thickness) first-order hexahedral finite element arrangement is used for the discretization of each expanded block. A node-to-surface contact algorithm [83], which relating each discrete point on one surface (purple circles in the figure) to multiple points on the other (pink surfaces in the figure), is used to model the zero-thickness joints. The external nodes of the meshed expanded blocks serve as the contact points in the joints. The discretization resolution adopted for the expanded blocks, along with the three thickness contact points incorporated in the joints, has been shown in the previous studies of the authors [52,53,84,85] to realistically and accurately simulate responses from material to wall level, while also optimizing computational demands.

2.2. Mechanical constitutive behaviors

Expanded blocks are assigned nonlinear behaviors in both compression and tension, as shown in Fig. 1b, capturing crushing of URM assemblies and tensile failure of units. The isotropic material model known as Concrete Damaged Plasticity (CDP) [86], which adopts the Drucker-Prager type multi-yield surface [87], is used to formulate these nonlinear behaviors. The nonlinearity is formulated using. The compressive behavior consists of an initial elastic phase, followed by a plateau, then linear softening, and finally a plateau. The tensile behavior consists of an elastic branch, followed by linear softening, and a final plateau. In both regimes, the final plateaus retain 10% residual strength to prevent numerical instability. Under cyclic loading, expanded blocks exhibit elastic unloading in the elastic regimes and the first compressive plateau, while showing reduced-stiffness unloading in the softening regimes. The reduction in unloading stiffness in each softening regime is proportional to the corresponding strength loss. Coupling is considered between tensile and compressive behaviors: strength loss in one regime induces proportional softening in the other, and vice versa. Dilatancy is accounted for using a non-associative flow rule in the expanded blocks [86].

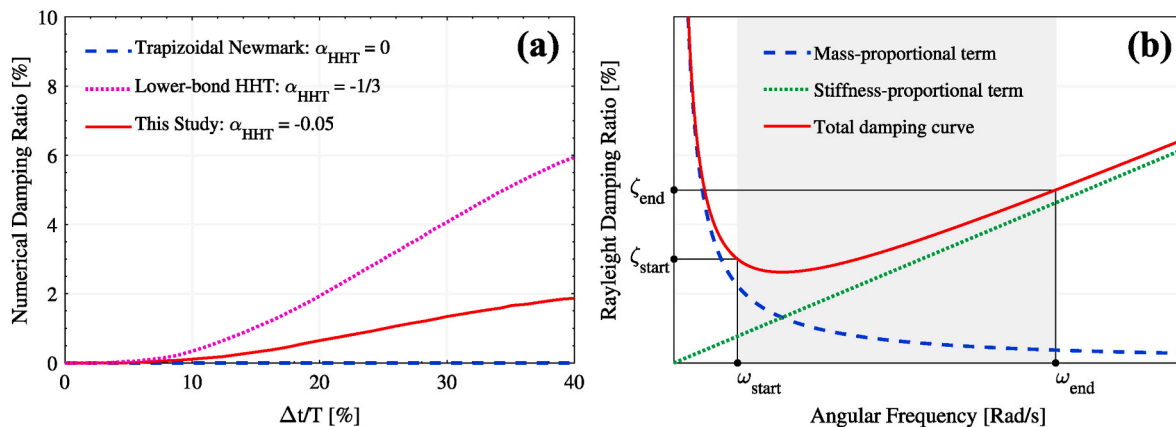


Fig. 2. Damping strategy adopted for dynamic simulations: numerical damping through the HHT implicit solver (a) and Rayleigh damping implemented in the blocks (b).

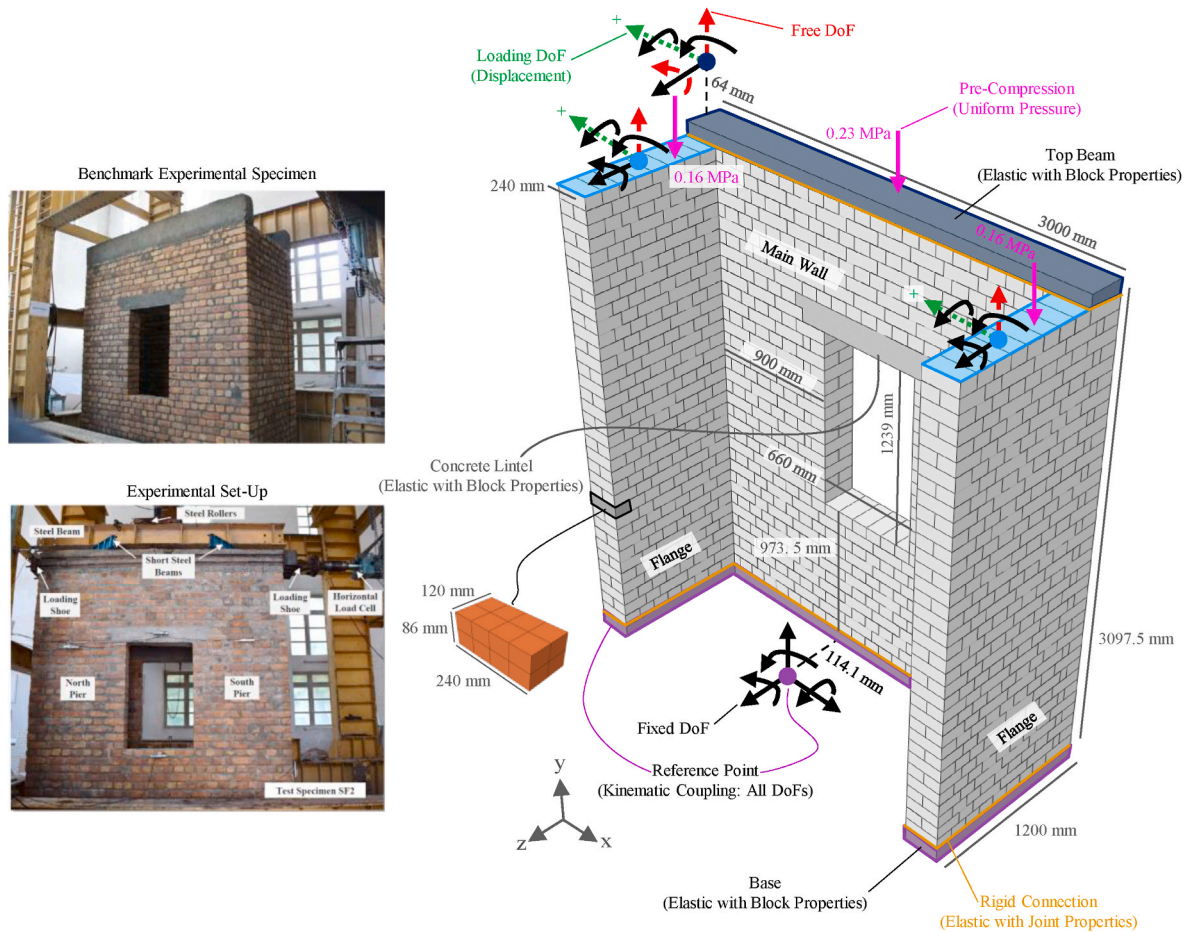


Fig. 3. Geometrical configuration and boundary conditions of the numerical specimen produced to simulate SF2 wall tested in Ref. [97]. The abbreviation “DoF” stands for degree of freedom.

The non-dilatant zero-thickness joints exhibit nonlinear behaviors in both the tangential and normal directions, and simulate shear and tensile failure of mortar-unit interfaces and mortar, respectively, based on relative displacement of the contact nodes. To enable simulation of large separations, sliding, and rotation of the surfaces, finite-sliding is employed [88]. As shown in Fig. 1c, the tangential response is characterized by an isotropic cohesive-frictional shear behavior. The cohesive component consists of an initial elastic branch followed by softening to zero strength. The frictional component includes an initial elastic slip, starting at the onset of cohesive softening, and a subsequent constant-friction plateau. Depicted in Fig. 1d, the response in the normal direction includes elastic overclosure and nonlinear tensile behaviors. A large stiffness is assigned to the overclosure regime to prevent block compenetration. The tensile regime consists of an elastic branch followed by softening to zero strength. Coupling is considered between the tensile and shear cohesive behaviors: softening in one leads to strength loss in the other, and vice versa. The overall cyclic response of the joints is characterized by elastic unloading in shear and overclosure, and secant unloading to the origin point in tension.

2.3. Solution procedure

For static analyses, such as those under gravity, vertical loading, and the simulation of quasi-static loadings used in experiments, an implicit-based static solution procedure is used [51]. For dynamic analyses, the Hilber-Hughes-Taylor- α (HHT- α) direct integration implicit solver with minimal numerical damping is adopted [89]. Both solvers employ automatic stepping: load increment stepping in the static solver and

time-stepping in the dynamic solver. This approach reduces computational demand by allowing rapid progression through increments with low nonlinearity and enhances numerical stability by enabling small step sizes in increments with high nonlinearity. A maximum allowable step size ($\Delta t_{\max}$) is imposed to prevent step sizes from becoming excessively large and to allow their effective downscaling when necessary. In dynamic analyses, $\Delta t_{\max}$ is set equal to the sampling rate used in the loading signal [52], which helps prevent artificial numerical damping (Fig. 2a) and supports high-fidelity data collection comparable to benchmark physical experiments. A minimum step size ($10^{-11} \times \Delta t_{\max}$), a maximum number of per-step iterations (10,000), and a maximum number of re-stepping attempts (15) in case of non-convergence with the current step size are defined for the solvers. The first two parameters ensure that high levels of nonlinearity are reliably managed without compromising stability. The third ensures that, in case of physical instability of the numerical specimen, the analysis is correctly halted and the generation of purely numerical responses is avoided.

2.4. Dynamic energy dissipation

Rayleigh damping is introduced in the expanded blocks to dissipate kinetic energy during dynamic analyses [90]. Both mass- and stiffness-proportional components of damping are applied without increasing computational demands, unlike in explicit-based solvers, owing to the use of the implicit-based HHT- α solver. The modal characteristics of each specific specimen are used to calibrate the parameters shaping the damping curve shown in Fig. 2b, i.e., α_R for the

mass-proportional and β_R for the stiffness-proportional components. Previous studies by the authors have demonstrated that a target damping ratio (ζ_R) of 5% yields sufficiently accurate simulation of dynamic OOP responses of one- and two-way spanning walls, as well as gables [52,55,82]. Therefore, the same target ratio is adopted here. One limitation of the current damping methodology, as noted in Ref. [53], must be acknowledged: the damping calibration is performed at the beginning of the analysis and remains unchanged throughout dynamic loading. When stiffness degrades due to progressive damage, the natural frequencies of the specimen under study shift, creating a mismatch between the real frequency range that should be damped and the range actually damped. This results in over-damping of the damaged response and may alter the response during collapse, such as by reducing rigid body motions. However, the results of this study are not expected to be affected by this limitation for two reasons. First, capturing of dynamic collapse progression, which is most influenced by this limitation, is not the objective of this study and only the onset of collapse is investigated. Second, an incremental dynamic analysis approach is used here where dynamic responses were simulated in single-run analyses, unlike in previous applications in Refs. [52,55,82] where multi-step sequential dynamic analyses were performed. This prevents run-to-run damage accumulation, which would otherwise intensify the effects of the limitation. The limitation explained in this section is not inherent to Rayleigh damping; Other damping strategies used in similar studies are also calibrated prior to the analyses and kept constant throughout the simulation [91]. Nevertheless, previous studies of the authors, including the simulation of the response of one- [52] and two-way [53] spanning URM walls under multi-step dynamic shake table loading, as well as the successful blind prediction of the collapse behavior of URM gables [55] and 3D-printed masonry-like sub-structures [56], show that, when calibrated properly, strategies such as Rayleigh damping can still allow correct approximation of highly nonlinear structural behaviors under damage accumulation and up to failure.

3. Model validation: simulation of a quasi-static cyclic IP experiment flanged wall

The modeling approach has already been extensively validated against quasi-static cyclic IP [92] and dynamic OOP [93–95] experiments of URM walls in Refs. [51,82,96] and [52,55,82], respectively. Therefore, these validations are not repeated here for brevity. Instead, the approach is validated against the quasi-static cyclic IP loading of a multi-wythe flanged wall reported in Ref. [97]. This validation is considered because some of the wall geometries used in the subsequent parametric analysis (see Section 4.1) include perpendicular return elements, also known as flanges, at their lateral ends, which may affect their IP response. Although the approach has previously been validated against an experiment on a full-scale building model [51], demonstrating its ability to simulate damage transfer between perpendicular walls, the simulation of the flanged wall presented here serves as further reinforces its reliability.

3.1. Test setup and geometry

The simulated specimen, identified as the SF2 wall in the reference campaign [97], is shown in Fig. 3 and features a $\perp$ -shaped geometry with flanges attached to one side of the main wall, oriented perpendicular to the loading direction. The specimen is built in full-scale using solid $230 \times 76 \times 110 \text{ mm}^3$ (length $\times$ height $\times$ thickness) Clay bricks and 10 mm thick cement-based mortar, with two bricks laid across the wall thickness along their thickness. Half-thickness bricks are also used at the wall-flange intersections. A centrally positioned symmetrical window is included at mid-length, and the wall is constructed in an English bond pattern. The concrete lintel above the window spans two brick rows in height, extends beyond the opening by half a brick length on each side, and is cast in-place. The IP loading is applied under 0.23 MPa and

0.16 MPa pre-compression applied to the top of the main wall and the flanges, respectively, via a concrete beam cast in-place at the top of the main wall. The wall is fixed to the base via high-strength adhesive. Cantilever boundary conditions are assigned to the main wall, while the flanges adopt a double-clamped configuration. Lateral loads are also transferred to the flanges through top steel beams connecting the main wall to the flange tops.

The numerical specimen constructed here follows the exact brick layout of the experimental benchmark. However, the wall thickness is increased to 240 mm: an additional 10 mm is added to the original 230 mm length of the blocks to account for mortar thickness along the length of the walls, and the same block geometry is used in the thickness direction for simplicity. Additionally, the thickness of each expanded block is extended by 10 mm, yielding an overall size of $240 \times 86 \times 120 \text{ mm}^3$ (length $\times$ height $\times$ thickness). The main wall measures 3000 mm in length, each flange is 1200 mm long, and the total wall height is 3097.5 mm. The window is 660 mm wide and 1230 mm high, positioned 900 mm horizontally from the interior face of the flanges and 973.5 mm vertically from the bottom of the wall. The wall base follows the same planar geometry and is modeled as an elastic solid with the same material properties as the bricks. It is connected to the wall using a high-stiffness elastic interface with linear properties of regular mortar and coupled to a central reference point with all DoFs constrained. The top beam is simulated as a reduced-height elastic part, using the same material properties as the bricks for simplicity. These properties are also assigned to the lintel. The top beam is kinematically coupled to a reference point for easier application of cantilever boundary conditions (free rotation about the z-axis in the figure). The coupling assumes the beam to act as a rigid body with deformations compatible with the DoFs of its reference point. The top reference point is located at the position of the original experimental loading point, i.e., the left edge of the wall at mid-height of the loading beam, to maintain the correct shear span and rotation center under cantilever loading. Each flange is similarly connected to a reference point placed at its center of geometry, aligned with the top surface of the flange, to impose the double-clamped boundary conditions (no rotation about the z-axis in the figure). Vertical uplift is permitted at all three top reference points. The simulation proceeds in three steps: application of gravity, vertical pre-compression via uniform pressure at experimentally prescribed levels, and IP loading, which is applied equally to the top reference points of the main wall and flanges. The full experimental loading history is applied, as it consists of a single cycle per target displacement amplitude. The static solver described in Section 2.3 is used in all loading steps.

3.2. Mechanical calibration

In the previous studies of the authors, the mechanical properties of the expanded blocks and zero-thickness joints are calibrated via the simulation of material-level characterization tests conducted on the building components of the wall under study in Refs. [51,52]. This approach has proven to lead to accurate prediction of wall-to-building-level responses with not micro-adjustment required [53]. However, the results of material-level experiments for the constituents of the wall simulated here are almost entirely absent from the reference publication [97]. Not only that, only the compressive and tensile strengths of the URM assembly are provided in Ref. [97]. Hence, a different material calibration approach is adopted in the simulation, with help from the experimental campaign conducted at Delft University of Technology (TU Delft, Netherlands) on similar Clay URM materials [98,99]. Accordingly, the material input parameters of the numerical model are calibrated via the procedure outlined below:

- 1 Use the input parameters calibrated in previous works of authors [82,96] simulating URM assemblies made with Calcium Silicate solid bricks as a basis;

Table 1

Material input parameters used for simulating the IP experiments on the wall tested in Ref. [97]. Values highlighted in bold are those assumed.

Expanded Blocks							
Elastic Behavior		CDP Parameters		Compressive Behavior		Tensile Behavior	
E_m [MPa]	3000^c	ψ [°]	10^a	f'_m [MPa]	4.34^b	f_{bt} [MPa]	0.65^d
ν_m [–]	0.16^a	ϵ [–]	0.1^a	ϵ_{mp} [–]	0.006^c	ϵ_{btk} [–]	0.002^c
Density [kg/m ³]	1708^a	f_{b0}/f_{c0} [–]	1.16^a	ϵ_{mk} [–]	0.012^c		
		ρ [–]	2/3^a				
Zero-Thickness Joints							
Overclosure Behavior		Tensile Cohesive Behavior		Shear Cohesive Behavior		Shear Frictional Behavior	
K_{no} [N/mm ³]	221^a	k_{nt} [N/mm ³]	221^a	k_s [N/mm ³]	87^a	$\tan \phi$ [–]	0.87^c
		f_t [MPa]	0.085^c	c [MPa]	0.15^c	δ_e [mm]	0.001^a
		u_k [mm]	0.6^d	δ_k [mm]	0.6^c		

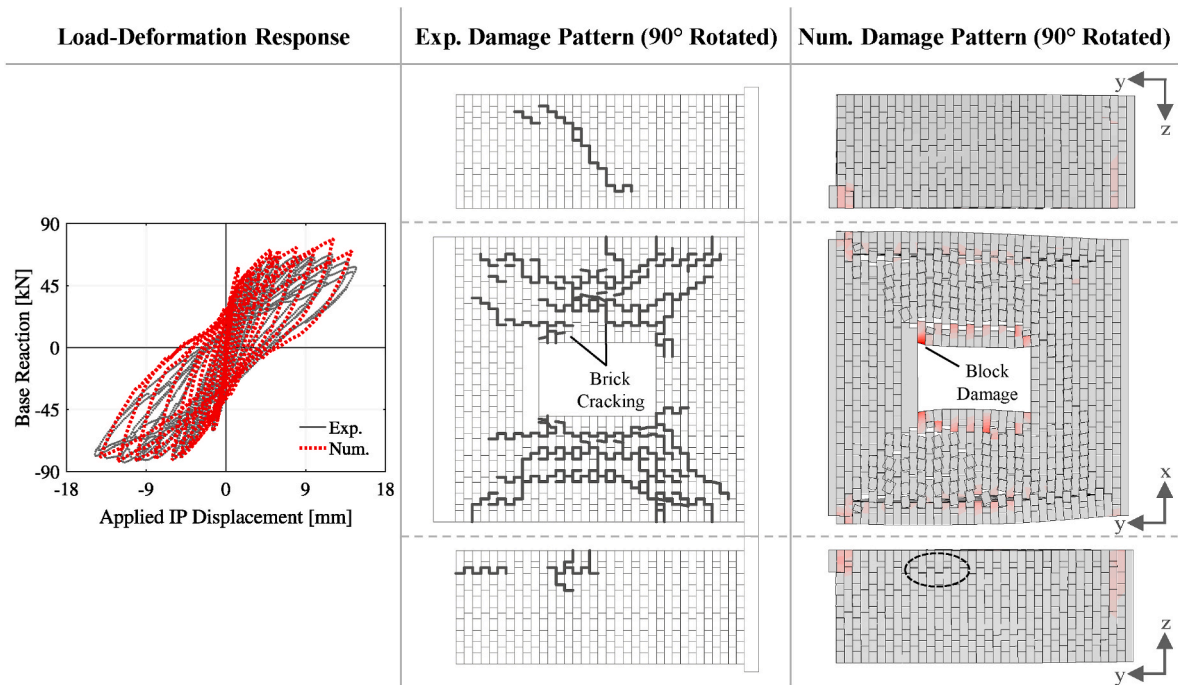
^a From previous simulations of URM assemblies made with Calcium Silicate bricks [82,96].^b From publication of the benchmark wall experiment [97].^c From experiments of TU Delft campaign on Clay URM [98,99].^d From equations proposed in Ref. [100] based on TU Delft experiments.

Fig. 4. Comparison between the experimental response of the SF-2 wall tested in Ref. [97] and the numerical model developed in this study. The abbreviation “Exp.” refers to the benchmark experimental data, and the “Num.” refers to output of the numerical model of this study. Experimental damage pattern is adapted based on [97] and is rotated counterclockwise by 90° for the clear visualization. The numerical deformed shape is magnified for clear visualization. Compressive and tensile damages in the numerical expanded blocks are highlighted in red. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

- 2 Retain the value of URM compressive strength (f'_m) reported in the reference experimental publication [97]; and,
- 3 Modify the missing input parameters requiring Clay URM-specific fitting based on values recorded in the TU Delft Clay URM experiments [98,99].

The procedure yields the input parameters listed in Table 1, where the values highlighted in bold represent the assumed parameters corresponding to the missing data. It is important to note that these assumed values are directly taken as the average results from the TU Delft Clay URM experiments, without any further adjustment. The remaining, not highlighted, parameters, except f'_m , are those used from previous studies of the authors [82,96]. It should be noted that, while not modified, the non-highlighted parameters are typically assigned

similar values regardless of the type of URM simulated (CDP parameters, K_{no} , k_{nt} , k_s , and δ_e), or they have minor effect on the wall-level quasi-static response (density and ν_m). For instance, CDP parameters are given values typical to quasi-brittle materials such as Clay or Calcium Silicate URM [86,101,102]: eccentricity (ϵ) of 0.1, biaxial-to-uniaxial initial compressive strength ratio (f_{b0}/f_{c0}) of 1.16, and tensile-to-compressive meridian ratio (ρ) of 2/3. Similarly, the 10° dilatancy angle (ψ) is based on prior experimental and numerical studies [103,104]. Moreover, joint stiffness and elastic slip are assumed values that prevent major elastic deformations regardless of the type of URM material. In fact, these parameters have already been successfully used to simulate sand-based URM-like structures as well [56], showing their applicability beyond Calcium Silicate URM. It should be noted that similar properties are given to head and bed joints, assuming equal construction quality across the specimen, following the previous studies of the authors [51,

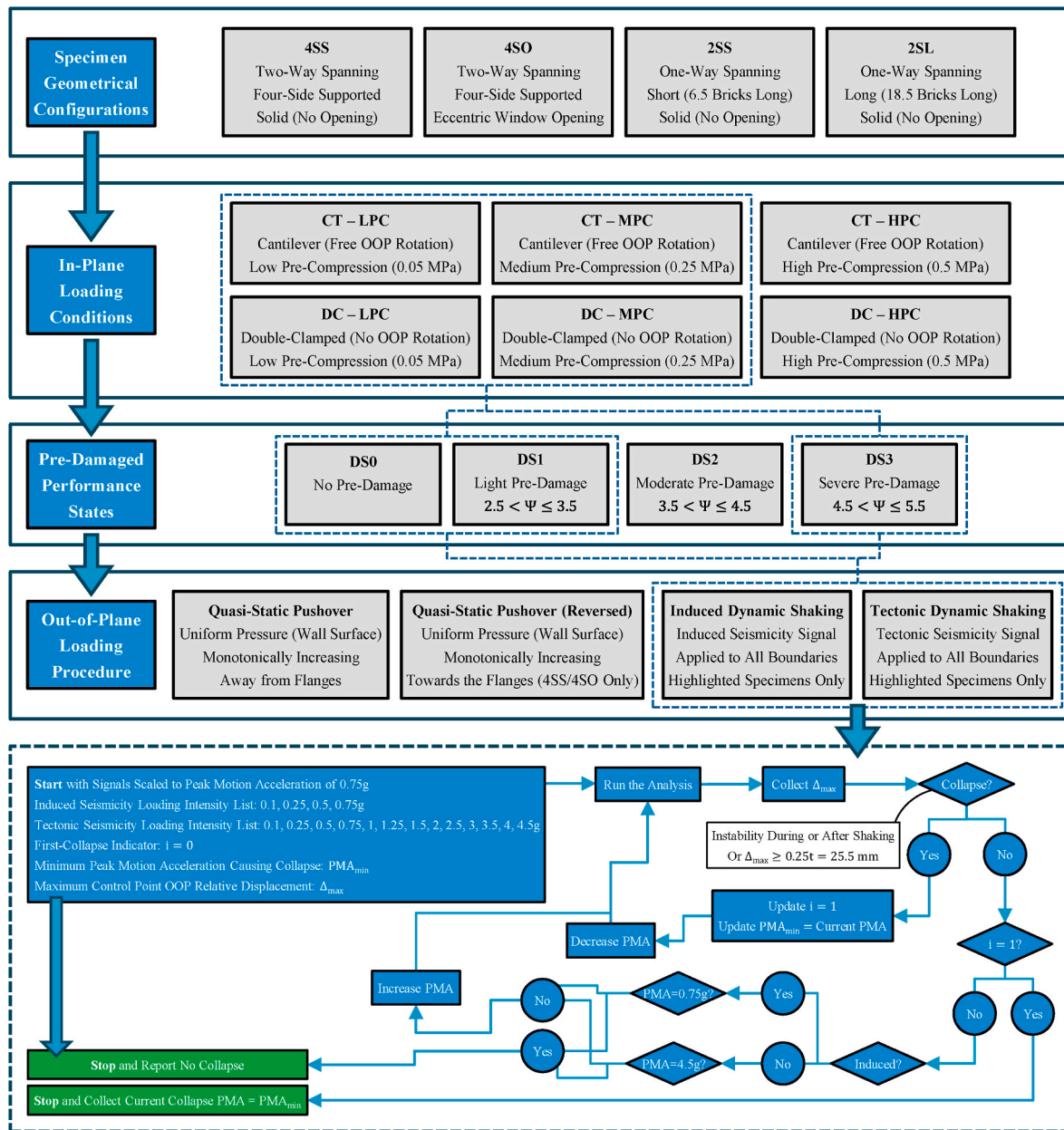


Fig. 5. Parametric analysis setup. In the pre-damaged performance states, Ψ is the damage parameter proposed in Ref. [105].

52].

3.3. Results and comparison

The simulation results are presented in Fig. 4 and demonstrate very good agreement with the experimental observations in terms of both load-deformation response and damage pattern (rotated counterclockwise by 90° due to space limits), without the need for adjusting the assumed material properties. The simulation not only captures the envelope of the load-deformation curve with high accuracy but also provides a reliable estimate of the energy dissipation in each loading cycle. Additionally, the major cracks observed in the experiment, which lie in the piers adjacent to the window and the spandrel below it, and the vertical cracks in the left flange at the wall-flange connection, are well-reproduced numerically. Notably, the simulation of brick cracking highlights the ability of the numerical approach in capturing damage transfer between discrete joints and continuum blocks. It should be noted that the responses of both the experimental and numerical

specimens are asymmetric in the positive and negative loading regimes, which is due to the loading point not being located at the mid-length of the wall.

Despite the overall good agreement, the numerical simulation shows a slightly lower level of alignment with the experimental results in the positive direction, and the right pier fails does not show the diagonal crack observed during testing. These deviations may also be attributed to the eccentricity of the loading point as well as to the variations in vertical compression of the flanges during the experiment, neither of which are reflected in the numerical model. Besides, the diagonal crack of the pier is a secondary damage not expected to have affected the simulation accuracy, given that the rest of the response is captured well. The overall comparison highlights the excellent predictive capability of the proposed modeling approach, even when applied to benchmark cases with limited information regarding their constituent materials. Moreover, since the simulated specimen had a non-flanged counterpart in the original campaign [97] that exhibited a measurably different IP response, it can be concluded that the modeling approach developed in

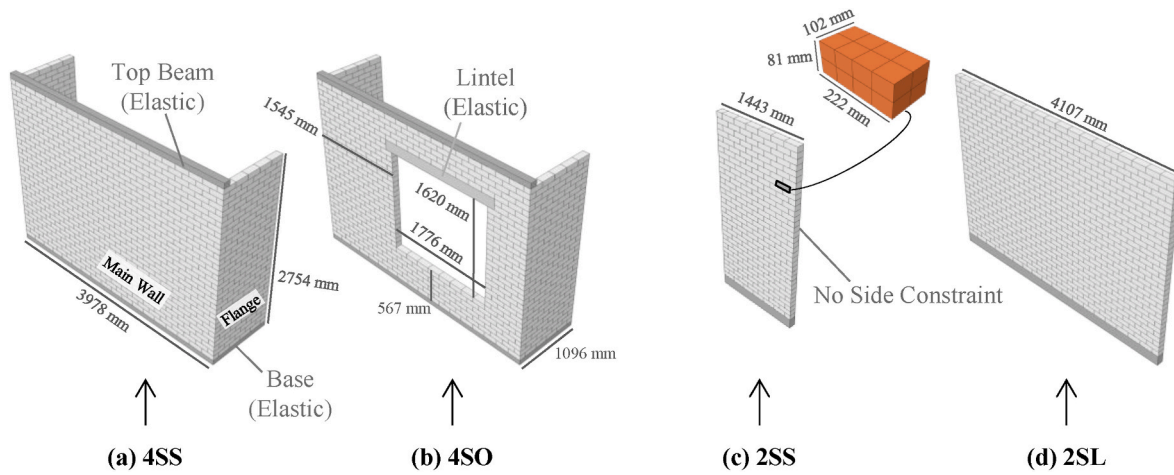


Fig. 6. Geometry of specimens used in the parametric analysis.

this research effectively captures the influence of flanges on IP response.

4. Parametric analysis on OOP response of walls with seismic pre-damage

Fig. 5 shows the setup of the parametric analysis conducted on the seismic OOP response of URM walls have been pre-damaged by past seismic events. Different wall geometries are subjected to cyclic IP loading under varying boundary conditions and overburden levels. Several performance states in the IP response of the walls are identified and used as initial conditions for the subsequent OOP analyses. Both static and dynamic OOP loadings are applied, with the latter conducted in the form of incremental dynamic analysis (IDA). More details regarding each component of the study are provided in the following sections. Nonetheless, it is important to note that many decisions made at various stages of the study are aimed at ensuring its applicability to the assessment of the residential URM building stock in the Groningen province of the Netherlands, subjected to induced earthquakes due to gas extraction [106].

The current setup of the parametric study, combined with the use of a validated high-fidelity modeling approach, introduces improvements to previous works [39–42] in multiple fronts: (1) adoption of a more realistic cyclic IP pre-damaging protocol, better representing seismic actions, (2) a multi-variational pre-damaging framework that captures a broad range of damage scenarios, (3) the use of material properties grounded in experimental benchmarks, rather than relying on numerically calibrated assumptions, (4) inclusion of dynamic OOP loading, alongside static analyses, allowing for cross-validation, and (5) extension of the investigation to two-way spanning wall configurations, which have received limited attention in prior studies despite their

relevance in real structures.

4.1. Specimen geometries and mechanical properties

The four wall geometries depicted in Fig. 6 are used in the parametric analysis, all being full-scale single-leaf and assumed to be at the ground story of a building. Two specimens are two-way spanning walls connected to flanges at their lateral boundaries and a beam at the top, one of which includes an eccentric window opening. The other two specimens are one-way spanning walls, restrained only at their top and bottom boundaries, with different lengths. The numerical procedure outlined in Section 1 is used for the construction of the specimens via $222 \times 81 \times 102 \text{ mm}^3$ (length $\times$ height $\times$ thickness) expanded blocks in a running bond configuration. The two-way spanning walls follow the geometry as the specimens tested in the experimental campaign in Ref. [93] under sequential dynamic shake table loading, and simulated in Ref. [53] for validating the modeling approach. The main wall is $3978 \times 2754 \times 102 \text{ mm}^3$, while the flanges are 1096 mm long with the same height and thickness as the main wall. In the specimen with the window, the opening size is $1776 \times 1620 \text{ mm}^2$ (length $\times$ height), covering 26% of the surface of the main wall. The height of the bottom spandrel is 567 mm, and the length of the large pier is 1545 mm. The lintel above the window has the height of two blocks and extends beyond each side of the opening by half a block length. One of the one-way spanning specimens follows the geometry of the specimen tested in Ref. [107] under sequential dynamic loading and simulated in Ref. [52] to validate the modeling approach. It has the same height as the two-way spanning walls and a short length of 1443 mm. The second one-way spanning specimen is based on the geometry of the walls tested under quasi-static cyclic IP loading [92] and simulated in previous

Table 2
Mechanical characterization input parameters for the parametric analysis.

Expanded Blocks							
Elastic Behavior		CDP Parameters		Compressive Behavior		Tensile Behavior	
E_m [MPa]	4800	ψ [°]	10	f'_m [MPa]	6.35	f_{bt} [MPa]	1.83
ν_m [–]	0.16	ϵ [–]	0.1	ϵ_{mp} [–]	0.004	ϵ_{bt} [–]	0.004
Density [kg/m ³]	1708	f_{b0}/f_{c0} [–]	1.16	ϵ_{mk} [–]	0.016		
		ρ [–]	2/3				
Zero-Thickness Joints							
Overclosure Behavior		Tensile Cohesive Behavior		Shear Cohesive Behavior		Shear Frictional Behavior	
K_{no} [N/mm ³]	221	k_{nt} [N/mm ³]	221	k_s [N/mm ³]	87	$\tan \phi$ [–]	0.57
		f_t [MPa]	0.096	c [MPa]	0.11	δ_e [mm]	0.001
		u_k [mm]	0.3	δ_k [mm]	0.3		

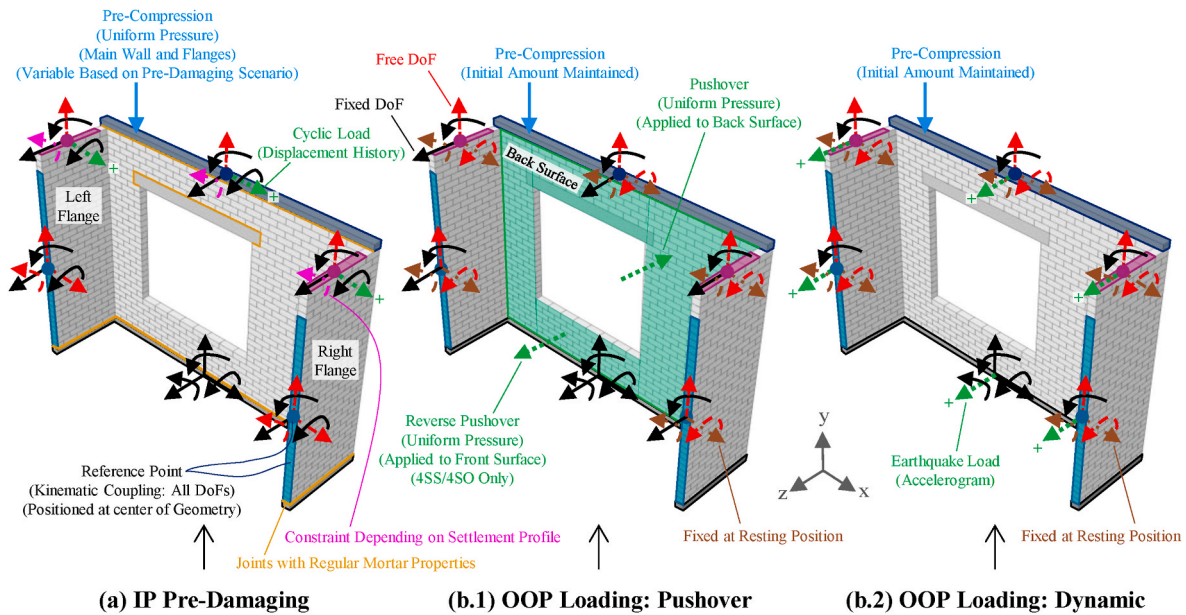


Fig. 7. Boundary conditions used in IP and OOP loading steps in the parametric analysis. The 4SO wall is shown as it represents the most complex geometry used in this study.

works by the authors [82,96]. This specimen also shares the same height as the others but has a longer length of 4107 mm. The long one-way spanning specimen is specifically included to capture a broader range of IP pre-damage patterns for walls loaded in one-way OOP bending: the short specimen is expected to be governed by flexural IP response regardless of the pre-damaging boundary conditions due to its length, whereas the IP response of the longer specimen can be shifted to a more shear-dominant one through changes in vertical overburden and pre-damaging boundary conditions [92]. It should be noted that all these geometries reflect real-world dimensions of walls from the residential URM building stock in the Groningen province of the Netherlands [108]. For brevity, an 'iSX' naming convention is used to refer to each specimen. The number of supported edges (i.e., 4 for two-way spanning walls or 2 for one-way spanning walls) is denoted by 'i,' and the presence of an opening on two-way spanning specimens (O for opening, S for solid) or the wall length in one-way spanning specimens (S for short, L for long) is indicated by 'X.'

Table 2 lists the input parameters for the mechanical behaviors of the expanded blocks and zero-thickness joints (shown in Fig. 1). These values are obtained by simulating the material characterization experiments of [92] on URM constructed with Calcium Silicate solid bricks and general-purpose cement-based mortar, typical constituents of low-rise buildings in the Groningen region [109]. Compressive and tensile behaviors of the expanded blocks are calibrated by modeling wallett compression and unit three-point bending tests, respectively. The overclosure response of the joints is also calibrated via the wallett compression simulation. Shear and tensile behaviors of the joints are calibrated by simulating triplet shear tests and wallett four-point bending tests, the latter being the test parallel to the bed joints. Consistent with the simulation in Section 3, values typical of URM materials are assigned to the CDP parameters and the dilatancy angle. Moreover, density and Poisson's ratio are taken directly from the experimental results in Ref. [92]. This calibration procedure, using simulated material tests to define input parameters for wall-level analyses, has been shown in previous studies [51–53] to enable reliable and accurate prediction of both IP and OOP responses under seismic loading. For simplicity, the same elastic properties assigned to the blocks are also applied to the base (all specimens), the top beam (4SS/4SO), and the lintel (4SO). Moreover, the finite element discretization of these regions follows that of the expanded blocks to ensure compatibility at the contact points.

Additionally, identical properties are used for the head and bed joints, as well as for the joints between the blocks and the base, the top beam, and the lintel. Finally, each pristine specimen is subjected to modal analysis under self-weight to identify the natural modes deemed most influential for its OOP response, and the natural frequencies associated with these modes are used to calibrate Rayleigh damping.

4.2. IP loading procedure

Fig. 7a shows the boundary conditions used during IP pre-damaging. Only the 4SO specimen is depicted, as it has the most complex geometry. The main walls are in the x-y plane, and the flanges extend in the positive direction of the z-axis. Following the simulations of one- and two-way spanning specimens in Refs. [52,53], several reference points are introduced at different locations to facilitate the application and transition of boundary conditions from IP pre-damaging to OOP loading (Fig. 7b). In the 4SS/4SO specimens, the top of each flange is given a reference point. Each point is kinematically coupled to, and located at, the geometric center of its entire corresponding flange cross-section, excluding the corner block at the wall-flange connection. The back of each flange, except the top three block rows, is also kinematically coupled to a reference point located at the geometric center of the coupled surface. Finally, the top beam in the 4SS/4SO specimens, and the top surface of the 2SS/2SL specimens, are each kinematically coupled to a reference point located at the mid-length and mid-thickness of the main wall, at the top edge of the top block row. The bottom boundary conditions are directly applied to the base parts without the use of reference points.

Before IP loading, gravity and vertical overburden are applied, the latter as a pressure uniformly applied over the entire top surface of the flanges and the main wall (i.e., the top of the top beam in the 4SS/4SO specimens, and the top surface in the 2SS/2SL specimens). Three overburden variations are considered, representing typical conditions in URM buildings in Groningen [92], in order to activate different damage patterns during IP loading. These include a low 0.1 MPa pre-compression (LPC) representing a wall supporting only the roof in a one-story building, a moderate 0.25 MPa pre-compression (MPC) simulating a ground-floor wall in a two-story building supporting the upper floor and other structural elements, and a high 0.5 MPa pre-compression (HPC) representing a ground-floor wall in a three-story

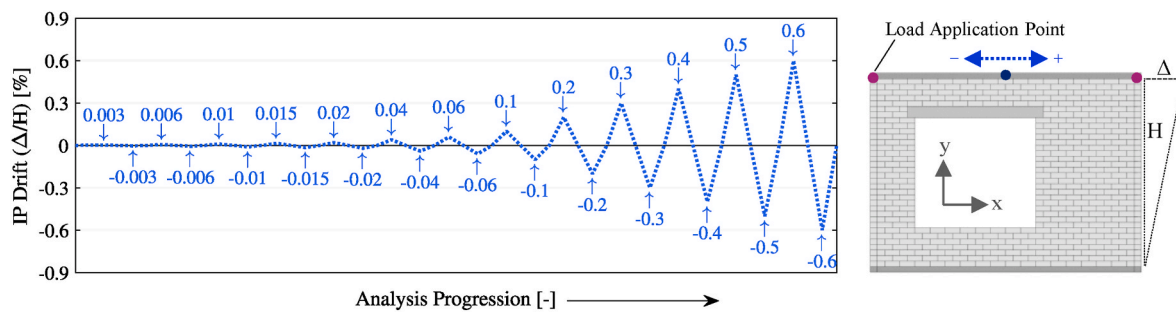


Fig. 8. Displacement history used in the IP loading step. The displacement is equally applied to all top reference points.

building. The HPC variation is an extreme scenario included specifically to activate potential crushing during IP deformation [107]. The use of such variations, grounded in the real-world conditions of URM walls, is preferred over the adoption of zero or excessively large vertical loads commonly used in previous studies on IP/OOP interaction effects [40]. The static solution procedure described in Section 2.3 is used for the gravity and vertical pre-compression steps, with loads applied monotonically up to the target values. During gravity and vertical loading steps, the base is fully fixed, while all DoFs of reference points are left free. During subsequent IP and OOP loadings, the level of pre-compression is maintained constant at its prescribed value.

IP loading is applied in a static analysis step and is imposed on all top reference points. During IP pre-damaging, all DoFs of the base are fixed, along with the rotation DoF about the y-axis, the OOP translational DoF along the z-axis, and the IP rotational DoF about the x-axis of the flange and top reference points. Uplift (along the y-axis) of all reference points, as well as IP translation (along the x-axis) and OOP rotation (about the z-axis) of the flange back reference points, are permitted. Two variations are considered for the OOP rotation (about the z-axis) of the top reference points. In the first, this rotation is allowed, simulating a cantilever (CT) boundary condition that enables flexural response. In the second, it is constrained, simulating a double-clamped (DC) boundary condition that enforces a shear-dominated response. These two variations are consistent with those used in physical IP testing of URM walls [92]. The cyclic displacement history shown in Fig. 8 is used for IP loading. It is adapted from experiments conducted in Ref. [92] on walls with similar geometries, boundary conditions, and material properties. Each cycle is symmetric in the positive and negative directions, meaning that the specimens are deformed to equal displacement levels in both directions. The target displacement of each cycle increases relative to the previous one, with no repetition of cycles at the same intensity. The final loading cycle reaches a peak top drift of 0.6% (calculated as the IP displacement divided by the 2754 mm height of the specimens), a level expected to reveal the full damage pattern and potentially induce collapse [92]. The IP loading is continued until either the final loading cycle is completed or the analysis diverges due to excessive nonlinearity (particularly crushing), whichever occurs first. It should be noted that the use of only one cycle per IP drift intensity, instead of multiple cycles per intensity as typically adopted in experiments [92], is done to reduce computational demand during the parametric analysis. Although, single-cycle-per-intensity loading protocols have also been used in previous experimental [97] and numerical [110] studies, without compromising the outcomes.

It should be noted that this study marks the first instance of IP pre-damaging of two-way spanning walls using cyclic loading. Previous studies on IP/OOP interaction effects applied simplified monotonic loading to two-way spanning walls [40], while cyclic pre-damaging has only been used for one-way spanning walls [39,41]. Although it should be noted that the kinematic coupling adopted at the back of the flanges, as well as the flange length used in this study, may not accurately represent actual structural behaviors. In real buildings, flanges are less restrained, and have a larger effective length participating in the

Table 3

Criteria used to identify IP seismic damage states.

Classification				C_w [113, 114]	Ψ [20]
State	Damage Class	Description	Repair [112]		
DS0	No Damage	–	–	–	–
DS1	Light ^a	First Visible Cracks	Easy	1–5 mm	2.5–3.5
DS2	Moderate	Many Visible Cracks	Medium	5–15 mm	3.5–4.5
DS3	Severe ^b	Full Crack Patterns	Extensive	15–25 mm	4.5–5.5
DS4	Near Collapse ^c	Large Deformations	Rebuild	>25 mm	>5.5

^a Also known as Slight or Negligible.

^b Also known as Heavy.

^c Also known as Very Severe or Very Heavy; not included in the parametric analysis.

deformation, allowing them to deform OOP during IP loading of the main wall and to develop more cracks [111]. Nevertheless, the current setup is maintained to simplify the application of boundary conditions, keep the OOP simulations consistent with the OOP dynamic experiments [93] against which the modeling approach has been calibrated, and to represent an extreme structural response scenario characterized by damage localization at the wall-flange connections due to limited flange deformability.

4.3. Pre-damage states: initial conditions for OOP loading

The OOP analyses of each specimen are not performed at the final stage of IP loading but instead at various levels of pre-damage. These levels, called hereafter pre-damage states, are selected from the IP response of each specimen based on the criteria provided in EMS-98 [112] and are listed in Table 3. It should be noted that EMS-98 is used here as a qualitative reference for defining ordered pre-damage states, while the actual identification of the pre-damage states is performed based on the scalar damage parameter Ψ , as described in the next paragraph. Five levels are defined in Ref. [112], each denoted as “DS_i,” where “i” ranges from 0 to 4 to indicate the severity of pre-damage. Each state is defined using qualitative descriptors and quantitative crack measures, such as a representative crack width (C_w). DS0 is the pristine (no pre-damage) condition. DS1 indicates light pre-damage, with C_w up to 5 mm and the appearance of the first visible cracks. DS2 represents moderate pre-damage, characterized by more cracked regions and C_w up to 15 mm. DS3 reflects a severe pre-damage state, where the full crack pattern has developed and C_w reaches up to 25 mm. DS4 denotes very severe (or near-collapse) pre-damage condition, with large deformations and even higher C_w . In this study, DS4 is not considered in order to maintain the practical relevance of the outcomes: structures exhibiting this DS4 state are deemed unsafe and are expected to undergo intervention prior to exposure to further seismic events [113–115].

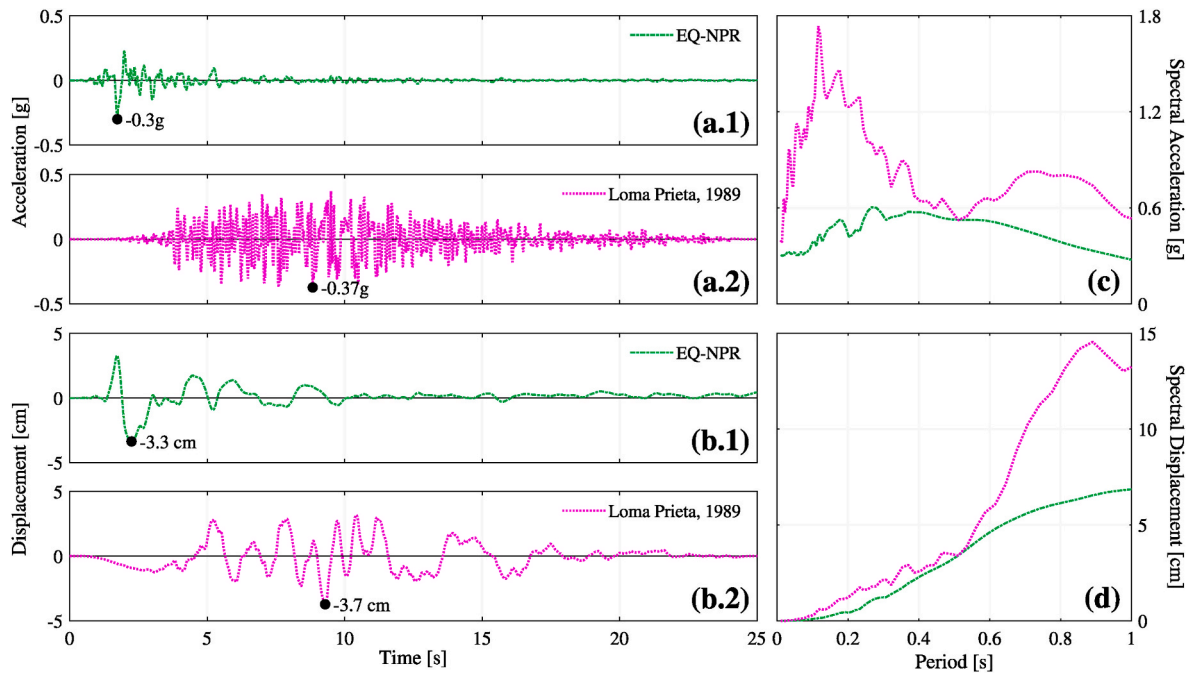


Fig. 9. Dynamic loading signals used in the parametric analysis: acceleration time histories (a) and resultant displacements (b), and 5%-damped acceleration (c) and displacement spectra (d) of the induced-seismicity [108,117] (1) and tectonic-seismicity [118] (2) signals.

The scalar damage parameter (Ψ) proposed in Ref. [105] is used to identify the pre-damage states. The use of the Ψ parameter provides an objective methodology in which the level of damage is estimated based not only on crack width but also on crack frequency and length. It is preferred over traditional approaches that rely solely on a representative crack width [116], as it is considered to estimate more realistically the severity of damage. Specifically, traditional methods may under-estimate damage in specimens with small-width but long cracks that penetrate through the body, and over-estimate it in specimens with localized damage consisting of short cracks with large widths [116]. The Ψ parameter is calculated via Equation (1), incorporating the number (n), the length ($C_{L,i}$ in mm), and the width ($C_{W,i}$ in mm) of cracks, with each crack assigned a numeral identifier “i.” The values of $C_{W,i}$ include both shear and normal components of the crack width.

$$\Psi = 2n^{0.15} \left(\frac{\sum_{i=1}^n C_{L,i} C_{W,i}^2}{\sum_{i=1}^n C_{L,i} C_{W,i}} \right)^{0.3} \quad (\text{Eq. 1})$$

Each pre-damage state is identified by tracking crack developments during IP loading, calculating the Ψ parameter at the peak deformation of each loading cycle, and comparing the calculated value with the ranges recommended in Ref. [20] and listed in Table 3. The final state of the loading cycle (see Section 4.4.5.1) for which the calculated Ψ falls within the range corresponding to a specific pre-damage state is saved and used as the initial condition for subsequent OOP loadings. Specimens with different levels of pre-damage are also subjected to modal analyses in order to investigate the effects of IP loading on their natural modes and frequencies.

4.4. OOP analysis procedure

Before the application of OOP loading and following IP pre-damaging, the specimens are given a relaxation period via a static analysis step. During this relaxation phase, the IP load is removed, allowing the specimens to return to a zero-lateral-reaction (or zero-residual-stress) condition. Afterward, the boundary conditions are updated to those corresponding to the OOP loading phase, as shown in

Fig. 7b, for the subsequent analysis steps. Based on the classification proposed in Ref. [17], the interaction scenario investigated here, the influence of seismic IP pre-damage on the dynamic OOP responses in non-framed URM walls during earthquakes, constitutes a direct IP/OOP interaction under sequential loading, referred to as “SQ (IP, OOP)” for short. Specifically, the inclusion of the relaxation period between the IP and OOP loading stages aligns with this classification.

Two types of OOP analyses are performed. The first type is conducted in the static regime by applying a monotonically increasing uniform pressure to the surface of the main wall, as shown in Fig. 7b.1. Due to the asymmetric OOP response of the 4SS/4SO specimens, two sets of OOP pushover analyses are carried out: one pushing the main wall in the weak direction (away from the flanges, i.e., in the negative z-direction), and the other in the strong direction (towards the flanges, i.e., in the positive z-direction). The second set is called “reversed pushover” hereafter. These static analyses are conducted to assess their applicability for predicting true dynamic responses, offering a more computationally efficient alternative to dynamic loading. The static solver introduced in Section 2.3 is used for the OOP pushover steps. During pushover, the base remains fixed, while uplift (along the y-axis) and IP rotation (about the x-axis) are allowed at the different reference points. All other DoFs at the reference points are fixed at their final positions from the relaxation step. These boundary conditions are consistent with those used in the one- and two-way spanning walls tested in Refs. [93, 95] and simulated by the authors in Refs. [52,53]. Free IP rotation at the top simulates a connection to flexible diaphragms, representing the weakest connection scenario expected to yield the lowest OOP resistance [54]. It should be noted that this OOP loading setup and boundary conditions neglect the effect of the floor slab on the OOP stiffness and strength. Hence, future studies are suggested to consider specimens in which the floor slab is explicitly simulated to achieve more realistic approximations of real-world responses. For simplicity, the direction of applied pressure is kept perpendicular to the surface of each block to which it is applied. Collapse during the pushover analyses is defined as the point at which the structure fails to maintain static equilibrium, either due to large deformations or physical detachment and falling of wall segments.

The second type of dynamic simulation involves two sets of incre-

mental dynamic analyses (IDAs) on each specimen. Each set uses one of the dynamic loading signals shown in Fig. 9 and is performed via the HHT- α solver introduced in Section 2.3. The first signal represents induced earthquakes in the Groningen province of Netherlands. This accelerogram, is taken from Refs. [108,117] and is called EQ-NPR therein, which reports a shake table experiment conducted on a full-scale building typical of the region. The signal is specifically selected because one of the objectives of the current study is to estimate the effects of pre-damage on the seismic OOP response of the URM building stock in Groningen. Moreover, it has a relatively large peak motion acceleration (PMA) compared to other available induced ground motions [95], which allows it to be scaled to even higher intensities for the purposes of the IDA conducted in the current study. The second signal belongs to the 1989 Loma Prieta tectonic earthquake in Central California (USA) [119], and is collected from the NGA-West2 database [118] (record number 811). This signal is included in the parametric analysis to examine the influence of the inherent differences between induced and tectonic seismic hazards on IP/OOP interaction effects. Moreover, it has been specifically selected because of its relatively large PMA, which supports its scaling in the IDA, as well as its discrepancies with EQ-NPR, which allow revealing the largest differences between pre-damage effects under induced and tectonic earthquakes. Although both signals have comparable PMAs, as shown in Fig. 9a, the tectonic signal has a longer significant duration and contains multiple acceleration peaks. Additionally, it exhibits larger spectral accelerations (Fig. 9c). The signals are used without post-processing, as they have already been cleaned in the source references. Representing ground motions at the base of the building, the signals are applied to the base and all reference points of the numerical specimens, with their positive direction aligned with the positive z-axis (Fig. 7b.2). The remaining boundary conditions match those used in the static pushover analyses: uplift and IP rotation are allowed at all reference points, while the remaining DoFs are fixed at their final positions from the relaxation period. With the current direction of dynamic loading, the PMAs of both signals act toward the weak OOP direction of the 4SS/4SO specimens, which is expected to maximize their sensitivity to IP/OOP interaction effects. A $\Delta t_{\max} = 0.0025$ s is used in the IDAs and is equal to the sampling interval of both loading signals. It should be noted that other loading signals have not been covered in the present study to maintain feasibility in terms of computational demand and time. However, this decision may limit the generalizability of the outcomes of the parametric analysis, as record-to-record variability is not considered.

The procedure by which the IDAs are performed is shown in Fig. 5. Each pre-damaged or pristine specimen is initially subjected to the dynamic loading signal scaled to a PMA of 0.75g. Based on the outcome of the simulation, collapse or no-collapse, one of the following steps is followed:

- In the case of collapse, the loading PMA is reduced and a new analysis is conducted on the pristine specimen. This process is repeated iteratively until a no-collapse condition is reached. The lowest PMA causing collapse is then identified as the collapse PMA of the specimen.
- In the case of no-collapse, a new analysis of the specimen, restarting from the initial pre-damage state, is conducted with a higher PMA. This process continues until the specimen collapses. The PMA used in the final analysis that results in collapse is recorded as the collapse PMA.

The PMA values follow the predefined lists shown in Fig. 5. For simulations under induced seismicity, a maximum PMA limit of 0.75g is imposed to keep the study practical: Real-world induced earthquakes generally exhibit low amplitudes, with intensities rarely exceeding 0.5g [120,121]. As a result, further increases of the PMA for the induced signal, although eventually causing collapse, does not yield observations that are applicable in practice. Therefore, if collapse is not reached for a

specimen under the already high-intensity induced signal with PMA of 0.75g, the IDA is halted and a no-collapse outcome is reported.

While IDAs typically define collapse as the failure to maintain equilibrium due to physical falling of wall segments or very large deformations [54], this study adopts a more practical numerical collapse criterion: dynamic collapse is declared when the peak OOP deformation observed in a simulation exceeds 25% of the wall thickness ($t = 102$ mm). This criterion allows a more practical interpretation of the outcomes: in some simulations, specimens may exhibit severe damage or large displacements yet still maintain equilibrium, leading to unrealistic outcomes such as excessively large numerical deformations or artificial stability. These behaviors may be the result of overdamping or the absence of element deletion mechanisms [53]. The 25% limit is not intended to represent a universal physical collapse displacement threshold. It is adapted from the studies of the authors on one- and two-way bending walls and gables [52,53,55] and corresponds to deformation levels that initiate significant nonlinearity in the OOP response in dynamic OOP experiments of walls [93,107] with similar geometries, material properties, and loading conditions to the specimens considered herein. Moreover, it reflects real-world practice, where walls with such large OOP deformations are classified as severely damaged and considered in need of immediate intervention, such as repair or strengthening [122–125]. The threshold, validated in a blind prediction competition participated in by the authors [56], serves as a filter for distinguishing collapsed and non-collapsed numerical responses, enabling a more accurate estimation of actual collapse occurrences. It should be noted that since the OOP deformation capacity of URM walls is governed by several parameters, such as height-to-thickness slenderness ratio, top constraints, and pre-compression [126–130], depending on the considered wall configuration, actual OOP collapse may occur at deformation levels other than 25% of the wall thickness. Such discrepancy can influence the absolute values of the PMA at which collapse is detected. Nevertheless, since the same criterion is consistently applied to all specimens and pre-damage states within the parametric analysis, the comparative trends discussed in this study regarding the effects of pre-damage remain valid.

Only specimens under DS0, DS1, and DS3 pre-damage states are considered for IDA; specimens at DS2 are not included in order to keep the study feasible in terms of time and computational demand. Additionally, specimens subjected to HPC loading are excluded, as they are deemed to be in extreme pre-damage conditions, such as significant block crushing or cracking, which would require repair before being exposed to subsequent earthquakes and are therefore not representative of real-world conditions. A limitation of the current IDA setup should be acknowledged: the use of equal boundary motions across all reference points does not fully replicate real-world conditions, where dynamic motions vary spatially due to the filtering effects of the structural response of the building the walls belong to Ref. [131]. However, this simplification is deemed acceptable for the current study, as all specimens are assumed to be on the ground floor, where such differences are typically smaller than those at upper stories. That said, future work is required to extend the investigation to upper-floor walls, incorporating the effects of relative support motions for a more comprehensive understanding of dynamic response.

5. Results and discussion

The results of each step of the parametric analysis are presented and discussed in this section. The structural response range under IP loading and the variations in damage progression among different specimens is explored in Section 5.1. The modal analysis results for both pristine and pre-damaged specimens are covered in 5.2 and are interpreted in terms of the effects of IP pre-damage on OOP stiffness. The static pushover OOP analysis results and comparisons of peak OOP strength and failure modes between pre-damaged and pristine specimens are detailed in Section 5.3. Finally, the dynamic OOP analysis outcomes and the

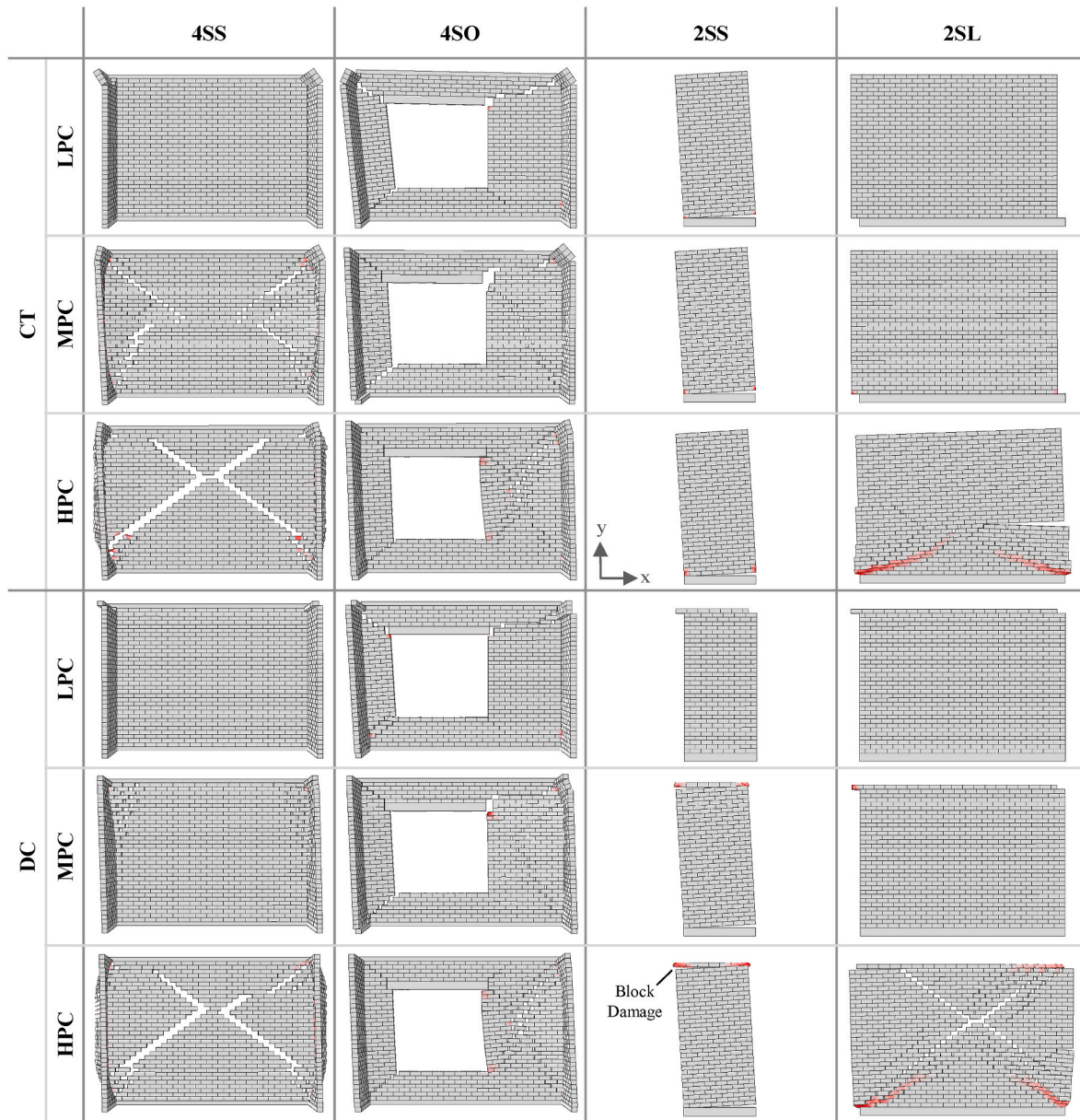


Fig. 10. Final crack patterns under IP loading. Damage in blocks (i.e., crushing or tensile cracking) is highlighted in red. Deformed shapes are magnified with different scales for clear visualization of crack paths. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

comparison between specimen responses under dynamic and static loading are discussed in Section 5.4.

5.1. In-plane loading: damage patterns and pre-damage states

Fig. 10 shows the final damage patterns obtained under different IP pre-damaging scenarios. The deformed shapes correspond to the final moments of the IP loading analyses, i.e., either at the end of the loading cycles or at the point of numerical divergence. The lowest sensitivity to IP loading conditions is observed in the 2SS specimen as, due to its large height-to-length aspect ratio ($h/l = 1.9$), it deforms primarily in flexure and shows base cracking. Although, more corner crushing and additional top cracking are observed in this wall when DC boundary conditions are used, and also shear sliding is observed under DC-LPC loading. A similar dominance of specific failure mechanisms is observed in the 2SL and 4SO specimens. In the 2SL specimen, the lower aspect ratio ($h/l = 0.67$) promotes a shear failure mechanism across all loading

scenarios, primarily characterized by shear sliding at the base or top of the wall. Exceptions occur under CT-HPC loading, where a flexural response with diagonal cracking and toe crushing develops, and under DC-HPC loading, where shear failure manifests as diagonal cracking. The responses observed in the 2SS and 2SL specimens are consistent with IP experimental results for walls with similar geometries and materials reported in Ref. [92]. In the 4SO specimen, the presence of the opening significantly influences crack patterns across all scenarios. Under MPC and HPC loading, damage includes cracks originating from the window corners and extending to the wall corners, along with diagonal cracks in the long pier. Under LPC loading, similar damage occurs, except for the absence of diagonal cracks in the long pier, which are replaced by two horizontal cracks at the top and bottom of the pier. The 4SS specimen, although primarily showing shear response due to the presence of flanges, exhibits more varied damage patterns. Under LPC loading, shear sliding occurs at the top of the wall. Under HPC loading, diagonal cracks develop. Under MPC loading, diagonal cracking and

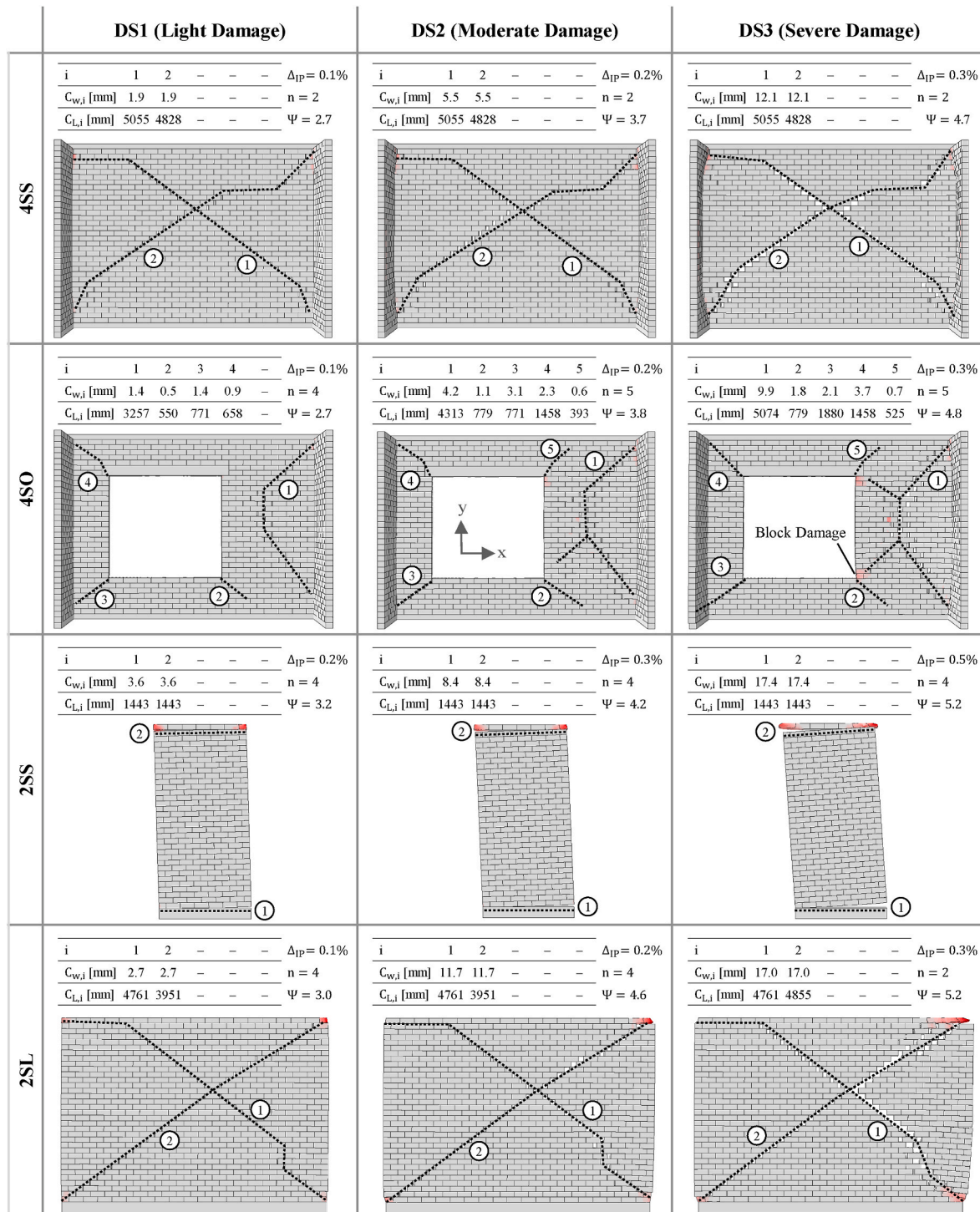


Fig. 11. Example of damage state identification for specimens under DC-HPC loading. Deformed shapes are $\times 20$ magnified. Damage in blocks (i.e., crushing or tensile cracking in units) is highlighted in red. The Δ_{IP} is the value of peak drift in the loading cycle at which the damage state is identified. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

minor base cracking are observed with CT boundary conditions, while damage concentrates at the wall-flange intersections under DC boundary conditions. In both the 4SS and 4SO specimens, cracking is also observed at the top and base of the flanges due to their OOP deformation and damage propagation from the main wall. Overall, a wide range of responses are captured during IP pre-damageing.

Examples of the pre-damage identification procedure (see Section 4.3), applied to the specimens under DC-HPC loading, are shown in Fig. 11. Crack identifiers are kept consistent across different pre-damage

states for clarity. All cracks, including those in the base joint, flanges, and main wall, are considered in the calculations and four simplifying (and conservative) assumptions are applied. First, vertical cracks at the back ends of the flanges (not completely visible in Fig. 11 since only back views are presented) are excluded, as they are assumed to be artificial artifacts caused by the fixed vertical rotation (about the y-axis) used in this study, and not representative of real-world damage. Second, cracks that originate from the same location but branch in different directions are treated as a single crack; examples can be seen in the long

Table 4
IP drifts corresponding to light-to severe-damage states of the specimens.

Pre-Compression	IP Drift (Displacement/Height) [%]											
	4SS			4SO			2SS			2SL		
	DS1	DS2	DS3	DS1	DS2	DS3	DS1	DS2	DS3	DS1	DS2	DS3
CT-LPC	0.04	0.3	0.5	0.06	0.2	0.4	0.10	0.3	0.5	0.04	0.3	0.5
CT-MPC	0.06	0.3	0.5	0.06	0.2	0.4	0.10	0.3	0.5	0.10	0.3	0.5
CT-HPC	0.20	0.3	0.4	0.06	0.1	0.2	0.10	0.3	0.5	0.20	0.3	0.4
DC-LPC	0.04	0.3	0.5	0.06	0.2	0.4	0.06	0.3	0.5	0.04	0.3	0.5
DC-MPC	0.10	0.3	0.5	0.10	0.2	0.4	0.10	0.3	0.5	0.06	0.3	0.5
DC-HPC	0.10	0.2	0.3	0.10	0.2	0.3	0.20	0.3	0.5	0.10	0.2	0.3

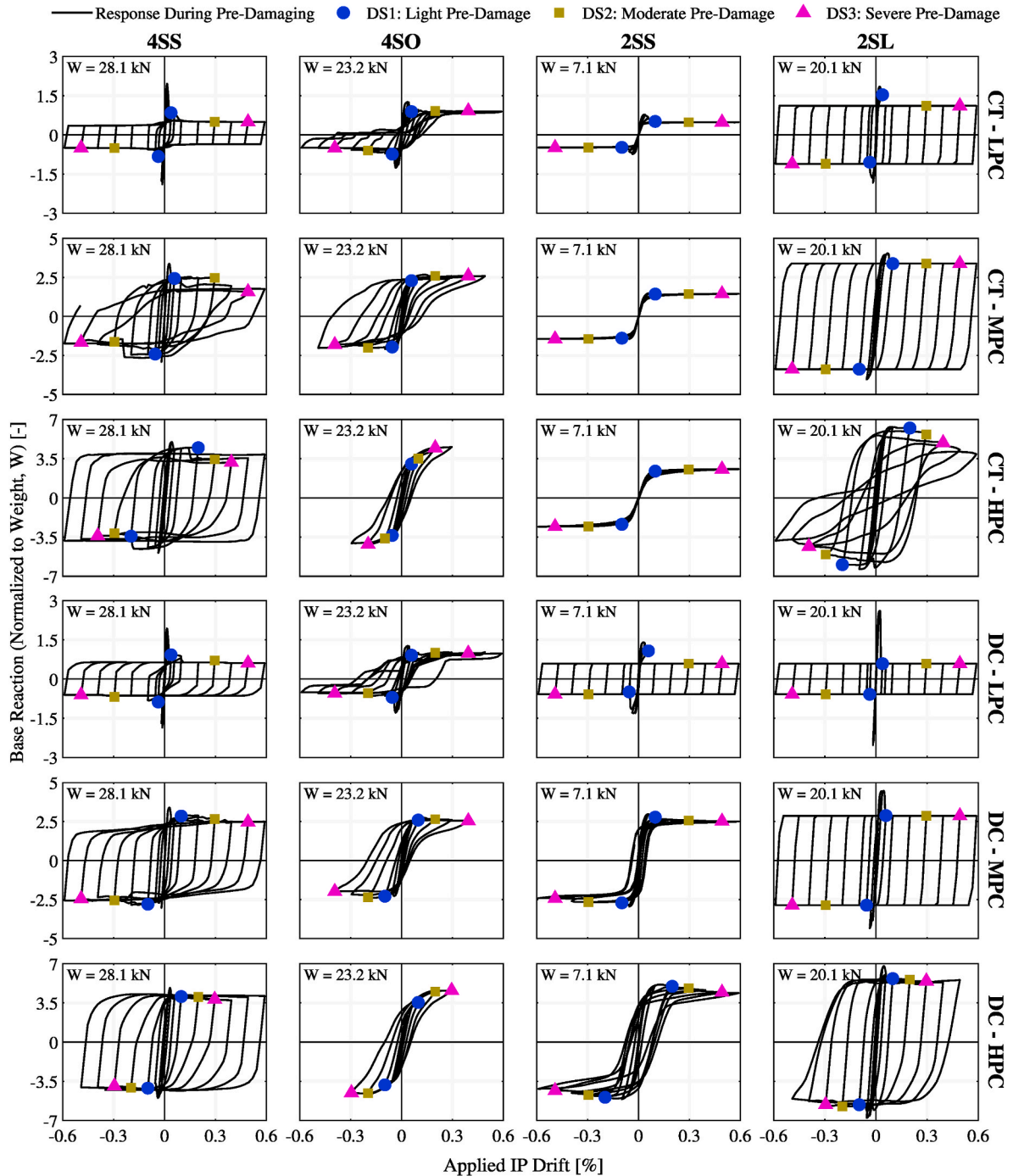


Fig. 12. Load-displacement response of different specimens during IP pre-damaging. The response of each specimen is normalized to its weight, W, for easier comparison. W is the weight of each specimen excluding the top beam and the base, and is 28.08 kN for 4SS, 23.16 kN for 4SO, 7.1 kN for 2SS, and 20.1 kN for 2SL.

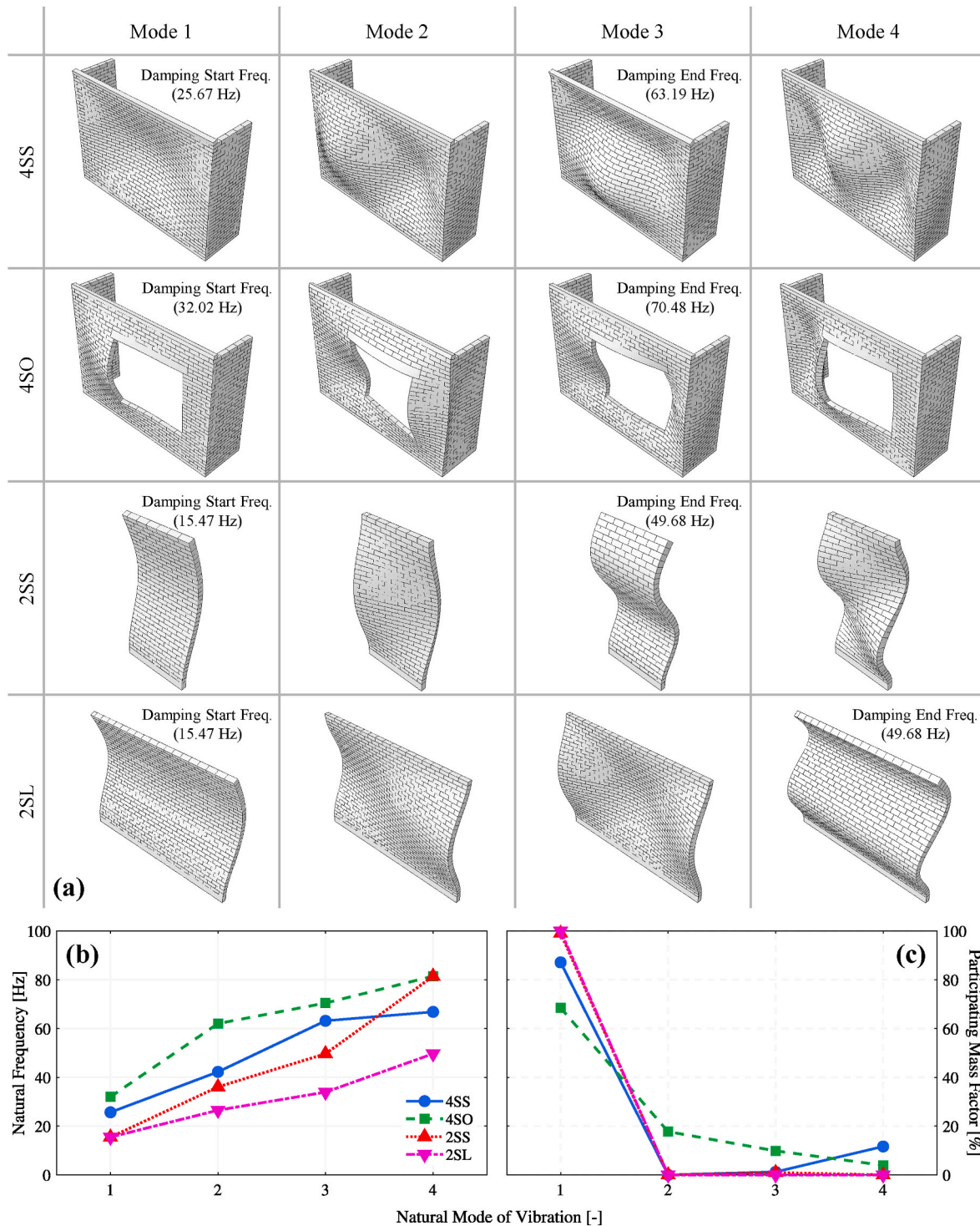


Fig. 13. Modal analysis results for pristine numerical specimens: mode shapes (a), natural frequencies (b), and participating mass factors (c) of the first four modes of vibration. Rayleigh damping is calibrated via the frequencies shown in the deformed shapes.

pier of the 4SO specimen. Third, cracks which change direction during propagation, such as the diagonal cracks in the 4SS specimen, are also considered as one. Fourth, the maximum width observed along the length of a crack is used as $C_{w,i}$. The versatility of the Ψ parameter compared to traditional approaches is demonstrated in DS3 of the 4SS specimen and DS2/DS3 of the 4SO specimen. In these cases, although the maximum width of any individual crack does not fall within the C_w range associated with the identified damage state, the collective frequency and severity of cracking already satisfy the qualitative criteria.

In other words, a traditional approach relying on a single C_w value would have, for example, identified DS3 for these walls only at higher IP drift levels.

Table 4 lists the IP drift ratios at which different pre-damage states are identified in the IP response of the specimens. Across all specimens, similar pre-damage states are reached at relatively similar drifts: DS1 is reached at 0.04–0.1% IP drift, DS2 is mostly reached at 0.2–0.3%, and DS3 is identified at 0.3–0.5%. These ranges were previously identified in Ref. [132] and align well with experimental results for walls of similar

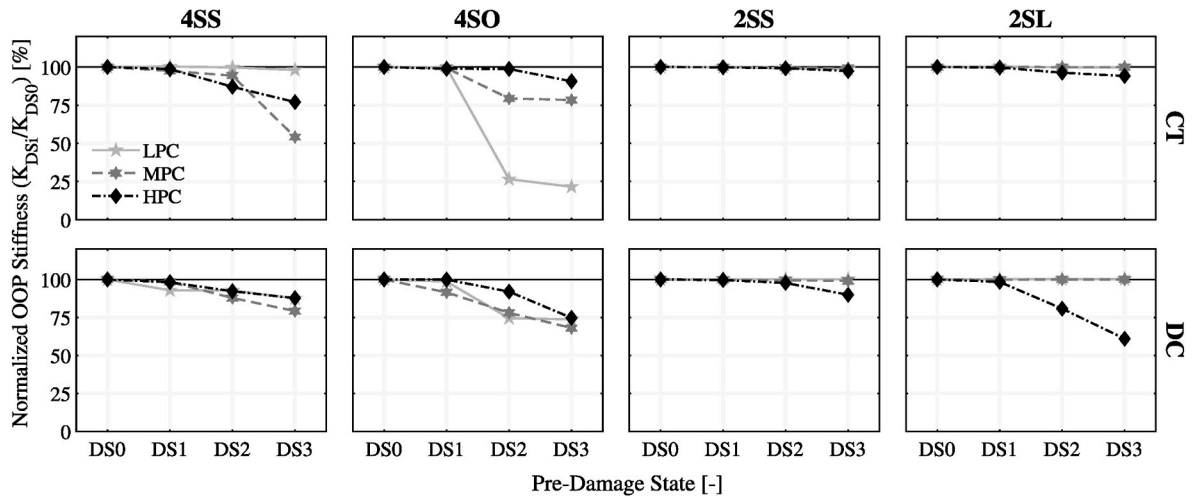


Fig. 14. Variations in OOP stiffness of specimens under different IP seismic pre-damage. The square of the ratio of first-mode frequency in pre-damaged walls to that of the pristine walls is shown.

geometry and material properties [92], which report drifts of 0.2–0.45% for DC specimens and up to 0.55% for CT specimens in their final loading cycles. However, some discrepancies are observed. First, the 2SS specimen under DC-HPC loading reaches DS1 only at a relatively large 0.2% drift. This is attributed to the combined effect of the DC and HPC boundary conditions. In early cycles, the HPC condition promotes a shear-dominated response with compressive toe damage, limiting crack openings typically associated with flexural response. As the loading progresses, the response transitions to flexure, and large cracks develop only at higher IP drifts. Second, both the 4SS and 2SL specimens reach DS3 under HPC loading at lower drifts compared to other loading scenarios, primarily due to expedited crushing mechanisms triggered by the higher pre-compression. Third, the 4SO specimen reaches DS2 and DS3 at lower drift levels than the other specimens. This is due to the presence of the opening, which leads to earlier localization and evolution of damage.

The IP drift levels at which different damage states are identified are superimposed on the force-displacement response of each specimen in Fig. 12. The curves within each subfigure, representing the response of the specimens under different IP loading conditions, are normalized to the weight of each specimen (W). The values of W exclude the top beam and the base, and are 28.08 kN for the 4SS specimen, 23.16 kN for the 4SO specimen, 7.1 kN for the 2SS specimen, and 20.1 kN for the 2SL specimen. The points of the response in the positive and negative loading regimes corresponding to the observation of different damage states are highlighted with different markers. DS1 is generally observed to occur at or shortly after the achievement of peak load, except for the 4SO specimens subjected to HPC loading, where it appears within the post-elastic branch before softening. DS2 typically corresponds to a point within the residual tail of the envelope of the load-deformation curves, while DS3 is identified in the final one or two loading cycles, beyond which the specimens collapse (defined as excessive crushing in HPC-loaded specimens and physical instability of others). The early attainment of DS3 in the 4SO specimen under HPC loading is noted, which corresponds to its earlier collapse relative to the other specimens, due to crushing failure. The identification of DS1, although consistent with the Ψ parameter thresholds [20], does not fully align with its qualitative description: at the point where DS1 is identified, the complete crack pattern is nearly formed, whereas DS1 is associated with only minor cracks (see Table 3). Two factors can explain this mismatch: the characteristics of the specimens studied, and the resolution of the damage states adopted. Specifically, the specimens exhibit brittle joint response and rapid softening progression, for which a more accurate identification of DS1 would require a denser sequence of loading cycles

at intermediate IP drift levels not included in this study. Additionally, the use and identification of intermediate light pre-damage states, such as negligible, very Slight, and slight [16], may have better satisfied the qualitative criteria for DS1. Nonetheless, the damage states identified here are considered valid and appropriate for the objectives of this study for three reasons. First, the focus is not specifically on DS1, but rather on the evolution of interaction effects up to DS3. Second, the DS1 levels identified provide a conservative estimate of the interaction effects under light pre-damage. Third, the current DS1 also reflects the real-world conditions of buildings where exposure to environmental effects already reduces cohesion in the joints and leads to rapid crack opening during subsequent actions [133,134].

5.2. Modal analyses: effects of pre-damage on stiffness

The outcome of modal analyses in specimens with no pre-damage are presented in Fig. 13. For each specimen, modal shapes, natural frequencies, and participating mass factors for the first four natural vibration modes are illustrated. Modal analyses are performed using the boundary conditions shown in Fig. 7b.1 and used for static OOP analyses. In all specimens, the majority of the participating mass (Fig. 13c) is concentrated in the first three modes, except for the 4SS specimen, where the fourth mode also contributes by 12% of the total mass. Additionally, modes beyond the third (except in the 2SL specimen) correspond to higher-order triple-curvature (in 4SS) or torsional (in 4SO and 2SS) modes, deemed not to significantly influence the OOP response of the considered specimens. In the 2SL specimen, the increased wall length introduces a horizontal triple-curvature deformation in the third mode and shifts the vertical double-curvature mode to the fourth mode. Accordingly, the natural frequencies of the first and third modes are used for Rayleigh damping calibration in the 4SS, 4SO, and 2SS specimens, while the first and fourth modes are used for the 2SL specimen, consistent with the approach adopted in previous works of authors [52, 53]. A simplification in the current study should be noted: damping in all specimens, pristine or pre-damaged, is calibrated using the modal frequencies of the non-damaged configurations, without accounting for the stiffness degradation and potential damping changes introduced by pre-damage. While this simplification facilitates more straightforward dynamic analyses and avoids uncertainties associated with specimen-specific damping calibration, it may also lead to overdamping in the OOP response of pre-damaged specimens. This issue should be addressed in future studies. It should also be noted that the lowest first-mode period among the specimens ($T = 1/32.02$ s, in 4SO) results in a $\Delta t_{\max}/T = 8\%$ in the dynamic analyses, which ensures that damping

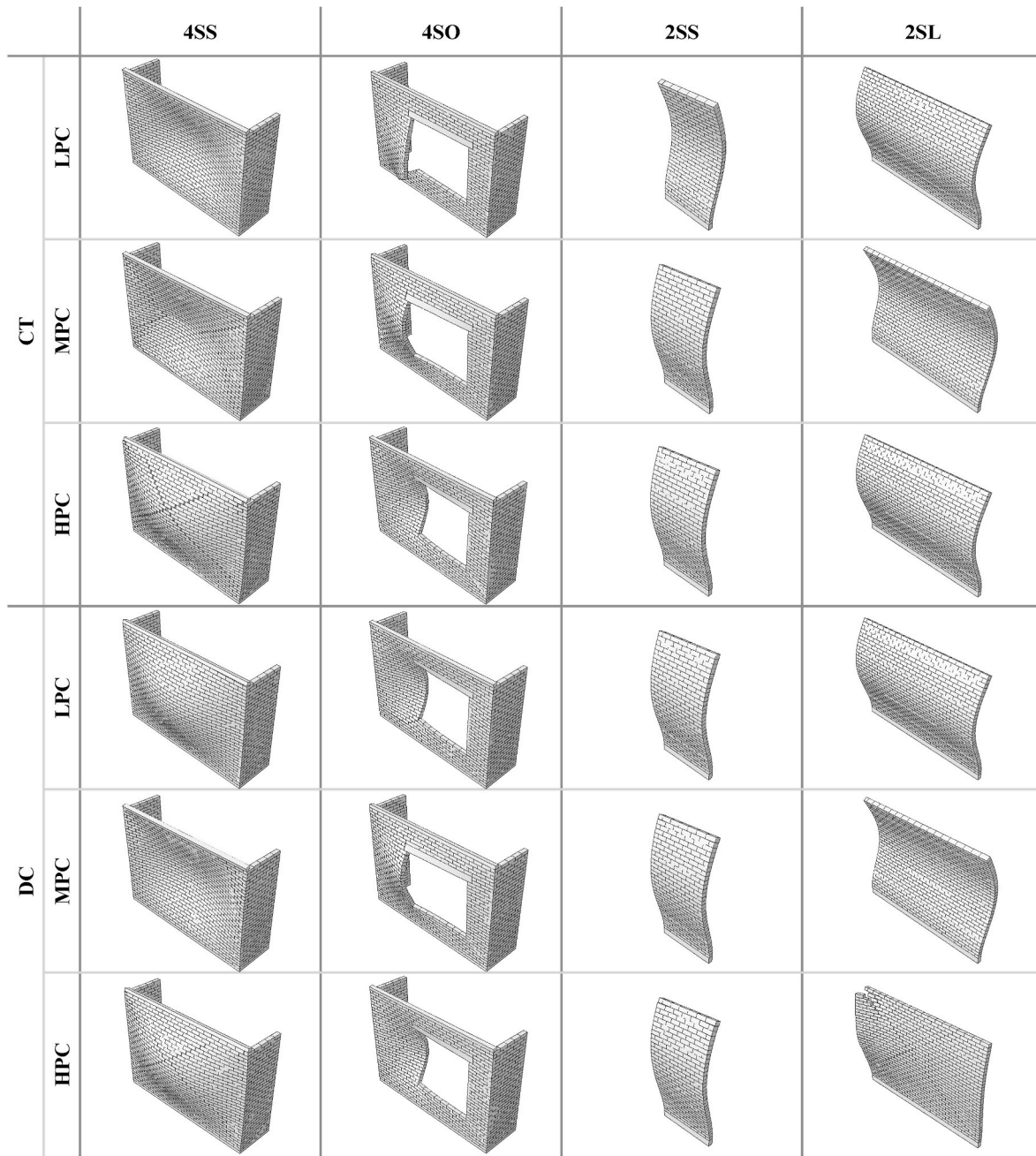


Fig. 15. Modal shapes of first natural deformation mode in specimens at DS3 seismic IP pre-damage state.

is not artificially introduced through the HHT- α solver (see Fig. 2a).

Based on the results of modal analyses on pre-damaged specimens and idealizing each specimen as a single-degree-of-freedom system where the fundamental relationship $f = \sqrt{K/m}$ holds, Equation (2) is used to calculate the change in OOP stiffness of each specimen (K_{DSi}) at different pre-damage states (DSi), based on the change in their first-mode frequency (f_{DSi}), relative to their pristine properties (K_{DS0} and f_{DS0}). Since the mass of each specimen (m_i) remains constant during all analyses, the use of Equation (2) is considered valid.

$$\frac{K_{DSi}}{K_{DS0}} = \left(\frac{f_{DSi}}{f_{DS0}} \right)^2 \quad (\text{Eq. 2})$$

Fig. 14 presents the results of the calculations, comparing the OOP stiffness of pre-damaged specimens with their pristine stiffness under various pre-damage scenarios and states. Moreover, Fig. 15 shows the

modal shape of the first natural deformation modes of the specimens at DS3 pre-damage state. Different sensitivities are observed, with pre-damage causing stiffness reductions between 0 and 32% at DS3, and even as high as 79% in one specimen (4SO under CT-LPC loading). The 2SS and 2SL specimens exhibit minimal changes in stiffness, particularly under LPC and MPC conditions. This is attributed to their IP loading predominantly causing loss of cohesion at a single horizontal joint without significantly altering their vertical confinement. Only the 2SL specimen under CT-HPC loading and the 2SS specimen under DC-HPC loading show minor stiffness drops (up to 10%) at DS3, due to the appearance of crushing. The most significant sensitivity among the one-way spanning specimens is observed in the 2SL specimen under DC-HPC loading, where the loosening of the triangular portion between the left diagonal cracks leads to a 40% stiffness reduction at DS3. The 4SS and 4SO specimens demonstrate higher sensitivity, largely due to the separation of their flanges from the base. However, most stiffness drops

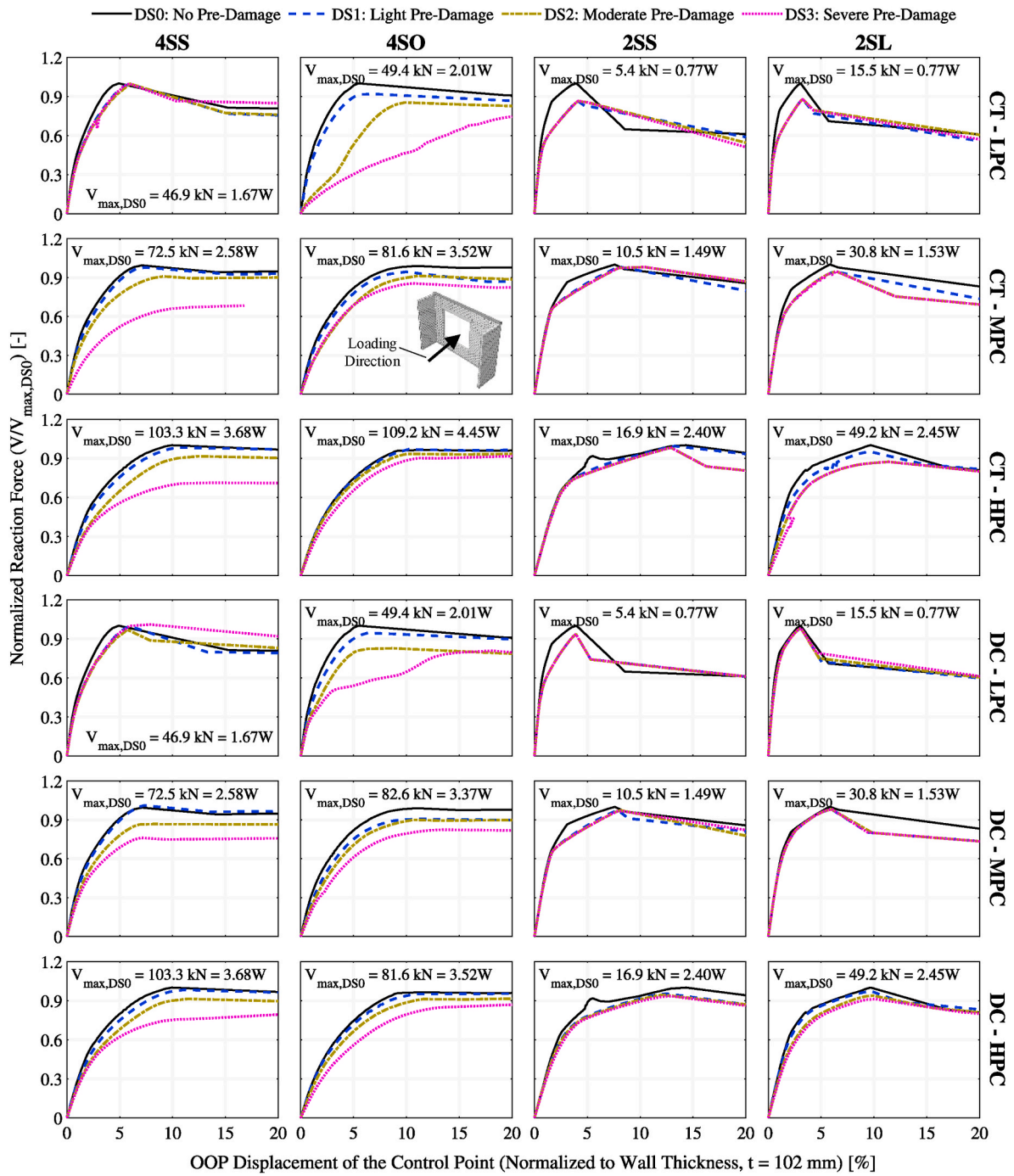


Fig. 16. Force-displacement curves from static pushover OOP analyses of specimens under IP seismic pre-damage. W is weight of the specimens, 28.08 kN for 4SS, 23.16 kN for 4SO, 7.1 kN for 2SS, and 20.1 kN for 2SL.

remain within 25%. A notable exception is the 4SO specimen under CT-LPC loading, which experiences a particularly large stiffness drop (75% already at DS2), caused by horizontal cracks originating from the window corners that result in the loosening of the long pier. The 4SS specimen under CT-MPC pre-damaging also shows a significant 46% reduction in stiffness, primarily due to the additional crack formations at the base of the main wall compared to other pre-damage scenarios. Overall, these observations indicate that seismic pre-damage can lead to OOP stiffness reductions of up to 25–30%, with even higher reductions when wall segments become loosened due to the progressive extension of cracks within the wall. The results further reveal that most changes in OOP stiffness develop at DS2 and continue into DS3, suggesting that DS1

pre-damage has only a minor influence on the OOP stiffness.

5.3. Pushover analyses: effect of pre-damage on the static OOP performance

This section presents the outcomes of the static pushover and reversed pushover OOP analyses, along with a discussion of the findings. Figs. 16 and 17 show the results in terms of OOP force-displacement curves under pushover and reversed pushover loading, respectively. The response of each specimen is presented in a column of subfigures, with each row corresponding to a different pre-damaging scenario. The curves within each subfigure, representing the response at different pre-

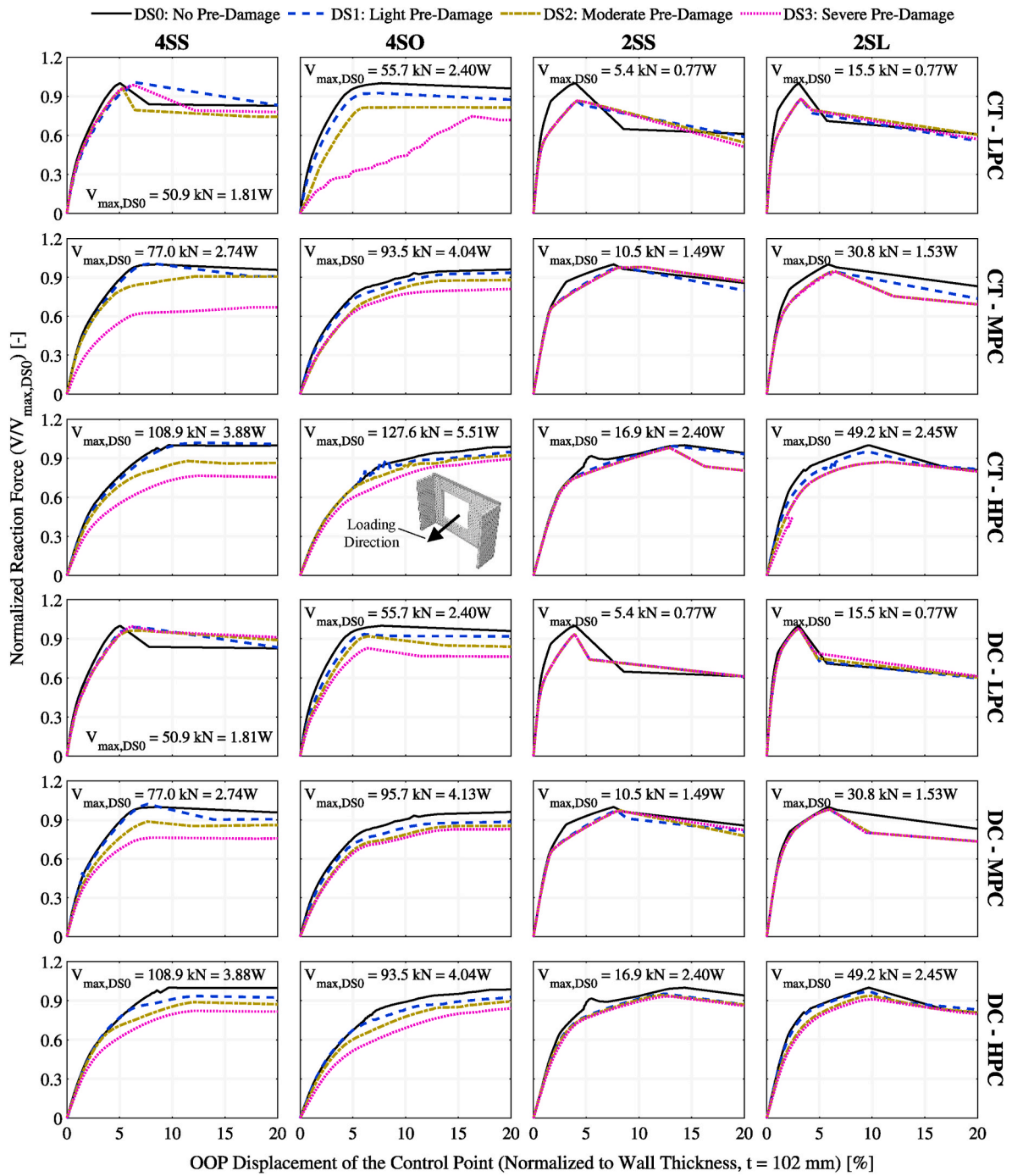


Fig. 17. Force-displacement curves from static reversed pushover OOP analyses of specimens under IP seismic pre-damage. W is weight of the specimens, 28.08 kN for 4SS, 23.16 kN for 4SO, 7.1 kN for 2SS, and 20.1 kN for 2SL. Although 2SS and 2SL specimens are not subjected to reversed pushover, their results from the regular pushover analyses are repeated in this figure for ease of comparison with the 4SS and 4SO specimens.

damage states, are normalized to the peak strength of the corresponding pristine variation ($V_{\max,DS0}$), which is indicated in the subfigure as a fraction of the specimen weights, W. Displacements are normalized to the thickness of the specimens ($t = 102$ mm for all specimens). Fig. 18 summarizes the changes in OOP strength due to pre-damage, expressed as a percentage of the strength of the corresponding pristine specimen. Failure mechanisms of DS0 specimens are shown in Fig. 19, while those for the pre-damaged DS3 specimens, under both pushover and reversed pushover, are depicted in Figs. 20 and 21, respectively. The displacement control points used to generate the pushover curves are also indicated in the figures and remain consistent across all analyses. These

include the center of the main wall for the 4SS specimen, the mid-height point on the window edge adjacent to the long pier in the 4SO specimen, and mid-height mid-length points in the 2SS and 2SL specimens. In the visualized deformed shapes, scaled with different factors for clarity, damage in blocks (representing crushing and tensile cracking of units) is highlighted in red. Although the 2SS and 2SL specimens were not subjected to reversed pushover analysis, the results of their response during the regular pushovers are shown in Figs. 17 and 21 as well, for accessibility.

The changes in static OOP strength show the same trends as those identified in the effects of pre-damage on OOP stiffness. Strength loss

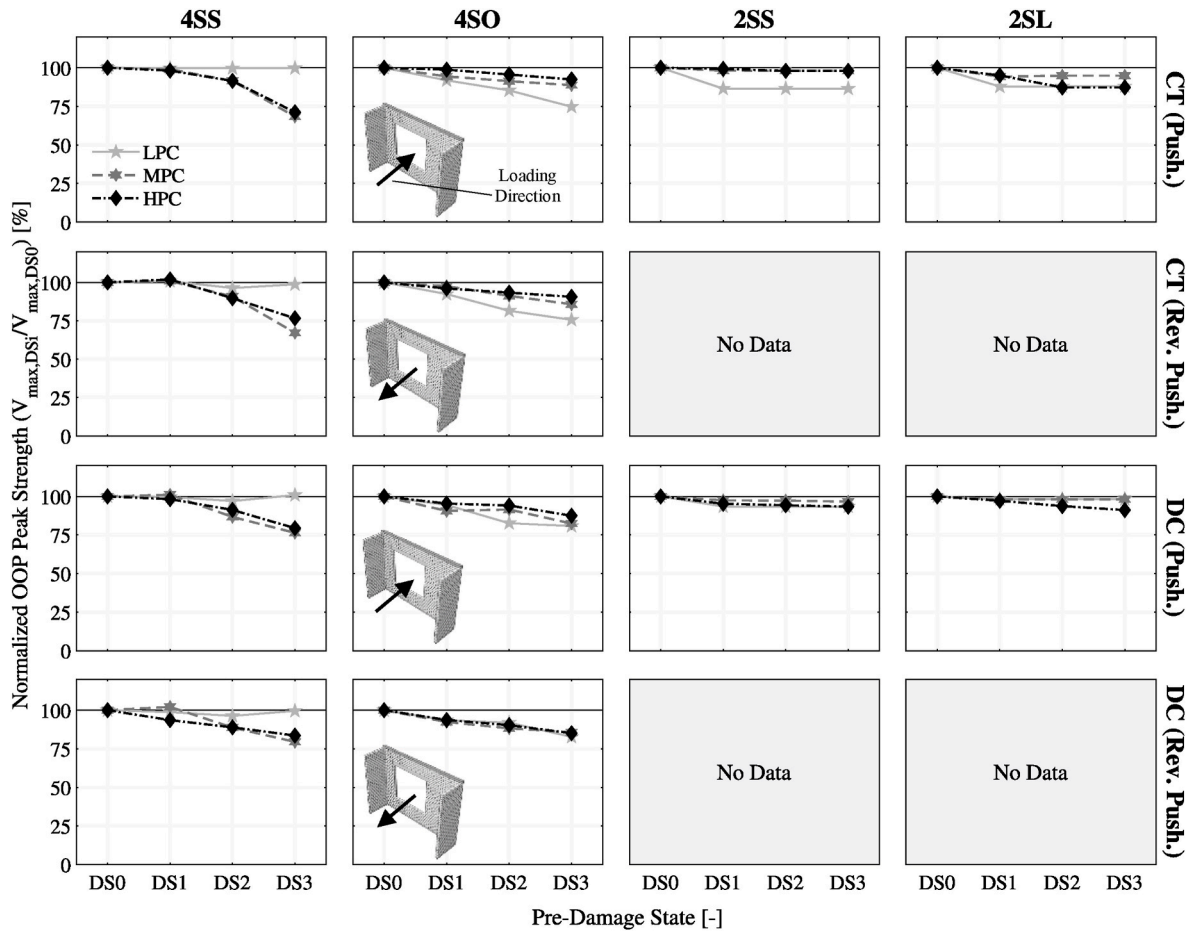


Fig. 18. Variations in static pushover OOP strength of specimens as a result of IP seismic pre-damage. Peak strengths are normalized to those of pristine specimens.

predominantly emerges for specimens pre-damaged at DS2 level and fully develops for specimens pre-damaged at DS3 level. An exception is observed in the LPC-loaded 2SS specimen, which shows earlier strength degradation when pre-damaged at DS1 level. In this case, the flexural crack at the base fully forms at DS1, with no additional cracking at the subsequent pre-damage states. Overall, the pre-damaged strength of the 2SS and 2SL specimens changes only marginally, up to 14% when pre-damaged at DS3 level, due to the concentration of damage in these walls being limited to the top or base joints. Previous reviews on IP/OOP interaction effects have indicated that OOP performance degradation becomes prominent only when the pre-damage caused by IP loading spatially overlaps with the damage caused by subsequent OOP action [17]. In the 2SS and 2SL specimens, although top and base cracks overlap with the typical cracking from one-way OOP bending, the pre-damage does not affect the wall mid-height, where the main crack responsible for OOP collapse forms. The 4SS and 4SO specimens exhibit greater sensitivity, up to a 33% strength reduction when pre-damaged at DS3 level. The LPC-loaded 4SS specimen shows negligible strength drop, while its 4SO counterpart experiences the most reduction under LPC loading (up to 25% when pre-damaged at DS3 level), primarily due to the looseness of the long pier. The largest strength reductions occur in the 4SS specimen under MPC and LPC scenarios, where the strongest overlap of IP and OOP cracking patterns is observed. In particular, the CT-MPC variation exhibits base cracking that reduces the force required for collapse by 33% when pre-damaged at DS3 level. These comparative observations are also reflected in the failure mechanisms of the pre-damaged specimens. Most retain collapse mechanisms similar to their pristine counterparts, with the most prominent deviations occurring in the LPC-loaded 4SO specimen. In summary, the results suggest

that seismic IP pre-damage can reduce the OOP strength of two-way spanning walls by up to 33%, with walls with opening under LPC and solid walls under MPC/HPC conditions being more affected. In contrast, one-way spanning walls show only minor strength reductions, limited to 14%.

Design and assessment guidelines for non-framed URM walls currently disregard the effects of seismic pre-damage, and no analytical formulations exist specifically addressing pre-damage in non-framed URM walls [17]. Therefore, the results of this study are compared against the analytical equation proposed in Refs. [135,136] for one-way spanning framed (infilled) URM strips, which has already been incorporated into existing guidelines [137]. While the current study shows no significant reduction in the OOP strength of one-way spanning walls, the equation in Refs. [135,136] predicts a 25% strength loss even when walls are pre-damaged at DS1 level. Additionally, although the two-way spanning walls in this study show strength losses of up to 30% when walls are pre-damaged at DS3 level, the analytical equation predicts a 60% reduction, an overly conservative estimate. The existing literature on IP/OOP interaction in non-framed URM walls predominantly focuses on the effects of IP pre-deformation rather than pre-damage [31,138,139], the distinction being that specimens are held at peak IP deformation during OOP loading in the former [17]. These studies also tend to significantly overestimate the effects of similar pre-deformation levels compared to the pre-damage states examined in this work, often predicting complete loss of OOP strength. Although the limited number of pre-damage and loading scenarios examined in this study precludes the development of generalized predictive equations, the results clearly indicate that existing analytical approaches are overly conservative. However, it should also be noted that, until further validated through

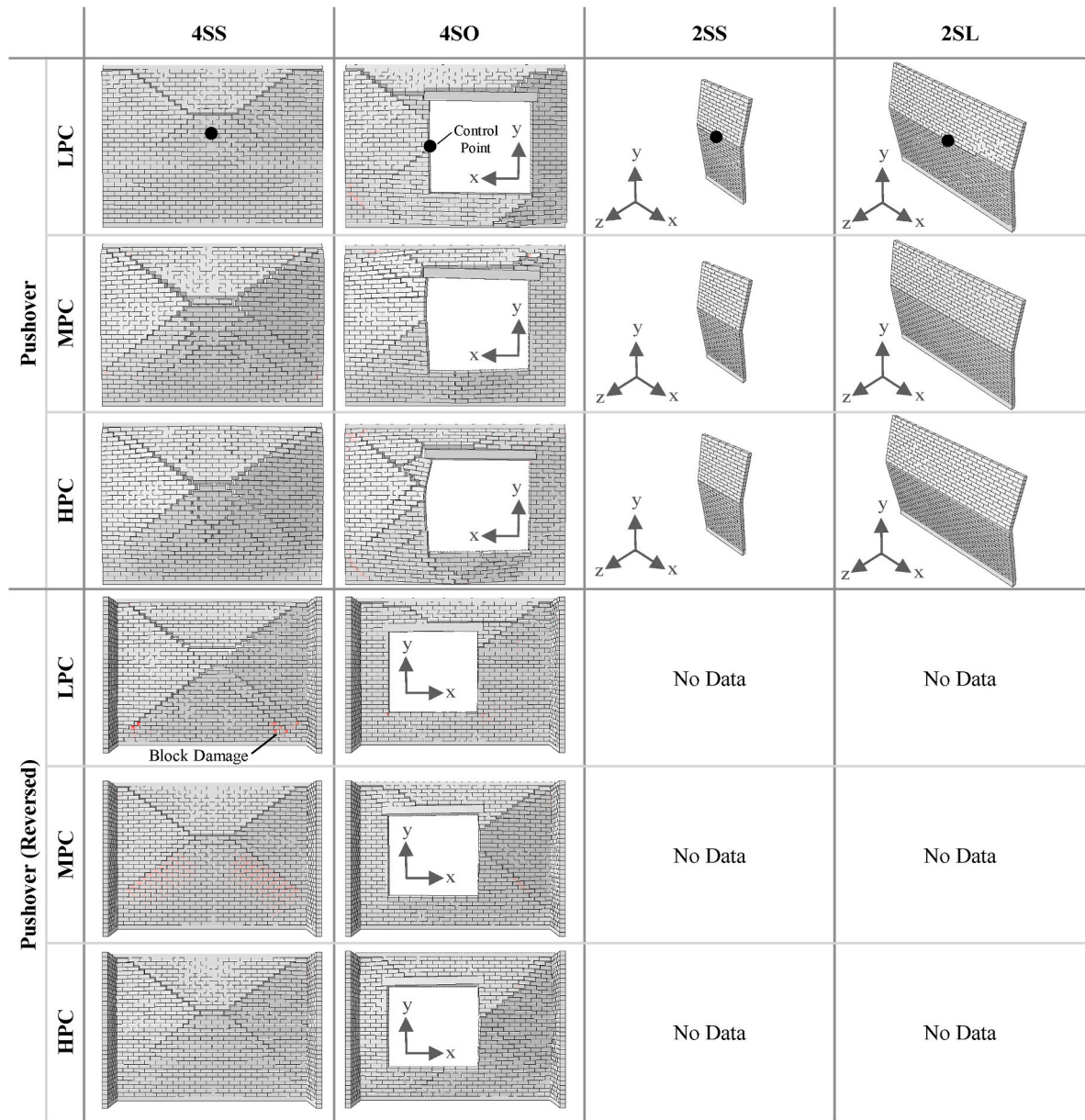


Fig. 19. Damage patterns from static pushover OOP analyses of pristine specimens under (no pre-damage). Block damage (crushing or tensile cracking of units) is highlighted in red. Deformed shapes are scaled with various factors for clear visualization. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

experiments or more rigorous statistical and probabilistic analyses, the comparisons presented herein, especially the extent of the difference between the interaction effects estimated in the current study and those predicted by analytical equations, should not be treated as generalizable, and should only be regarded as indicators of the need for further investigation into IP/OOP interaction effects in non-framed URM walls.

5.4. Dynamic analyses: comparison with static pushover analysis results

This section presents and discusses the results of the incremental dynamic analyses. Under the induced seismicity loading signal and the intensities considered in this study (scaled up to a PMA equal to 0.75g), most specimens did not experience collapse. Not only that, almost no new damage or considerable OOP deformation was observed under induced seismicity. Only the 2SS and 2SL specimens with pre-damage at DS1 and DS3 levels collapsed at a PMA of 0.75g, exhibiting the same failure mechanisms previously observed in the static analyses, while heir

pristine DS0 counterparts did not collapse. These results suggest that the reduction of wall capacity determined by IP pre-damage may not be sufficient to provoke structural OOP collapse under realistic intensities of induced seismicity. The rest of the section focuses on the outcomes of the IDAs under tectonic seismicity. Table 5 presents the collapse PMAs, and Fig. 22 illustrates the dynamic failure mechanisms. Collapse is monitored at the same control points used for the static pushover curves, as shown in Fig. 22. It should be noted that while the collapse of 4SS/4SO is identified by the exceedance of the 25% deformation threshold described in Section 4.4, the 2SS/2SL specimens exhibited collapse in the form of falling of wall portions. Additionally, in the 4SS and 4SO specimens, not only was the 25% threshold exceeded during the analyses, but the deformations greatly increased, reaching as much as 50% of the wall thickness, confirming that collapse was correctly detected. All specimens demonstrate similar failure patterns between pristine and pre-damaged states, consistent with the static analyses. A reduction in collapse PMA is observed only at DS3, and only in the two-way spanning

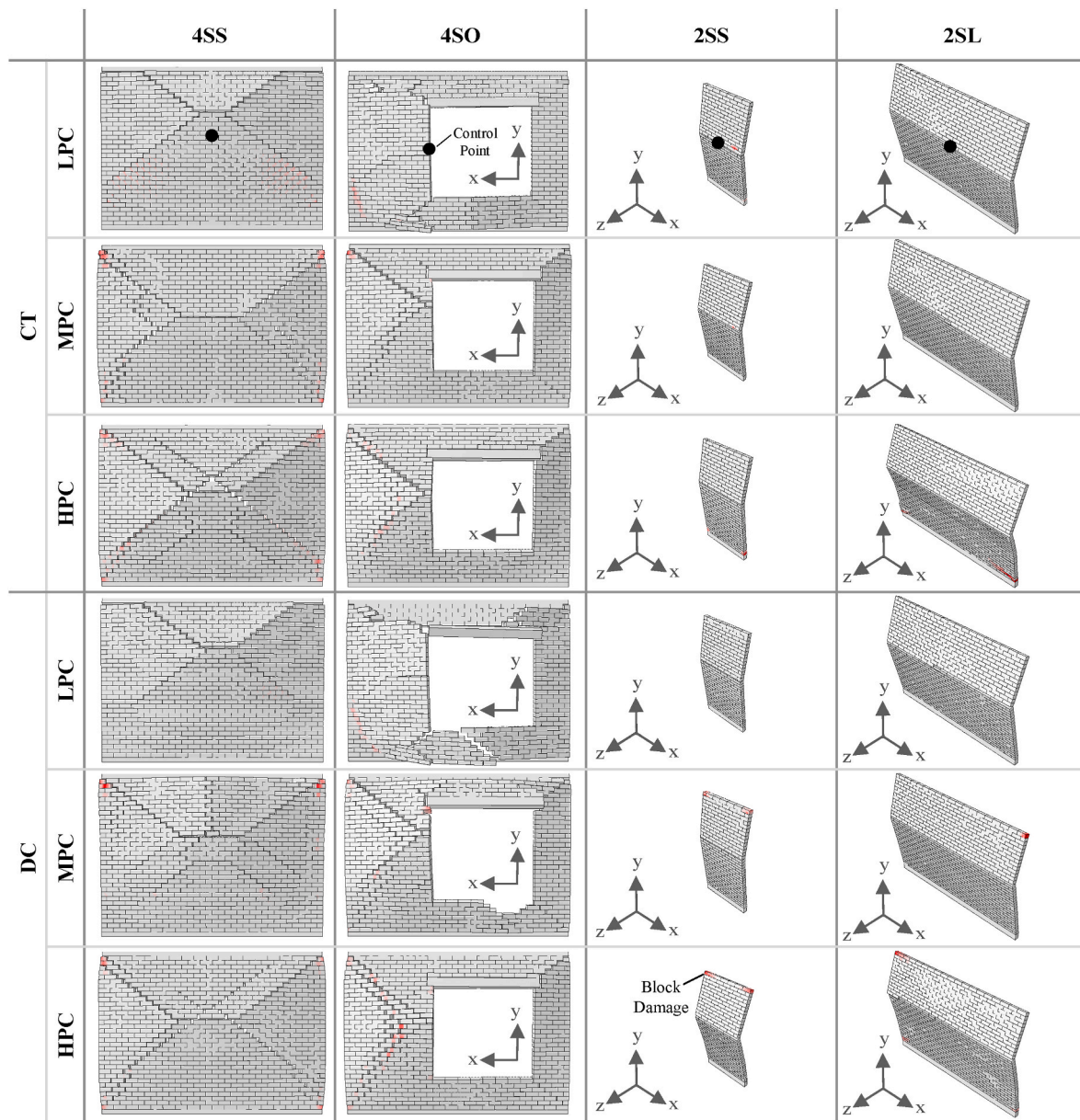


Fig. 20. Damage patterns from static pushover OOP analyses of specimens under DS3 IP seismic pre-damage. Block damage (crushing or tensile cracking of units) is highlighted in red. Deformed shapes are scaled with various factors for clear visualization. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

specimens previously identified as the most sensitive in the static pushover analyses. These findings reinforce the conclusions drawn from static loading and are not further elaborated here for brevity. Instead, the focus shifts to comparing the performance of the static and dynamic analysis procedures.

The variations in OOP strength estimated from static analyses are compared with those obtained from the IDAs in Fig. 23. The dynamic results are presented as a range of potential collapse PMAs, normalized to the collapse PMA of the corresponding pristine specimen. This approach is used to address a limitation of the current parametric analysis: IDAs are conducted using discrete loading intensities, and no further refinement is performed to precisely pinpoint the collapse PMAs [54]. As such, the actual collapse PMA is assumed to fall within the range between the recorded collapse PMA and the immediately lower loading intensity in the IDA list of PMAs. The shaded range shown in Fig. 23 reflects this assumption. Further refinement is also considered for the 2SL/2SS specimens to achieve a more accurate estimation of

dynamic collapse intensity: an additional dynamic simulation is conducted with a loading intensity below the collapse PMA by 0.125g. Since none of the specimens collapsed, the lower-bound of the ranges in Fig. 23 reflects the PMA of the new analyses.

Strong agreement is observed between the IP/OOP interaction effects captured by the static and dynamic analyses. In all specimens, the normalized strength of the pre-damaged specimens either lies within the dynamically expected range, or coincides with the upper or lower bounds of that range. Furthermore, the failure mechanisms observed in the static analyses, for both pristine and pre-damaged specimens, match closely those yielded from dynamic simulations. These observations show that, for the specimens studied, static pushover analyses reliably estimate both the failure mechanisms and the strength reductions associated with IP/OOP interaction. Additionally, static analyses accurately capture the pristine OOP strength of the 4SS and 4SO specimens, while they slightly overestimate the strength of the 2SS and 2SL specimens. This discrepancy is attributed to the force-controlled nature of

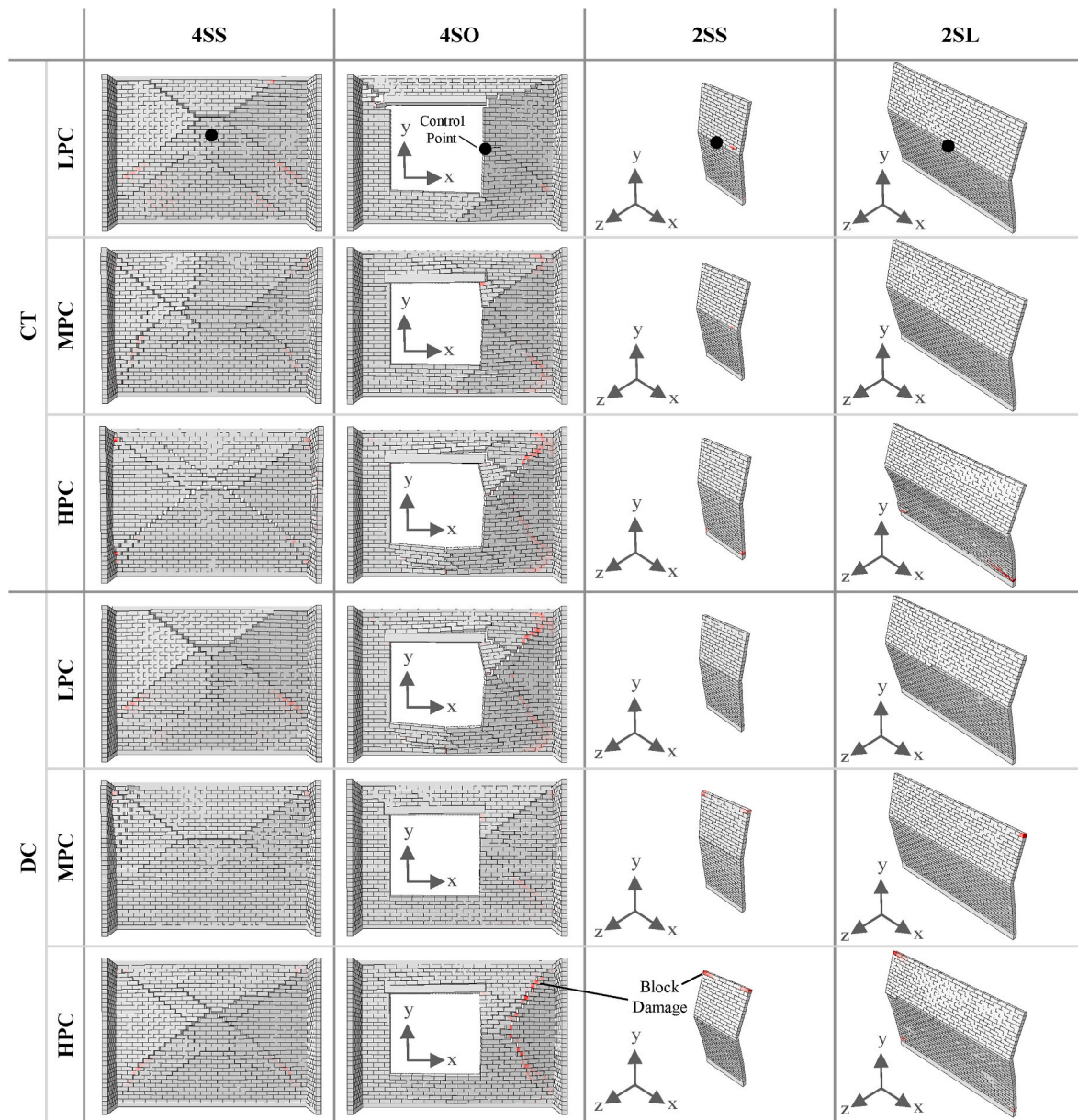


Fig. 21. Damage patterns from static reversed pushover OOP analyses of specimens under DS3 IP seismic pre-damage. Block damage (crushing or tensile cracking of units) is highlighted in red. Deformed shapes are scaled with various factors for clear visualization. Although 2SS and 2SL specimens are not subjected to reversed pushover, their results from the regular pushover analyses are repeated in this figure for ease of comparison with the 4SS and 4SO specimens. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

Table 5

Collapse PMAs of specimens under tectonic-seismicity IDA.

Pre-Damage Scenario	Collapse PMA ^a [g]											
	4SS			4SO			2SS			2SL		
	DS0	DS1	DS3	DS0	DS1	DS3	DS0	DS1	DS3	DS0	DS1	DS3
CT-LPC	2.0	2.0	2.0	2.5	2.5	2.0	0.75	0.75	0.75	0.75	0.75	0.75
CT-MPC	3.0	3.0	2.5	4.0	4.0	3.5	1.25	1.25	1.25	1.25	1.25	1.25
DC-LPC	2.0	2.0	2.0	2.5	2.5	2.0	0.75	0.75	0.75	0.75	0.75	0.75
DC-MPC	3.0	3.0	2.5	4.0	4.0	3.5	1.25	1.25	1.25	1.25	1.25	1.25

^a Peak motion acceleration causing physical collapse or OOP deformations exceeding 25% of thickness.

OOP response in 4SS/4SO specimens [140], compared to the displacement-controlled response in 2SS/2SL [141]. Specifically, collapse in two-way spanning walls occurs when the applied force, arising from inertia under dynamic loading or from pressure under static

loading, exceeds the wall strength, after which the walls show brittle failures involving shear and torsional cracks [140], a behavior that can be successfully captured by static analyses. By contrast, the response of one-way spanning walls is governed not only by the applied load but

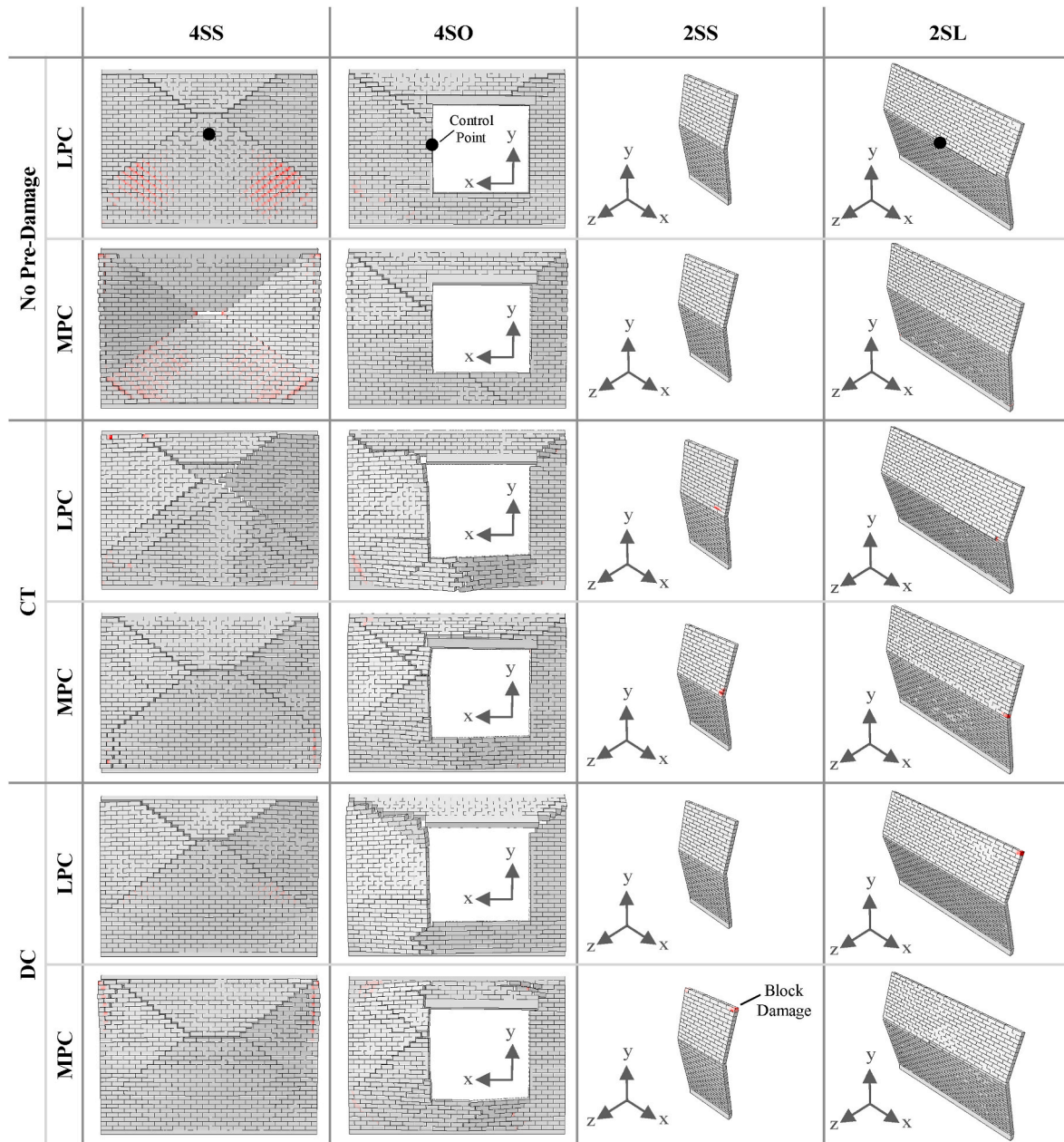


Fig. 22. Damage patterns from IDA of specimens under DS3 seismic IP pre-damage and tectonic seismicity. Block damage (crushing or tensile cracking of units) is highlighted in red. Deformed shapes are scaled with various factors for clear visualization. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

also by the magnitude of the resulting deformation, which leads to collapse once a critical instability threshold is reached after cracking at critical regions [141]. Because static analyses apply the load gradually and neglect inertial effects, they may have delayed the attainment of this threshold, thus predicting collapse at higher-than-necessary loading magnitudes. Despite these differences, static simulations show good accuracy in predicting IP/OOP interaction effects in the pre-damaged configurations of both one- and two-way spanning walls, in the former case particularly because the one-way spanning walls (2SS/2SL) exhibit low sensitivity to pre-damage and maintain almost their entire strength due to the lack of overlap between their primary IP and OOP failure paths.

A lower computational demand of static analyses compared to dynamic ones is noted in this study, increasing their practical advantage. For instance, each loading run in the tectonic IDAs for the MPC-loaded 4SS specimens at DS0 takes as similarly long as its static analysis (3.3

vs 2.8 h on 16 CPU cores @ 4.2 GHz clock speed), leading to the full IDAs of the specimens, which include nine analyses with PMAs starting from 0.75g and increasing to 4g, to require about 970% more time (roughly 30 h) than the static simulations. Although, this gap is considerably smaller for specimens with lower dynamic collapse PMAs, such as the LPC-loaded 2SS specimens with DS3 pre-damage, which collapse during their first dynamic loading run at a PMA of 0.75g.

It should be noted that the results of this study may not be generalizable to a wider range of scenarios due to certain limitations. Specifically, the outcomes presented here may be influenced by the specific geometry and material properties of the specimens, the characteristics and direction of the loading signals, and the amount of damping used in the analyses. Regarding damping, the uncertainties associated with selecting an appropriate target damping ratio could not be explored in this study due to time constraints. Additionally, the specimens investigated are not based on an experimental benchmark from which a more

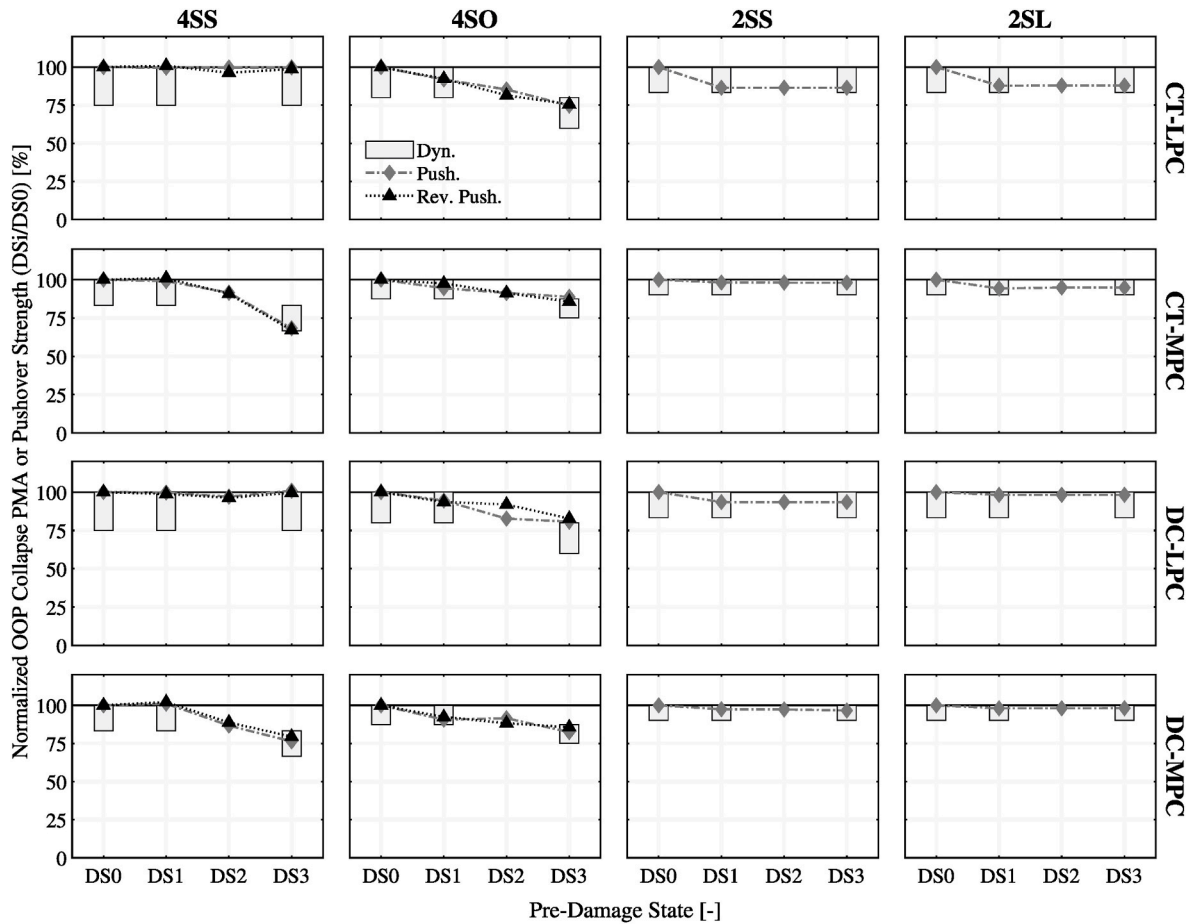


Fig. 23. Comparison of strength drops predicated via dynamic (tectonic-seismicity) and pushover analyses. For 4SS/4SO specimens, dynamic analysis results are shown as a range of expected collapse PMAs between the numerically obtained value and one loading intensity below it. For 2SS/2SL specimens, the lower loading intensity is from additional dynamic analyses with a 0.125g lower PMA than that of collapse.

accurate damping ratio could be derived. Although a 5% damping ratio has previously demonstrated suitability in capturing the dynamic OOP response of both one- and two-way spanning walls [52,53], lower damping levels are expected to lead to earlier collapse [56]. Nevertheless, the magnitude of IP/OOP interaction effects is expected to remain consistent and largely unaffected by the damping ratio. In addition to the potential damping-dependency of the current outcomes, pushover analyses do not appropriately capture complex dynamic responses, such as those of upper-floor walls and gables [94,142] affected by relative support motions. Therefore, dynamic analyses are still more appropriate in such instances.

6. Conclusions

This study investigates how pre-existing damage from seismic activity influences the seismic out-of-plane (OOP) response of unreinforced masonry (URM) walls. A high-fidelity numerical modeling approach, previously developed and validated by the authors for static and dynamic analysis of URM walls, is employed in a sensitivity analysis to examine these effects. The study includes four wall configurations: two two-way spanning walls with and without a window opening, and two one-way spanning walls of short and long lengths. The geometries and material properties are selected to represent typical residential URM buildings in the Groningen province of the Netherlands. Each specimen is subjected to static cyclic in-plane (IP) loading under low, moderate, or high vertical overburden, and with either cantilever or double-clamped boundary conditions. Based on crack severity observed during IP loading, distinct performance levels are defined and used as initial

conditions, termed pre-damage states, for the subsequent OOP analyses. These states correspond to light, moderate, and severe damage levels. Both static pushover analyses, in two loading directions, and incremental dynamic analyses (IDAs) using records from induced and tectonic earthquakes are conducted. Modal analyses are also performed on the pre-damaged specimens to evaluate changes in OOP stiffness. The outcomes are assessed in terms of how pre-damage affects OOP stiffness, static strength, and dynamic capacity of the specimens, as well as their failure mechanisms. The main findings are as follows:

- The effects of IP pre-damage primarily emerge when walls are loaded OOP starting from the moderate pre-damage state and fully develop only at the severe pre-damage state. Light pre-damage has negligible influence on the OOP response of the specimens studied.
- One-way spanning specimens exhibit minimal sensitivity to IP pre-damage, as the damage from IP loading did not spatially coincide with the subsequent cracking observed during OOP analyses. Only in exceptional cases, where segments became loose during IP pre-damage, is a notable reduction in OOP stiffness observed, reaching up to 40%.
- Two-way spanning specimens show moderate to high sensitivity to pre-damage, exhibiting reductions of up to 30% in OOP stiffness and 33% in OOP strength under severe pre-damage conditions.
- The presence of a window led to high sensitivity of two-way bending OOP stiffness to pre-damage under low pre-compression, with reductions of up to 75% observed under moderate pre-damage, primarily due to the looseness of the piers adjacent to the opening.

- The solid two-way spanning specimen exhibits higher sensitivity to pre-damage compared to the specimen with an opening in terms of collapse PMA, due to greater conformity between IP pre-damage patterns and OOP failure mechanisms.
- Current guidelines disregard the effects of pre-damage on the OOP seismic response of non-framed URM walls, while existing analytical formulations are overly conservative with respect to the current study, predicting strength losses of up to 100% at severe pre-damage states.
- Induced seismicity results in dynamic collapse only when applied at a very high loading intensity of 0.75g, and only in the one-way spanning walls, suggesting a potentially lower significance of IP pre-damage under induced earthquake conditions.
- Static analyses effectively captured the influence of pre-damage on OOP strength and failure mechanisms, providing predictions of dynamic collapse intensity that were either accurate or slightly conservative. This agreement is likely due to the force-controlled nature of the two-way OOP bending responses (brittle shear and torsional failure after the load passes the strength) governed by their dominant mode of natural deformation, which can be simulated via static analyses, and the overall low sensitivity of the one-way spanning walls investigated in this study, which leads to almost no interaction effect for the static analyses to capture.

The results of this study demonstrate that static analyses perform well in efficiently and reliably predicting IP/OOP interaction effects under dynamic loading. They also highlight the limited influence of IP pre-damage in regions affected by induced seismicity. However, these preliminary findings should be reinforced in future research. Specifically, additional dynamic loading signals should be used to repeat the analyses and address uncertainties related to dynamic response. Furthermore, a broader range of wall geometries and material properties should be considered, including varied opening configurations and combinations such as weak block-strong joint assemblies. It should also be noted that the findings of this study may have been influenced by the specimen lengths, which provided substantial horizontal confinement in the two-way spanning walls. Such extensions are necessary before reliable prescriptions could be incorporated in guidelines.

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CRediT authorship contribution statement

Amirhossein Ghezelbash: Conceptualization, Data curation, Formal analysis, Investigation, Methodology, Resources, Software, Validation, Visualization, Writing – original draft, Writing – review & editing. **Satyadhrik Sharma:** Conceptualization, Data curation, Methodology, Supervision, Validation, Writing – review & editing. **Antonio Maria D’Altri:** Conceptualization, Funding acquisition, Methodology, Supervision, Validation, Writing – review & editing. **Jan G. Rots:** Conceptualization, Funding acquisition, Methodology, Resources, Supervision, Validation, Writing – review & editing. **Francesco Messali:** Conceptualization, Funding acquisition, Methodology, Project administration, Resources, Supervision, Validation, Writing – review & editing.

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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The loading signals used for the dynamic simulations of this study are adapted from the experimental campaign conducted at EUCENTRE foundation (Pavia, Italy) on full-scale URM buildings [95,108].

Data availability

Data will be made available on request.

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