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INVESTIGATING THE SEISMIC RESPONSE OF URM WALLS WITH IRREGULAR OPENING LAYOUT THROUGH DIFFERENT MODELING APPROACHES

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Abstract

The façade and internal walls of unreinforced masonry (URM) buildings often present an irregular opening layout, due to architectural reasons or modifications to the structure, which make the expected seismic damage pattern less predictable a priori. Therefore, the discretization of the walls in structural components is not standardized, conversely to cases with a regular opening layout for which the available modeling methods are corroborated by seismic damage surveys reporting recurrent failure patterns. The structural component discretization is a relevant step for the code-conforming seismic assessment, typically based on comparing the internal forces and drifts of each component to strength criteria and drift thresholds. Therefore, the lack of well-established approaches can significantly influence the assessment. The issue is even more evident when the structural components must be identified a priori in the modeling stage, namely for equivalent frame models. The applicability of available methods for discretization of URM walls with irregular opening layout has been already investigated in literature, but a conclusive judgment requires further studies.
In this context, this paper presents an overview of the preliminary results addressing the numerical modeling of this type of walls within the framework of the DPC-ReLUIS 2022-2024 project (Subtask 10.3), funded by the Italian Department of Civil Protection. The Subtask aims to propose consensus-based recommendations for researchers and practitioners which can contribute to harmonize the use of different modeling approaches. Seven research groups are involved in the research, adopting different modeling approaches and computer codes, but similar assumptions and the same analysis method (pushover) are used. The benchmark URM structure illustrated in the paper is a two-story wall from which four configurations with increasing irregularity of opening layout were derived. The results of four modeling approached are presented. Three of them reproduce the mechanical response of masonry at the material scale by means of FE models implemented in OpenSees, DIANA and Abaqus software, while the remaining approach describes the mechanical response of masonry at the macro-element scale in 3DMacro software. Results were compared in terms of capacity curves, predicted failure mechanisms and evolution of internal forces in piers. The adoption of consistent assumptions among the different approaches led to an overall agreement of predictions at both wall and pier scales, particularly in terms of damage pattern with higher concentration of damage at the ground story. Despite that, differences on the pushover curves have been highlighted. They are mainly due to some deviations of the internal forces in squat piers deriving from a complex load flow in these elements.

Keywords: URM walls, Irregular opening layout, Modeling approaches, Constitutive models, Pushover analysis.

1 INTRODUCTION

The opening layout of walls in existing unreinforced masonry (URM) buildings is often irregular with horizontal, vertical and offset misalignments due to differences in the dimensions, shape, and position of the openings; see Figure 1a-b. An irregular opening pattern can also result from a variable number of openings per consecutive stories; see Figure 1c. In some cases, these irregularities are original, namely the different geometry and/or position of the openings was due to aesthetic or functional reasons at the time of the building construction. However, in most cases irregular opening layouts derive from architectural changes, additions, modifications to building use, reconstruction after earthquakes. Irregularities are found particularly amplified for internal walls.

Parisi and Augenti [1] established the first classification of the most recurrent types of opening layout irregularities (horizontal irregularity, vertical irregularity, offset irregularity and variable opening number irregularity). They also proposed an index indicating to which extent the irregular opening layout differs from the corresponding regular one. An expanded classification and a similar quantifying index was adopted by Berti *et al*. [2]. In both research works (see [1, 2]) the authors have assessed the implications of specific types of irregularities on the seismic response of 2-story benchmark walls by performing pushover analysis. In fact, the wall seismic response, and therefore the building response, can be significantly influenced by the presence and extent of opening layout irregularities (see also [3]).

(a) (b) (c)

Figure 1 Façade walls with irregular opening layout: (a) horizontal irregularity, (b) offset irregularity and (c) different opening number per story.

Moreover, the damage pattern of walls with an irregular opening layout is not so easily predictable a priori. In this case, the discretization of structural components, namely in terms of geometry and reference cross-sections of piers and spandrels, is not standardized, conversely to walls with a regular opening layout. In the latter case, the discretization criteria available in the literature (mainly focused on defining the effective height of piers) are corroborated by seismic damage surveys reporting recurrent failure patterns. Since all code-conforming seismic assessments are based on comparing the internal forces and drifts of each structural component with the corresponding strength criteria and drift thresholds, a different discretization has implications on the verifications (see [4, 5]). The user definition of the geometry of piers and spandrels (and the corresponding cross-sections to monitor the internal forces and drifts) is applied to all modeling approaches, typically when post-processing the results. In these cases, the assessment may vary because users choose from a wide range of different but reasonable assumptions. In the Equivalent Frame (EF) method, the discretization is established in the modeling stage and, therefore, the prediction even more depends on the initial assumptions.

These modeling issues are not properly addressed in available design codes to consider for the large variability of opening layouts, even for a single type of irregularity. In fact, there is no comprehensive information and data of failure patterns for walls with an irregular opening layout. The issue is amplified by the fact that the limits to the applicability of the EF approach are still not well-established in the literature. Siano *et al*. [6] proposed to apply this approach only if a defined geometrical parameter, related to the dimensions of pier and spandrels at each story, is smaller than a given value. In this way, URM walls are classified not only for the regularity of the opening layout but also for the frame-like geometry.

Different works (see [3, 5, 7]) compared the seismic response of EF models with the one from a reference model at the wall scale, for the most recurrent types of opening irregularities. In these works, models were developed by considering some of the available methods for defining the effective height of piers (e.g., [1, 8, 9]). In [7], a continuum micro-modeling based on the Finite Element (FE) method was taken as a reference while a FE macro-modeling and other based on the Applied Element method were taken as a benchmark in [3, 5]. In all cases, the wall seismic response was assessed by means of pushover analysis with assuming a mass-proportional load distribution. However, the comparison parameters are different from work to work, e.g., the secant stiffness is calculated at different percentages of the peak base shear force, as well as the displacement capacity. A detailed comparison was made by Cattari *et al.* [5] with quantitative judgments about matching the EF results to the reference FE predictions, in terms of damage patterns and internal forces of the structural components. Benchmark walls were idealized from existing URM buildings with two stories and two bays for which the seismic response is mainly controlled by few structural components.

The literature review has evidenced that, although some recommendations for modeling walls with irregular opening layouts were proposed in those works, they are still not conclusive and cannot be generalized to different wall geometries. In this context, the topic of walls with irregular opening layout and the related open issues fall within the framework of the DPC-ReLUIS 2022-2024 project (Subtask 10.3), promoted by the Italian Department of Civil Protection. The Subtask focuses on benchmarking different modeling approaches for the code-conforming seismic assessment of URM structures. This benchmarking aims to contribute to the harmonization of the available approaches, proposing consensus-based recommendations for researchers and practitioners. The seven participant Research Groups (RGs) are affiliated to the: University of Genova coworking with the University of Minho, University of Pavia, University of Bologna, University of Chieti-Pescara G. D'Annunzio, University of Catania, University of Napoli Federico II, and Politecnico di Milano. The RGs have already worked together within the DPC-ReLUIS 2019-2021 project "*URM nonlinear modeling – Benchmark project*" (see [10, 11]). In this paper, the numerical analyses on four 2-story benchmark walls with multiple bays evidencing offset irregularity and variable opening number are presented. Note that these walls were idealized from existing URM buildings already considered as case study buildings in the previous Benchmark project [10].

After presenting the framework of the current ReLUIS project (Subtask 10.3) and its methodology, the paper gives an overview of the preliminary results from some RGs (4 out of 7) in terms of pushover curves and damage pattern for the benchmark walls. Although different modeling approaches and software were adopted by the participant RGs, the dispersion of predictions was limited by considering similar modeling assumptions and performing the same analysis (mass-proportional pushover). Moreover, for all the modeling strategies, the pushover response at the pier scale was validated against the available strength domains, as already tested in [12]. Despite the harmonization of the modeling hypotheses, in some cases, differences in results were still obtained. Additional remarks on the results and particularly on the complex failure mechanism of squat piers and modeling the RC ring beam are drawn.

2 METHODOLOGY

A brief description of the methodology and the highlights of the work are presented in this section. First, a priority was assigned to each irregularity type defined by Parisi and Augenti [1] according to its recurrence in existing URM buildings. The recurrence was evaluated by using a photographic database of damage patterns of walls with an irregular opening layout, built by the University of Genova after the Central Italy (2016–17), Emilia (2012) and L'Aquila (2009) earthquakes. Note that irregularities resulting from contiguous walls in building aggregates were not considered. Buildings with two openings vertically aligned in the same story or 2-story openings are not considered as well. Moreover, a higher priority was assigned to those irregularity types involving a series of critical issues when modeling, as already discussed in Section 1. Here, only offset and variable opening number irregularities are addressed.

For these irregularity types, four 2-story benchmark walls were idealized from the internal walls of two existing URM buildings in Italy: the Town Hall in Pizzoli and P. Capuzi school in Visso. Details on the case study buildings are given in the *Annex I-Benchmark Structures Input Data* [10]. The first wall has a regular opening layout, labeled as P1.0 in Figure 2, while the other wall configurations, P1.1–P1.3, present an increasing level of irregularity. The benchmark walls consist of traditional stone masonry, and the structural details allow assuming that the seismic response is mainly dependent on the in-plane behavior of its components (piers and spandrels).

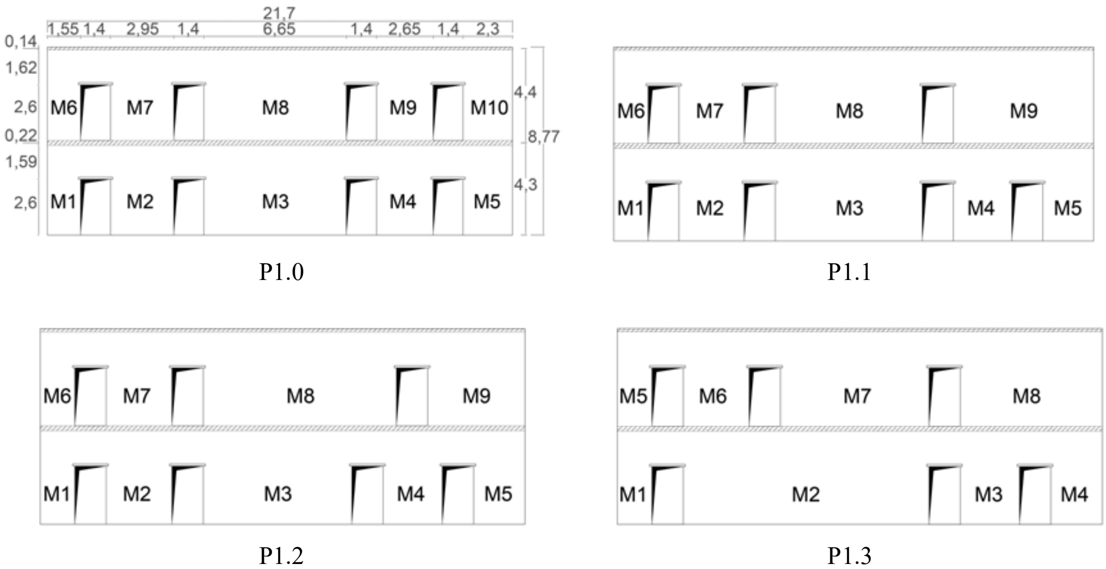


Figure 2 Geometry of benchmark walls (dimensions in m).

The wall is rectangular in shape with dimensions of 21.7 m × 8.77 m, and thickness of 0.55 m. The positive sign of the horizontal direction points to the right. Each pier is identified with a tag M#, as shown in Figure 2. Note that coupling spandrels were considered to limit the uncertainty on the capacity models of these components. Indeed, Reinforced Concrete (RC) ring beams were modeled at each floor level. The width of these beams has the same thickness of the wall. Details on the material properties of concrete and steel reinforcements can be found in the *Annex I-Benchmark Structures Input Data* [10]. The stone masonry is assumed consistent with the "hammer-dressed stone masonry with good bonding" of Table C8.5.I in the 2019 Italian Commentary [13]. According to this Commentary, when considering a normal knowledge level, the design value of the compressive strength is 2.67 MPa while the one for the tensile strength associated to the diagonal cracking mechanism is 0.081 MPa. Elastic modulus of 1.74 GPa and weight density of 2141 kg/m^3 are assumed.

The structure, representative of an internal wall, is assumed to support floors on both sides; therefore, the total tributary length is equal to 5 m. The load of a one-way lightweight RC ribbed slab is considered at the 1st level. The load sum of a steel-clay slab and roof structure is applied at the 2nd level. Details on these loads are given in the *Annex I-Benchmark Structures Input Data* [10], although a residential live load (and corresponding partial factor for the quasi-permanent combination) is adopted here.

Since the participant RGs use different modeling approaches and software, similar assumptions were considered to limit the dispersion of predictions. The coupling spandrels hypothesis is also intended to this purpose. Moreover, a cross-comparison of the numerical push-over response at pier scale was made between the approaches and against the strength criteria defined in the Italian Building Code (NTC 2018) [14] and its 2019 Commentary [13]. For the pushover analysis (with mass-proportional load pattern), all the RGs simulated the equivalent seismic forces as distributed forces on the RC ring beams at both levels. This choice is consistent with the assumptions adopted in EF models for the sake of future comparison. The near collapse limit state was defined for the wall damage at the ultimate displacement (d_u) corresponding to the 20% reduction of the peak base shear force (V_{max}).

The capacity curves and the damage patterns corresponding to V_{max} and d_u were requested as comparison output. The pier internal forces and drifts at certain cross-sections were asked as well. The results were collected in datasheets to allow a centralized post-processing, which preliminary results are presented in Section 4. The next step in Subtask 10.3 is to measure the match of the wall response predicted through different EF idealization criteria (existing and new proposals by the RGs) against the response from the advanced modeling approaches. An example of this procedure is adopted in [5]. It will allow to assign a weighted score to qualitative parameters of the pushover response and eventually assess which method/proposal is more appropriate for a specific opening layout irregularity.

3 OVERVIEW ON THE MODELING APPROACHES AND MATERIAL MODELS ADOPTED BY THE RESEARCH GROUPS

Four modeling approaches have been considered to simulate the selected benchmark URM walls. Three approaches reproduce the mechanical response of masonry at the material scale by means of FE models implemented in OpenSees, DIANA and Abaqus software; see references [15–17] and Table 1. Differently, RG2 team represents the mechanical response of masonry at the macro-element scale (in this case corresponding to the structural component scale) in 3DMacro software [18].

Regarding the FE approaches, those used by teams RG3 and RG4 idealize masonry as an equivalent nonlinear homogeneous continuum material. For both cases, an isotropic behavior which is able to simulate tensile cracking and compressive crushing of the material is adopted. In particular, the approach by RG3 team uses a rotating total strain crack model (see [16, 19]) on 8-node plane stress FEs, while RG4 team adopts the concrete damaged plasticity model (see [20]) on 4-node solid FEs; see Table 2. Conversely, the approach used by RG1 team adopts a damage plasticity model (see [21]) on 4-node thick shell FEs. This model requires the mechanical properties of the bricks and mortar joints to be explicitly modeled. Thus, the textured continuum material allows to consider for the orthotropic nature of masonry with all its failure mechanisms, i.e., opening and sliding of mortar joints and compressive crushing of bricks.

For the model by RG2 team, a discrete macro-element approach developed by Caliò *et al.* [22] was adopted. It is based on an equivalent mechanical scheme of the masonry panel (consisting of an assembly of nonlinear springs) able to simulate rocking, diagonal cracking, and sliding failure modes. Details on the modeling approaches by teams RG1, RG2 and RG4, as well as the methodology for the mechanical properties validation at the panel scale, are given in [12]. This validation was also adopted by RG3 team with reference to the benchmark piers in [12]. A similar validation procedure is described in [23], where the material model adopted by RG3 is also explained in details.

	RG1	**RG2**	**RG3**	**RG4**
Modeling approach	FE textured continuum model	Discrete macro-element model	FE continuum model	FE continuum model
Software	OpenSees	3DMacro	DIANA	Abaqus
Mesh	Quadrilateral 4-node FEs	Macro-elements	Quadrilateral 8-node FEs	Tetrahedron 4-node FEs
	Size of 5 cm for bricks and 1 cm for mortar joints	Pier and spandrel size	Size of 10–20 cm	Size of 25 cm
Masonry model	At material scale	At panel scale	At material scale	At material scale
	Damage plasticity micro model TC3D	Plane macro-element with uniaxial nonlinear springs	Rotating total strain crack model	Concrete damaged plasticity model
	Orthotropic	Orthotropic	Isotropic	Isotropic
Failure modes	Opening and sliding of mortar joints and compressive crushing	Flexural and diagonal cracking	Tensile cracking and compressive crushing	Tensile cracking and compressive crushing

Table 1: Description of the modeling approaches and material models adopted by participant RGs.

All teams assumed a linear elastic behavior for the RC ring beams as a simplification for the preliminary analyses presented in this paper. However, an investigation on the effect of considering the nonlinear behavior of these beam elements was performed by teams RG1 and RG3. The study also involves other teams, but their results are not presented here for brevity. The nonlinear behavior of RC ring beams will be assumed in all the advanced modeling approaches to establish the final reference solution which the EF models will be compared to. When considering the nonlinearity in these elements, the RC ring beams present a nonlinear mechanical constitutive law for concrete and explicitly include the reinforcements, both longitudinal rebars and stirrups. For these reinforcements, embedded truss FEs were used.

Lintels above openings are modeled only in the models by teams RG1 and RG3 (see Table 2). Both teams considered these elements having a linear elastic behavior and assumed a soft connection to the side and upper masonry portions through frictional interfaces. The approach used by teams RG2 and RG4 neglects the presence of lintels, as a simplification. Note that the lateral resistance and the corresponding drift capacity of coupling spandrels also depend on the interaction with lintels.

	RG1	RG2	RG3	RG4
RC ring beam modeling	Linear concrete and no reinforcements Nonlinear concrete and embedded reinforcements	Linear beam	Linear concrete and no reinforcements Nonlinear concrete and embedded reinforcements	Linear concrete and no reinforcements
Spandrel modeling	Linear lintels with soft interface	No lintels	Linear lintel with Coulomb-friction interface	No lintels

Table 2: Additional features on the RC ring beam and spandrel modeling for different RGs.

4 SEISMIC RESPONSE OF BENCHMARK WALLS

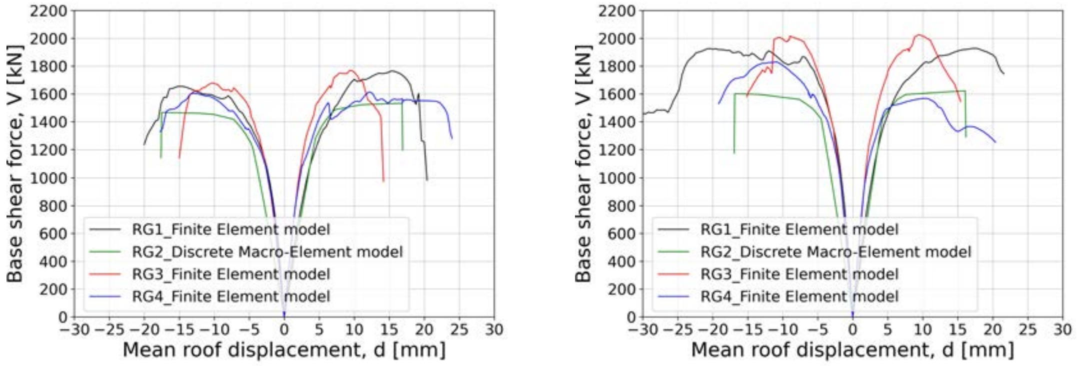
The pushover response of the benchmark walls predicted by 4 RGs is presented in this section. The wall response is cross-compared in terms of secant stiffness at 60% of V_{max}, ($k_{60\%Vmax}$), V_{max}, d_u, and damage patterns. Note that this cross-comparison is made only for Walls P1.1 and P1.3 for brevity. The implications on results of assuming a linear or nonlinear behavior of RC ring beams for these walls are shown as well. An investigation into the evolution of the internal forces in one squat pier (M3 in Wall P1.1) is also reported.

The pushover curves are shown in Figure 3 and the corresponding synthetic parameters ($k_{60\%Vmax}$, V_{max}, d_u) are presented in Table 3 and Table 4 for Walls P1.1 and P1.3, respectively. The differences in $k_{60\%Vmax}$ between the curves predicted by teams RG1, RG3 and RG4 (in the negative direction) are limited, while the estimated value of $k_{60\%Vmax}$ by RG2 team is much smaller than the previous ones, e.g., 281 kN/mm (RG2 team) vs 459 kN/mm (RG1 team) for Wall P1.1; see Table 3. This fact depends on the adoption of a cracked shear modulus by the last team (RG2). Contrarily to all other predictions, the values of $k_{60\%Vmax}$ for RG1 team change significantly from positive to negative direction for both walls, e.g., 299 kN/mm vs 459 kN/mm for Wall P1.1; see Table 3. The difference depends on the development of different damage patterns for the two directions, therefore different stiffness degradations, as well as the use of an orthotropic material model.

The value of V_{max} predicted by RG2 model is the lower bound for Wall P1.1. The same consideration applies for Wall P1.3, except in the positive sense for which V_{max} by RG4 team is slightly smaller; see Table 3 and Table 4. The result depends on the diagonal cracking strength domain of the squat piers in the first model (RG2), which is calculated according to the Turnšek and Čačovič [24] criterion. The corresponding resistance is smaller than those predicted by other models and therefore V_{max} is smaller as well, as further explained in the following through the detailed comparison of the base shear of the central squat pier. For Wall P1.1, the results by teams RG1 and RG3 in terms of V_{max} considerably differ from the positive to the negative direction, e.g., 1764 kN vs 1654 kN for RG1 team, respectively. Reduced differences between V_{max}^+ and V_{max}^- are found for the estimates of teams RG2 and RG4.

The seismic response of Wall P1.3 is more asymmetrical than the one of Wall P1.1 due to the more irregular opening layout. Thus, the differences between V_{max}^+ and V_{max}^- are relevant, especially for the predictions of RG4 team (1567 kN vs 1809 kN); see Table 4. The smallest values of d_u for Wall P1.1 are associated to the models by RG3 team. This fact depends on the activation of the compressive strength reduction option in the corresponding material model (see [25]). If this prediction is neglected, the values of d_u for Wall P1.1 are consistent among models RG1, RG2 and RG4, e.g., ranging from 18–24 mm in the positive direction (see Table

3). For Wall P1.3, the value of d_u predicted by RG1 team is twice the one by RG4 team (10 mm vs 26 mm in the negative direction); see Table 4.



Wall P1.1 Wall P1.3

Figure 3 Pushover curves predicted by different RGs with different modeling approaches.

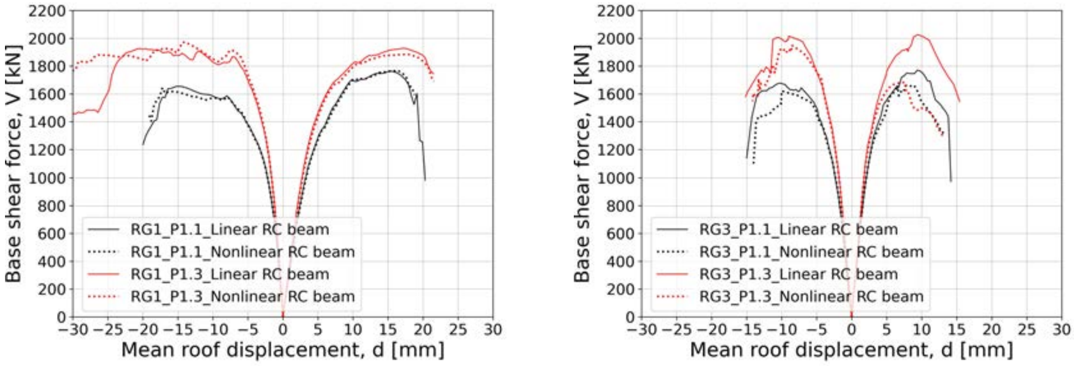
	$k_{60\%Vmax}^{+}$ [kN/mm]	$k_{60\%Vmax}^{-}$ [kN/mm]	V_{max}^{+} [kN]	V_{max}^{-} [kN]	d_u^{+} [mm]	d_u^{-} [mm]
RG1	299	459	1764	1654	19	19
RG2	298	281	1533	1466	17	18
RG3	462	424	1769	1678	14	15
RG4	457	441	1613	1604	24	18
min-max	298–462	281–459	1533–1769	1466–1678	14–24	15–19

Table 3: Secant stiffness, peak base shear force and ultimate displacement predicted by different RGs and corresponding ranges for Wall P1.1.

	$k_{60\%Vmax}^{+}$ [kN/mm]	$k_{60\%Vmax}^{-}$ [kN/mm]	V_{max}^{+} [kN]	V_{max}^{-} [kN]	d_u^{+} [mm]	d_u^{-} [mm]
RG1	344	568	1928	1926	22	26
RG2	341	325	1621	1601	16	17
RG3	535	509	2026	2015	15	15
RG4	563	524	1567	1809	13	10
min-max	341–563	325–568	1567–2026	1601–2015	13–22	10–26

Table 4: Secant stiffness, peak base shear force and ultimate displacement predicted by different RGs and corresponding ranges for Wall P1.3.

The effect of assuming a linear or nonlinear behavior of RC ring beams on the pushover response is given in Figure 4 for teams RG1 and RG3. The same walls configurations (P1.1 and P1.3) are considered. Note that the mesh size adopted by the two RGs does not allow the accurate simulation of the shear failure mechanism of the RC ring beams. A trend of decreasing the values of $k_{60\%Vmax}$, V_{max} and d_u is observed for RG3 team when moving from linear to nonlinear behavior of RC ring beam. The response particularly changes for Wall P1.3 in the positive sense due to a different damage pattern. On the contrary, the pushover curves predicted by RG1 team for the different behavior of the RC ring beam are almost overlapped for both walls. The only exception relates to the negative sense of Wall P1.3, for which the value of d_u increased from around 26 mm to 33 mm.

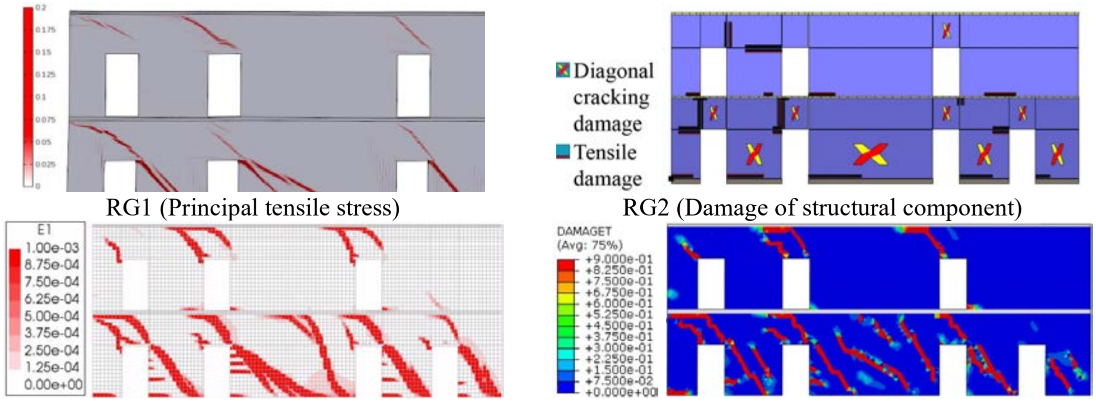


RG1 RG3

Figure 4 Pushover curves for Walls P1.1 and P1.3 with linear or nonlinear behavior of RC ring beams.

The predicted damage pattern is rather consistent among the models within the same wall configuration; see Figure 5 for Wall P1.1 and Figure 6 for Wall P1.3. Note that only the positive sense is considered here for brevity. As expected, there is a higher concentration of damage at the ground story due to the mass-proportional load pattern. This load distribution allows to simulate a severe damage state in a structure, but it limits the force sharing between consecutive stories. However, the dominant failure mechanism for piers is diagonal cracking, except for pier M1 failing in flexure. The diagonal cracking mechanism was predicted also for spandrels combined with cracks running slantingly or pseudo-vertically from the windows corners to the floor levels (models by teams RG3 and RG4). A multiple failure mechanism is predicted for the squat piers at the ground story, namely for pier M3 in Wall P1.1 and pier M2 in Wall P1.3 (see pier labels in Figure 2). These damage patterns and the corresponding pier response are discussed in the following.

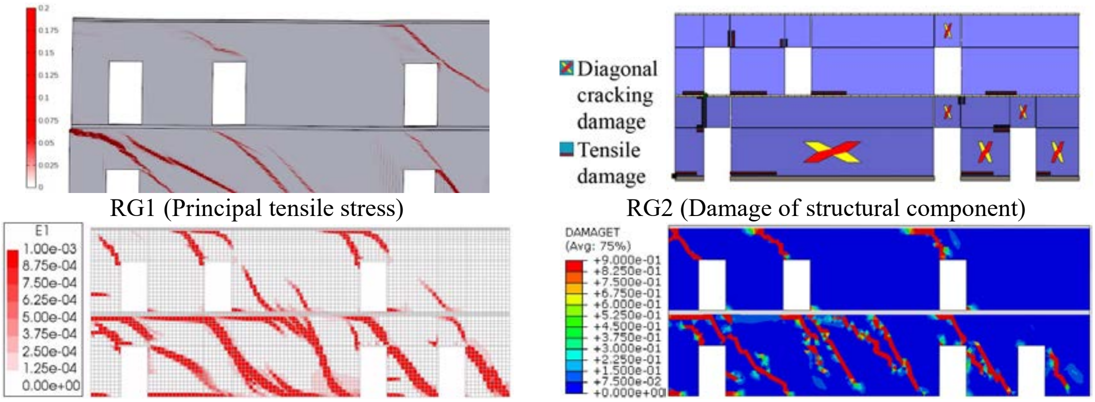
The differences on the pushover curves of predicted wall response when varying the modeling approach mostly depends on a different lateral response at the pier scale. For this reason, the internal forces were integrated at the significant cross-sections of all piers at the wall base and then compared with the flexural and diagonal cracking strength criteria. However, only M3 pier in Wall P1.1 is addressed here. The diagonal cracking resistance is calculated according to the Turnšek and Čačovič [24] criterion due to the irregular pattern of the stone masonry. The stress-block hypothesis with reduced compressive strength (0.85 of the design value) is adopted for the flexural strength domain (see NTC 2018 [14]). Moreover, both cantilever and double bending configurations were considered for the flexural domain, as the pier response is within this range.



RG1 (Principal tensile stress) RG2 (Damage of structural component)

RG3 (Principal tensile strain) RG4 (Tensile damage parameter)

Figure 5 Predicted damage pattern at peak base shear for Wall P1.1 by different RGs.



RG1 (Principal tensile stress) RG2 (Damage of structural component)

RG3 (Principal tensile strain) RG4 (Tensile damage parameter)

Figure 6 Predicted damage pattern at peak base shear for Wall P1.3 by different RGs.

The dominant strength domain, i.e., the one associated with the smallest shear resistance, for most of axial loads of M3 pier, is diagonal cracking. Note that the predictions by teams RG1, RG3 and RG4 using continuum FE models are above the limit of the Turnšek and Čačovič [24] domain; see Figure 7a. Note that this criterion was adopted to simulate the pier shear resistance in the discrete macro-element model by RG2 team. Indeed, the Turnšek and Čačovič [24] criterion seems to underestimate the shear resistance of very squat piers, and its validity range is limited. Just to name a few limitations: the pier aspect ratio is assumed higher than 0.67, the intersecting point of the pier diagonals is taken as representative of the stress state of the entire pier, the horizontal normal stresses are considered negligible. When looking at the damage pattern of M3 pier (Wall P1.1) for teams RG1, RG3 and RG4, see Figure 5, three fully developed mechanisms are predicted in different masonry portions. The lateral parts evidenced flexural damage with horizontal cracks due to rocking of the unloaded pier side, and vertical cracks due to compressive crushing on the loaded side. In the mid-part, there are multiple diagonal cracks not completely centered on the pier and not perfectly aligned with the pier diagonals.

The difference between the shear resistance by Turnšek and Čačovič [24] criterion and the one predicted by the continuum FE models is evident when comparing the evolution of the pier shear force during the analysis; see Figure 7b. This difference should be considered when comparing advanced modeling approaches with EF models, in addition to the influence of other modeling disparities. However, the numerical results could not be fully validated because there are no comprehensive experimental campaigns investigating the lateral response of very squat piers with different boundary and loading conditions. Note that the resistance values obtained from EF models are smaller than those from FE models, and therefore on the safe side.

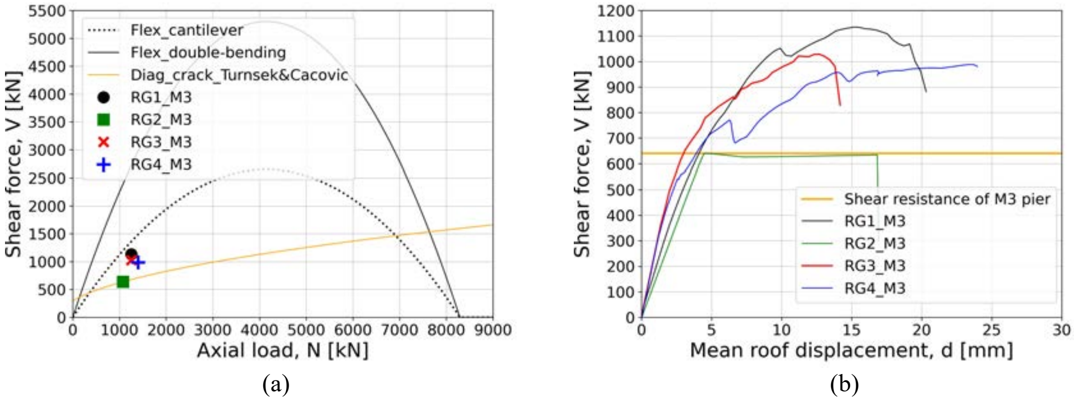


Figure 7 Pier M3 of Wall P1.1: (a) numerical results compared with strength domains and (b) shear force evolution during the pushover analysis.

5 FINAL REMARKS AND FUTURE DEVELOPMENTS

The paper intends to provide an overview of the framework, objectives and preliminary results of the Subtask 10.3 within DPC-ReLUIS 2022-2024 project. Specifically, the Subtask focuses on modeling issues for equivalent frame discretization of URM walls with irregular opening layouts. The final goal is benchmarking the pushover response of reference walls estimated by different methods for the discretization of structural components, i.e., piers and spandrels, using existing and new proposed approaches, against the predictions of advanced modeling approaches.

For the intended goal, various benchmark walls were idealized from existing URM buildings that present offset irregularity and a variable number of openings per story. In addition to the wall description, a specific section of the paper focuses on the main features of the adopted modeling approaches. This section aims to understand the reasons for the response disparities, although similar assumptions among approaches were made in the modeling and analysis stages. The models were in fact implemented in different software packages, both masonry specific and general purpose. The adopted models have different levels of complexity and detail, therefore with significant differences in the required input. The preliminary results for some RGs (4 out of 7) were presented here in terms of pushover curves, damage patterns and internal forces.

Despite using similar assumptions to limit the dispersion of results, unmatched predictions were initially obtained, highlighting the need for result control even among researchers. The preliminary results have shown moderate differences in the pushover parameters for both walls. Specifically, the scatter in secant stiffness, peak base shear and displacement capacity is larger for Wall P1.3 which opening layout is more irregular than Wall P1.1. Note that these parameters, especially the secant stiffness, influence the seismic safety assessment, e.g., based on the N2 method by Fajfar [26]. However, the predicted damage patterns are consistent among all the models within the same wall configuration despite some differences. There is a higher concentration of damage at the ground story due to the mass-proportional load distribution for the pushover analysis.

The investigation on the RC ring beam confirms the influence of the corresponding modeling assumptions on the pushover results. Note that the shear failure mechanism of these beams can be captured only by using an extensive number of finite elements, which were not modeled in this work. There are additional limitations (and underestimation of capacity) in considering the RC ring beams without the tributary floor portion which contributes to higher stiffness and resistance. In fact, the available damage survey after recent earthquakes evidenced light damage on these elements for low-rise buildings and confinement of the spandrel masonry portions.

The study of the internal forces and damage of a squat pier highlighted some potential limits in the applicability of the Turnšek and Čačovič [24] criterion. Note in fact that the corresponding shear resistance is smaller (on the safe side) than the one predicted by the advanced modeling approaches. For these approaches, a multiple failure mechanism was predicted with rocking damage on the unloaded side of the pier, diagonal cracks not perfectly aligned to the pier diagonals in the central part, and compressive crushing on the loaded side. However, the numerical results must be validated against those from experimental campaigns on squat piers before making conclusive judgments. Unfortunately, these campaigns are still limited.

The next steps of the project will focus on the detailed comparison between the results of the advanced modeling approaches and those of different methods for equivalent frame discretization. This comparison aims to propose practical recommendations for the selected opening layout irregularities. Furthermore, the set of benchmark structures will be increased by considering walls with different characteristics, namely higher aspect ratios. Investigations into the effect of RC ring beam modeling and different distributions of the equivalent seismic forces will also be included.

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REFERENCES

[1] F. Parisi, N. Augenti, Seismic capacity of irregular unreinforced masonry walls with openings. *Earthquake Engineering & Structural Dynamics*, **42**(1), 101–121, 2013. doi: 10.1002/eqe.2195.

[2] M. Berti, L. Salvatori, M. Orlando, P. Spinelli, Unreinforced masonry walls with irregular opening layouts: reliability of equivalent-frame modelling for seismic vulnerability assessment. *Bulletin of Earthquake Engineering*, **15**(3), 1213–1239, 2017. doi: 10.1007/s10518-016-9985-5.

[3] C. Morandini, D. Malomo, A. Penna, Equivalent frame discretisation for URM façades with irregular opening layouts. *Bulletin of Earthquake Engineering*, **20**(5), 2589–2618, 2022. doi: 10.1007/s10518-022-01315-0.

[4] D. Ottonelli, C. F. Manzini, C. Marano, E. A. Cordasco, S. Cattari, A comparative study on a complex URM building: part I—sensitivity of the seismic response to different modelling options in the equivalent frame models. *Bulletin of Earthquake Engineering*, **20**, 2115–2158, 2022. doi: 10.1007/s10518-021-01128-7.

[5] S. Cattari, A. M. D'Altri, D. Camilletti, S. Lagomarsino, Equivalent frame idealization of walls with irregular openings in masonry buildings. *Engineering Structures*, **256**, Article no. 114055, 2022. doi: 10.1016/j.engstruct.2022.114055.

[6] R. Siano, V. Sepe, G. Camata, E. Spacone, P. Roca, L. Pelà, Analysis of the performance in the linear field of equivalent-frame models for regular and irregular masonry walls. *Engineering Structures*, **145**, 190–210, 2017. doi: 10.1016/j.engstruct.2017.05.017.

[7] G. Camata, C. Marano, V. Sepe, E. Spacone, R. Siano, M. Petracca, P. Roca, L. Pelà, Validation of non-linear equivalent-frame models for irregular masonry walls. *Engineering Structures*, **253**, Article no. 113755, 2022. doi: 10.1016/j.engstruct.2021.113755.

[8] M. Dolce, Schematizzazione e modellazione degli edifici in muratura soggetti ad azioni sismiche (Seismic modeling of masonry buildings) [in Italian]. *L'industria Delle Costruzioni*, **25**, 44–57, 1991.

[9] S. Lagomarsino, A. Penna, A. Galasco, S. Cattari, TREMURI program: An equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Engineering Structures*, **56**, 1787–1799, 2013. doi: 10.1016/j.engstruct.2013.08.002.

[10] S. Cattari, G. Magenes, Benchmarking the software packages to model and assess the seismic response of unreinforced masonry existing buildings through nonlinear static analyses. *Bulletin of Earthquake Engineering*, **20**, 1901–1936, 2022. doi: 10.1007/s10518-021-01078-0.

[11] S. Cattari, G. Magenes, G. Camata, I. Calio', B. Calderoni, S. de Miranda, A. Saetta, S. Degli Abbati, D. Ottonelli, C. F. Manzini, P. Morandi, E. Spacone, C. Marano, B. Pantò, F. Cannizzaro, G. Occhipinti, E. A. Cordasco, G. Castellazzi, A. M. D'Altri, L. Berto, A. Doria, D. Talledo, G. Milani, Uso dei software di calcolo nella verifica sismica degli edifici in muratura v1.0 (Use of software codes in the seismic safety assessment of masonry buildings). [in Italian]. ReLUIS project - Accordo Quadro DPC 2019-2021. WP10 - Contributions to the Italian Building Code associated to existing masonry buildings, Rete dei Laboratori Universitari di Ingegneria Sismica (ReLUIS), 2020. [Online]. Available: http://www.reluis.it/index.php?option=com_content&view=article&id=692:uso-dei-software-di-calcolo-nella-verifica-sismica-degli-edifici-in-muratura-v10&catid=34:news-reluis&lang=it

[12] A. M. D'Altri, F. Cannizzaro, M. Petracca, D. A. Talledo, Nonlinear modelling of the seismic response of masonry structures: Calibration strategies. *Bulletin of Earthquake Engineering*, 2021. doi: 10.1007/s10518-021-01104-1.

[13] Circolare 21/01/2019, no. 7 C.S.LL.PP. Instructions for the application of the updated "Technical standards for constructions". (Istruzioni per l'applicazione dell'Aggiornamento delle "Norme tecniche per le costruzioni" di cui al D.M. 17/01/2018), [in Italian]. 2019.

[14] NTC 2018. Updated Technical standards for constructions. (Norme Tecniche per le Costruzioni, D.M. 17/01/2018, Ministero delle Infrastrutture e dei Trasporti, S.O. no. 8 alla G.U. del 20/2/2018 no. 42, Rome, Italy), [in Italian]. 2018.

[15] F. McKenna, G. L. Fenves, M. H. Scott, OpenSees. Open System for Earthquake Engineering Simulation. University of California, Pacific Earthquake Engineering Research Centre, Berkeley, 2000.

[16] DIANA FEA. DIsplacement ANAlysis finite element software. Release 10.6. Delft, The Netherlands, 2022.

[17] Abaqus/CAE, Finite Element software. Release 2022.

[18] 3DMacro, Il software per le murature (3D computer program for the seismic assessment of masonry buildings), Gruppo Sismica S.r.l. Release 2022.1.3. Catania, Italy, 2022. [Online]. Available: https://www.grupposismica.it

[19] F. J. Vecchio, M. P. Collins, The Modified Compression-Field theory for concrete elements subjected to shear. *ACI Journal*, **83**(22), 219–231, 1986.

[20] J. Lee, G. L. Fenves, Plastic-Damage Model for Cyclic Loading of Concrete Structures. *Journal of Engineering Mechanics*, **124**(8), 892–900, 1998. doi: 10.1061/(ASCE)0733-9399(1998)124:8(892).

[21] M. Petracca, L. Pelà, R. Rossi, S. Zaghi, G. Camata, E. Spacone, Micro-scale continuous and discrete numerical models for nonlinear analysis of masonry shear walls. *Construction and Building Materials*, **149**, 296–314, 2017. doi: 10.1016/j.conbuildmat.2017.05.130.

[22] I. Caliò, M. Marletta, B. Pantò, A new discrete element model for the evaluation of the seismic behaviour of unreinforced masonry buildings. *Engineering Structures*, **40**, 327–338, 2012. doi: 10.1016/j.engstruct.2012.02.039.

[23] F. Parisse, R. Marques, S. Cattari, P. B. Lourenço, Finite Element and Equivalent Frame modeling approaches for URM buildings: Implications of different assumptions in the seismic assessment. *Journal of Building Engineering*, **61**, Article no. 105230, 2022, doi: 10.1016/j.jobe.2022.105230.

[24] V. Turnšek, F. Čačovič, Some experimental results on the strength of brick masonry walls. *Proceedings of the 2nd International Brick Masonry Conference*, 149–156, Stoke-on-Trent, UK, April 12–15, 1971.

[25] F. J. Vecchio, M. P. Collins, Response of reinforced concrete to in-plane shear and normal stresses. University of Toronto, Toronto, Ontario, Canada, Scientific Report 82–03, 1982.

[26] P. Fajfar, A nonlinear analysis method for performance-based seismic design. *Earthquake Spectra*, **16**(3), 573–592, 2000. doi: 10.1193/1.1586128.