Multiscale procedure to assign structural damage levels in masonry buildings from observed or numerically simulated seismic performance

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Abstract

The damage level (DL) is a fundamental metric in earthquake engineering and is particularly effective for multiple purposes, such as correlating structural damage to consequences and losses (e.g. economic, casualties, etc.) as well as to physical and mechanical variables quantifiable from both experimental and numerical tests. Despite the relevance of the topic, the conversion of complex information on the damage spreading over a 3D building into a synthetic DL remains an open issue for which there are currently no agreed-upon scientific criteria in the literature. Within this context, the study focuses on unreinforced masonry (URM) buildings by providing an analytical multi-scale approach that integrates structural damage at the panel-scale and macro-element size (where the macro-element is intended as an assemblage of components, like vertical walls or diaphragms). The approach is specifically conceived to be applied consistently to both observed and numerically simulated damage. As a result, the method is particularly effective for supporting the validation of numerical models or synthetically interpreting the huge amount of results from nonlinear analyses as much as feasible using an objective basis. The paper proposes a comparison of DL interpretation arising from other approaches in the current literature, highlighting their pros and cons. The effectiveness of the proposed procedure is then assessed using four URM case studies, all of which exhibited primarily a ‘box-like behaviour’. They were specifically chosen based on the extremely detailed documentation available for both the damage and geometric/mechanical features of the buildings as well as aiming to reduce the uncertainty in the seismic input, making them ideal candidates also for validation purposes. To this end, the applicability of the procedure also to the damage simulated by nonlinear dynamic analyses was proven by adopting equivalent frame models of the case studies, developed and calibrated in previous researches. Definitely, the results encourage the adoption of the proposed procedure to interpret as analytically as possible the real damage that occurred on URM building, when accurate damage observations from field surveys are available, as well as that coming from numerical simulations.

Keywords  Damage metric · Observed damage · Numerically simulated damage · Masonry · Benchmark case studies · Nonlinear analyses

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1 Introduction

Post-earthquake damage data constitutes an invaluable source of information for assessing the seismic vulnerability of the exposed building stock. The observed damage can be considered as direct evidence of the actual seismic capacity of buildings under earthquakes. The assignment of a synthetic damage level (DL) is the starting point for numerous engineering impact goals, in addition of being an important tool for understanding the structural response of buildings.

In the field of loss estimation and risk analysis, for example, DLs are the parameters to which most of the fragility curves and consequence functions are associated, as implemented in many *ad-hoc* tools, such as HAZUS (FEMA 2003), SELENA (Molina et al. 2010) or PACT (FEMA P-58 2018). As recently applied by Dolce et al. (2021) within the seismic risk assessment of residential buildings in Italy, the attainment of specific DLs becomes the main factor to correlate the consequent impacts in terms of unusable and collapsed buildings, casualties and direct economic losses.

The availability of a large amount of data from real earthquakes allows for the development of robust and reliable correlation laws, as discussed for example in De Martino et al. (2017), where a regression analysis was used to propose a correlation between empirical damage and actual repair costs; in that case, results were based on the data collected after the reconstruction process of the 2009 L’Aquila event (see also Di Ludovico et al. 2017a, b). Furthermore, in most cases, the physical character of the Performance Levels (PLs) or Limit States (LSs) specified by Codes and Standards to execute the safety seismic verification/design of structures is closely represented by concepts connected to DLs. In fact, while the final goals of PLs usually rely on other targets, such as the operational level guaranteed by the building or the safety of people, the criteria recommended for defining their attainment are usually linked to the damage that occurred at local or global scales. However, it is worth noting that numerical models provide continuous damage variables, and identifying discrete DLs/PLs is a difficult process. For example, in ASCE (2017) the attainment of PLs is treated by checking the corresponding DLs in each single element, whereas in the EC8 (CEN 2004) and (NTC 2018) by considering interstorey drift thresholds and/or heuristic criteria on the stiffness/strength degradation of the pushover curve obtained by Nonlinear Static Analyses (NSAs), assumed to be representative of the structural performance of the building.

All these factors highlight the importance of establishing unambiguous and homogeneous criteria for interpreting the structural response also in terms of damage levels: this is necessary not only to ensure an objective evaluation, but also to reduce uncertainty when comparing risk assessments conducted by different researchers and using different approaches.

Despite this obvious concept, the current state-of-the-art for masonry structures is highly multifaceted, and converting the complex information on the damage spreading over a 3D building into a synthetic DL still constitutes a tricky issue. An interesting proposal in this direction is presented by Novelli and D’Ayala (2019) where the LOG-IDEAH tool is presented. The latter is a web-Knowledge Based System aimed at simulating the surveyors’ decision-making processes in identifying failure modes by interpreting crack patterns (regarding both severity and extension), obtained from post-earthquake building inspections. Aside from the lack of a univocal criterion for assigning the damage level, the damage scale itself offers several alternatives, including nomenclature, description clarity, the number of damage levels. Hill and Rossetto (2008) compared all the available methods
in-depth, attempting also to establish some equivalence rules in the process. Recently, also in Rezaie et al. (2022) a comparison among different DL definitions available in the literature both at global scale and that of single elements for URM buildings has been reviewed. In this paper, the discretization in five levels, as adopted in the macroseismic EMS 98 scale (Grünthal et al. 1998), is taken as reference, namely: 1—negligible to slight damage; 2—moderate damage; 3—substantial to heavy damage; 4—very heavy damage; 5—destruction.

The criterion utilized to assign such DLs is the difficult part, regardless of whether empirical/numerical approach is used. Indeed, establishing a robust correlation between damage observed at a smaller scale (i.e. severity and diffusion in various components of the building, such as walls, floors and roofing systems, stairs, non-structural elements) and the global scale is difficult and, although employed in several applications (e.g. recently discussed in Cook and Liel 2021), it remains an open issue, as discussed further in Sect. 2.

Within this context, this paper aims to provide a new methodology for assigning the DL to URM buildings accounting for both the entity and extension of damage at different scales (from the lower one, i.e. the panel scale, to the higher one, i.e. the global scale). The main strength of the proposed procedure regards the possibility of being applied consistently to both empirical/real data and numerical data (the latter obtained by nonlinear analyses). The methodology, as described in detail in Sect. 3, aims to minimize subjectivity thanks to a standardized and well-replicable procedure, that is inspired by the EMS-98 scale in terms of the damage classification (i.e. in five different levels) and damage extension. For the latter, additional analytical criteria have been proposed in the paper with respect to the qualitative definitions proposed in the EMS-98, with the main goal of making the procedure applicable also to the numerical context. At this stage of the research, the proposal is only focused on the damage to structural components. In fact, in the case of URM buildings, such elements are the most important in determining the overall damage (Ottonelli et al. 2020). This represents a relevant distinction from other structural typologies (like reinforced concrete), in which nonstructural components play a significant role, particularly for the slight to moderate DLs (e.g. as debated in Del Gaudio et al. 2017; Abrahamczyk et al. 2004; Al Hanoun et al. 2019; Schwarz et al. 2015).

The proposed procedure reflects the possibility to validate the numerical simulation through a parameter (i.e. the DL) that is directly comparable with the experimental evidence. Of course, numerical models are inevitably affected by both aleatory and epistemic uncertainties making it almost impossible to reproduce exactly the observed damage for each individual structural element. In this regard, the different scales established in the proposed procedure provide the opportunity to assess the reliability of the numerical simulation to different extent.

The applicability of the proposed method is demonstrated in the paper through a data set of four real seismic-damaged URM buildings, which are introduced in Sect. 4 with an overview of their geometric/mechanical features. As better defined in Sect. 4, these case studies were properly selected on the basis of: the very accurate documentation available on the damage; the limited uncertainty of the seismic input; and the possibility to replicate them also by other researchers through accurate models thanks to the additional data provided in other literature works. The buildings were selected also because they exhibited a prevailing global box-behaviour, dominated by the role of vertical walls and their in-plane (IP) damage. The possible role of diaphragms in defining the global DL is also discussed in the paper (see Sect. 3). The application of the procedure to the real case-studies is provided in Sect. 5. The potential of the proposed procedure for validation goals is demonstrated in Sect. 6, where numerical simulations of the four case studies were performed through an
Equivalent Frame (EF) modelling approach. Nonetheless, the proposed approach could be equally adopted in other numerical implementations (i.e. different EF methods, Finite or Discrete Element methods, as reviewed in D’Altri et al. (2020)).

2 Overview of the literature debate on the definition of damage levels

Evidence that the debate on the topic is still ongoing and no unanimously recognized scientific approaches are available can be inferred from: (i) the several damage conversion metrics adopted in the literature, for example, to interpret the large number of data coming from post-earthquake observation surveys (e.g. employed for the calibration of empirical fragility curves); and (ii) the various criteria developed to interpret the results of nonlinear dynamic analyses, aimed at associating specific damage levels to their numerical results (e.g. useful for developing fragility curves or performing seismic safety assessments).

Regarding the first field of evidence (i), on-site surveys that support decision-making processes about the safety of structures in post-earthquake scenarios usually adopt synthetic approaches, which collect basic information on single building components (i.e. walls, diaphragms, stairs, etc.) with the goal of assigning a usability rating rather than a global damage level. Examples of that are the AeDES form, proposed by Baggio et al. (2007) and extensively adopted in Italy since 2003, or the ATC-20 (2005), developed in the U.S.A. and used in many countries across the world.

Thus, damage conversion rules must be introduced to provide a proper conversion into a synthetic DL. Focusing on unreinforced masonry (URM) structures, as recently discussed also in Ioannou et al. (2021), the available methods differ in the considered component types and in the approach adopted to combine them. In particular, the method proposed by Dolce et al. (2017a, b, 2019) and implemented in the D.a.DO platform (http://egeos.eucentro.it/danno_osservato/web/danno_osservato) refers to the peak damage level observed on the vertical bearing walls, whereas the one proposed by Rota et al. (2008) and extended by Rosti et al. (2018) refers to the maximum DL observed among walls, floor diaphragms, and roof (without considering its extension). The choice adopted in Rota et al. (2008), to use the maximum damage among the structural components, was justified by the authors by the fact that it can be the main element influencing the evaluation of the building usability and its repair cost. However, despite the importance of considering more components, that choice appears not consistent in terms of structural vulnerability, especially if reference is made to the grades established by the EMS98, as the vertical elements may have a higher importance in the definition of the DL of the building. Indeed, it is tough to think that the collapse of a very small portion of diaphragms can significantly affect the vulnerability of the entire building and, therefore, the definition of the global DL when, instead, vertical elements overall suffer a low/medium DL. Recently, the method proposed in Lagomarsi no et al. (2021) proposed a weighted average of the damage observed in structural elements, taking into account their relative importance in the global estimation through proper weights (further differentiated based on the accuracy of the survey). However, when the damage is mainly concentrated on a specific structural element or the high damage is characterized by a low extension, that procedure may lead to an underestimation of the global DL (as discussed more in detail in Sect. 5.4).

Passing to the second field of evidence (ii), in the context of numerical approaches, the so-called Engineering Demand Parameters (EDPs) are usually introduced. They represent a measurable quantity that may be monitored by the numerical model (such as the roof
drift, interstorey drift ratio, peak floor accelerations, etc.). Their application in defining the attainment of given DLs necessitates the establishment of distinct thresholds aimed at representing the specific building performance. However, the definition of specific or single EDPs may not be exhaustive enough to represent the actual overall behavior of the structure whereas the definition of thresholds is characterized by a certain level of subjectivity, which represents a strong uncertainty in the seismic assessment. While literature works regarding reinforced concrete (RC) structures are quite unanimous in assuming the interstorey drift ratio (or at most the peak floor accelerations, for assessing the damage of acceleration sensitive non-structural components), in the case of URM structures, in the last two decades, a progressive evolution regarded the use of single EDP, like as interstorey drift ratio (e.g. as proposed in Calvi 1999), or combination of EDPs/conditions. For the latter case, Mouyiannou et al. (2014) tested various criteria to define DLs through nonlinear dynamic analyses (NDA) combining various approaches and variables differentiated as a function of the increasing level of damage severity (until DL3). Lagomarsino and Cattari (2015) and Marino et al. (2019) developed a multiscale approach that always combines in an integrated way different criteria and checks at various scales by checking the spread of damage in masonry elements (piers and spandrels), the response of masonry walls and the global behavior of the building (described by its pushover curve). More specifically, in (Marino et al. 2019) the concept of the so-called “minimum DL” was introduced to replace the adoption of the interstorey drift thresholds at the wall scale; in particular, such a proposal assigns a damage level to the wall based on the minimum damage level attained by all the elements of a certain level. Finally, in the RINTC project (Iervolino et al. 2018) for URM structures (Cattari et al. 2018), the usability-preventing damage was defined according to the achievement of specific levels of damage attained in masonry elements assessed by nonlinear analyses on the numerical model of the structure; then, in Lagomarsino et al. (2022) additional checks have been introduced to monitor also the damage occurring in diaphragms.

3 Proposed methodological criteria for interpreting the seismic damage

3.1 Overview on the proposed multi-scale approach

The proposed procedure aims to define the damage level of URM buildings struck by earthquakes as analytical as possible and to permit the application of a unique and consistent method for the interpretation of both observed and simulated damage.

In seismic risk analyses, the scenario of the built environment can be expressed in terms of Damage States (DS), representing a discrete qualitative description of the overall damage of structural and/or non-structural elements of a building that obviously progresses by increasing the seismic input level. From an engineering point of view, as discussed also in Lagomarsino and Cattari (2014), DLs conceptually represent the thresholds that separate various DSs. Figure 1 graphically shows this concept by referring to a pushover curve obtained from a nonlinear static analysis: the position of DLs on the curve has to be intended as purely indicative, although it wants to imply the repercussions that the attainment of a given DL (that corresponds to a certain severity and extension of cracks on the structure) usually has also on the overall structural capacity of the building in equilibrating the external seismic forces.
The proposed procedure consists of the definition of DLs—graduated into five levels as proposed in the EMS98 scale—starting from different level scales (i.e. at the panel-scale and macroelement scale, where the macroelement is intended as an assemblage of components like as vertical walls or diaphragms) with the ultimate goal of assigning the global damage to the investigated building (DLG). The damage at smaller scales (i.e. at panel or macroelement scales) is also set on five levels (i.e. DLP for panels, DLW for walls and DLD for diaphragms) although the physical meaning varies from one scale to another. The only one that strictly conceptually refers to the description of EMS98 is the global damage, DLG.

Usually, in the literature, the seismic response of URM buildings is ascribed to two main categories: the in-plane—IP—response of walls (see Fig. 1) and the occurrence of local mechanisms (mainly associated with out-of-plane—OOP—response of single parts, see Fig. 3).

Focusing on the IP response of masonry walls with openings, they can be firstly considered as an assemblage of two main structural components (panels): piers (i.e. vertical resistant elements) and spandrels (i.e. parts of walls between two vertically aligned openings, that couple piers as the beams in a RC frame). Indeed, the seismic damage observation shows that cracks and failure modes are mainly concentrated in those elements (see for example Augenti and Parisi 2010; D’Ayala and Paganoni 2011; Cattari et al. 2012; Penna et al. 2014; Sorrentino et al. 2019). Although the idealization of the wall in piers and spandrels is not strictly required to assign a DL from real observed data, it is convenient and effective for associating a clear engineering meaning to the crack propagation and, also, to the residual structural capacity of walls. The logical-scheme of the proposed procedure is depicted in Fig. 2 and is described in detail in the following Sects. 3.2 and 3.3. After the assignment of the DL at the panel-scale (I, DLP for piers and DLs for spandrels, respectively), one can define first the DL for each wall of the analyzed building (II, DLW for vertical walls and DLd for diaphragms, respectively) and, finally, that of the entire building (III, DLG), after having associated with each wall its relevance to the overall structural vulnerability.

Fig. 1 Example of the capacity curve of a URM building with the indication of both the damage states (DS) and damage levels (DL)
The proposed procedure is conceived to interpret the damage of buildings that mainly exhibited a box-behavior (i.e. dominated by IP response) and, therefore, with local failure mechanisms extended to a limited portion of the building or none at all. The damage produced by possible OOP failures of parts of the building insufficiently connected to the rest of the structure is herein simplistically reconducted to the attainment of the damage level 5 of piers and/or spandrels of such portions (namely $D_{L_p,5}$ and $D_{L_s,5}$, as clarified in Sect. 3.1). Indeed, the attribution of intermediate DL related to the OOP mechanism is difficult to provide, as that mechanism is mainly associated with the equilibrium condition and, therefore, to the entity of the damage rather than the progressive damage evolution or the attainment of specific mechanical strength limits. In particular, after the initial rocking phase of the panel, which can determine a low $D_{L_W}$ (up to 2), the achievement of a certain acceleration led to an almost sudden overturning phase, able to determine a very severe $D_{L_W}$ (from 3 to 5, that can be only conventionally differentiated); for example, according to the Italian Structural Code (NTC 2018; MIT 2019), $D_{L_W}=5$ can be assumed attained for $0.6 d_0$, where $d_0$ is the distance between the rotation point O and the centre of the mass. As an example, Fig. 3 shows the constitutive model adopted in (Angiolilli et al. 2021) on
the basis of the one proposed in (Lagomarsino 2015) for the characterization of the OOP mechanisms; that is also consistent with other literature proposals and recommendations of Codes (Sorrentino et al. 2017; Abrams et al. 2017; Degli Abbati et al. 2021). Moreover, the quality of masonry may significantly affect the morphology of mechanism (as discussed for example in De Felice (2011)), with possible disintegration of external leaves (in case of poor masonry) or ascribable to the rigid block idealisation; also, the type of structure details plays a crucial role as debated in D’Ayala (2013).

Other limitations of the proposed procedure regard the implicit inclusion of the damage contribution of diaphragms, roofs and not structural components in the attribution of the overall damage level of the building. The reasons that motivate these simplified assumptions are debated in depth in the following.

Firstly, the repercussion of neglecting the damage of diaphragms in the assignment procedure of the global DLg is discussed. To this aim, the damage data available in the Da.Do. platform and obtained for the municipalities struck by L’Aquila 2009 earthquake (Dolce et al. 2017a, b) has been investigated. It is worth noting that, such a database was produced for all the buildings for which the macroseismic intensity was higher than VI, as well as for buildings whose inspection was requested by the owners. In the present study, about 50,000 survey forms of residential masonry buildings were taken into account from the database; this subset has been selected by neglecting mixed structures (i.e. R.C.-masonry buildings). These data have been collected thanks to the availability of the AeDES forms (Baggio et al. 2007). Please take note that AeDES forms are not available for all historic Italian earthquakes within the Da.D.O. platform since its extensive use dates back to 1998. Among those available, the L’Aquila database has been selected since it is currently the most comprehensive for taking both regular and irregular masonry types into consideration as well as various diaphragm types; moreover, it is extremely robust as demonstrated by many studies that used it to derive empirical fragility curves (e.g. Rosti et al. 2021; Lagomarsino et al. 2021).

It is worth recalling that the AeDES form: a) contains a section addressed to provide basic information on the structural details (i.e. masonry quality – if regular of irregular, diaphragms type and presence of tie rods or RC tie beams); b) provides information on the damage differentiated for the vertical walls, diaphragms, roofs, stairs and infills. For each of these components, the damage is graded into three main grades (i.e. slight -D1, medium/severe—from D2 to D3 and very heavy- from D4 to D5) and three main levels of extension (i.e. < 1/3, 1/3–2/3, and > 2/3). The form doesn’t require the attribution of a final DLg, that instead would require the introduction of some damage conversion metric. In the examined stock, the “irregular” masonry, namely made of irregular texture or poor-quality masonry (i.e. rubble stone/fieldstone), were predominant (about 80% of the total number of the considered masonry buildings) as compared to “regular” masonry, which is characterized by regular layout and good-quality masonry (i.e. simple stone, massive stone, brick masonry). The types of horizontal structural systems are classified into vaults (with and without chains), flexible (e.g. wooden diaphragms), stiff (e.g. steel beams with hollow flat blocks), or rigid (e.g. reinforced concrete slabs). That distinction as well as the assignment of the grade/extension of the damage were carried out in the survey forms by technicians. The statistical analysis of the database is summarized in the pivot matrixes illustrated in Fig. 4a, b, in which a correlation between wall- and diaphragm- DL is reported. In particular, DLw was computed according to the Da.Do. criterion (Dolce et al. 2017a, b), whereas DLd was computed by considering the greatest DL that occurred for the building diaphragms and their corresponding extension. The damage of vertical walls has been directly summarized through the Da.Do approach to make more effective the correlation.
between vertical and horizontal components and condense some of multiple possible combinations (resulting from the extension and severity also on vertical walls); however, it has been verified that this choice doesn’t substantially alter any conclusion of the relationship between DLW and DLD. The effect of both the diaphragm type and masonry type is reported in Fig. 4a, b, respectively. One can see that the DL distribution among walls and diaphragms is very similar for vaults, flexible and stiff diaphragms because, despite the different possible types, a clear prevalence of irregular stone masonry can be noted for those cases. Indeed, the masonry type appears to be predominant in affecting the DL distribution among vertical and horizontal elements, as compared to the diaphragm type. For each diaphragm type, one can observe a very small percentage (<1%) of occurrence of severe (or very severe) DLD combined with slight (or null) DLW. On the contrary, one can observe that severe (or very severe) DLW occurs more frequently even if, at the same time, the DLD is slight (or null). Furthermore, for vaults, flexible, and stiff diaphragms, as well as, in general, for irregular masonries, a higher data percentage can be observed along the diagonal of the pivot matrixes. This trend highlights that, when a structure with those characteristics suffers severe or very severe global DL, the damage is almost equally distributed between the two structural elements and that the DL generated by diaphragm weaknesses is intrinsically considered in the DLW. Indeed, during seismic actions, in the case of total/partial collapse of diaphragms, the latter can no longer properly distribute the seismic inertial force to the walls, which are therefore subjected to a higher DL. Hence, when building surveys are carried out after a natural disaster, a greater DLW is expected to be surveyed in the case of diaphragm failures. Another important aspect regards the correlation of the DL between vertical and horizontal components in the case of buildings with rigid diaphragms. In particular, one can observe that its damage distribution among the structural elements is very similar to that of regular masonry (Fig. 4b, in which all the diaphragm typologies are considered), as that diaphragm type is mainly associated with regular masonry (about 70%)

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**Fig. 4** Relation between the observed DL computed for walls (according to the Da.Do. procedure) and floors (in terms of the maximum grade of damage and its corresponding extension) obtained by analysing the L’Aquila 2009 damage database: **a** effect of the floor types; **b** effect of the masonry types
and vice versa. In both the cases, the greatest DL is mainly focused on the wall. Indeed, one can see that a higher percentage of severe DLW is reached when DLD is null or at most slight.

Definitely, one can conclude that the DLW has a higher importance in the assignment procedure of DLG to the entire structure, as compared to the DLD. Indeed, the DLD is statistically lower or at most equal to the DLW. Thus, to consider also the DLD as an additional factor influencing the DLG may not be essential. Other possible procedures, such as assigning a DLG based on the maximum DL observed among walls, floor diaphragms, and roof (the latter can be considered simplistically as a further horizontal structural element) (as in Rosti et al. 2018), may be unnecessary and certainly not as robust as referring only to vertical walls due to the statistical trend observed and afore discussed. Hence, the DLD was not taken into account in the proposed procedure. However, further investigation should be performed on this critical aspect in future studies. Obviously, this does not mean that the diaphragm does not have an important role in the overall structural vulnerability of the building rather than structural repair costs. For example, the works done by Del Gaudio et al. (2019) and Rosti et al. (2021) and based on the empirical evidence of L’Aquila 2009 earthquake have highlighted that there is a general tendency for vulnerability to decrease with increasing diaphragm stiffness (provided the quality of masonry is above a minimum).

As far as the damage to the stairs is concerned, it was taken into account implicitly in the damage of the piers on which they were fixed.

Furthermore, the damage to non-structural elements (infill walls, false ceilings) was not considered because, although these elements can have a primary influence on the performance response of other structural typologies (e.g. infills for the structural operativity of reinforced concrete buildings, as discussed in Del Gaudio et al. 2017), their presence is limited in URM buildings and their damage does not significantly affect their overall DL, as recently highlighted in Ottonelli et al. (2020).

An important (and innovative) aspect of the proposed procedure consists of the attribution of a certain DLG also accounting for the possible concentration of the damage in a single direction of the building. Indeed, when that occurs, it is reasonable to attribute to that damage a strong impact on the overall capacity of the entire building in sustaining the external seismic actions that would be underestimated by considering an average DLG between both the building directions. This concept is in line with the procedure also adopted in the codes for the seismic safety verification of the building, for which the most limiting capacity curve computed between the two main directions of the building is assumed as reference. In line with that principle, the proposed procedure also suggests investigating the DLG separately for each building level (for a specific building direction) aiming to detect possible soft-story mechanisms that may significantly affect the structural vulnerability. The influence of the damage concentration in assigning the final DLG is considered only when a substantial to heavy damage can be identified for the investigated building (i.e. DLG > 2). In fact, damage localization to specific structural elements when DLG ≤ 2 is considered less robust in detecting the actual zone in which possible soft-story mechanisms may occur in a strong nonlinear phase.

### 3.2 Criteria for assigning the damage at the panel-scale

The present section defines the criteria adopted to assign the n-th damage level (n= {0, 1, . . . , 5}) on the j-th pier (DLP, j) and k-th spandrel (DLS, k) that compose a URM wall, mainly subjected to IP response. The adopted criteria summarized in Table 1 also consider the
| $n$ | Piers (DL$_{P,j}$) | Compressive—Flexural failure (Fig. 5b) | Spandrels (DL$_{S,k}$) (Fig. 5c) |
|-----|------------------|----------------------------------------|-------------------------------|
| 0   | No cracks        | No cracks                              | Low crack-band at the end elements       |
| 1   | Low crack-band at the end elements | Low crack-band at the end elements       | Larger cracks at the end elements or occurrence of shear failure in the central part of the element (achievement of the strength for that portion of the masonry) or diagonal crack that occurred at the center of the panel (achievement of the maximum strength) |
| 2   | Slight diagonal crack ($< 1$ mm) at the center of the panel (<em>achievement of the maximum strength</em>) | More extended crack-band that occurred on the mortar joint at the end elements | Significative degradation phase that occurred for the masonry portion; the residual resistance is related to the lintel capacity |
| 3   | Diagonal crack propagation with openings having up to 10 mm (<em>beginning of the softening behavior</em>) | Significative diagonal cracks with significative crack opening, permanent plastic deformations and/or spalling phenomenon (<em>very limited residual capacity to equilibrate horizontal forces</em>) | Crushing at the compressed corner |
| 4   | Partial collapse of portions of the structural element (<em>almost null residual capacity to equilibrate horizontal forces and close to the loss of vertical-load-bearing capacity</em>) | Partial collapse of portions of the structural element (<em>close to the loss of vertical load-bearing capacity</em>) | Lintel collapse |
experimental evidence that indicates different behaviors in terms of strength degradation and crack development, varying: (i) the main failure mode (shear or compressive-flexural); (ii) the element type, i.e. piers or spandrels.

As far as the issue (i) concerns, focusing attention on piers, the compressive-flexural behavior may involve two different failure modes. If the applied vertical load is low with respect to compressive strength (the most frequent case), the horizontal load produces tensile flexural cracking at the corners and the pier begins to behave as a nearly rigid body rotating about the toe (rocking). If no significant flexural cracking occurs, due to a high applied vertical load, the pier is progressively characterized by a widespread damage pattern, with sub-vertical cracks oriented towards the more compressed corner (crushing). In both cases, the ultimate limit state is obtained by failure at the compressed corners. While the latter condition is typically fulfilled in laboratory experiments, in actual damage only cracks caused by the opening of horizontal mortar joints are typically noticeable (Fig. 5b). It is rather uncommon to observe the total crushing of the compressed toe. In the case of shear, damage is usually differentiated into: sliding shear failure, mainly associated with the development of cracks on horizontal mortar joints, possibly with permanent plastic deformation; and diagonal cracking, with the formation of a diagonal crack, which usually develops at the centre of the pier and then propagates towards the corners. The crack may pass prevailing through mortar joints assuming the shape of a ‘stair-stepped’ path in the case of a regular masonry pattern (type A in Fig. 5a), or through the blocks or both components (type B in Fig. 5a), varying their relative mechanical properties and the masonry bond. Actually, in real buildings, the sliding shear failure modes are quite rare to be observed as compared with the diagonal cracking: this is the reason because only the latter one has been illustrated in Fig. 5. Of course, many factors (e.g. the slenderness, the boundary conditions, the axial load, the mechanical properties of units and mortar joints, …) influence the actual dominant failure mode in the panel.

Fig. 5 Interpretation of the damage at the panel scale (ideal backbone curve and type/extension of damage) for: a) squat pier elements under compressive-flexural action generating diagonal cracks; b) slender pier elements under compressive-flexural action; c) spandrel elements
As far the issue (ii) concerns, experimental tests revealed that the spandrel behavior is affected by other factors not pertinent for piers, for instance, the type of architrave and the interlocking phenomena at end sections (i.e. Beyer 2012; Beyer and Dazio 2012; Parisi et al. 2013, 2014). These factors slightly alter the layout of cracks with respect to piers. In particular, in the case of compressive-flexural behavior, the cracks at the end section cannot follow a rigorous vertical path due to the interlocking among units (thus, cracks pass through the bed and head joints or through the units according to types A and B in Fig. 5c) and, for the same reason, the sliding is not likely to occur. Moreover, it should be noted that the term "lintel" was used in Table 1 to refer to any kind of structural component intended to support the spandrel portion. It might actually be a real lintel (made of wood, steel, or R.C. beam), or it might even be a flat or segmental arch. In fact, according to experimental tests made by, for example, Beyer and Dazio (2012), the inclusion of a chain or curb can change the failure preponderance in flexural or shear mechanisms, but not substantially the phenomenology of the cracks (to which the current work was focused on).

In all cases, the severity of damage may be ascribed to the opening of cracks (e.g. following the indication of the user manual of the AeDES form in Baggio et al. (2007)) and to the fact that they involve both sides of the panel (as usually systematically occurs from the damage level 3) or not. It is worth noting that, despite the criteria listed in Table 1, in some cases (i.e. DL = 2 and DL = 3 for piers characterized by compressive-flexural failure as well as DL = 3 and DL = 4 for spandrels) it is difficult to really differentiate subsequent damage levels in the observational field. Moreover, despite the convenience in such a classification of damage modes, it is obvious that in the reality hybrid failure mechanisms often occur and it is much more difficult to clearly distinguish one mechanism from the other at their early stage of activation (namely, in DL1).

The actual structural capacity of the element in carrying both the horizontal and vertical loads may be then related to the progression of nonlinear response, namely the spreading and worsening of cracks. Experimental evidence on piers (e.g. Augenti et al. 2011; Bosiljkov et al. 2003; Petry and Beyer 2014; Kržan et al. 2015; Vanin et al. 2017; Rezaie et al. 2020; Morandi et al. 2018; Boschi et al. 2019)) and spandrels (Gattesco et al. 2008, 2016; Parisi et al. 2014; Calderoni et al. 2011; Beyer and Dazio 2012; Knox 2012; Graziotti et al. 2012) testified a quite pronounced softening phase in the case of diagonal cracking (much or less sudden depending on the masonry type), as well as a not so degrading response in the case of the flexural-compressive failure of piers. In the case of the flexural-compressive failure, the degrading phase is associated with the crushing of the corner (in presence of high axial loads or low compressive strength of masonry) or to second order phenomena (for very high horizontal displacements, usually not attained in the seismic response of existing buildings). In the case of spandrels, the softening phase is basically governed by the residual strength provided by the architrave (Beyer 2012) and not so huge differences with respect to piers are obtained by passing from the flexural to the shear response. Recently, in Rezaie et al. (2022) the use of simple machine learning models was explored to predict the degradation of stiffness, strength and displacement capacity on piers as a function of the observed crack features; that may constitute an interesting direction in the use of image-data for the automated post-earthquake assessment of URM buildings.

Figure 5 shows in a qualitative way such types of behavior in terms of both ideal backbone curves of a cyclic response as well as the type and crack width for the various DLs.

According to the common approach proposed in the literature (as set in Vanin et al. 2017; Morandi et al. 2018; Rezaie et al. 2020 based on a huge dataset of experimental tests) and codes (ASCE 2017; CEN 2004; MIT 2019), the transition from moderate to very severe damage levels is regulated in mechanical models by introducing proper drift
thresholds. More specifically, in the aforementioned documents reference is usually made on specific conditions easily measurable from experimental tests (e.g. the drift associated with the base shear decay equal to 20% or 50%) more than to specific damage levels. With respect to that, the only document that establishes a precise correspondence among the latter ones is the DT212 (2013) issued by the Council of National Research in Italy.

3.3 Criteria for assigning the damage at the wall-scale

The damage level for each \( i \)-th wall (\( DL_{W,i} \)) of the building is then computed as a combination of damage to each structural element composing it (\( DL_{P,j(i)} \) and \( DL_{S,k(i)} \)).

In particular, the contribution of the two panel types was computed by considering their relative importance in the seismic response of the wall, i.e. considering that piers are the main resistant elements carrying both vertical and lateral loads, whereas spandrels are usually considered as secondary elements affecting the boundary conditions of piers (by allowing or restraining end rotations). The overall stiffness, base shear, and ductility of the building may change as a result of the resulting static scheme of piers (which obviously may change during the seismic event due to progressing damage in spandrels). By way of example, Fig. 6 shows the pushover curves obtained for two structures varying the structural configurations of spandrels, namely: (A) spandrels not coupled to any tensile resistant horizontal element at floor level; (B) spandrels in presence of horizontal steel tie rods; (C) spandrels coupled to reinforced concrete (r.c.) tie beams; (D) shear-type ideal scheme. For further details, the interested readers may refer to Cattari and Magenes (2022). Although such an important role must be recognized for spandrels, the final capacity of the building to equilibrate the horizontal forces produced by the earthquake is ascribable to piers. This is why for the definition of \( DL_{W,i} \) it appears consistent to assume higher importance in the damage of such elements rather than that of spandrels. More specifically, in this study, the damage level that occurred in the spandrels (\( DL_{S,k} \)) was treated as an additional factor influencing the \( DL_{W,i} \) through the application of a weight parameter \( \alpha_S \). Here, for each \( i \)-th wall, \( \alpha_{Si} \) was defined as the ratio between the surface area of the spandrels (\( A_s \)) and that of the wall (\( A_W \)): this parameter represents the rate that the possible damage to spandrels has with respect to that of the entire wall. The computation of the spandrel area requires to assume a geometry for such elements; usually, it is defined on the basis of the overlapping of two vertically aligned openings (Lagomarsino et al. 2013), as also indicated in Fig. 7

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**Fig. 6** Influence of the structural details on the pushover curve passing from A to D configuration for two complex structures inspired by the Visso’s and Pizzoli’s buildings analyzed in Sect. 6 (results taken from Cattari and Magenes 2022)
that depicts the case of $\alpha_{S,i} = 0.2$. The more the opening layout is regular and the more the number of spandrels increases, and the more the role of spandrels becomes relevant in affecting the structural capacity of the wall (as evident also from Fig. 6 considering the distance from cases A and C/D, respectively, varying the building configuration). Moreover, in the presence of tensile resistant elements coupled to spandrels (such as tie rods or reinforced concrete tie beams), the strength capacity of spandrels increases reducing their potential to be damaged; in these cases, $\alpha_{S,i}$ can be assumed in a simplified way as null.

Based on the above considerations, the average damage of each $i$-th wall ($DL_{W,i}$) is herein estimated as:

$$DL_{W,i} = DL_{P,i} + \alpha_{S,i} DL_{S,i} \leq 5$$  \hspace{1cm} (1)

where $DL_{P,i}$ and $DL_{S,i}$ consist in the overall damage contribution provided to the wall by all piers and spandrels that compose it. They are computed as:

$$DL_{P,i} = \sum_{j=0}^{\eta_{P(i)}} \left( \frac{A_{P,j(i)} DL_{P,j(i)}}{A_{P,tot(i)}} \right)$$  \hspace{1cm} (2)

$$DL_{S,i} = \sum_{k=0}^{\eta_{S(i)}} \left( \frac{DL_{S,k(i)}}{n_{S(i)}} \right)$$  \hspace{1cm} (3)

where $A_{P,j(i)}$ is the area of the cross-section of $j$-th piers within the $i$-th wall, $A_{P,tot(i)}$ is the total area of the pier cross-sections, $n_{P(i)}$ and $n_{S(i)}$ are the total number of the pier and spandrel elements, respectively. In practice, the damage contribution of piers is weighted by accounting for their horizontal cross-sectional resistant area (conventionally assumed as representative of their potential role in equilibrating the external forces), while the damage of spandrels is simply computed as the average with respect to the number of these elements. This approach is consistent with other literature proposals as described in DT 212 (2013), Lagomarsino and Cattari (2015), Mouyiannou et al. (2014) and Lagomarsino et al. (2022).

![Fig. 7 Definition of the spandrel- and wall-surface areas for the computation of $\alpha_{S,i}$](image)
The upper limit equal to 5 in Eq. (1) is considered in the case of severe damage to both piers and spandrels. The adequacy of such an approach is further corroborated by the application to real buildings illustrated in Sect. 5.

Thus far, the above-described criteria could not consider a possible soft-story mechanism, which occurs when the damage is concentrated on a specific building level. However, the activation of soft-storey mechanisms could be reasonably associated with a higher DL of the entire building. Hence, to overcome this drawback, a useful rule acting on wall damage at the interstorey level can be considered, monitoring the mean damage of piers at the $L$-th level, as follows:

$$DL_{W,Li} = \frac{\sum_{j=0}^{n_{p,L}} \left( A_{p,Lj(i)} DL_{p,Lj(i)} \right)}{A_{p,Ltot(i)}} + \alpha \frac{\sum_{k=0}^{n_{S,L}} \left( DL_{S,Lk(i)} \right)}{n_{S,L}(i)} \tag{4}$$

where $A_{p,Lj}$ is the sectional area of the $j$-th piers and $L$-th level, $A_{p,Ltot}$ is the total area of the piers at the $L$-th level, $n_{p,L}$ and $n_{S,L}$ are the total number of the pier and spandrel elements at the $L$-th level, respectively.

### 3.4 Criteria for assigning the damage at the global scale

The last step involves, firstly, the combination of damage levels attributed to lower scales to pass to the global one and, secondly, the establishing of a clear correspondence with the EM98 damage grades. As summarized in the first three columns of Table 2, the EMS-98 supplies a linguistic description of the expected damage and its extension to the building. In particular, the extension is graduated through the following adjectives associated with the walls: “very few”, “many”, “most”. The description of each grade is then completed by the expected severity of cracks, ranging from “hair-line”, “large and extensive”, “serious” cracks until the “serious failure” and “collapse”. The approach adopted in this study numerically interprets such linguistic descriptions. In particular, according also to the fuzzy set theory, the qualitative definitions associated with the extension can be interpreted through a membership function (as also originally introduced in EMS98 and discussed in detail in Lagomarsino and Giovinazzi (2006)) leading to the following ranges of conversion: “very few” up to 10–20%; “many” from 15% up to 50%-60%; “most” over 55%. Even though they presuppose some overlapping, they constitute a robust reference useful to introduce the explicit statistical meaning of the occurrence of a specific damage. Instead, concerning the severity of cracks, the correspondence is established between the adjective used in the description of each EMS98 damage grade and the corresponding “main” damage level expected at wall scale, namely: “hair-line” to $DL_{W} = 1$; cracks to $DL_{W} = 2$; “large and extensive” to $DL_{W} = 3$; “serious failure” to $DL_{W} = 4$; “collapse” to $DL_{W} = 5$. The use of “main” refers to the fact that it cannot be excluded the presence even of higher severe cracks in the walls but localized in a smaller percentage with respect to that associated with the adjective used in the corresponding damage grade for the extension. For example, in the case of damage grade 2 that refers to the presence of “cracks in many walls”, it cannot be excluded that “large and extensive” cracks are present in “few” walls. Furthermore, the ranges of conversion values reported in Table 2 do not consider range overlapping to avoid ambiguity especially when the proposed procedure is applied for the investigation of the simulated damage. For example, the upper limit of the second condition for $DL_{G} = 1$—and consequently the limit of the first condition for $DL_{G} = 2$—was assumed 0.3 to represent the central value of the extension “many” defined by the EMS98.
Table 2  Definition of the damage at the global scale from the EMS98 criteria (l = or; BN = but not) only associated with the structural vertical elements

| DL\(_G\)  | EMS98—description and quantification            | Proposed conversion criteria to assign DL\(_G\) |
|---------|-----------------------------------------------|-----------------------------------------------|
| 1       | Negligible to slight damage (no structural damage, slight non-structural damage) | \(\sum_{DL_{W}} \geq 1 \geq 0.2\) l  
|         | Hair-line cracks in a very few walls          | \(0.1 \leq \sum_{DL_{W}} \leq 0.30\)  
|         | \(\sum_{DL_{W}} \leq 3 < 0.50\)  
|         | BN \(\sum_{DL_{W}} \geq 4 > 0\)  
| 2       | Moderate damage (slight structural damage, moderate non-structural damage) | \(\sum_{DL_{W}} \geq 2 \geq 0.30\) l  
|         | Cracks in many walls                         | \(0 \leq \sum_{DL_{W}} \leq 0.50\)  
|         | \(\sum_{DL_{W}} \leq 4 < 0.30\)  
|         | BN \(\sum_{DL_{W}} \geq 5 > 0\)  
| 3       | Substantial to heavy damage (moderate structural damage, heavy non-structural damage) | \(\sum_{DL_{W}} \geq 3 \geq 0.50\) l  
|         | Large and extensive cracks in most walls     | \(0 \leq \sum_{DL_{W}} \leq 0.30\)  
|         | \(\sum_{DL_{W}} \leq 5 < 0.50\)  
| 4       | Very heavy damage (heavy structural damage, very heavy non-structural damage) | \(\sum_{DL_{W}} \geq 4 \geq 0.30\) l  
|         | Serious failure of walls                     | \(0.2 \leq \sum_{DL_{W}} \leq 0.50\)  
| 5       | Destruction (very heavy structural damage)   | \(\sum_{DL_{W}} \geq 5 \geq 0.50\)  |


Based on these considerations, the conversion criteria proposed in the last column of Table 2 have been introduced; the upper and the lower limits merely represent a conversion of the EMS98 extension, considering also that the DL aims to establish a transition from a damage state and the next one. As clarified in the table, to monitor the extension of damage reference is made to the cumulative rate of walls that attained a specific damage level. To this aim, it is quite convenient—not to say necessary—to convert the continuous values of \( DL_{Wi} \) (see Sect. 3.2) to discrete values \( DL_{Wi}^* \) (the * symbol indicates the integer-discretized version of the variable). In particular, in this study, it is proposed to adopt the binomial distribution (Lagomarsino and Giovannazzi 2006), assuming the discretization criteria reported in Table 3.

Finally, it must be observed that the shear capacity of each masonry wall is different to each other because of their different section-area and material properties. As a first attempt, conventional criteria, based on the qualitative estimate of the sectional area of piers that compose each wall, may be adopted to include this effect in the evaluation of the cumulative rate. Moreover, the weight assigned to wall may represent a useful artifice to account for the role of possible torsional effects or discontinuity able to produce a concentration of significant damage, thus making it worthwhile to be considered with particular attention in the DL assessment. Conversely, if a structural model is available for the building under investigation (as in the examined cases, proposed as real benchmark to validate numerical models), specific “weights” \( W_{wi} \) may be derived from the results of the nonlinear static analyses (NSAs), i.e. as the ratio between the shear base reaction of the \( i \)-th wall \( V_{Wi} \) and the total base shear reaction \( V_{TOT} \). It is worth noting that a slight redistribution of the base shear of the walls can occur also in the initial phase of the NSA due to the crack propagation in the elements, as observed in Fig. 8. Therefore, in this study, the \( W_{wi} \) parameters were computed at the initial phase of the NSAs, considering the average values between the results of the analyses performed through the application of uniform load pattern distributions proportional to the structural mass in negative and positive directions. However, other procedures can be assumed as well without losing the generality of the proposed approach.

At this point one can sum, for each \( i \)-th wall within a specific direction of the building, the \( W_{wi} \) for which a certain \( DL_{wi}^* = n \) (being \( n = \{0,1,…,5\} \)) occurred aiming to compute the percentage of occurrence (extension) of a specific \( DL_n \) (as also depicted in the logical scheme of Fig. 3), as follows:

\[
DL_n = \sum_{i=1}^{i} W_{wi}(DL_{wi},i^* = n)
\]

| \( DL_{wi}^* \) | Adopted criterion |
|-----------------|-------------------|
| 1               | \( 0.7 < DL_{wi},i \leq 1.6 \) |
| 2               | \( 1.6 < DL_{wi},i \leq 2.5 \) |
| 3               | \( 2.5 < DL_{wi},i \leq 3.4 \) |
| 4               | \( 3.4 < DL_{wi},i \leq 4.3 \) |
| 5               | \( 4.3 < DL_{wi},i \leq 5 \) |

Table 3 Criteria to convert \( DL_{wi} \) into discrete values \( (DL_{wi}^*) \)
The latter criterion allows to refine the evaluation of the cumulative rate of walls which attained a specific damage level and thus to apply the criteria illustrated in Table 2.

4 Data set of the real damaged buildings

4.1 Selection criteria for the investigated buildings

This section defines the criteria adopted to select the four URM buildings struck by real earthquakes as references to apply the proposed procedure. The case studies were selected following specific requirements mainly related to the completeness and accuracy of available data and the opportunity to provide as a whole a comprehensive example of diversified damage levels.

For all the case studies, very accurate documentation on the real damage as well as geometric and mechanical features of the buildings was available thanks to full on-site surveys and/or available documents. More specifically, one can assume a degree of completion of the survey equal to 100% for all the external walls of all the selected buildings. As far as the damage identification of the internal walls is concerned, for the San Felice’s and Caldarola’s buildings was not possible to survey the attic and the upper level, respectively. For this reason, one can assume a degree of completion of the survey equal to 80% and 90% for those case studies. For the other two case studies, the identification of the real damage was very accurate (100%) also for the internal walls. Hence, it was possible to assign for each structural element (piers and spandrels) a reliable damage level, according to the criteria established in Sect. 4.1. Obviously, reliability on the knowledge about the main characteristics of the case studies is characterized by inevitable uncertainty in the survey procedure, data handling etc.

It is worth noting that, aiming also to reduce the uncertainty in the numerical models and, therefore, increase the reliability of the simulations, structures tested by shaking tables...
may represent an ideal case to be considered, as all information is known with extreme precision (seismic input, structural details, weight of the structure, etc.). However, such cases can represent only simplified configurations (i.e. low number of walls, maximum weight of the structure, etc.) if compared to those of the real cases. For these reasons, they were not considered in this study.

Furthermore, all the case studies were selected because they showed a global response (the so-called “box behavior”) during the earthquakes, with a substantial activation of the in-plane behavior of the load-bearing walls, presenting negligible local mechanism failures.

The four case studies were also selected because they were characterized by different damage levels, from slight to very severe damage. Considering a wide damage level scenario was a fundamental aspect, on the one hand, to more robustly test the proposed procedure, and, on the other hand, to take into account for the validation of the numerical simulation, as the latter usually tends to overestimate the real damage especially when seismic input is characterized by low intensity.

Moreover, although the four selected buildings cannot be exhaustive of all possible masonry types and architectural configurations, they are emblematic examples of masonry typologies quite recurring in the existing heritage, at least in Italy and European countries, namely from irregular stone masonry (i.e. fieldstone) or manufactured stone (with horizontal courses) to unreinforced brick.

Finally, since the uncertainty on the seismic input may affect the dynamic response of the structures as well as the damage propagation on the structural elements, all the case studies presented a high reliability on the seismic input, too. A case study (the San Felice’s one, that was struck during the Emilia 2012 earthquake) was chosen because it was very close (less than 300 m) from the National Accelerometric Network (RAN) seismic registration. Other two case studies (Visso and Pizzoli, struck during the Central Italy 2016/2017 earthquake) were instrumented by a permanent accelerometric monitoring system (Dolce et al. 2017b; Cattari et al. 2019). In particular, the Visso’s building represents an interesting case for which a detailed study on the site and soil-foundation interaction effects was already investigated in (Brunelli, et al., 2021). The last case study (the Caldarola’s one, that was struck during the Central Italy 2016/2017 earthquake) was characterized by the absence of accelerometric stations placed in the municipality as well as the absence of a permanent monitoring system during the seismic event. However, some study on the site amplifications was already conducted in Cattari et al. (2022a) and the data from the seismic microzonation study were available (MZS3 2018), ensuring a good potential to well characterize the seismic input.

4.2 Main structural and geometrical features of the selected buildings

The four case studies were built around 1920 and 1960 and present similar features (Fig. 9): they are all masonry structures, quite regular in elevation and with continuous masonry piers and openings generally vertically aligned. The buildings are instead characterized by different plan configurations, as defined in Table 4, in which are also summarized the general and other geometrical information of the four cases. The detailed description (geometry, material and observed damage) of the San Felice’s building, Visso’s school and Pizzoli’s town hall and Caldarola’s school are reported in Cattari and Lagomarsino (2013), Brunelli et al. (2021), Degli Abbati et al. (2022), Cattari et al. (2022a), respectively. In addition, the San Felice’s building was selected as a case study in the DT 212 (2013), while Visso’s school and Pizzoli’s town hall were included in the benchmark cases analyzed in
the “Nonlinear URM modelling – Benchmark Project” (Cattari and Magenes 2022) where the results achieved by several modelling strategies were compared in Ottonelli et al. (2022), for the Visso’s school, and in Degli Abbatì et al. (2022), for the Pizzoli’s town hall. The data contained in the aforementioned documents make feasible to replicate the modelling of these buildings also to other interested researchers. Furthermore, Table 5 summarizes the structural details of the four analyzed cases.

The San Felice’s structure (Fig. 9a) is a residential building constructed in the ‘60 s and located in the North-Central Italian area. Due to its geometrical configuration and adopted material, the building can be considered as representative of the existing masonry stock of the entire area that suffered the highest damage after the 2012 Emilia earthquakes (Mucciarelli and Liberatore, 2014). The other three case studies represent the existing masonry heritage aimed at schools or municipal buildings, presenting moderate—(Caldarola’s case of Fig. 9b) or significative—interventions (Visso’s and Pizzoli’s cases of Fig. 9c, d, respectively) during the years. In particular, for the Pizzoli’s case (see details of the masonry type in the infrared thermography reported in Fig. 10d that was necessary to individuate the masonry type due to the plaster), some bearing walls were demolished and some new partitions were constructed. For the Visso’s case, some extensive interventions were provided after the 1997 Umbria-Marche earthquake sequence, that mainly consisted of: mortar injections through some internal walls; stiffening of front openings with steel bracings opened at the bottom; insertion of some tie-rods, improvement of the walls-to-roof connection; and replacement of part of the timber roof structure, deteriorated due to aging. It is worth noting that all the analysed buildings were characterized by almost rigid floor diaphragms despite they were characterized by different typologies.

All the case studies mainly showed a global structural response during the seismic events, favored also by a quite good wall-to-wall and wall-to-diaphragms connection and quite stiff diaphragms able to ensure a redistribution of actions among walls. Moreover, also the quality of masonry was good (Fig. 10) thanks also to a general good transversal
| Case study | Seismic event | Building age | Num. floors | Plan configuration | Interstory height (m) | Dimensions (m) |
|------------|---------------|--------------|-------------|-------------------|----------------------|----------------|
| San Felice | 2012 Emilia   | 1960s        | 2A + 1A     | Squared           | 3.9–3.9×2.6          | 11.8 × 12.4 |
| Pizzoli    | 2016–17 Central Italy | 1920s | 1U + 2I     | Elongated rectangular with small wings | 4.1–3.4–3.6 | 37 × 12 |
| Visso      | 2016–17 Central Italy | 1930s | 1U + 3I     | “T-shape”         | 2.8–4.15–4.15–2.0   | 40 × 24 |
| Caldarola  | 2016–17 Central Italy | 1930s | 2A + 1A     | Elongated rectangular with small wings | 4.5–3.5–3.0 | 26 × 58 |

U = underground level, I = intermediate level, A = attic floor
### Table 5  Structural details of the four case studies

| Case study | Masonry type                  | Floor slab                                      | R.C. curbs      | Chains  | Lintels          | Roof                               |
|------------|--------------------------------|------------------------------------------------|-----------------|---------|------------------|------------------------------------|
| San Felice | regular clay brick; one-leaf   | Steel beams and hollow flat blocks              | No              | No      | Clay brick       | Wooden beams                       |
| Pizzoli    | cut stone with horizontal courses | Steel beams and hollow flat blocks with R.C. infill | R.C. beams**    | Sporadic* |                  | R.C. beams                         |
| Visso      | hewn stone; two-leaf           | One-way or two-way hollow block slab            | Yes; R.C. beams | Sporadic | Flat arch; clay brick | Wooden beams                       |
| Caldarola  | irregular stone; two-leaf      | Thin vaults (L0-L1) and steel beams + R.C. and hollow tiles mixed floor (L2) | L-shaped steel elements on the back facade | Sporadic | Flat arch; clay brick | R.C. beams with steel-reinforced hollow flat blocks |

*Not verifiable, **presumable, but not directly ascertained
connection between wall facings, apart the Caldarola’s case study characterized by a very poor adherence between units and mortar and two-leaf not connected (Fig. 10b/c).

However, it is worth specifying that: partial OOP collapse involved only some perimeter walls of the Visso’s school due to the activation of the overturning mechanism during the second mainshock of 26th October 2016; a partial collapse of the external facing in the top part of one of two small wings of the Caldarola’s school occurred (visible also in (Fig. 9b). Indeed, Visso’s school was severely and repeatedly struck by subsequent seismic events: the damage was first produced by the 24 August 2016 shock and was further aggravated by the 26 and 30 October shocks. For this reason, the Visso’s case study can represent an emblematic example of structural damage accumulation.

5 Application of the proposed procedure for interpreting the real damage

5.1 Description of the observed damage

As already commented in Sect. 4, all the analyzed structures mainly showed a box behavior during the seismic events, with out-of-plane mechanisms eventually limited to small portions.

In particular, the S. Felice’s building showed severe damage concentrated on the spandrels, as illustrated in Fig. 11a/b. Only a slight-moderate damage was instead observed for
some pier. Furthermore, it was observed a heavy damage level in a small portion of the floor, as illustrated in Fig. 11h. In general, that building was interested by a moderate damage level.

The Pizzoli’s building exhibited a slight/moderate damage after the 2016/2017 Central Italy earthquake, in particular referring to the event of 18th January 2017. The damage mainly occurred on pier elements of the ground floor; both slight diagonal/flexural cracks occurred on these elements. An almost horizontal crack also occurred on the upper level of the lateral side of the building, involving both pier and spandrel elements.

For the Visso’s building, a very heavy damage was observed after the third event (30th October 2017), including the partial collapse of a façade that contributed to that of the upper floors. Damage was mainly concentrated in pier elements by the occurrence of significant diagonal cracks (Fig. 11f/g). However, a moderate damage occurred also in the spandrels (see Fig. 11c). Furthermore, some floors of both the ground and first levels suffered a partial collapse (see Fig. 11j). The high damage level observed for the
building was influenced by damage accumulation phenomena and it was also related to site amplification phenomena, as already investigated in Brunelli et al. (2021), because of the poor properties of the soil present in that area.

The Caldarola’s building showed a damage concentration on the structural elements of the first floor with severe diagonal cracks (Fig. 11d/e). This occurred for all the external piers oriented in X direction. Instead, only slight diagonal cracks developed on the piers of the ground floor. Furthermore, no damage was observed for the spandrels located along the perimetrical walls, whereas moderate damage was observed for some spandrels located in the halls and corridors. As far as the horizontal structures are concerned, cracks and detaches involve only the external plaster of the thin vaults of diaphragms (Fig. 11k). The few sub-horizontal cracks at the floor-to-wall interface occurred because of the sliding mechanism of the rear wall.

Table 6 summarizes both the level and extension of the apparent damage observed for each case study.

In particular, the damage is classified separately for the different structural components, such as piers, spandrels, floors, and roofs as well as non-structural elements (infills and partitions) and it is represented by three levels (slight: D1, severe: D2–D3, and very severe: D4–D5) as indicated by the AeDES form. As better clarified in Sect. 5.4, this damage classification will turn out very useful to compare the results with other damage metrics available in the literature and, anyhow, it is compatible with the meaning introduced in Sect. 3.1. The extension of that damage grade with reference to the entire building is also reported following the AeDES indications (i.e. < 1/3, 1/3–2/3, and > 2/3). It is worth noting that, the AeDES form does not classify separately the damage that occurred for piers and spandrels but considers the average damage that occurred for the entire wall. Furthermore, the damage that occurred for the stair components was directly computed within the damage observed for the pier elements. When interpreting the global damage level of the investigated buildings, as motivated in Sect. 4, only structural damage was considered, ignoring at this stage non-structural damage. However, for sake of completeness, it is mentioned that only a few partitions located at the ground floor of the Caldarola’s building collapsed during the earthquake.

| Case study | Piers | Spandrels | Floors | Roof | Non-structural elements |
|------------|-------|-----------|--------|------|-------------------------|
| San Felice | Slight (1/3–2/3) | Slight (< 1/3) | Slight (1/3–2/3) | Slight (> 2/3) | – |
| Pizzoli | Slight (1/3–2/3) | Slight (< 1/3) | Slight (1/3–2/3) | Slight (> 2/3) | – |
| Visso | Slight (< 1/3) | Slight (< 1/3) | Slight (1/3–2/3) | Slight (> 2/3) | – |
| Caldarola | Slight (< 1/3) | Slight (< 1/3) | Slight (> 2/3) | – | Very severe (< 1/3) |
whereas the severe damage for nonstructural elements of the buildings in Visso and Pizzoli regards the partition walls (see Table 6).

5.2 Attribution of the damage level to the panels

The damage pattern observed for the four case studies is depicted on the architectural plan views of Fig. 12, by interpreting the DL through the criteria described in Sect. 3.2. Moreover, Fig. 13 focuses on the damage that occurred on the piers (i.e. DLp). From these figures it is evident that the damage to the S. Felice’s case was concentrated on the first two floors of the building and mainly in the X direction. For the Pizzoli’s case the damage was concentrated almost similarly on both the levels but especially along the Y direction. For the Visso’s case the pier elements placed along the Y direction suffered a higher damage level than the X direction. Finally, for the Caldarola’s case the damage was mainly concentrated on the piers at the first floor.

5.3 Assignment of the global damage level from the damage walls

Figure 14a–d summarizes from a quantitative point of view all the information about the damage level that occurred in the walls as well as that in piers and spandrels for the four case studies. According to the procedure described in Sect. 3, the first column of Fig. 14 represents the damage to each wall DLWi (in blue) and their correspondent weights WW,i (in orange). This combined representation is useful because it clarifies the effective importance of the specific damage that occurred on each wall in contributing to that at the global scale of the building. The horizontal dotted lines indicate the sum of the DLWi multiplied by WW,i computed for the walls along the two directions (X and Y).

These values can also represent an alternative way to obtain the DL at the global scale, without the need of adopting a conversion criterion to transform the DLs from continuous to discrete values in the first part of the data processing, i.e. in the assignment of damage at the wall scale, with the consequent risk to slightly alter the results in the definition of the global damage level. However, that alternative procedure is investigated only for a comparative purpose with the results obtained by the procedure described in Sect. 3.1, which is assumed to be more consistent with the EMS98 criteria. Then, the second and third columns of Fig. 14 compare the extension of a specific DL that occurred on the wall (in blue), as a combination of the pier and spandrel damage levels, and the DL that occurred for the individual elements (in orange for the piers and in green for the spandrels) in the two main directions of the buildings (X and Y), respectively. Obviously, the extension of the DL computed for the walls is more similar to that computed for the piers rather than that computed for the spandrels because of the assumption described in Sect. 3.3. Hence, in some cases, one can observe a certain extension of DLw also for grade values higher than that computed for the walls. For instance, in the San Felice’s case (X direction) one can see that a certain damage extension appears in DL equal to 3, 4, and 5, although the maximum extension of DLw was in DL = 2.

DLW represents a synthetic parameter to express the spread damage occurred to the building. Hence, one can observe that a certain extension of DLp also occurred for grade values higher than that computed for the walls. For instance, in the San Felice’s case (X direction) one can see that a certain damage extension of the piers appears in DL = 3 although the maximum extension of DLw was in DL = 2.
Fig. 12 Observed damage to the front and back views for the buildings located in San Felice (a), Pizzoli (b), Visso (c) and Caldarola (d)
Table 7 lists all the additional information about the DL occurred on piers and spandrels as well as the $\alpha_{s,i}$ values that are not obtainable from the histograms of Fig. 14.

Table 8 summarizes the extension (expressed in percentage) of the damage observed for the walls as well as piers and spandrels of the four case studies, highlighting that
the Pizzoli’s building is characterized by the lowest DL, whereas the Visso’s building by the highest DL. In the table, the rate of DL1 is reported in brackets since its attribution is subjected to uncertainty and not always, from photos, it was simple distinguish between DL0 and DL1.

As already described in Sect. 3.3, with the aim of also considering the occurrence of possible soft-story mechanisms, it is necessary to investigate separately for each L-th level of the building the DL occurred for piers, spandrels and, therefore, walls. For the sake of simplicity, the same weights (\(W_{W,i}\)) were assumed for the walls of the different floors and equal to the shear-force equilibrated at the bottom of the panel at the ground floor although a small redistribution of them can be obtained along the height of the building, especially in case of irregular opening. Figure 15 illustrates both the damage extension and cumulative damage by comparing the results between the individual floors and the “global” building. For the latter, the damage is intended smeared on all the levels.
Table 7 Definition of the DL occurred for piers and spandrels (DL_{p,i} and DL_{s,i}) and the coefficient \( \alpha_{s,i} \) of the \( i \)-th wall

| Case study | ID | Wall | 1     | 2     | 3     | 4     | 5     | 6     | 7     | 8     | 9     | 10    | 11    | 12    | 13    | 14    | 15    | 16    | 17    | 18    | 19    | 20    |
|------------|----|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| San Felice |    | DL_{p,i} | 1.6   | 1.2   | 0.3   | 0.9   | 0.7   | 0.3   | 1.1   | 0.9   | 0.3   | 0.3   | –     | –     | –     | –     | –     | –     | –     | –     | –     | –     | –     |
|            |    | DL_{s,i}  | 2.1   | 1.2   | 0.4   | 1.4   | 0.9   | 2.0   | 2.0   | 2.0   | 0.0   | 0.0   | –     | –     | –     | 2.0   | 2.0   | 0.0   | –     | –     | –     | –     | –     |
|            |    | \( \alpha \) | 0.2   | 0.1   | 0.1   | 0.3   | 0.3   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | –     | –     | –     | 0.0   | 0.0   | 0.0   | –     | –     | –     | –     | –     |
| Pizzoli    |    | DL_{p,i}  | 1.7   | 1.0   | 0.5   | 0.5   | 0.5   | 1.7   | 1.1   | 0.2   | 0.7   | 1.8   | 1.8   | 0.0   | 1.0   | 1.9   | –     | –     | –     | –     | –     | –     | –     |
|            |    | DL_{s,i}  | 0.6   | 1.0   | 0.0   | 0.2   | 0.1   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | –     | –     | –     | –     | –     | –     | –     |
|            |    | \( \alpha \) | 0.1   | 0.2   | 0.1   | 0.1   | 0.2   | 0.1   | 0.2   | 0.1   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | –     | –     | –     | –     | –     | –     | –     |
| Visso      |    | DL_{p,i}  | 3.9   | 1.6   | 2.8   | 1.8   | 3.3   | 0.9   | 2.4   | 3.8   | 2.2   | 4.0   | 4.0   | 3.7   | 4.3   | 3.9   | 2.6   | 2.5   | 0.5   | –     | 4.4   | 3.0   | –     |
|            |    | DL_{s,i}  | 2.0   | 0.4   | 1.3   | 0.2   | 0.0   | 2.0   | 1.3   | 0.0   | 0.7   | 0.0   | 0.0   | 0.0   | 3.4   | 5.0   | 1.3   | 1.0   | 3.0   | 2.0   | –     | 0.0   | 0.0   |
|            |    | \( \alpha \) | 0.1   | 0.1   | 0.1   | 0.4   | 0.0   | 0.0   | 0.4   | 0.0   | 0.4   | 0.0   | 0.0   | 0.0   | 0.3   | 0.0   | 0.3   | 0.2   | 0.2   | 0.0   | –     | 0.0   | 0.0   |
| Caldarola  |    | DL_{p,i}  | 2.0   | 2.1   | 2.8   | 1.5   | 1.9   | 1.0   | 1.6   | 1.3   | 2.7   | 1.8   | 2.0   | 2.0   | 1.5   | 3.2   | 1.0   | 1.5   | 3.0   | 2.0   | 3.0   | –     | –     |
|            |    | DL_{s,i}  | 2.0   | 0.3   | 1.2   | 1.0   | 0.5   | 1.3   | 1.2   | 0.3   | 0.2   | 0.0   | 0.1   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | –     |
|            |    | \( \alpha \) | 0.1   | 0.1   | 0.2   | 0.2   | 0.2   | 0.2   | 0.2   | 0.2   | 0.2   | 0.0   | 0.1   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | 0.0   | –     |
Table 8  Percentage of the observed damage at the wall-, piers- and spandrel- scale (DL1 values indicated within the commas)

| DL | S. Felice | Pizzoli | Visso | Caldarola |
|----|-----------|---------|-------|-----------|
|    | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] |
|    | dir.X     | dir.Y   |       | dir.X     | dir.Y   |       | dir.X     | dir.Y   |       | dir.X     | dir.Y   |       |
| 0–1| 67 (54)   | 48 (18) | 85 (38)| 100 (46)  | 73 (41) | 95 (8) | 32 (32)   | 27 (0)  | 68 (20) | 1 (1)    | 35 (35) | 41 (41) | 84 (16) |
| 2  | 33        | 0       | 9     | 38       | 0       | 27     | 5        | 42       | 9       | 13       | 64       | 49       | 28       | 13       |
| 3  | 0         | 0       | 0     | 5        | 0       | 0      | 0        | 0        | 16      | 29       | 23       | 10       | 35       | 16       | 21       | 2       |
| 4  | 0         | 0       | 0     | 5        | 0       | 0      | 0        | 0        | 36      | 38       | 8        | 0        | 0        | 5        | 0        |
| 5  | 0         | 0       | 0     | 3        | 0       | 0      | 0        | 0        | 8       | 3        | 2        | 0        | 0        | 0        | 6        | 0       |
For the San Felice’s case (Fig. 15a), one can see that there was a damage concentration at the ground floor for both the directions (especially in X-direction). In direction X, one can see that the extension of the maximum DLw equal to 3 is almost the same for both the ground and first floors (about 40%). Instead, one can see that all the walls at the second floor (i.e. 100% of extension) are characterized by DLW = 0–1. This result depends on the fact that the damage that occurred at the second floor interested only the spandrels (lintel collapse), which give a smaller contribution in the definition of the DLw (see Sect. 3.3). Furthermore, in direction X, the cumulative damage of the walls at the ground floor resulted higher than that of the first floor, as well as that of the second floor, as illustrated in the 2nd column of Fig. 15a. In direction Y, the damage that occurred at the ground floor was higher (DLw = 3 with extension of less than 30%) than that which occurred for both first and second floors, as illustrated in 3rd and 4th columns of Fig. 15a. Definitely, by neglecting the upper floor, a higher global damage could be obtained for the San Felice’s building (DLG equal to 3 instead of 2 in X direction). However, as specified in Sect. 3, the procedure of investigating possible damage localization can be consistent only when

![Fig. 15 a–d Extension damage occurred for the four analysed buildings at the different levels in X and Y directions (1st and 3rd columns). Cumulative damage at the different levels in X and Y directions (2nd and 4th columns)](image_url)
$DL_G > 2$. Hence, for the San Felice case, $DL_G$ should be assumed equal to 2 (neglecting the effect of the damage concentration to the ground floor). For the Pizzoli’s case (Fig. 15b), one can observe a slightly higher damage concentration at the ground floor of the building (especially in the Y direction) reaching $DL_w = 3$ (with an extension less than 20% of the wall) instead of $DL_w = 2$ of the global damage, which is intended as the damage smeared on all the levels.

For the Visso’s case (Fig. 15c) one can see a higher damage concentration at the first floor of the building in both the directions. However, that concentration does not change the maximum DL occurred for the building but only the cumulative damage.

Finally, for the Caldarola’s case (Fig. 15d) one can observe the highest damage concentration, as compared to the other buildings. For both the building directions, a small percentage of the walls of the first floor attained $DL = 5$ (7% and 16% of extension in X and Y, respectively), whereas the maximum global damage is equal to 3. However, the building direction that suffered the highest damage concentration was the X, as in that case also $DL = 4$ occurred for about 25% of the walls. By applying the criteria of the proposed procedure (Table 2), one can obtain $DL_G$ equal to 4 for the X direction (two grades more with respect to the result obtained without focusing on the damage concentration), and still equal to 2 for the Y direction (the damage concentration was not so consistent). However, when the investigation of the damage concentration highlights possible soft-story mechanisms, the authors suggest increasing $DL_G$ by a maximum of one grade. Hence, for the Caldarola’s case, $DL_G$ should be assumed equal to 3.

### 5.4 Definition of the global damage level and comparison with other procedures available in literature

In Table 9 is reported the global damage level $DL_G$ computed for the analyzed buildings, following the criteria defined in Table 2. Particularly, the effective $DL_G$ between the two directions is shown in bold (i.e. the maximum value between direction X and Y). A comparison with the results obtained through both the methods proposed in EMS-98, the Da.Do platform Dolce et al. (2019), Lagomarsino et al. (2021) and Rota et al. (2008) is also reported. In particular, for the San Felice’s, Pizzoli’s and Visso’s cases, one can see that the $DL_G$ values obtained by the procedure proposed in this study are coincident with those obtained by EMS-98, Da.Do. and Rota (when the highest damage among the two directions is considered). On the contrary, for the Caldarola’s case, despite the proposed procedure led to an increase of the $DL_G$ because of the soft story mechanisms at the first floor, one can obtain a lower $DL_G$ (equal to 3), as compared to the other above-mentioned methods (for which $DL_G = 4$).

Furthermore, one can see that the Lagomarsino procedure (under both the assumptions of the survey accuracy: full and partial) led to the lower $DL_G$ for all the cases. Indeed, that method underestimates the damage of the buildings, as it assigns reductive weight (0.6 or 0.8 for full survey -FS- and partial survey -PS-, respectively) to the wall damage aiming to consider the damage contribution of the other structural elements, such as stairs and roof, for which weights are assumed equal to 0.2/0 (i.e. FS/PS) and 0.2/0.2, respectively. Hence, when the damage is merely concentrated in the walls with negligible damage occurred for the other two structural elements, the adoption of the Lagomarsino method may lead to an underestimation of the $DL$, as compared to the other methods. Furthermore, in that method, the coefficients assumed for the damage extension (i.e.1/6 for an extension < 1/3;
| Method                  | dir.X | dir.Y | dir.X | dir.Y | dir.X | dir.Y | dir.X | dir.Y |
|-------------------------|-------|-------|-------|-------|-------|-------|-------|-------|
| Catassi and Angiolilli (2022) | 2     | 1     | 1     | 2     | 2     | 2     | 2     | 2     |
| EMS-98                  | 2     | 2     | 2     | 4     | 4     | 3*    | 3*    |       |
| DaDo                    | 2 (pier; floor: 2) |       | 2 (pier; floor: 2) |       | 4 (floor: 4) |       | 4 (floor: 4) |       |
| Rota et al. (2008)      | 2     | 2     | 2     | 4     | 4     | 4     | 4     | 4     |
| Lagomarsino et al. (2021)—FS | 0.8 (floor: 2)** | 0.8 (floor: 2)** | 2.3 (floor: 2)** | 2.3 (floor: 2)** | 1.8 (floor: 2)** | 1.8 (floor: 2)** | 2.0 (floor: 2)** | 2.0 (floor: 2)** |
| Lagomarsino et al. (2021)—PS | 0.8 (floor: 2)** | 0.8 (floor: 2)** | 2.6 (floor: 3)** | 2.6 (floor: 3)** | 2.0 (floor: 3)** | 2.0 (floor: 3)** | 2.0 (floor: 3)** | 2.0 (floor: 3)** |

*One grade more due to the damage floor localization; **by adopting the conversion criteria of Table 2.
1/2 for an extension ranging from 1/3 to 2/3; 5/6 for an extension > 2/3) also contribute to underestimate the DLG.

The difference among the four methods is evident especially when a small concentration of very severe damage occurs to the building. For example, in the case of vertical structures characterized by DL = 5 with an extension lower than 30% of the walls (i.e. < 1/3), the DLG would be: equal to 3 through both the Cattari and Angiolilli procedure and the Da.Do procedure; equal to 4 through the EMS-98 and Rota procedures; less than 1 through the Lagomarsino procedure (also if the full survey accuracy is considered).

The results show a non-negligible variability in the results as a function of the different conversion metrics adopted. In general, approaches referring to the peak damage level may tend to slightly overestimate DLG while simplified integral ones may tend to slightly underestimate it. A judgment on the “optimal” metric is out of the scope of this paper and also difficult, also because in a wider perspective it cannot be excluded that different metrics may produce a better or worse correlation with different types of losses (e.g. usability or repair costs).

6 Potential of the proposed procedure for validating numerical simulations

This paragraph is intended to exemplify how the above-described procedure can be applied univocally also to interpret the simulated damage level of URM building obtained by performing NDAs on the numerical model representing as close as possible the real building in terms of both geometric and mechanical features. This can be useful not only to validate numerical models but also to assign a synthetic DL value to the huge amount of data from a time-history with the aim of deriving fragility curves or tools useful to support the decision-making process in the post-earthquake scenario. While in this paper only the first goal is exemplified, examples of the second ones are provided in Brunelli et al. (2022) and (Sivori et al. 2022), respectively.

The numerical models of the case studies presented in Sect. 5 and herein used were developed in previous research through the Tremuri software (Lagomarsino et al. 2013). The latter works according to the Equivalent Frame (EF) approach and is particularly suitable in the case of URM buildings dominated by the box-like behaviour. The Tremuri software was largely validated in previous studies (e.g. Brunelli et al. 2021; Degli Abbati et al. 2022; Penna et al. 2016) and is particularly effective in performing NDA (Cattari et al. 2018). According to the piecewise-linear constitutive law adopted for panels and implemented in this software (Lagomarsino and Cattari 2013), the shear-drift relationship governing the stiffness and strength deterioration of masonry panels to cyclic loading and damage is based on a phenomenological approach and allow to easily keep track of the DL reached in each deformable structural element (piers and spandrels) during the seismic input (see Fig. 16a–c). Moreover, a hysteretic response is well reproduced through a phenomenological approach that can capture the differences among the various possible failure modes (mainly flexural type, shear type or even mixed) and the different responses of piers and spandrels. More precisely, the achievement of a DL in the structural element (i.e. DL_E) is monitored by the exceeding of drift thresholds defined by the analyst and differentiated for spandrels and piers and also varying the failure mode (if flexural, shear or hybrid); in the case of DL_E equal to 3, 4, and 5 a strength reduction is also assigned. Thus, these drift thresholds represent in the numerical model the practical way to establish the
relationship with the actual physical evolution of damage, i.e. to the widening and propagation of cracks as discussed in Sect. 3.1.

Nonetheless, the proposed procedure can be equally adopted in other equivalent-frame (i), finite element (ii) or discrete element (iii) implementations by properly analyzing the ex-post processing of the data (e.g. Castellazzi et al. 2021). In particular, it may be made: in the (i) and (ii) cases, by monitoring the drift occurred on each structural element according to the same DL-threshold values; as well as in the (iii) case, by the possibility to evaluate also the effective crack opening. The choice of the most effective modelling strategy must be addressed by the specific features of the building under examination together with the examination of the damage mechanisms that may be activated by assessing the capability of the model in reproducing them (D’Altri et al. 2020; Cattari et al. 2022b).

6.1 Overview of the numerical models

Figure 16a–g illustrates the EF models of the four analyzed buildings, where the piers and spandrels are represented in orange and green, respectively. As already mentioned, these models were already developed in previous research and in particular in (Cattari and Lagomarsino 2013; Degli Abbatì et al. 2022; Brunelli et al. 2021; Cattari et al. 2022a) for the San Felice’s-, Pizzoli’s-, Visso’s and Caldarola’s buildings, respectively.

Of course, when dealing with the numerical simulation of the actual response, the impact of modelling uncertainties on the analytical damage prognosis should be accurately evaluated (e.g. Bracchi et al. 2015). All the models herein adopted were subjected to an in-depth calibration phase (e.g. based on data from ambient vibration tests, if available, as for the Visso’s and Pizzoli’s buildings) and also to extensive sensitivity analyses to investigate the effect of both aleatory and epistemic uncertainties on the structural response.
A discussion on the sensitivity of the results regarding some of epistemic uncertainties that characterize the modelling process (e.g. the effective height of piers or the effectiveness of the wall-to-wall connection) is described in Ottonelli et al. (2020), considering as benchmark case study just the Visso’s school; moreover, in Cattari et al. (2022a) further exemplifications of parametric analyses addressed to explore the effects of uncertainties on mechanical parameters and softening phase of constitutive laws of piers are illustrated for the Caldarola’s school. Indeed, the selected case studies constitute very emblematic examples for investigating also other relevant and tricky issues that may affect the seismic response of existing buildings, such as the soil-structure interaction and cumulative damage due to repeated shocks (for the Visso’s school) or the site effect (for Caldarola’s school). A detailed assessment of all these aspects is out of the scope of this paper and interested readers are invited referring to the aforementioned papers. In the following, the main assumptions on such numerical models are briefly recalled.

Firstly, a good coupling between orthogonal walls was assumed. This was consistent with the geometry and constructive details as well as the post-earthquake damage observation that suggested a box-type behavior for all the buildings. Floor diaphragms were modelled as 3- or 4-node finite elements with linear orthotropic membrane formulation in plane stress conditions.

The model requires the definition of a set of parameters for the description of the initial stiffness of the panel and its progressive degradation, the maximum strength of the panel \( V_y \), and the progress of nonlinear response for increasing damage levels \( \theta_{E,i} \)—drift value at the attainment of the \( i \)th DL, \( \beta_{E,i} \)—corresponding fraction of the residual shear strength, with respect to \( V_y \). The different values of \( \theta_{E,i} \) and \( \beta_{E,i} \) are assigned for describing a prevailing flexural or shear response of the panel and can be differentiated in the case of spandrel and pier elements. The values assumed for the drifts and the corresponding strength decay are consistent with experimental data in the literature (Beyer and Dazio, 2012; Vanin et al. 2017; Graziotti et al. 2018; Rezaie et al. 2020). Table 10 lists the main mechanical properties and the nonlinear parameters adopted in the numerical simulations, that have been derived from the aforementioned calibration process and sensitivity analyses carried out by Cattari and Lagomarsino (2013), Degli Abbati et al. (2022), Brunelli et al. (2021) and Cattari et al. (2022a).

### 6.2 Comparison between the simulated- and observed- responses

This subsection analyzes the difference between the simulated and observed damage level. In particular, the first column of Fig. 17 illustrates the comparison in terms of DLW. In those histograms are also reported the horizontal dotted lines indicating the sum of the DLW of along the two directions (X and Y). The second column of Fig. 17 illustrates the comparison in terms of the DLW multiplied by the respective weights Ww. These histograms provide more accurate information about the difference between the observed and numerical global damage. For example, for the San Felice’s case, the wall n.6 is characterized by a DLW value higher than that of wall n.7 (first column of Fig. 17a) though the latter more scatters the difference in the observed-simulated global damage of the building because its importance (weight Ww) is higher than that of wall n.6. Finally, the third column of Fig. 17 depicts the extension of a certain DL occurred for the walls along the two main directions of the buildings.

The results of Fig. 17 are summarized in Table 11, in which is also represented the damage extension discretized for piers and spandrels. In general, one can observe that
### Table 10  Mechanical parameters adopted in Tremuri for the masonry panels

| Case study | $f_m$ [MPa] | $\tau_0$ [MPa] | E [MPa] | G [MPa] | $\theta_{\varepsilon 3}$ [%] | $\theta_{\varepsilon 4}$ [%] | $\theta_{\varepsilon 5}$ [%] | $\beta_{\varepsilon 3}$ [-] | $\beta_{\varepsilon 4}$ [-] |
|------------|-------------|----------------|---------|---------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|
| S. Felice  | 2.8         | 0.1            | 675     | 225     | 0.3/0.2 (SH)             | 0.5/0.6 (SH)             | 0.7/2 (SH)               | 0.3/0.5 (SH)             | 0.6/0.5 (SH)             |
|            |             |                |         |         | 0.6/0.2 (FL)             | 1.0/0.6 (FL)             | 1.5/2.0 (FL)             | 1/0.5 (FL)               | 0.15/0.5 (FL)            |
| Pizzoli    | 5.95        | 0.12           | 2262    | 754     | 0.45/*** (SH)            | 0.7/1 (SH)               | 0.9/1.5 (SH)             | 0.6/0.7 (SH)             | 0.1/0.7 (SH)             |
|            |             |                |         |         | 0.5/0.6 (FL)             | 0.8/1 (FL)               | 1.1/1.5 (FL)             | 1 (FL)                   | 0.8/0.7 (FL)             |
| Visso*     | 4.94        | 0.096          | 2574    | 858     | 0.45/*** (SH)            | 0.7/1.5 (SH)             | 1.48/2.0 (SH)            | 0.4/0.3 (SH)             | 0.8/0.3 (SH)             |
| Visso**    | 5.7         | 0.111          | 2701    | 991     | 0.60/*** (FL)            | 0.8/1.5 (FL)             | 1.81/2.0 (FL)            | 0/0.3 (FL)               | 0.15/0.3 (FL)            |
| Caldarola  | 1.4/2.0     | 0.0245/0.0430  | 1230/1230 | 410/410 | 0.25/1 (SH)             | 0.42/2 (SH)             | 0.59/2.5 (SH)            | 0.7/0.1 (SH)             | 0.9/0.1 (SH)             |
|            |             |                |         |         | 0.25/1 (FL)             | 0.42/2 (FL)             | 0.59/2.5 (FL)            | 1/0.3 (FL)               | 1/0.1 (FL)               |

$f_m$: compressive strength of masonry; $\tau_0$: shear strength of masonry to be used in criteria analogous to that proposed by Turnšek and Čačovič (1970) and Turnšek and Sheppard (1980); $E$: Young modulus of masonry; $G$: shear modulus of masonry

*Original masonry; **strengthened masonry; ***defined by assigning a prefixed value of ductility equal to 2
the simulated damage is similar to the observed damage and that the simulated damage of piers is more scattered than that of spandrels. Moreover, for all the cases, the simulated damage of spandrels slightly overestimates the real damage. It is worth noting that, in some cases, the large difference can be misleading, as for example in the piers of the wall n. 6 of the Caldarola’s building. Indeed, the weight $W_W$ of that wall is very small (see Fig. 14d) and, therefore, its contribution in the definition of the $DL_G$ is almost negligible.

Finally, Table 12 summarizes the damage level at the global scale calculated by both the observed-based and numerical procedures. It is worth noting that for the computation of the simulated damage it was not taken into account the possible soft story mechanisms that occurred for the buildings (i.e. for the Caldarola’s building). In particular, except for the Caldarola’s case, the same observed- and simulated- $DL_G$ can be noticed by comparing the maximum damage computed among the two main directions of the buildings. For the Caldarola’s building one can observe a higher simulated $DL_G$ (equal to 3) instead of the real one (equal to 2 without considering the soft-story mechanisms). Nevertheless, one can
Table 11  Extension of the simulated damage at the wall-, piers- and spandrel-scale

| DL | S. Felice | Pizzoli | Visso | Caldarola |
|----|-----------|---------|-------|-----------|
|    | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] | W [%]     | P [%]   | S [%] |
|    | X | Y | X | Y | X | Y | X | Y | X | Y | X | Y | X | Y |
| 0–1| 0 (0) | 19 (19) | 11 (11) | 0 (0) | 100 (46) | 10 (0) | 73 (41) | 95 (8) | 0 (0) | 0 (0) | 6 (0) | 1 (1) | 0 (0) | 0 (0) | 3 (3) | 22 (3) |
| 2  | 92 | 81 | 79 | 0 | 0 | 90 | 27 | 5 | 30 | 0 | 51 | 55 | 39 | 71 | 73 | 78 |
| 3  | 8 | 0 | 66 | 0 | 0 | 0 | 0 | 0 | 70 | 74 | 30 | 42 | 61 | 29 | 17 | 0 |
| 4  | 0 | 0 | 0 | 4 | 0 | 0 | 0 | 0 | 0 | 18 | 9 | 2 | 0 | 0 | 6 | 0 |
| 5  | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 8 | 4 | 0 | 0 | 0 | 1 | 0 |

DL1 values indicated within the commas
see that the building direction in which the greatest damage occurred is well captured by simulations.

The data acquired at various scales according to the multi-scale procedure proposed in this paper may also be efficiently adopted to compute other synthetic indexes useful for validation aims and investigate the effectiveness of the modelling strategies. For example, in Cattari et al. (2022a) the Matching Index (MI) concept (a synthetic number varying from 0 to 1, see Angiolilli et al. 2022) has been adopted for the Caldarola’s school to define the most reliable numerical model reproducing the real damage by varying the mechanical proprieties of masonry and drift thresholds adopted for piers.

### 7 Conclusions

The correct interpretation of the damage level of existing URM buildings is a crucial aspect for several goals in the earthquake engineering and, moreover, it may constitute a useful support for the validation of numerical simulations. First, the paper defines a new methodology for an analytical definition of the damage level of URM buildings mainly characterized by a global behavior, taking into account for the failure type that occurred in individual structural elements (piers, spandrels and diaphragm) whose information is then combined to define the damage level at the global scale. The latter is conceived to consider the qualitative criteria defined in the European Macroseismic Scale (EMS98) only regarding classification and extension of the damage levels.

The statistical analysis of the database available on the DaDo platform from which only residential masonry buildings struck by the 2009 earthquake were taken into account, highlighted important correlations between floor and wall damage. In particular, the masonry typology appears to be predominant in affecting the DL distribution among vertical and horizontal elements, as compared to the floor typology. Moreover, from that analysis, it was justified the exclusion (at least as first approximation) of the floor damage in the assignment procedure of the global DL, as the diaphragm DL is statistically lower or at most equal to the wall DL.

Hence, the described procedure can be applied to interpret the real damage that occurred on existing buildings when accurate damage observations from field surveys or historic documentation are available.

A comparison of the damage level interpretation performed by other methods present in the current literature is presented, highlighting the main differences and the limitations of each procedure. In particular, when the weakest direction of the building is considered as representative of the vulnerability of the entire building, the results of the proposed method are in line with those obtained by the Da.Do. and Rota methods (which define the global

### Table 12

Comparison between observed and simulated damage at the global scale through the new procedure

|        | S. Felice | Pizzoli | Visso | Caldarola |
|--------|-----------|---------|-------|-----------|
| dir.X  | dir.Y     | dir.X   | dir.Y | dir.X     | dir.Y |
| DL$_{G,obs}$ | 2       | 1       | 2     | 4         | 3*    |
| DL$_{G,sim}$ | 2       | 2       | 1     | 3         | 3     |

*One grade more due to the damage floor localization
damage level strictly based on the interpretation of the AeDES form and assume as most representative the peak of damage).

Finally, the potential of the new method was also tested to interpret the damage simulated by the numerical model reproducing the selected case-study buildings through nonlinear dynamic analyses. In general, the global damage level of the building is correctly reproduced by numerical models herein considered. Indeed, the simulated damage level of the individual elements tended to be only slightly higher (i.e. less than one grade) than the observed one, especially in the spandrel elements. This highlighted that, when detailed data is available, the proposed procedure can be very useful for the validation of simulated-based damage assessments. Moreover, in future studies, the procedure should be extensively applied to nonlinear dynamic numerical analyses in order to post-process the huge amount of data they provide and define a synthetic parameter (DLG) conceptually consistent with the one attributable from the observed damage. This potential of the proposed procedure appears particularly relevant within the context of seismic risk analyses to reduce the inconsistencies often produced by the use of different approaches in defining DLs from numerical and empirical approaches.

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Availability of data and materials Additional data on the discussed URM buildings may be requested to the Authors of the paper for helping the reproducibility of the proposed case studies.

Code availability Not applicable.

Declarations

Conflict of 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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