A comparative study on a complex URM building. Part II: issues on modelling and seismic analysis through continuum and discrete-macroelement models

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Abstract: The paper presents the comparison of the results obtained on a masonry building by nonlinear static analysis using different software operating in the field of continuum and discrete-macroelement modeling. The structure is inspired by an actual building, the "P. Capuzi" school in Visso (Macerata, Italy), seriously damaged following the seismic events that affected Central Italy from August 2016 to January 2017. The activity described is part of a wider research program carried out by various units involved in the ReLUIS 2017/2018 - Masonry Structures project and having as its object the analysis of benchmark structures for the evaluation of the reliability of software packages. The comparison of analysis was carried out in relation to: global parameters (concerning the dynamic properties, capacity curves and equivalent bilinear curves), synthetic parameters of structural safety (such as, for example, the maximum acceleration compatible with the life safety limit state) and the response in terms of simulated damage. The results allow for some insights on the use of continuum and discrete-macroelement modeling, with respect to the dispersion of the results and on the potential repercussions in the professional field. This response was also analyzed considering different approaches for the application of loads. URM building.

Keywords: Continuum model, discrete-macroelement model, benchmark structures, comparative study, masonry, nonlinear static analysis.

1 Introduction

The paper presents the work carried out by various research units about the analysis of benchmark structures for the evaluation of the reliability of software packages within the ReLUIS 2017/2018 project - Masonry Structures Project [1][2] that aims at seismic analysis of unreinforced masonry buildings.
In particular, in this article, the results of the evaluations carried out through nonlinear static analysis on an actual case study, inspired by a real building the school "P. Capuzi" of Visso (Macerata, Italy), are shown. The case study represents the benchmark n.5 (BS5) analyzed in the research program [2].

This building dates back in the '30s and was strengthened following the Umbria-Marche 1997 earthquake. Then it was severely damaged, mainly due to the in-plane response of the walls, by the seismic events that affected Central Italy in 2016/2017. The structure, now demolished because of the serious damage suffered, was the subject of a permanent seismic monitoring systems by the Seismic Observatory of Structures [3]. The BS5 has been modeled with three software packages: two in the field of continuum modeling and one in the field of discrete-macroelement. In the following, owing to anonymous presentation and discussion of results the software will be named as SW8, SW9, SW10. Results will also refer to outcomes obtained within the companion paper, see [4] about seven software operating in the field of equivalent frame modeling, available at professional level, here named software of Group1 (in the following SWG1). Similarly, SW8, SW9, SW10 are named software of Group 2 (in the following SWG2).

The input data and some modeling choices were shared at an early stage by the researchers involved in order to limit the potential dispersion of the results and make the comparison more robust. It is important to note that the reconstruction of the actual damage is an important feedback to carry out considerations on the reliability of the software used in predicting and capturing the damage mode activated or, more generally, the seismic response.

The synergy between the Part I and Part II of the present work allows for a critical comparison of the solutions that can be adopted in order to achieve a consistent modeling of the same structure using different modeling strategies. The paper is organized as follows: in Section 2 the case study is presented, in Section 3 describes the seismic response of the structure obtained, with the same modeling choices, from the different SWs in terms of masses, dynamic parameters, pushover curve and damage modes for the different piers. Section 4 presents the results obtained for the calculation of bilinear equivalents with respect to the modeling techniques used in the present work. Section 5 describes the repercussions on seismic assessment, implemented by evaluating the ultimate capacity of the structure in terms of peak ground acceleration (PGA), considering the sensitivity to different SWs and modeling contributions.

2 Brief description of the benchmark case study and modelling hypotheses adopted

A brief description of the case study is reported here, but the interested reader is addressed also to the Part I paper [4] for further details.

The BS5 structure was selected as a benchmark since it possesses interesting features: (i) regular distribution of openings; (ii) the availability of a detailed reconstruction of the damage; (iii) the
structure experienced a prevalent global response with damage concentrated mainly in piers and in
the walls.

The severe damage experienced by the structure, also caused by damage accumulation, led to the
activation of a tilting mechanism in a very limited portion of the building. I worth to mention that the
structure was provided by a permanent monitoring system by the Seismic Observatory of Structures
[3] and was the subject of other research as part of ReLUIS projects funded by the Department of
Civil Protection (ReLuis - Task 4.1 Workgroup (2018)). The building is characterized by an irregular
"T" shape and possesses load-bearing unreinforced masonry and rigid floors organized into two levels
plus an attic floor and an underground part.

The walls are made mainly of stone masonry (pietra a spacco) whose properties are presented into
the companion paper Part I [4] and whose collocation is briefly recalled in Figure 1. Within the
Masonry Structures Project [2,5] two hypotheses have been analyzed for the benchmark structure
BS5 in order to investigate the results dispersion provided by the different software due to the
variation of different structural configurations recurring in existing buildings:

- **BS5/A**: absence of tensile strength elements coupled to the spandrels, to simulate a so-called
  weak spandrel behavior.
- **BS5/C**: spandrels coupled to tensile strength elements (reinforced concrete ring beams).

In the present paper, only the BS5/C case will be addressed and discussed. In particular, in-situ
investigations revealed a full thickness reinforced concrete ring beam, so the most plausible modeling
hypothesis is that the spandrels interrupt at the level of the ring beam forming under- and over-
window elements.

To push the results comparison collected in both the companions papers Part I [4] and Part II, and in
order to reduce the results dispersion, the following common assumptions holds for both the studies:

- the geometrical data (such as the wall thicknesses, as synthetically reported in Figure 1 for the
ground floor);
- the distribution and values of the floor loads, including that of the roof, considered only as an
  equivalent additional load and not through the explicit modelling of each single structural
  element;
- the mechanical properties of all materials.

For the sake of paper length, the material properties are summarized into the companion paper only
(see Table 1) along with the comprehensive description of the cracking pattern and pictures of the
building collected after each ones of the multiple seismic events that allows for the precise inspection
of cracking pattern evolution due by the accumulation of subsequent damage.

This aspect is of fundamental importance in the following when we aim at being comparing this
cracking pattern with the outcome of the present modeling: we will make use of the conventional
push-over analysis to study the structural behavior and not of the nonlinear dynamic analysis could be more appropriate for the comparison of modeled and real cracking patterns. Moreover, the application of push-over analysis, due to its intrinsic conventionality, is not straightforward when dealing with continuum modeling and requires assumptions that can have a decisive impact on the results, such as the selection of the element type, the choice of convergence criteria, the definition of the load history and the method of load application.

The following software have been used in this study:

- SIMULIA ABAQUS 6.14 distributed by Dassault Systemes, where masonry nonlinear behavior is modeled using an homogeneous isotropic plastic-damaging continuum. This model, hypothesizes independent tensile and compressive behaviors ruled by tensile damage ($0 \leq d_t < 1$) and compressive damage ($0 \leq d_c < 1$) variables. The masonry wall panels are modeled with three-dimensional four-nodes tetrahedral finite elements, see Figures 2(a)(b);

- MIDAS FEA 2016 Ver. 1.1, distributed by MIDAS Information Technology Co. where masonry nonlinear behavior is modeled using the Total Strain Crack Model, assuming a fixed reference system for the cracked element called fixed crack model. The masonry wall panels are modeled with two-dimensional plate finite elements, see Figures 2(c)(d).

- 3DMacro Ver. 4.7, distributed by Gruppo Sismica, that works for plane discrete elements and adopts a macro-element formulation in which the different failure modes are managed by different types of nonlinear springs. The panels are described by discrete two-dimensional elements, see Figures 2(e)(f). The coupling of incident walls is managed through appropriate "corner elements", characterized by rigid prismatic elements connected via interfaces (which can be calibrated by the user) to other connected elements. The floors can be modeled as orthotropic slabs or, alternatively, adopting the ideal solution of infinite stiffness. The program also allows for the out-of-plane contribution of stiffness and panel strength. The convergence algorithm used in nonlinear analyzes is an event-to-event strategy.
The calibration of the mechanical parameters of the material is the preliminary step necessary to establish a correspondence between the parameters of the material and the behavior of the element (the scale at which the checks are usually carried out).

The parameters that usually must be calibrated are the elastic modulus of the material, the compressive strength, the tensile strength, and the fracture energy of the material. These values affect the stiffness, strength, and ultimate deformation of the material. It worth to remember here that all the software have been prior tuned and calibrated to reproduce simple conventional benchmark results, see [6]. A discussion of these aspects is addressed in the paper [6] where a simple and practitioners-friendly calibration strategy to consistently link target panel-scale mechanical properties (that can be found in national standards) to model material-scale mechanical properties is presented.
Then, in the following, the assumption made for the modeling of the structures follow these preliminary setting. The interested reader could get some further complementary information in the following references [7-17].

The structural models of the BS5 are illustrated in Figure 2 for the above-mentioned software along with the association of the specific software package, but no further specification will be addressed in the following sections to link the computed results to their software and comments and discussion will try to preserve the anonymous illustration.

The geometrical representation of the vertical structure is here driven by the sole geometry of the building without introducing specific simplification of the structure (i.e., dimension of the opening), whether the effective length of the walls is accounted for models that employs conventional representation of the floor thickness.

Some general simplifications are also considered for all the software and reflect the assumptions made for the research activity of the "URM nonlinear modeling – Benchmark project", including that of neglecting the explicit modeling of the roof and of the basement floor which also involved only a minor part of the floor plan.

3 Sensitivity of results to different software packages with standardized modeling assumptions

In this section the sensitivity of the numerical modeling is studied with respect to some specific modeling assumption that could affect the characterization of the structural dynamic behavior of the BS5 benchmark. To extend the comparison to results provided in the companion paper [4], the same parameters are used here:

- the total mass of the building;
- the periods, participant masses and modal shapes obtained from the modal analysis;
- the global pushover curves and the synthetic parameters that unequivocally define the equivalent bilinear of those curve (i.e. the stiffness $K_s$, the base shear $V_y$ and the ultimate displacement $d_u$);
- the cracking pattern and the damage corresponding to the ultimate displacement capacity ($d_u$) evaluated on the pushover curves has been also compared with the actual one occurred in the “P. Capuzi” school.

For a comprehensive description of the input parameters please refer to the Annex- BS Input Data reported in [2].

3.1 Masses and dynamic properties comparison

A comparison of the structural dynamic behavior provided by the software programs is discussed here by means of modal analysis. For this purpose, the (i) effective masses, (ii) periods, and (iii) shape
modes of the first three modes are compared and discussed. The discussion is also including the results reported in Part I by means, where applicable, of averaged result. As anticipated, structural model steams from the geometry definitions, then, if modeling simplification are introduced, the computed volume of the structure will lead to possible differences in the mass definition, as reported in Table 1. The mass variation computed using the reference value, computed manual, produces very small errors confined below 1.5%, (for equivalent frame softwares the error is confined in general below 5%, see [4]).

| Software | Mass [Kg] | err. % |
|----------|-----------|--------|
| SW8      | 3333550   | 0%     |
| SW9      | 3295517   | -1%    |
| SW10     | 3295064   | -1%    |
| Reference| 3336031   |        |

Despite this little variation, it worth to note that, the calculation of the dynamic properties will depend on the effective mass distribution consequent of the discretization procedure. Then, considering the mesh reported in Figure 2, it is possible to note that, owing to different modelling approaches, the software will produce then different mass distribution. With this in mind, in order to compare the dynamic properties computed by the different software ten modes are extracted and analyzed, and then a comparison is carried out on similar modes with similar participating masses. The overall inspection of mode shape (equivalent frame, continuum, and macro-element models) reveals three significant common mode shapes: X, Y and X-Y. Among the ten computed modes, comparison is performed using the three modes that are found similar in terms of mode-shape and highest participation masses. This comparison, carried out by means of global modes, is justified since the structure examined is characterized by very rigid slabs, which are, therefore, able to effectively couple the walls. The modal analyses eigenvectors show a flexural-torsional vibration mode (identified by the "X-Y" subscripts, characterized by significant percentages of activated participation mass along both X and Y) and two vibration modes of a predominantly translational type (identified by the "X" and "Y" subscripts, respectively).

Table 2 summarize the participation mass values associated with the three modes estimated by each software.

It worth to mention that modal analyses on equivalent frame models are generally carried out by adopting cracked stiffness values, (applying a reduction coefficient of 0.5). For this reason, in this paper, the analyses were run considering both cracked-stiffness and uncracked-stiffness for macro-element and continuum modeling, respectively.
Then Table 3 also reports the reference value obtained for each of the three modes considered, assessed as the average of the software estimates for each of the two groups (SW\(_G_1\) and SW\(_G_2\)), as they operate under the same modeling approach.

| First Mode | Second Mode | Third Mode |
|------------|-------------|------------|
| \(M_x\) [%] | \(M_y\) [%] | \(M_x\) [%] | \(M_y\) [%] | \(M_x\) [%] | \(M_y\) [%] |
| SW8        | 2           | 73         | 17         | 11         | 68         | 0          |
| SW9        | 29          | 20         | 9          | 56         | 41         | 0          |
| SW10       | 52          | 30         | 29         | 54         | 1          | 0          |

It is observed that the periods of "Y" and "X-Y" modes, in the uncracked case, are rather close together, while the periods of the two modes that can be classified as translational in "X" and "Y" are more dissimilar.

| SW     | Mode X-Y | Mode Y | Mode X |
|--------|----------|--------|--------|
| SW8    | Mode 2   | Mode 1 | Mode 3 |
| SW9    | Mode 1   | Mode 2 | Mode 3 |
| SW10   | -        | Mode 2 | Mode 1 |

| modes   | \(T_{X-Y}\) | \(T_Y\) | \(T_X\) |
|---------|-------------|--------|--------|
| SW\(_G_1\) cracked | 0.251   | 0.253  | 0.229  |
| SW\(_G_2\) cracked | 0.241   | 0.235  | 0.229  |
| SW\(_G_2\) uncracked | 0.172   | 0.173  | 0.156  |

- Mode not identified

Figure 3(a) illustrates the percentage changes relative to the reference averaged value of the modes in X, Y and X-Y.

The comparison is excellent between the estimates offered by the equivalent frame models, continuum and macro-element models since the percentage error is confined below 5%.

The extraction of some eigenvectors components, available for all the modes, allows the conventional reconstruction of mode shape that are sketched in Figure 3(b), (c) and (d) for X-Y, Y and X mode shapes, respectively.

A general matching of the modal shapes can be tracked: two configurations obtained from SW8 and SW9 about Mode 3 almost overlaps, see Figure 3(d).
Figure 3 - Modal shapes and undeformed layout for the first floor - Percentage changes relative to the reference value of the modes in X, Y and X-Y (a) Mode XY (b), Mode Y (c), Mode X (d)

3.2 Global Pushover Curve Comparison

Nonlinear static analyses were conducted in the main X and Y directions, considering both positive and negative directions of seismic action, without considering the effect of additional accidental eccentricity. Two different distributions of lateral forces are considered: proportional to the mass distribution (uniform) and to the distribution of the product of the masses for the relative elevations (inverse triangle). Therefore, eight global pushover analysis are performed whose curves are illustrated in Figure 4 and 5 for uniform and triangular load distribution, respectively. The curve comparison is carried out including the equivalent frame model results (SWG1) summarized by envelope results of the pushover curves for each load distribution. Some details in terms of panel stiffness and strength definition are discussed in the following to push the comparison.

3.2.1 Panels stiffness definition

The initial stiffness is set according to the two usual strategies:

- Conventional degradation: assuming a restrictive elastic stiffness coefficient equal to 0.5 for the fully reactant sections (usually for frame models).
- Progressive degradation: due to the evolution of the spread of damage and to different degrees depending on the damage mode activated in the panels (compression-bending or shear).

Continuum modeling of masonry supposes a homogeneous isotropic material idealization, whether macro-element modeling is based on the separation of the possible failure mechanisms of masonry
Dealing with an isotropic continuum, the well-known relationship $G = \frac{E}{2(1 + v)}$ links the 3 elastic constants, i.e., Young’s modulus ($E$), Poisson’s coefficient ($v$), and shear modulus ($G$). However, masonry shows an anisotropic response also in terms of stiffness. Accordingly, values of $E$ and $G$ experimentally measured or suggested in standards for masonry panels would often lead, in an isotropic model, to unrealistic values of $v$ (which is typically included within the range 0.15-0.25 for masonry). Here, following the stiffness calibration strategy adopted for isotropic models in [6], for all the models we consider a realistic value of $v$, typically 0.2. Accordingly, the value of $G$ is herein assumed equal to the target shear modulus (e.g., $G = 580$ MPa as from Table 1 in [4]) and the value of $E$ is computed for the isotropic model. Furthermore, for the macro-element model, the suggested value of the Young’s modulus is not modified to account for the cracked conditions, as often done in the case of equivalent frame models, since the loss of lateral stiffness associated to the rocking mechanism is gradual and associated with the progressive plasticization of the nonlinear links that belong to the interfaces.

Figure 4 - Global pushovers in X directions comparison: summarized results for $SW_{G1}$ are reported in light gray as envelope.
Figures 4 and 5, owing to a productive comparison of software, we propose two stiffness configurations for the macro-element model: same parameters as for the continuum models (SW10*); reducing the shear modulus by a factor 2 (SW10). As reported in [6], unlike the continuous models, the macro-element allows to define separately the parameters that characterize the shear and bending behavior. Once the elastic Young’s modulus $E$ was determined based on the shear modulus and Poisson’s ratio values, the model was analyzed even in the presence of a 50% reduction in the modulus $G$ in order to evaluate the response considering a cracked state. The above is due to the use of an elastic-plastic shear bond assigned to two diagonal springs [6] which is not sufficient to simulate the degradation of stiffness. On the other hand, the bending behavior is linked to discrete interfaces of non-linear links so that, even in the presence of an elastic-plastic bond, the progression of damage along the aforementioned interfaces does not require a reduction of $E$.

3.2.2 Strength panel definition

Following the calibration procedures proposed in [6] for continuum modeling, the target panel strength is deduced by standard well-known analytical strength criteria [18]: strength characterization
is basically governed by the definition of the tensile and compressive uniaxial stress-strain curves. Uniaxial compressive strength is here assumed equal to the target masonry compressive strength, while compressive strain values are defined according to available literature. Together with the shear stiffness value $G$, for the macro-element model we select two values of the masonry friction coefficient. Respectively the use of the value 0.3 for which the response is close to that of the equivalent frame models [4], although lower than the peak values of the continuous models. For this reason, an increase of the parameter up to the value of 0.5 was adopted with a consequent increase in peak values and a better overlap on the results of the continuous models.

4 Calculation of bilinear equivalent curves

The calculation of the bilinear equivalent curves is presented and discussed, with respect to $X$ and $Y$ directions, for uniform distribution of load for the sake of paper length.

In some cases, the adoption of continuous models with nonlinear material can lead to shear-displacement curves without a marked softening branch. This aspect does not usually allow to clearly identify the ultimate displacement with respect to a criterion based on the base shear decay. In order to overcome that we adopt a different criterion that controls the structure ultimate displacement instead.

In particular, following the standards direction, the panel drift is computed and controlled for a significant number of masonry piers. It worth to mention that this is an ex-post operation for continuum models and requires specific extraction of data based on the custom selection of reference sections.

Then, the construction of the bilinear equivalent curves is based on three parameters: the stiffness $K$, the shear $V_Y$ at yielding and the ultimate displacement $d_u$. These parameters are computed using the pushover curves by means of the following common criteria:

- Stiffness $K$ was assessed by imposing the intersection of the equivalent bilinear curve to the point of the pushover curve corresponding to the base shear equal to 70% of the maximum value.
- Ultimate displacement $d_u$ was identified at 20% decay of the base shear from the maximum value; this level of displacement is assumed to be representative of the Ultimate condition.
- Base shear at yield $V_Y$ was determined by imposing the equality of the areas under the original pushover curve and of the bilinear curve until the ultimate displacement.

Figures 6 and 7 show the computed bilinear curves where results for the macro-element model are reported using both the configuration about the definition of stiffness and strength of the panel.
Table 5 reports the mean values conventionally adopted to compare the three parameters that define the equivalent bilinear curves (derived from the data presented in Section 3.2 according to the criteria illustrated in Section 3.1).

To foster the comparison between SW$_{G1}$ and SW$_{G2}$ software results in Figures 8 and 9 the percentage variation of parameters is computed and illustrated with respect to the averaged quantities provided by SW$_{G1}$ for $X$ and $Y$ directions, respectively. This choice is done for comparison purpose only since no judgment of reliability is reported or discussed. With this in mind we track that: (i) stiffness ($K$): the percentage variations relative to the reference value reach the 140%; the percentage variations
relative to their average (i.e. average of SWG2) reach 22% if SW10 is disregarded, whether it is limited to a maximum of 16% for SWG1.

Table 5: Mean reference values of the three parameters which define the bilinear equivalent curves

| Analyses    | $V_y$ [kN] | $K$ [kN/mm] | $d_u$ [mm] |
|-------------|------------|-------------|------------|
| $X_+$ uniform | 8487       | 2467        | 14         |
| $X_+$ triangular | 7766    | 1998        | 19         |
| $X_-$ uniform | 8719       | 2343        | 15         |
| $X_-$ triangular | 7956    | 1866        | 19         |
| $Y_+$ uniform | 7874       | 2455        | 17         |
| $Y_+$ triangular | 7045    | 1999        | 23         |
| $Y_-$ uniform | 8244       | 2385        | 15         |
| $Y_-$ triangular | 7105    | 1922        | 22         |

Figure 8 Percentage variation of the three parameters that define the bilinear equivalents for $X$ direction analyses.

Percentage variations are reported on a purely conventional and comparative basis respect to the reference value calculated as average of the SWG1 estimates.

The significant difference between the results obtained from the two groups of models is due to the different stiffness degradation, as previously illustrated. (ii) overall base shear ($V_y$): the percentage variations relative to the reference value reach the 39%; the percentage variations relative to their average reach 9% if SW10 is disregarded, whether it is limited to a maximum of 16% for SWG1. (iii) Ultimate displacement ($d_u$): the percentage variations relative to the reference value reach the 71%; the percentage variations relative to their average reach 23% if SW10 is disregarded, whether it is limited to a maximum of 42% for SWG1.
Percentage variations are reported on a purely conventional and comparative basis respect to the reference value calculated as average of the SW_{G1} estimates.

4.1 Cracking pattern comparison

As illustrated in [6] the parameters required to input the continuous model influence the element’s capacity curve in terms of stiffness, resistance, and ultimate drift. Since the calibration is very complex, it is essential to ensure that the capacity and drift of individual panels are within acceptable ranges and verifiable using the data available in the literature. Careful evaluation of the uncertainties related to the parameters to be used and of the complex mathematical model is necessary. A possible means of evaluation is by developing different models to assess the impact that the various hypotheses have on the results. Therefore, it is useful to make subsequent verifications to assess the reliability of the solution with simplified manual calculations and controls.

In addition, it should be noted that the verification format usually adopted in the regulatory field refers to magnitudes assessed at the scale of the panels; to finalize the verification, it is necessary to perform an ex-post reworking of the results (choosing predetermined sections in which to carry out integration operations, or for the interpretation of the damage modes). Moreover, the above-mentioned calibration of parameters together with the different modeling approach could lead to different stress distributions, and then, to different cracking pattern or failure modes.

Then, it becomes necessary to assess the validity of the proposed criteria, based on the ultimate drift of a significative number of masonry piers by the comparison with the cracking patterns and failure modes experienced by the real structure, see Figure 10.
Figure 10 Damage survey on Wall N.8 (a). Line thickness is proportional to the damage level (the thicker the line the more severe is the damage): severe damage (b), significant damage (c), light damage (d). The gray area refers to a collapsed part after the seismic event occurred the October 26th 2016.

Further difficulties may arise when reworking results for comparison with the equivalent frame models regarding the rendering of the damage framework at different analysis steps, intended as severity and type of damage suffered. In fact, continuous models can often obtain very detailed information (position and width of cracks, location of damaged portions in each wall panel, etc.), which are generally not easily unambiguously comparable with simplified damage frameworks obtainable using equivalent frame models.

In Figures 11, 12 and 13 the real cracking pattern is compared with the simulated cracking pattern produced by the software in terms of stress, strain damage or failure modes for walls W8-W10, W1, and W3-W5-W7-W9 respectively.
Figure 11: Pushovers performed in $Y$ direction: reference geometry with indication of damage survey over walls W8 and W10 (a): severe damage (red), significant damage (orange), light damage (yellow); (b) SW8 $Y+$; (c) SW8 $Y-$; (b) SW9 $Y+$; (c) SW9 $Y-$; (b) SW10 $Y+$; (c) SW10 $Y-$;
Figure 12: Pushovers performed in X direction: reference geometry with indication of damage survey over wall W1 (a): severe damage (red), significant damage (orange), light damage (yellow); (b) SW8 X-; (c) SW8 X+; (b) SW9 X-; (c) SW9 X+; (b) SW10 X-; (c) SW10 X+;
Figure 13: Pushovers performed in X direction: reference geometry with indication of damage survey over walls W3-W5-W7-W9 (a): severe damage (red), significant damage (orange), light damage (yellow); (b) SW8 X+; (c) SW8 X−; (b) SW9 X+; (c) SW9 X−; (b) SW10 X+; (c) SW10 X−;
5 Sensitivity to different modeling assumptions on the global response

5.1 Contribution of the load application method

The numerical response of the structure summarized by means of the pushover curve could change significantly if the external load is applied following different conventional mode of application. Conventionally, regardless the load patterns (i.e., triangular or uniform) load could be applied uniformly over the structure (condensing masses to each node of the mesh) or concentrating its action at floor levels (condensing level by level at the floor mid-plane). Note that the second choice is usually employed for equivalent frame models. It has been proven that this aspect of more pronounced when the wall ratio thickness to height tend to increase.

The BS5 is characterized by thick walls of 65cm of thickness of average, then it worth to assess whether the different choice could lead to significant different results.

The analyses were completed using two software programs from among those considered in the context of the study presented in the general document, operating within the framework of a FEM continuous modeling approach (SW8 and SW9), particularly versatile for examining the application of concentrated or distributed actions. The same mechanical properties and hypotheses for modeling the structure’s geometry described in Section 3.1 are adopted here.

A horizontal load distribution proportional to the masses, assuming a positively directed seismic action in the X direction, without accounting for the effects of accidental eccentricity, was applied to the structure. Figure 14 shows pushover curves. It is evident how the pushover curves attained from both software programs, considering the distributed actions, are characterized by peak loads substantially higher with respect to those obtained using concentrated actions. Specifically, softwares register an increase up to 22% of the maximum load when moving from lumped actions to distributed actions at floor level.

![Figure 14: Influence of the application of actions either lumped or distributed at floor level for Visso’s School. Comparison between SW8 and Software SW9.](image-url)
use of distributed horizontal actions. However, this effect favors safety, so the use of horizontal actions concentrated at the floor level appears to be an acceptable simplification in the pushover analysis of masonry buildings, although, as the thickness of masonry piers increases, the capacity of the structure is gradually underestimated.

5.2 Maximum acceleration calculation compatible with various limit states

The comparison of the maximum acceleration (PGA) is carried out for different performance levels (PL): (i) \(PGA_{vy}\) at yield point of the equivalent bilinear curve and (ii) \(PGA_{du}\) at the ultimate displacement computed for pushover curves. Considering the uniform distribution only.

As previously established, the N2 method was used for the evaluation of PGA. According to that method, the differences that play a role in the final calculations of the PGAs are:

• the differences of the parameters \((F, K, d_u)\) that define the equivalent bilinear form;

• the differences of the conversion factors of system SDOF\((\Gamma, m^*)\), which allow the conversion of the global base shear and the stiffness in system yield acceleration \((A_y = F_y / \Gamma m^*)\) and in period \(T^*\).

The final value is also affected by the assumed value for \(T_C\) (the period separating the spectrum regions at constant acceleration and velocity) and its relationship to \(T^*\).

The result is useful to illustrate how differences in the representation of the nonlinear response of the structure can affect the final safety assessment. The following parameters have been adopted to calculate the spectral form: (i) \(S=1.52\); (ii) \(T_C=0.714\); (iii) \(F_o=2.363\). Figure 15 shows the relationship obtained for \(T^*/T_C\). In each case, the relationship between \(T^*/T_C\) was less than 1; thus, the calculation of the expected seismic demand is made with reference to the region with maximum response spectrum amplification.

![Figure 15: \(T^*/T_C\) relationship for different software, for both directions and uniform distribution.](image)

For the calculation of \(\Gamma\) (Figure 16), for the SW\(_G2\) (i.e. SW8, SW9 and SW10) there are no differences between the different distribution of forces adopted. This is consistent with the assumption that this modal participation factor is calculated by most of the software with reference to the eigenvector form corresponding to the first mode (approximated, where appropriate, with the deformation resulting from the application of a system of forces proportional to the distribution of the product of the nodal...
masses for their dimensions on the modeled structure in the elastic field,) regardless of the forces applied in the nonlinear static analysis. This assumption, as highlighted in the companion paper, is not adopted by all the SWG1, because for two of them a unitary value of $\Gamma$ is assumed as default in case of uniform load pattern.

Figures 17, 18 and 19 shows values in different software with varying force distributions applied and differentiated analysis directions ($X$ and $Y$, positive and negative) for $M^*$, $\Gamma M$ and $q^*$ respectively.
Finally, figures 20 and 21 show the PGA and percentage variations obtained: it is worth noting that the percentage variation is assessed according to the reference values obtained by the SWG1 on a conventional and purely comparative basis. It can be seen that the percentage variations reach a maximum of 48%, and with a mean (considering the four analyses treated) of about 10% for PGA-$V_y$; and up to a maximum of 30%, and with a mean (considering the four analyses treated) of about 11% for PGA-$d_u$. 

Figure 20: PGA values obtained for the two performance levels examined.
Figure 21: Percentage variations of PGA respect to the reference value calculated as average of $SW_{G1}$ estimates for the two performance levels

6 Conclusions

The paper presents the comparison of the results obtained on a masonry building by nonlinear static analysis using different software operating in the field of continuum and discrete-macroelement modeling. The activity described is part of a research carried out by several research teams involved in the “URM nonlinear modelling - Benchmark project” on the benchmark structure BS5, inspired by the “P. Capuzi” school in Visso (MC, Italy), seriously damaged following the seismic events that affected Central Italy in 2016/2017. The benchmark structure has been selected in order to explore the influence of modeling approaches adopted by different software at both research and professional levels by means of nonlinear static analyses. Although not exhaustive, three commercial software were considered and compared. Furthermore, the results are compared with the outcomes obtained from equivalent frame models reported in the companion paper (Part I). The comparison of analyses was carried out in relation to: global parameters (concerning the dynamic properties, capacity curves and equivalent bilinear curves), synthetic parameters of structural safety (such as, for example, the maximum acceleration compatible with the performance levels) and the response in terms of simulated damage.

The results allow for some insights on the use of continuum and discrete-macroelement modeling, with respect to the dispersion of the results and on the potential repercussions in the professional field. This response was also analyzed considering different methods for the application of loads. With respect to the comparative study reported in the companion paper Part I the results confirm that the
dispersion achievable when different software packages are used is not completely negligible, especially when comparing different modeling techniques although it is generally contained within acceptable ranges when the consistency of the modeling assumptions are ensured.

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Conflicts of interest/Competing interests

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.

Availability of data and material

The benchmark structure analyzed in the paper (BS5) can be replicated by other researchers and analysts thanks to the input data provided in the paper made by Cattari and Magenes [2] as supplementary electronic material (Annex I-Benchmark Structures Input Data). Some additional data
support the findings of this study are available from the corresponding author upon reasonable request.

Authors’ contributions

GC: interpretation of results, writing-review, conceptualization, supervision; numerical analyses, data curation, writing-original draft, methodology, comparisons of results made by the research teams; BP: numerical analyses, data curation, writing-original draft, methodology, comparisons of results made by the research teams.; GO: numerical analyses, data curation, writing-original draft, methodology, comparisons of results made by the research teams; DT: numerical analyses, data curation, writing-original draft, writing-review, methodology, comparisons of results made by the research teams; LB: numerical analyses, data curation, writing-original draft, writing-review, methodology, comparisons of results made by the research teams; GC: numerical analyses, data curation, writing-original draft, methodology;

References

[1] S. Cattari, D. Ottonelli, S. Degli Abbati, G. Magenes, C.F. Manzini, P. Morandi, E. Spacone, G. Camata, C. Marano, I. Caliò, B. Pantò, F. Cannizzaro, G. Occhipinti, B. Calderoni, E.A. Cordasco, S. de Miranda, G. Castellazzi, A.M. D'Altri, A. Saetta, D.A. Talledo, and L. Berto. Uso dei codici di calcolo per l'analisi sismica non lineare di edifici in muratura: confronto dei risultati ottenuti con diversi software su un caso studio reale. In editor, editor, Proc. XVIII ANIDIS, , Ascoli Piceno, Italy., 2019.

[2] S. Cattari and G. Magenes. Benchmarking the software packages to model and assess the seismic response of URM existing buildings through nonlinear analyses. Bulletin of Earthquake Engineering, submitted:

[3] M. Dolce, M. Nicoletti, A. De Sortis, S. Marchesini, D. Spina, and F. Talanas. Osservatorio sismico delle strutture: the Italian structural seismic monitoring network. Bulletin of Earthquake Engineering, 15(2):621-641, 2017.

[4] D. Ottonelli, S. Cattari, C. Marano, C.F. Manzini, and B. Calderoni. Nonlinear modelling of masonry structures: calibration strategies. Bulletin of Earthquake Engineering, (submitted)

[5] B. Calderoni, I. Caliò, S. de Miranda, G. Magenes, G. Milani, A. Saetta, and G. Camata. Nonlinear modelling of masonry structures: critical aspects. Bulletin of Earthquake Engineering, (submitted)

[6] A.M. D’Altri, F. Cannizzaro, M. Petracca, and D.A. Talledo. Nonlinear modelling of masonry structures: calibration strategies. Bulletin of Earthquake Engineering, (submitted).

[7] Jeeho Lee and Gregory L. Fenves. Plastic-damage model for cyclic loading of concrete structures. Journal of Engineering Mechanics, 124(8):892-900, 1998.

[8] ABAQUS/Standard User's Manual, Version 6.18. Dassault Systemes Simulia Corp, United States, 2018.
[9] G. Castellazzi, A.M. D'Altri, S. de Miranda, A. Chiozzi, and A. Tralli. Numerical insights on the seismic behavior of a non-isolated historical masonry tower. Bulletin of Earthquake Engineering, 16(2):933-961, 2018.

[10] A.M. D'Altri, F. Messali, J. Rots, G. Castellazzi, and S. de Miranda. A damaging block-based model for the analysis of the cyclic behavior of full-scale masonry structures. Engineering Fracture Mechanics, 209:423-448, 2019.

[11] Amir Mirmiran and Mohsen Shahawy. Dilation characteristics of confined concrete. Mechanics of Cohesive-Frictional Materials, 2(3):237-249, 1997.

[12] G. Milani, M. Valente, and C. Alessandri. The narthex of the church of the nativity in Bethlehem: A non-linear finite element approach to predict the structural damage. Computers and Structures, 207:3-18, 2018.

[13] Frank J. Vecchio and Michael P. Collins. Modified compression-field theory for reinforced concrete elements subjected to shear. Journal of the American Concrete Institute, 83(2):219-231, 1986.

[14] R.G. Selby and F.J. Vecchio. A constitutive model for analysis of reinforced concrete solids. Canadian Journal of Civil Engineering, 24(3):460-470, 1997.

[15] Midas FEA 2016 v1.1 - Build: Nov. 06, 2018. Nonlinear and detail FE Analysis System for Civil Structures. Midas Information Technology Co. Ltd., 2016.

[16] I. Caliò, M. Marletta, and B. Pantò. A simplified model for the evaluation of the seismic behavior of masonry buildings. Proceedings of the 10th International Conference on Civil, Structural and Environmental Engineering Computing, Civil-Comp 2005, pages 1-17, 2005.

[17] Ivo Caliò, Massimo Marletta, and Bartolomeo Pantò. A new discrete element model for the evaluation of the seismic behavior of unreinforced masonry buildings. Engineering