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Modeling Approaches for the Assessment of Seismic Vulnerability of Masonry Structures: The E-PUSH Program

Maria Luisa Beconcini, Paolo Formichi, Linda Giresini, Filippo Landi *, Benedetta Puccini and Pietro Croce *

Department of Civil and Industrial Engineering, University of Pisa, Largo Lucio Lazzarino 2, 56122 Pisa, Italy; maria.luisa.beconcini@unipi.it (M.L.B.); paolo.formichi@unipi.it (P.F.); linda.giresini@unipi.it (L.G.); benedetta.puccini@ing.unipi.it (B.P.)
* Correspondence: filippo.landi@ing.unipi.it (F.L.); p.croce@ing.unipi.it (P.C.)

Abstract: The assessment of seismic performance of existing masonry structures is a key aspect for the risk mitigation strategies of existing buildings and preservation of historical heritage. The increasing availability of modelling approaches for the assessment of seismic response of masonry structures calls for the need of verifying their reliability and correct use. In fact, these procedures are very sensitive to modelling hypotheses, so that the results of the assessment could vary in a wide interval depending on the adopted software and on the user's skill. Aiming at enhancing the classical software packages for the structural analysis of masonry buildings, especially in terms of easiness of use, simplicity of modelling and limited computational demand, the authors developed a reliable and sound push-over program, called E-PUSH, which allows a quick and nearly user-independent assessment of the seismic risk index. In the paper, available commercial codes for the seismic assessment of unreinforced masonry buildings are illustrated and discussed, in comparison with the E-PUSH program, highlighting the differences in terms of modelling assumptions, choice of masonry mechanical parameters and failure criteria, focusing on the impact of the assumptions adopted for the estimation of capacity curves and seismic risk index of a simple benchmark structure. Then, a relevant case study, consisting in the assessment of the "Niccolò Machiavelli" masonry school in Florence, is investigated adopting two different software packages, the original E-PUSH and a commercial one, discussing the sensitivity of the results on the assumptions made by the user in the modelling phase.

Keywords: existing masonry structures; seismic risk; modelling techniques; equivalent frame models; pushover analysis

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1. Introduction

The dramatic consequences of recent seismic events in southern Europe emphasized once more the high seismic vulnerability of the built environment [1–4]. This seismic vulnerability is particularly significant not only for critical modern structures and infrastructures [5], but also for existing constructions erected following mainly empirical rules, in the absence of specific seismic provisions. The set-up of reliable procedures for the evaluation of the seismic performance of masonry structures is thus fundamental for the definition and planning of interventions identifying risks and priorities for seismic upgrading [6]. Evidently, the outcomes of seismic vulnerability evaluations significantly impact not only the engineering practice at the scale of the single existing building, but also the mitigation and management policies implemented at broader scales by public administrations and owners of large construction portfolios [7].

A reliable seismic assessment requires the knowledge of the masonry mechanical properties [8,9] and availability of appropriate mechanical models for the behavior of masonry elements [10,11], combined with coherent modeling strategies.

At present, depending on the intended field of application, several different approaches for modelling masonry structures are available, based on macro-modelling, or micro-modelling, characterized by various levels of complexity and refinement. These methods aim at assessing the building response considering both in-plane modes and out-of-plane modes. Firstly, in existing masonry buildings out-of-plane modes should be investigated to exclude local mechanisms which would impede a global response, also through probabilistic approaches [12]. Recently, an earthquake resistant device was conceived to mitigate the seismic vulnerability of out-of-plane modes [13,14], properly assessing the influence of stiffness and damping parameters according to site and design parameters [15]. Among the methods to investigate the in-plane behavior of masonry buildings, macro-modeling approaches, based on one-dimensional elements and simulating the structural response by means of an equivalent frame analysis, are often adopted in current engineering practice.

Starting from the first significant computer program for the non-linear static analysis of unreinforced masonry structures, the so-called POR method [16,17], proposed in the 1970s, several macro-modelling software packages have been proposed, especially in recent years [5,18–25]. Notwithstanding the increasing availability of commercial software packages for the structural analysis of masonry buildings, they are generally heavily dependent on the user's choices. Consequently, there is a strong need for assessing their reliability, their correct use, and their sensitivity on the modelling approach. The benchmarking of practitioner-oriented software packages is thus becoming an important task, since the different modelling assumptions and the analysis methods adopted by each software may lead to large discrepancies in the results, with significant consequences for the assessment of seismic vulnerability [24,25].

In the paper, first, macro-element methods for seismic analysis of masonry buildings are presented and discussed, highlighting the relevant assumptions in terms of masonry modelling, masonry mechanical properties and failure criteria, and stressing their influence on the evaluation of capacity curves and on the estimation of seismic risk index on a typical benchmark structure. The main software packages currently available are investigated together with an ad-hoc original and open-source software, previously developed by the authors, called E-PUSH [6,23], with the objective to refine the classical pushover approach for masonry buildings, improving easiness of use, clarity of the modelling and reducing the computational demand, allowing a quick and nearly user-independent evaluation of the seismic risk index.

A significant case study, the "Niccolò Machiavelli" school in Florence, is then presented. This case study has been selected within a large set of masonry buildings studied by the authors in the framework of a research agreement funded by the Municipality of Florence, aimed at assessing the seismic vulnerability of masonry school buildings. This case study is investigated adopting two different software packages, the original E-PUSH and the commercial one Aedes PCM [26], discussing the influence of the assumptions made by the user in the modelling phase.

2. Macro-Elements Methods for Structural Analysis of Masonry Buildings

Modelling strategies, analysis methods and assessment criteria are the key points for the structural assessment of existing masonry buildings. In fact, the identification of the underlying structural model can be difficult for such structures, often built according to empirical rules, without following codified provisions. Moreover, in many cases, modifications and alterations occurred over times are neither evident nor documented.

The structural analysis of existing buildings requires, first, suitable, reliable, user-friendly and not computationally expensive mechanical models of the structure. The definition of such numerical models, describing the non-linear mechanical behavior of masonry panels, is not trivial; in fact, it depends on complex constitutive laws, requiring the knowledge of several mechanical parameters, whose evaluation is generally affected by various sources of uncertainty. Therefore, the choice of the most appropriate model is not univocal. Appropriate modeling strategies include macro-element methods [19–26], 2D continuum models [27,28], and, in the case of a high level of modeling detail, advanced micro-modelling approaches considering masonry as a heterogeneous medium, made of mortar and bricks joined by interfaces [29]. The detailed discussion of such approaches is out of the scope of the present paper, but an extended review of modelling approaches for masonry structures can be found in [29,30].

It must be underlined that micro and meso models have been mostly used for the analysis at element scale and for research purposes [25], while in current engineering practice, macro-element methods, requiring more synthetic input data and lower computational efforts, are generally adopted to assess seismic performance at the building scale. When macro-element modeling is adopted, the structure is idealized into mono-dimensional structural components characterized by a phenomenological nonlinear response [26]. In any case, despite their apparent simplicity, these models often require very skilled users: in fact, on a given structure, they often provide results which are heavily dependent on the individual assumptions setup by the user in the modeling phase.

Macro models commonly rely on the definition of an equivalent frame, in which walls are divided into macro-elements: piers and spandrels, or lintels, clamped at nodes, and provided at their ends with suitable rigid links, if necessary. This Equivalent Frame Model approach leads to a 3D model of the structure, enabling a global analysis. Vertical frame elements (columns) and horizontal frame elements (beams) reproduce mass, stiffness, and strength of the corresponding elements: wall panels, spandrels (or lintels). Most commercial software programs are based on the Equivalent Frame Model approach [26,31–35], but spring-based approaches were also developed in recent years [21,36]. In any case, the outcomes of these software programs can be very scattered especially in terms of seismic performance [25] due to different assumptions about the discretization of structural components, the material properties, and the load distributions.

Evidently, the definition of the model requires one to discretize each masonry wall in structural frame components: deformable parts of piers and spandrels or lintels, and rigid links, duly connecting the flexible parts to the nodes. The default modelling scheme mostly adopted is the one proposed by Dolce in [37], but, since the effective length of the equivalent frame elements can be defined in different ways, its default value depends on the software, even if the length of elastic beams and rigid links can be suitably modified by the user, like in Aedes PCM [26]. The schematization is clearly different for spring-based models [21,36] where all the structural components are modelled as deformable, and the wall discretization results from the openings layout. An example of the discretization of structural components, piers and spandrels, adopted by the most common commercial software packages for a simple wall, is shown in Figure 1.

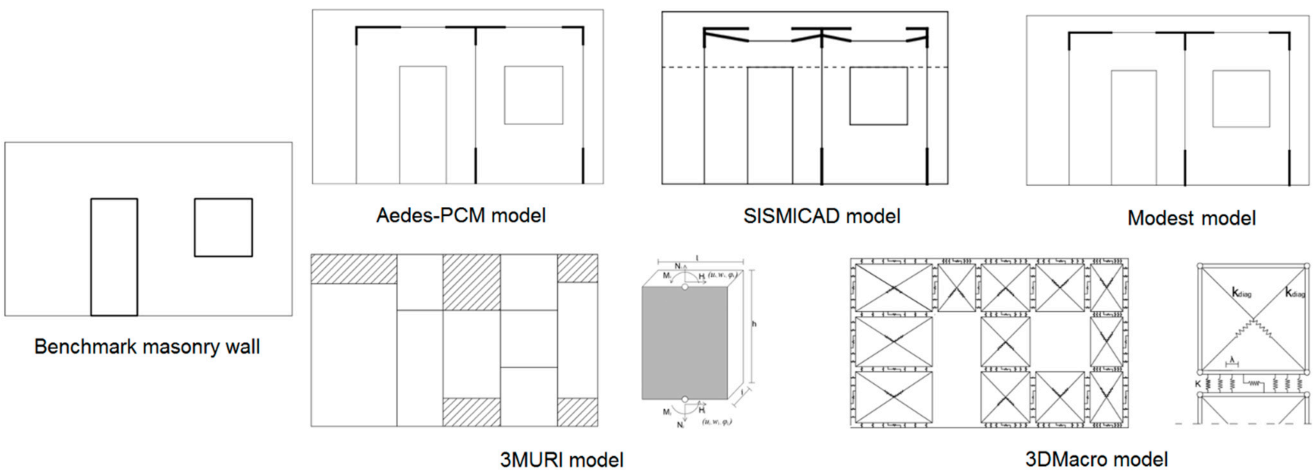


Figure 1. Examples of macro-elements models for the benchmark masonry wall according to different software packages: Aedes-PCM, SISMICAD, Modest, 3MURI, and 3DMacro.

A large variation is observed again regarding the modeling of horizontal diaphragms, ranging from the lack of floors to finite stiffness diaphragms in case of flexible floors, and fully rigid diaphragms in the case of r.c. slabs or horizontal bracings able to ensure a 3D "box" behavior of the structures.

Once the structural model is defined, the further key element is the adoption of a suitable constitutive law to describe the non-linear response of masonry elements. The shear behavior is generally idealized with a bilinear elastic–plastic curve, characterized by an initial elastic slope defined by the lateral stiffness k_e and by a plastic plateau limited by the elastic inter-story drift δ_e and by the ultimate inter-story drift δ_u. The evaluation of the shear rigidity and of the shear resistance is the basis for a sound assessment of the seismic vulnerability of a masonry building [6,8,9,38,39]. However, significant uncertainties affect the results in terms of capacity curves and masonry material properties [40–43], such as elastic and shear modulus and shear strength, to be used for modelling the overall response of a structural component.

The resistance, the effective stiffness, and the shear failure mechanisms of the wall depend on the masonry type, the quality of mortar, the compressive stresses, and other influencing parameters, [40–47]. As in-situ experimental tests are necessarily limited for practical and economic reasons, the available results should be supported by visual inspection methods [48] and engineering judgements. A Bayesian methodology for the evaluation of masonry classes and the associated probability density functions for mechanical parameters is proposed in [7]. The procedure, starting from the analysis of a large database of test results, allows the updating of masonry parameters considering local information on masonry quality obtained by visual inspection and limited in situ compression tests performed with flat jacks. The achieved calibration of masonry mechanical properties provides the basis for a more refined seismic assessment.

To the aim of adequately modelling the shear behavior of walls, further to the essential estimation of masonry material properties, the appropriate definition of the deformation capacity of masonry walls plays a key role [49–51]; this capacity depends on several factors, such as the boundary conditions of the wall, the axial load ratio, the height of the wall, the loading history and the strain rate [49]. Nowadays, the drift capacity is generally defined as a percentage of the inter-story height depending on the failure mode: see, for example, the Guidelines for the application of the Italian Building Code [52,53], EN 1998–3 [54], New Zealand NZSEE [55] and FEMA 273 [56], while in the past, it was associated with the definition of a ductility factor μ = 1.5–2 [16,17].

Table 1 summarizes the main modeling assumptions for the in-plane response of masonry walls made by Aedes PCM [26], 3MURI [22,33] and 3DMacro [21,36]. A complete review of non-linear methods for the seismic assessment of masonry buildings based on macro modelling can be found in [24], where the most used commercial software packages in Italy, namely, Aedes PCM, 3MURI, Modest [34], Sismicad [35] and 3DMacro, are presented and compared with the E-PUSH software package on different case studies.

Table 1. Strength criteria for masonry walls implemented in the macro-element programs, Aedes PCM, 3MURI and 3DMacro. In the formulas, l is the length of the wall, t is the thickness, l' is the length of the compressed part of the wall, τ_k is the shear strength, f_{vk0} is the sliding shear strength, c is the cohesion coefficient, μ is the friction coefficient, f_c is the compressive strength, σ_0 is the compressive stress on the wall and b is the stress distribution factor.

Program	Failure Mode	Strength Domain	Description
Aedes PCM	Sliding shear Diagonal shear Rocking/Crushing	$V_{Rd} = l' t (f_{vk0} + 0.4\ \sigma_0)$ $V_{Rd} = l\ t \frac{1.5\ \tau_k}{b} \sqrt{1 + \frac{\sigma_0}{1.5\ \tau_k}}$ $M_{Rd} = l^2\ t \frac{\sigma_0}{2} \left(1 - \frac{\sigma_0}{0.85\ f_c}\right)$	The limit domain is obtained by assuming an elastic perfectly plastic constitutive law (bi-linear curve) where the strength is determined by the minimum value predicted by flexural and shear responses. The deformation capacity of each wall can be determined by means of a ductility check multiplying the elastic displacement by a ductility factor μ = 1.5–2 , or setting the limit value of the inter-story drift equal to 0.004 h and 0.006 h, in case of shear or rocking failure, respectively.
3Muri	Sliding shear Diagonal shear Rocking/Crushing	$V_{Rd} = l'\ t (c + \mu\ \sigma_0) \leq V_{blocks}$ $V_{Rd} = l\ t \frac{1.5 \tau_k}{b} \sqrt{1 + \frac{\sigma_0}{1.5\ \tau_k}}$ $M_{Rd} = l^2\ t \frac{\sigma_0}{2} \left(1 - \frac{\sigma_0}{0.85\ f_c}\right)$	The limit domain is obtained by assuming an elastic perfectly plastic constitutive law (bi-linear curve) where the strength is determined by the minimum value predicted by the flexural and shear responses. The deformation capacity is determined by the inter-story drift limit.
3D Macro	Sliding shear Diagonal shear Rocking/Crushing	$V_{Rd} = l'\ t\ (c + \mu\ \sigma_0)$ $V_{Rd} = l\ t \frac{1.5 \tau_k}{b} \sqrt{1 + \frac{\sigma_0}{1.5\ \tau_k}}$ $M_{Rd} = l^2\ t \frac{\sigma_0}{2} \left(1 - \frac{\sigma_0}{0.85\ f_c}\right)$	The flexural mode is controlled by the orthogonal nonlinear links. Two elastic-plastic springs in series are defined for adjacent panels with stiffness k_1 and k_2, respectively, $k_1 = \frac{E\ \lambda\ t_1}{h_1};\ k_2 = \frac{E\ \lambda\ t_2}{h_2}$ where t_i is the thickness of the panel and λ is the di stance between two nonlinear links.The sliding-shear failure mode is governed by the longitudinal nonlinear springs of the interfaces, which are modelled by means of a rigid-plastic constitutive behavior governed by a Mohr–Coulomb yielding surface. The diagonal-shear failure mode is defined by two diagonal non-linear springs with initial stiffness given by $k_{diag} = \frac{G\ A}{2\ h \cos^2 \alpha}$ The deformation capacity is given by a specific value of the ultimate angular deformation: $\gamma_u = \delta_u / h$.

2.1. E-PUSH Software Package

The E-PUSH program is an original method for nonlinear static analysis of masonry building, developed by the authors [6,23] with the aim to enhance at the same time the easiness of use and the user friendliness of classical pushover procedures for the analysis of masonry buildings. The program relies on a simple structural model, which is nearly independent of the users' skill [38] and requires a limited number of input data: namely, the geometry and the location of the walls, the compressive stress σ_0 induced on them by the quasi-permanent load combination, as defined in Eurocode EN1990 [57], the material properties (the elastic modulus, E, the shear modulus, G, the shear strength, τ_k and the compressive strength, f_c).

The E-PUSH program allows one to perform various types of analysis, depending on the effectiveness of wall connections and on the in-plane stiffness of floors, according to the following hierarchy:

- in case of rigid and resistant floors, leading to a box behavior of the building, a global 3D non-linear static analysis can be performed;
- in case of flexible floors, and aligned wall panels connected by resistant masonry spandrels, or lintels, or by r.c. curbs, only a 2D non-linear static analysis can be performed, focused on the masonry panels part of the considered alignment;
- in case of no adequate connections between adjacent walls, a simple linear static analysis can be carried out.

Obviously, the possibility of performing a given type of analysis implies the possibility to develop the hierarchically inferior types of analysis. Each structural element is assessed according to different failure criteria (diagonal shear, in-plane and out-of-plane bending), as defined in the pertinent structural codes [52–56].

Focusing on 3D and 2D non-linear static procedures, the program is based on the following usual assumptions:

- only wall panels extending from a given floor to the foundations are taken into account;
- each shear wall is assumed to be effective only in its longitudinal direction; therefore, only the lateral stiffness of the wall is considered, disregarding the transverse (out-of-plane) stiffness;
- the capacity curve of each wall is approximated by a bi-linear elastic-plastic curve, where the plastic plateau, defined by the ultimate shear resistance given by the diagonal or sliding shear failure, is bounded by the elastic drift δ_e and the ultimate drift δ_u, for which different formulations can be set (e.g., in terms of a ductility factor or considering and inter-story drift limitation);
- the equivalent SDOF (single degree-of-freedom) system bi-linear force-displacement capacity curve of the whole structure is considered, to perform verification according to the N2 method [58] on the Acceleration Displacement Response Spectra (ADRS) plane.

During the analysis, at each step of the iterative procedure, which proceeds from the highest floor to the foundation, the lateral forces are increased, and the inter-story drift of each shear resistant wall is compared with the elastic drift, δ_e, and the ultimate drift, δ_u, considering the three possible situations:

- the wall is still in elastic phase, as its drift, δ, satisfies $\delta \leq \delta_e$: the stiffness of the wall is the elastic one, k_e, and the shear force H is $H = k_e\, \delta$;
- the wall is in the plastic phase, $\delta_e < \delta \leq \delta_u$: the shear force is equal to the wall resistance and an apparent stiffness k can be assumed, given by:

$$k = \frac{\delta_e k_e}{\delta} \tag{1}$$

- the wall is collapsed, $\delta > \delta_u$: its shear resistance and its stiffness are set to zero and the wall is assumed to sustain only vertical loads.

The analysis terminates when the base shear resistance reduces to a given percentage of the relative maximum base shear resistance, for example, 80%, or the walls pertaining to the same floor collapse. In this way, the force-displacement capacity curve of the whole structure is derived.

The E-PUSH program has been validated focusing on its capability to simulate the seismic response of masonry building prototypes tested in the laboratory [6] and has been tested on several real case studies, comparing the numerical outcomes with those obtained by means of commercial software [24,38]. The software is distributed "open source" under a Creative Commons License (CC BY-ND 4.0) and its implementation in a BIM environment is currently ongoing [59]. In the following sub-section, the E-PUSH software is tested on the benchmark exercise described in [25].

2.2. Assessment of Seismic Performance of a Benchmark Structure

As anticipated, the seismic assessment of masonry buildings is highly influenced by assumptions and methods adopted for the study. The results of a benchmark exercise aiming at comparing the outcomes of different software packages for the analysis of an unreinforced masonry building has been recently presented and discussed in [25]. The benchmark exercise, firstly introduced in a Special Session of the 16th European Conference of Earthquake Engineering [60], is also a reference for other researchers to assess modelling strategies, not directly considered in the comparison.

The main characteristics of the building, geometry and structural configuration, and the masonry mechanical properties are shortly recalled here. Complementary input data, presented in the original document, are provided as supplementary material in [25].

The three-story masonry building, 11.3 m high, is a typical example of Mediterranean and Central European existing units [25] and it is characterized by a rectangular layout, 8.5 m $\times$ 10 m in plan. Two different, but typical, structural configurations were considered, varying the typology of the wall panels:

- Case (A) double-leaf cut stone walls, bonded with lime mortar, connected by transverse stones; the wall external walls at ground and first floor were 0.55 m thick, while all the remaining walls were 0.45 m thick. The floors were flexible wooden diaphragms (dead load 1 kN/m^2 and live load 2 kN/m^2).
- Case (B) English bond solid clay brick walls, bonded with lime mortar: the wall external walls at ground and first floor were 0.38 m thick, while all the remaining walls were 0.265 m thick. The floors were rigid r.c. diaphragms (dead load 3.5 kN/m^2 and live load 2 kN/m^2).

The same roofing system was assumed for both configurations: a double slope roofing type consisting of a timber structure with rafters supported by joists. The assumed dead load of the roof was 1.25 kN/m^2. The relevant masonry mechanical properties pertaining to the two configurations can be found in [25], while the plan layout of the shear resistant walls at each story is shown in Figure 2, referring to the mentioned Case B.

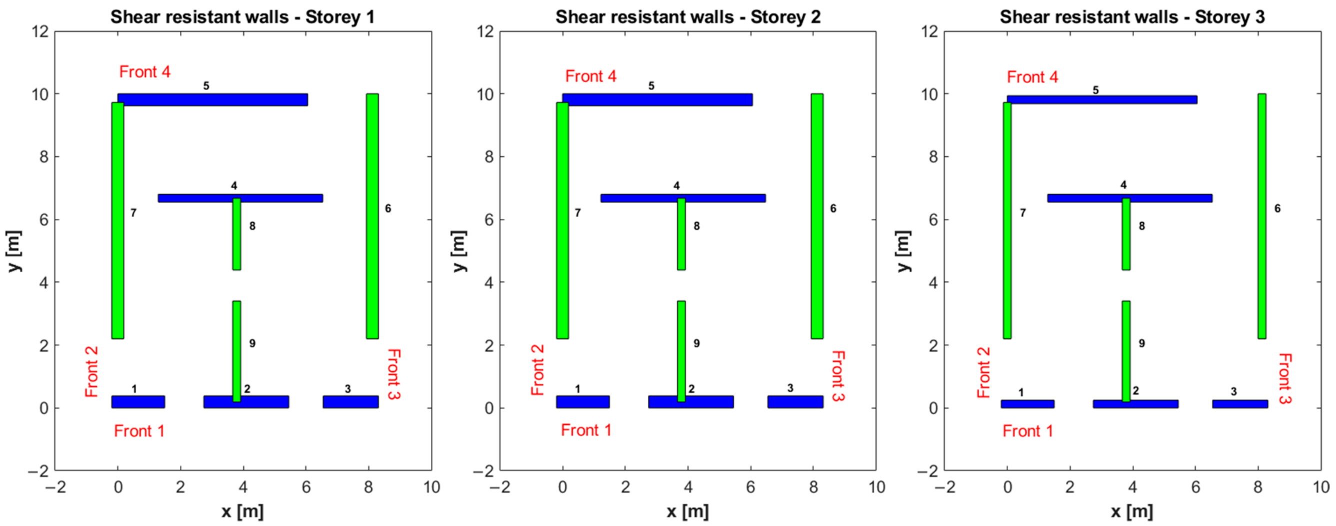


Figure 2. Layout of the shear resistant walls of the benchmark masonry building (wall thicknesses refer to Case B).

The benchmark structure, represented in Figures 2 and 3, has been modeled by means of the E-PUSH program, and a 3D non-linear static seismic analysis has been performed for the Case B configuration, which allows one to consider a 3D box behavior, being characterized by rigid diaphragms. The relevant mechanical properties are recalled in Table 2.

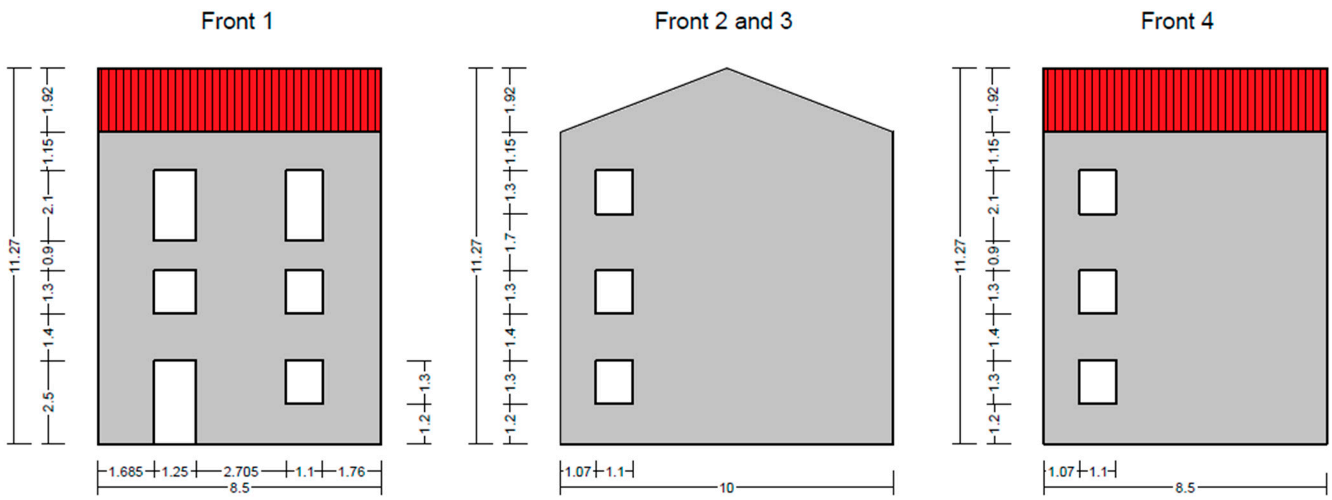
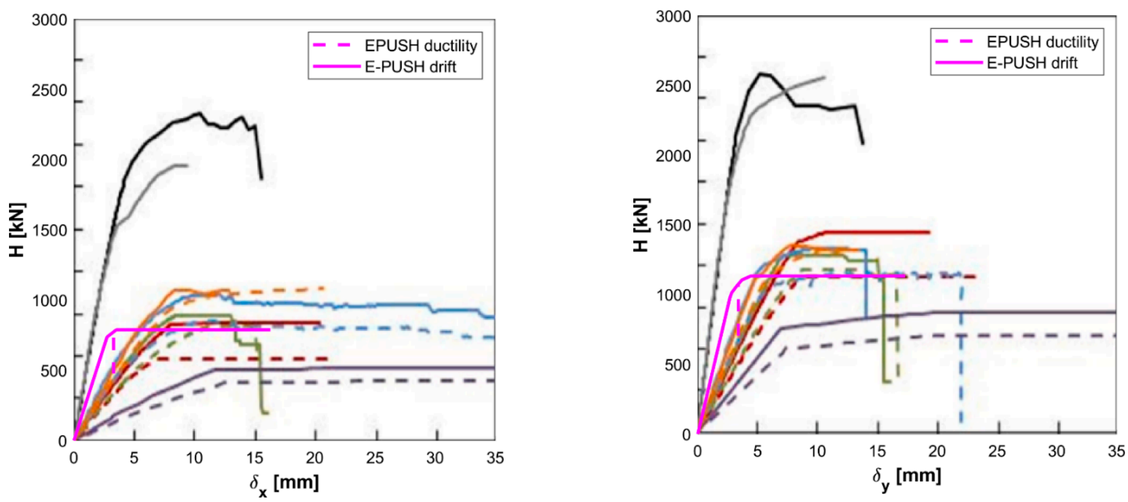


Figure 3. Front and side views of the benchmark masonry building (unit: m).

Table 2. Masonry mechanical properties of Case B configuration: elastic modulus E, shear modulus G, specific weight w, masonry compressive strength f_m, brick compressive strength f_{bm}, mortar joint friction coefficient μ, mortar joint tensile strength f_{mt}, mortar joint initial shear strength f_{v0}.

E (MPa)	G (MPa)	w (kN/m^3)	f_m (MPa)	f_{bm} (MPa)	μ	f_{mt} (MPa)	v_0 (MPa)
1500	500	18	7.5	14	0.6	0.12	0.2

The capacity curves obtained by means of the E-PUSH program are shown in Figure 4, for the longitudinal direction (x-axis in Figure 4a) and the transversal direction (y-axis in Figure 4b). In Figure 4, they are also reported the capacity curves obtained in [24] according to different software packages (Aedes.PCM, ANDILWall [32], 3MURI, 3DMacro, LUSAS [61], MIDAS Gen [62]). It must be stressed that the above-mentioned reference capacity curves are reported in [25] in anonymous form, i.e., they are not associated to a specific software.



(**a**) (**b**)

Figure 4. Comparison of capacity curves obtained from the benchmark exercise Case B in [24] and those obtained from the E-PUSH software (in magenta) (**a**), Longitudinal direction X, (**b**) Transversal direction Y. (Longitudinal direction is the shorter dimension in the plan.)

Considering the near collapse (NC) limit state, the equivalent bi-linear elastic–plastic capacity curves provided by E-PUSH, according to the N2 method, show that the building can sustain a peak ground acceleration (PGA) of 0.285 g in longitudinal direction, and of 0.37 g in transversal direction. It can be remarked that the results are very encouraging; moreover, the predicted PGA values are very close to the mean values predicted by means of the investigated software packages in [25]. However, it must be stressed that results provided by various software are very scattered.

3. Case Study: The Secondary School "Machiavelli"

To further validate it, the outcomes provided by the E-PUSH program have been compared with the outcomes of the commercial software Aedes-PCM referring to a very relevant and rather complex real case study, the secondary school "Machiavelli" in Florence, as discussed in the following.

3.1. Description of the Building

The secondary school "Machiavelli" is a masonry building located in the Municipality of Florence (IT). The building, built at the beginning of the twentieth century, is characterized by a horseshoe layout: the area of the first two stories is about 2000 m^2, while the third story extends over a more limited portion, which area is about 400 m^2, located at the ends of the central block. The basement, ground and first floors differ only in the presence of the gymnasium at the ground floor. A picture of the school is shown in Figure 5.

Figure 5. The secondary school "Machiavelli" in the Municipality of Florence.

The load-bearing walls of the ground floor have a height variable between 5.50 m and 6.15 m, excluding the gymnasium, whose height is around 6.80 m, and the eaves and the ridge, whose height is 8.20 m. On the first floor, the height is 5.35 m, while on the second floor, the minimum height is 3.70 m. The plan of the ground floor, a relevant section (section A-A), and the west elevation are illustrated in Figure 6.

The perimeter walls at the first two levels are made of stone masonry with brick course. These external walls are usually about 60 cm thick, excluding those in proximity of the "small towers", which are about 100 cm thick. Excluding some isolated walls, made of solid bricks and lime mortar, the internal walls are mainly characterized by stone masonry with brick courses: they are about 50 cm thick. The perimeter and internal walls at the third level are also made of stone masonry; they are about 50 cm thick.

Rigid horizontal diaphragms, composed by r.c. slabs, are present at the first floor and, for limited portions, at the second floor. No horizontal diaphragm is present at the last level under the roof. The roofing system is a double slope timber structure, where rafters are supported by wooden trusses.

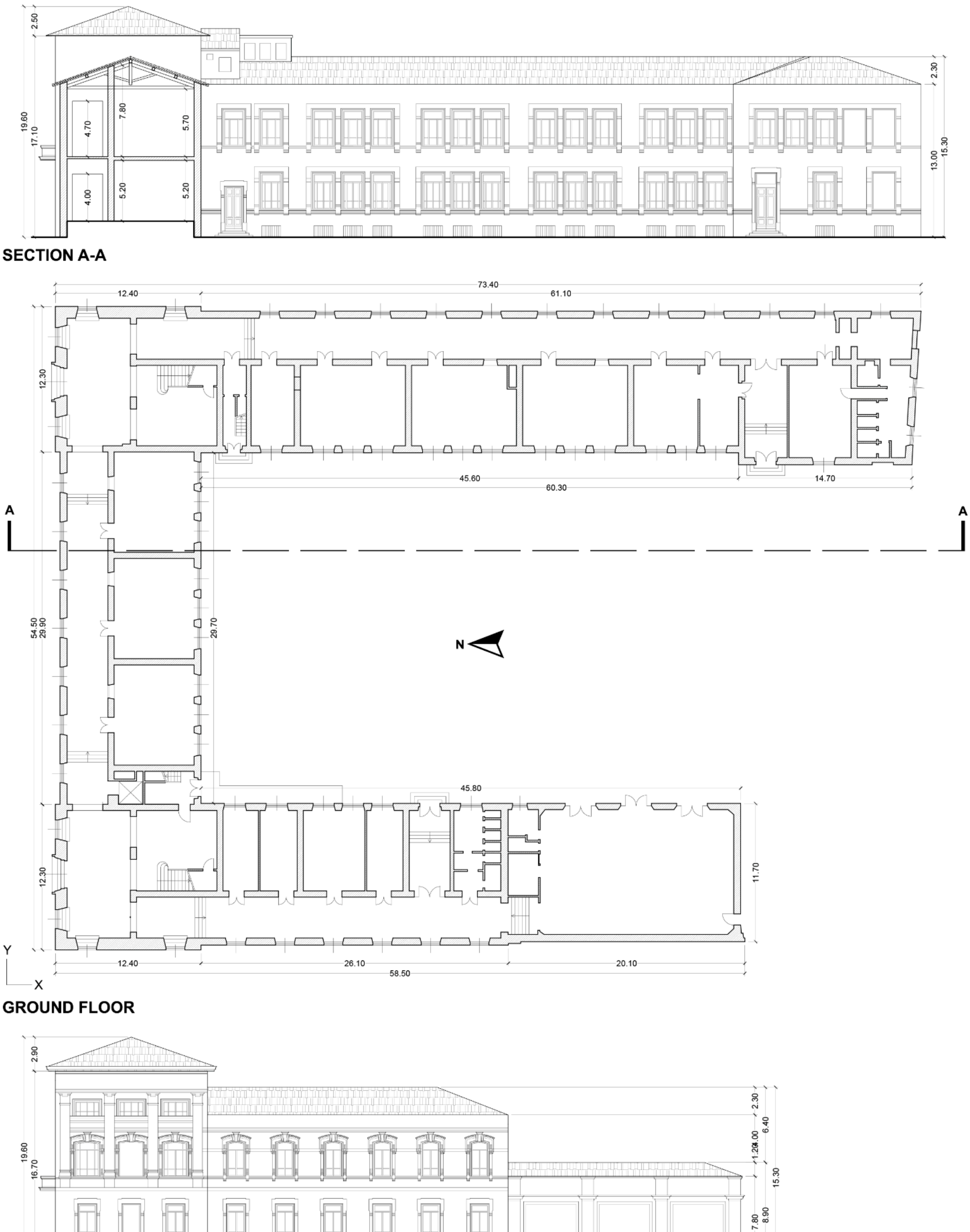
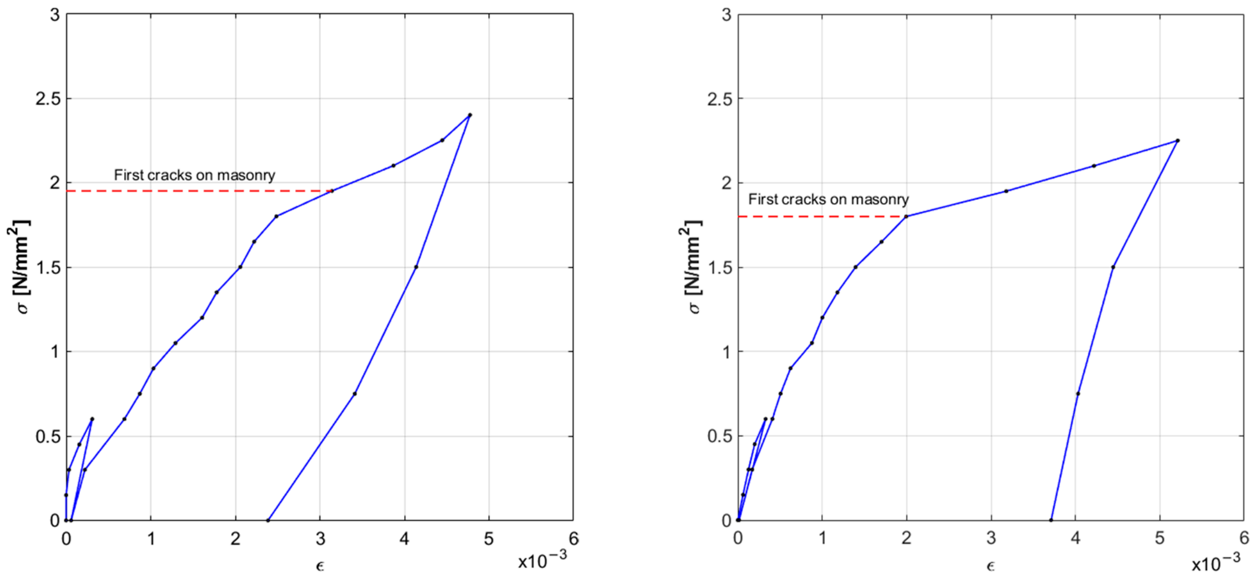


Figure 6. Section A-A, plan layout of the ground floor and west elevation of the secondary school "Machiavelli" (unit: m).

3.2. Evaluation of Masonry Properties

As already highlighted, the evaluation of masonry properties is crucial for one to perform a reliable structural assessment. With this aim, experimental data obtained from in-situ tests, which are necessarily limited to preserve the structural and architectural integrity of the construction, should be combined with information provided by a relevant database of test results on similar masonry type [8,39], supplemented by visual inspection methods [48] and engineering judgements. Referring to the case study presented here, the masonry quality and the most relevant masonry mechanical properties, i.e., elastic and shear modulus, compressive and shear strength, were obtained, duly combining reference literature and code information [52,53] with the results of in-situ single and double flat jack tests performed on two stone walls at the ground level.

The stress-deformation diagrams obtained by the two double flat jack tests are shown in Figure 7. Looking at the diagrams, it can be observed that the masonry compressive strength, which corresponds to the occurrence of first visible cracks on the masonry surface, is about 1.95 N/mm^2 in the first test, and 1.8 N/mm^2 in the second test. These values, combined with the qualitative information obtained by visual inspection methods, confirm that the masonry can be considered as quite homogenous.



(**a**) (**b**)

Figure 7. Stress deformation diagrams obtained by the two double flat jack tests: (**a**) DFJ1, (**b**) DFJ2.

From the diagrams has also been derived the value of the secant elastic modulus $E_{10\text{–}70}$, evaluated considering the stress range $(10\%\ \sigma_{\max},\ 70\%\ \sigma_{\max})$, which resulted as equal to 758 N/mm^2 in the first test, and 1065 N/mm^2 in the second test.

The experimental values of f_m and E so obtained were then used for the Bayesian updating of the mean values provided by the Guidelines for the application of Italian Building Code [53] for the given masonry type. The other relevant mechanical properties of masonry needed for the assessment, shear modulus, G, and shear strength, τ_k, were obtained by means of the experimental relationships derived in [40] from the analysis of a large dataset of experimental tests, in the form:

$$G = 0.15\,E;\ \tau_k = \frac{G}{2000} \tag{2}$$

The results of the experimental tests and the assumptions about masonry properties adopted in the assessment are summarized in Table 3, where they are also compared with the corresponding values suggested in [53] for stone masonry panels characterized by horizontal brick courses. For the updating of the mean values of material properties based on test results, the formula provided by the Guidelines for the Application of the Italian Building Code [53] is adopted:

$$\mu'' = \frac{n\overline{X} + \kappa\mu'}{n + \kappa} \tag{3}$$

where $\overline{X}$ is the average value of test results, n is the number of tests, μ' is the average value of the interval provided by the Guidelines for the same masonry type, and κ is a coefficient, taking into account the ratio between the variance of the investigated material properties determined by material tests and the variance adopted in the prior distribution [63]. Values of κ equal to 1.5 and 2 are recommended, in the same Guidelines [53], for the estimation by means of double flat jack tests of elastic modulus and compressive strength, respectively.

Table 3. Masonry mechanical properties obtained from the tests and adopted in the assessment: elastic modulus E, shear modulus G, masonry compressive strength f_m, masonry shear strength τ_k, specific weight w. Elastic and shear modulus are considered in cracked conditions.

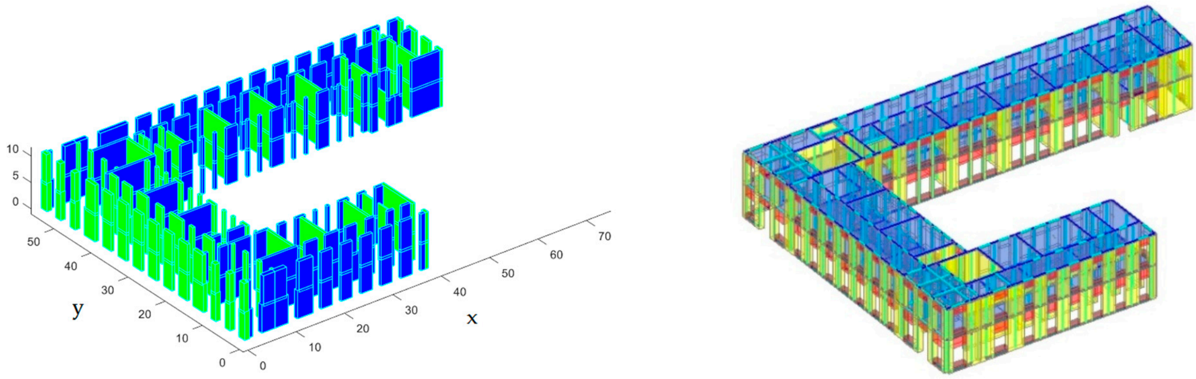
Case	E (MPa)	f_m (MPa)	G (MPa)	τ_k (MPa)	w (kN/m^3)
DFJ 1	758	1.95	-	-	-
DFJ 2	1065	1.8	-	-	-
Guidelines of application Italian Building Code [44] [1]	510–720	2–2.4	170–240	0.035–0.061	20
Adopted	784	2.32	118 [2]	0.059 [3]	20

[1] Correction factors for the presence of horizontal brick courses are considered. [2] Shear modulus G is set equal to 0.15 E [40]. [3] Shear strength τ_k is set equal to $G/2000$ [40].

3.3. Definition of the Structural Model and Results of the Assessment

As anticipated, the seismic performance of the investigated structure has been assessed by means of the original software E-PUSH [6,23] and the commercial software Aedes PCM. Considering the huge scattering of software output described before, we underline that Aedes-PCM has been selected, only for comparison purposes, among those software packages leading to results situated in the average band of the diagrams in Figure 4. Obviously, any validation or judgement on the soundness of the outputs provided by different commercial software packages is outside the scope of the present study.

The global 3D structural models used in the analyses are shown in Figure 8: more precisely Figure 8a displays the E-PUSH model, Figure 8b, the Aedes PCM one.



(**a**) (**b**)

Figure 8. 3D layout of the structural models developed by means of: (**a**) E-PUSH (dimensions in m), (**b**) Aedes PCM.

Owing the fact that there is no rigid diaphragm at the upper floor, preliminarily local verifications of individual masonry walls are carried out, and safety coefficients are determined, considering different failure criteria: diagonal shear, in plane and out of plane bending. In this case, a nil redistribution capacity is assumed.

Subsequently, a 2D non-linear static analysis is carried out for the alignments, each one represented by aligned and well-connected masonry panels, relying on the plastic redistribution capacity of the ensemble. Nine alignments are identified for the investigated building, as illustrated in Figure 9. Capacity curves provided by the two software packages are thus derived.

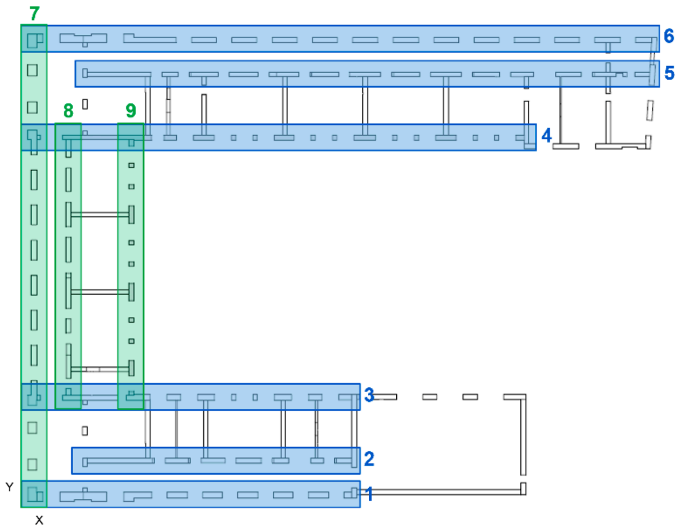
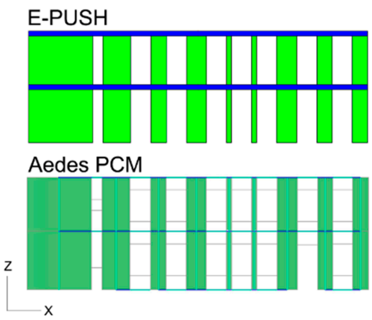
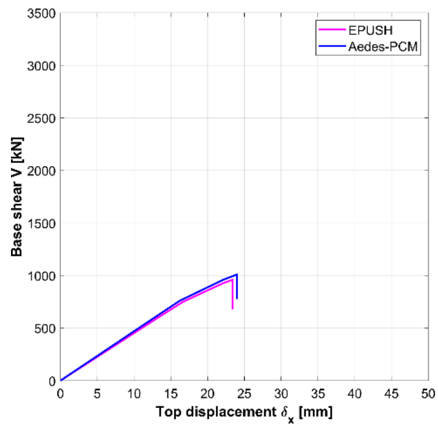


Figure 9. Definition of wall alignments in the investigated structure.

As an example, the capacity curves provided by the considered software programs for the wall alignments denoted as No. 3, 5, 7 and No. 9 in Figure 9 are compared in Figures 10–13, respectively, assuming the ductility failure criterion. In the left part of the figures, the models of the alignments are compared, while in the diagrams on the right, the base shear-displacement curves provided by Aedes PCM (blue lines) and by E-PUSH (magenta lines) are shown. Evidently, the force-displacement curves provided by the two software packages for these wall alignments are in good agreement, both in terms of base shear resistance and ultimate displacement. Even if not reported here, similar results have been obtained for the other alignments.

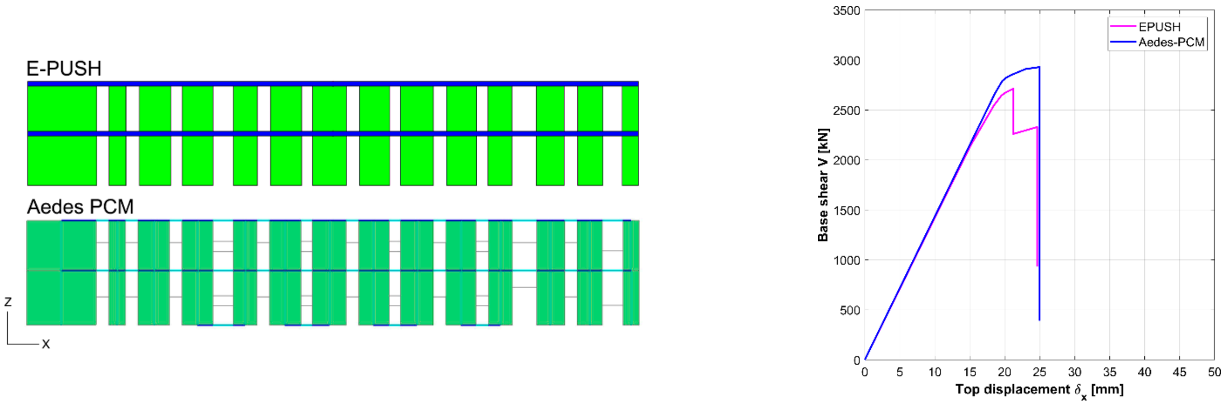


(**a**)



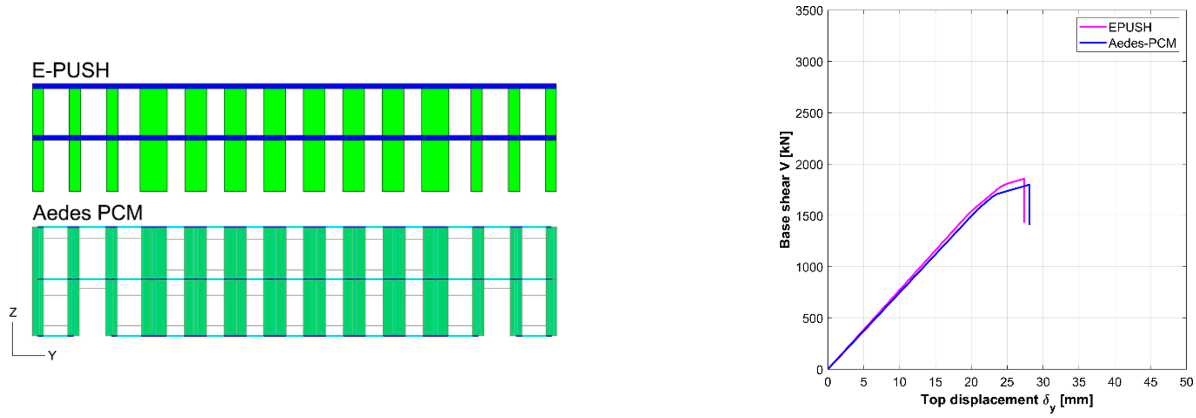
(**b**)

Figure 10. Comparison of capacity curves provided by Aedes PCM and E-PUSH for the wall alignment No. 3 in Figure 9: (**a**) model of the wall alignment, (**b**) capacity curves.



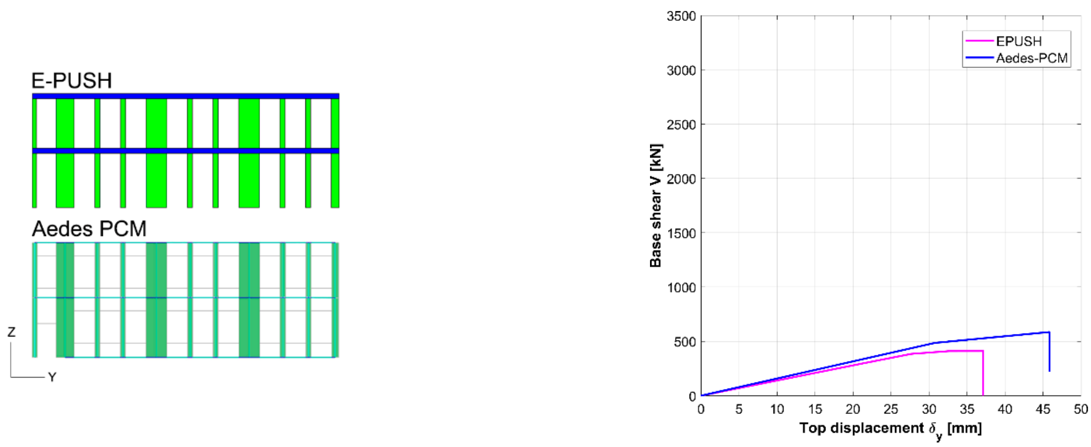
(**a**) (**b**)

Figure 11. Comparison of capacity curves provided by Aedes PCM and E-PUSH for the wall alignment No. 5 in Figure 9: (**a**) model of the wall alignment, (**b**) capacity curves.



(**a**) (**b**)

Figure 12. Comparison of capacity curves provided by Aedes PCM and E-PUSH for the wall alignment No. 7 in Figure 9: (**a**) model of the wall alignment, (**b**) capacity curves.

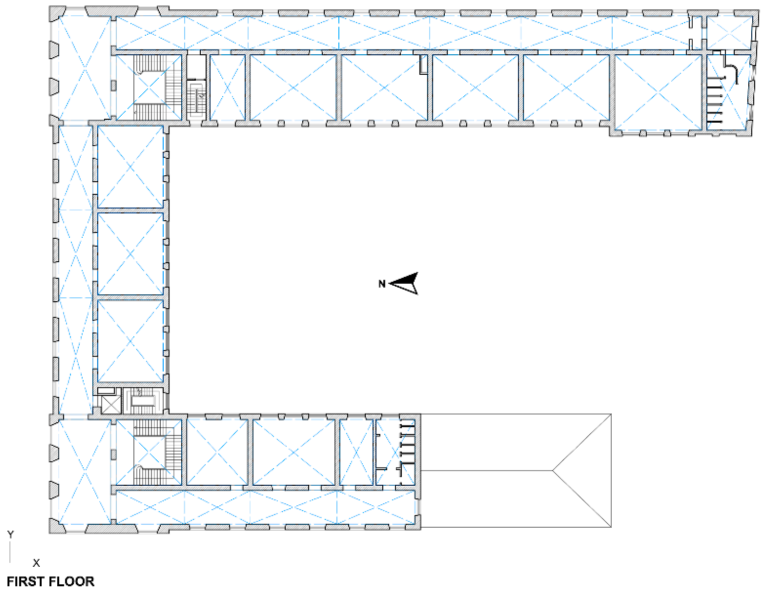


(**a**) (**b**)

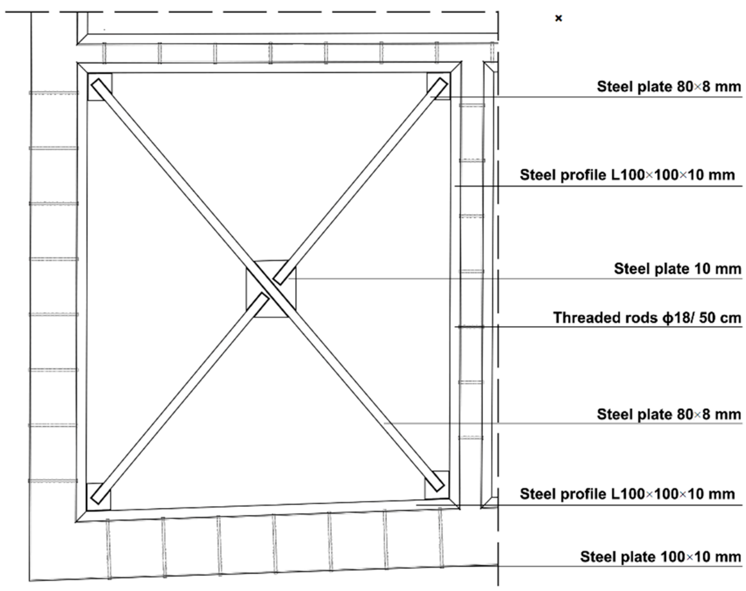
Figure 13. Comparison of capacity curves provided by Aedes PCM and E-PUSH for the wall alignment No. 9 in Figure 9: (**a**) model of the wall alignment, (**b**) capacity curves.

On the basis of the abovementioned results, the seismic risk index I_R given by E-PUSH, i.e., the minimum ratio between the maximum peak ground acceleration resisted by the alignments and the design peak ground acceleration in the actual condition, is $I_R = 0.34$ for seismic excitation acting in the y-direction (alignment No. 9 in Figure 9), and $I_R = 0.49$ for seismic excitation acting in the x-direction (alignment No. 4 in Figure 9). The seismic risk index of the building is thus $I_R = 0.34$, stressing the need of strengthening interventions.

Among possible solutions to improve the seismic performance of the building, even integrated to enhance mechanical and thermal performances [64], a simple and very effective intervention, often proposed, consists in increasing the rigidity of the floors, to ensure a 3D box behavior. In the present case, the introduction of a steel bracing diaphragm at the upper floor of the building not only allows one to consider all floors as rigid, so guaranteeing such a global behavior, but also contributes to preventing out of plane failure of the walls. A possible layout of the proposed steel bracing system is shown in Figure 14.



(**a**)



(**b**)

Figure 14. Layout of the proposed steel bracing system (**a**) and example of the structural scheme (**b**).

To assess the efficiency of such kinds of intervention, a global 3D non-linear static analysis is carried out again with E-PUSH and Aedes PCM. To discuss the influence of the failure criterion on the results, the ultimate displacement of the masonry walls is defined adopting, in turn, the ductility, or the drift, failure criterion. The results in terms of force-displacement capacity curves are compared in Figure 15a, referring to the seismic excitation directed along the x-axis, and in Figure 15b, referring to the seismic excitation directed along the y-axis.

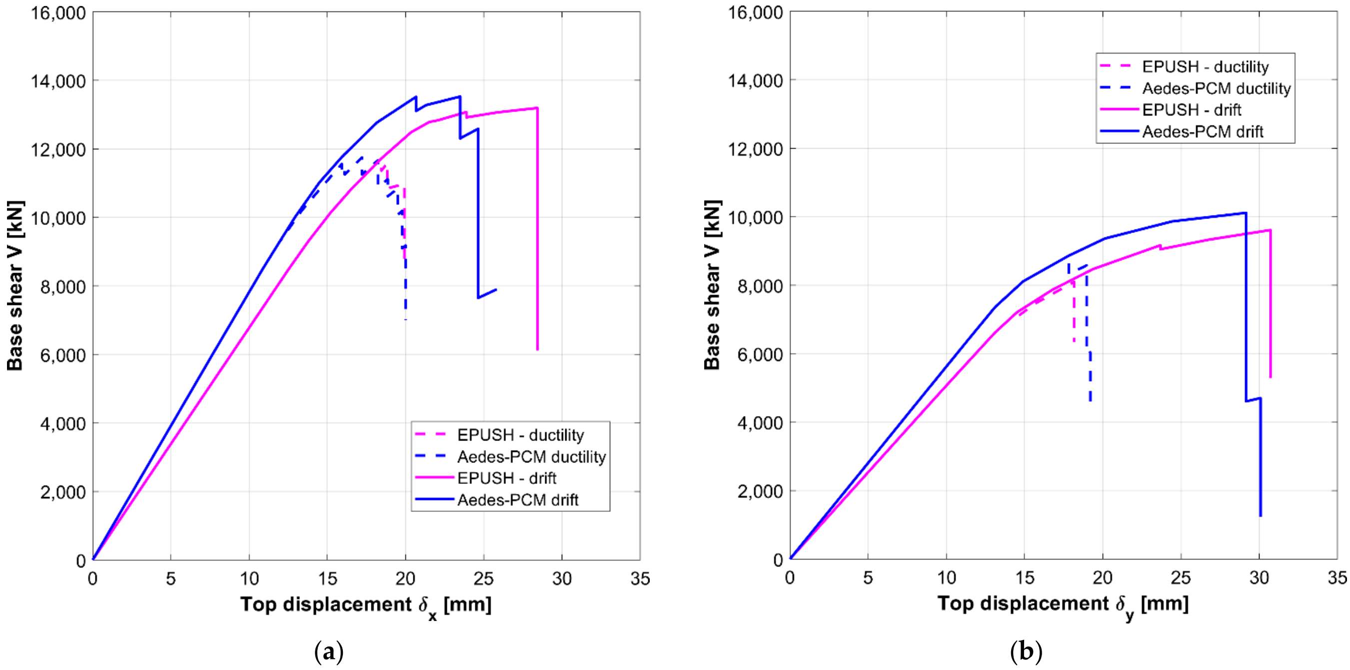


Figure 15. Comparison of capacity curves obtained by means of the E-PUSH software (in magenta) and Aedes PCM (**a**), x-direction (longitudinal), (**b**) y-direction (transversal).

In the diagrams, magenta lines represent E-PUSH results, while blue lines represent Aedes PCM results; the curves obtained adopting the drift failure criterion are drawn by solid lines, whereas dashed lines refer to curves computed using the ductility failure criterion. It must be once more remarked that, given the failure criterion, the force-displacement curves provided by E-PUSH and by Aedes PCM are in good agreement. On the contrary, but it is not surprising, the assumptions of different failure criteria significantly impact the results. In fact, the seismic risk index I_R provided by E-PUSH is:

- $I_R = 0.67$ for seismic excitation in the x-direction, $I_R = 0.52$ for seismic excitation in the y-direction, so that the seismic risk index of the building is $I_R = 0.52$, according to the ductility failure criterion;
- $I_R = 0.97$ for seismic excitation in the x-direction, $I_R = 0.91$ for seismic excitation in the y-direction, so that the seismic risk index of the building is $I_R = 0.91$, according to the drift failure criterion.

These values should be compared with that obtained from the 2D non-linear of the alignments, $I_R = 0.34$, so emphasizing the efficiency of the intervention.

The results of the pushover analyses refer to the post-intervention conditions and are reported here only aiming to show the implementation of the software under different modeling hypotheses, and to demonstrate its capabilities in comparison with commercial software on a complex building. Clearly, in consideration of the non-symmetrical building's plan, for a complete structural assessment, which is outside the scope of the paper, specific attention should also be focused on the torsional behavior of the structure; with this aim, the contribution of various mode shapes, evaluated by means of linear dynamic analysis, should be duly considered.

4. Discussion and Conclusions

In the paper, macro-elements methods for modelling masonry structures and assessing their seismic performance in the engineering practice are presented and discussed in comparison with an original program developed by the authors, called E-PUSH. The adoption of different methods highlighted a rather impressive dispersion of results for a simple benchmark structure [25], which can be even enhanced for real case studies considering the uncertainties about masonry mechanical properties [8,9].

The dispersion of the results and its dependance on the modelling assumptions may significantly impact the assessment of the seismic risk index I_R, with obvious consequences: this aspect impacts not only the need or the opportunity of strengthening interventions, but also, at a bigger scale, the planning and adoption of mitigation and incentive policies, which depend on the seismic risk index distribution of the built environment. Moreover, available programs for seismic analysis of masonry buildings are often so complicated, that results are extremely dependent on the modelling assumptions and on the skills of the user.

Even in comparison with other software packages, main advantages of the E-PUSH program, which is distributed open source, are: "robustness", user-friendliness, low computational time, clear modeling system, limited user skill requirements, ease interpretation of the results. Moreover, requiring low computational time, it results particularly efficient when sensitivity analyses and uncertainty evaluation are concerned [6].

In the paper, the outcomes of the E-PUSH program are critically discussed. In a first phase, a benchmark structure, recently investigated in [25], resorting to sever commercial packages, is assessed: the study demonstrates that capacity curves provided by E-PUSH satisfactorily agree with the ones reported in [25], belonging to the mean range of the other software ensemble. Subsequently, the use of the program is illustrated referring to a relevant real case study: the seismic assessment of a school masonry building, the "Machiavelli" school located in Florence (IT). In this case, the results of the seismic assessment are compared in terms of capacity curves and seismic risk indexes with those obtained with the Aedes PCM software by "expert" users. The results provided by the two programs are very close, confirming that the results obtained with E-PUSH are in line with the average results provided by other software, encouraging further developments.

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