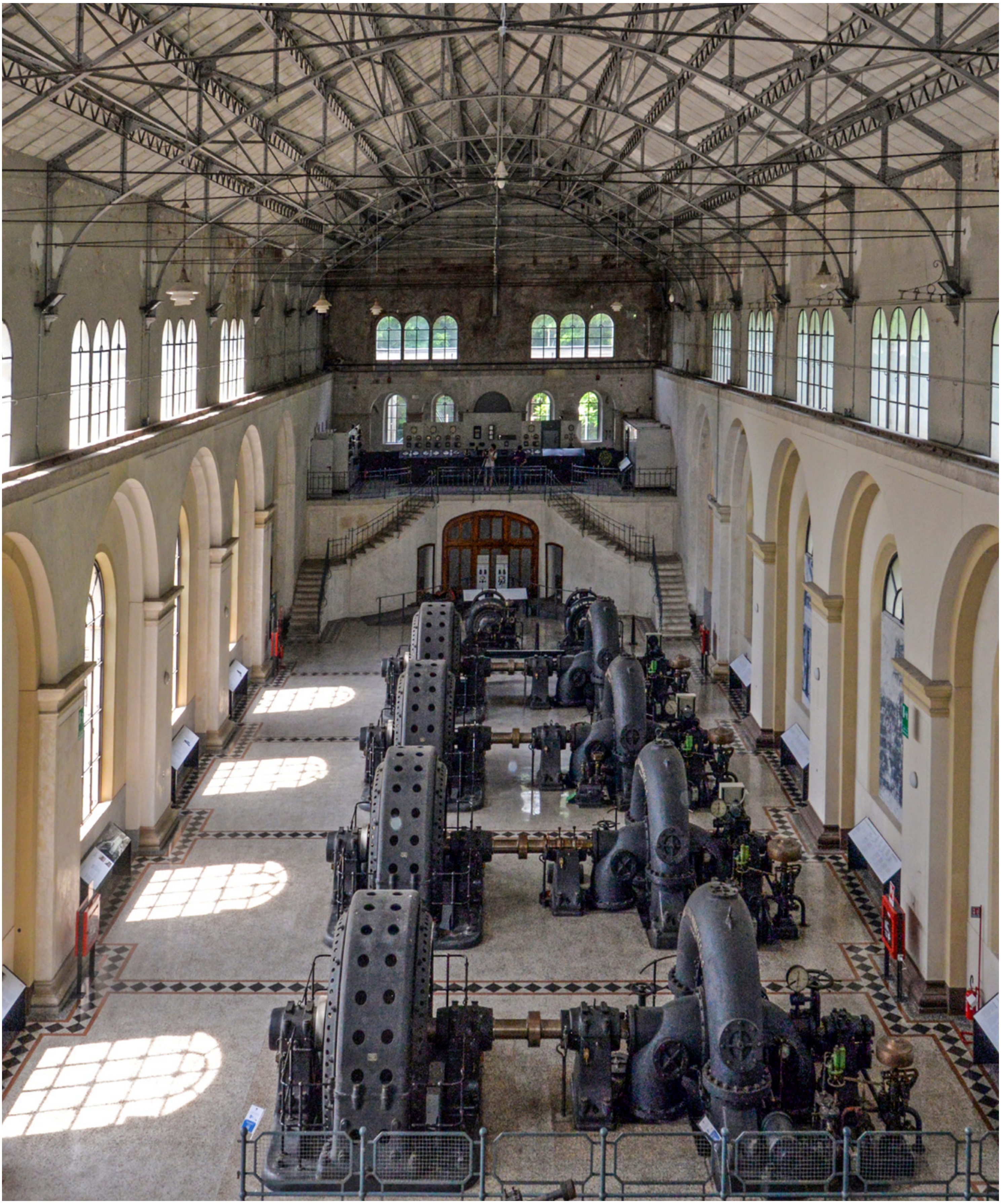

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A NOVEL SEISMIC VULNERABILITY ASSESSMENT OF MASONRY FAÇADES: FRAMING AND VALIDATION ON CALDAROLA CASE STUDY AFTER 2016 CENTRAL ITALY EARTHQUAKE

Letizia Bernabei, Generoso Vaiano, Federica Rosso, Giovanni Mochi

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Highlights

A novel seismic vulnerability assessment of historic masonry facades is described.
Out-of-plane damages that interfere with the usability of the rescue roads and the safety of evacuees are targeted.
A rapid application also by non-expert technicians for macro-scale analysis of historical built environment for developing emergency planning and mitigation strategies is allowed.

Abstract

An important portion of the historical built environment, which is characterized by un-reinforced masonry, is particularly vulnerable to collapse in case of earthquakes, as demonstrated by recent events. Strategies to target the facades of the most vulnerable buildings need to be tailored for retrofitting and emergency planning. In this research, a novel expeditious vulnerability assessment method, particularly suitable for historical masonry aggregates, is proposed. The method allows assessing the vulnerability index based on information available from external surveys on the building, thus facilitating and speeding up the investigation. If other more precise information (e.g., curbs and tie rods effectiveness) is available, the vulnerability estimation can be improved. The method focuses on out-of-plane mechanisms of the facade, which cause debris to fall on adjacent streets, impeding emergency response. The expeditious method is tailored starting from analytical methods applied on a large sample of historical buildings hit by earthquakes, and validated by means of comparison with kinematic analysis and observed damage state on a relevant case study, Caldarola (Macerata, Central Italy, which was struck by the earthquake in 2016). Results show a good agreement between the proposed method, the kinematic analysis, and the observed damage state of the considered case study, with 75% and 87.5% correspondence, and the method is especially precise for evaluating highly damaged facades.

Keywords

Seismic vulnerability of aggregates, Masonry façade assessment method, 2016 Central Italy Earthquake, Out-of-plane damages, Expeditious vulnerability assessment method.

Letizia Bernabei*
Dipartimento di Ingegneria Civile ed Ambientale, Università degli Studi di Perugia, Perugia (Italy)

Generoso Vaiano
Dipartimento di Ingegneria Civile ed Ambientale, Università degli Studi di Perugia, Perugia (Italy)

Federica Rosso
Dipartimento di Ingegneria Civile ed Ambientale, Università degli Studi di Perugia, Perugia (Italy)

Giovanni Mochi
Dipartimento di Ingegneria Civile ed Ambientale, Università degli Studi di Perugia, Perugia (Italy)

** Corresponding author:*
e-mail: letizia.bernabei@unipg.it

1. INTRODUCTION

The dramatic impacts of the 2016 Central Italy earthquake underline the importance of adopting a seismic risk assessment method from a holistic and interdisciplinary perspective that integrates both macro-scale (i.e., territorial, urban) and micro-scale (i.e., buildings) aspects to ensure the safety of urban settlement and promote a resilient response to disasters [1, 2].

Damages of masonry buildings determine the highest proportion of impacts in earthquakes considering both casualties and losses in architectural heritage [3], especially in Historical Built Environment (HBE), which is a vulnerable building stock as buildings are characterised by Un-Reinforced Masonry (URM) structures that are more prone to collapse [4]. Moreover, such buildings are generally part of an aggregate of buildings with structural deficiencies due to the historical evolution and transformation processes, unmanaged stratifications, and obsolescence that affect the overall behaviour to seismic ground shaking [5–7]. These constructive conditions prevent the box-like behaviour due to the lack of connection between the façade itself and the orthogonal walls, thus triggering Out-Of-Plane (OOP) collapse mechanisms [8, 9].

The high vulnerability of the HBE building stock affects the safety of individuals both directly, by the total collapse of buildings, and indirectly during the evacuation process, due to the streets' blockage by the debris of overturning façade [2, 10]. Particularly, the interference of the built fronts with open spaces (e.g., streets and squares) strongly influences the whole emergency management in the immediate post-earthquake framework: streets link parts of the urban fabric, ensuring pedestrian evacuation and rescue operations, and squares act as outdoor temporary gathering areas for evacuees [11, 12].

Therefore, the seismic vulnerability assessment has great relevance to the damage scenarios prediction for the development of effective risk reduction and mitigation strategies aimed at strengthening interventions of the building stock to reduce failures and at an adequate emergency planning [2, 4, 13].

A significant number of vulnerability assessment methodologies are already available in the literature [14, 15]. Macro-scale evaluations are generally carried out by means of empirical methods based on observational damage data from past earthquakes. These empirical methods are scoring methods (i.e., Vulnerability Index Method-VIM) composed of parameters based on the geometric and construction characteristics of the considered buildings. Scoring methods provide a rapid analysis of the global behaviour of Structural Units (SU) of buildings' aggregate [9, 16]. They are commonly combined with the macro-seismic methods for the prediction of damage scenarios through the mathematical function of the mean damage (μD) [17]. Although these methods are successfully adopted for the analysis of historical urban centres, the accuracy of the results relies on expert judgment, i.e., the knowledge and expertise of surveyors in assigning the scores and the parameters. Such a procedure may seem prone to errors, especially in the absence of verified information about the structural characteristics when on-site surveys from outside are not sufficient. Indeed, such methods cannot derogate from the specific knowledge of the SUs, which depends on the availability of data about the construction characteristics (e.g., the roof type, masonry quality, degree of connection to orthogonal walls, and horizontal structures) provided by on-site survey campaigns and wall inspections.

Analytical methods represent an attempt to overcome the uncertainties associated with the empirical approach, thank to the structural-engineering approach based on time-consuming mechanical models, thus being more appropriate for the micro-scale analysis. Among these, some analytical methods [18, 19] provide vulnerability indices based on kinematics analysis, which identifies collapse load factor multipliers of a given configuration of macro-elements and loads. Even if analytical methods directly identify the occurrence of possible OOP failure modes, they rely on a considerable number of parameters referring to detailed characteristics of structural elements, which require invasive inspections, as this information is generally not exposed.

With the aim of enabling rapid vulnerability assessment at the urban scale, the current paper proposes the definition of a novel seismic vulnerability assessment method for the façades of historic masonry aggregates for strictly identifying those buildings' fronts that are more prone to OOP mechanisms through a simplified procedure. The proposed method, rooted in the existing empirical methods, aims to allow for higher reliability, comparable to that of mechanical model-based approaches (analytical methods). Indeed, the results of the method are compared to those of kinematic analyses (analytical methods). Contrary to kinematic analyses, the method relies on a few geometric and construction parameters influencing the OOP behaviour of the macro-elements and is experimentally calibrated on a diffuse set of post-seismic damages observed after the 2016 Central Italy earthquake in a high number of examples. Moreover, it is then applied to the relevant case study of a historical building aggregate located in Caldarola (Macerata, Central Italy) to validate the reliability of the assessment procedure in comparison with the real damage of the specific case study. One of the main objectives of the proposed method is having an expeditious nature, to enable its rapid application for vulnerability assessments, feasible at the entire urban scale even by non-expert technicians, as the information required can be detected by external inspections or even dimensioned facades images.

2. METHODOLOGY

The methodological framework that is adopted in this study is illustrated in detail in the following subsection and is depicted in Figure 1, which describes the stages of the definition of the novel expeditious vulnerability assessment method proposed in this study.

2.1. DEFINITION OF THE NOVEL EXPEDITIOUS METHOD

The novel seismic vulnerability assessment for historic masonry facades is tailored by first setting the initial assumptions for the theoretical framing. Then the geometry and construction expeditious parameters are set, and the weights to balance their importance in the method are defined. Finally, the damage scale for OOP collapse mechanisms is illustrated. Once the assessment method, which is described in greater detail in the subsections below, is set, the relevant case study is presented in section 0, and the method is applied and discussed in the results section. The method has undergone two validations, one analytical, where the results were compared with those of the kinematic analyses (section 4.2), and one empirical, where the results were compared with the direct observations of damages (section 4.3).

2.1.1. ASSUMPTIONS FOR VULNERABILITY ASSESSMENT

The proposed method allows estimating the seismic vulnerability of the facade of a masonry building starting from its geometric characteristics. Indeed, the research aims at tailoring an expeditious method, calibrated based

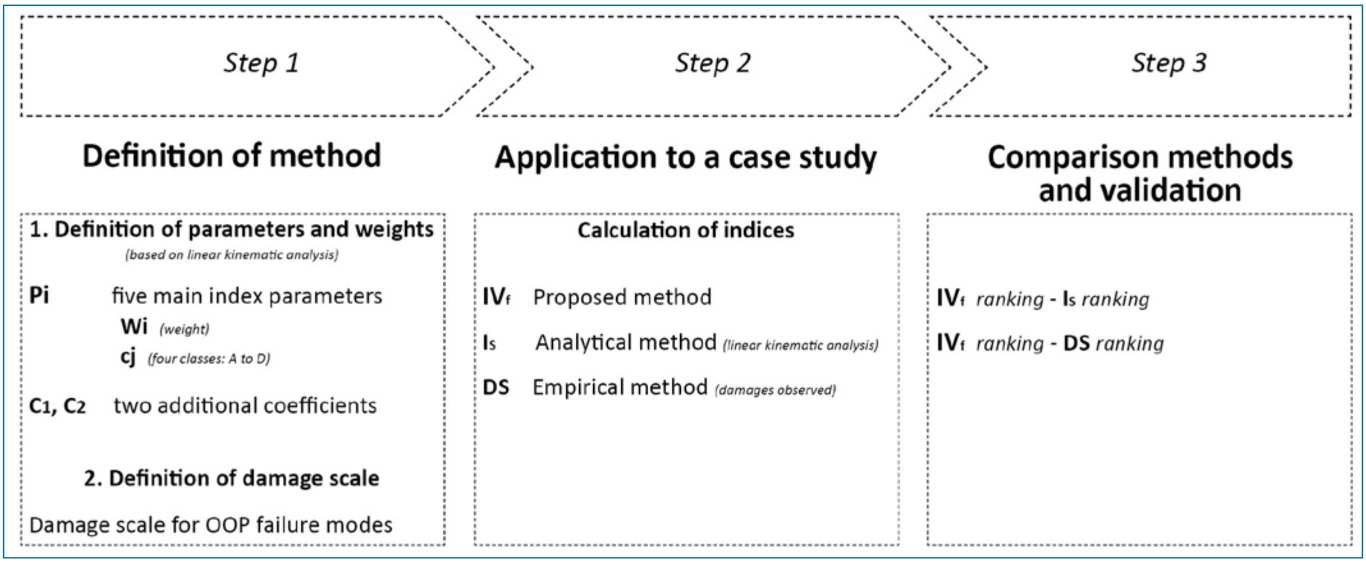

Fig. 1. *Diagram flow of the novel expeditious vulnerability assessment method of masonry façades.*

on linear kinematic analysis, as defined by the Italian Codes [20, 21].

In so doing, two assumptions are made, (i) the connections between orthogonal walls and (ii) between the wall facade and floors are poor. With the above-mentioned assumptions, it is, therefore, possible to hypothesize the activation of one local collapse mechanism OOP of the wall, i.e., façade overturning. This mechanism affects the walls of one or more levels of the building, involving the entire façade or parts of it, as well as the full thickness of the walls or part of it. However, due to the hypotheses underlying the kinematic analysis – namely the presence of blocks referred to as macro-elements and their consideration as rigid bodies – the examination of the local collapse mechanisms is applicable when the mechanical characteristics of the masonry do not allow the block disintegration.

In this study, specific consideration is given to the horizontal multiplier of the loads acting on the masonry façade, which leads to the activation of the overturning kinematics of the entire façade. Such evaluation is conducted based on the knowledge of the characteristics of the building. The multiplier is the ratio between the horizontal forces and vertical weights and is evaluated as follows [22] (Eq. 1):

$$\alpha = \frac{W\cdot\frac{s}{2}+F_V\cdot d_V+P_S\cdot d+T\cdot h_t-F_H\cdot h_V-P_H\cdot h_p}{W\cdot y_G+F_V\cdot h_V+P_S\cdot h_p} \qquad (1)$$

The α coefficient is dimensionless, as both the denominator and the numerator are forces multiplied by length. Therefore, the measurement units for both force and length can be chosen arbitrarily. For example, for the forces [F], the following units are generally used: kg, N, daN, and kN. Instead, for lengths [L], the following units are generally used: m, cm, and mm.

The acronyms are as follows (Fig. 2) [23]:

- W_i *[F]*, weight of the i^{th} macro-element;
- s_i [*L*], wall thickness;
- F_{Vi} *[F]*, thrust vertical component of arches or vaults in correspondence of the i^{th} floor;
- d_{Vi} [*L*], horizontal distance of arches or vaults thrust at the i^{th} floor.
- P_{Si} *[F]*, floor weight of the i^{th} level;
- d_i [*L*], horizontal distance of generic vertical load transmitted on the macro-element;
- T_i *[F]*, action of metallic tie rods at the i^{th} floor;
- h_{ti} [*L*], vertical distance from the application point of the action transmitted by the floor and/or the metallic tie rod of i^{th} floor to the hinge of the i^{th} macro-element;
- F_{Hi} *[F]*, thrust horizontal component of arches or vaults in correspondence of the i^{th} floor;
- h_{Vi} [*L*], vertical distance of arches or vaults thrust at the i^{th} floor;
- P_{Hi} *[F]*, static thrust transmitted by the top floor;
- h_{pi} [*L*], vertical distance from the action application point transmitted by the floor of the i^{th} level to the base hinge;
- y_{Gi} [*L*], vertical distance between the centroid of the ith wall and the wall base.

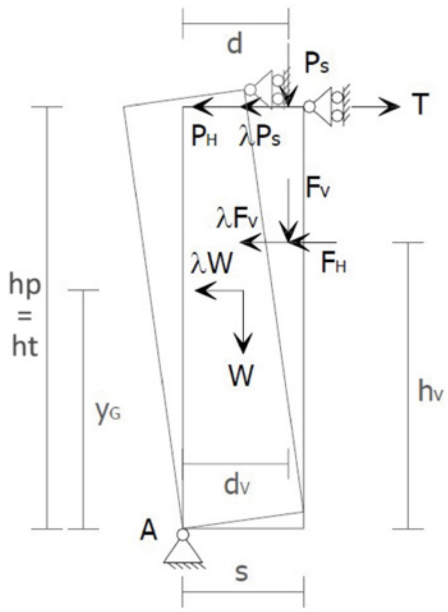

Fig. 2. *Graphical representation of the forces in Equation 1.*

2.1.2. MAIN INDEX PARAMETERS

The vulnerability index is calculated starting from 5 parameters (*Pi*) to which a weight (W_i) is assigned. Each parameter is divided into 4 classes (A to D, from the least to the most vulnerable). To each class, a coefficient (c_j), which is the weight percentage, is assigned.

The assessment of c_j and W_i for each parameter is made through parametric analyses (according to section 2.1). In particular, to define the coefficient c_j of the four classes, the following procedure is adopted. Firstly, the mean value related to the geometric and mechanic characteristics of the parameters is identified considering the average condition of a "standard" building. The building has the following characteristics: three floors, 3.2 meters of inter-storey height, masonry specific weight of 18 kN/m^3, facade width (or distance between two orthogonal walls) of 7 meters, wood floor with main beams orthogonal to the façade, 60 cm thick walls, façade slenderness of 11.5, slightly-pushing roof, 15% of openings to the total façade surface. Then, the horizontal multiplier of the loads (Eq. 1) is calculated by varying from the standard to the best (i.e., corresponding to A-class) and the worst (i.e., corresponding to D class) condition the features related to one of the five parameters, while the other four remain fixed. Then, the percentage variation of the horizontal multiplier α between classes A, B, and C are evaluated compared to class D, to which the maximum weight coefficient of 1 is assigned, and the value of the coefficient is consequently determined according to such variation. Instead, the weight W_i of a parameter is determined by observing the percentage difference of the multiplier between the lowest value, hence the best class (Class A), and the highest, hence the worst one (Class D), that are then normalized from 1 to 100.

Finally, the vulnerability index (IV_f) is calculated as the sum of the weights of the parameters multiplied by the relative class coefficients, as follow (Eq. 2):

$$IV_f = \sum W_i \cdot c_j \quad (2)$$

The investigated parameters refer to the geometric features of the masonry façade, which can be easily detected by both external in-situ survey (e.g., photogrammetry tools) and remote access techniques (e.g., google street view and post-processed images of the facade); thus, the goal is to provide a seismic vulnerability index through an objective judgment. Obviously, the availability of accurate investigations such as in-situ tests, historical-critical analyses, and detailed surveys can better direct the choice of parameter classes. When this information is not available, we assume the typical average characteristics of the location. The five parameters are reported below.

- *P1: Floors number*

The number of floors has a great influence in assessing masonry facade vulnerability. Indeed, this is the parameter with the greatest weight. In particular, as the number of floors increases, the horizontal multiplier of the loads decreases because, according to equation 1, the denominator increases faster than the numerator. Floors that are simply leaning on the walls are considered. Therefore, facade walls are not capable of counteracting the wall overturning against horizontal thrusts. The study of this parameter considers ordinary masonry buildings that do not exceed 4-5 floors.

The 4 classes and their coefficients are reported below (Tab. 1):

Classes	Description	c_1
Class A	One floor buildings	0.18
Class B	Two floor buildings	0.35
Class C	Three floor buildings	0.51
Class D	Four or more floor buildings	1

Tab. 1. *P1 class coefficients.*

- *P2: Specific weight*

The specific weight is the parameter that has the least influence in seismic vulnerability index assessment. As this parameter increases, the horizontal multiplier of the loads increases too.

In the case of the absence of accurate on-site test procedures, the facade masonry types have been assumed according to the indications provided by Table C 8.5.I of the Italian code "Instruction for application of the NTC18" [21]. The selection of the masonry type can be made through direct observation of the facade, using a reduction factor defined confidence factor (FC) equal to 1.35, which corresponds to a level of knowledge LC1 (limited knowledge) [20, 21]. Table 2 illustrates the considered masonry types.

Masonry Type	Specific weight [kN/m^3]
Disorganized irregular stone	19
Barely cut stone	20
Roughly cut stone	21
Irregular soft stone	13-16
Stone square blocks	22
Bricks and lime mortar	18
Hollow bricks with cementitious mortar	15

Tab. 2. *Masonry types according to [21].*

The 4 classes and their coefficients are (Tab. 3):

Classes	Specific weight [kN/m³]	c_2
Class A	greater than 23	0.71
Class B	from 20 to 22	0.77
Class C	from 17 to 19	0.83
Class D	from 11 to 16	1

Tab. 3. *P2 class coefficients.*

- *P3: Slenderness*

Slenderness is the ratio between the total height and the thickness of the facade. Usually, thickness is reduced on higher floors. In this case, it is possible to refer to its average value or the minimum one. The latter case is the most unfavorable one, therefore to be preferred.

This parameter indirectly considers the number of floors, and for this reason, has a strong influence on the assessment of the vulnerability index. The 4 classes and their coefficients are (Tab. 4):

Classes	Total slenderness	c_3
Class A	less than 9	0.19
Class B	from 10 to 12	0.32
Class C	from 13 to 17	0.51
Class D	greater than 18	1

Tab. 4. *P3 class coefficients.*

- *P4: Roof type*

Roof reaction is defined as "pushing" when the thrust is transferred to the top of the façade wall. A pushing roof has extremely negative effects on the seismic response of the building because:

- the horizontal action due to vertical loads are added to the horizontal seismic action;

- the vertical seismic component increases the horizontal thrust.

The problem of pushing roofs mainly concerns wooden ones. With the exception of particular cases, reinforced concrete (RC) roofs are no-pushing structures. In the table below, some roof types are shown.

The 4 classes and their coefficients are shown in Table 6:

Classes	Roof types	c_4
Class A	Flat roof	0
Class B	No-pushing roof	0.21
Class C	Slightly-pushing roof	0.41
Class D	Pushing roof	1

Tab. 6. *P4 class coefficients.*

- *P5: Openings*

Facade overturning mechanisms activation depends on the distance between the wall centroid and the cylindrical hinge (located at the base of the wall). Generally, the presence of openings in the wall modifies the position of the centroid involving a variation in the evaluation of the overturning actions. The presence of an opening is evaluated as the ratio between the total surface of the façade and that of the openings one. Its influence is lower than that of parameters P1, P3, and P4, but greater than P2. Table 7 shows the 4 classes and their coefficients.

Classes	Openings [%]	c_5
Class A	less than 4% (no windows)	0.54
Class B	from 5% to 10%	0.77
Class C	from 11% to 18%	0.88
Class D	greater than 18%	1

Tab. 7. *P5 class coefficients.*

A summary of all the parameters with the relative weights and classes is proposed in Table 8.

Non-pushing roof	Slightly-pushing roof	Pushing roof
Trusses roof	Gable roof having top beam	Gable roof
Roof beams parallel to the facade	Gable roof with central wall up to the top	Hip roof

Tab. 5. *Roof types and their behavior.*

	Parameters P_i	Class c_j				Weight W_i
P1	Floors number	*1 floor*	*2 floors*	*3 floors*	*4 or more floors*	29.69
		0.25	0.36	0.52	1	
P2	Specific weight [kN/m³]	*greater than 23*	*from 20 to 22*	*from 17 to 19*	*from 11 to 16*	8.59
		0.76	0.80	0.83	1	
P3	Slenderness	*less than 9*	*from 10 to 12*	*from 13 to 17*	*greater than 18*	30.08
		0.23	0.30	0.49	1	
P4	Roof type	*flat roof*	*non-pushing roof*	*slightly-pushing roof*	*pushing roof*	22.66
		0	0.42	0.59	1	
P5	Openings	*less than 4%*	*from 5% to 10%*	*from 11% to 18%*	*greater than 18%*	8.98
		0.54	0.77	0.88	1	

Tab. 8. *Parameter list of the proposed method.*

2.1.3. ADDITIONAL COEFFICIENTS

Two additional coefficients are further set to consider qualitative aspects that influence the OOP behavior and are described below. They could aid in reaching a more accurate vulnerability index. Therefore, the final vulnerability index (IV_f) is calculated as follows:

$$IV_f = \left(\sum W_i \cdot c_j\right) \times C_1 \times C_2 \quad (3)$$

Where:

- *C1: State of conservation*

C1 increases the vulnerability index and assumes three values according to the crack pattern (Tab. 9). However, the choice of the score relies on the expertise of the surveyor in assessing the severity of the cracks.

- *C_2: Anti-seismic devices*

C2 coefficient reduces the IVf since the presence of anti-seismic devices hinders or prevents OOP failures. The presence of ties is detected by an external survey, but there is no information on their structural effectiveness. Thus, such coefficient considers the possible poor performance of ties (case 1 – Tab. 9) if these are not well distributed within the façade, under-dimensioned or deteriorated [8] (e.g., stainless-steel ties with endplate were applied in the 20th century, while the wrought iron cross ties are historical reinforcement, usually less effective). Otherwise, different weights (case 2 – Tab. 9) are set if the effectiveness of ties is demonstrated according to current regulations.

Additional qualitative coefficients			
C_1	**State of conservation**	absence of any cracks	1
		few OOP cracks	1.05
		several OOP cracks	1.10
C_2	**Anti-seismic devices** case 1	present on all storeys	0.55
		present on the top **and** *n-1* storeys	0.65
		present on the top **or** *n-1,2* storeys	0.85
C_2	**Anti-seismic devices** case2	present on all storeys	0.26
		present on the top **and** *n-1* storeys	0.31
		present on the top **or** *n-1,2* storeys	0.43

Tab. 9. *Additional qualitative coefficients.*

2.1.4. DAMAGE SCALE FOR OOP COLLAPSE MECHANISMS

The existing European macro-seismic damage scale EMS-98 [24] provides a description of failure modes and crack patterns with respect to the global behaviour of a masonry building. As such, it does not allow a direct correlation with the damages suffered by the single façade. According to the proposed method, the definition of a specific damage scale referring exclusively to local failure mode is necessary. The method includes OOP failures, the overturning of the total façade, or a portion of the façade.

Therefore, six Damage States (DS) have been identified, and the description of the related typical crack pattern is addressed as follow (Fig. 3):

- Collapse (C): fall of at least 75% of the overturning façade.
- Near Collapse (NC): fall up to 25% of the overturning façade; detachment and dislocation of macro-elements; irreversible OOP deformations. Although the proposed

methodology has not been computed on specific partial failure mechanisms, such as overturning of the upper horizontal spandrel, a gable, or arch failure mode (caused by the hammering of the roof on the façade), typical crack patterns related to these are also included.

- Severe Damage (SD): several vertical cracks on both the sides of macro-elements, related to OOP failure modes.
- Medium Damage (MD): few vertical cracks related to OOP mechanisms; detachment and loss of plaster or decorative elements.
- Light Damage (LD): no cracks due to OOP mechanisms.
- No Damage (ND): absence of any cracks.

The proposed damage classification is also set according to the limit states adopted by the Italian Building Code [20], which provides performance targets for building design and are directly related to the extent of damages. In the case of Damage Limit State (DLS), buildings can be affected by negligible to slight damages that do not affect the structural resistance, and in this case, we refer to ND and LD. In the case of Life Safety Limit State (LLS) instead, structural elements have light to moderate cracks, and small parts of plaster or stones fall (corresponding to MD and SD). In the case of Collapse Limit State (CLS), serious damages to load-bearing walls lead to partial or total collapse, as in NC and C.

2.2. COMPARISON METHODS AND VALIDATION

The proposed method is validated by comparing the results obtained with those of the analytical method, i.e., by means of linear kinematic analysis, and with those of the empirical method, i.e., by means of direct comparison of observed damages after the 2016 earthquake according to the OOP damage scale (section 2.1.4).

The analytical method allows the local collapse mechanisms verification when the macro-element is ground-connected; the following expression is adopted (Eq. 4) [20]:

$$\alpha_0^* \geq \frac{a_g(P_{VR})*S}{q} = \alpha_{0,min}^* \tag{4}$$

Where:

- α_0^*, is the spectral acceleration;
- *ag (PVR)*, is the seismic ground acceleration function of PVR which is the probability of exceeding a LS (limit state) and the building reference life (VR).
- *q*, is the behavior factor, which is equal to 2;
- *S*, is the product of the stratigraphic amplification coefficient and topographic one.

The safety index (I_S) [-] is the ratio between demand and capacity in terms of spectral acceleration, as follows (Eq. 5):

$$I_s = \frac{\alpha_0^*}{\alpha_{0,min}^*} \tag{5}$$

If $I_S \geq 1$, the safety check is satisfied, while if If $I_S < 1$, seismic upgrading is required.

The kinematic analyses require highly detailed information on the construction features according to the confidence levels (FC=1.35, 1.20, 1.00) and increasing knowledge level (LC1, LC2, or LC3) and adopting the

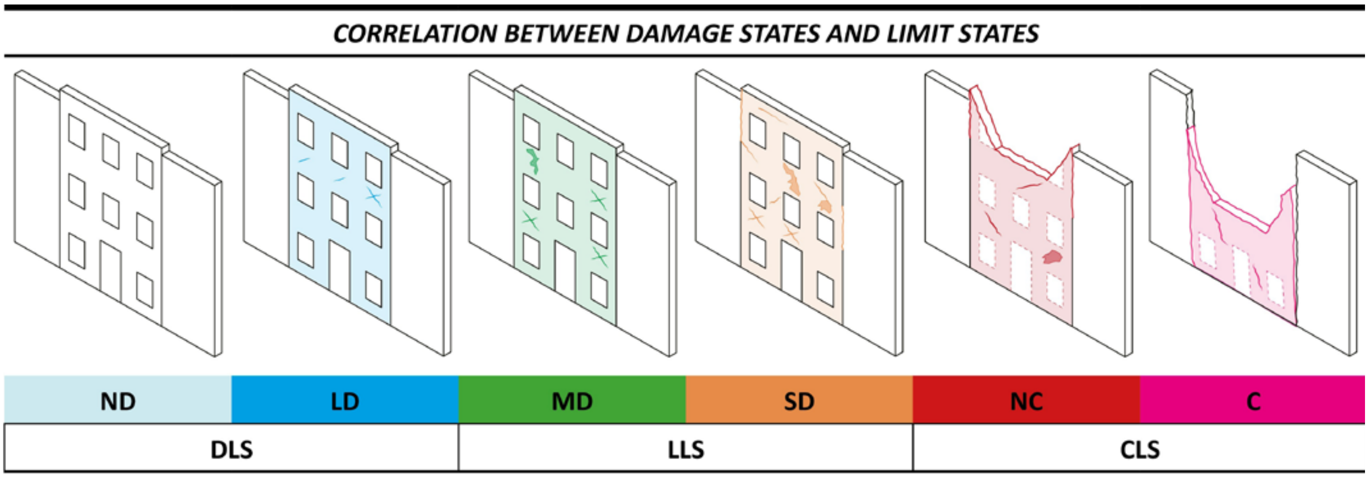

Fig. 3. *Damage scale for OOP failure modes and correlation with the limit state of Italian building code [20].*

characteristics of the site, as indicated by NTC2018 [20]. Moreover, it is necessary to insert all the information about anti-seismic devices. As mentioned above, a qualitative assessment of their effectiveness can be based on the period of their introduction into the building.

The empirical method instead consists in the validation of the results of the vulnerability assessment by comparing the damage assessment based on the OOP failures (section 2.1.4) due to occurred earthquakes. The comparison is carried out by ranking vulnerability index values and by ranking the severity of damage states suffered by the SUs.

The results of both the comparisons and the subsequent validation are presented in the results section.

3. CASE STUDY

The historical centre of Caldarola, a medieval small town in the province of Macerata (Central Italy), is well-known for the historical significance of its architecture and for the surrounding natural scenery. Although it was repeatedly affected by earthquakes along the centuries (e.g., in 1997 and in 1936 as the epicentre of a VI intensity event), the entire urban system has proved inefficient in responding to the shock of the 30 October 2016 (Mw 6.5), which led it to the isolation of the town because the of widespread building collapse along the mobility paths [1]. There were, fortunately, no injuries, as the centre had already been fully evacuated following the seismic swarm that began on 24 August 2016. The area affected by the aftershocks is approximately 40 km far from the epicentre, and the estimated ground motion severity was around the VII-VIII grade of Macro-seismic Intensity and 0.16 PGA [g] [25]. The whole historical centre was indeed closed for two years due to investigations on damaged buildings since 90% of the buildings became unusable.

The most critical condition occurred at the urban block located at the entrance of the historical centre. The only access road, “Via Roma”, was blocked by debris due to the overturning façade of three SUs belonging to aggregate n. 1 and aggregate n. 2 (Fig. 4).

Previous studies of expeditious vulnerability assessment of both aggregates have provided results consistent with the level of real damage suffered [1]. However, such analyses are focused on the global response of the aggregates and fail to highlight the probability of typical OOP failure modes occurring. The current study is focused on the aggregate n. 2 since it was affected by OOP failures of the façades. The analysis was carried out by an external in-situ survey in 2019 and with the support of all existing buildings’ documentation (gathered until 2016) for the application of the analytical method (section 4.2). The aggregate is a historic construction from the 17th century, characterised by URM buildings. The facades have a poor connection with orthogonal wall and horizontal structures since the aggregate has been subject to transformation processes over time. The load-bearing walls consist of two leaves of roughly irregular stone blocks randomly mixed with bricks poorly connected (ir-

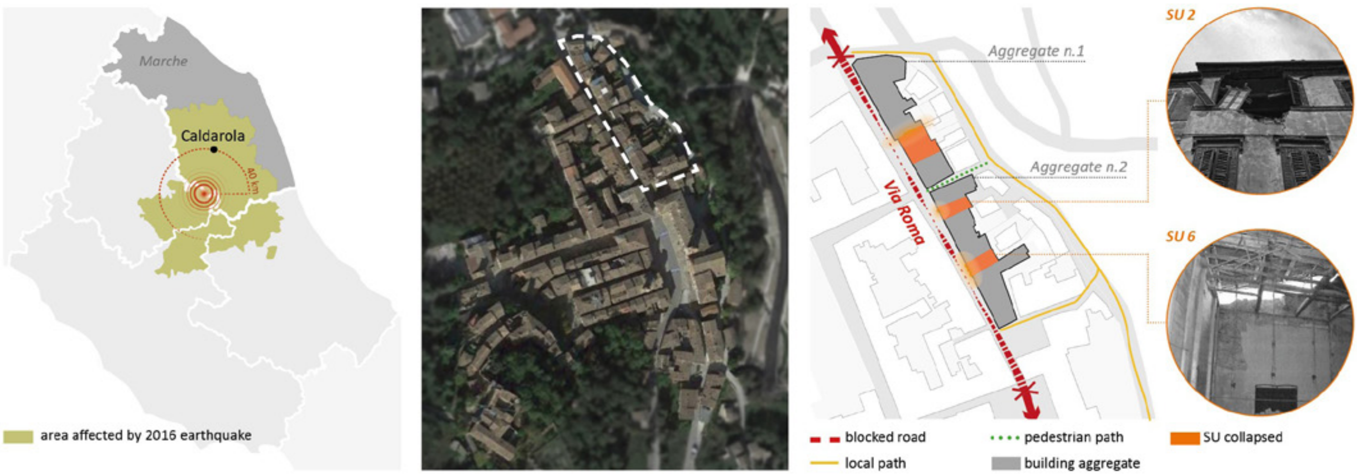

Fig. 4. *The city of Caldarola (Central Italy). The access road, Via Roma, to the historical centre blocked from the debris of overturning façade: SU2 and SU6 (view from inside).*

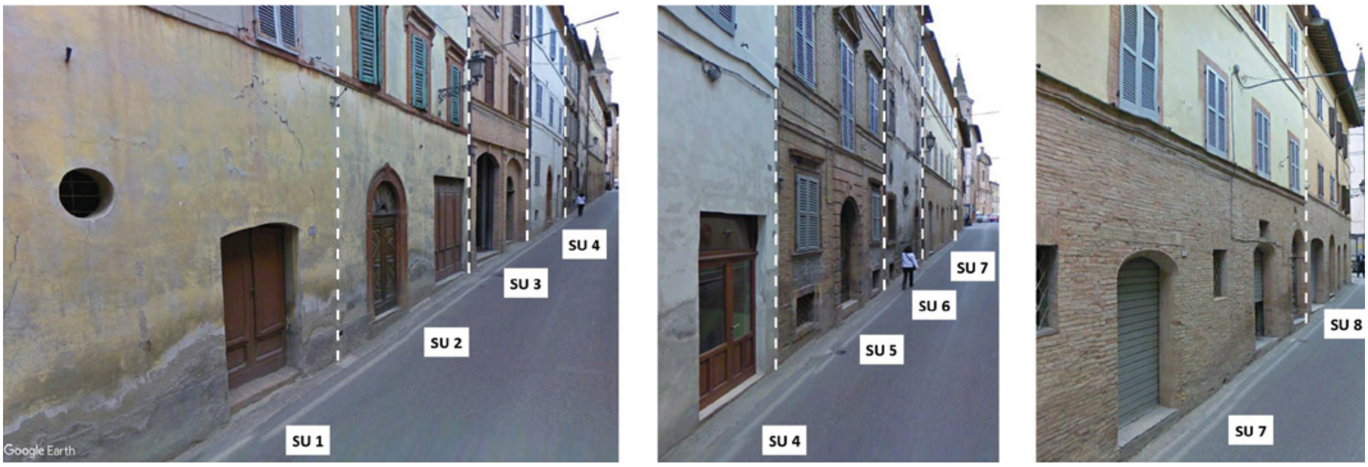

Fig. 5. *Google street view images (2011) of the eight SUs before the 2016 earthquake.*

regular blocks without headers elements, mud, and rubble inner core, weak mortar) as the prevailing masonry wall type called "rubble masonry" of Central Italy [8]. The constructions present regular elevation (e.g., three storeys), homogeneity of geometric features (e.g., regular opening layout), and construction techniques (e.g., roof types). However, different states of conservation of the facades emerge from pre-seismic facades images on Google street view according to the buildings being probably strengthened after the earthquake occurred in 1997 (Fig. 5).

4. RESULTS

The results of the research are illustrated in this section. First, the vulnerability index for the case study is calculated with the proposed method. In the following sub-sections, such a result is compared with the analytical and the empirical method results. Finally, the results and comparisons are discussed.

4.1. PROPOSED METHOD

When considering the description of the case study, all the parameters are assessed for eight SUs of the Caldorola aggregate n.2 (Fig. 4). According to the proposed methodology (section 2), specific information on construction techniques is not necessary since the procedure relies on a few parameters regarding the façade. In the absence of data about the wall (thickness, specific weight), reference is made to the prevailing typology in the area, i.e., "rubble wall" of 60 cm wall thickness and a specific weight of 19 kN/m^3. The resulting IV_f indices are comprised between 49 and 92, with being 92 the highest vulnerability, for SU2, and 49 being the lowest, for SU4.

4.2. ANALYTICAL METHOD

According to section 2.2, the analytical method has been applied for the eight SUs of Caldarola according to FC=1.35 and LC1. The documentation of past retrofitting interventions on the SUs supported the analysis of the effectiveness of the anti-seismic devices. Indeed, the tie rods of SU4 and 8 were applied before NTC2008, but certainly after the 1997 earthquake; those of SU6 are pre-1997; instead, tie rods of SU1 are inserted after 2011 as they are not in 2011 images. As by section 2.1.3, the effectiveness and quality of such devices are more probably better after the introduction of 2008 regulations

Vulnerability index of facades - IV_f							
SU 1	SU 2	SU 3	SU 4	SU 5	SU 6	SU 7	SU 8
66.54	92.54	57.38	53.37	70.34	90.76	74.38	63.23

Tab. 10. *Vulnerability indices of facades (IV_f).*

Safety index - I_s							
SU 1	SU 2	SU 3	SU 4	SU 5	SU 6	SU 7	SU 8
0.91	0.17	-	0.55	0.42	0.23	0.31	0.48

Tab. 11. Safety index of facades (I_s).

(NTC2008), which is therefore selected as a date for determining qualitative information on the anti-seismic devices.

Moreover, SU3, as from additional documentation, has RC slabs and curbs at intermediate floors and roof steal curbs that are well connected to the façade, thus preventing the OOP mechanisms from occurring. The following safety index (I_s) have been found:

4.3. EMPIRICAL METHOD

The empirical method consists in assessing damage after the 2016 Central Italy earthquake (Fig. 6). Facades of the aggregate are characterised by diverse crack patterns since they have different states of conservation. On the other hand, SUs 1, 2, and 6 were evidently in a state of neglect before the seismic event of 2016. However, the anti-seismic devices, even if not recently installed, seem to have been effective in avoiding the total collapse of the facade. Two SUs had a partial collapse. In SU2 there is an arch failure mode caused by the punching effect of the roof and the absence of an orthogonal load-bearing wall. In SU6, there is the overturning of the upper horizontal spandrel and fall of plaster portions. Therefore, the damage state NC has been assigned to these SUs. SU1 suffered significant damage comparable to SD, and further seismic shock could trigger OOP failures. The presence of anti-seismic design certainly prevented the occurrence of such mechanisms. The other SUs had a slight crack pattern comparable to MD.

5. COMPARISON AND DISCUSSION

The comparison of the methods is performed with the purpose of validating the proposed expeditious method.

The first comparison is conducted between the proposed method (section 2.1) and the analytical one (section 2.2) by ranking decreasing IV_f and increasing I_S. Then, the proposed method is compared with the empirical one (section 2.2) by ranking decreasing DS severity for each SU of the case study. The results of both comparisons are reported in Figure 7. The percentage of agreement is obtained by considering the correspondence of the indices calculated with different methods of the entire sample as 100% agreement, while lower agreement percentages are evaluated based on the number of SUs with corresponding results. By comparing the proposed method (IV_f) and the analytical one (I_S) there

Fig. 6. *Damage pattern and corresponding DS of each SU.*

	I_s ↓	
I	SU 2	0.17
II	SU 6	0.23
III	SU 7	0.31
IV	SU 5	0.42
V	SU 8	0.48
VI	SU 4	0.55
VII	SU 1	0.91
VIII	SU 3	-

75% matching

	IV_f ↑	
I	SU 2	92.54
II	SU 6	90.76
III	SU 7	74.38
IV	SU 5	70.34
V	SU 1	66.54
VI	SU 8	63.23
VII	SU 3	57.38
VIII	SU 4	53.37

	DS ↑	
I	SU 2	NC
II	SU 6	NC
III	SU 1	SD
IV	SU 7	MD
V	SU 5	MD
VI	SU 8	MD
VII	SU 3	MD
VIII	SU 4	MD

87.5% matching

Fig. 7. *Comparison between the vulnerability indices (IV_f), the safety indices (I_s) and the damage states (DS). The % of matching is defined by the jumps among the classes (I-VIII) in the rankings.*

is 75% coincidence, showing a good agreement among them. By looking at the two indexes, they have different trends due to their intrinsic definition, as I_s, which is referred to the safety level of the SU, is growing, while IV_f, which indicates the vulnerability to damage of the SU, is decreasing. With respect to the comparison of IV_f and the DS, as by the empirical method, there is an 87.5% agreement between the two. In this case, IV_f and DS have the same trend and comparable values (especially for higher damaged and vulnerable SUs).

Moreover, it is also crucial to highlight that when there are differences among the two, the IV_f overestimates the vulnerability/damage, thus allowing to have a prudential approach. There are two exceptions in the similarity of results from the comparison: that of SU3 and SU1. These exceptions indicate the limit of the proposed method, but they can be explained. The SU3 has curbs preventing OOP mechanisms. Indeed, it is not possible to detect some detailed construction features when they are not visible from the outside (e.g., they are covered by plaster). In the case of SU1, from the kinematic analyses it turns out as the least vulnerable SU (with respect to overturning), which was assigned after a critical-historical study on the documentation of the SU. From the documentation, it was possible to know that the ties were realized according to NTC2008, thus most probably showing high effectiveness and adherence to regulations. As the proposed method would not have this information, vulnerability is overestimated as a consequence (IV_f=66.54). A more precise vulnerability assessment could be achieved by means of experts' judgment, in which case the consideration of the ties would allow adding the reduction coefficient for anti-seismic devices equal to 0.43, thus leading to IV_f=33.66, which is in line with the result of the kinematic analysis. Eventually, the proposed method overestimates the SUs vulnerability, thus leading to a favorable precautionary approach to vulnerability assessment.

6. CONCLUSIONS

In this research, a new expeditious vulnerability assessment method for the facades of masonry buildings is proposed. It is a scoring method based on five parameters related to geometry and construction features, as well as on two additional qualitative coefficients. The method does not aim at defining a precise global behaviour of seismic vulnerability but proposes a rapid and effective procedure to target those buildings that are more prone to OOP damages that obstruct passage on adjacent streets.

The effectiveness of this method is demonstrated through the application in a case study, an aggregate composed of 8 structural units located in the historic center of Caldarola (Macerata, Marche Region), stricken by the 2016 Central Italy Earthquake. The results obtained with the proposed method are validated by means of comparison with the results of an analytical approach, which is linear kinematic analysis, and are also validated by means of comparison with the results of an empirical approach, which is the direct observation of post-earthquake damages. The correspondence is quite satisfactory, as 87.5% of SUs show consistent results with the empirical method and 75% with the analytical ones. However, these comparisons have underlined some limitations. Indeed, the method overestimates vulnerability in the presence of anti-seismic devices. The presence of inter-storey curbs and the effectiveness of tie rods can create evaluations uncertainties as they are not easily detectable by an external survey. Of course, an in-depth analysis of the building, such as in materials' inspections or historical-critical surveys, allows a more precise choice of the class for some parameters (i.e., specific weight of masonry type, wall thickness for slenderness calculation, and type of roof) and the additional qualitative coefficient regarding the effectiveness of anti-seismic devices.

In conclusion, the proposed method demonstrates high accuracy, especially for the application in historic masonry aggregates, as in most cases, they present poor connections between façade, orthogonal walls, and horizontal structures.

The proposed method could thus be of interest for local administrations and professionals involved in the retrofit and mitigation of seismic risk of historic masonry buildings, as well as for researchers in the field of expeditious methods for seismic vulnerability assessment and emergency planning.

Credit Authors statement

Conceptualization, L.B., G.V., F.R., and G.M.; Formal Analysis, L.B. and G.V.; Funding acquisition, G.M.; Investigation, L.B. and F.R.; Methodology, L.B., G.V. and G.M.; Project administration, G.M.; Supervision, G.M.; Validation, G.V.; Visualization, L.B.; Writing – original draft, L.B., G.V. and F.R.; Writing – review & editing, F.R.

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