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Numerical calibration of an easy method for seismic behaviour assessment on large scale of masonry building aggregates

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ABSTRACT

The paper deals with the numerical calibration of a speedy procedure for large scale seismic vulnerability assessment of masonry building aggregates, which are typical building compounds diffused within historical centres of many Italian towns. First of all, based on several numerical analyses developed with the 3MURI calculation program, this simplified assessment procedure has been implemented, it being derived from the well known vulnerability form for masonry buildings integrated by five parameters accounting for the aggregate conditions among adjacent units. Later on, the set-up procedure has been validated through an application to a single building aggregate in the Vesuvius area. Since the results previously achieved have been again confirmed, subsequently the procedure has been used to investigate a wide area of the historical centre of Torre del Greco, allowing for the knowledge of the buildings most at risk under earthquake.

Finally, the methodology has been applied to the historical centre of Poggio Picenze (AQ), damaged by the recent Italian earthquake (2009), in order to prove its effectiveness to foresee the damage level experienced by other types of masonry aggregates under seismic actions.

1. Introductory remarks

Masonry is the most diffused construction material in the Italian historical centres, which are often the result of an uncontrolled urban development based on buildings erected in continuity to each other, so resulting into aggregates of constructions. These were generated by the progressive transformation of the urban tissue, in which elevation floors were added to existing constructions and plan extensions were made by adding structural units to the existing ones, so that often adjacent units shared the same boundary walls. Therefore, it is very difficult, if not impossible in some cases, to distinguish the structurally independent units and also to identify the global response of the building compound. So, seismic vulnerability assessment of masonry aggregates in the Italian historical centres represents a specific and very actual problem to be solved in order to foresee their behaviour under earthquake and, where deficiencies occur, to implement seismic protection measures.

The main difficulties of this task are related to the low knowledge level of these structures, which were in many cases built without anti-seismic design regulations, particularly due to the absence of drawings and/or reports. In addition, the careful analysis of these building complexes should take into account all structural units. This can be performed from the research point of view only by using either very complex numerical approaches [1,2] or experimental dynamic tests [3,4]. On the contrary, this is an activity complicated to be developed at the design level by engineers and architects for seismic vulnerability analysis of these building groups.

Furthermore, the recent and innovative technical Italian code (NTC 2008) [5] does not provide reliable methodologies to solve problematic issues connected to this topic.

On the other hand, in literature, starting from "codes of practice" for different historical city centres proposed by Giuffrè [6], some interesting papers have analysed the current topic in order to evaluate the behaviour of masonry buildings grouped into aggregates.

In 2005 Binda and Saisi [7] gave a general methodology to be followed for seismic vulnerability assessment and protection of historical masonry buildings. In particular, they prepared a report on the state of the art of research carried out in Italy in the field of cultural heritage restoration and conservation, also by focusing

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their attention on building compounds. After the classification of typologies of historic buildings was presented and the materials and masonry construction technologies were discussed, several mathematic models for structural analysis were provided. Finally, appropriate repair and improvement techniques for different type masonry buildings were given.

In 2004 Ramos and Lourenço [8] addressed the seismic analysis and vulnerability of historical city centres by treating the case study of the 18th century downtown part of Lisbon. Different finite element method analyses considering the non-linear behaviour of materials were performed on a selected building compound aiming at evaluating its stability with respect to overturning mechanisms.

Analysis results showed that the ‘‘aggregate effect’’ is felt in two ways: globally, since the force distribution obtained from analysis of each building is different from the one calculated on the whole compound, and locally, considering pounding damages due to change of building stiffness resulting from the insertion of new reinforced concrete and steel members in the structure. It was found that individual buildings are more flexible than the compound and have lower safety factors. So, ‘‘compound effect’’ is beneficial for buildings which can be studied as isolated in order to reduce the computational efforts. However, the mentioned approaches can be usefully applied when local analysis on single masonry building compounds are of concern only.

Instead, about large scale analysis of building aggregates, the work of Pagnini et al. [9] is noteworthy. The paper discusses in particular a mechanical model for vulnerability assessment of masonry building compounds in the historical city centre of Coimbra considering uncertainties related to different factors, such as building parameters, seismic demand and model error. Capacity curves were assessed according to a probabilistic approach taking into account the variability of both structural response and seismic demand. In addition, by representing seismic demand as response spectra, vulnerability analysis was carried out with reference to several random limit states. Finally, fragility curves were derived taking into account the influence of uncertainties of different parameters examined.

Nevertheless, the need to have simpler approaches for large scale seismic vulnerability assessment of masonry building aggregates is particularly felt aiming at providing effective management tools to be used by Municipalities, especially in the prevention phase from earthquakes, for directing retrofitting interventions. In addition, the individuation of most vulnerable aggregates allows also to address aids in a rational way during the post-earthquake emergency phase.

To this purpose, a quick procedure for seismic vulnerability assessment of masonry compounds opportunely calibrated on the basis of numerical analyses performed at different urban scale levels, namely small scale (single aggregates) and large scale (parts of historical centres), has been implemented and proposed in the current paper. This should be the first step towards the implementation of a rigorous methodology to evaluate the seismic vulnerability of single buildings grouped into aggregates.

2. A simplified seismic vulnerability assessment methodology

2.1. The proposed form

Aiming at implementing a speedy seismic evaluation procedure for masonry aggregates, the starting point has been represented by the Benedetti and Petrini’s methodology, widely used in the past as a quick technique, based on collecting into an appropriate form some information on single buildings, for investigating their vulnerability under earthquake [10,11].

This form is based on ten parameters used to recognise the main structural system and its fundamental seismic deficiencies.

The first parameter ‘‘Organization of vertical structures’’ identifies features of the building structural apparatus, defined as the system withstanding more than 70% of the seismic forces.

The second parameter ‘‘Nature of vertical structures’’ appraises the structural system quality with respect to different criteria, such as construction materials, workmanship features and execution efficacy.

The third parameter ‘‘Location of the building type and foundation’’ evaluates the influence of both consistency and slope of soil category and height difference between foundations on the building seismic performances.

The fourth parameter ‘‘Distribution of plan resisting elements’’ is based on the ratio between the acting base-shear, gotten by the elastic response spectrum, and the structure resistant base-shear, representative of the system shear resistant capacity.

The fifth parameter ‘‘In-plane regularity’’ takes into account both the building plane configuration and the seismic-resistant elements mass and stiffness distribution.

The sixth parameter ‘‘Vertical regularity’’ considers the mass change among levels and possible discontinuities in the positioning of vertical seismic-resistant systems.

The seventh parameter ‘‘Type of floor’’ accounts for the in-plane stiffness of floors and their connections with the vertical seismic-resistant systems.

The eightieth parameter ‘‘Roofing’’ judges the roof typology and the possible pushing actions applied to masonry walls.

The ninth parameter ‘‘Details’’ classifies non-structural elements as internal (partition walls, furniture, flush ceilings, etc.) and external (antennas, cornices, parapets, chimneys, balconies, etc.) elements that may or may not collapse partial or totally depending on the connection quality to the resisting elements in the structure.

The tenth parameter ‘‘Physical conditions’’ evaluates structural imperfections and damages into both in-elevation load-bearing systems and foundations.

Based on this approach, which requires external inspection of buildings only, the vulnerability index of an isolated masonry building was calculated according to the following expression [12]:

$$I_{V,I} = \sum_{i=1}^{10} s_i \cdot w_i \tag{1}$$

where s_i and w_i are the score and weight, respectively, of the form generic parameter. Four scores (from A, minor, to D, major) are used to describe the vulnerability classes of each parameter, whereas weight (ranging from 0.25 to 1.50) represents the less or more importance of the parameter in quantifying the building vulnerability.

This vulnerability assessment form, whose basic parameters are reported in Table 1 with white background, has been adopted with some small adjustments by the Italian National Group Against Earthquakes as first screening tool for vulnerability assessment of masonry and r.c. buildings belonging to historical centres [12].

In order to consider the structural interaction among adjacent buildings, not considered in the cited method, a new form has been ideated [13]. This is resulted from adding to the basic ten parameters of the original form new five parameters taking into account interaction effects among aggregate structural units under earthquakes. These factors, in part derived from previous studies found in literature [14], are:

1. In elevation interaction.
2. Plan interaction.
3. Number of staggered floors.

Table 1
The new vulnerability assessment form proposed for buildings in aggregate.

Parameter	Class score (s)				Weight (w)
	A	B	C	D	
1. Organization of vertical structures	0	5	20	45	1
2. Nature of vertical structures	0	5	25	45	0.25
3. Location of the building and type of foundation	0	5	25	45	0.75
4. Distribution of plan resisting elements	0	5	25	45	1.5
5. In-plane regularity	0	5	25	45	0.5
6. Vertical regularity	0	5	25	45	0.5–1
7. Type of floor	0	5	15	45	0.75–1
8. Roofing	0	15	25	45	0.75
9. Details	0	0	25	45	0.25
10. Physical conditions	0	5	25	45	1
11. Presence of adjacent buildings with different height	−20	0	15	45	1
12. Position of the building in the aggregate	−45	−25	−15	0	1.5
13. Number of staggered floors	0	15	25	45	0.5
14. Structural or typological heterogeneity among adjacent structural units	−15	−10	0	45	1.2
15. Percentage difference of opening areas among adjacent facades	−20	0	25	45	1

4. Structural or typological heterogeneity among adjacent structural units.
5. Percentage difference of opening areas among adjacent facades.

The new survey form is depicted in Table 1, where new five parameters appear on a grey background.

In order to achieve a form totally homogeneous to the previous one, scores and weights assigned to these five additional parameters have been numerically calibrated on the basis of the results of specific numerical parametric analyses. Such analyses have been performed by the 3MURI non linear numerical software, which uses the Frame by Macro-Elements (FME) computational method [15]. So, the reference model of masonry structures is a three-dimensional equivalent frame, where walls are schematized as framed systems composed of piers and spandrel beams connected to each other by means of rigid links. In fact, by considering the location of openings, walls can be divided into vertical areas corresponding to different levels, so identifying masonry piers and spandrels where both deformability is concentrated and damages are detected under earthquakes. In particular, spandrel beams can be modelled only if they are adequately notched by the walls, supported by structurally efficient architraves, and if a possible strut resistant mechanism could be activated. Structural parts not susceptible to be damaged under seismic actions are modelled as rigid links, which allow to join masonry deformable parts in order to define a wall model completely comparable to that of a plane frame.

In the current work, a masonry structural unit typical of the urban tissue of Sessa Aurunca, a small Italian city within the Campania region, has been modelled with the 3MURI FME computational approach.

The examined construction (Fig. 1) has a vertical structure made of 60 cm thick squared tuff stones with a reduction of 10 cm in thickness at each floor. The mechanical properties of tuff masonry have been taken from Table C8A.2.1 of the Italian Ministerial Circular (2009) [16] by considering a LC1 knowledge level.

Horizontal structures are made of mixed r.c. – hollow tile floors connected to walls by r.c. tie beams. So, a box-type structural behaviour of the building has been considered, without investigating the first mode collapse mechanisms of masonry walls.

The elastic spectrum of the building site has been achieved from appendix of the NTC 08 [5], by considering a subsoil type A, a topographic category T_1 and a 2nd class of use (ordinary building with a nominal life of 50 years).

For each new form parameter, several numerical pushover have been performed in order to reproduce different boundary conditions among adjacent structural units. According to the original form, the purpose of this activity was the definition of four classes with the corresponding scores for each new parameter, which an appropriate weight has been assigned to.

These results have been collected by performing pushover analyses with horizontal forces proportional to the first vibration mode along the longitudinal (X) and transverse (Y) directions of the building considered before isolated and subsequently inserted within the aggregate. The building behaviour has been assessed in both cases by defining the mechanical vulnerability index I_M as:

$$I_M = \frac{D_{max}}{D_u} \tag{2}$$

where D_{max} is the maximum horizontal displacement required by earthquake (demand) and D_u is the building displacement in ultimate conditions (capacity).

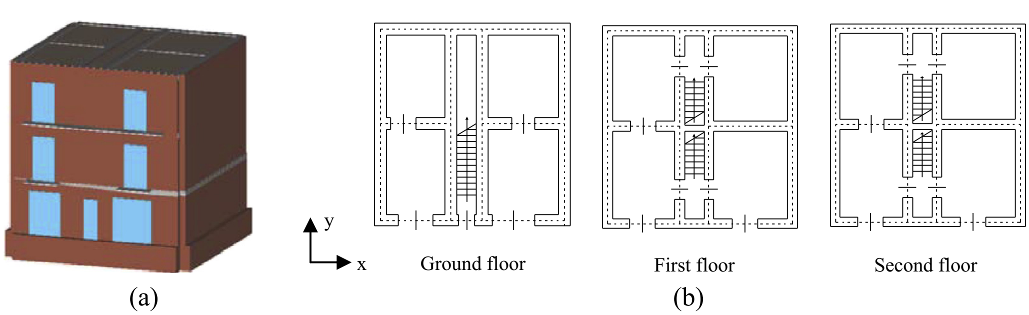


Fig. 1. A typical structural unit of Sessa Aurunca: the 3MURI numerical model (a) and plan layouts at different floors (b).

Therefore, the scores have been determined so that the difference among the indexes associated with different classes of each parameter is proportional to the difference among the corresponding mechanical vulnerability index values obtained in the analyses performed in the most severe direction. In particular, also negative scores have been introduced in the new form in order to take into account some positive effect deriving from the aggregate condition. Instead, for weight assignment, as a first step the absolute value maximum differences among vulnerability indexes related to the several classes of each parameter have been considered. Then, the weights have been assigned to each new parameter proportionally to these differences and, finally, they have been homogenised with the original form ones.

First of all, the interaction in height among adjacent buildings has been evaluated by considering six different analysis cases (Fig. 2).

The analysis results, expressed in terms of vulnerability indexes and collapse mechanism of analysed buildings, are respectively reported in Figs. 3 and 4.

From Fig. 3 it is apparent that the building vulnerability is higher in direction *X* than direction *Y*. Also, the most dangerous cases are when the building is within two shorter constructions (one and two floors). In fact, in these cases, since the constraining action of adjacent buildings is partially provided only, the central building is free to deform laterally at last levels.

Later on, the plan interaction parameter has been investigated considering four different positions of the building, namely isolated (Fig. 5a), within other buildings (Fig. 5b), in the aggregate corner (Fig. 5c) and in a leading position of the aggregate (Fig. 5d).

All the achieved results are summarised in Table 2, whereas in Fig. 6 the failure modes of the structural unit within aggregate are shown. It is apparent that the lower vulnerability index is attained when building is within two edifices.

In order to calibrate the parameter related to the influence of staggered floors among aggregated buildings, the following five conditions have been modelled:

a. total absence of staggered floors (Fig. 7a).
b. presence of one staggered floor (Fig. 7b).
c. presence of two staggered floors at the same level (Fig. 7c).
d. presence of two staggered floors at different levels (Fig. 7d).
e. presence of four staggered floors (Fig. 7e).

For each case, static non-linear analyses have been performed in order to evaluate the structural ductility and the vulnerability indicator I_M. All the results are summarised in Table 3 and in Fig. 8, where graphical representation of vulnerability indexes is depicted.

It is clear that staggered floors have a little influence on the global in-plane behaviour of masonry aggregates under study, since vulnerability indexes are very similar each other in both directions examined. However, even if in a negligible way, as the number of staggered floors increases, the vulnerability index augments.

Later on, the parameter regarding either structural or typological heterogeneity among adjacent structural units has been examined. In order to standardise this parameter, the following four possible conditions have been taken into account:

a. aggregate buildings are homogeneous from typological and structural viewpoints (Fig. 9a).

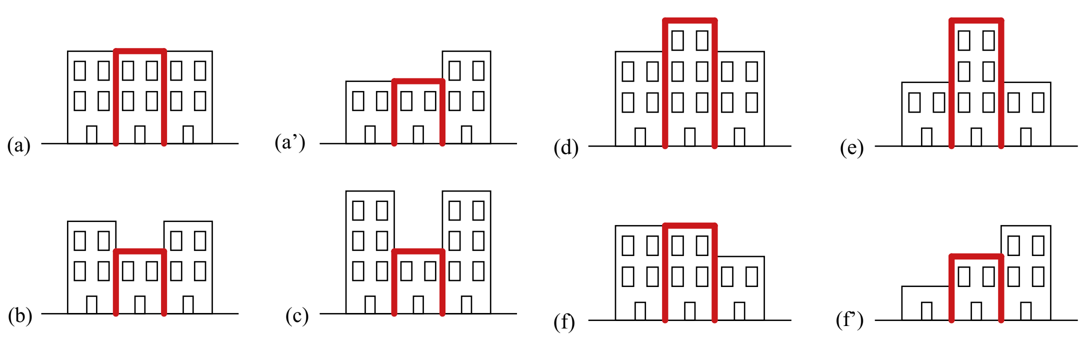


Fig. 2. Different conditions regarding the in elevation interaction among buildings in aggregates.

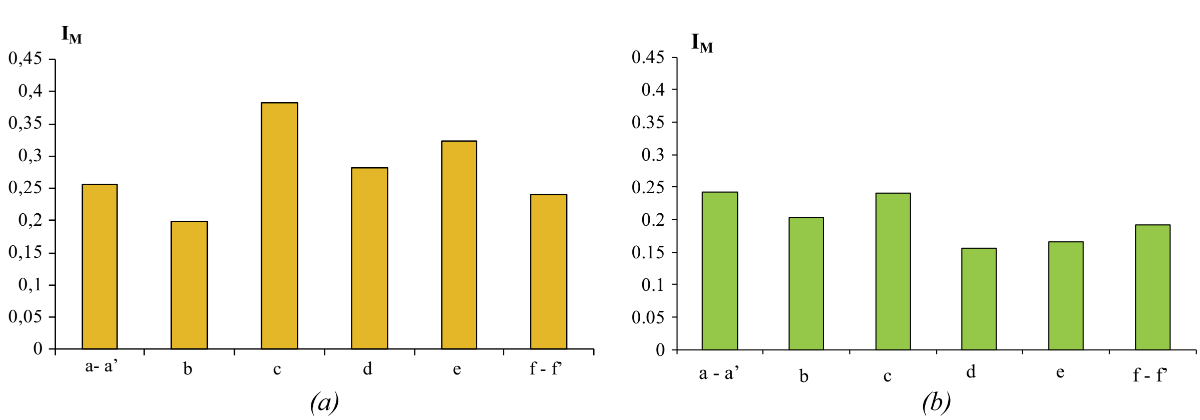


Fig. 3. Vulnerability index I_M in the (a) longitudinal (*X*) and (b) transverse (*Y*) directions.

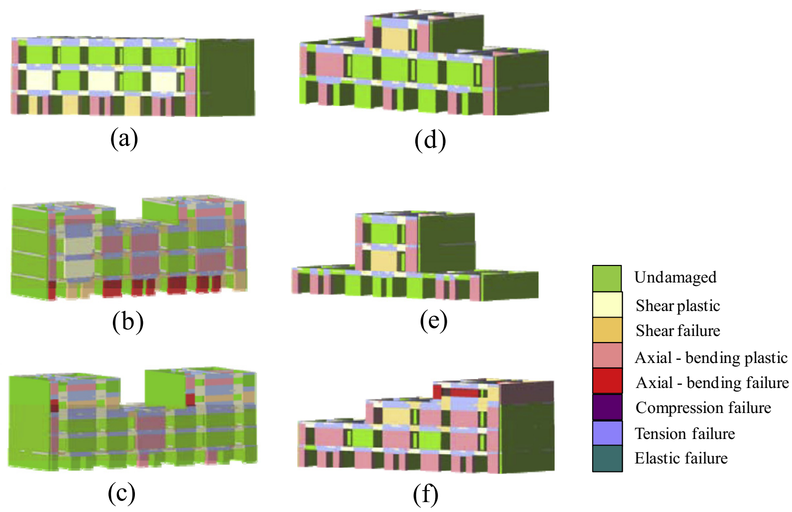


Fig. 4. Collapse mechanisms of different FEM models (in elevation interaction).

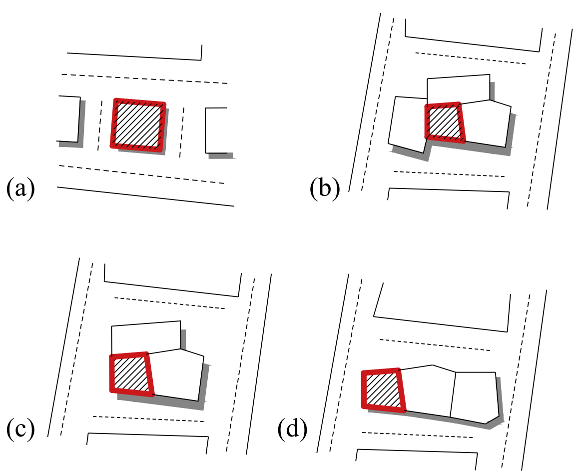


Fig. 5. Possible positions of the building in the aggregate.

b. the building is adjacent to buildings made of the same material but erected with a construction technique worse than the examined one (Fig. 9b).
c. the building is close to buildings made of the same material but erected with a construction technique better than the examined one (Fig. 9c).
d. the building has a structural typology very different from that of adjacent buildings (Fig. 9d).

The pushover curves in the most vulnerable direction (*X*) related to the above analysis cases are plotted in Fig. 10.

The main results in terms of vulnerability indexes and collapse mechanisms are depicted in Figs. 11 and 12, respectively.

It is noticed that in both analysis directions the worst condition is when the base structural unit is next to units made of materials having greater strength. On the other hand, when a r.c. building is adjacent to the base structure, the lower vulnerability index is achieved.

Finally, the parameter accounting for the percentage difference of opening areas among adjacent structural units has been assessed by considering in the FEM analyses the following five conditions:

a. no difference among adjacent facades (Fig. 13a).
b. difference more than 50% (Fig. 13b).
c. difference more than 25% (both sides) (Fig. 13c).
d. difference less than 25% (Fig. 13d).
e. difference more than 25% (from one side only) (Fig. 13e).

The results in terms of vulnerability indexes I_M and distribution of damage within the building aggregate deriving from pushover analyses are reported in Figs. 14 and 15, respectively. It is apparent that the worst condition is achieved when the opening area of the study unit is less than 25% the adjacent units one.

2.2. Assignment of weights and scores to the new form parameters

On the basis of pushover analysis results, appropriate weights have been assigned to each additional parameter of the new form. These weights have been established in proportional way to the maximum absolute difference Δ, in the most vulnerable direction, among the vulnerability indexes of four classes considered for each parameter (Table 4). In this table the parameter calculated weight *cw* has been obtained through the following relationship:

$$cw = \frac{\Delta}{0.06} \tag{3}$$

where Δ is the maximum absolute difference among vulnerability indexes achieved for each parameter in the performed analyses.

Table 2
Non linear behaviour of the structural unit considering plan interaction.

Direction	Longitudinal (*X*)				Transverse (*Y*)			
Configuration	D_{max} (cm)	D_u (cm)	Ductility [D_u/D_{max}]	I_M	D_{max} (cm)	D_u (cm)	Ductility [D_u/D_{max}]	I_M
(a)	0.641	1.74	3.10	0.368	0.242	1.141	2.72	0.212
(b)	0.383	1.499	3.73	0.256	0.244	1.141	2.61	0.214
(c)	0.427	1.499	3.78	0.285	0.281	1.313	2.91	0.214
(d)	0.441	1.51	3.54	0.292	0.264	1.141	2.58	0.231

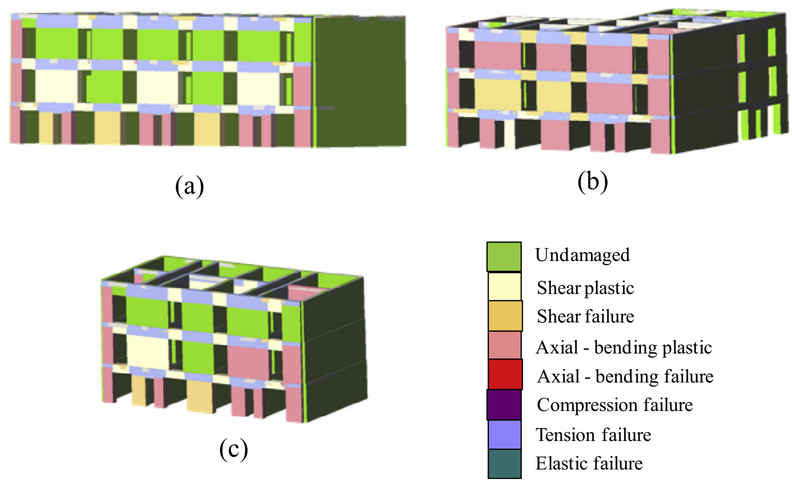


Fig. 6. Collapse mechanisms of different FEM models (plan interaction).

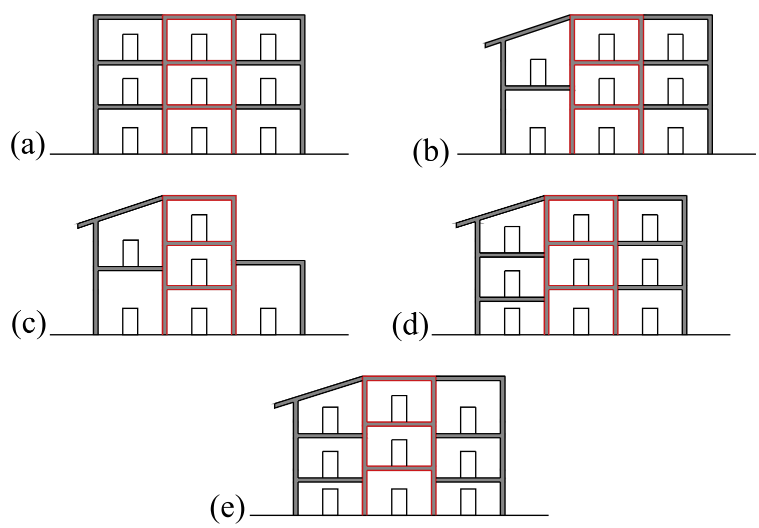


Fig. 7. Possible positions of staggered floors in the examined models.

Table 3
Pushover analysis results on the FEM models considering the presence of staggered floors.

Direction	Longitudinal (X)				Transverse (Y)			
Configuration	D_{max} (cm)	D_u (cm)	Ductility [D_u/D_{max}]	I_M	D_{max} (cm)	D_u (cm)	Ductility [D_u/D_{max}]	I_M
(a)	0.383	1.499	3.73	0.256	0.277	1.141	2.61	0.243
(b)	0.389	1.44	3.59	0.270	0.284	1.141	2.57	0.249
(c)	0.396	1.44	3.74	0.275	0.282	1.141	2.58	0.247
(d)	0.397	1.44	3.75	0.276	0.283	1.141	2.60	0.248
(e)	0.401	1.44	3.75	0.278	0.285	1.141	2.60	0.250

Starting from these calculated values, weights have been assigned to parameters trying to homogenise the achieved values with the form ones.

In particular, the following changes to weights, reported in Table 4 as assigned weight (*aw*), have been considered:

– *Parameter 2:* assigned weight is reduced to 1.50 since this is the form maximum weight.
– *Parameter 3:* weight has been increased as respect to calculated value considering that height difference between staggered floors can be larger than the ones of the case studies examined.
– *Parameter 4:* weight has been reduced to 1.20 considering the difficulty to find into historical centres structural units into aggregates with very large scatters of masonry mechanical properties.

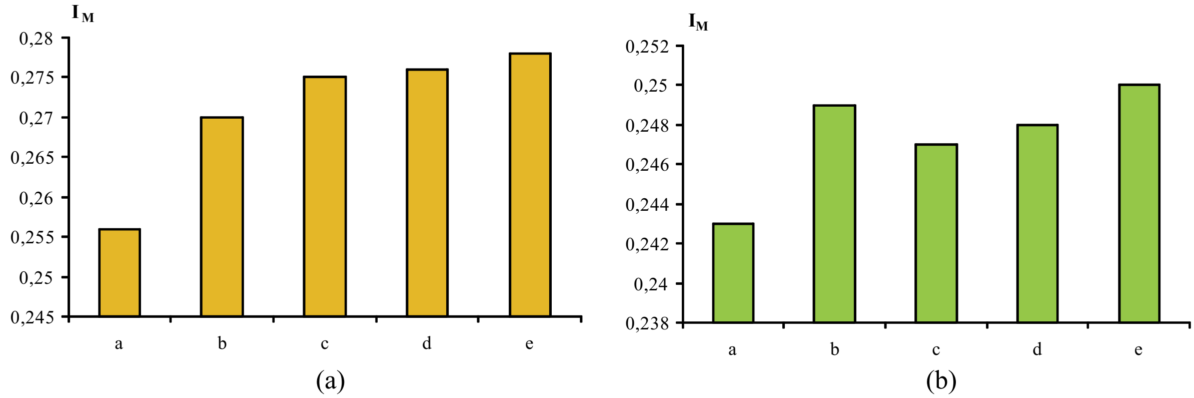


Fig. 8. Distribution of the vulnerability index I_M related to the influence of staggered floors: longitudinal (X) direction (a) and transverse (Y) direction (b).

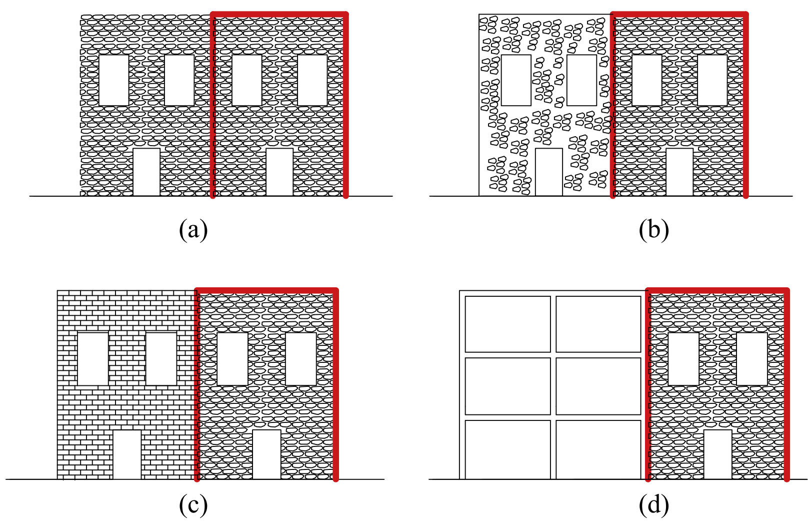


Fig. 9. Structural or typological heterogeneity among adjacent structural units.

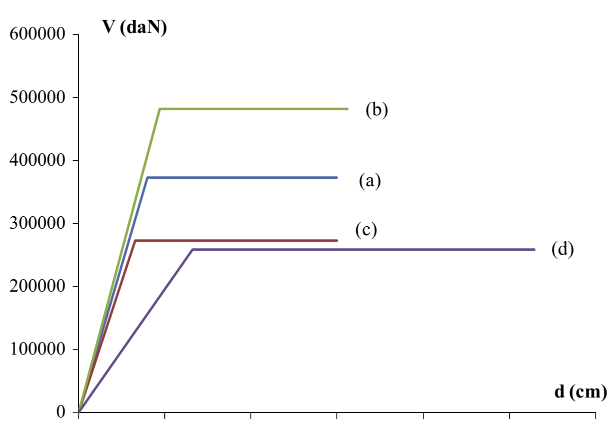


Fig. 10. Capacity curves of the examined configurations in the longitudinal direction (structural heterogeneity).

Instead the scores related to four classes of each parameter have been assigned proportionally to the differences in terms of vulnerability index among the different aggregate conditions foreseen in the numerical analyses before examined.

So, the results of pushover analyses have been grouped in order to contemplate four classes (A, B, C and D) for each new parameter of the survey form.

The aggregate conditions among adjacent structural units contemplated by the additional five parameters of the form are illustrated from Figs. 16–20.

Based on the above considerations, a new type of form composed of fifteen parameters has been therefore developed, as shown in Table 1. According to this simple large scale vulnerability assessment technique, the aggregate building vulnerability index $I_{V,A}$ is calculated according to the Eq. (1) considering the contribution of the new form fifteen parameters. The minimum and maximum values of $I_{V,A}$, achieved by summing respectively lowest and highest scores of each parameter multiplied by self weight, are −125.5 and 515.25.

2.3. The method application: a case study in the historical centre of Sessa Aurunca (CE)

The effectiveness of the proposed procedure has been proved by analysing a masonry aggregate located in the historical centre of Sessa Aurunca, a district in the province of Caserta, a town in the South of Italy.

The case study consists on a building aggregate composed of five different units having a vertical structure made of squared tuff masonry stones (Fig. 21). The compound expands along a curtain on the street and has an elongated shape. The buildings are developed on 3–4 storeys, having generally vaults at the ground floor, mixed r.c.-tile or steel-tile floors on other storeys and pitched

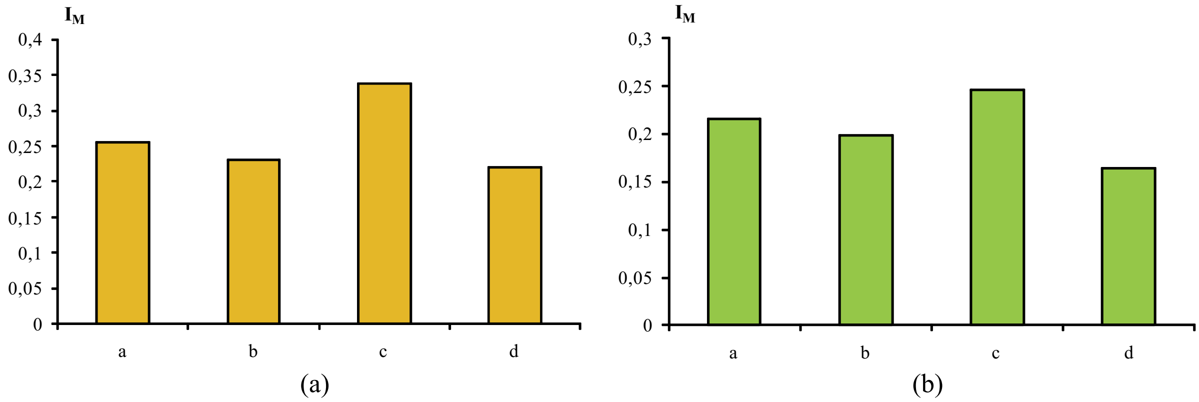


Fig. 11. Distribution of the vulnerability index I_M related to the structural heterogeneity: longitudinal (X) direction (a) and transverse (Y) direction (b).

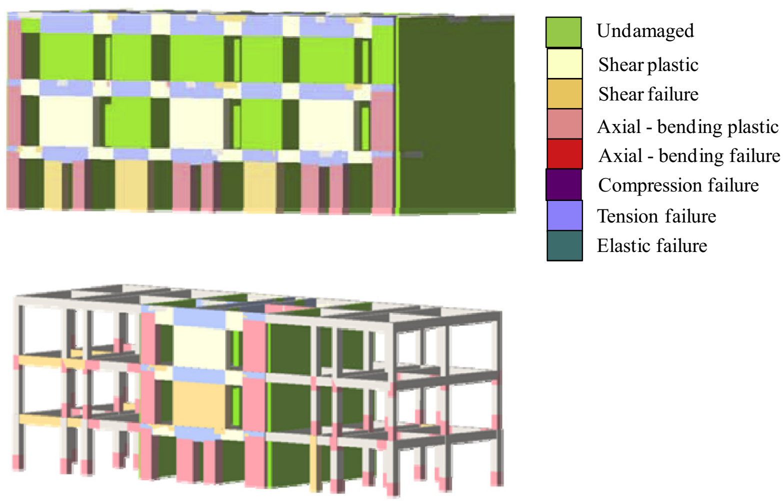


Fig. 12. Possible collapse mechanisms of some of the examined FEM models (structural heterogeneity).

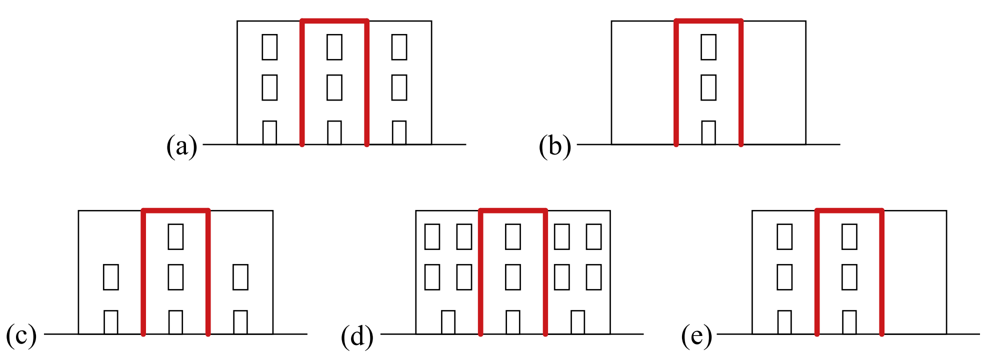


Fig. 13. Difference in terms of opening areas among adjacent facades.

wooden roofs with tie beams. Units n. 2, 4 and 5, visible in Fig. 21, were subjected to retrofitting interventions after the 1980 Irpinia earthquake. More details on this case study are reported in [17].

A numerical model of the building aggregate has been implemented by means of the 3MURI software in order to compare the achieved results with the ones deriving from the form application.

The pushover analyses have been performed by modelling each of the single structural unit both as isolated (Fig. 22) and as part of the compound (Fig. 23).

The seismic behaviour of buildings has been assessed along their longitudinal (X) and transverse (Y) direction considering the force distribution proportional to their first vibration mode. Two vulnerability indexes have been computed for each structure: the isolated building index ($I_{M,I}$) and the aggregate building one ($I_{M,A}$). The obtained results are summarised in Table 5.

Later on, the vulnerability index of the study structural units has been also calculated by using the quick procedures given by both the Benedetti and Petrini's form (isolated building – index $I_{V,I}$) and the implemented form (buildings in aggregate – index $I_{V,A}$).

The comparison among results of two different methodologies is displayed in Fig. 24. It is noted that when the structural unit is within the compound its seismic vulnerability is reduced if

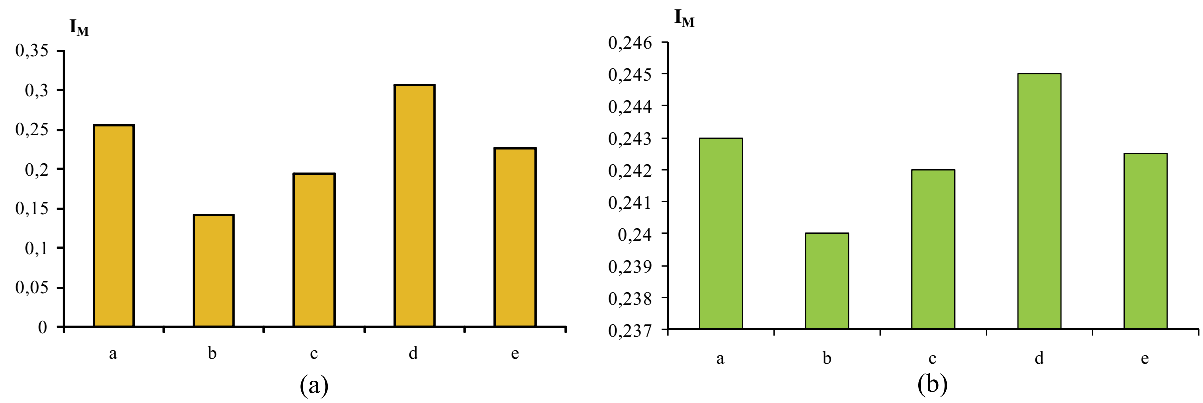


Fig. 14. Distribution of the vulnerability index I_M related to the different percentage of windows among adjacent facades: longitudinal (X) direction (a) and transverse (Y) direction (b).

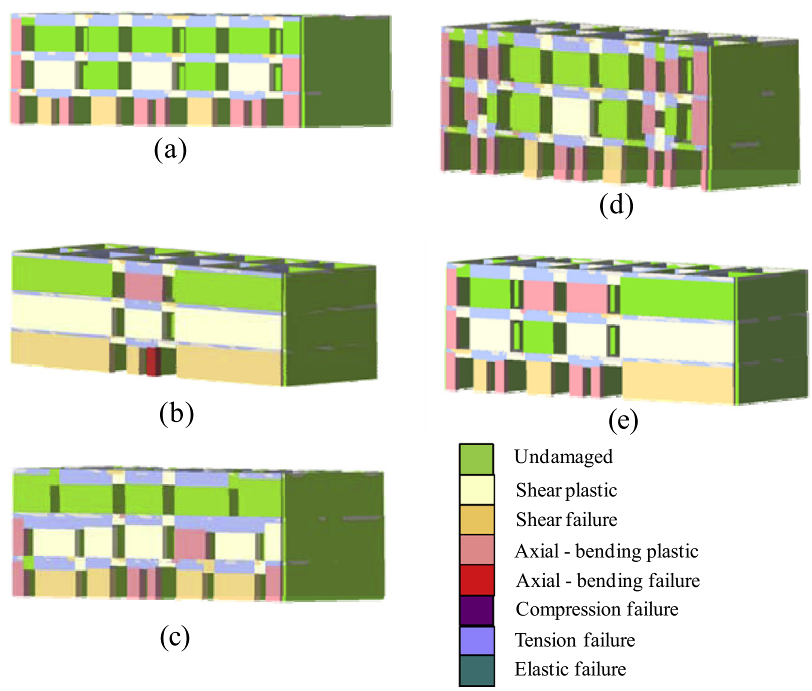


Fig. 15. Collapse mechanisms of the examined FEM models (difference in terms of opening area among adjacent units).

Table 4
Weights assigned to the new form parameters.

Parameter	Maximum difference [Δ]	Calculated weight (*cw*)	Assigned weight (*aw*)
1. Presence of adjacent buildings with different height	0.06	1.00	1.00
2. Position of the building in the aggregate	0.11	1.83	1.50
3. Number of staggered floors among aggregated buildings	0.02	0.33	0.50
4. Effects of either structural or typological heterogeneity among adjacent structural units	0.08	1.33	1.20
5. Percentage difference of opening areas among adjacent facades	0.06	1.00	1.00

compared to the one of the same building considered as isolated. In addition, the building ranking in terms of vulnerability is the same with the two applied methods. This proves the effectiveness of the proposed form.

In addition, it is apparent that the simplified technique provides the same vulnerability classification as respect to the mechanical method results provided in the longitudinal direction. Therefore, even if this first application has provided good results, further case studies have been developed aiming at its validation.

3. The method calibration: analysis in the vesuvius area

3.1. General

The proposed methodology has been used to assess the seismic vulnerability of a part of the historical centre of Torre del Greco (Fig. 25a). This city, about 20 km far from Naples, is one of the municipalities most exposed to the Vesuvius risk, since it was destroyed in the eruption of 79 AD. For this reason, the WG4 “Risk

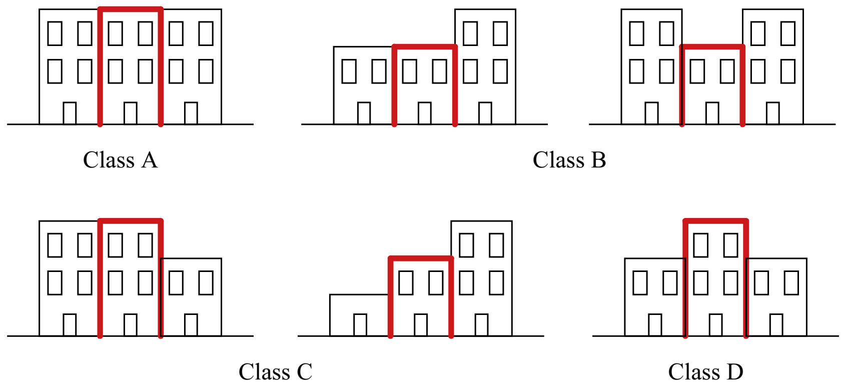


Fig. 16. In elevation interaction classes.

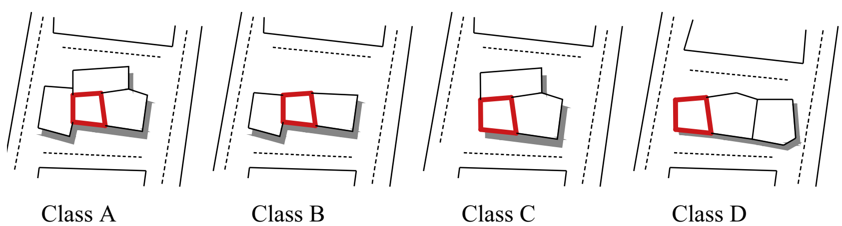


Fig. 17. Plan interaction classes.

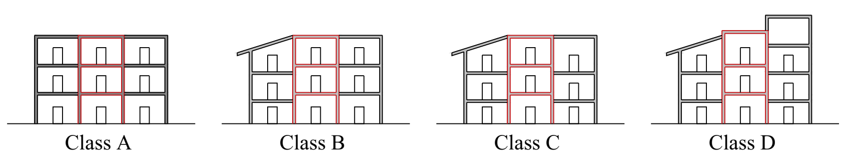


Fig. 18. Staggered floor classes.

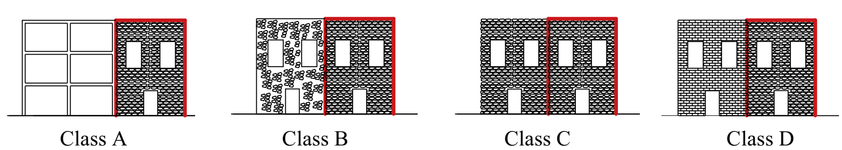


Fig. 19. Structural heterogeneity classes.

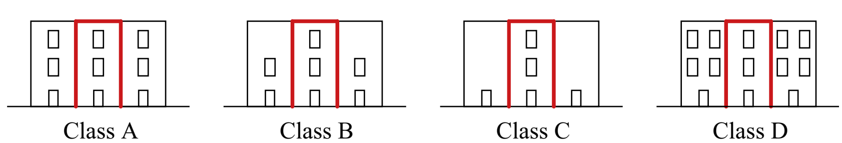


Fig. 20. Classes related to the percentage difference among opening areas of adjacent facades.

Assessment for Catastrophic Scenarios in Urban Areas" of the COST C26 Action "Urban Habitat Constructions under Catastrophic Events" [18] has selected the Vesuvius region, and in particular the historical centre of Torre del Greco, as investigation area, in order to evaluate the impact of the volcano on built up for both minimising life losses and implementing protection measures for both cultural heritage and ordinary building.

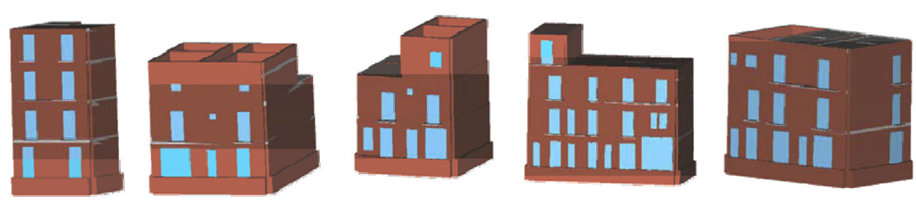
The data used for the methodology application were collected during specific visual *in situ* inspections of aggregates of the investigated zone.

The activity of *in situ* data collection related to the study area was done by WG4 members, with the contribution of the PLINIVS Centre (Hydrological, Volcanic and Seismic Engineering Centre, Director prof. Giulio Zuccaro) [19]. The data were collected by means of the compilation of a synthetic form, elaborated and commonly used by Italian Civil Protection Department, subdivided into eight different sections regarding the morphological, geometrical and structural properties of constructions from seismic and volcanic viewpoints.

In the post-survey phase, all the collected data have been organised and put in a database. In particular, a suitable elaboration of data acquired over the whole pilot area has been carried out in the GIS environment. By processing these data, homogeneous groups of buildings have been identified, their main features being

(a) (b)

Fig. 21. The aggregate investigated in Sessa Aurunca: plan layout (a) and 3D view (b).



1 2 3 4 5

Fig. 22. FEM models of the structural units of the study aggregate in Sessa Aurunca.

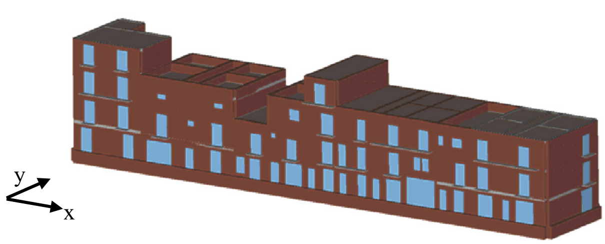


Fig. 23. The numerical model of the entire aggregate in Sessa Aurunca.

Table 5
Numerical vulnerability indexes obtained considering the building both as isolated and as part of the aggregate.

Building	Direction	Isolated			Within the aggregate		
		D_{max} (cm)	D_u (cm)	$I_{M,I}$	D_{max} (cm)	D_u (cm)	$I_{M,A}$
1	*X*	1.176	2.323	0.51	0.713	1.222	0.58
	Y	0.563	2.324	0.24	0.447	2.242	0.20
2	*X*	0.590	1.561	0.38	0.564	1.601	0.35
	Y	0.261	1.799	0.15	0.282	2.320	0.12
3	*X*	0.654	1.186	0.55	0.515	1.039	0.50
	Y	0.153	1.182	0.13	0.160	0.420	0.38
4	*X*	1.858	2.742	0.68	1.780	2.560	0.70
	Y	0.263	0.801	0.33	0.231	0.481	0.48
5	*X*	0.750	1.599	0.47	0.652	1.440	0.45
	Y	0.258	1.381	0.19	0.209	0.830	0.25

displayed in several thematic maps created by means of the ArcGIS software [20].

In the investigation area, the main structural typology is represented by masonry buildings (80%), while r.c. buildings are in a few percentage (9.5%). More than one half of the buildings, which developed mainly on three floors and are in a mediocre conservation state, were erected before 1919 and have mixed steel-hollow tile floors. Major details on the performed activity are available in [21].

3.2. Small scale application

3.2.1. Typological and geometrical features

Aiming at validating the proposed survey form, the analysis of a case study represented by an existing masonry building aggregate in the historical centre of Torre del Greco has been done. The examined masonry compound (Fig. 25b), which consists of five tuff masonry stones units structurally dependent each other, has a regular plan layout and covers a total area of about 877 m^2.

During the *in situ* inspection, a survey of the above selected masonry compound has been carried out in order to evaluate the structural characteristics of its constituent units and, at the same time, to enable the collection of data necessary to apply the proposed vulnerability assessment procedure.

The buildings, identified with numbers from 1 to 5 in Figs. 26 and 27, have the following peculiarities:

– The building n. 1 is placed at the internal corner of the compound, so it shows two free sides only. The surface is about 97 m^2. It is composed of 4 storeys: the ground level height is 4 m, while other inter-storey heights are 3.20 m.

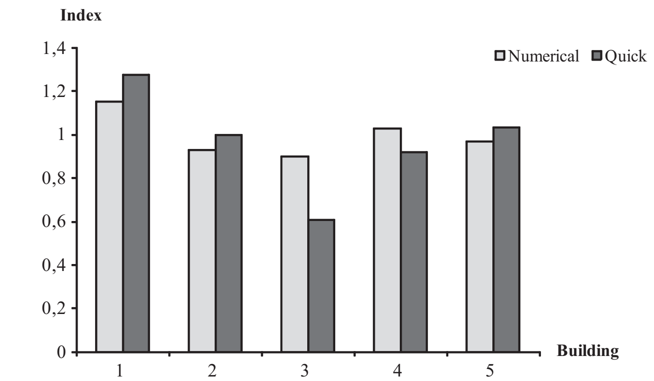


Fig. 24. Vulnerability of the study aggregate structural units in Sessa Aurunca according to the two implemented assessment procedures.

– The building n. 2 is placed at an internal position of the compound, framed within three different buildings. The surface is about 137 m^2. It develops on 2 storeys having height of 4.00 m and 3.20 m at the ground level and the first level, respectively.
– The building n. 3 is placed at the compound corner, so it shows two free sides. The surface is about 194 m^2. It is developed on 3 storeys with a constant inter-storey height of 4.00 m.
– The building n. 4 is adjacent to buildings 2, 3 and 5 and presents two free sides opposite each other. The surface is about 164 m^2. It develops on 3 storeys having the same inter-storey height of 4.90 m.
– The building n. 5 occupies an external position of the compound, so it is free on three sides. The surface is about 163 m^2. It is erected on 3 storeys: the height of the ground level is 5.40 m, while other inter-storey heights are equal to 4.30 m.

Fig. 25. The investigation pilot area (a) and the bird-eye view of the study masonry aggregate (b) of the historical centre of Torre del Greco.

Fig. 26. The aggregate investigated in Torre del Greco: (a) plan view; (b) building n.1; (c) building n.2; (d) building n.3; (e) building n.4; (f) building n.5.

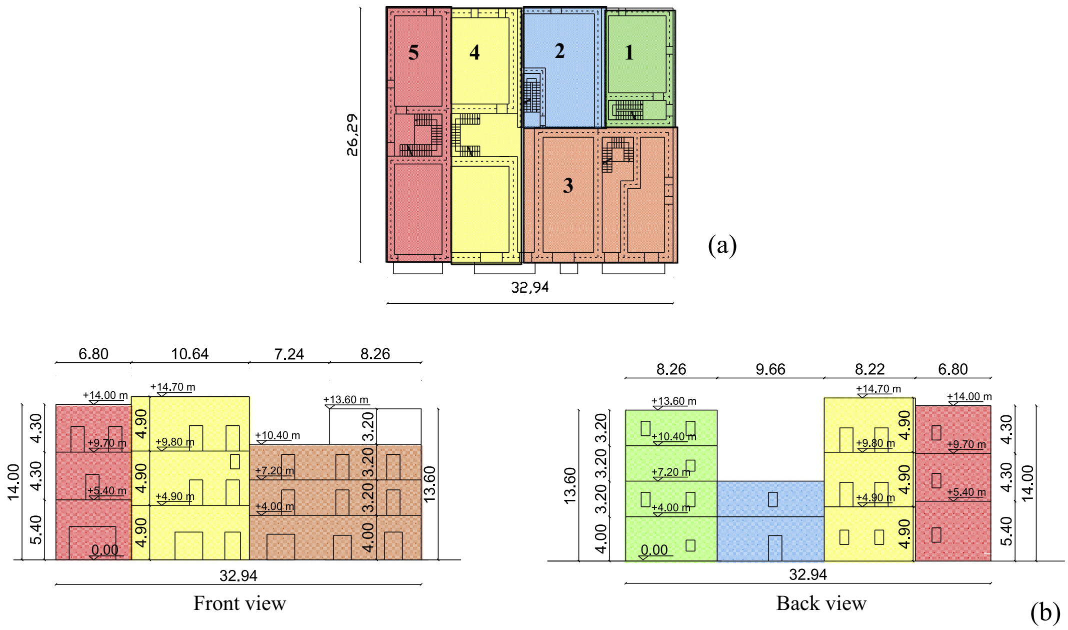


Fig. 27. Subdivision in structural units (a) and external views (b) of the masonry compound in Torre del Greco.

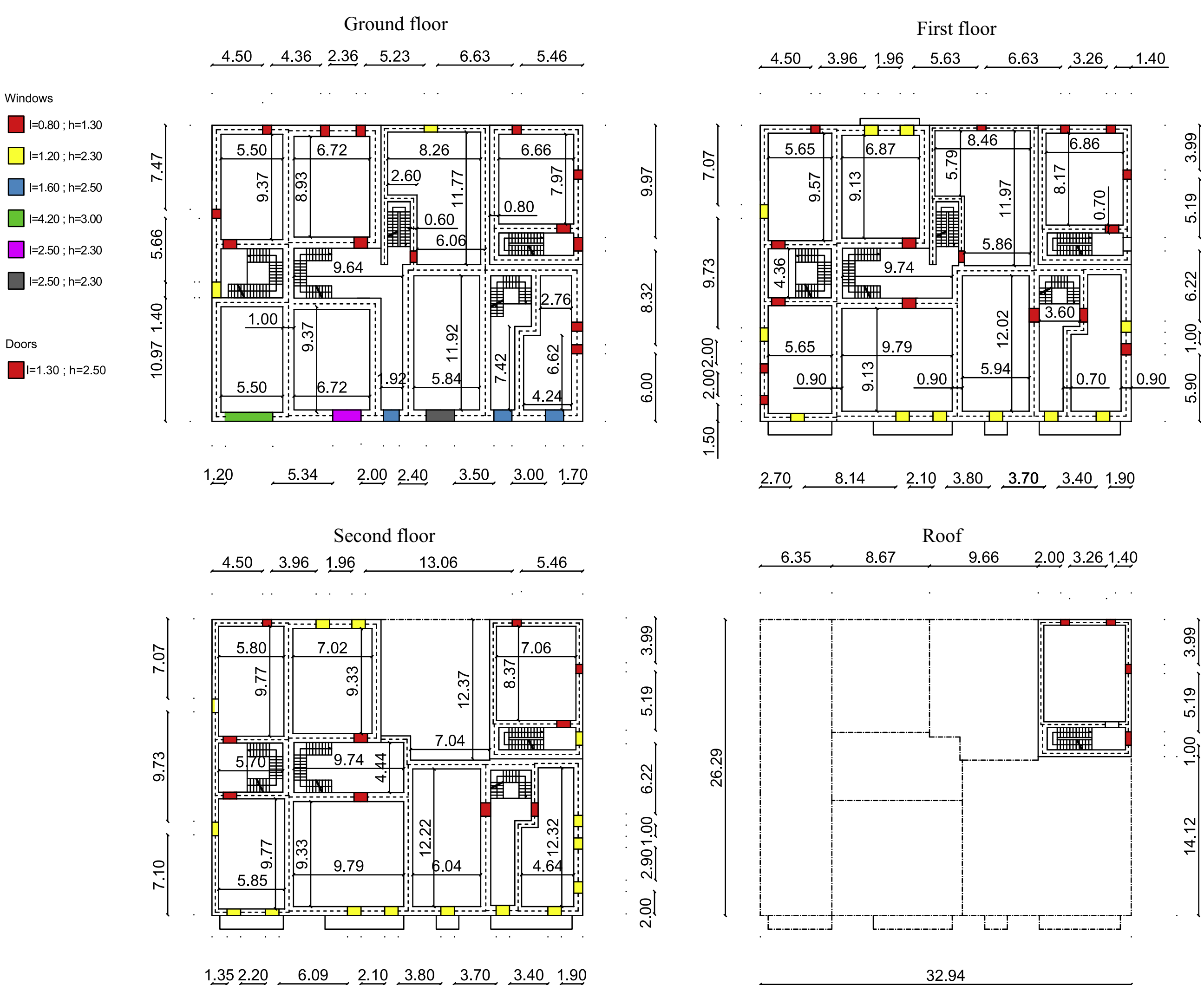


Fig. 28. Plan layouts of the masonry compound in Torre del Greco.

The plan layouts of the building aggregate are reported in Fig. 28.

Each building has a tuff masonry structure with different quality; the mechanical parameters of each masonry kind are shown in Table 6.

The horizontal structures consist of vaults without tie beams at the first level and mixed steel-hollow tile floors at the other levels, while roofing structures have a plain configuration.

3.2.2. Numerical analyses

As in the previous case study, seismic analysis of the selected masonry compound have been carried out by means of the 3MURI numerical software.

Several non linear static analyses have been performed on the aggregate numerical model (Fig. 29). Particularly, two different load conditions have been considered: forces proportional to either masses or the first vibration mode. Afterwards, these pushover curves have been compared with the demand spectra provided by the Italian code in order to obtain a vulnerability indicator.

In particular, three different earthquake design spectra have been used, the Life Safety Limit State (LLS), the damage limit state (DLS) and the operational limit one (OLS).

Such spectra are referred to the zone of Torre del Greco for ordinary buildings (class use II) with a life of 50 years and located on a ground type *C* of a topographic category T_1.

Each structural unit has been modelled both as isolated and as part of the building aggregate.

Two different directions, namely the longitudinal (*X*) and the transverse (*Y*), have been examined. The collapse mechanism of the entire aggregate is depicted in Fig. 30. The behavioural difference between a single isolated unit and the same unit in aggregate is plotted in Fig. 31.

From the same figure, it is apparent that the seismic behaviour of buildings in aggregate exhibits an increment of stiffness and resistance. In particular, the strength increase is equal to about five times the one of the same structure considered as isolated, since the aggregate condition takes into account the contribution of other structural unit walls.

From the vulnerability indexes shown in Table 7, it can be noted that the building aggregate condition reduces the seismic vulnerability of the same building considered as isolated.

Table 6
Masonry mechanical properties of the aggregate structural units in Torre del Greco.

Building	f_m (N/cm^2)	τ_0 (N/mm^2)	E (N/mm^2)	G (N/mm^2)	w (kN/m^3)
1	100	3.5	1080	180	16
2	80	2.8	900	150	16
3	110	3.5	1020	170	16
4	100	3.5	1080	180	16
5	120	4.2	1260	210	16

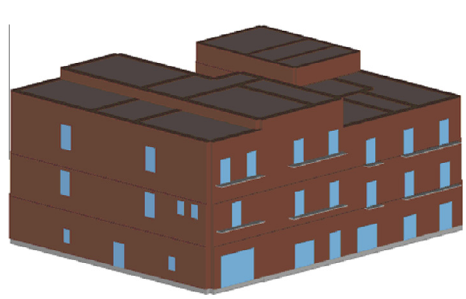


Fig. 29. FEM model of the masonry compound in Torre del Greco.

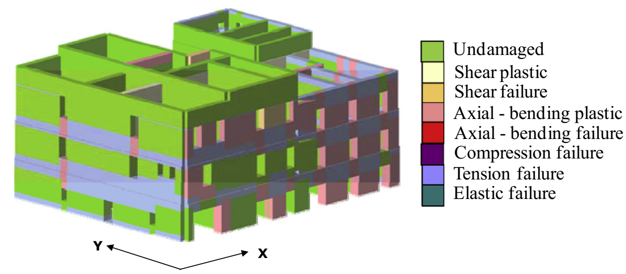


Fig. 30. Collapse mechanisms of the aggregate examined in Torre del Greco (dir. *X*).

As in the case of Sessa Aurunca, also here the vulnerability indexes $I_{v,rel}$, according to the quick procedure developed in the previous section, have been calculated for the aggregated units in order to compare the achieved results with the ones deriving from numerical analyses $I_{M,A}$.

The comparison among vulnerability index values related to the two examined methodologies with reference to the aggregate condition of buildings is shown in Table 8, where it is perceptible that in the numerical analyses the most vulnerable direction is the longitudinal one.

If we observe the mechanical vulnerability classification in this direction, it is noticeable that the building n. 1 has the greatest vulnerability index. On the other hand, the building n. 2 is less vulnerable than others. In fact, such a structure is made of a good quality of tuff masonry and has a plan vertical regularity. Furthermore, the building is protected by three taller buildings and it occupies an internal position in the aggregate. All these features reduce the seismic vulnerability, leading to the improvement of the building performance against earthquake.

In addition, if the results deriving from applying the two methods are compared, it is noticeable that the simplified technique provides the same vulnerability classification as respect to the mechanical method applied in the longitudinal direction (Fig. 32). Therefore, the comparison allows to validate the proposed quick evaluation methodology, so to consider the form as a reliable indicator of the seismic vulnerability of masonry aggregates into historical centres. This methodology does not allow to evaluate the damage grade that the building into masonry aggregates should suffer under earthquakes, but permits to identify the most vulnerable units in order to program retrofitting interventions.

3.3. Large scale application

Once the analysis method implemented for building aggregates has proved its effectiveness, it has been used to investigate the seismic vulnerability of the entire pilot area of the historical centre of Torre del Greco (Fig. 25a).

First, the seismic vulnerability of the built up area has been estimated on the basis of the Benedetti and Petrini's form and, later on, by means of the one herein proposed. Therefore, vulnerability maps according to the two methods have been depicted in the GIS environment (Fig. 33).

From figures, it is noted that the original methodology overestimates the effective seismic vulnerability of the buildings in aggregate, since it does not take into account the significant parameters typical of the interaction among adjacent constructions. In fact, it is apparent that, in the case under question, the aggregate condition makes the structural units one level less vulnerable than the same units considered as isolated. So, the aggregate condition improves the seismic performance of the single buildings compared to the isolated ones. In addition, the analysis results show that the most vulnerable buildings, whose vertical structures sustaining either

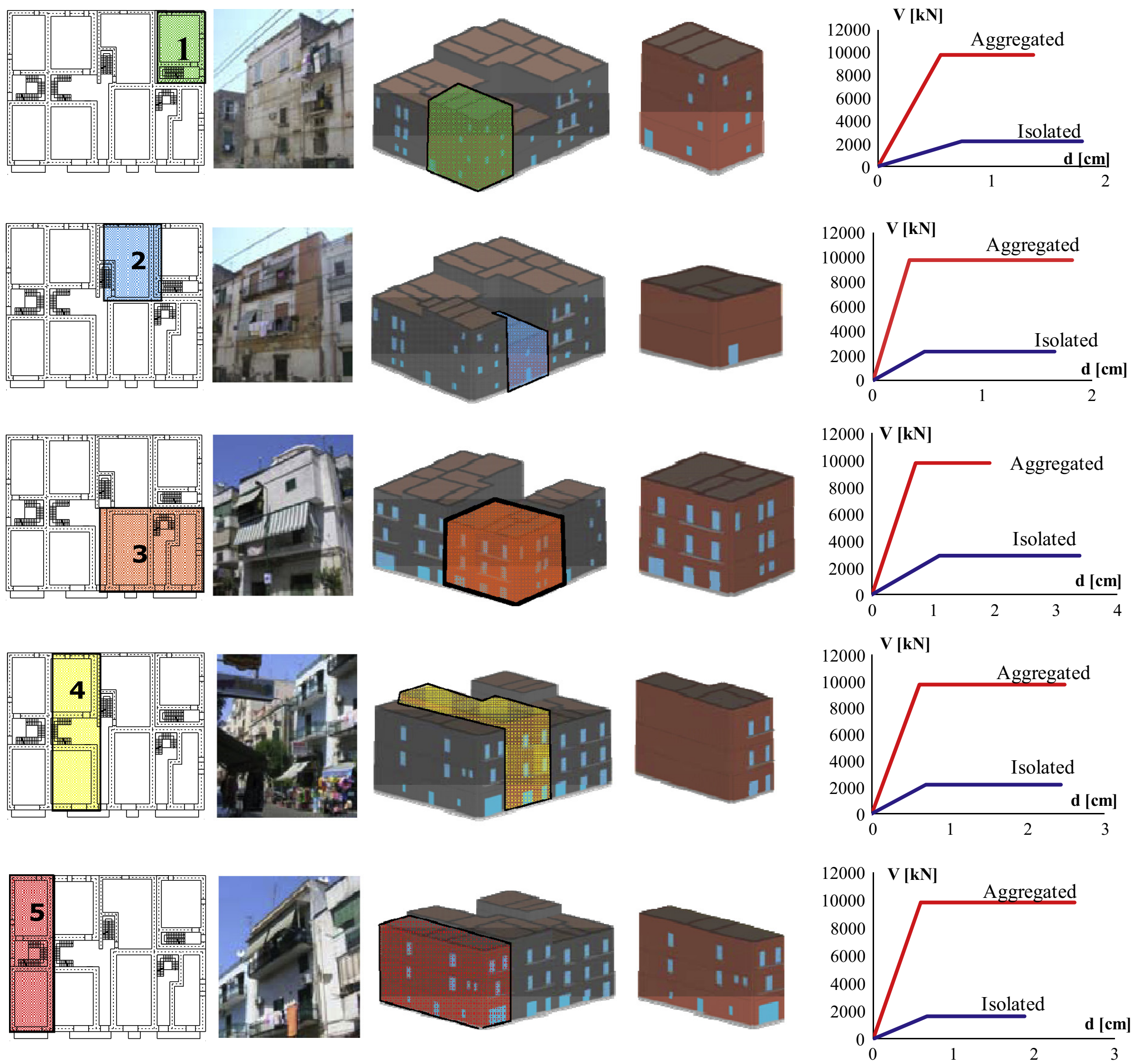


Fig. 31. Comparison among responses of isolated units and ones grouped into the study aggregate in Torre del Greco.

Table 7
Vulnerability indexes obtained by numerical analyses for the building aggregate of Torre del Greco.

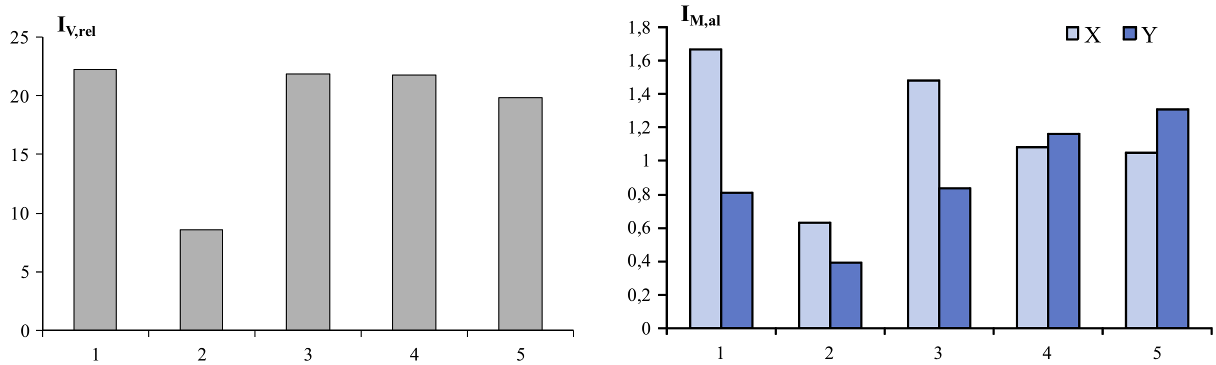
Building	Direction	Isolated			Aggregate		
		D_{max} (cm)	D_u (cm)	$I_{M,I}$	D_{max} (cm)	D_u (cm)	$I_{M,A}$
1	X	2.58	1.79	1.44	2.27	1.36	1.67
	Y	1.98	2.04	0.97	1.78	2.20	0.81
2	X	1.17	1.65	0.71	1.14	1.81	0.63
	Y	0.54	1.75	0.31	0.69	1.77	0.39
3	X	3.58	3.40	1.05	2.85	1.92	1.48
	Y	1.77	2.27	0.78	1.67	2.00	0.84
4	X	3.86	2.44	1.58	2.70	2.49	1.08
	Y	1.62	2.26	0.72	1.75	1.51	1.16
5	X	3.89	1.87	2.08	2.62	2.50	1.05
	Y	0.99	1.44	0.69	1.53	1.16	1.31

mixed steel-hollow tile floors or vaults are made of sack masonry, were built before 1919, develop on 3–4 storeys and have a poor or mediocre conservation state. Instead, regarding the aggregate condition, it is apparent that the most vulnerable buildings are the ones comprised between lower constructions and placed at either the corner or the end of the compound.

Table 8
Comparison between vulnerability indexes achieved with the two assessment methods for the aggregate in Torre del Greco.

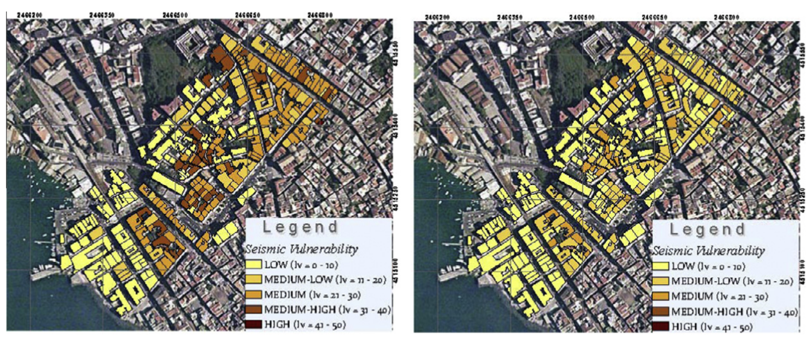
Building	$I_{v,rel}$	$I_{M,A}$	
		Longitudinal (X)	Transverse (Y)
1	22.24	1.67	0.81
2	8.58	0.63	0.39
3	21.84	1.48	0.84
4	21.75	1.08	1.16
5	19.82	1.05	1.31

Once the new seismic vulnerability indexes of buildings are known, a deterministic correlation between them and the expected seismic input has been done in order to estimate the seismic damage within the examined area. In fact, considering that the vulnerability index does not give information about the damage level caused by earthquakes, a deterministic correlation based on previous studies [22] can be employed to evaluate the mean damage grade (μ_D) of buildings on the basis of the following trigonometric expression:



(a) (b)

Fig. 32. Vulnerability indexes of the aggregate units belonging to the study aggregate in Torre del Greco according to the quick (a) and the mechanical (b) procedures.



(a) (b)

Fig. 33. Seismic vulnerability maps of the Torre del Greco pilot area developed by the Benedetti and Petrini's procedure (a) and the proposed survey form for buildings in aggregate (b).

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{S + 6.25 \cdot V_I - 13.1}{Q}\right)\right] \tag{4}$$

where μ_D is the mean damage grade, Q is the ductility factor equal to 2.3, S is the macro-seismic level ranging from 1 to 12 [23] and V_I is the vulnerability index correlated to the $I_{V,A}$ value by means of the following relationship:

$$V_I = \frac{113.66 + I_{V,A}}{619.59} \tag{5}$$

The earthquake scenario for different seismic intensities has been considered by changing the macro-seismic level S, so to obtain different values of μ_D related to different values of the seismic acceleration a_g (Table 9). In particular, the μ_D value has been calculated for 4 different seismic events, defined in the new technical Italian code and identified in Table 9 by both their return period (T_R) and the corresponding macro-seismic level.

The variation of the mean damage grade μ_D of the investigation area buildings for each of the seismic events listed in Table 9 is shown in the GIS damage maps (Fig. 34), where the damage levels foreseen in [23] can be identified.

Table 9
Earthquakes considered in the damage analysis within the Torre del Greco centre.

T_R (years)	a_g (m/s^2)	MCS scale	Macro-seismic level S (MMI scale)
101	0.83	VII	8
475	1.61	VIII	10
975	2.06	IX	11
2475	2.72	X	12

In the first earthquake considered, light cracks in very few walls and fall of both small pieces of plaster and loose stones from upper part of buildings should occur.

Instead, light and moderate damages, the latter represented by cracks into many walls, fall of large pieces of plaster and partial collapse of chimneys, should happen under earthquakes occurring after the Vesuvius eruption, having a degree lower than tectonic quakes.

Subsequently, under Life Safety Limit State (LLS) earthquake, the major part of buildings should suffer a heavy damage with large and extensive cracks in most walls and failure of roof tiles and chimneys.

Furthermore, if we observe the damage map at the Collapse Limit State (CLS), it is noticed that heavy and very heavy damages will be observed within the major part of the city centre constructions.

Finally, under an exceptional earthquake, very heavy damages, together with significant cases of collapse, should take place in the investigated built up area.

4. The case study of Poggio Picenze in the post-earthquake scenario

4.1. The L'Aquila earthquake

On April 6, 2009 at 3:32 a.m. an earthquake (M_L = 5.8 and M_w = 6.3) stroke the city of L'Aquila, the capital of the Abruzzo region with about 73.000 people, and the surrounding villages. The earthquake was generated by a normal fault, located in a valley contained between two parallel mountain located along the

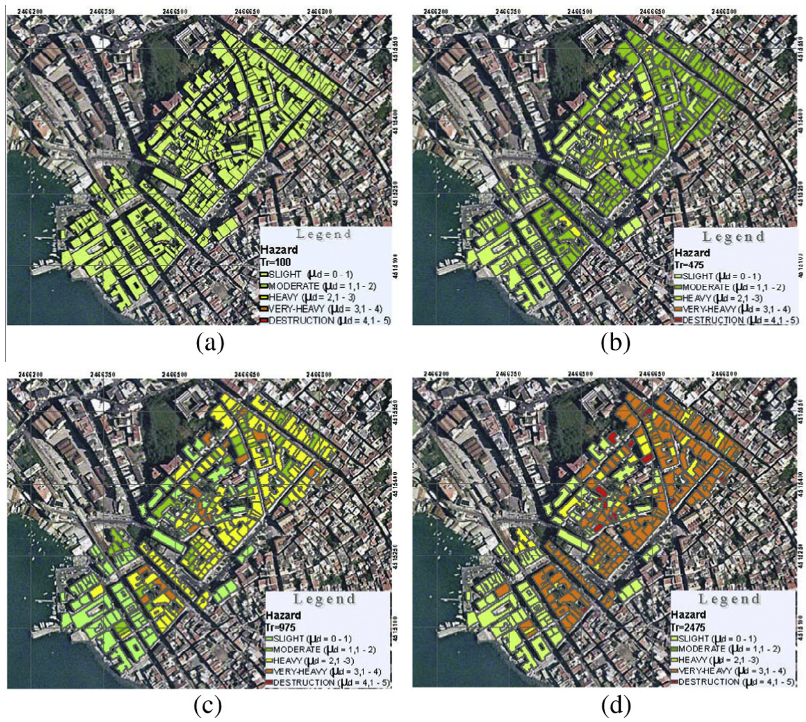


Fig. 34. Damage map of the Torre del Greco pilot area for an earthquake with $T_R = 101$ years (a), $T_R = 475$ years (b), $T_R = 975$ years (c) and $T_R = 2475$ years (d) (speedy procedure).

direction North–South (Fig. 35a) [24], with a maximum vertical dislocation of 25 cm and hypocentre depth of about 8.8 km.

It was the third main earthquake recorded in Italy since 1972, after the Friuli event (1976; $M_w = 6.4$) and the Irpinia one (1980; $M_w = 6.9$). Also, this event was the strongest among a sequence of 23 earthquakes having $M_w > 4$ and occurred between 2009 March, 30th and 2009 April, 23rd (Fig. 35b), it providing strong motion recordings from accelerometer stations placed very close to the epicentre, that is 4–5 km. Within the epicentre area, the maximum recorded horizontal and vertical acceleration components were larger than PGAs of the elastic spectra given by the Italian code.

The earthquake occurred when most people were sleeping. So, a large number of people were killed (305) or injured (1.500). Moreover, the earthquake produced the temporary evacuation of 70.000–80.000 residents and 24.000 of them remained without home [25].

A total of 81 municipalities were affected by the earthquake. The whole population of the towns listed in the official earthquake damage declaration was 60.352.

Forty-nine towns were characterised by a damage level from VI to X according to the Mercalli–Cancani–Sieberg (MCS) classification. On the contrary, damage did not exceed the MCS Scale VI grade nearly anywhere to the northwest of L'Aquila. This concentration of the damage pattern towards south probably reflects a combination of rupture directivity and seismic local amplification effects. Major details on earthquake effects on L'Aquila and its surroundings are provided in [26–28].

In the current work the aforesaid earthquake effects on the masonry aggregates of the historical centre of Poggio Picenze have

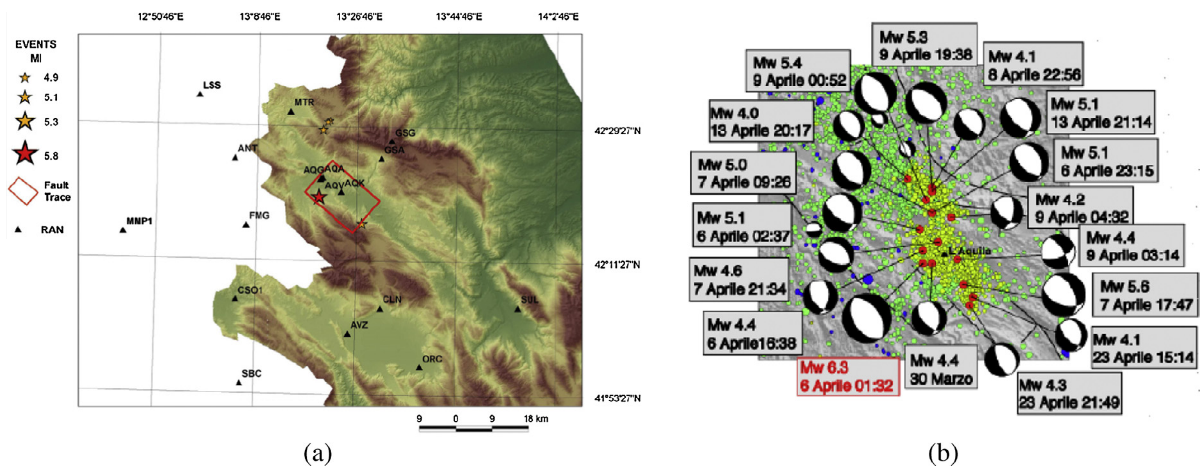


Fig. 35. L'Aquila earthquake: the fault geometry (a) and sequence of seismic events occurred in the l'Aquila district (Italian National Institute of Geophysics and Volcanology – INGV) (b).

been considered and analysed. The study is particularly interesting since it allows to make a comparison between the damaging level expected within masonry compounds according to the implemented damage assessment method for urban aggregates and the effectively occurred one after the earthquake.

4.2. The historical centre of Poggio Picenze

Poggio Picenze is a small town with about 1000 inhabitants situated on the top of a hill, 760 m above sea level, and located about 10 km to the South-East of L'Aquila along a slope at the left (north) side of the river Aterno valley.

The historical centre is the result of the process of continuous urban growth from the ancient times up to the present days. In particular, the farming town can be divided into two different urban areas (Fig. 36). The oldest nucleus was founded by Piceni around the 3rd century BC on the slope of Mount Picenze. The subsequent urban configuration developed around the medieval castle built approximately in the 1st century AC. Originally, the ancient castle had fortified walls and six towers, including a high one in the middle. Therefore, in the oldest part, the urban planning is typical of a medieval town with buildings arranged in almost concentric arrays which follow the contours.

On the contrary, the other area, which is the new one, has an irregular urban plan with some important palaces, like the mercantile Medieval House, built in the 13th century.

The entire town suffered heavy damages during the 1762 October 6th earthquake, when the castle of Poggio Picenze became unsafe and it was demolished. Ruins of this structure are still visible in the oldest part of the town.

The most important monumental buildings of the town are the three churches, namely San Felice Martire, Visitazione and St. Giuliano and two palaces, namely Galeota and Ferrari.

More information on the history and the most important buildings of Poggio Picenze are reported in [29].

Nowadays, the historical centre consists of masonry aggregates generally ranging from 2 to 3 stories. The inter-storey height is about 3.00–4.00 m for the first levels and 3.00–3.50 m for other floors. Masonry walls usually generally have constant thickness along the building height, it varying between 50 and 70 cm.

Sack stone masonry with chaotic texture inside and bad quality mortar is the typical structure for load-bearing walls, which are, in some cases, connected to each other by metal ties.

About horizontal structures, masonry vaulted ceilings largely covered the lower storey of the buildings, spanning along one or two directions. Other floor types with flexible diaphragms are made of steel beams and vaulted or flat tiles. Instead, roofing structures are often composed of double frame timber beams with clay tile covering (Fig. 37).

Most of the centre of Poggio Picenze was partially destroyed by the L'Aquila earthquake, which produced significant damages to buildings and caused the death of 5 people.

Fig. 36. Urban morphology and monumental constructions of Poggio Picenze.

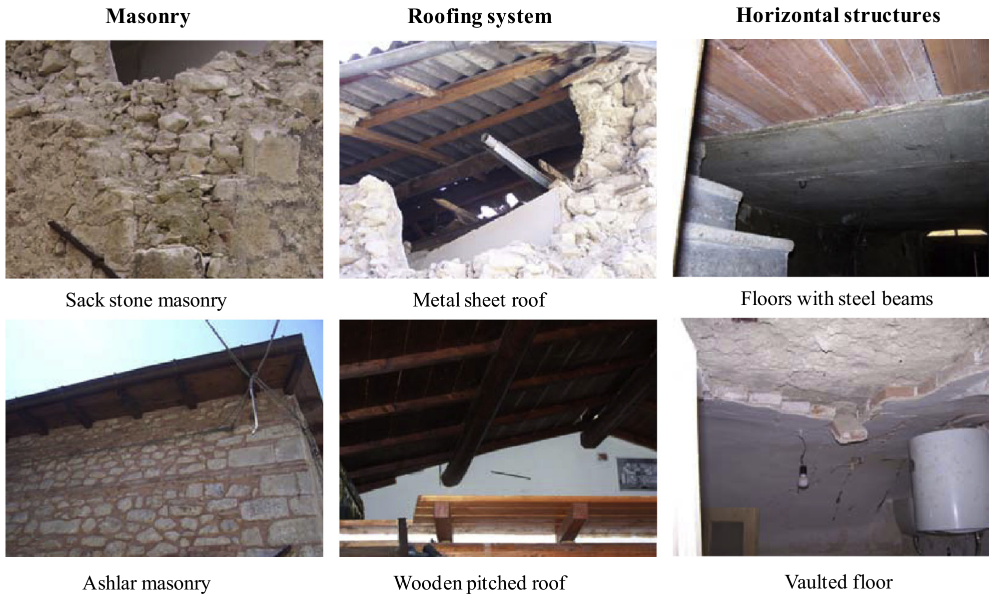


Fig. 37. Main structural features of masonry aggregates of Poggio Picenze.

During *in situ* investigations of several masonry buildings, important failure patterns into both vertical and horizontal structures have been detected. In particular, the following main collapse mechanisms have been identified (Figs. 38 and 39):

(1) *global in-plane mechanisms*, consisting of storey shear failures due to diagonal shear cracks in the masonry piers; local crushing of the masonry with or without expulsion of material.
(2) *global out-of-plane mechanisms*, characterised by either whole or partial wall overturning or walls bending collapse, generally triggered by vertical cracks at the wall corners; rocking.
(3) *other global mechanisms*, such as irregularity among adjacent structures and floor and roof beam unthreading, due to permanent deformation of either tie-beams or their anchorages; vertical cracks along the interface between two adjacent buildings.
(4) *local mechanisms*, especially consisting of lintel or masonry arch failure, local weakness, corner overturning in the upper building part caused by diagonal and vertical cracks within the masonry spandrels or cracks in the keystone arches.

4.3. Seismic vulnerability and damage assessment

The seismic vulnerability analysis of the historical centre of Poggio Picenze has been performed by means of the simplified procedure already applied to the pilot area of Torre del Greco.

The work is aimed at the extension of the quick approach calibrated on the built-up of the Campania Region to geographical zones recently affected by earthquake, where the foreseen damage derived from Eq. (4) by using the new form vulnerability index can be compared with real ones. In the current context, considering that parameter scores and weights deriving from FME analysis results performed by Authors on some masonry building aggregates located in Poggio Picenze [30] and San Pio delle Camere [31] (L'Aquila districts) were found to be coincident with previously obtained ones, the same form calibrated on building compounds in the Campania region has been consequently used. However, for the method generalisation, analyses on other building compounds located into different seismicity areas must be performed. This will allows to determine scores and weights of the form parameters independent on the site seismicity.

The simple vulnerability evaluation method has been therefore applied to 51 masonry aggregates, composed of 284 structurally

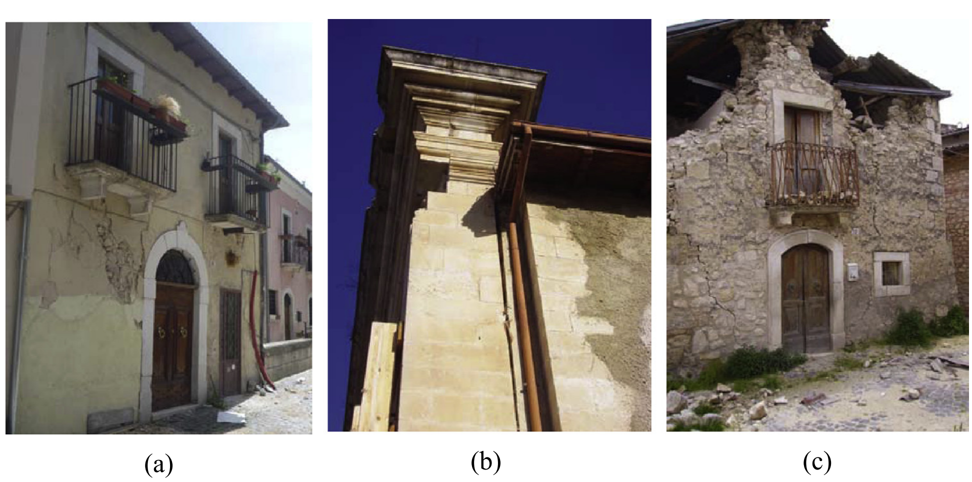


Fig. 38. Main global collapse mechanisms: (a) in-plane, shear failures due to diagonal shear cracks in the masonry piers; (b) out-of-plane overturning; (c) diagonal wedge and horizontal bending.

(a) (b) (c)

Fig. 39. Main local mechanisms: (a) lintel failure; (b) collapse of the masonry external layer; (c) vertical cracks in masonry piers.

independent units (Fig. 40). For each building, the vulnerability index has been computed by filling the form reported in Section 2.1.

Afterwards, as in the previous case study, a seismic damage analysis has been carried out by evaluating the mean damage grade according to Eq. (4). This procedure has permitted to estimate the aggregate damage level, comparing it to the effectively suffered one related to the L'Aquila earthquake seismic intensity.

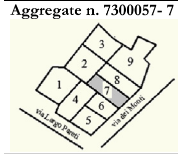
In particular, since seismic registrations have revealed that, depending on the ground nature of the site, the quake intensity range detected in Poggio Picenze was between VII and IX grade of the MCS scale, different values of the quake motion grade have been considered. In fact, the western side of the town is settled on a coarse-grained Pleistocene formation, whereas most of the historical centre is founded over the carbonate silt formation of San Nicandro, locally covered by layers of the Pleistocene gravel. This latter formation outcrops even at the toe of the hill [32]. An approximate geological SW–NE cross-section of Poggio Picenze is shown in Fig. 41.

The post-seismic damage of masonry aggregates has been estimated on the basis of their external visual inspection by assigning to each structural unit a mean damage μ_D grade ranging between 0 and 5 according to the EMS 98 scale:

– light damages: $0 < \mu_D \leqslant 1$.
– moderate damages: $1 < \mu_D \leqslant 2$.
– heavy damages: $2 < \mu_D \leqslant 3$.
– very heavy damage: $3 < \mu_D \leqslant 4$.
– destruction: $4 < \mu_D \leqslant 5$.

From visual survey it was noticed that in the castle zone, the old buildings were heavily damaged, whereas minor damage was

Aggregate n. 7300057- 7



Parameter	Classes A	B	C	D	Weights K	Scores V
1. Organization of vertical structures	0	5	20	45	1	3,72
2. Nature of vertical structures	0	5	25	45	0,25	1,16
3. Location of the building and type of foundation	0	5	25	45	0,75	3,49
4. Distribution of plan resisting elements	0	5	25	45	1,5	1,39
5. Plain regularity	0	5	25	45	0,5	4,18
6. Vertical regularity	0	5	25	45	1	0,00
7. Type of floor	0	5	25	45	1	4,65
8. Roofing	0	15	25	45	0,5	2,32
9. Details	0	0	25	45	0,25	0,00
10.Physical conditions	0	5	25	45	1	0,93
11.Presence of adjacent buildings with different height	-20	0	15	45	1	0,00
12.Position of the building in the aggregate	-45	-25	-15	0	1,5	-12,55
13.Number of staggered floors	0	15	25	45	0,5	4,18
14.Effect of either structural or typological heterogeneity among adjacent structural units	-15	-10	0	45	1,2	-2,23
15.Percentage difference of opening areas among adjacent facades	-20	0	25	45	1	-3,72
Vulnerability Index						**7,52**

(a) (b)

Fig. 40. Seismic vulnerability assessment of the historical centre of Poggio Picenze: the examined aggregates (a) and a typical form filled for a given aggregate structural unit (b).

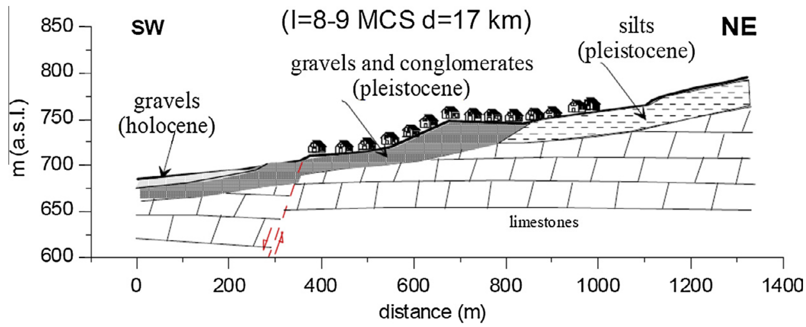


Fig. 41. Schematic geological cross-section of Poggio Picenze [32].

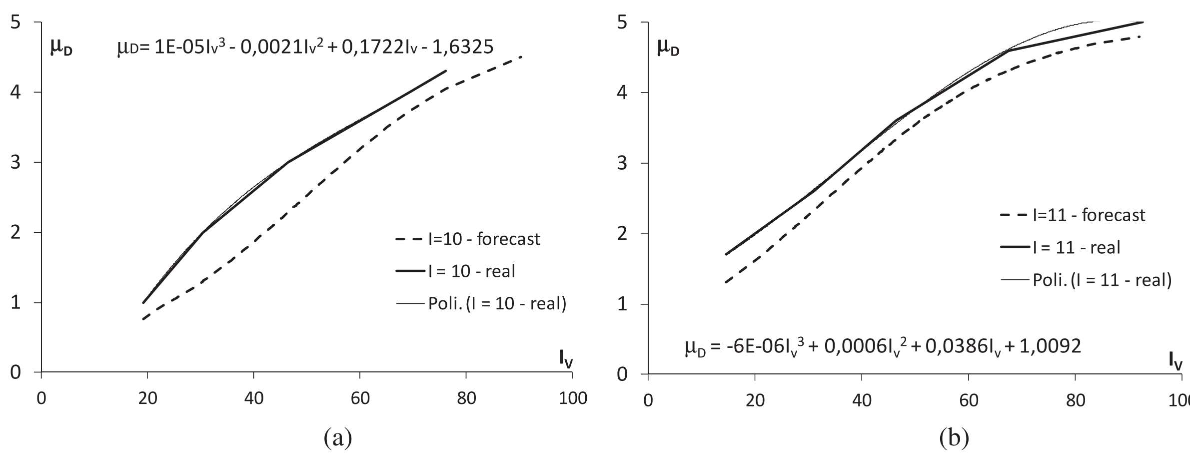


Fig. 42. Comparison between the expected mean damage grade and the occurred one within the building aggregates of Poggio Picenze: the west zone (a) and the castle area (b).

detected in the western and downhill parts of the town, where the foundation soil is based on the coarse-grained Pleistocene formation.

So, based on the damages detected in the old centre of Poggio Picenze, numerical relationships between the mean damage and seismic vulnerability and macroseismic intensity have been derived. In particular, considering the two macroseismic intensity levels detected in the centre ($I = 10$ and 11), third-order polynomial equations between the average damage degree and the expected level of vulnerability, have been derived (Fig. 42).

The curves originated from really detected damages have been then compared with those obtained by using the damage–vulnerability relationship (Eq. (4)) together with the vulnerability index calculated according to the form proposed in [17] and converted following the Eq. (5). The comparison between the real damages and the estimated ones which took place in two areas of the old town of Poggio Picenze is graphically illustrated in Fig. 42.

The comparison shows that the proposed procedure applied to the literature damage–vulnerability relationship does not provide in some few cases a conservative estimation of the building aggregate behaviour under earthquake. This result could be produced from coupling near-fault conditions with site effects induced by the complex geological structures, which further contributes to increase the complexity of the earthquake ground motion effects. However, in the whole, foreseen damages of structural units are in a good agreement with real damage levels experienced by the same units under earthquakes. Moreover, the proposed vulnerability index applied to the recommended third-order polynomial damage–vulnerability equations allows to fit very well the damage level suffered by buildings under occurred seismic event (Fig. 42).

The analysis of additional Abruzzo historical centres affected by the 2009 earthquake, as well as the careful evaluation of site effects, represent one of the future developments of the study, which will have as target the definition of a seismic damage – vulnerability law taking into account the actual seismic hazard of the investigation site. All these implemented studies will be preparatory to reach the ultimate goal of the research activity, where appropriate reversible and sustainable retrofitting measures for vertical structures [33] and floors [34] will be defined and successfully applied to masonry building aggregates.

5. Conclusions

In the present paper a simplified procedure for seismic vulnerability assessment of masonry building aggregates, typically diffused within the historical centres of many Italian towns, has been numerically calibrated and applied to some study cases on small and large scales.

First, such a simplified assessment procedure has been implemented, it being derived from the well known vulnerability form for isolated masonry buildings integrated by five parameters accounting for the aggregate conditions among adjacent units. Based on several FE analyses developed with the 3MURI calculation program, weights and scores of these new parameters have been determined. In particular, differently from the original form, also negative scores have been used, they considering the beneficial effects deriving from the aggregate condition on the seismic behaviour of a masonry building within an aggregate.

Second, the set-up procedure has been validated by performing a study in the Vesuvius area. Before, it has been applied to a single building aggregate, confirming the effectiveness of the implemented procedure, since the simplified technique has provided the same vulnerability ranking as respect to the numerical analysis one in the most vulnerable direction. Afterwards, the procedure has been applied to a wide area of the historical centre of Torre del Greco, also by employing the original vulnerability assessment survey form. It is noted that this latter methodology overestimates one level the effective seismic vulnerability of the buildings in aggregate. In addition, the analysis results have shown that the most vulnerable buildings were built before 1919, develop on 3–4 storeys and have a poor or mediocre conservation state.

Instead, regarding the aggregate condition, it is found that the most vulnerable buildings are the ones comprised between lower constructions and placed at either the corner or the end of the aggregate.

Third, in order to evaluate also the damage expected in the investigated area under four different earthquakes considered in the new Technical Italian Code, a damage analysis has been carried out on the basis of a damage vulnerability–quake intensity relationship already available in literature. The analysis results have shown that few damages will occur under low intensity earthquakes, whereas under less frequent earthquakes, damages from moderate to heavy (LLS), heavy to very heavy (CLS) and very heavy to destruction (exceptional earthquake) will be detected within the majority of the investigated buildings.

Finally, the new seismic vulnerability assessment procedure on large scale has been further applied to the historical centre of Poggio Picenze (AQ), damaged by the recent Italian earthquake (2009).

Seismic vulnerability and damage analysis of the building aggregates have been done by applying the proposed procedure. The achieved results in terms of damage have been compared with really detected damages.

The comparison among results has shown that the literature damage–vulnerability relationship does not provide an estimation on the safe side of the seismic behaviour of building aggregates. This result could be produced from coupling near-fault conditions with site effects induced by the complex geological structures of Poggio Picenze, which further contributes to increase the complexity of the earthquake ground motion effects on built-up.

For this reason, a third degree polynomial relationship between vulnerability index and mean damage grade has been derived for each of the two different historical centre zones of the town, namely the west area and the castle zone. The achieved relationships have fit very well the damage level suffered by buildings under occurred seismic event.

In conclusion, since the proposed method has been applied only to some highly seismic historical centres of Abruzzo and Campania regions of Italy, in order to have a more general procedure to be used on the whole Italian territory, additional analyses on other building compounds located into different seismicity areas will be performed as the research final step, they allowing to assess scores and weights to the form parameters independently on the site seismicity.

Furthermore, the analysis of additional Abruzzo historical centres affected by the 2009 earthquake will allow to define a seismic damage–vulnerability law for masonry building aggregates useful to individuate cases most at risk to be subjected to retrofitting interventions.

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