Non-linear static analyses on an Italian masonry housing building through different calculation software packages

Antonio Formisano, Nicola Chieffo

Abstract— This paper aims at assessing the seismic response of a typical residential masonry building located in Mirabello, a district of Ferrara damaged by the earthquake that in 2012 hit the Emilia Romagna Region of Italy. The selected case study is a masonry building representative of the class B typology, namely ordinary masonry buildings equipped with seismic protection systems. After the geometrical and mechanical characterization of the building, non-linear static analyses are carried out by using different calculation programs (Pro_Sap, 3Muri and 3D Macro) to assess the most probable seismic response of the investigated housing construction. Finally, analytical and empirical fragility curves are defined in order to obtain a synthetic parameter of the seismic damage suffered by the building under different earthquakes.

Keywords—Masonry building, Seismic vulnerability assessment, Non-linear static analysis, Calculation software packages, Fragility curves.

I. INTRODUCTION

The study of seismic risk is a useful tool for assessing the susceptibility of a sample of buildings to overcome, in a given period of time, a given seismic event with a certain entity. It can be seen as the combination of three factors, namely *Exposure* (E), *Vulnerability* (V) and *Hazard* (H). Exposure is connected to the nature, quantity and value of the properties and activities of the area that can be influenced directly or indirectly by the seismic event. Vulnerability can be defined as the intrinsic potential of buildings to suffer a certain level of damage, when subjected to a seismic event of defined intensity. Finally, Hazard is understood as the occurrence probability of the asymptomatic event with a given intensity in a specific site and it depends mainly on both the geographic position and the geological characteristics of the site where the event is expected [1], [2], [3]. These three factors are interconnected to each other, but they are influenced by uncertainties due to the indecision of involved variables, represented by buildings in this case.

Damages due to seismic events in the last decades, particularly in cities with dense urban fabrics, have raised the interest of emergency planners in estimating the seismic risk associated with future earthquakes [4]. Surveys show that collapse of buildings and other constructions during an earthquake can cause huge social, economic and human disasters [5]. Vulnerability estimation is a complex process which has to take into account and damages caused to the building, if any [6], [7], can be determined not only from the inappropriate use of design rules, but also from the deterioration of materials [8].

Residential buildings vulnerability is up to five times larger than that affecting commercial buildings and industrial ones [9]. A structure with high vulnerability is likely to suffer severe seismic damages [10]. The Global Earthquake Model (GEM) initiative [11] identified different factors like building height, age, design and construction as main indicators for the building stability.

In August 1942, the Italian Law n. 1150 was promulgated. This law, other than introducing the first urban planning discipline in Italy, provided the development of a new urban technique and conceived urban planning techniques. Post-war reconstruction led to the birth of the Italian urban planning. So, the urban built-up were placed where the people would have to resume their peaceful social activities. Through the Fanfani's Plan, a plan for housing construction was developed in Italy by means of the management of the I.N.A. CASA insurance, which developed the architectural language of Italian social houses [12].

The large number of social houses present on the Italian territory requires a thorough knowledge of the residential building heritage in order to both manage and direct the intervention strategies for the protection of housing and for the planning of appropriate structural interventions. However, it is not possible to investigate all buildings with the same level of detail. In fact, the seismic vulnerability of residential constructions is a complex task, which implies different aspects and affects structural and non-structural building components [13].

The seismic vulnerability of Italian masonry building heritage often arises from congenital structural deficiencies, such as the poor mechanical quality of masonry and the lack of connection between structural elements. These issues appear as a result of seismic events with catastrophic results. In the case of existing economic and social housing buildings, it is possible to notice that there are many structural deficiencies, such as the poor use of mortars, the use of wall hangings without connections, the use of raw bricks and the

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lack of diatones, which inevitably contribute to increase the vulnerability of structures in case of earthquakes [14].

In fact, the investigation of the seismic properties of large public buildings, as social housings, requires specific strategies to determine quick and easy classification rules. In most cases, large stocks of similar constructions are featured by different ages, constructive properties and maintenance levels [15]. Generally, focusing on such criticalities, it is noted that the most widespread dwelling type is given by two or three-storeys buildings with stretcher bond masonry walls having thickness varying from 25 to 30 cm.

The evaluation of seismic vulnerability of this type of buildings has become really considerable in the last decades due to both the high number of people living there and the frequent occurrence of earthquakes, which have demonstrated that the number of victims and the amount of economic losses depend significantly on their seismic behaviour. These buildings are generally made of masonry structures, which are the most diffused types of construction in Italy thanks to the rapidity of realization, cheapness and the employment of the usual workmanship. Nevertheless, recent destructive earthquakes that affected the Italian territory showed very high seismic deficiencies of these buildings, which are associated with a wide number of structural aspects, together with local geotechnical features [16].

On the basis of these premises, the present study focuses on the seismic vulnerability evaluation of a brick masonry housing building representative of the constructive technique of the Italian Emilia-Romagna Region, with the aim of analysing its seismic response through development of non-linear static analyses and derivation of fragility curves.

II. THE 2012 EMILIA-ROMAGNA EARTHQUAKE

The Emilia-Romagna earthquake occurred in 2012 consisted of a series of mainshocks located in the seismic district of the Po plain, predominantly in the provinces of Modena, Ferrara, Mantua, Reggio Emilia, Bologna and Rovigo, which cover a very large area of the Northern Italy (Figure 1) [17].

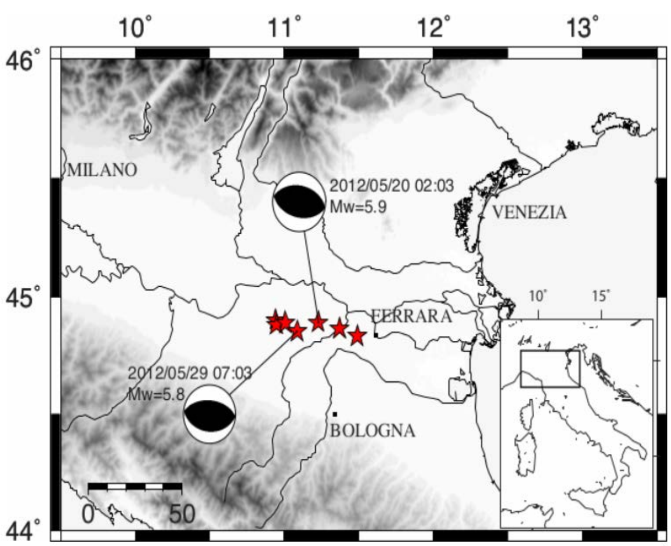
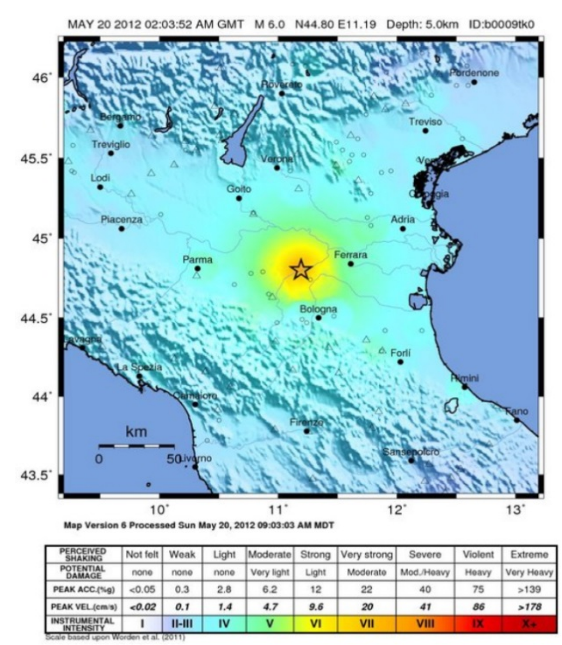


Fig. 1. The epicentral area of the 2012 May seismic sequence [17].

The series of main earthquakes that hit the Emilia-Romagna Region of Italy on 2012 May 20th (02:03 UTC MI = 5.9–13:18 UTC MI = 5.1) and on May 29th (07:00 UTC MI = 5.8–10:55 UTC MI = 5.3) involved an area of about 9.000 km^2, traditionally considered as a low seismic hazard region [18]. These shocks were followed by several seismic aftershocks with variable magnitudes smaller than those of previous quakes. Nevertheless, another 5.1 magnitude shock was reported throughout North Italy on 2012 June 3rd at 21:20:43 GMT (19:20:43 UTC), with epicentre in Novi of Modena. Peak accelerations recorded by the Mirandola accelerometer during the strongest shocks of May 20th and May 29th were respectively equal to 0.31g and 0.29g, values that, according to the existing seismic hazard maps, correspond to a seismic event in the same area with a return period of about 2500 years. The spatial distribution of epicentres extended for about 50 km along the E-W direction, while hypocentre depths ranged from 0 to 40 km. Furthermore, the majority of hypocentres were located within a depth of 10 km.

Moreover, the ShakeMap of Figure 2 displayed the immediate shake levels and reported the peak values recorded by accelerometers and seismometers, mainly provided by both the National Accelerometer (RAN) [19] of the Italian Department of Civil Protection and the National Seismic Network (RSN) of the Italian Geophysics and Volcanology National Institute (INGV) located in the earthquake area. These maps adopt the same colour scale for comparable intensities by providing, with the judgements “weak”, “strong” and “severe”, an initial idea of earthquake ground shake levels.

In practice, with “weak” (blue colour) the shaking was just felt. With “strong” (green-yellow colours) the shock was felt in a very distinct and strong way, causing in some cases slight damages (e.g. cracks in the plasters). Finally, when the shaking is “severe” (orange-red colours), very strong quakes occurred producing large damages and collapses when orange and red colours are reached, respectively (Figure 2).



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	<0.02	0.1	1.4	4.7	9.6	20	41	86	>178
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Fig 2. The ShakeMap of 2012 Emilia-Romagna seismic events [19].

The seismic sequence affecting Emilia-Romagna was undoubtedly a casualty, because this area was classified, according to the MPS04 model, as a low-medium seismic zone with predictive ground acceleration (PGA) values, having a probability of exceeding 10% in 50 years, ranging from 0.10g to 0.15g (see Figure 3) [20].

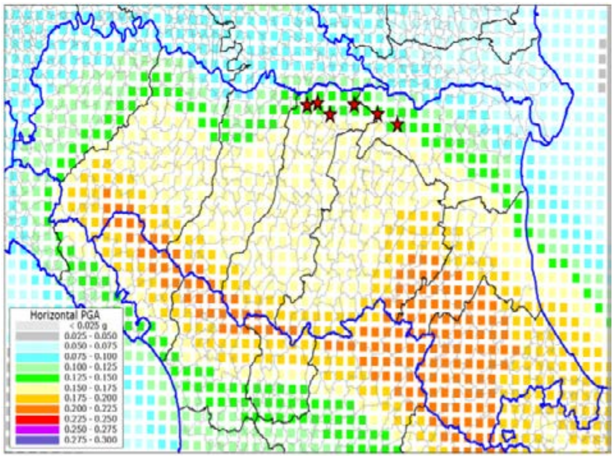


Fig. 3. Seismic hazard map of the Emilia-Romagna Region [20].

III. CENSUS OF HOUSING BUILDINGS AND PRESENTATION OF THE CASE STUDY

Through surveys performed by the Italian Statistical Institute (ISTAT) [21], it is achieved a national census that provides information on citizens, buildings and housing. In particular, in the "14th General Census of Population and Housing" provided by the ISTAT in 2001 [22], information on the number of storeys, as well as on the characteristics of residential (and in some cases even non-residential) buildings, were provided [23]. Through this census, it was possible to classify the built-up in 448 municipalities hit by the 2012 Emilia-Romagna earthquake so to have an idea about prevailing structural typologies, number of storeys and construction age (Figure 4).

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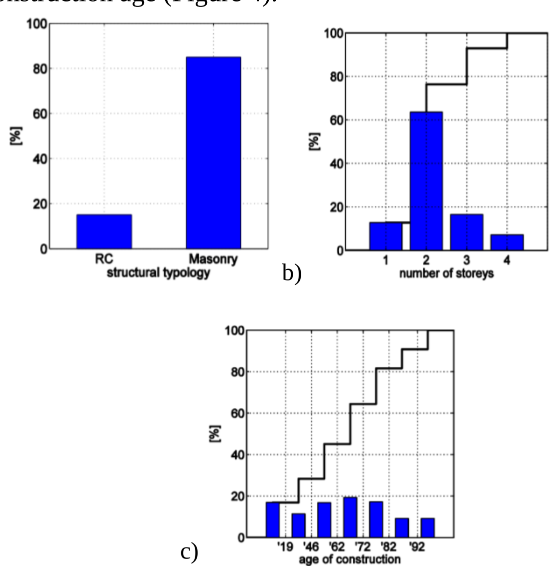

Fig. 4. Building classification in the areas affected by the 2012 Emilia-Romagna earthquake: a) structural typologies; b) number of storeys and c) age of constructions [23].

Focusing on ordinary masonry buildings, constructions located in the Emilia-Romagna Region are usually made of brick masonry with regular shape elements and horizontal mortar rows. This is detected in the case study herein presented, dealing with a building with residential use located in the municipality of Mirabello, a town in the district of Ferrara. It has a rectangular plan developed on a surface of 18.22 m x 8.80 m and covered by a double pitched roof with a maximum height from the ground of about 13.60 m (Figure 5).

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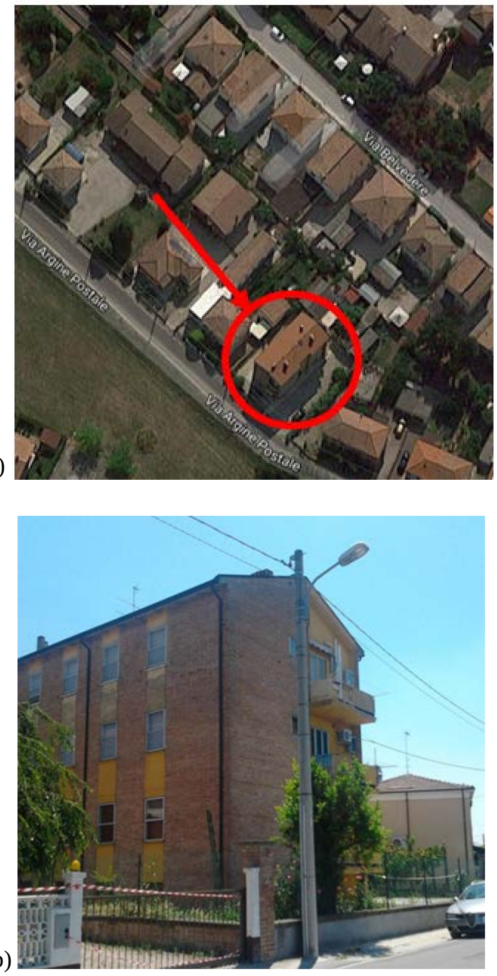
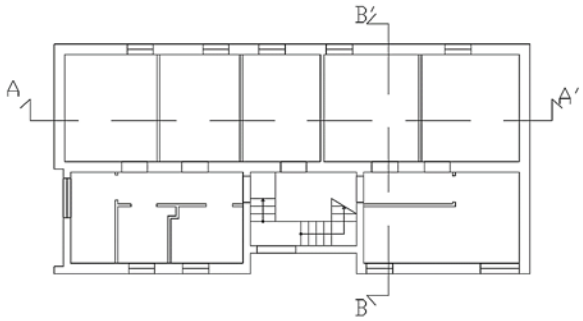


Fig. 5. a) Bird-eye view and b) street view of the housing building under investigation.

The housing building under study is composed of a basement floor, three storeys and a not habitable attic floor. It accommodates n. 6 real estate units, each of them including a cellar in the basement floor and a car box not integrated in the building structure. Plan layouts at the different storeys and the longitudinal sections of the building are depicted in Figure 7.

The main vertical structure consists of brick masonry walls with thickness of 42 cm (heart bond) along the building perimeter and thickness of 30 cm (stretcher bond) and 12 cm (layer course) in the basement floor. Moving from the ground floor to the upper storeys, the heart bond walls are reduced to stretcher bond ones, maintaining the external alignment and the central alignment for perimeter and interior walls, respectively.



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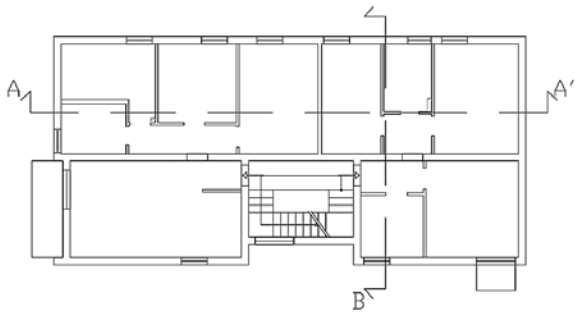

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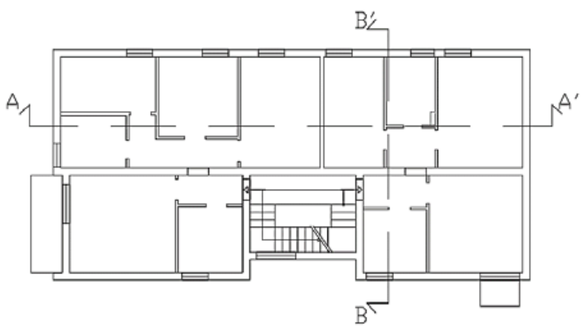

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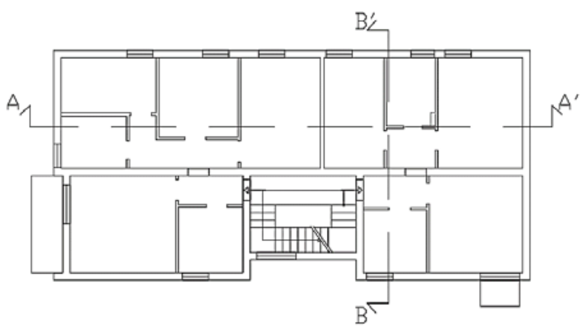

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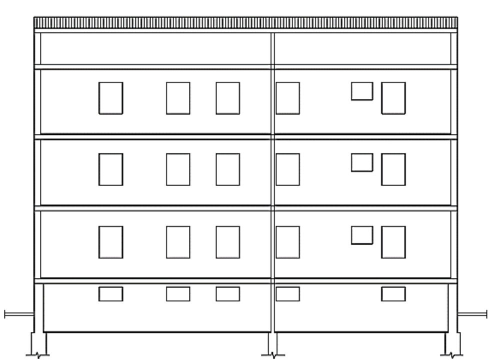

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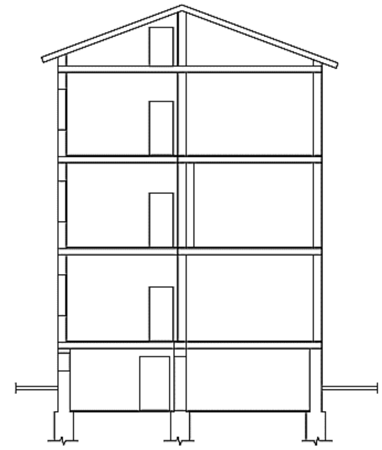


f)

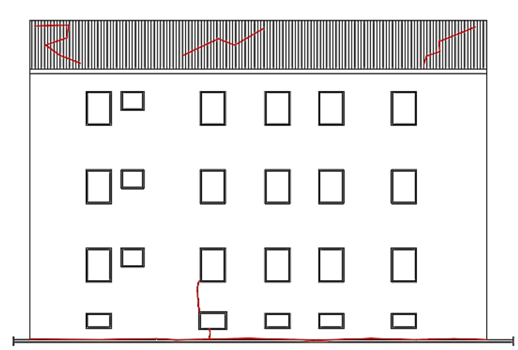
Fig. 6. Building geometrical drawings: a) basement floor; b) first floor; c) second floor; d) third floor; e) longitudinal section A-A'; f) transverse section B-B'.

The intermediate horizontal structures are made of mixed reinforced concrete-tile floors without rigid slabs, considering the building age and the constructive technique used at the erection time of the construction. The same structure type is used for roof, which is covered by a layer of tiles. The different storeys are connected vertically by a masonry staircase. The foundation structure is made of brick masonry stones forming inverse *T* cross-section beams placed about 1.80 m below the ground. The partition walls are made of hollow bricks.

After the 2012 Italian earthquake, the building was classified in class B (temporary unusable) by the investigators of the Regional Evaluation Nucleus acting under the directives of the Italian Civil Protection Department.

The seismic damages detected were: - failure of masonry chimneys with the consequent collapse of some portions of the roof; - settlements of the foundation structure along the short sides of the building, resulting in the consequent detachment of all perimeter sidewalks; - numerous medium size cracks in the bearing masonry walls and in the masonry corners; - slight detachment between floors and masonry walls; - detachment between the roof and the masonry walls below; - detachment of roof covering elements and subsequent infiltration of rainwater causing damages to both real estate units and staircases.

Some damages detected in the housing building after the 2012 seismic sequence are shown in Figure 7.



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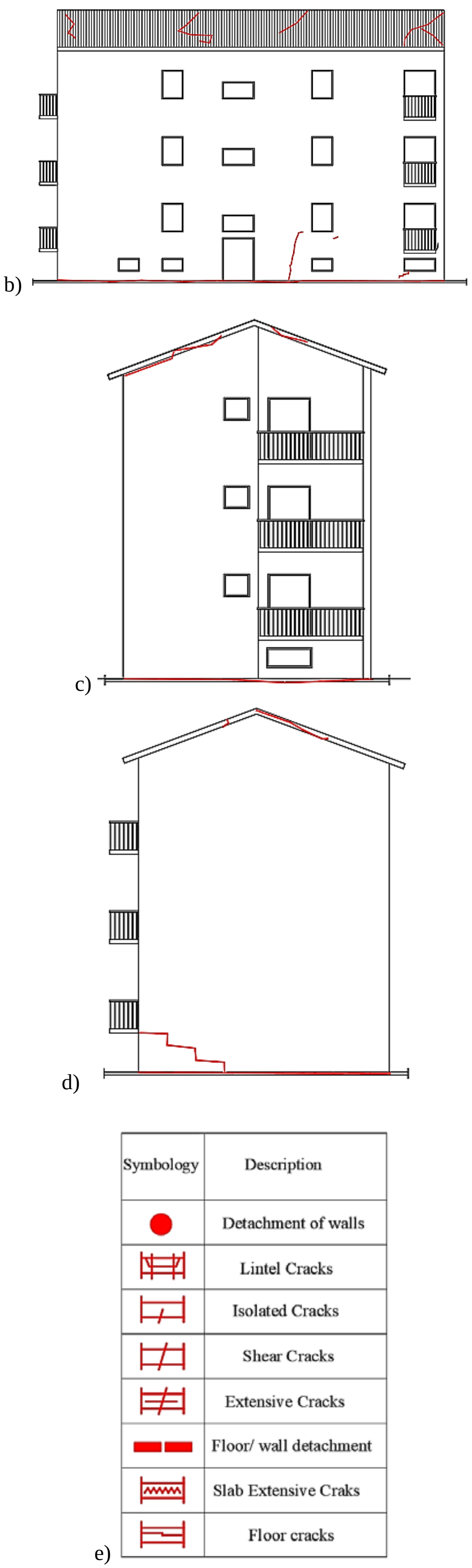


Fig. 7. Damage state: a) North - East facade; b) South - West facade; c) North - West facade; d) South - East facade; e) legend of cracks.

IV. NUMERICAL ANALYSES

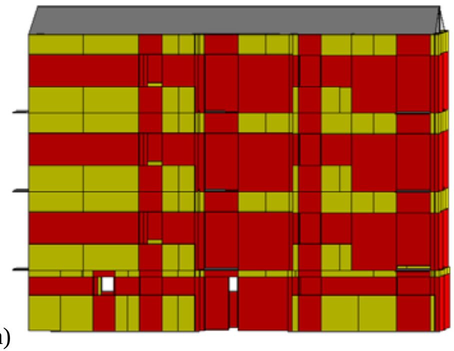
Non-linear static analyses have been performed by using 3 different software packages, namely Pro_Sap, 3Muri and 3D Macro, indicated, for the sake of simplicity, in the following with PRS, 3M and 3DM, respectively. About numerical modelling, according to the geometrical survey performed, the height of the basement floor is assumed to be 232 cm, while the heights of other levels are 300 cm. Mixed reinforced concrete – tile floors with thickness of 20 cm have been considered as horizontal structures at each level. The foundation structure has been modelled through an inverse *T* beams system placed below the walls. Mechanical properties of masonry have been defined considering a limited knowledge of the structure, that is assuming a LC1 knowledge level, with a confidence factor FC=1.35, according to the Italian New Technical Codes for Constructions (NTC08) [24]. For numerical analyses a subsoil category "*C*" and a design spectrum referred to the Life Safety limit state have been considered. Dead and variable loads applied at the different structural levels, as well as partial safety factors for gravity loads combination at the Ultimate Limit State, are shown in Table 1.

Table 1: Loads acting on the structure.

Static load	Intermediate floor	Roof	Balcony	Partial safety factor
	[KN/m^2]	[KN/m^2]	[KN/m^2]	-
G_1	3	3	3	1,3
G_2	2	1	1	1,3
Q_k	2	0.5	4	1,5

Concerning the structural models, each software schematises the structure through a series of macro-elements interconnected to each other, in some cases leading towards the definition of the so-called "equivalent frames" [25]. These macro-elements allow to simulate the seismic behaviour of masonry structures, providing all the information required for their static linear analyses. Generally, for non-linear analyses in the most of the used programs for masonry constructions, the macro-elements are transformed into beam elements, leading to 3D framed structures similar to those typical of RC and steel buildings. According to this modelling approach, masonry walls are reduced to linear vertical elements called piers, which are connected to the spandrels, horizontal linear elements acting as beams or trusses depending on their capacity to resist or less tensile actions, through rigid nodes, which represent zones generally undamaged by seismic actions [26].

The PRO_SAP software, developed by the 2S.I. company, considers in the modelling phase of the equivalent frame, composed of sections with shear and bending rigidities, two infinitely rigid materials: the first, having the same masonry density, used for rigid parts of masonry columns, and the second, with null specific weight, used for rigid parts of beams [27]. The FEM models used by this calculation software for the study structure are shown in Figure 8.



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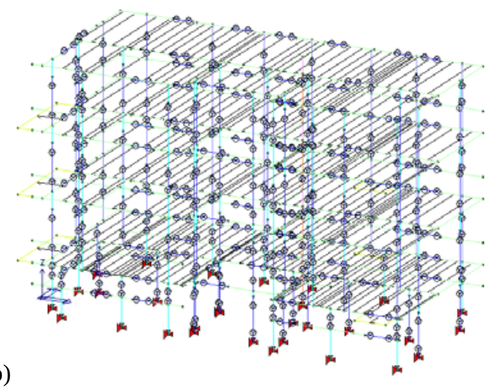

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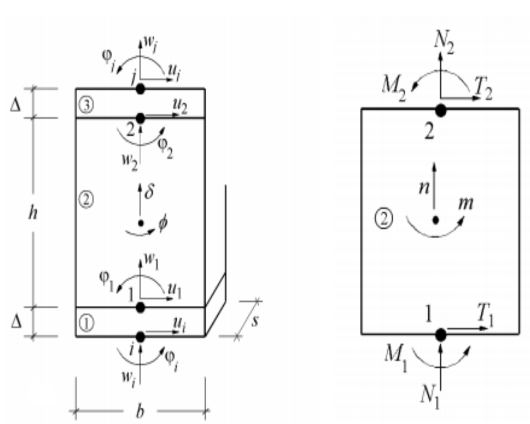
Fig. 8. a) Macro element and b) non-linear models of the inspected building developed with the Pro_Sap program

The 3Muri software, developed by the S.T.A DATA company [28], uses macro-elements to generate the three-dimensional model of the structure, which is then automatically transformed into an assemblage of 3D equivalent frames to perform pushover analyses.

The typical macro-element used for static linear analyses is schematised with the kinematic model reported in Figure 9a. The 3D model of the examined housing building, where it is apparent that masonry walls are modelled through a mesh of masonry piers and spandrels, is depicted in Figure 9b.



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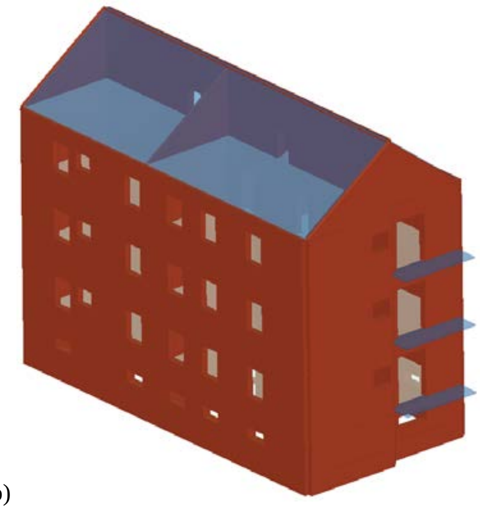

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Fig. 9. a) The macro-element kinematic model and b) the 3D building model with macro-elements setup through the 3Muri software.

The last software used is 3D Macro, developed by the Gruppo Sismica company [29]. The proposed model consists of masonry walls structured with quadrilateral schemes connected to contiguous walls by an array of springs distributed along the perimeter of the four-sided shape. Each quadrilateral scheme, composed of infinitely rigid sides connected to each other by hinges, is stiffened by two diagonal springs arranged according to the St. Andrew's cross configuration. It is connected to the ground by means of a bed of springs. Such a modelling approach is able to take into account the major failure mechanisms, namely shear diagonal and sliding shear, of a wall portion subjected to horizontal actions in its own plane.

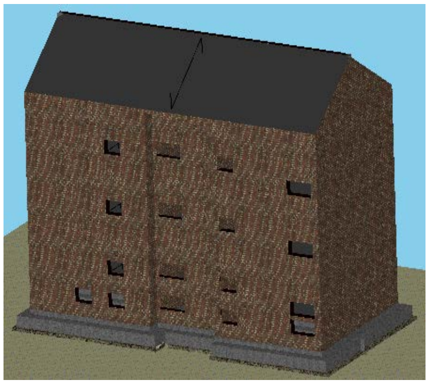
The macro-elements computational model used by 3D Macro for numerical modelling of the inspected housing building is illustrated in Figure 10a.

After this modelling phase, the software automatically transform macro-elements into the above described quadrilateral elements connected to each other by distributed springs (Figure 10b).

Non-linear static analyses have been performed for each software in the two main directions (X and Y), taking also into account the effect of accidental eccentricities. The worst analysis results in terms of SDoF capacity curves are shown in Figure 11.

From the comparison of results it is seen that the three used software calculation packages provide curves with a very similar stiffness (see Figure 12) and only small variations in terms of maximum strength (base shear).

On the other hand, the results show large uncertainties about the estimation of yielding and ultimate displacements, as shown in Figure 13.

a) 

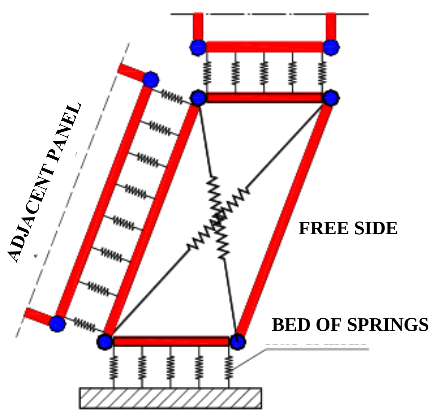


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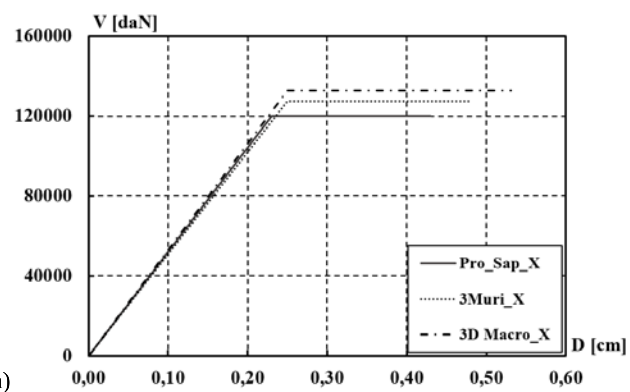
Fig. 10. a) 3D model and b) modelling of an end panel by means of the 3DMacro software.



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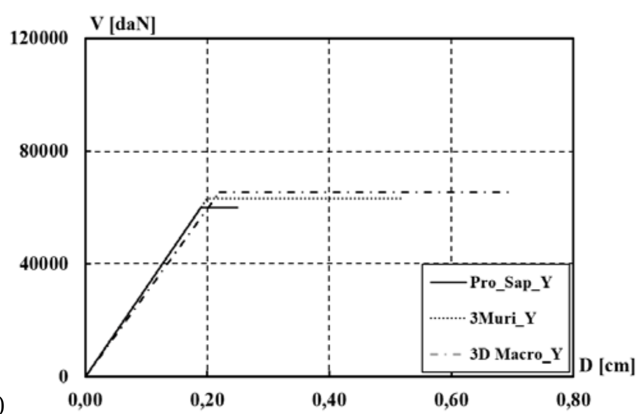

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Fig. 11. The building capacity curves in a) X and b) Y directions.

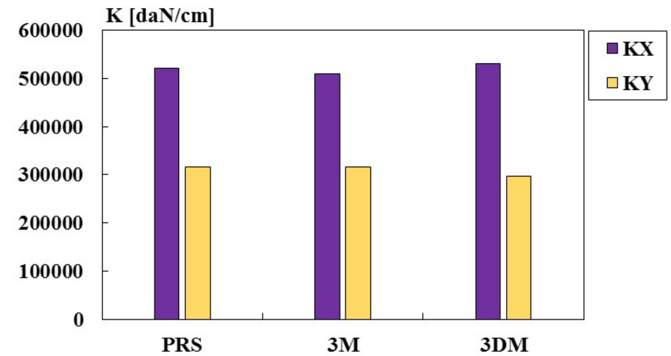

Fig. 12. Comparison among pushover analysis results in terms of initial stiffness.

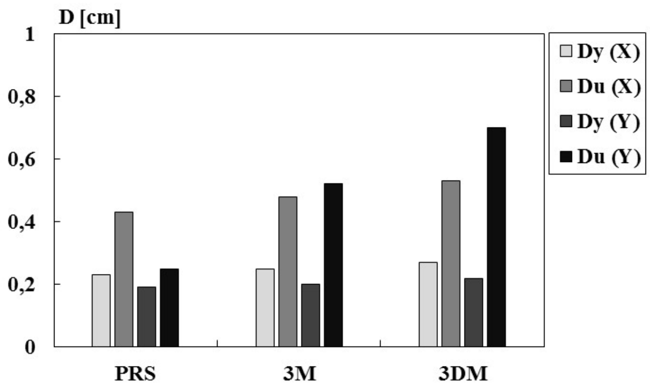

Fig. 13. Comparison among pushover analysis results in terms of estimation of displacements.

With reference to the results obtained by the PRO_SAP (PRS) software, it is possible to evaluate the percentage variation in terms of stiffness and displacements achieved with the other software packages used.

In particular, in terms of stiffness, in direction X, 3Muri (3M) provides a decrease of 2.3%, while 3D Macro (3DM) gives an increase of 1.7%. In direction Y, 3M provides a percentage increase of 0.02%, while 3DM gives a decrease of 5.8%.

On the other hand, with respect to PRS yielding displacements, in directions X and Y there are percentage increases of 8.7% and 5.3% with 3M and 17.4% and 15.8% with 3DM, respectively. Instead, regarding the PRS ultimate displacements, percentage differences achieved in directions X and Y are equal to 11.6% and 108% with 3M and 23.3% and 180% with 3DM, respectively.

From analysis results in terms of displacements, it is therefore noticed that PRS provides the smallest values, that appear to be too conservative. Contrary, 3DM gives the largest displacements, which are of the same magnitude order of those offered by 3M, which provides intermediate displacement values.

However, given such uncertainties in terms of displacements, the seismic vulnerability assessment has been carried out by considering a mean capacity curve, obtained by assuming the average values provided by the three programs in terms of strength, yielding displacement and ultimate displacement (Figure 14).

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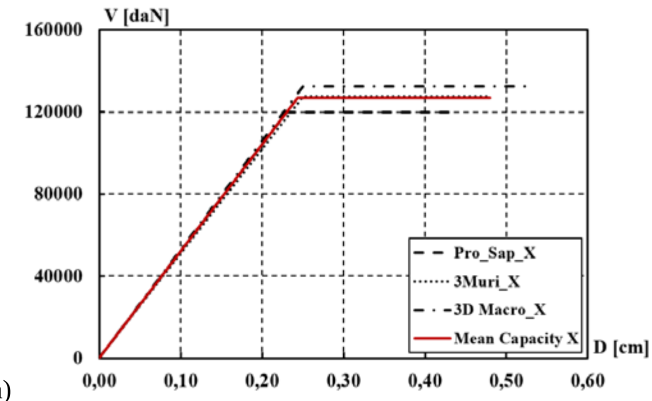

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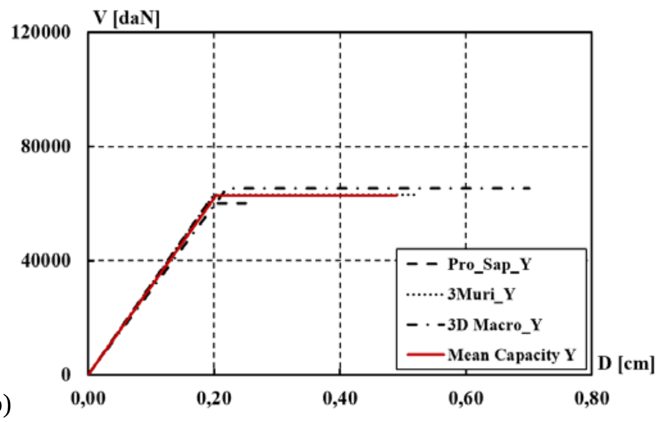

Fig. 14. Mean capacity curves in a) X and b) Y directions.

The Capacity Spectrum Method (CSM) is the most applied procedure to analyse, starting from a non-linear static analysis, the seismic response of a structure. It was developed by Freeman [30] by means of a graphical procedure which compares the structure capacity with the earthquake ground motion demand.

The capacity of the structure is represented by the base shear - displacement curve obtained from the pushover analysis. The base shear and top displacement values are converted into the spectral accelerations and spectral displacements of an equivalent Single Degree Of Freedom (SDOF) system, respectively. These spectral values give rise to the so-called capacity spectrum.

The demand of the seismic ground motion is represented by the earthquake elastic spectrum, providing the so-called demand spectrum.

Capacity spectrum and demand one are reported together in the Acceleration Displacement Response Spectrum (ADRS) format, where the line passing from the axis origin and having the same slope of the capacity curve stiffness represents the vibration period of the structure [31]. The intersection between the capacity spectrum and the demand one provides an estimate of the displacement demand which the structure should be subjected to under the considered earthquake.

Based on the above considerations, the seismic checks of the structure along the main analysis directions are shown in Figure 15.

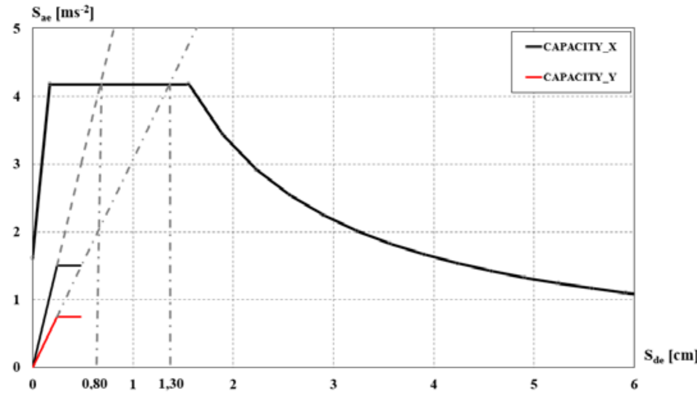

Fig. 15. Seismic checks by means of the CSM.

From the analysis of results it is noticed as in both directions the structure is not verified under the Life Safety limit state earthquake considered by the Italian code NTC08 in the site of Mirabello. In fact, in the direction X the capacity displacement is 0.48 cm, while the demand one is 0.8 cm, whereas in direction Y, the capacity displacement and the demand one are 0.49 cm and 1.30 cm, respectively. Therefore, the seismic vulnerability indices, intended as the ratio between demand displacement and capacity one, in the directions X and Y are respectively I_{Vx}= 1.66 and I_{Vy}= 2.65. This confirm the highest vulnerability of the inspected housing building along its transverse direction.

Later on, for the assessment of damages suffered by the structure under different grade earthquakes, the fragility curves are computed. They are continuous functions that express, for each level of ground motion, the likelihood of reaching specific levels of damage. In particular, the fragility function gives the probability that a generic Limit State (LS) is reached when a value of the Intensity Measure (IM), generally represented by the PGA, is given [32], [33].

The fragility function is thus defined by two parameters, namely IM_{LS} and β_{LS}. The mean intensity IM_{LS} can be obtained from the statistical analysis of data from either damage observation after earthquakes (empirical method) or by a mechanical model (analytical method), which is considered representative of the average seismic behaviour of buildings belonging to a particular class. On the other hand, the dispersion β_{LS} depends on the different contribution of uncertainties in the seismic demand, such as the uncertain definition of the Limit State threshold and the capacity variability of buildings that belong to the considered vulnerability class.

In the current work a comparison between two methods for defining fragility curves is made. The first method, called discretised method, is based on the functional linkage degree between average damage (μ_{D}) and Peak Ground Acceleration (PGA). The degree of average damage is estimated for each step of acceleration by making the ratio between the seismic demand and the structure capacity. The second method, instead, defines the fragility curves by means of a lognormal function characterised by mean value and standard deviation. According to the FEMA P-58 code developed in 2012, the total uncertainty is estimated to be 0.6 [34].

Afterwards, damage thresholds, which different damage states (D) correspond to, are defined. Four damage states are usually considered: D1 (slight), D2 (moderate), D3 (near collapse) and D4 (collapse) [35]. Based on these considerations, fragility functions are plotted, as shown in Figure 16.

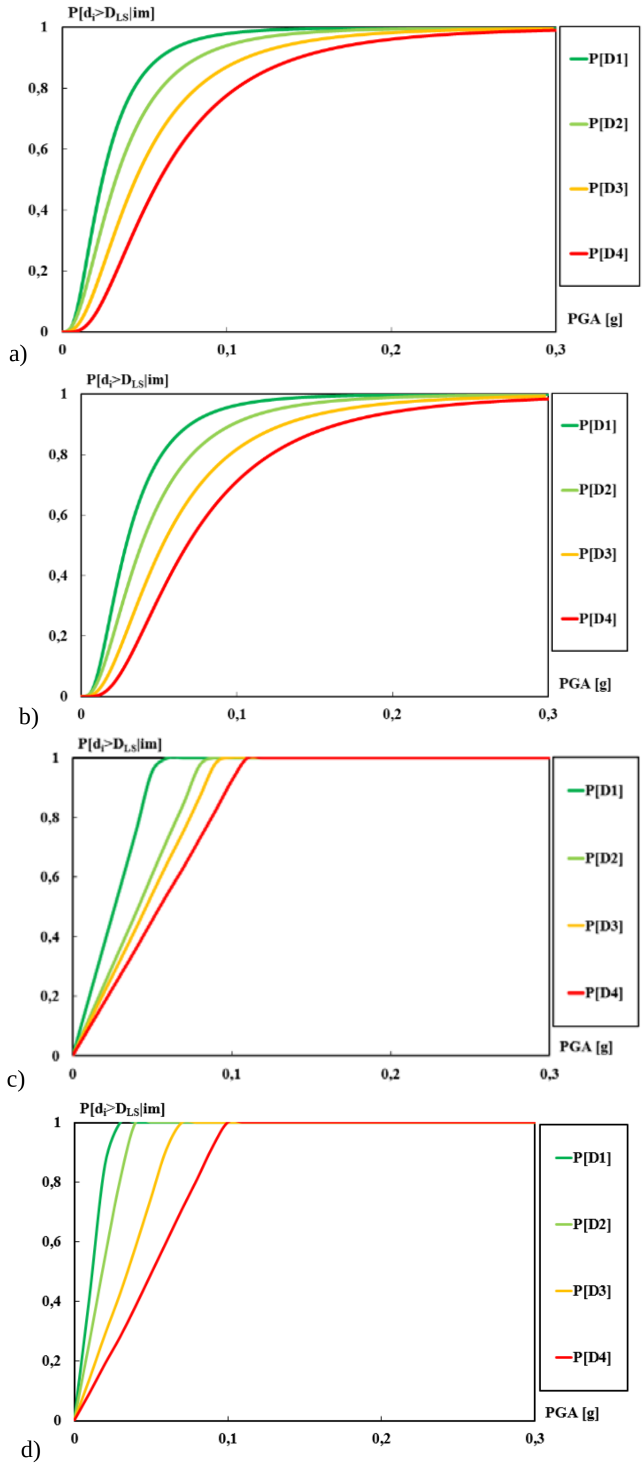

Fig. 16. Fragility curves: lognormal distribution in *X* (a) and *Y* (b) directions and discretised method in *X* (c) and *Y* (d) directions.

Subsequently, once fragility curves obtained through the two applied methods are derived, the results in terms of expectation of damage, or rather the probability of exceeding a certain level of damage due to an assigned seismic acceleration, are put in the same diagram and compared to each other. In particular, it is possible to investigate the methodology providing the most conservative definition of the expected damage level (Figure 17).

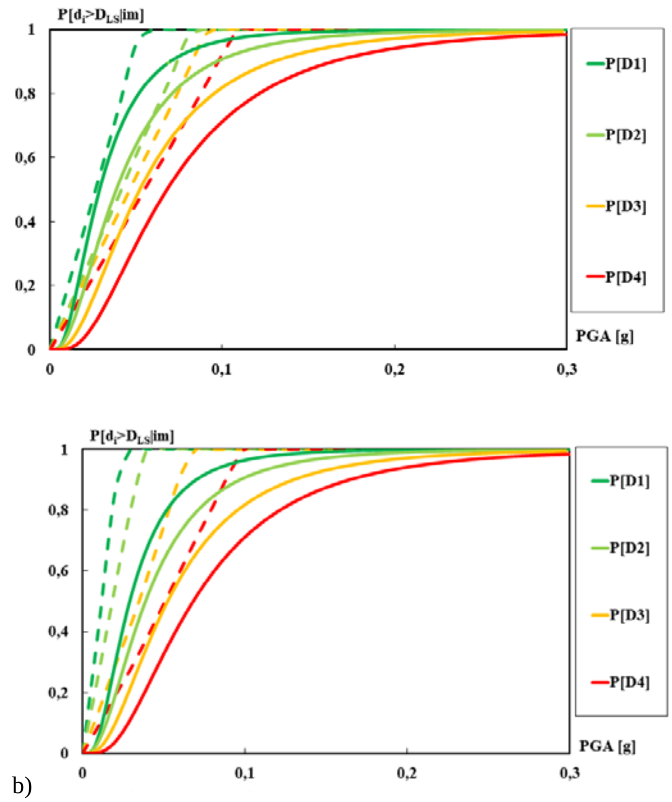

Fig. 17. Comparison among fragility curves in a) X and b) Y directions.

From the comparison of results it is seen how the procedure for defining the fragility curves by means of the discretised procedure leads towards overestimation of damages. In fact, considering the value of a_g = 0.16g, which is the spectral acceleration value for the site of interest, in *X* direction it is noticed that, with reference to the D4 damage state (collapse), the discretised method provides a likelihood of occurrence of 100%. Contrary, with the lognormal distribution method, there is a probability of occurrence of 80%. Therefore, the discretised method overestimates the damage of 25% with respect to the lognormal distribution method. In the *Y* direction, always with reference to the same limit state, the discretised procedure overestimates the results of about 22% in comparison to the other method.

V. CONCLUSIONS

The study herein presented has allowed to assess the vulnerability and seismic damage of a masonry residential building in the centre of Mirabello, which was damaged by the 2012 May seismic sequence. Through the use of three different calculation software packages, namely Pro_Sap, 3Muri and 3D Macro, the capacity curves of the buildings have been gotten and compared to each other. The comparison has shown that about the same stiffness is predicted by the three programs, whereas only slight differences in terms of ultimate base shear have been noticed. On the other hand, in terms of ultimate displacements, it has been seen that Pro_Sap provides too conservative results, while the largest values are given by 3D Macro. Therefore, to take into account the uncertainties of obtained results, the average capacity curves of the building in the two main analysis directions have been considered by taking the mean values of ultimate base shear and yielding and ultimate displacements of curves derived from the three calculation programs. Subsequently, by using the Capacity Spectrum Method, the structure vulnerabilities in the two analysis directions have been evaluated. The results obtained have provided vulnerability indices of 1.66 and 2.65 in X and Y directions, respectively, so highlighting the highest building vulnerability along its transverse direction.

Finally, for the assessment of damages suffered by the building under different grade earthquakes, the fragility curves have been defined according to two different approaches, namely a discretised method and a lognormal distribution based procedure. Considering the maximum PGA occurred in Mirabello at the Life Safety limit state (0.16g), from the comparison of achieved results it has been detected as, for the collapse (D4) condition, the discretised method provides the highest damage forecasts, with damage exceedance probabilities greater (25% and 22% in directions X and Y, respectively) than those achieved with the lognormal distribution based procedure.

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