SEISMIC FRAGILITY ASSESSMENT OF UNREINFORCED MASONRY AGGREGATE BUILDINGS IN ROW

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Abstract

Masonry buildings in Italy represent the majority of the historical and cultural heritage and they very often represent the result of an unbridled urban growth, due to the need to fill all the possible urban spaces. For this reason, aggregate masonry buildings have been generated over the years, allowing the aggregation of different inhomogeneous structural units, interacting under seismic action. The present work is focused on the study of unreinforced masonry aggregates in row, considering structural units along the aggregate with geometrical differences each other, generated starting from the medium values of the variables used to study masonry aggregates with identical structural units in row. Once some distributions of those variables were defined, a set of aggregations of different structural units in row is generated, following the rules of the Response Surface (RS) statistical method, based on the definition of a statistical model expressing a response parameter (for instance the spectral acceleration corresponding to the attainment of the Life Safety Limit State) as a function of a set of variables. The RS model is calibrated through numerical data obtained by non-linear static analyses, with reference to buildings whose geometrical and mechanical properties are varied in prescribed distributions. A group of registered accelerograms was used to consider the variability of the seismic action, for defined ranges of magnitude and epicenter distances referred to the considered earthquakes. Finally, the data obtained from the simulations were used to plot the fragility curves, by applying full Monte Carlo simulations, in order to obtain the conditional probability of failure for different values of the peak ground accelerations (PGA_C). The results showed considerable differences in the fragility if different directions of the seismic action are considered, due to the geometrical properties and the arrangement of the resisting masonry walls of the aggregate and to the torsional effects deriving from the aggregation of structural units in row. Moreover, the various structural units along the aggregate exhibit different seismic fragility, according to their position in the masonry aggregate in row.

Keywords: Masonry aggregate; Variability; Response Surface; Non-linear analysis; Fragility curves

1. Introduction

The study of the seismic fragility is one important goal for civil engineers, using fragility curves as a tool to assess seismic risk. The structural capacity of buildings is required to calibrate these curves, and sometimes it is affected by structural and geometrical variabilities and uncertainties [1]. Aggregates made of old masonry structures represent a very common structural typology in many cities of the world. The structural units along the aggregates are characterized frequently by structural and geometrical heterogeneities. But, urban growth led to the formation of many historic centers with similar structural typologies. Thus, the present study aims to assess the seismic vulnerability and fragility of masonry building aggregates observed in the city of Bologna, in Italy, chosen as prototypes having similar characteristics and representative of some classes of buildings, starting from the idea that buildings located in similar geotechnical conditions and with similar geometrical and structural properties are expected to have similar seismic performances. The structures are unreinforced masonry aggregates in row, made by structural units having geometrical differences each other. The purpose is to analyze how the considered differences affect the seismic response in the global behavior of the aggregate structures, evaluating which are the parameters most influencing the seismic behavior of the various structural units sited in different positions along the aggregates.

The effect of some variabilities and uncertainties involved in the problem is taken into account through the Response Surface (RS) statistical method, where the expected value of a response parameter (the peak ground acceleration (PGA_C) corresponding to the attainment of the life safety limit state) is approximated through a polynomial function of a set of chosen variables. The Response Surface model [2, 3] is calibrated through numerical data obtained by non-linear static analyses and used to determine the fragility curves, by applying full Monte Carlo simulations.

In order to consider the possible variability of the seismic action in the site and the uncertainty related to the definition of the ground motion, a group of registered accelerograms referred to past earthquakes was used, also considering two different and orthogonal directions of the seismic action, to analyze the differences on the global seismic behavior due to the different geometrical configurations of the resisting masonry walls in the two directions.

2. Seismic Fragility

The evaluation of the seismic fragility is a key issue in seismic engineering because the use of fragility curves allows to evaluate the probability of exceeding certain levels of damage of a structural system, when exposed to an assigned seismic action. Fragility analysis requires the estimate of the structural capacity and the structural demand of the considered system. Thus, the use of the limit state function allows to determine if the failure is reached when the difference between the structural capacity (C) and the structural demand (D) is less than or equal to zero. In this work, the quantities C and D are expressed in terms of spectral acceleration and do not depend on time [4, 5, 6]:

$$g = S_{a,\mathrm{C}}(\mathbf{x}) - S_{a,\mathrm{D}} \tag{1}$$

where $S_{a,\mathrm{C}}$ is the spectral acceleration corresponding to the attainment of the life safety limit state and $S_{a,\mathrm{D}}$ is the spectral demand acceleration. The structural capacity depends on the properties of the structure, which are defined in statistical terms. In this way, the Response Surface (RS) statistical model is used to represent the structural capacity $S_{a,\mathrm{C}}$ ($\mathbf{x}$) in Eq. (1). The data to calibrate the polynomial function of the RS model are estimated by means of non-linear static analyses (push-over) in order to approximate the dependence of $S_{a,\mathrm{C}}$ to $\mathbf{x}$. Finally, a range of values of the spectral demand acceleration is chosen and the generated values of the spectral capacity acceleration are used to solve Eq. (1) using full Monte Carlo simulations in order to develop the fragility curves [7].

3. The Response Surface

The response surface method is based on a probabilistic procedure defining a statistic model expressing a response parameter as a function of a set of variables [8, 9]. In this work the response parameter is represented by the spectral acceleration corresponding to the attainment of the life safety limit state [10], the variables (called *factors*) are random and they are defined in prescribed ranges, using a priori selected normal distributions.

In order to reduce the number of random variables in the RS, they are divided in two groups: explicit x_E and implicit x_I variables [8, 11]. The first ones are accounted for explicitly as random variables in the RS, while the latter are considered implicitly [1]. Considering *N* observations, i.e. the results of numerical simulations, it is possible to approximate the expected response value of a response parameter *Y* through a polynomial function of a set of explicit variables. In matrix form, the definition of the polynomial function is defined as [1]:

$$Y = \mathbf{X}\beta + \sum_{j=1}^{r} \mathbf{Z}_j \delta_j + \varepsilon \tag{2}$$

where Y is the vector collecting the response values $Y=[Y_1,\ldots,Y_N]$, β is the vector of the regression parameters, $\mathbf{X}$ is the design matrix containing the values assumed by the explicit variables and ε is the vector of the statistical errors. Each vector δ_j, which represents a different implicit variable, is divided in b_j blocks; a block is a homogenous group of simulations affected by the same value of the variable δ_j. Finally, each $\mathbf{Z}_j$ is a Boolean matrix, with values equal to 1 if the corresponding block is associated to the considered observation or otherwise equal to 0. According to Searle et al. [8] and Khuri and Cornell [11], the implicit variables δ_j and the errors ε are normally distributed with zero mean and constant variance ($\sigma^2_{\delta j}$ and σ^2_{ε} respectively).

In order to obtain the data necessary to calibrate the Response Surface, in the present study, a set of non-linear static analysis (push-over) was performed. The Design of Experiment Theory [6, 7] allows to define the criteria necessary to establish the number of simulations (each simulation is referred to a different analysis) and the region of interest (typically the region is cuboidal) for the explicit variables influencing the response.

In this work all the variables are given by a normal distribution and, in order to simplify the construction of the region of interest, a non-dimensional form for the variables was used. These *coded variables* x_i are defined by Eq. (3) [7]:

$$x_i = \frac{2X_i - (X_{iL} + X_{iH})}{X_{iH} - X_{iL}} \tag{3}$$

where X_{iL} and X_{iH} are two chosen values of the *i-th* variable. In this work these values were selected as the mean value ± 1.5 times the standard deviation. In this way a 2^k factorial design was generated (k is the number of the explicit variables), where each variable x_i can assume two different values, $x_i = \pm 1$. In order to obtain 2^k set of values of the variables and 2^k simulations, all the possible combinations of the two values of k variables were considered. But the 2^k factorial design is a first-order design, i.e. it is suitable to polynomial models of order one. In order to use polynomial models up to order two, the Central Composite Design (CCD) [8], was used. It is a second-order design and it is constituted by: (i) a 2^k factorial design ($x_i = \pm 1$); (ii) a number n_0 of center points ($x_i = 0$); (iii) two axial points on each axis of the variables, at distance equal to α from the center ($x_i = \pm \alpha$), with $\alpha > 1$.

4. The aggregation of structural units in row

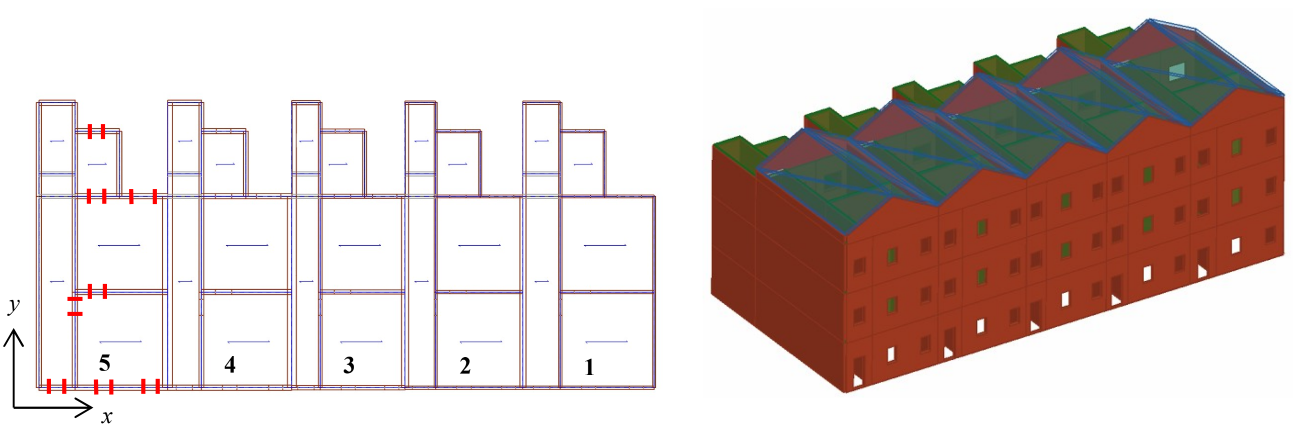
Starting from the study of the isolated structural unit (ISU), performed by Battaglia et al. [10] and that of the aggregations in row of 5 identical structural units (AS) performed by Battaglia et al. [12], the main purpose of this work is the assessment of the seismic fragility of masonry aggregate structures made by structural units with geometrical differences each other. It is very common to find aggregations of different, but similar, masonry structural units in row in the Italian historic centers, commonly due to the urban growth characterized by the development of similar construction techniques in the same historic period.

Thus, this work is focused on the study of unreinforced masonry aggregates in row, considering structural units differing each other, generated starting from the medium values of the variables used for RS models studied in Battaglia et al. [10, 12]. The same methodology was applied: once the simulations of the RS model were defined, a set of non-linear static analyses was performed using the software "TreMuri" [13], considering two orthogonal directions of the seismic action; afterwards, the data obtained from the analyses were used to plot the fragility curves.

The purpose is to analyze how the considered differences affect the seismic response in the global behavior of the aggregate structures, evaluating which are the parameters most influencing the seismic behavior of the various structural units sited in different positions along the aggregates.

4.1 The structure

The masonry aggregate buildings object of this work were generated starting from the same structural units analyzed in Battaglia et al. [10, 12]: three-storeis masonry buildings, with clay brick walls, hollow-core concrete slabs and pitched roof made by timber beams. Fig. 1 shows the structural plan of the ground floor and a tri-dimensional view of the masonry aggregate, referring to a model chosen as example of one of the row-aggregations of different structural units analyzed in this work. In Fig. 1(a) the differences in terms of thickness of the walls and distance between the walls in *x*-direction are highlighted; the thickness of the masonry walls between two adjacent buildings is equal to the summation of the two thicknesses, to ensure that the aggregate structure is a combination of the structural units. Moreover, in order to better understand the geometry of the masonry panels, the positions of the openings (the same in all the Units, from 1 to 5) are highlighted in Fig. 1(a) with red lines. The definition of the simulations to calibrate the Response Surface model is carried out by means of the choice of the explicit and implicit variables, explained in the following paragraph.



(a) (b)

Fig. 1 – (a) Plan of the structural ground floor and (b) model of the 3D masonry aggregate with different structural units.

4.2 Selection of the variables

In this work it was assumed the choice of the same explicit variables defined for the RS model in Battaglia et al. [10] (mean masonry shear strength (τ) and mean distance between external walls in *x*-direction (d)), except for the mean slab elastic modulus considered in this application in a deterministic way with a fixed value, as it was shown that it does not affect the seismic response. The two explicit variables (τ and d) were defined with the same normal distributions and the same assumption of the values adopted in Battaglia et al.

[10], as showed in Table 1. The other structural masonry properties (masonry compressive strength (f_m), masonry elastic (E) and shear (G) modulus) are direct function of τ according to the values reported in Table C8A.2.1 of the Italian Code [14].

Table 1 – Definition of the normal distributions adopted for the explicit variables.

Variable (X_i)	Distribution	μ	COV	σ
τ	N	0.063 (MPa)	0.2	0.012
d	N	6.90 (m)	0.1	0.690

As already mentioned, the values of the variables are selected following the Design of Experiment Theory to calibrate the RS model. Therefore, using Eq. (3) and according to the Central Composite Design, the total number of a group of simulations is 11; it is repeated several times, according to the definition of the blocks for the implicit variables. Table 2 gives the definition of the group of 11 simulations, setting the coded variables as x_1 = τ and x_2 = d.

Table 2 – Definition of the group of 11 simulations using the coded variables x_i.

x_1	1	-1	1	-1	0	0	0	1.33	-1.33	0	0
x_2	1	1	-1	-1	0	0	0	0	0	1.33	-1.33

Regarding the implicit variables, the uncertainty of the seismic action (δ_{sis}), the uncertainty of the distance between the walls in *x*-direction (δ_d) and the uncertainty of the thickness of the walls (δ_s) are chosen in this work.
As far as δ_{sis} is concerned, a group of 48 accelerograms was used in order to consider the variability of the seismic action (more details in Section 6). For each of the 11 simulations, according to the Design of Experiment Theory, 2 accelerograms were associated to the factorial region and 1 accelerogram was associated to the axial and central points. Thus, each group of 11 simulations is related to 3 blocks δ_{sis} and it is repeated 16 times (the total number of blocks δ_{sis} is 48).

δ_d and δ_s represent the implicit variables defining the different geometrical properties of the structural units along the aggregate. δ_d is the uncertainty of the distance between the walls in *x*-direction (d) and it allows to define a different value of d for each structural unit along the aggregates. For each of the 5 values of d a normal distribution was defined and 8 groups of 5 values (5 as the number of the structural units in row) was randomly selected in the distributions: in total 40 groups δ_d were selected, defining 40 different aggregate configurations.

As far as δ_s is concerned, the thickness of the walls *s* was considered as implicit variable and the variation of its values depends on the variation of the values of the distance d: each aggregate configuration was generated in such a way as to have greater *s* with greater d. The values of the thickness *s* are the same used in Battaglia et al. [12]: in this application they were divided in 5 groups of 3 values (Fig. 2), from s_1 to s_5, and for each generated aggregate configuration, 5 random values of *s* (5 as the number of the structural units in row) were obtained from the 5 groups of *s*. In total 40 groups δ_s were selected.

Thus, each aggregate configuration was generated selecting every time 5 random different values of the distance (d) from the correspondent distribution of d and 5 random values of *s* from the correspondent group of *s*, in such a way as to have the correspondence between d_i and s_i (i assumes values from 1 to 5). 40 aggregate configurations are obtained with the selections of d and *s* for each structural unit (1 to 5) along the aggregate structures (Fig. 1). Each aggregate configuration is represented by a block δ_d and a block δ_s.

The partition in blocks, associated to the groups of explicit variables, generates 176 simulations in total.

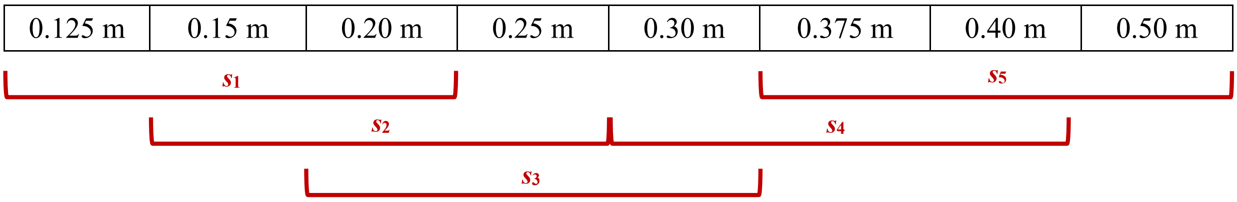


Fig. 2 – Group of *s* for the definition of the blocks δ_s.

5. Push-over analyses

The aggregate configurations obtained were analyzed performing non-linear static analyses (push-over), using the software "TreMuri", to obtain the data required to calibrate the Response Surface models. Two orthogonal directions (*x* and *y*) of the seismic action are considered (Fig. 1) and the distribution of the forces applied (proportional to the masses) was considered with both signs (+F and -F), generating 176 capacity curves for each studied case. Furthermore, in the *y*-direction the analyses over the attainment of the LS limit state were performed, to evaluate the collapse of the structural units in different positions along the aggregate.

Figures 3(a) and (b) show the capacity curves obtained from the analyses in *x*-direction (+ F_x) and *y*-direction (– F_y) respectively, as they represent the weaker cases. In fact, looking at the geometrical configuration of the walls in *x*-direction (Fig. 1), the left-sides are the weaker due to a greater presence of the openings: thus, if the forces + F_x are considered those weaker portions of the walls are the most solicited, causing the progressive decrement of the total capacity of the walls, mainly subjected to the flexure failure mechanism. Thus, an increment of the collapse PGA (PGA_C) is expected if the application of the seismic forces - F_x is considered. Otherwise, in *y*-direction the walls are stocky and the main failure mechanism is the shear one: the two behaviors considering the two different cases (+ F_y and – F_y) are very similar, due to the presence of the openings just in one panel (Fig. 1). Considering the non-regularity in plan, the building results to be weaker to the negative seismic action in *y*-direction (- F_y), due to the torsional effects more accentuated, in this case depending on the asymmetry resulted in the upper part (in plan) of the model. Thus, an increment of the PGA_C is expected if the application of the seismic forces + F_y is considered.

Fig. 3 highlights that the same buildings referred to the 176 simulations exhibit greater capacity in *y*-direction, due to the arrangement and the geometry of the walls in this direction: they have a greater length and most of them are without openings.

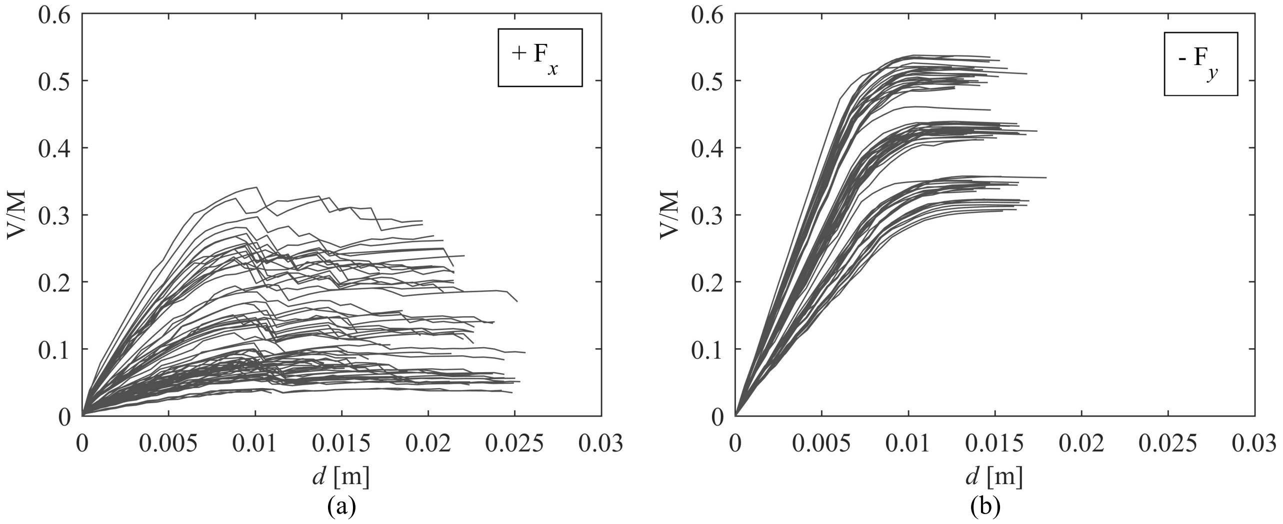


Fig. 3 – Capacity curves from the analyses in (a) *x*-direction (+ F_x) and (b) *y*-direction (- F_y).

The curves are plotted in terms of total shear divided by the total mass (V/M). The displacement refers to an average of the displacements of the nodes located in the top of the buildings (d), weighted by their associated masses.

The geometrical configuration in the y-direction results in distinct levels of vulnerability of the different structural units across the aggregate structure, and they are affected differently depending on their relative position. If the attainment of the life safety (LS) limit state is considered as the limit for the analysis, only the resisting walls of the external Units 1 and 2 fail for shear. Due to the torsional effects, the external Units 1 and 2 (Fig. 1) reach larger displacements with respect to units 3, 4 and 5. Hence, the displacement can still increase until the walls of the other structural units experience the failure for shear. Therefore, according to Battaglia et al. [12], the analyses over the attainment of the LS limit state were carried out in this work, to allow the resisting walls in y-direction of the other Units (3, 4 and 5) to reach the shear collapse. Therefore, this type of analysis allows to evaluate the different vulnerabilities of the masonry structural units along the aggregate structure and to define a hierarchy of collapse of the various structural Units. Thus, larger values of PGA_C are expected for the structural units occupying the internal positions along the aggregate, being affected by lower torsional effects and showing a greater stiffness against the seismic action in y-direction.
In order to preserve the reliability of the analysis, it was decided to neglect the failure related to Unit 5.

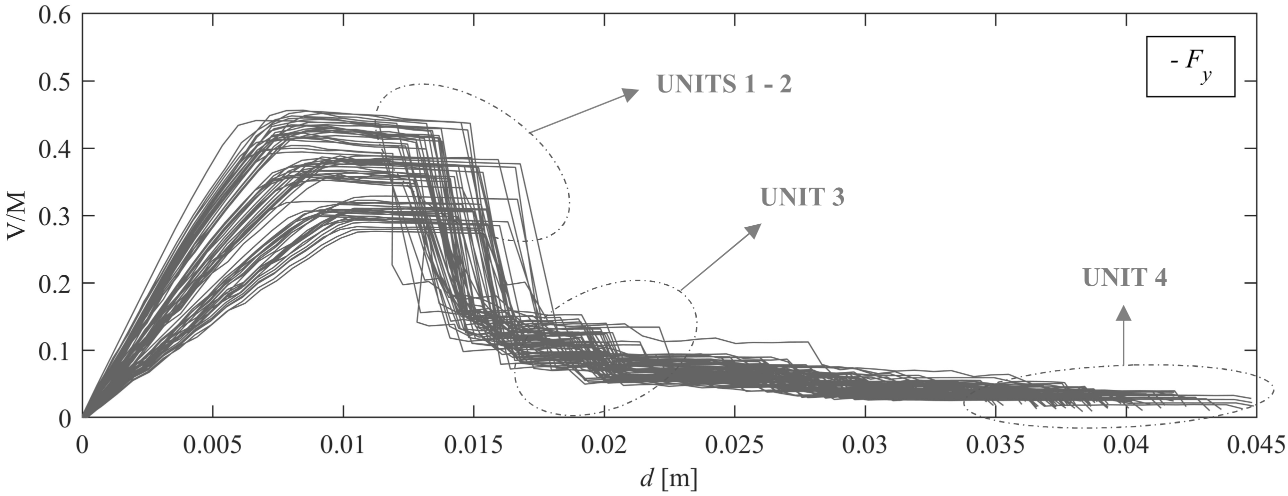


Fig. 4 – Capacity curves from the analyses in y-direction (- F_y) over the attainment of the LS limit state.

6. Definition of the seismic action and evaluation of the collapse PGA (PGA_C)

In this work, in order to consider the possible variability of the seismic action in the site and the uncertainty related to the definition of the ground motion, a group of registered accelerograms referred to past earthquakes was considered. The accelerograms were chosen based on the data referred to previous earthquakes present in the PEER Ground Motion Database (https://ngawest2.berkeley.edu), created in collaboration with the NGA project [15].

According to the Design of Experiment and the division in blocks for the definition of the Response Surface, a group of 48 accelerograms is defined. The accelerograms were scaled to the same reference peak ground acceleration of the considered site (a_g = 0.166 g), imposing some limits to the scaling in such a way as to be compatible with the LS limit state spectrum in that site, in the range period between T = 0.1s and T = 1.0s, but also usable until T=3.0s. Furthermore, the selection was done avoiding recordings with impulsive characteristics, considering fixed ranges of epicentral Joyner-Boore distance (0 km < D_{JB} < 30 km) and fixed ranges of the average shear wave velocity Vs_{30} (200 m/s < $v_{s,30}$ < 700 m/s) [14 ,16] in such a way to make the selections compatible with the considered site. Fig. 5 shows the scaling factors used to scale the accelerograms, in order to make them compatible with the spectrum of Bologna.

Starting from the accelerograms, the correspondent spectra were obtained: Figure 6(a) shows the group of 48 scaled acceleration spectra and Figure 6(b) shows the group of 48 scaled displacement spectra, obtained dividing the spectral accelerations for the frequency squared (ω^2). In the Figures the acceleration and displacement spectra defined by the Italian Code [14] are also reported.

The 48 different acceleration spectra were applied to each of the models described in order to obtain the PGA_C associated with each of them. As proposed in the Eurocode [16] and in the Italian Code [14], and according to Fajfar [17], the N2 Method was used to obtain the structural capacity and the structural demand of the models, measured in terms of displacement, starting from the 48 displacement spectra. Afterwards, the ratio between the displacement capacity and the displacement demand was used to scale the acceleration spectra [10]. According to this procedure, the acceleration corresponding to the structural failure of the model (PGA_C) can be obtained from the product between that ratio and the peak ground acceleration of Bologna (a_g).

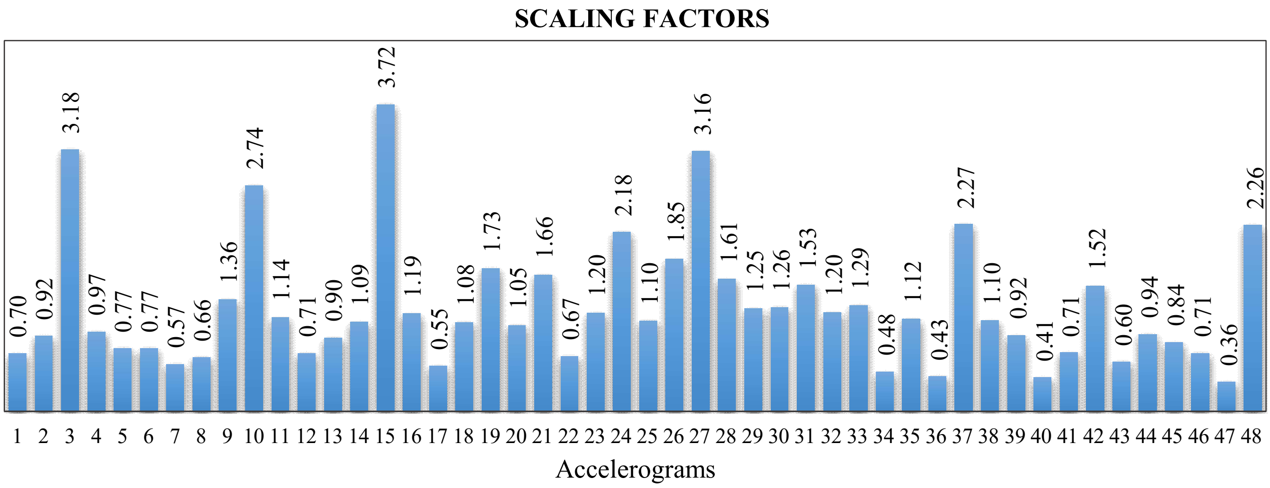


Fig. 5 – Scaling factors used to scale the accelerograms of Bologna: max = 4 and min = 0.25.

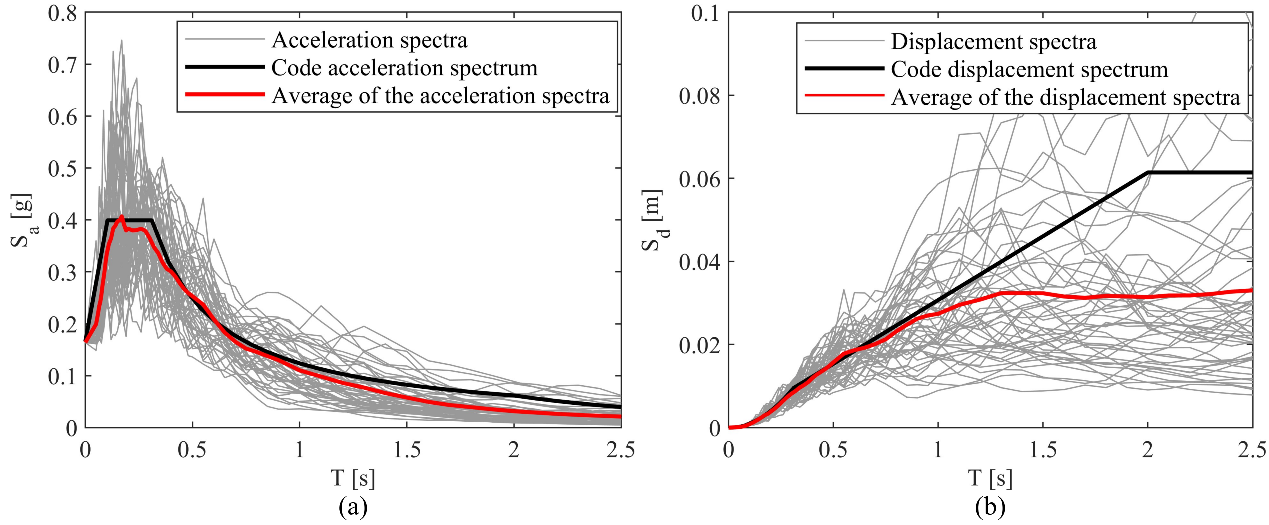


Fig. 6 – (a) Group of 48 acceleration spectra and (b) displacement spectra for the site of Bologna.

7. Response Surface models

The data obtained from the push-over analyses allow to calibrate the Response Surface models, defined by means of a quadratic polynomial, whose equation used to study the masonry aggregate structures is quadratic and it is set as:

$$\log(PGA_{C,i,j,k,l}) = \beta_0 + \beta_1 x_{1,i} + \beta_2 x_{2,i} + \beta_3 x^2_{1,i} + \beta_4 x^2_{2,i} + \delta_{sis,j} + \delta_{d,k} + \delta_{s,l} + \varepsilon_{i,j,k,l} \quad (4)$$

where i stands for the i-th simulation, j for the j-th δ_{sis} block, k for the k-th δ_d block, l for the l-th δ_s block an ε represents the errors. The regression is obtained through the Ordinary Least Squares method and, in order to avoid the prediction of negative values of the variables, the natural logarithm of PGA_C ($\log(PGA_C)$) was used as response parameter for the calibration.

In the following the results referred to seismic action in x-direction (+ F_x) and in y-direction (- F_y) are given, in order to show how the parameters chosen as variables affect the seismic response of the selected masonry aggregate structures with different structural units, also highlighting the differences considering two orthogonal directions of the seismic action.

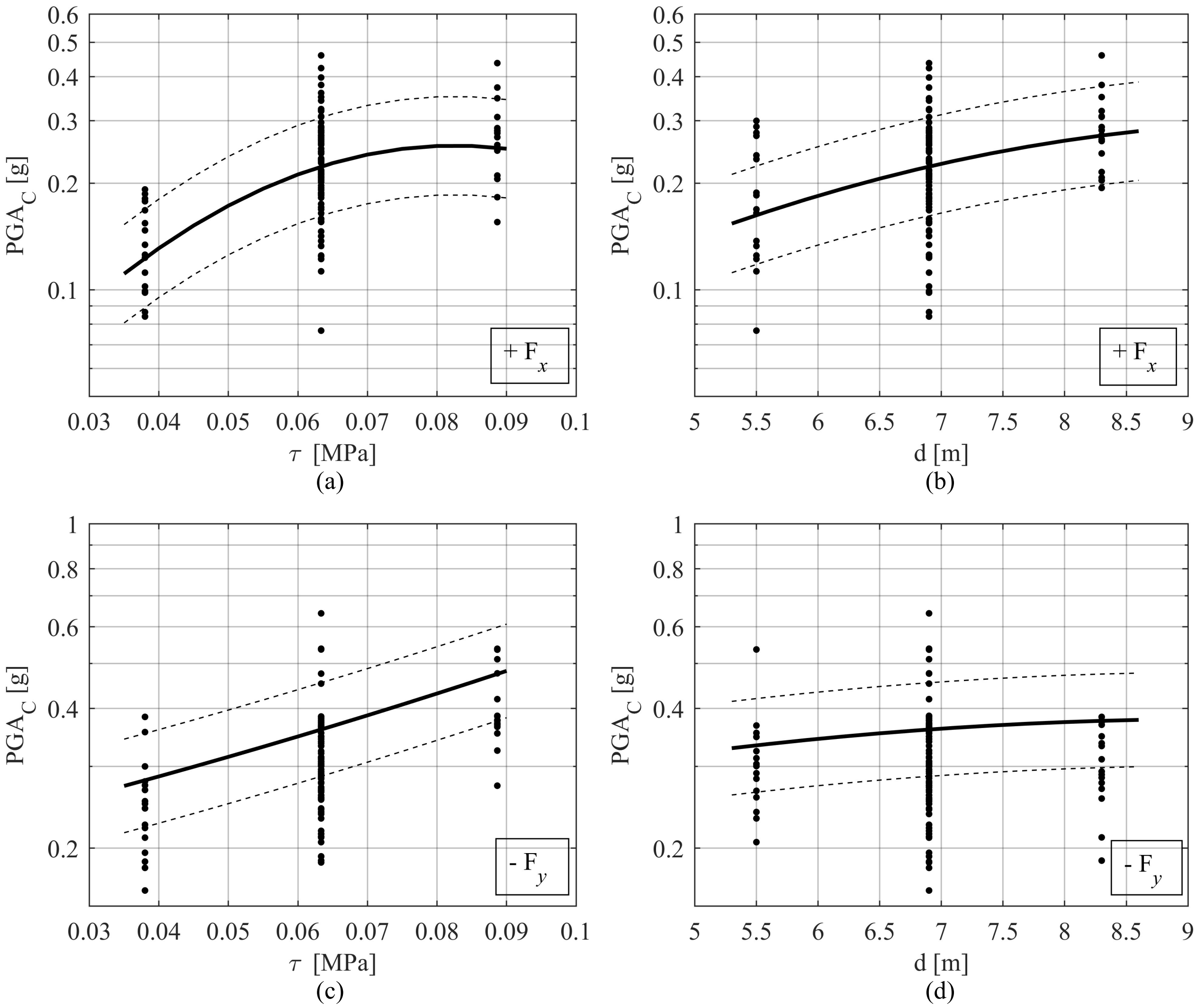


Fig. 7 – RS sections obtained varying τ, considering (a) the seismic forces + F_x and (c) the seismic forces - F_y; RS sections obtained varying d, considering (b) the seismic forces + F_x and (d) the seismic forces - F_y.

The Figures show the sections of the RS models (continuous lines) obtained changing the values of the variable τ (Fig. 7(a) and (c)) and the variable d (Fig. 7(b) and (b)) and setting the values of the other explicit variable (d and τ respectively) and of the implicit variables to their mean values. The sections indicated with the dashed lines are obtained adding and subtracting the RS variance $\sigma = \sqrt{\sigma^2_\varepsilon + \sigma^2_{sis} + \sigma^2_d + \sigma^2_s}$; the points are those corresponding to the various simulations used to calibrate the RS models.

The results confirm that the values of the PGA_C for the *y*-direction are larger than those obtained for the *x*-direction, due to the different geometrical properties of the resisting walls in the two directions. The values of the regression parameters indicate that the shear strength (τ) is the variable most influencing the response; the value of the variable d (mean distance between the masonry walls in *x*-direction) is also affecting the PGA_C, but through a smaller regression parameter. This latter parameter is positive as expected, because since the main failure mechanism is the flexural one, if d increases the length of the walls in *x*-direction increases, and the structure can better behave against the seismic action in *x*-direction.

As for the *y*-direction, the RS indicates a qualitatively similar relationship between the response parameter and the explicit variables, except for the variable d: if d increases, the length of the wall in *x*-direction increases, implying an increment of the slab length in the same direction. As a consequence, the capacity of the walls in *y*-direction is expected to decrease, leading to lower values of the PGA_C. However, differently from what was shown in Battaglia et al. [10], in this application if d increases the values of the PGA_C increases, as well. This is due to the fact that the relation between d and PGA_C is also influenced by the thickness of the walls (*s*), whose values increase as the values of the distance (d) increase. Conversely, in the definition of the RS showed in Battaglia et al. [10], the association between d and *s* is more random and simulations with high values of d associated to low values of *s*, and vice versa, were obtained. Therefore, the trend to decrease of the PGA_C, if d increases, is mitigated by the effect of the thickness of the walls (*s*), making the RS curves relating d and PGA_C flatter.

According to these motivations, the relation between d and PGA_C depends on the ratio between the values of δ_d and δ_s, randomly selected to obtain the 40 aggregate configurations. Fig. 8 shows the 200 (5 Structural Units times the 40 aggregate configurations) relations between d and *s*, highlighting the trend to have greater values of d with greater values of *s*. In the Figure below the thicknesses *s* are divided according to the definition of the 5 groups of *s*, given in Fig. 2. The regressions of the RS models show that the increment of the values of d and *s* leads to an increment of the PGA_C: it is expected that the PGA_C increases as the ratio d/*s* decreases.

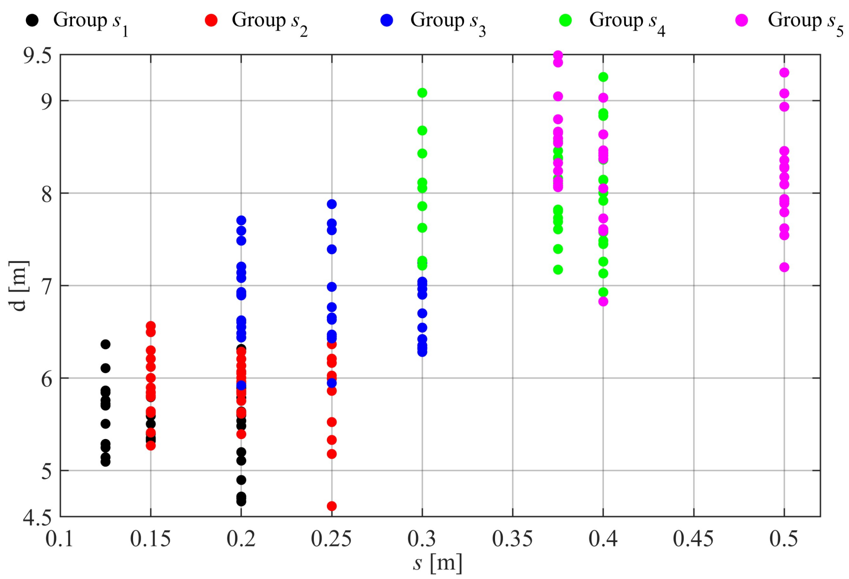


Fig. 8 – Relations between the 200 values of d and *s*, randomly selected.

8. Fragility curves

The obtained RS models were used to estimate the fragility curves of the masonry aggregate structures with different structural units in row. The fragility analysis was assessed using the same procedure adopted in Battaglia et al. [10, 12] using the limit state function in Eq. (1), rewritten in the form:

$$g(\mathbf{x}_E, \beta, \varepsilon, \delta_{sis}, \delta_d, \delta_s \mid PGA_D) = \log(PGA_C) - \log(PGA_D) = \mathbf{f}(\mathbf{x}_E{}^T)\beta + \delta_{sis} + \delta_d + \delta_s + \varepsilon - \log(PGA_D) \quad (5)$$

Eight fragility curves were obtained for seismic action in *x*- and *y*-direction (Fig. 9). They give the conditional probability of the structural failure (P_f) for different values of the structural demand (PGA_D). Thus, once obtained the collapse PGA_C, fixed PGA_D and being the behavior of the structures non-linear, in order to solve the Eq. (5), Monte Carlo method was used. These curves confirm that the masonry aggregate structures are more vulnerable against the seismic action in *x*-direction (red curves) because of the geometrical properties, which have already been discussed in the previous Sections, with respect to the curves obtained for the *y*-direction (blue curves) showing the attainment of the P_f for higher values of PGA_D. These latter curves give the fragility of the external Units 1-2, affecting by substantial torsional effects, decreasing the total PGA_C, referring to the attainment of the LS limit state for the global aggregate structures. However, continuing the analyses to allow the other Structural Units the attainment of the shear failure, the green curves give the fragility of the internal Unit 3 and Unit 4, showing their higher stiffness against the seismic action in *y*-direction. In Fig. 9 the continuous lines are related to the analyses carried out with the positive seismic forces, the dash dot lines to those with the negative seismic forces, highlighting that the presence of the openings and their positions in the masonry walls make the aggregate structure more fragile against the positive forces in *x*-direction and against the negative forces in *y*-direction.

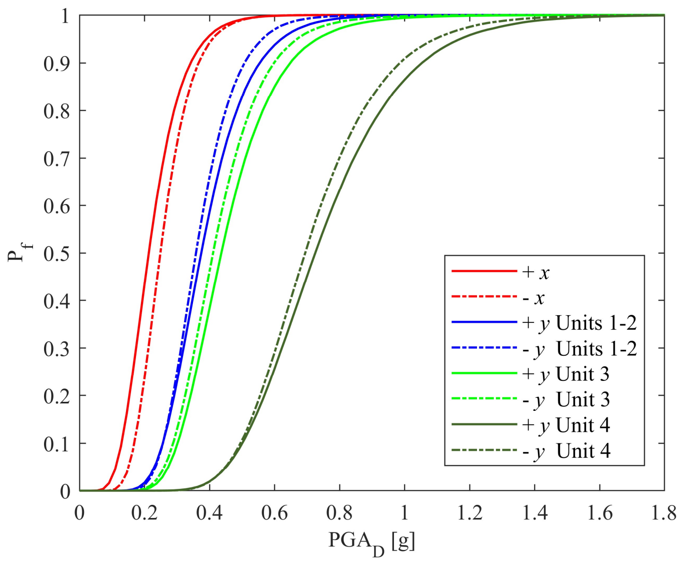


Fig. 9 – Fragility curves of the masonry aggregate structures with different structural units in row.

9. Conclusions

This work is focused on the fragility analysis of masonry aggregate structures, made by different structural units each other, by statistical procedures. In order to account for variabilities and uncertainties involved in the problem, the Response Surface statistical method is used and it is calibrated using numerical data obtained by non-linear static analysis using the “TreMuri” software. Fragility curves are developed, by applying Monte Carlo simulation. The results showed that the parameter most influencing the seismic response is the shear strength (τ) of the resisting masonry walls, followed by the geometrical properties of the structure, showing as the arrangement and the geometry of the masonry walls are determining factors in the seismic performance of unreinforced masonry buildings. Moreover, considerable differences were found between the two directions due to the different configurations of the masonry walls. Furthermore, the disposition of the openings in the walls makes some of their parts weaker against the seismic action, leading to different seismic performances if positive or negative seismic forces are considered. Higher values of the collapse PGA are associated to the internal structural units along the aggregate structure, obtained continuing the analyses over the attainment of the LS limit state and allowing the walls of the more internal Units to reach the shear collapse. These analyses allowed to make a hierarchy of collapse of the various structural units along the aggregate, for the presence of the rigid slabs: the fragility curves show a decrement of the fragility if more internal units are considered.

Finally, the regressions of the RS models showed that the seismic response is affected by the ratio between the distance between the masonry walls (d) and their thicknesses (s): the PGA_C increases as the ratio d/s decreases; however, in this work this trend is influenced by the variability of the seismic action and the irregular shape of the spectra and the variation of the mechanical properties.

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