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NUMERICAL FRAGILITY OF TYPICAL ITALIAN INDUSTRIAL PRECAST RC FRAMES: EFFECT OF GEOMETRY AND NON-STRUCTURAL COMPONENTS

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Abstract

Two mainshocks and a series of smaller earthquakes struck the Emilia region, in Northern Italy, in May 2012. The earthquake sequence has highlighted the vulnerability of prefabricated industrial RC buildings, which were quite numerous in a region that is one of the most productive in Italy. Remarkable damage was observed during the field surveys that followed the mainshocks. According to some estimates, almost 70% of reinforced concrete precast buildings in the affected areas collapsed or were severely damaged by the earthquake. The main reason of the poor seismic behavior observed is arguably the lack of seismic provisions at the time of design and construction. In fact, the area affected by the events was not considered a "seismic region" until recently.

Scope of this study is to assess the structural fragility of this building typology and it is a first step in a more comprehensive study that aims to understand the seismic behavior of these industrial precast structures, taking into account various failure modes and the interaction between structural and non-structural components. In fact, in spite of their apparent simplicity, there is still a critical lack of information on their seismic performance. On the other hand, these buildings present many commonalities, suggesting that useful information about the vulnerability of the building stock could be obtained by analyzing a number of prototype structures, whose geometrical and mechanical characteristics are representative of those found in the area of interest. In fact, when a vulnerability assessment is required at a regional scale, analyzing single buildings would become extremely onerous and thus impracticable due to both time and economic constraints.

In this contribution, the seismic vulnerability and expected performance of this typology of industrial buildings is assessed through seismic fragility curves. Different geometries as well as a range of possible material properties are considered in order to assess their influence on the overall seismic behavior. Moreover, the interaction with non-structural infills is investigated.

Following such an approach, valuable information about the seismic behavior of single-storey precast RC frame structures built without specific seismic provisions can be obtained. Hence, indications about the most critical structural configurations can be provided, helping to identify and prioritize the required repairing and/or retrofitting interventions either in the post-earthquake emergency and reconstruction phase as well as a part of a risk-reduction national plan of preventive retrofit interventions.

Keywords: Industrial precast buildings; Collapse fragility functions; Parametric vulnerability assessment

1. Introduction

Two mainshocks and a series of smaller earthquakes struck the Emilia region, in Northern Italy, in May 2012. The two main earthquakes were characterized by a moment magnitude of 6.1 an 6.0 respectively. During these earthquakes large values of horizontal Peak Ground Accelerations were recorded, in fact at the stations closest to the epicenters values as large as 259cm/s^2 (May 20th, epicentral distance R_e=12.3km) and 411cm/s^2 (May 29th, R_e=1.4km) were registered. Moreover, very large peak ground vertical-acceleration values were recorded (up to 841cm/s^2) [1].

This earthquake sequence has highlighted the vulnerability of the industrial precast buildings, as this building typology is quite frequent in the region which is one of the most productive in Italy. Remarkable damage was observed in the field surveys that followed the mainshocks. According to some estimates, almost 70% of reinforced concrete precast buildings in the affected areas collapsed or were severely damaged by the earthquake [2]. The main reason of the poor seismic behavior observed reflects the lack of seismic provisions of the codes applicable at the time of design and construction of the majority of the buildings. In fact, the area affected by the events was not considered seismic until recently and it was included in the hazard maps as a low-to-moderate seismicity region only in 2006, becoming mandatory for designers from 2008 [3]. For this reason, the majority of the precast buildings were mainly designed to sustain vertical loads, since the prescribed horizontal load was only a very small fraction of the total weight of the building (e.g. 2% according to the code effective in 1992). Columns typically have small cross-sections with the minimum longitudinal and transverse steel reinforcement allowed by the codes, and the connection between columns and beams are based solely on friction.

Field surveys that followed the earthquake sequence allowed to identify recurrent typologies of failures. In general, failure in the connections between elements due to the lack of mechanical connectors between various precast monolithic elements was the main cause of most of the collapses, inducing loss of support of the precast beams from the columns or of the roofing elements from the beams. In many cases, moreover, the collapses were caused by the interaction between the structure and non-structural components (such as masonry infills). Buildings with regular infills along the four sides of the building (Fig. 1a), and the roof sufficiently rigid in its own plane to transfer horizontal actions to the wall panels, rarely suffered serious damage, being the in-plane stiffness of the masonry curtain walls in general very high compared with the column stiffness. In these cases, only minor damage was observed in the masonry walls. However, in many cases, an irregular layout of the infills, due, for example, to the presence of a strip-window between the infill wall and the beam, generated irregularities and concentrated forces that had influenced the global behavior of the structure, producing a short-column effect that triggered in many cases failure due to loss-of-support of the beam (Fig. 1b) or due to flexure in the columns. Moreover, deficiencies in the structural elements (e.g. columns or foundations not designed to sustain seismic actions), plan-irregularities, deformable roofs led to the crisis a large number of structures.

(a) (b)

Fig. 1 – Examples of industrial precast structures inspected after the Emilia earthquake: (a) a frame structure with regular infills that suffered minor damages (b) a frame structure with irregular infills that experienced failure due to loss of support of the beam [2]

Scope of this study is to numerically assess the structural behavior under seismic loading of this building typology and it is a first step in a more comprehensive study that aims at understanding the seismic behavior of these industrial precast structures relying upon a combination of numerical analyses on a representative component of this typology of structures and field evidences and taking into account various failure modes and the interaction between structural and non-structural components.

These buildings present strong homogeneities, due to the highly standardized structural configurations and relatively limited "variations". This peculiarity, typical of industrial precast frame structures, suggests that useful information about the actual vulnerability of the building stock could be obtained by analyzing a number of prototype frames, whose geometrical and mechanical characteristics are representative of those found in the area of interest. In fact, when a vulnerability assessment is required at a regional scale, analyzing single buildings would become extremely onerous and thus impracticable due to both time and economic constraints. For this reason, in this contribution, seismic vulnerability is assessed through seismic fragility curves, that indicate the probability for a building or a building class of experiencing a certain level of damage as a function of the ground motion. With reference to a single storey and single bay 2-dimensional structure, different geometries as well as a range of possible material properties are considered in order to assess their influence on the overall seismic behavior. Moreover, as the presence and configuration of infills, together with the typology of connection between precast elements, plays a fundamental role in determining the seismic performance of these frame structures, alternative degrees of connection and the interaction with non-structural walls are also investigated.

2. Procedure outline

In this contribution, a procedure to assess the collapse vulnerability of the existing precast building stock has been developed. As already mentioned, a simple structural configuration is assumed, consisting in a one-storey and one-bay 2D structure since this is considered a representative component of the assessed structural typology. Hence, starting from this basic structure, a parametric study has been performed modifying different characteristics (e.g. geometry, materials, loads) to represent most of the possible structures of the same typology that could be found in the area of interest. Finally, the seismic response of each of them has been analyzed providing information about the seismic behavior of existing single-storey precast RC frame structures built without specific seismic provisions. The main steps involved in the procedure are briefly described in the following.

2.1 Selection of geometrical and mechanical characteristics

In order to evaluate fragility curves a simulation approach has been used. A virtual building stock has been created by considering combination of the following parameters: bay length in the longitudinal and transverse direction ($L_{col,x}$ and $L_{col,y}$ respectively); column height (H_{col}); concrete strength (R_{ck}); vertical load magnitude (q); friction coefficient in beam-column connections.

Table 1 – Geometrical and mechanical characteristics selected for the parametric study

		Distribution	μ [m]	β [ln(m)]	min [m]	max [m]	n_i
$L_{col,\,x}$		Lognormal	14.9	0.3	8	30	5
$L_{col,\,y}$		Lognormal	9	0.28	8	10	5
H_{col}		Lognormal	6.5	0.25	4	12	5
			Values considered				
R_{ck}	[Mpa]	Constant	35	40	45	50	4
Vertical load	[kN/m^2]	Constant	4.75	5.5	6.25	7	4
Friction coeff.	[-]	Constant	0.2	0.3	0.4		3

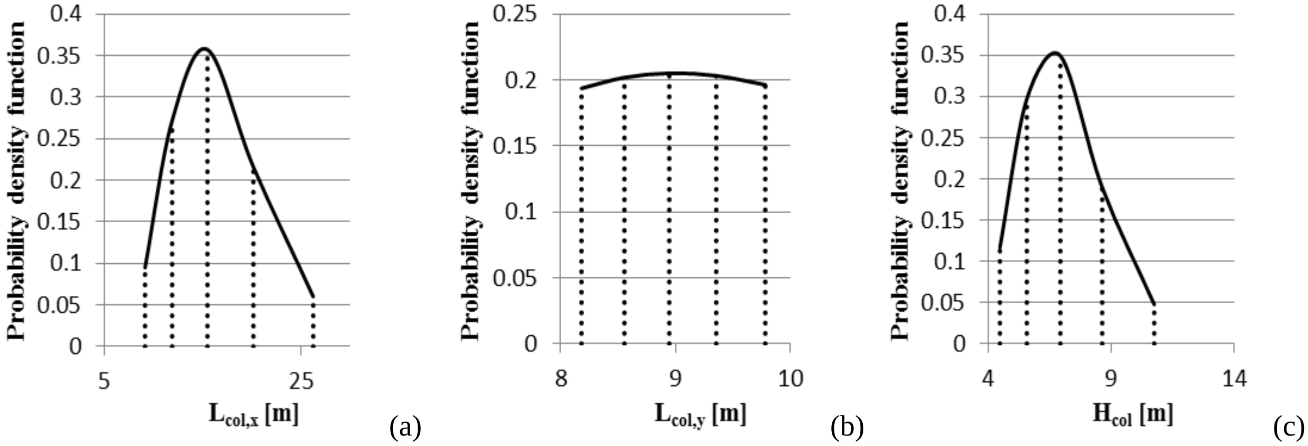

Fig. 2 – Distribution of the geometrical parameters and selected values: (a) bay length in the longitudinal direction; (b) bay length in the transverse direction; (c) column height

Literature results [4] have been adopted to identify probability distributions for the aforementioned parameters (Table 1). Geometrical parameters have been characterized by bounded lognormal distributions (see Fig. 2). Uniform distributions have been assumed for the concrete compressive strength, the vertical loads and the friction coefficient. Each of these distributions was divided into intervals whose central values were used in the simulations. The number of intervals used for each distribution is indicated in Table 1 as n_i.

Hence, all the possible combinations of these values have been considered, resulting in 6000 different configurations. Since each value is associated with a probability of occurrence, each configuration can also be associated with a probability of occurrence of the corresponding structure in the area of interest. This probability was computed as the product of the probabilities of the various intervals, i.e. assuming independence of the random variables.

Furthermore, at first, all the configurations have been studied considering a bare frame, then the analyses have been repeated assuming frames infilled with masonry curtain walls. An irregular layout of the infills have been considered, allowing for the presence of a strip-window just below the beam.

2.2 Design of the structures

Each combination of the independent parameters described in Section 2.1 corresponds to a possible structure. Hence, following the provisions of the code effective in Italy in the early ’90 [5], all the considered structures have been designed according to the allowable stress approach. As already mentioned, the only horizontal force considered at the time was 2% of the weight and since the seismic prescriptions did not apply for the area considered, the beam have been designed to be simply supported by the column with no additional mechanical connection. The minimum support depth allowed by the code was provided, which is 8cm+L/300, where L is the clear length of the beam [6]. Due to the very low values of shear expected on the columns, the minimum area and spacing of stirrups were always supplied, as for this level of loading a shear verification was never required.

2.3 Definition of ground motions

The next step in the procedure is the selection of a number of ground-motion records representing the seismic demand. In order to capture the different response of buildings due to record-to-record variability, the seismic demand on the structure has been represented by a suite of 100 recorded accelerograms randomly selected. Since an assessment procedure is undertaken, the time histories have not been selected to be strictly compatible with the Uniform Hazard Spectrum (Fig. 3). At this stage of the research, the vertical component of the ground motion was not included in the analyses. However, it will be taken into account in further developments of the study.

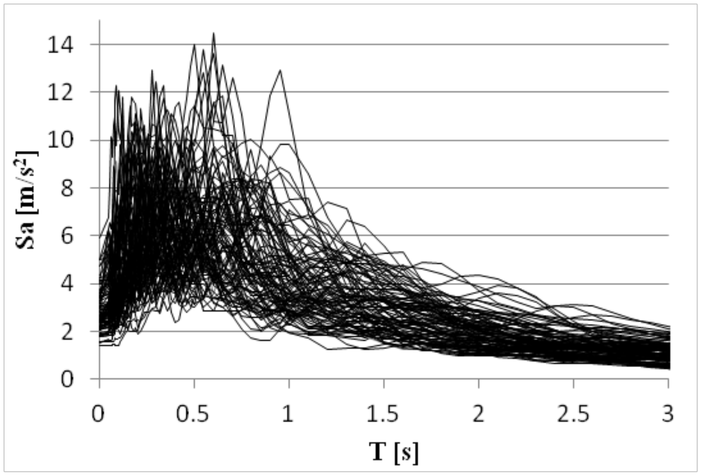

Fig. 3 – Pseudo-acceleration response spectra for the selected ground motions

2.4 Definition of collapse fragility functions for each frame

The probability of collapse is typically estimated by means of a collapse fragility function, which expresses the probability of building collapsing as a function of the ground-motion intensity (IM).

The development of collapse fragilities can take place according to different methods with different degrees of complexity, from Incremental Dynamic Analysis (IDA) [7] to engineering "expert judgement". Regardless of the method adopted to obtain these fragilities, the definition of collapse itself can be challenging. In fact, collapse is generally associated with either local or global failure of the gravity load resisting system, but the criteria adopted to indicate failure are often affected by great degrees of uncertainty. Especially when non code-conforming existing buildings are assessed, uncertainties in collapse capacity reflect record-to-record variability and limited knowledge of the parameters governing the elements' non-linear behavior.

In this contribution, collapse fragilities have been developed in a simplified way, assessing the capacity of the analyzed structure through pushover analyses and then comparing the capacity and the demand according to the Capacity Spectrum Method (CSM) [8]. The methodology implemented is presented in the following and it is summarized in Fig. 5.

2.4.1 Modelling of the structure

The structures are modelled using Ruaumoko [9], which relies on lumped-plasticity models, while the sectional analyses have been performed with CUMBIA [10].

Given the regular layout of the structures, and assuming that the roofing is not rigid in its plane, a 2D model is deemed accurate and hence a single-bay frame is modelled to represent each structure. Columns are modelled using concrete beam-column members, which allow for Moment-Axial load interaction while the beam is assumed elastic. A translational spring characterized by an elastic-perfectly plastic behavior has been introduced to model the connection between the beam and the top of the column, which is assumed to rely on friction (see Fig. 4a and b). The stiffness of the elastic branch of the translational spring is taken as 10^6 times the slope of the elastic branch of the Moment-Curvature relationship of both beam and column connected by the spring.

Infills have been modelled introducing an elastic truss member acting in compression. The nonlinear behavior of curtain walls was not modelled since significant damage of these elements was seldom observed during the field surveys. The height of its cross section has been taken as 25% of the diagonal length of the panel, following the suggestions of Paulay and Priestley [11]. In order to simulate an irregular layout of the infill panel, due to the common presence of a strip-window, the diagonal truss element connects a node at an intermediate height of a column (placed at 1.2 meters from the top) to a node at the bottom of the other column (Fig. 4b).

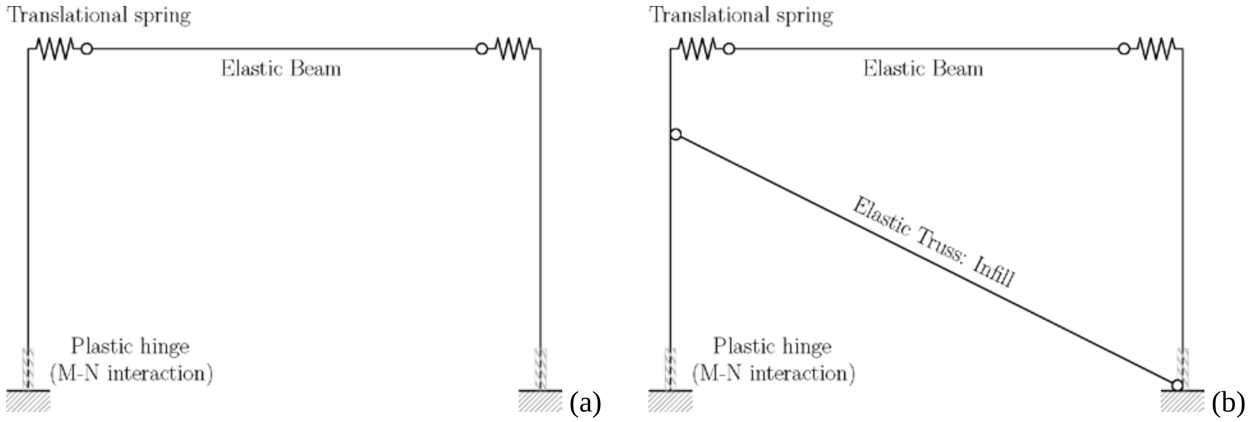

Fig. 4 – Schematic representation of the structural models: (a) bare frame; (b) infilled frame.

2.4.2 Pushover analysis and identification of the failure mode

Once the models have been prepared, they have been subjected to displacement-controlled quasi-static non-linear analyses which allow to capture the (possible) degrading behavior of the plastic hinges that could develop at the base of the columns. Large displacements are considered in the analyses.

Three different failure modes have been investigated, namely, shear failure, flexural failure and loss of support of the beam from the top of the column. In particular, the ultimate chord rotation has been adopted to assess the flexural capacity of the columns. Failure due to sliding of the beam is observed when the relative displacement between the end of the beam and the supporting column exceeds the threshold value of support depth assumed during the design phase (see Section 2.2). The ultimate point of the pushover curve for each model corresponds to the smallest displacement at which either shear, flexural or sliding failure occurs.

2.4.3 Comparison between capacity curve and seismic demand

Once that the Force-Deformation relationship has been established, the performance assessment is carried out according to the Capacity Spectrum Method (CSM) described by the ATC-40 [8].

According to the CSM, the structural response is represented in acceleration-displacement response spectrum (ADRS) format, and the result is termed capacity curve of the structure. The capacity curve can then be plotted against the ground motion, as the seismic demand can be also represented in ADRS format.

The method relies on the basic assumption of equivalent linearization methods, which states that the maximum displacement of a non-linear SDOF system can be estimated from the maximum displacement of a linear elastic SDOF system characterized by an appropriate period and damping: the equivalent period and equivalent damping ratio, respectively. The Capacity Spectrum Method assumes that the equivalent damping of the system is proportional to the area enclosed by the capacity curve and the equivalent period is taken as the secant period intersecting the capacity curve at its maximum displacement. Both parameters are function of the ductility capacity of the structure.

The ATC-40 [8] gives guidance for the evaluation of the equivalent damping ratio based on the hysteretic behavior and ductility capacity.

The seismic action, multiplied by an appropriate reduction coefficient R accounting for the damping of the structure reaching the selected limit state (damped spectrum), is scaled to match the ultimate point of the capacity curve (i.e. performance point at collapse). Since the ground-motion intensity measure commonly adopted in the vulnerability assessment typically refers to a 5% conventional damping, the scaled spectrum matching the collapse performance point is divided by the factor R, resulting in the reduced spectrum at 5% damping that would induce the collapse of the structure.

With reference to this final spectrum, both Peak Ground Acceleration (PGA) and Spectral acceleration at the fundamental period of the structure ($Sa(T_1)$) can be identified. By repeating the same procedure for each of the simulated structures and for each ground-motion, a population of IM values, representing the capacity of each structure, has been obtained and hence a collapse fragility function could be defined. Typically, these functions are assumed to follow a lognormal distribution and, as such, they can be fully described by a median IM and a dispersion term. The structures considered in this study are characterized by a wide range of fundamental periods and, for this reason, fragility functions are presented in terms of PGA. This Intensity Measure, in fact, allows a straightforward comparison among functions corresponding to different structures and, eventually, of the obtained numerical fragilities with empirical ones.

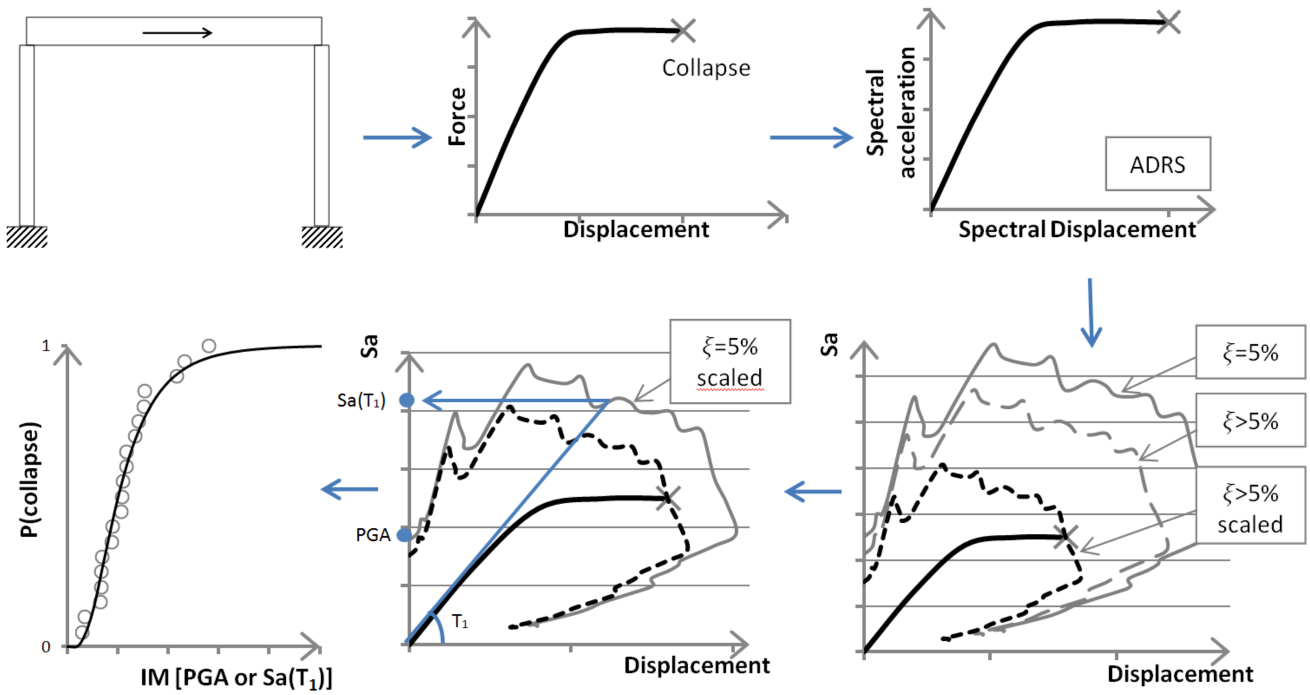

Fig. 5 – Capacity Spectrum Method for the definition of collapse fragilities

2.5. Definition of collapse fragility functions for groups of structures

The final step of this procedure is to “aggregate” the collapse fragility functions obtained for each frame to obtain information about the overall vulnerability of the existing building stock. This has been done by weighing the fragilities according to the probability of occurrence of the corresponding structure within the selected inventory/building stock (see Section 2.1), or, in other words, by computing the contribution to the global probability of collapse of each typology of the precast structures that can be found in the region. Hence, all the contributions can be summed up, resulting in a global collapse fragility function, useful to estimate the seismic vulnerability and potential risk (for a given seismic intensity) of industrial precast structures at a regional scale. Moreover, these fragilities could be combined with an hazard model available for the area under investigation, allowing to evaluate the actual probability of collapse of the building stock on an annual basis. This further development will constitute the subject of future contributions.

Adopting a similar procedure, fragility functions can be obtained for groups of frames with common characteristics (such as the type of concrete adopted or the column height). In this way, it is possible to evaluate the effect of the considered geometrical and mechanical characteristics on the seismic behavior of precast structures.

3. Results

In the following section the results of this study are summarized.

3.1 Bare Frames

Fig. 6 presents the results of the analyses performed on the bare frames.

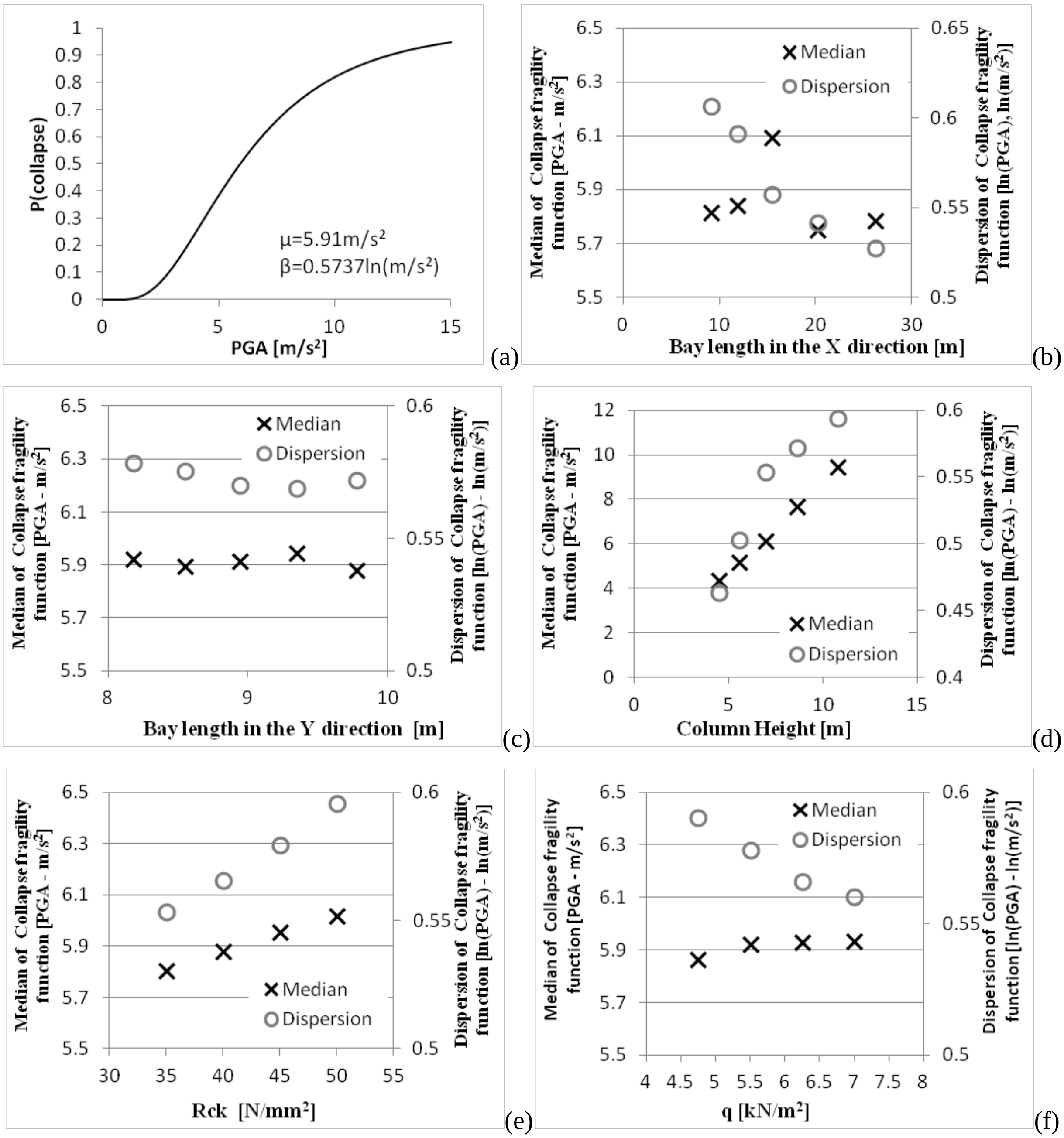

Fig. 6 – (a) Global fragility function for precast bare frames; (b)-(f) Median and dispersion of fragility functions for bare frames: effect on the seismic vulnerability of geometrical and mechanical properties

For this typology of structures, no shear or loss of support failure were observed, since the force required to reach flexural collapse was always smaller than the one required to overcome friction or to induce shear issues. Moreover, for simplicity, no vertical seismic demand was considered during the analysis. However, it is acknowledged that the presence of vertical ground-motions could lead to a premature sliding failure, since the vertical loads transferred from the beam to the column could be strongly reduced.

According to this analyses, the global fragility function has a median value of 5.91m/s^2 and a dispersion term of 0.5737ln(m/s^2) (Fig. 6a).

The fragility functions developed for the different values of bay length in the longitudinal and transverse direction (Fig. 6b and Fig. 6c respectively) do not show a clear trend. This indicates that these two parameters are not directly related to an increase or decrease of the expected vulnerability. Similarly, the structures designed to sustain different levels of vertical loads do not appear to be characterized by significantly different fragility functions (Fig. 6f). On the other hand, the height of the columns and, to a minor extent, the compressive strength of the concrete adopted affect the vulnerability of the analyzed structures. In particular, as the concrete compressive strength increases, the vulnerability of the building stock decreases while, probably counter-intuitively, as the height of the columns increases, the vulnerability decreases (Fig. 6e and Fig. 6d respectively). The latter finding is mainly due to the higher displacements that taller structures can sustain prior to reaching the ultimate chord rotation, and hence collapse, with respect to shorter ones.

3.2 Infilled Frames

For what concerns infilled frames, a much more vulnerable behavior with respect to bare frames was observed, with a global fragility characterized by a median and dispersion of 2.26m/s^2 and 0.452ln(m/s^2) respectively (Fig. 7a).

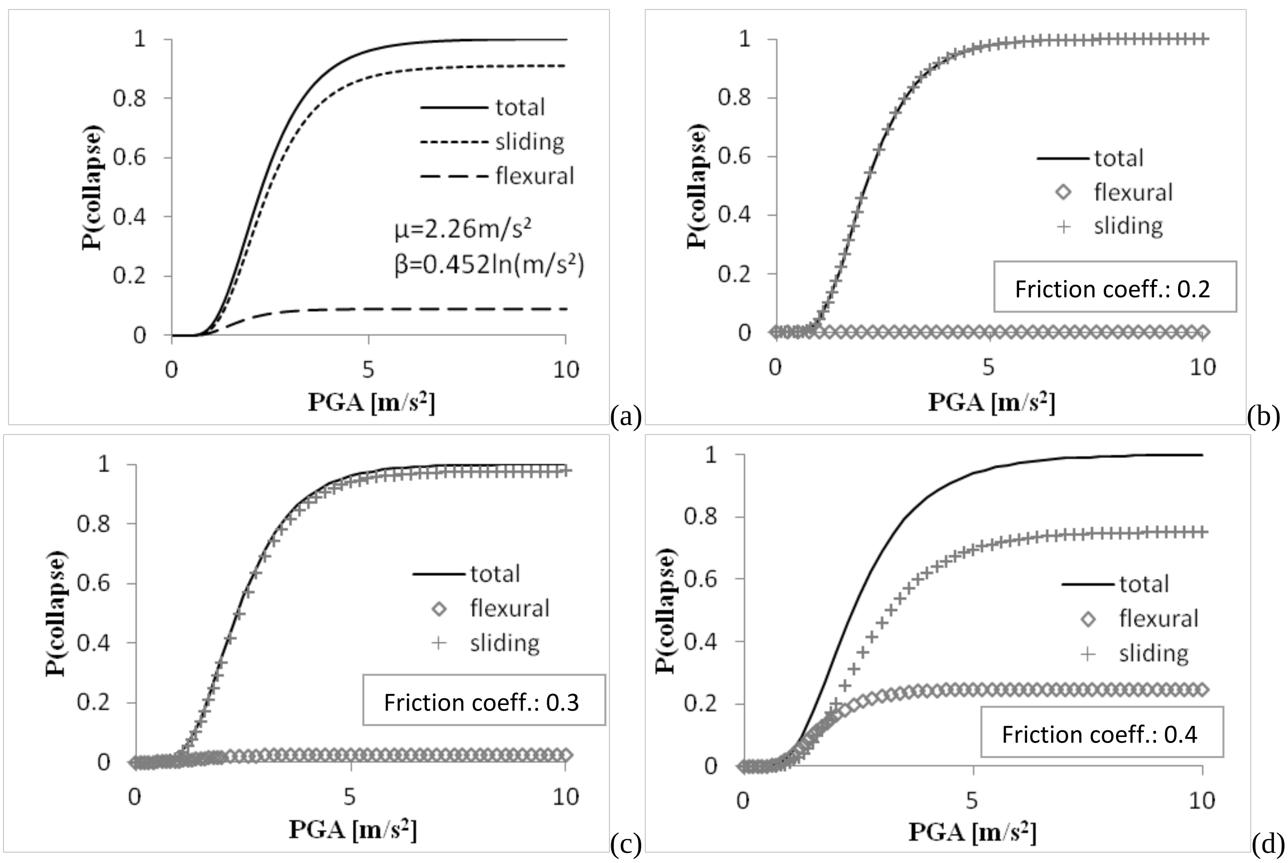

Fig. 7 – (a) Global fragility function for precast infilled frames; (b)-(d) Fragility functions for infilled frames: effect on the seismic vulnerability of the adopted friction coefficient

Shear failures in the columns were never observed. However, due to the presence of the stiffening wall only up to a certain height of the structure, short column effects could be observed and sliding failures appeared to be the most probable failure mode even though the vertical acceleration effects have not been accounted for in the analyses. The amount of structures experiencing collapse due to loss of support with respect to those experiencing flexural failure is dependent on the friction coefficient adopted during the analyses. Fig. 7b, Fig. 7c and Fig. 7d show the collapse fragilities obtained for a friction coefficient of 0.2, 0.3 and 0.4 respectively, together with the contribution of sliding and flexural failure. As expected, the lower the friction coefficient, the higher the probability of having sliding.

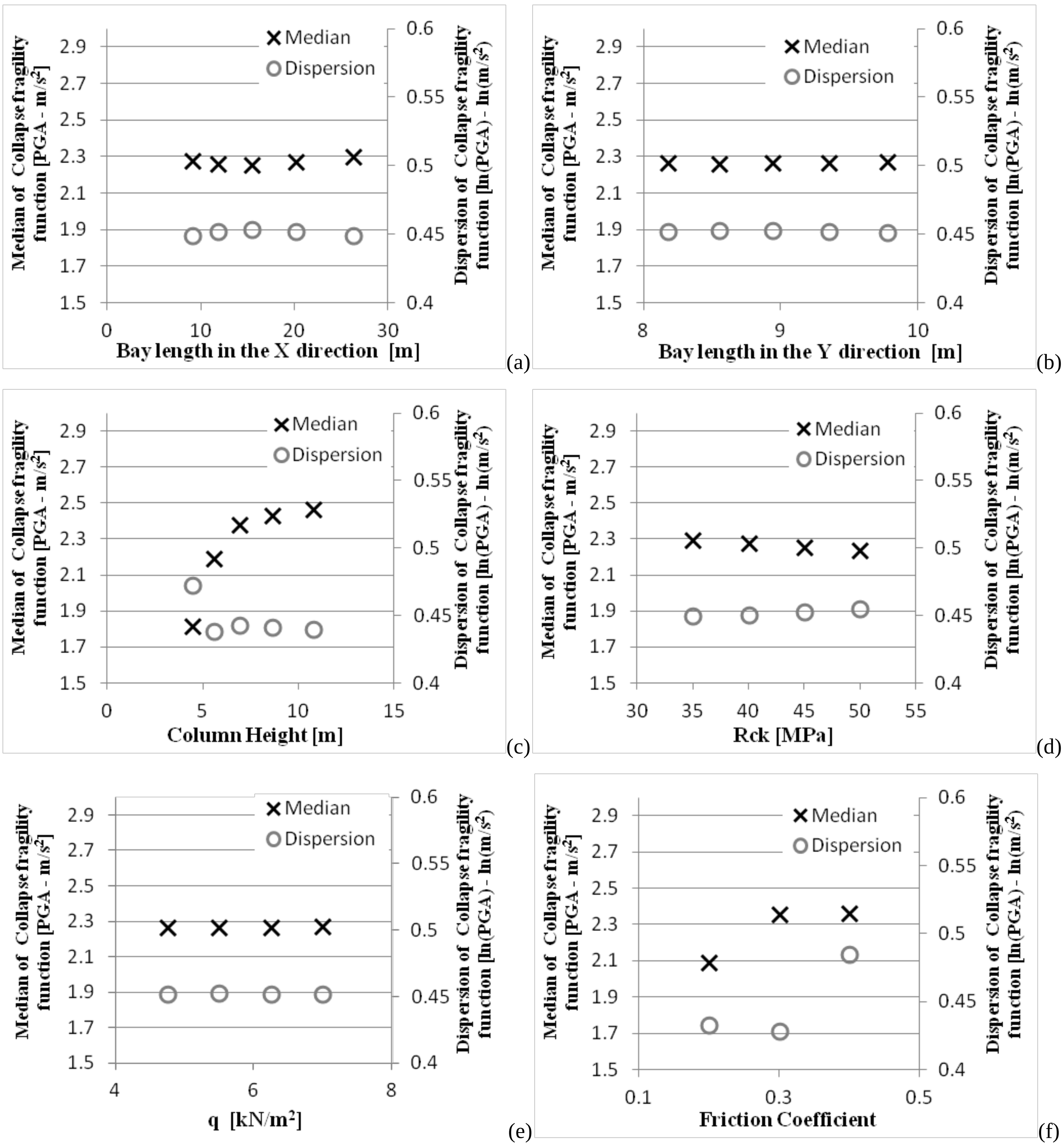

Fig. 8 –Median and dispersion of fragility functions for infilled precast frames: effect on the seismic vulnerability of geometrical and mechanical properties

Finally, the influence of the geometrical and mechanical properties on the seismic vulnerability has been assessed for the infilled frames. In this case, the only two parameters that affect significantly the seismic behaviour are the friction coefficient and the column height. A smaller friction coefficient and shorter columns imply higher vulnerability (Fig. 8f and c). However, since the strip-window is assumed at the same height for all the models, the length of the column portion in contact with the masonry infill has a little effect on the structural response. Hence, the median value of the collapse fragility has a smaller dependence on the column height with respect to what was observed in the case of bare frames.

4. Conclusions

In this contribution, the seismic behavior of single-storey industrial precast RC frame structures typical of construction practice in Italy, Greece, Turkey and, in general, southern Europe, built without specific seismic provisions, has been studied through collapse fragility functions. A population of different frame geometries has been considered in order to cover a wide range of structural configurations that could be found in a typical industrial area of the Emilia region, in Italy. The frames have been designed according to the code that was effective in Italy in the early nineties and no seismic detailing have been considered since the area, at that time, was not considered as seismic. Collapse fragility curves were then obtained for each structure adopting a simplified approach based on the Capacity Spectrum Method and then combined considering the frequency of occurrence of the considered structures. In this fashion, a fragility curve applicable at a regional scale has been developed for both bare and infilled frames. As expected, the presence of a masonry infill together with a strip-window enhances the vulnerability of these simple precast structures with respect to the bare frames.

Lastly, the influence of some geometrical and mechanical properties on the global vulnerability has been investigated. The results show that the column height and the friction coefficient are the parameters that affect the most the structural vulnerability. On the other hand, the bay length in both directions, the weight of the beam and roof and the type of concrete adopted have a very limited effect on the building response. This may be due to many reasons: first, the seismic demand has been chosen almost randomly and hence the record-to-record variability is quite large with respect to the variability of the structural responses. Furthermore, all the structures have been designed according to the same criteria and the same code, and for this reason some level of variability due to different designers experience and preferences or constraints have been neglected.

Future developments will then include the assessment of other design codes and building typologies, including different variabilities (e.g. considering the variability of the collapse capacities of the structural elements, which are affected by a great degree of uncertainty) and different analysis methodologies. Moreover, some of the authors of the present paper have been working to develop empirical fragility functions for R.C. industrial precast structures based on observational data collected after the Emilia seismic sequence [12]. Hence, the comparison between computational and observational fragilities will constitute subject of future contributions. This work, in fact, represents the first step in a more comprehensive study aiming at understanding the seismic behavior of existing industrial precast buildings at a regional scale thanks to numerical results and their comparison with the empirical evidences collected by field surveys that followed the Emilia earthquake in 2012.

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