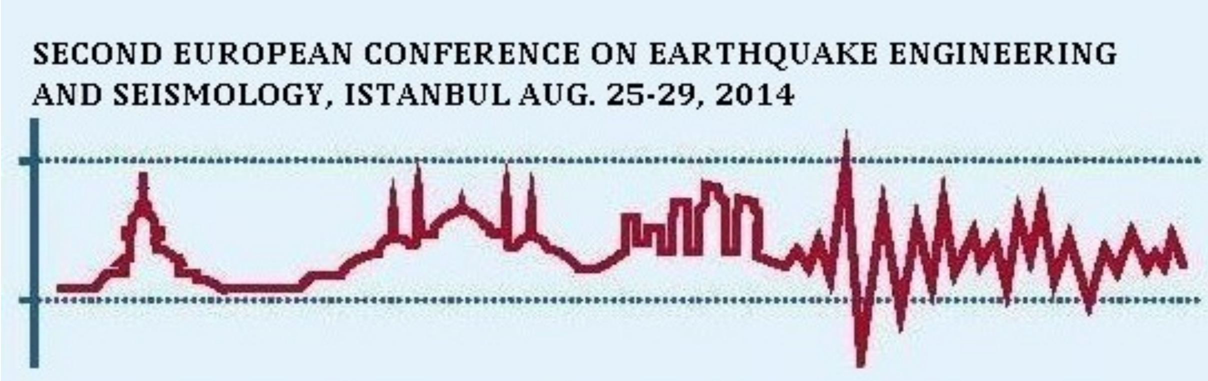

SCENARIO DAMAGE ANALYSIS OF RC PRECAST INDUSTRIAL STRUCTURES IN TUSCANY, ITALY

Chiara CASOTTO[1], Vitor SILVA[2], Helen CROWLEY[3], Rui PINHO[4], Roberto NASCIMBENE[5]

ABSTRACT

Reinforced Concrete (RC) precast industrial buildings have recently suffered excessive damage, with substantial direct and indirect losses, as highlighted during the 2012 Emilia-Romagna earthquakes in Northern Italy, as well as during past events in Turkey (Adana-Ceyhan and Kocaeli and Duzce earthquakes, in 1998 and 1999). The aim of this paper is thus the evaluation of the seismic vulnerability of an Italian RC precast building portfolio, to be used in a scenario damage assessment. The building population employed in this study was generated considering variability in the material and geometric parameters, whose probabilistic distributions were obtained from a database of field surveys comprising 650 warehouses and information available in the literature. An analytical methodology was used consisting in 1) random sampling of one hundred structures from the building stock being assessed, 2) pushover analyses to define the damage limit states, 3) dynamic nonlinear analyses and comparisons of the maximum response with the limit states to assess the damage state, 4) regression analyses on the cumulative percentage of buildings in each damage state for the intensity measure level typical of each seismic input to derive the statistical parameters of the fragility curves. Different modelling hypotheses were tested and verified in order to robustly represent beam-column connection failure, which was one of the principle causes of total collapse of such structures in recent events. It has been found that the choice of a simplified dynamic analysis that does not consider vertical acceleration could be inadequate for the representation of the connection failure, and therefore an investigation of the sensitivity of collapse fragility curves to the modelling assumptions is presented. The fragility curves have then been applied to predict preliminary damage distributions for a magnitude 6.5 (Mw) earthquake in South-Eastern Tuscany.

INTRODUCTION

In May 2012 the seismic events in Northern Italy revealed the seismic vulnerability of precast industrial buildings typical of Italian building practice. The seismic inadequacy of precast industrial buildings was also observed in the Adana-Ceyhan (1998), Kocaeli (1999) and Duzce (1999) earthquakes, in which 60% of the industrial facilities suffered substantial damage (Senel and Kayan, 2010). Arslan *et al.* (2005) indicated that the main causes of the poor response of Turkish precast buildings were the insufficient strength, lateral stiffness, and ductility (especially required by the

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fixed-base crown-hinged structural scheme, and by the high inter-storey height) and the incompatibility between the precast elements. In the post-earthquake inspections, plastic hinge formation at the base of the columns and overturning of the beams from the column support were observed, as a result of the inadequacy of the column cross-sections especially in the out-of-plane direction and the insufficiencies in detailing in the beam-column connections.

The seismic performance of Italian precast buildings has been shown to be equally inadequate (Magliulo *et al*., 2013) considering that, despite the observed moderate ground motion with respect to the intensity of the Turkish events, excessive damage occurred, resulting in substantial direct and indirect losses. The dominant failure mechanism has been the loss of support of the precast beams, due to the inability of the beam-column connection to sustain the seismic displacement demand associated with the structure's high flexibility, and that fact that these connections are often designed only to transfer horizontal forces by friction or through steel dowels. In fact, the first specific seismic design regulation for this typology was only introduced in Italy in 1987 and an adequate seismic hazard zonation of the Italian territory did not occur until 2004, leading to several structures that were designed before the enforcement of modern seismic regulations.

The structural deficiencies common to this class of buildings have thus emphasised the need for risk assessment to estimate potential damage for future earthquakes, given that there seems to be a very limited experience in the current literature on the fragility assessment of such structures with respect to traditional cast-in-place buildings. For this purpose, a set of fragility curves describing the probability of exceeding a certain number of damage limit states given the intensity of the ground motion have been derived and applied to predict preliminary damage distributions for a scenario earthquake in Southern Tuscany.

An analytical method was used in the seismic fragility study presented herein, because it allows the development of sensitivity studies to evaluate the influence of specific aspects, such as connection failure, on the overall structural response and it permits the vulnerability assessment of classes of buildings for which reliable statistical damage data from past events might not be available. Hundreds of structures were produced through Monte Carlo simulation to represent the "as built" in Italy, and subjected to ground motion records using nonlinear dynamic analysis. The building stock was classified into a number of typologies according to the geometric configuration and the design levels, which are correlated with the construction age. A pushover curve was performed for each frame and a set of limit states were estimated according to both the strain levels and the maximum top drifts. The capacity of the connections was calculated and connection failure was accounted for as a collapse limit state, estimated with both static and dynamic methods.

For the damage scenario risk analysis, the OpenQuake-engine developed by the Global Earthquake Model (GEM) for seismic hazard and risk analysis (Silva *et al.,* 2013; Pagani *et al*., 2014) was used. A single seismic event was simulated, considering a fault located in Southern Tuscany, and the distribution of damage for an industrial building stock in the region of interest was assessed.

An overview of the steps to perform the risk analysis is shown in Figure 1.

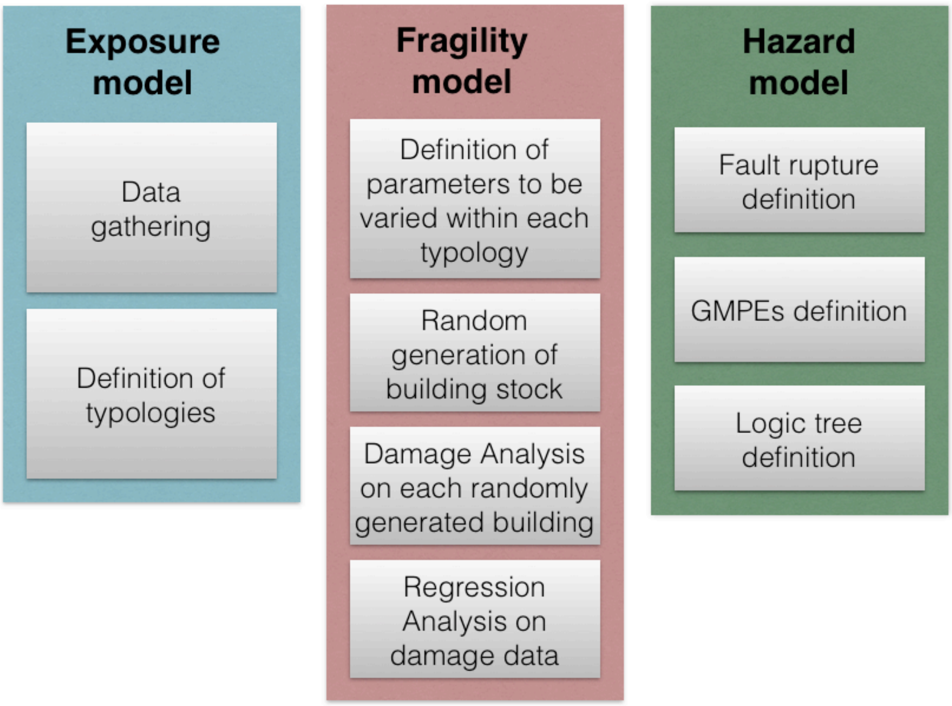


Figure 1 Overview of the risk analysis process

FRAGILITY CURVES METHODOLOGY

The analytical methodology followed for the fragility curve derivation can be summarised in the following steps:

Generation of precast RC structures: The characteristics of this type of structure were analysed according to a set of building typologies with homogeneous attributes. For each category, a probabilistic distribution for each geometric and material parameter was defined.

Design, numerical modelling and damage analyses: The randomly sampled single structures were designed according to the admissible tension method and the provisions of the code in place at the time of construction (either pre- or post-1996). The classification of the building stock and the design procedure outlined by Bolognini *et al.* (2008) was adopted. Each generated structure was modelled and subjected to nonlinear static and dynamic analyses using the structural analysis software OpenSEES (Mazzoni *et al.*, 2007). The static analysis served the purpose of defining a capacity curve, and the associated limit states for three levels of damage. A dynamic analysis was performed for the suite of selected accelerograms and the maximum response of each structure in terms of maximum top drift was extracted. These results (displacement demand) were compared with the limit states (displacement capacity), in order to allocate each structure into a damage state.

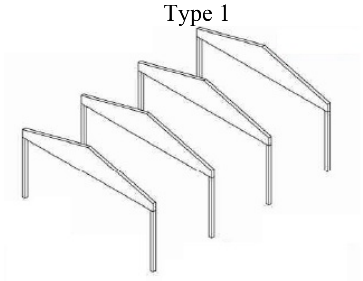
Fragility curves derivation: The distribution of buildings in each damage state for each ground motion record was used to derive the statistical parameters of each fragility function. This regression analysis was carried out using the Maximum Likelihood Method.

Generation of precast RC structures

The first step for the generation of the building stock is the classification of buildings with common characteristics into typologies. Two main factors were considered in the definition of the typologies: the year of construction, and thus the seismic design code to which they should conform to, and the geometric configuration. The building stock subdivided between pre-code and low-code design, for the structures that date back to before and after 1996, respectively. A first (unsatisfactory) attempt to include provisions for the design of reinforced concrete precast structures in seismic zones was done in 1987 and 1996, but an appropriate design code including a specific chapter concerning precast structures was introduced only in 2003 (OPCM 3274) and only legally enforced in 2009 following the release of the NTC (2008), and thus the vast majority of buildings constructed after 1996 can be defined as “low-code”.

Two main categories were selected to represent the most common geometric configurations of the Italian precast industrial building stock, as described in Table 1. The first typology, more traditional and frequently used, consists of a series of one-storey basic portal frames. Each portal is comprised of two or more columns fixed at the base and a saddle roof beam usually simply supported by the columns or with shear resistant connections. The second common typology consists of one-storey frames linked by perpendicular straight beams, which carry the main roof beams or directly support the large span slab elements. This classification was developed based on information from the available Italian literature, from precast element producers, design reports from Calvi *et al.* (2006, 2009), and from direct surveys of 650 warehouses located in Tuscany, Emilia Romagna and Piedmont regions, conducted by the Seismic Risk Prevention Area of Tuscany Region and by the Structural Analysis Section, at the EUCENTRE. The results of the investigations were used to derive the statistical distributions of the geometric properties for each building typology. Some information required for this study (e.g. material properties and design loads) could not be extracted from the surveys, and was instead extracted from the literature or estimated by expert opinion.

Table 1 Classification of the building typologies used in this study

Structural configuration	Code level	Design lateral load*	Id code
Type 1 	Pre-code	2%	T1-PC-2
	Low-code	4% 7% 10%	T1-LC-4 T1-LC-7 T1-LC-10
Type 2 	Pre-code	2%	T2-PC-2
	Low-code	4% 7% 10%	T2-LC-4 T2-LC-7 T2-LC-10

* as a percentage of the weight of the building

The second step for the generation of the building stock is the statistical characterisation of the material and geometric parameters to be varied within each building typology.

The concrete and steel characteristic compressive and tensile strengths were sampled according to the construction age. The material properties indicated by the reference codes (DM 3-03-1975, DM 16-01-1996) were used only to design each structure. Table 2 summarises concrete and steel characteristic strengths (Rck and fsk) used for the design.

Table 2 Material properties randomly sampled for the simulated design of the building stock

	Pre-code (all cases)	Low-code (all cases)
R_{ck} [MPa]	35, 40, 45, 50	45, 50, 55
fsk [MPa]	320, 380	380, 440

In order to detect the difference between the actual characteristic compressive and tensile strength of the materials to be used in the modelling and the design values indicated in the codes, the results from an experimental campaign on the actual mechanical properties of the materials were adopted (Verderame *et al.,* 2001). Unfortunately, this information was not available for all the materials, and therefore overstrength coefficients were applied to the design values of those materials for which statistical studies were not available. Table 3 provides the values used in the modelling process.

Table 3 Material properties randomly sampled to model the industrial building stock.

	Pre-code (all cases)		Low-code (all cases)
R_{ck} [MPa]	35, 40, 45, 50 multiplied by γ_c		45, 50, 55 multiplied by γ_c
	Smooth rebars	Ribbed rebars	Ribbed rebars
fsk [MPa]	$\mu = 356$ $\sigma = 67.8$	380 multiplied by γ_s	380, 440 multiplied by γ_s

The probabilistic distributions for the characterisation of the geometric properties of pre-cast structures were derived by the field data or by expert opinion when the probability distributions showed a poor fit with the field data [Bolognini, personal communication, 2012]. All the parameters are described in Table 4, minimum and maximum values were also defined to avoid sampling of unrealistic parameters.

Table 4 Geometric dimensions randomly sampled for the generation of the building stock: μ and σ are the mean and standard deviation of the associated normal distribution

Structural Configuration		Distribution	μ	σ	min	max	Source
Type 1	L_{beam} [m]	Lognormal	2.7	0.3	8	30	Tuscany database
	$L_{intercol}$ [m]	Lognormal	9	1	8	10	expert opinion
	H_{col} [m]	Lognormal	1.9	0.2	4	12	Tuscany database
Type 2	L_{beam} [m]	Normal	8.7	2	8	10	Tuscany database
	$L_{intercol}$ [m]	Normal	16.5	3.7	10	25	Tuscany database
	H_{col} [m]	Normal	6.5	1.3	4	11	Tuscany database

Design, numerical modelling and damage analyses

The structures were designed in compliance with the Italian codes DM 3-03-1975 and DM 16-01-1996, applied respectively to the pre-code and low-code typologies. The allowable tension method was employed for the design and the seismic action was applied as static horizontal loads, corresponding to a fraction of the total weight of the building, 2% for the pre-code design, and 4%, 7% and 10% for the low-code design, as a function of the seismic region where the structure is located (zones III, II and I, respectively). The full design methodology is thoroughly described in Casotto (2013).

The type of connection considered in this study consists of a simple corbel supporting the roof beams. Connections relying on friction are allowed in the pre-code design, but not in the low-code, as stated in the DM 03-12-1987 regulation. Thus, in pre-code buildings the capacity of the connections was calculated simply as the friction resistance. The definition of the friction coefficient, dependent on the material of the supporting surface, is a controversial matter, as in the literature it is found to vary between 0.6 and 0.9, whilst in some experimental campaigns lower values (between 0.1 and 0.5) have been observed (Magliulo *et al.,* 2011). Due to this uncertainty, a decision was made to adopt two fixed values of 0.2 and 0.3 and to estimate the effect of this parameter on the final fragility curves. For low-code structures, steel bolts or bars can be used to reinforce the connection. The resistance of the steel-concrete interaction is assumed to be the smallest between the shear resistance of the steel and that of the concrete.

The randomly generated structures were modelled in a 2D environment using the software OpenSEES, considering both geometric and material nonlinearities. Force-based fibre finite elements were used for the columns, with a mesh of 220 fibres and 4 integration points, and elastic elements for the pre-stressed beams, which are assumed to remain elastic given the pinned connection with the columns. Scott *et al.* (1982) and Menegotto and Pinto (1973) models were selected to represent concrete and steel nonlinear behaviour. A tangent proportional damping model was used in the dynamic analysis with a damping ratio of 2%.

Two types of nonlinear analyses were conducted: pushover analysis and nonlinear dynamic analysis. A pushover analysis was computed to estimate a set of limit state global drifts. Three damage states are identified in this case: none/slight damage, moderate/extensive damage and complete damage. The first limit state is characterised by the member flexural strength attainment, at first yielding of the reinforcement steel in the columns. For the definition of collapse the ultimate strain limits of concrete and steel (ε_c=0.005 and ε_s=0.015 as proposed by Crowley *et al.,* 2004) could not be used, as they lead to excessive levels of top displacement (almost 30 cm or 4% of top drift) and 3% inter-storey drift was thus set as the collapse limit state, as experimentally verified by Bellotti *et al.* (2009). The collapse limit state is related also with the beam loss of support condition, identified when the shear demand in at least one column exceeds the connection capacity. The connection collapse is evaluated with two different approaches: the capacity of the connection is computed either assuming a constant force, proportional to 40% of the axial load (Bolognini *et al.*, 2008) or as a function of the vertical component of the ground motion records.

Two types of nonlinear dynamic analysis were performed, considering only the horizontal input of the records or applying both the horizontal and the vertical acceleration, thus allowing the calculation of the connection capacity as a function of the vertical excitation. The maximum top drift of the structure was compared with the drift limits defined with the pushover analysis to allocate each frame into a damage state.

Seventy accelerograms were extracted from the PEER database, with magnitude ranging between Mw 4 and 6.5 and distances from 0 to 30 km, as indicated by the disaggregation analysis in Northern-Central Italy for the 2475 years return period hazard and for Sa(T=1.5sec), conducted by Iervolino *et al.* (2011). The accelerograms selected were all recorded on soft soil, which is the soil class at the site of interest according to the EC8 classification (Borzi and Di Capua, 2011).

Fragility curves derivation

The results from the nonlinear dynamic analyses were assembled in the Damage Probability Matrix, which contains the percentage of sampled structures in each damage state, for a set of intensity measure levels representing each ground motion record. Then, the cumulative fractions of structures in each damage state for each intensity level, expressing the probability of exceeding that damage state, were estimated and fitted with a lognormal cumulative distribution function. The regression analysis to find the parameters of the lognormal functions was carried out using the Maximum Likelihood Method.

In the first type of dynamic analysis, that hypothesises a 40% axial load reduction to account for vertical acceleration, a simplified methodology was employed to include the Probability of Exceedance (PoE) of connection failure. This latter probability was computed separately and then added to the PoE of flexural failure to obtain the total PoE of the ultimate limit state as shown in Figure 2. In this way it is possible to describe the two trends visible in the collapse fragility curves (Figure 2): a first trend when the demand is not enough to attain flexural capacity and collapse is due only to connection failure, and a second trend when flexural failure starts contributing to collapse, while connection failure is limited by the maximum percentage of frames featuring the connection collapse mechanism. The total PoE of collapse is the sum of the two conditional probabilities as expressed in Eq 1.

$$PC = P(C_{Connection})P(\text{Connection mechanism}) + P(C_{Flexure})(1-P(\text{Connection mechanism})) \qquad (1)$$

where $P(C_{Connection})$ and $P(C_{Flexure})$ are the probability of connection and flexural collapse respectively, and P(Connection mechanism) is the probability of presenting connection failure mechanism. Two values for the friction coefficient were considered in the collapse fragility curves (0.2 and 0.3). It is apparent from Figure 2 that there is a relevant difference when assuming 0.2 or 0.3 as the friction coefficient, due to the large sensitivity of the maximum percentage of structures presenting connection collapse mechanism to this factor.

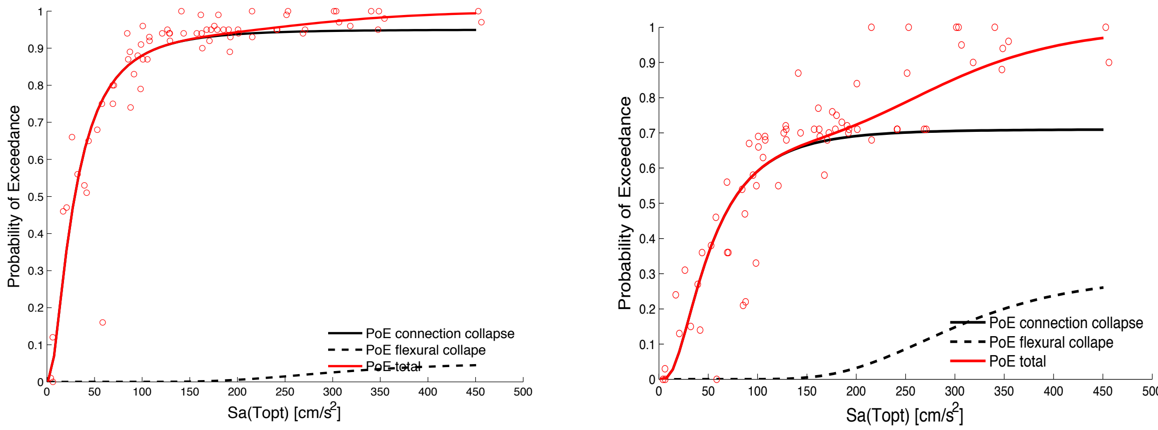


Figure 2 Total probability of exceedance for the collapse limit state as the sum of the two conditional probabilities of flexural collapse and connection collapse, T1-PC-2 with 0.2 (left) and 0.3 (right) friction coefficient.

In the second approach, where the connection capacity is affected by the vertical component of the acceleration, the transition between collapse fragility curves calculated using different levels of friction coefficient is smoother, as shown in Figure 3. The issue of the high impact of the friction coefficient was thus overcome by assessing connection collapse when the shear demand time history crossed the connection resistance time history. A unique regression analysis could be performed with the data comprising both flexural and connection failure, without the need to compute the PoE of connection and flexural collapse separately.

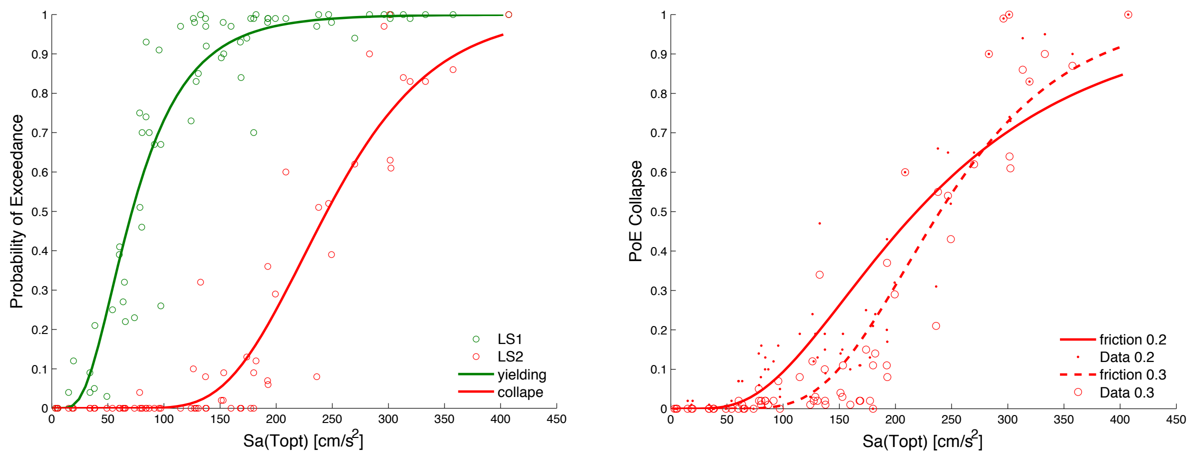


Figure 3 Probability of exceedance considering the vertical acceleration component. T1-LC-4 yielding and flexural collapse fragility curves (left), flexural and connection collapse curves (right) for the two friction coefficients.

In the fragility curve derivation various intensity measures were considered and compared, using the R^2 coefficient to quantify the correlation between the intensity levels and the cumulative percentage of frames for each limit state. A very large dispersion in the results using the peak ground acceleration (PGA) was found and a considerably increased correlation with damage using Spectral Acceleration (Sa). The period of vibration for which the Sa is computed and its influence on the variability of the fragility curves was thus investigated in the correlation analysis shown in Figure 4, where the coefficient R^2 is computed for a set of elastic periods (Silva *et al.,* 2013). The mean R^2 curve (the mean between the first limit state and the second limit state R^2 curves) is also presented and the optimal periods (the periods corresponding to the maximum correlation for each limit state and for the mean curves) are indicated with vertical lines. The spectral acceleration at the mean optimal period of vibration Sa(T_{opt}) was selected to be used as the intensity measure, because it increases the performance of the second limit state curve (which provides information about the damage with more influence on the losses) without substantially compromising the first limit state correlation.

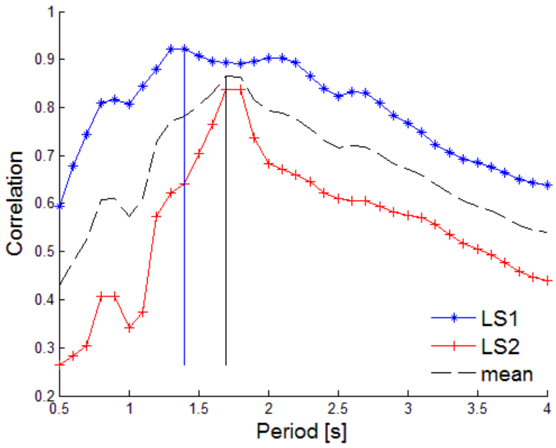


Figure 4 Correlation coefficient as a function of the period of vibration of S_a, for T1-PC-2 model.

RESULTS

The results derived with the first approach, using the hypothesis of 40% axial load reduction to account for vertical acceleration, and with the second approach, using vertical acceleration in the dynamic analysis are shown in Table 5 and Table 6, respectively. In order to be conservative, the final fragility curves were computed using the 0.2 friction coefficient.

Table 5 Results for the fragility curves hypothesizing 40% axial load reduction. Median (m) and logarithmic standard deviation (ζ), and coefficient of correlation (R^2) using friction coefficient of 0.2 and an intensity measure of Sa(Topt) in cm/s^2.

Typology	T_{opt}	LS1		R_{LS1}^2	LS2 flexure		$R_{LS2\ c}^2$	LS2 connection		$R_{LS2\ f}^2$	P cnn*
		m	ζ		m	ζ		m	ζ		
T1-PC-2	1.7	78.35	0.56	0.89	297.61	0.33	0.84	28.91	0.89	0.85	0.95
T1-LC-4	1.8	76.02	0.50	0.91	253.75	0.30	0.90	159.56	0.67	0.81	0.38
T1-LC-7	1.7	96.04	0.49	0.87	316.85	0.31	0.82	79.19	0.75	0.78	0.76
T1-LC-10	1.3	166.71	0.44	0.91	600.68	0.33	0.83	90.37	0.72	0.82	0.96
T2-PC-2	1.8	66.67	0.51	0.92	244.51	0.27	0.89	30.19	0.79	0.90	0.96
T2-LC-4	1.8	63.94	0.52	0.92	230.57	0.28	0.90	98.39	0.66	0.78	0.46
T2-LC-7	1.7	77.08	0.50	0.87	266.25	0.30	0.85	61.53	0.69	0.80	0.87
T2-LC-10	1.3	112.25	0.63	0.87	660.97	0.77	0.69	68.15	0.69	0.80	0.90

*Probability of presenting connection collapse mechanism

Table 6 Results for the fragility curves considering the vertical acceleration. Median (m) and logarithmic standard deviation (ζ), and coefficient of correlation (R^2) using friction coefficient of 0.2 and an intensity measure of Sa(Topt) in cm/s^2.

Typology	T_{opt}	LS1		R_{LS1}^2	LS2		R_{LS2}^2
		m	ζ		m	ζ	
T1-PC-2	2.2	37.33	0.76	0.88	62.21	1.16	0.84
T1-LC-4	1.8	70.82	0.55	0.90	218.74	0.59	0.83
T1-LC-7	1.3	141.26	0.52	0.91	251.56	0.80	0.83
T1-LC-10	0.9	177.69	0.49	0.93	197.33	0.65	0.92
T2-PC-2	2.1	46.31	0.65	0.92	65.87	0.93	0.88
T2-LC-4	2.1	55.76	0.47	0.94	171.86	0.55	0.84
T2-LC-7	2.1	54.74	0.53	0.90	110.01	0.89	0.82
T2-LC-10	0.9	149.73	0.55	0.88	177.37	0.77	0.88

It is clear from the comparison between collapse fragility curves with and without consideration of connection failure that connection collapse is an important issue that cannot be neglected and has to be addressed carefully. Moreover, the chosen method used to evaluate this particular failure mechanism can influence the results considerably, as explained in detail in Casotto *et al.* (2014). In the first methodology, connection failure is estimated based on constant column capacity and constant connection capacity (proportional to the initial axial load reduced by 40%). This method does not consider the possibility that the column capacity could be very close to the connection capacity and it could easily exceed it during an acceleration time-history, where both the capacities vary with the vertical acceleration. Furthermore, when following this methodology the number of frames with connection collapse mechanism is very sensitive to the friction coefficient, as demonstrated by the abrupt change in the fragility curves caused by a small variation in this parameter (see Figure 2). Given the large uncertainty in the friction coefficient definition and the consequent impact on the fragility curves, this method seems to be unreliable, and thus the fragility curves derived with the second method were used for the risk assessment.

SCENARIO RISK ASSESSMENT

For the damage scenario risk analysis, the OpenQuake-engine developed by the Global Earthquake Model (GEM) for seismic hazard and risk analysis (Silva et al., 2013; Pagani *et al*., 2014) was used. A single seismic event was simulated, considering a single fault rupture located on the Appenines in southern Tuscany. A normal fault which is capable of producing a magnitude 6.5 (Mw) earthquake was selected and the rupture length was computed according to Wells and Coppersmith (1994) as a function of the selected magnitude. The fault rupture is characterised by its geometry (trace, dip angle, upper and lower seismogenic depths) by the rake, which defines the type of fault, and by the magnitude the fault is capable of. These pieces of information were extracted from Basili *et al.* (2008). Two thousand ground motion fields for Spectral Acceleration at the optimal periods of the eight industrial building typologies were computed using the spatial correlation model developed by Jayaram and Baker (2009), and combined with the derived fragility models to assess the distribution of damage for the industrial building stock. Figure 5 represents the median ground motion field for rock obtained from the ground shaking analysis.

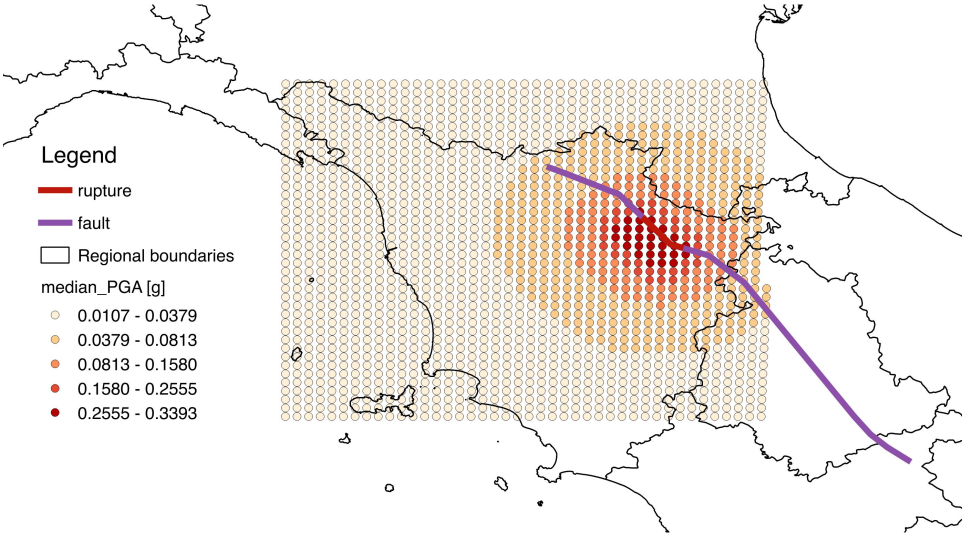


Figure 5 Median ground motion field for Sa(1s) units of g

The exposure model was constructed using the same building dataset developed from direct surveys of many industrial areas in Tuscany for the statistical characterisation of the geometric parameters of the building population. The dataset provides building by building information about location, date of construction, geometric and structural configuration.

By combining these three components (ground shaking at each location, fragility curves and distribution of industrial building) the mean and standard deviation of the expected damage distribution have been calculated for each building in the dataset. The total distribution of damage is depicted in Figure 6, while Figure 7 represents the spatial distribution of the number of collapsed buildings in the region of interest.

Figure 6 Distribution of damage for the buildings in the exposure model

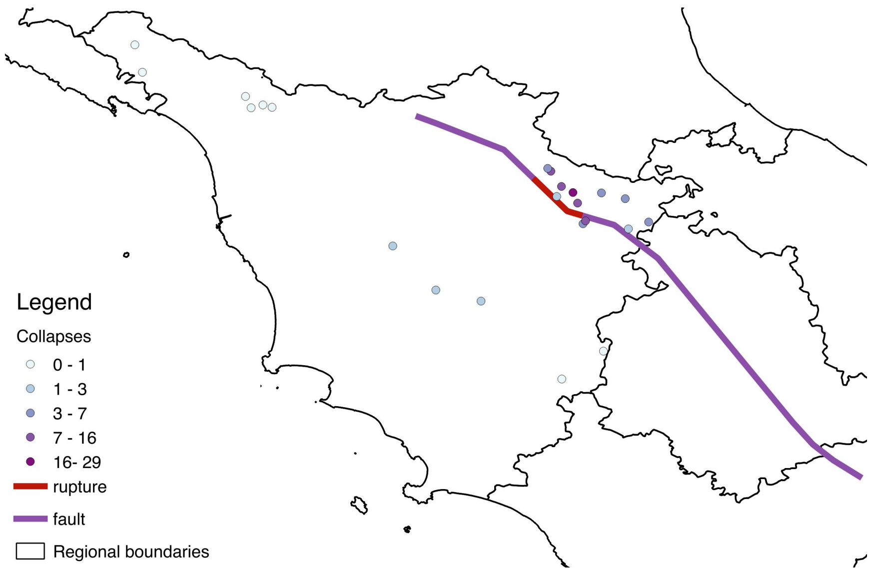


Figure 7 Number of collapses for the buildings in the exposure model

CONCLUSIONS

The aim of this study was to develop fragility functions for Italian RC precast industrial buildings, which are useful for earthquake loss assessment, such as the one described in the present work. Given the poor experience in the current literature in the fragility assessment of such structures, with respect to the good knowledge of the behaviour of regular cast-in-place buildings, special attention was given to the simplifications usually adopted in structural modelling.

For this reason, the main characteristic of these structures, (i.e. weak beam-column connection) which has been the principle cause of collapse in recent seismic events, was explicitly considered in the development of the fragility curves. Different modelling methods were studied: the frames were tested against horizontal accelerograms only in their main direction and the influence of the vertical acceleration was simulated both with a static simplified method, by reducing the axial load by 40% (Bolognini *et al.,* 2008) and considering the vertical input of the ground motion records.

The results of the fragility study have highlighted the importance of considering connection collapse and the need to represent it robustly. In fact, the analyses have shown some drawbacks in the use of the simplified method: the results demonstrated a higher sensitivity to the value of friction coefficient used to compute the capacity of the connection than to the ground motion intensity. For this reason, the second set of fragility curves were adopted for the scenario damage assessment. The results of this case-study indicated that 38% of the industrial building stock in the region of interest is likely to suffer extensive damage in a magnitude 6.5 (Mw) earthquake triggered by a nearby fault in Southern Tuscany. The considerable percentage of damaged buildings underlines the importance of an accurate vulnerability study of such structures and the need to implement risk mitigation actions on these structures, including structural retrofitting or strengthening interventions.

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