SEISMIC FRAGILITY OF PRECAST RC BUILDINGS IN PORTUGAL

R. Sousa(1), N. Batalha(2), H. Rodrigues(3), V. Silva(4), P. Fernandes(5), H. Varum(6)

(1) *Assistant Researcher, CDRSP, School of Technology and Management (ESTG), Polytechnic Institute of Leiria, Leiria, Portugal, romain.r.sousa@ipleiria.pt*

(2) *Phd Student, CONSTRUCT-LESE, Faculty of Engineering (FEUP), University of Porto, Porto, Portugal, up201809163@fe.up.pt*

(3) *Associated Professor, RISCO, Civil Engineering Department, University of Aveiro, Aveiro, Portugal, hrodrigues@ua.pt*

(4) *Associated Professor, RISCO, Civil Engineering Department, University of Aveiro, Aveiro, Portugal, vitor.silva@globalquakemodel.org*

(5) *Coordinating Professor, CERIS, School of Technology and Management (ESTG), Polytechnic Institute of Leiria, Leiria, Portugal, paulo.fernandes@ipleiria.pt*

(6) *Cathedratic Professor, CONSTRUCT-LESE, Faculty of Engineering (FEUP), University of Porto, Porto, Portugal, hvarum@fe.up.pt*

Abstract

Vulnerability functions are analytical functions that describe, in a probabilistic manner, the performance of a given typology as a function of the seismic intensity, and are a key parameter for the seismic risk analysis of structures. This paper describes a study carried out to derive, for the first time, fragility functions representative of the Portuguese reinforced concrete precast buildings. Such goal was achieved following an analytical methodology considering the result of hundreds of nonlinear static analysis, whose building models reflect both mechanical and geometrical characteristics of the Portuguese industrial building stock. Considering the specificities of this typology, namely in what regards the behavior of the connections between the structural members, a recently developed macro-element was employed enabling to explicitly simulate the contribution of the friction and dowel mechanisms. The results are analyzed in view of both structural and nonstructural limit states.

Keywords: reinforced concrete, precast buildings, non-linear static analyses, seismic fragility, risk assessment

1. Introduction

The recent earthquakes highlighted the vulnerability of industrial buildings built with reinforced concrete (RC) precast systems [1]–[4]. Senel & Kayhan [5] studied the properties of 65 precast existing buildings in Turkey, pointing that the parameters that govern seismic behavior, damage and collapse probabilities of precast buildings are lateral stiffness and ductility. Babič & Dolšek [6] referred that the 2009 L'Aquila and the 2012 Emilia-Romagna earthquakes revealed the seismic fragility of Italian industrial buildings, referring that many buildings built in the last decades have collapsed, totally or partly. Magliulo *et al.* [4] referred that in the Emilia-Romagna region about 5 billion euros on indirect losses and 1 billion euros on direct losses were estimated. Wilson *et al.* [7], based on experimental and analytical analysis, have highlighted that the global seismic performance of the precast building are mainly connection dependent and that the solution should be related with increasing the displacement capacity of the connections.

Casotto *et al.* [8] evaluated the seismic vulnerability of an Italian RC precast building. To achieve that, a set of fragility curves have been derived in order to evaluate the probability of exceeding a number of damage limit states given the intensity of the ground motion and to predict damage distributions for an earthquake scenario in Tuscany, Italy. Fig. 1 represents an overview of the steps to perform the risk analysis.

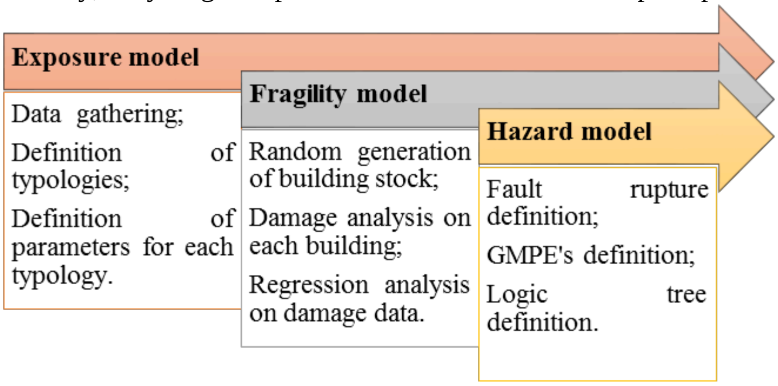


Fig. 1 – Risk analysis process

Rossetto & Elnashai [9] refers to the vulnerability curves relate the probability of exceed multiple damages states to a parameter of ground motion severity. Beilic *et al.* [10] mentioned the seismic vulnerability as a measure of how likely is a building to suffer damage for a given intensity of ground shaking, and it can be mathematically formulated by fragility curves. The fragility curves could be obtained by four different methods: *i)* empirical curves; *ii)* expert opinion-based curves; *iii)* analytical curves; and *iv)* hybrid curves. Empirical curves are obtained through the damage distribution observed in post-earthquake surveys, the expert opinion-based curves are, as the name implies, based in the opinion of expert professionals, while analytical curves are obtained through structural analysis of numerical models. Hybrid curves are basically those that combine any of the previous methods in order to fill in the gaps that some of them may have [11]. Silva *et al.* [12] mentions the analytical ones as a methodology to overcome the lack of post-earthquake data.

As stated before, the development of seismic risk studies requires the knowledge of the building characteristics of a given typology in order to characterize its main seismic vulnerabilities. The information of geometric and mechanical parameters allow the definition of reliable numerical models that can be used to derive fragility functions capable of describing the relation between the seismic intensity and building limit states. Contrarily to what is observed for residential buildings, limited information is available in what regards the properties of precast buildings. In Portugal, only the study developed for the European Commission in 2013 [13], presents a general description of the main typologies of the industrial building stock, but whose information is insufficient to enable the development of representative numerical models. In this sense, a study with the objective of characterizing the Portuguese industrial RC precast building was developed by [14]. The outcome of this study allowed to characterize this typology from a structural point of

© The 17th World Conference on Earthquake Engineering

view and was used to develop reliable numerical models in order to derive seismic vulnerability functions. Considering the similitude of the Portuguese properties with respect to the buildings found on other Mediterranean countries, namely Italy and Turkey, the fragility curves presented in this manuscript can be used to update current risk models and contribute to assess and mitigate the seismic vulnerability of this typology of buildings in this seismic prone region.

2. Definition of the buildings portfolio

The population of synthetic buildings was generated based on the properties of the Portuguese RC precast industrial buildings collected by Vitorino *et al.* [14], based on the Monte Carlo sampling of the information gathered from dozens of existing buildings. This study also indicates that some properties of the buildings are somehow correlated with others, namely with the period of construction and span length of the main beams.

Considering the larger variability in the building geometric properties, when compared to the ones observed in conventional residential buildings, a total of 1000 buildings were considered in the numerical study. Some variables served as the basis for the generation of the buildings that basically constitute the general layout of the buildings. Are they: *i)* number of bays in longitudinal direction; *ii)* number of bays in transverse direction; *iii)* number of storeys; and *iv)* building with or without claddings. Then a set of the independent variables was established: *i)* year of design project; *ii)* columns height; *iii)* span length in transversal direction; *iv)* columns height-to-length; and *v)* corbel span. In the Table 1 is presented the information of statistics adopted in the numerical model generation, regarding the fixed and independent variables, namely the mean value, the standard deviation, the minimum and the maximum value in each variable.

Noting that there is no specific regulation for this type of buildings, and that most of the properties are directly related with the natural evolution of the construction, several parameters were defined based on regressions with respect to the year of construction. In addition, the area of the dowels and the longitudinal beams self-weight were, in turn, derived based on the secondary correlations with the longitudinal span.

Table 1 – Statistics of the fixed and independent variables adopted for the generation on the numerical models

Parameter	*Mean*	*STDV*	*Min*	*Max*
Number of bays in longitudinal direction	1-2			
Number of bays in transverse direction	8.2	4.8	1	29
Year of design project	1990	17.3	1960	2020
Span length in transverse direction [m]	7.6	2.5	4.2	12.5
Column height [m]	7.7	3.4	3.0	23.0
Column height-to-length ratio	18.1	4.0	6.9	28.9
Column length-to-width ratio	1.4	-	-	-
Corbel span [mm]	29.4	8.4	15.0	50.0

Regarding the expected concrete compressive strength, following the NP EN 1992-1-1 [15] it was assumed that the mean value is 8 MPa higher than the characteristic value, whilst for the reinforcement yield strength, the expected value was assumed to be 10 % higher that the corresponding characteristic value, as suggested by Priestley *et al.* [16].

Table 2 presents a summary of the statistics adopted in the numerical model generation, regarding the year dependent variables, namely the mean value, the standard deviation, the minimum and the maximum value in each variable. In the table, *Y* indicates the year of the construction, whilst *L* is the span length in the main direction (longitudinal) and *Lc* is the column length in meters.

© The 17th World Conference on Earthquake Engineering

Table 2 – Statistics of the dependent variables adopted for the generation on the numerical

Parameter	***Mean***	σ_E*	***Min***	***Max***
Span length in the longitudinal direction, L [m]	$\mu = 0.23Y - 437.6$	7.7	5.5	50.0
Longitudinal reinforcement ratio [%]	$\mu = 0.018Y - 34.7$	0.65	0.3	3.7
Transverse reinforcement ratio [%]	$\mu = 0.0043Y - 8.34$	0.17	0.05	0.95
Concrete strength [MPa]	$\mu = 0.52Y - 999$	6.9	12	50
Beam self-weight [kN/m]	$\mu = 0.27L - 0.33$	0.82	2.8	10
Dowel area [mm^2]	$\mu = 20.3L + 229$	405	50	1608
Length-to-width column ratio	$\mu = 1.885L_C + 0.377$	0.48	1	4

* σ_E - Standard deviations

3. Description of the numerical analysis

3.1. Model Assumptions

The seismic assessment of the buildings was carried out through nonlinear static analysis of 3D models along the two main directions of the buildings using the structural analysis software OpenSees [17]. Considering the observation that the damages tend to concentrate at the bottom of the columns or at the beam-to-column connections [4], [18][3], the columns were modeled with force-based nonlinear beam elements whilst both longitudinal and transverse beams were modeled with elastic elements. In terms of materials, the concrete was modeled with the Concrete01 model, based on the Kent-Scott-Park concrete model [19][20] whereas the longitudinal reinforcement was simulated with the Steel02 model, based on the Giuffre-Menegotto-Pinto [21] material model. The number of integration points vary based on the properties of the columns, and were defined following the recommendations by Sousa *et al* [22].

The performance of the buildings was assessed considering different connection properties, based on the data provided by Vitorino *et al.* [14]. These variations were simulated through a macro-element proposed by Sousa *et al.* [23] which is capable of accurately describe the main mechanisms identified in conventional beam-to-column RC precast connections, namely the friction between the different elements, the steel dowels (when present) and the neoprene pad (always considered in the model). This macro-element consists of a zero-length element, i.e., the end node of the beam and column have the same coordinates, that represent the contribution of the different systems through different springs that are aligned in series or in parallel, depending on the manner these are activated in real structures. The spring arrangement, illustrated in Fig. 2, is defined for both horizontal directions, while the rotations at the connection node are released. This model was considered for both longitudinal and transversal beam-to-column connections.

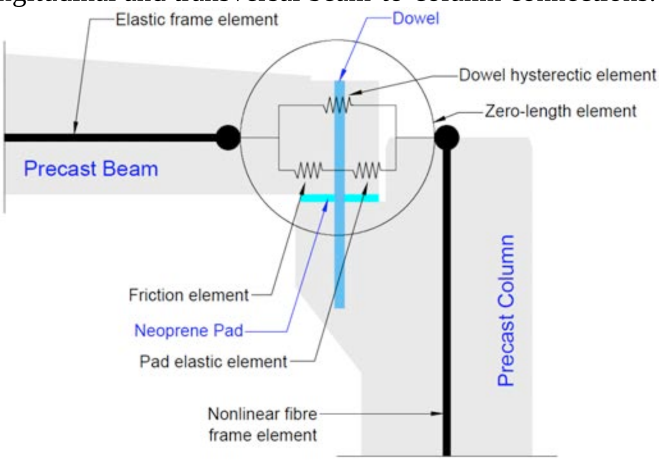


Fig. 2 – Numerical model adopted to simulate the behavior of the beam-to-column connections

© The 17th World Conference on Earthquake Engineering

In addition to the self-weight of the building an additional vertical load of 0.65 kN/m^2 was distributed on beams to simulate the self-weight of roof.

3.2. Definition of Limit States and Seismic Input

The seismic performance assessment of the buildings was carried out considering limit states associates with both structural and nonstructural elements. In terms of structural elements the analysis was focused on the response of both columns and beam-to-column connections, whilst for the nonstructural elements the damage was evaluated at the cladding connections. Each of these cases was assessed considering a damage control and a collapse prevention limit state following the thresholds recommended in past studies, as described in *Table 3*.

Table 3 – Limit states adopted for the different elements and performance level

	Structural limit states	
Columns	Collapse prevention	80 % drop F_{max}
	Damage limitation	80 % drop F_{max}
Connection	Collapse prevention	8 cm relative displacement [25]
	Damage limitation	3 cm relative displacement [25]
	Non-structural limit states	
Claddings	Collapse prevention	4 cm relative displacement between cladding connections [25]
	Damage limitation	1 cm relative displacement between cladding connections [25]

The seismic performance of every building was assessed considering a dataset with 250 records covering the Mediterranean region, which is consistent with the region under study. Considering the large period of vibration characteristic of this type of structures, all the records were scaled considering a maximum factor of 3.5 in order to reach seismic intensities capable of causing the structures to collapse. The scaled acceleration spectra are presented in *Fig. 3*.

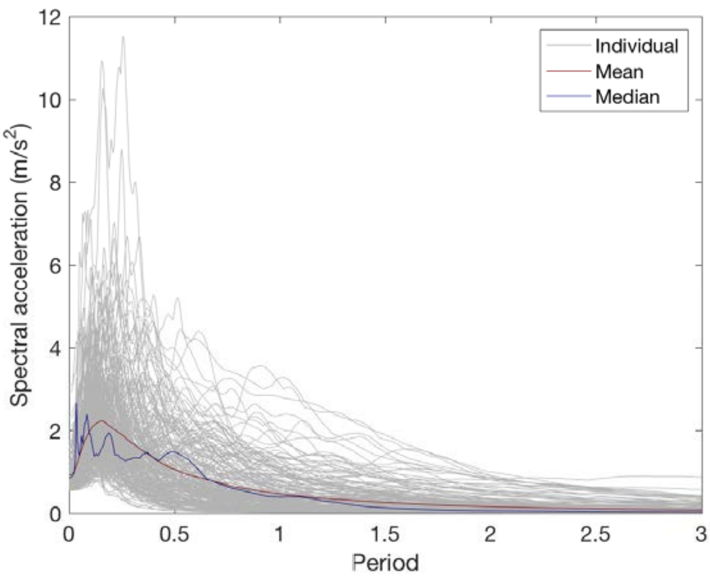


Fig. 3 - Scaled spectral accelerations considered as seismic input

© The 17th World Conference on Earthquake Engineering

4. Results

The fragility functions presented in the paper were derived from nonlinear static analysis carried out on 900 buildings, along the two main transverse directions, equally distributed across 3 different building classes (i.e. Pre-code, Moderate-code and Post-code), featuring different connection properties: (1) no dowels for pre-code buildings, (2) half of the buildings with dowels for moderate-code buildings and (3) all buildings with dowels for post-code buildings.

As illustrated in Fig. 4, in the absence of steel dowels, the seismic coefficient (defined as the ratio between the lateral strength and the self-weight of the building) is largely reduced to maximum values of about 0.1. Another relevant observation regards the large dispersion of the capacity curves, reflecting the variability (much higher when compared to conventional residential buildings) in the overall building geometry and cross-section of the vertical elements to accommodate different industrial requirements.

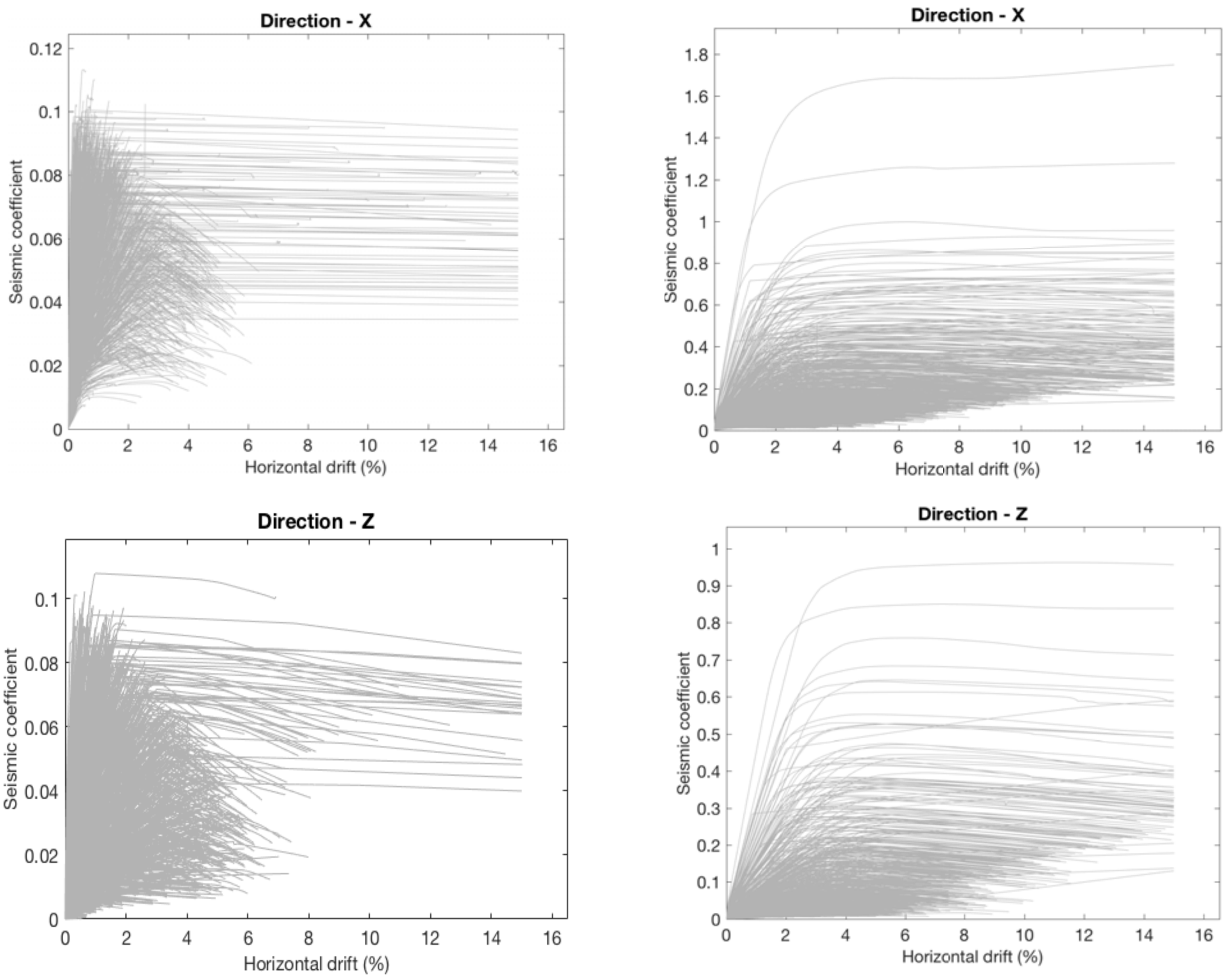
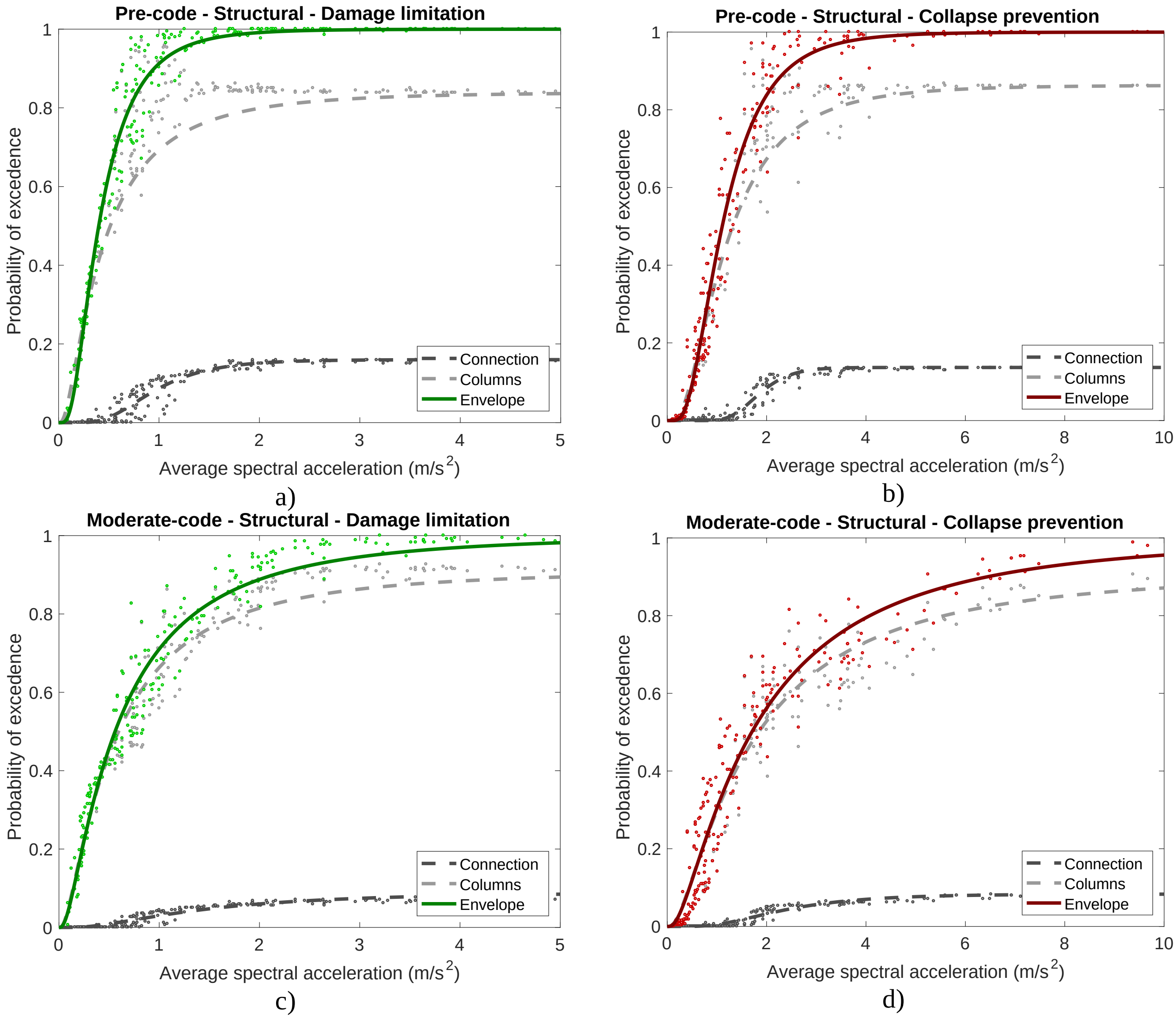


Fig. 4 – Relations between lateral drift and seismic coefficient along X- and Y- direction (top and bottom) and for models with (right) and without (left) dowels

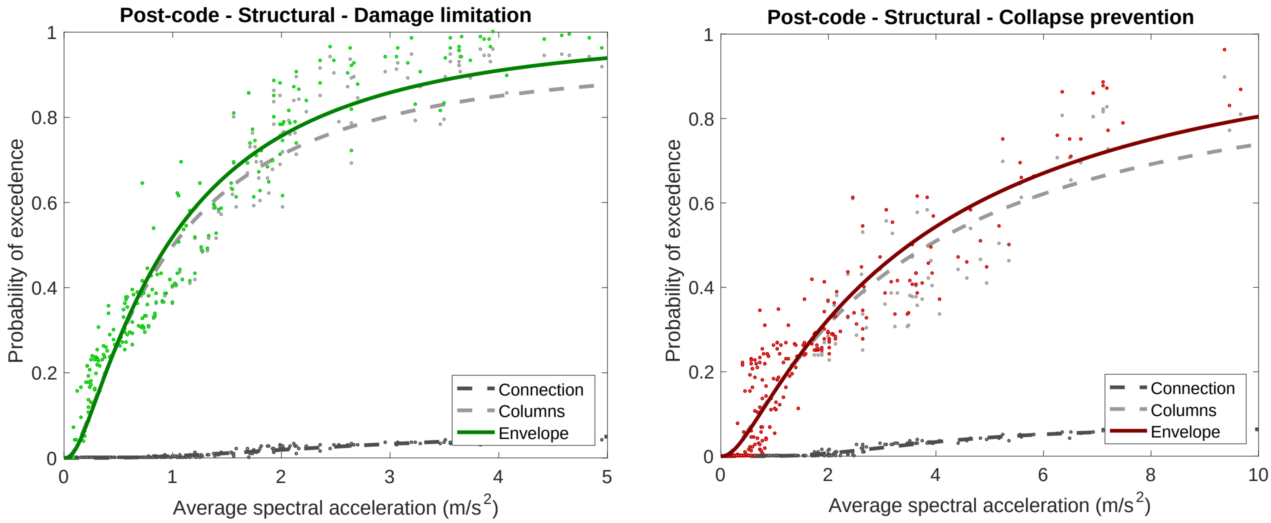
The results presented in Fig. 5 show the response of the individual industrial buildings together with the associated lognormal cumulative distribution associated with the structural limit states (i.e. damage limitation and collapse prevention) for the tree building classes, disaggregated in terms of conditioning mechanism (columns or connections). Each point in the plots represents the ratio of buildings within each class that reached a given limit state under analysis for each ground motion record, represented by the associated averaged spectral acceleration at the average elastic period of all the buildings (T=1.7 s).

© The 17th World Conference on Earthquake Engineering

The results confirmed that, in the presence of dowels, the response is generally controlled by the columns, while the failure at the connections is observed only in marginal cases (bottom plots in Fig. 5). On the contrary, in the absence of steel dowels (in all the Pre-code buildings and a fraction of the Moderate-code buildings), a larger number of buildings exhibit vulnerabilities at the connection level. The reason for this distinct behaviour relies on the reduced lateral strength of the columns analysed in this study. In the presence of particularly slender columns, the response tends to be governed by the columns' behaviour and the friction strength at the connection level is often enough to sustain the maximum shear forces developed in the columns. For the cases where the columns are more robust (with a local seismic coefficient higher that about 0.1), the friction at the connection in not enough to sustain the lateral loads and the beams experience large lateral displacements.



© The 17th World Conference on Earthquake Engineering



e) f)

Fig. 5 – Structural fragility functions for building models for a) Pre-code design for damage limitation state, b) Pre-code design for collapse prevention limit state, c) middle-code design for damage limitation state, d) middle-code design for collapse prevention limit state, e) Post-code design for damage limitation state, f) Post-code design for collapse prevention limit state

For what regards the non-structural components, all the typologies presented similar average spectral accelerations for both limit states and, therefore the results presented reflect the behaviour of the entire portfolio of buildings (see Fig. 6). The reduced variation observed among the different groups results from the columns slenderness (and hence the buildings initial lateral stiffness) being independent of the year of construction. Furthermore, given the low deformation level associated with these limit states, the damage in the cladding appears to be independent of the type of structural failure.

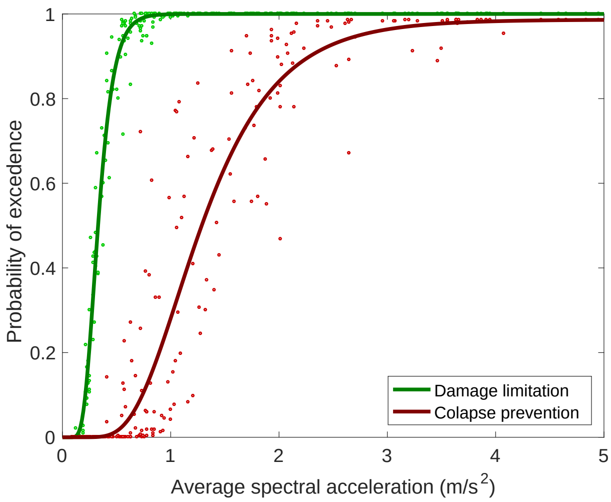


Fig. 6 – Claddings fragility functions for building models with pre- (left) and post- (right) code design

A summary of the statistics associated with each of the cases presented above is presented in *Table 4*.

Table 4 – Summary of the statistics associated with the fragility functions

Limit state	***Pre-code***		***Moderate-code***		***Post-code***	
	Mean	***Stdv***	***Mean***	***Stdv***	***Mean***	***Stdv***

© The 17th World Conference on Earthquake Engineering

Structural	Damage limitation	0.50	0.37	0.97	1.37	1.69	2.48
	Collapse prevention	1.33	0.87	2.92	4.07	7.35	13.78
Non-structural	Damage limitation	0.35	0.14	0.35	0.14	0.35	0.14
	Collapse prevention	1.50	0.77	1.44	0.68	1.40	0.63

Overall both damage limitation and collapse prevention limit states are attained for relatively small spectral accelerations, indicating the potential vulnerability of these structures, in particular those designed without steel dowels. Nonetheless, it is important to highlight that, as noted by Casotto *et al* [24], the definition of columns limit states are still object of debate and requires further investigation. Hence, it is possible that the results might be negatively affected by the consideration of inadequate drift limits. This observation is also supported by the results obtained in this study (Fig. 7), showing a large dispersion of the columns lateral drift measured at the instant where the columns reach its maximum capacity, here taken as reference.

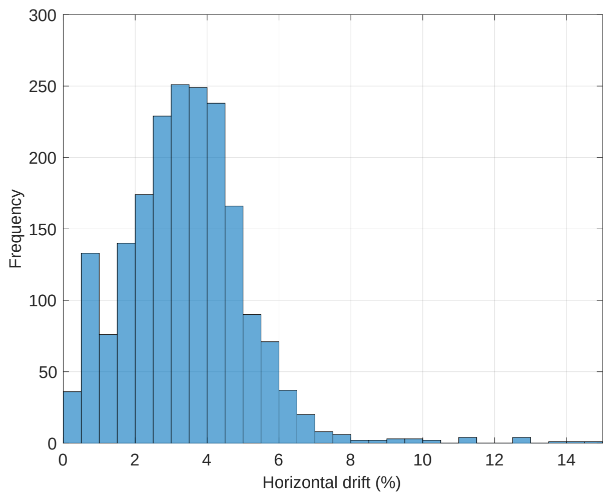


Fig. 7 – Distribution of columns horizontal drift at maximum shear capacity

5. Conclusions

Following the damage observed in past earthquakes, this paper presents the work carried out to assess the seismic fragility of the precast RC buildings. For that, a portfolio of hundreds of synthetic buildings was generated mimicking the material and geometric properties found in Portugal, which exhibit similar characteristics found in other Mediterranean countries. The numerical models were then subjected to nonlinear static analysis allowing to assess the main seismic vulnerabilities of this class of buildings. The results obtained showed that both structural and nonstructural damage are expected for low levels of spectral acceleration. This apparent vulnerability results from the high slenderness of the columns and the absence of steel dowels at the beam-to-column connections, more frequent in older buildings built before the introduction of seismic regulation. The results attained are naturally conditioned to the limit states adopted that, confirming previous indications, requires further investigation.

© The 17th World Conference on Earthquake Engineering

6. Acknowledgements

This work was financially supported by Project POCI-01-0145-FEDER-028439 – "SeismisPRECAST Seismic performance ASSessment of existing Precast Industrial buildings and development of Innovative Retrofitting sustainable solutions" funded by FEDER funds through COMPETE2020 - Programa Operacional Competitividade e Internacionalização (POCI) and by national funds (PIDDAC) through FCT/MCTES. This work was also supported by the Foundation for Science and Technology (FCT) - Aveiro Research Centre for Risks and Sustainability in Construction (RISCO), Universidade de Aveiro, Portugal [FCT/UIDB/ECI/04450/2020]. The second author acknowledged to FCT - Fundação para a Ciência e a Tecnologia namely through the PhD grant with reference SFRH/BD/139723/2018.

7. References

[1] A. Belleri, E. Brunesi, R. Nascimbene, M. Pagani, and P. Riva, "Seismic performance of precast industrial facilities following major earthquakes in the Italian territory," *J. Perform. Constr. Facil.*, pp. 1–31, 2014.

[2] H. Sezen and A. Whittaker, "Seismic Performance of Industrial Facilities Affected by the 1999 Turkey Earthquake," *J. Perform. Constr. Facil.*, no. February, pp. 28–36, 2006.

[3] L. Liberatore, L. Sorrentino, D. Liberatore, and L. Decanini, "Failure of industrial structures induced by the Emilia (Italy) 2012 earthquakes," *Eng. Fail. Anal.*, vol. 34, pp. 629–647, 2013.

[4] G. Magliulo, M. Ercolino, C. Petrone, O. Coppola, and G. Manfredi, "The Emilia earthquake: Seismic performance of precast reinforced concrete buildings," *Earthq. Spectra*, vol. 30, no. 2, pp. 891–912, 2014.

[5] S. M. Senel and A. H. Kayhan, "Fragility based damage assesment in existing precast industrial buildings: A case study for Turkey," *Struct. Eng. Mech.*, vol. 34, no. 1, pp. 39–60, 2010.

[6] A. Babič and M. Dolšek, "Seismic fragility functions of industrial precast building classes," *Eng. Struct.*, vol. 118, pp. 357–370, 2016.

[7] J. L. Wilson, A. J. Robinson, and T. Balendra, "Performance of precast concrete load-bearing panel structures in regions of low to moderate seismicity," *Eng. Struct.*, vol. 30, pp. 1831–1841, 2008.

[8] C. Casotto, V. Silva, H. Crowley, R. Pinho, and R. Nascimbene, "Scenario damage analysis of RC precast industrial structures in Tuscany, Italy," in *Second European conference on earthquake engineering and seismology*, 2014, pp. 1–12.

[9] T. Rossetto and A. Elnashai, "Derivation of vulnerability functions for European-type RC structures based on observational data," vol. 25, pp. 1241–1263, 2003.

[10] D. Beilic, C. Casotto, R. Nascimbene, D. Cicola, and D. Rodrigues, "Seismic fragility curves of single storey RC precast structures by comparing different Italian codes," *Earthquakes Struct.*, vol. 12, no. 3, pp. 359–374, Jan. 2017.

[11] G. M. Calvi, R. Pinho, G. Magenes, and J. Bommer, "Development of seismic vulnerability assessment methodologies over the past 30 years," no. September, 2006.

[12] V. Silva, H. Crowley, H. Varum, R. Pinho, and R. Sousa, "Evaluation of analytical methodologies used to derive vulnerability functions," 2013.

[13] A. Braconi *et al.*, "Prefabricated steel structures for low-rise buildings in seismic areas (Precasteel)," Brussels, 2013.

[14] H. Vitorino, N. Batalha, R. Sousa, H. Rodrigues, and P. Fernandes, "Typical properties of the Portuguese precast industrial buildings," Leiria, (under view).

© The 17th World Conference on Earthquake Engineering

[15] “NP EN 1992-1-1:2010, Eurocódigo 2 –Projeto de Estruturas de Betão –Parte 1-1: Regras gerais e regras para edifícios,” Instituto Português da Qualidade.
[16] M. Priestley, M. Calvi, and M. Kowalsky, *Displacement-Based Seismic Design of Structures*. Pavia, 2007.
[17] F. McKenna, “OpenSees: A Framework for Earthquake Engineering Simulation,” *Comput. Sci. Eng.*, vol. 13, no. 4, pp. 58–66, 2011.
[18] N. Batalha, H. Rodrigues, and H. Varum, “Seismic performance of RC precast industrial buildings — learning with the past earthquakes,” *Innov. Infrastruct. Solut.*, pp. 1–13, 2019.
[19] D. C. Kent and R. Park, “Flexural members with confined concrete,” *J. Struct. Div.*, vol. 97, no. 7, pp. 1969–1990, 1971.
[20] R. P. B. D. Scott and M. J. N. Priestley, “Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates,” *J. Proc.*, vol. 79, no. 1.
[21] M. Menegotto and P. E. Pinto, “Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending,” in *IABSE symposium of resistance and ultimate deformability of structures acted on by well-defined repeated loads, vol. 13, International Association of Bridge and Structural Engineering*, 1973, pp. 5–22.
[22] R. Sousa, J. P. Almeida, A. A. Correia, R. Pinho, and F. F. Modelling, “Shake Table Blind Prediction Tests : Contributions for Improved Fiber-based Frame Modelling Shake Table Blind Prediction Tests : Contributions for Improved,” *J. Earthq. Eng.*, vol. 00, no. 00, pp. 1–42, 2018.
[23] R. Sousa, N. Batalha, and H. Rodrigues, “Numerical simulation of beam-to-column connections in precast reinforced concrete buildings using fibre-based frame models,” *Eng. Struct.*, vol. 203, 2020.
[24] C. Casotto, V. Silva, H. Crowley, R. Nascimbene, and R. Pinho, “Seismic fragility of Italian RC precast industrial structures,” *Eng. Struct.*, vol. 94, pp. 122–136, 2015.
[25] F. Cornali, A. Belleri, A. Marini, and P. Riva, “Influence of modelling assumptions in the expected loss evaluation of a precast industrial building,” *Procedia Eng.*, vol. 199, pp. 3510–3515, 2017.

© The 17th World Conference on Earthquake Engineering