

Analytical Seismic Fragility Functions for Dual RC Structures in Bucharest

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Abstract Incremental dynamic analyses (IDA) of two dual RC structures (six and twelve stories in height), designed according to the provisions of relevant earthquake resistant design codes in force in Romania, are performed in order to derive analytical seismic fragility functions. The horizontal layout of the two structures is similar with five spans of 8.00 m in both orthogonal directions. In order to perform the nonlinear time history analyses (NTHAs) required by IDA method, the two structures are modelled with STERA 3D ver. 5.8. The hysteretic model employed in the NTHAs is a trilinear degrading model based on the Takeda model. The NTHAs are performed using 75 horizontal components of site-dependent simulated accelerograms that are randomly selected from a larger set developed for the INCERC site in Bucharest; the site is characterized by predominant long-periods of vibration of soil in strong earthquakes and the simulated accelerograms are reproducing this trend. The analytical fragility functions are obtained with the procedure given in Porter et al. (Earthquake Spectra 23(2):471–489, 2007) using the results of NTHAs.

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1 Introduction

Seismic fragility functions are a compulsory ingredient for the seismic risk assessment. The literature is abundant in seismic fragility functions for reinforced concrete (RC) structures. Full updated collections of fragility functions are provided in relevant report and papers (i.e. FEMA 2009; Kaynia 2013; Pitilakis et al. 2014). Nevertheless, the information is scarce for dual (moment resisting frames and structural walls) RC structures.

In order to fill this gap, the incremental dynamic analyses (IDAs) are performed for two dual RC structures and the results are reported in this paper. To this aim, two dual RC structures designed according to the provisions of relevant earthquake resistant design codes in force in Romania—P100-1/2013 (MDRAP 2013a) and CR2-1-1.1/2013 (MDRAP 2013b)—are subjected to IDAs in order to derive analytical seismic fragility functions. The first structure has a height of 18.0 m (six storeys high), while the second one is double in height (36.0 m) and in number of storeys, as well. According to FEMA (2009) typology, the first building is mid-rise and the second building is high-rise. The horizontal layout of the two structures is similar with five spans of 8.00 m in both orthogonal directions. In order to perform the nonlinear time history analyses (NTHAs) required by IDA method, the two structures are modelled with STERA 3D ver. 5.8 (<http://www.rc.ace.tut.ac.jp/saito/software-e.html>). This software is chosen because of its high speed of computation and because it allows the modelling of RC structures through elements with distributed plasticity. The hysteretic model employed by the software is a trilinear degrading model based on the well-known Takeda model.

The NTHAs are performed using mean values for the mechanical properties of the structural materials (concrete and reinforcing steel) and 75 horizontal components of site-dependent simulated accelerograms that are randomly selected from a larger set developed for the INCERC site in Bucharest; the site is characterized by predominant long-periods of vibration of soil in strong earthquakes and the simulated accelerograms is reproducing this trend. More details regarding the simulated accelerograms can be found in the companion paper of Pavel et al. (2016). The analytical fragility functions are obtained with the procedure given in Porter et al. (2007) using the results of NTHAs.

2 Buildings Description

Two building structures located in Bucharest were modelled, both having the same layout in plan and the same storey height. Both buildings have dual structures, with frames that bear mainly the vertical loads and shear walls that resist a large part of

the lateral loads and their tributary gravitational loads. The frames could provide a second line of defence when the buildings are subjected to very strong earthquakes and provide redundancy to the overall structure.

The floor plan is square so, in both directions there are five spans of 8.00 m. The storey height is 3.00 m. The first structure has six storeys and the second one is twelve storeys high. The location and the length of the walls are the same. The horizontal layout of the structural system of both buildings is shown in Fig. 1.

The structures were designed according to the provisions of earthquake resistant design codes in force in Romania: P100-1/2013 (MDRAP 2013a) and CR2-1-1.1/2013 (MDRAP 2013b). The design values for seismic design are: 0.30g peak ground acceleration with 20 % exceedance probability in 50 years, 1.6 s corner period of the response spectrum, and maximum dynamic amplification factor of 2.5 (MDRAP 2013a). One has to mention that for the higher building the importance factor is 1.2 because its height exceeds 28 m (MDRAP 2013a) while the value of the importance factor is 1.0 for the lower building. The seismic coefficient (ratio of the base shear to building weight in seismic combination) for the two structures is 0.111 for the six storey building and 0.133 for the twelve storey building. ETABS computer software (CSI 2015, <http://www.csiamerica.com/products/etabs>) was used for structural analysis.

The first building is 18.0 m in height. The basic design of the superstructure includes longitudinal and transversal beams of 300×700 mm, exterior and interior

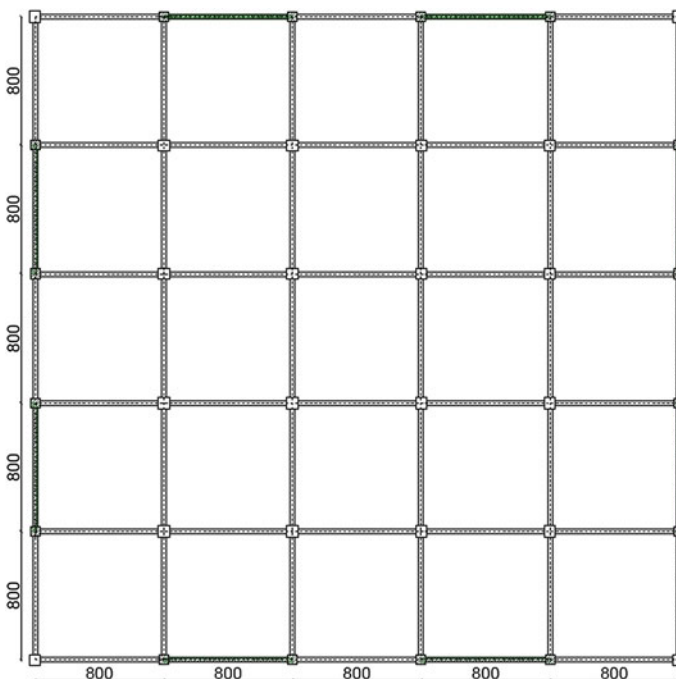


Fig. 1 Horizontal layout of structural system (RC structural walls are highlighted in green)

columns of 700×700 mm. All of the structural RC walls have a thickness of 300 mm. The materials used for this structure are: concrete C25/30 and reinforcing steel S500.

The reinforcement ratios of all structural members are detailed below in Tables 1, 2 and 3.

The total weight of the building is 134,400 kN and it was modelled using ETABS 2015 software, under different loading hypotheses. The eigenperiods of vibration along the two directions of the buildings (translation) are 0.41 and 0.28 s for general torsion.

The twelve-storey building has 700×700 mm columns, 300×700 beams and walls with 400 mm web width. The slab is 180 mm thick. The columns are integrated in the wall at the wall ends, providing enhanced ductility and anchorage for beams framing perpendicular to the wall. The concrete is of class C35/45 and the reinforcing steel is S500.

In Tables 4, 5 and 6 some of the details regarding structural members are given.

Total height of building is 36.00 m, total weight is 224,400 kN and the eigenperiods of vibration are 0.90 s (translation along the two principal directions) and 0.63 s for general torsion.

Table 1 Beams reinforcement

Beam dimensions (mm)	Location	Level	Bottom reinforcement ratio (%)	Top reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
300×700	Exterior	3, 4	0.38	0.63	0.52	$2\text{Ø}10/100$
300×700	Exterior	1, 2, 5, 6	0.38	0.47	0.52	$2\text{Ø}10/100$
300×700	Interior	3, 4	0.38	0.63	0.52	$2\text{Ø}10/100$
300×700	Interior	1, 2, 5, 6	0.38	0.47	0.52	$2\text{Ø}10/100$

Table 2 Columns reinforcement

Column dimensions (mm)	Location	Longitudinal reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
700×700	Corner and interior	1.72	0.60	$5.41\text{Ø}10/100$

Table 3 Structural walls reinforcement

Web thickness (mm)	Level	Edge reinforcement ratio (%)	Web reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
300	1, 2, 3	1.69	0.38	0.75	$2\text{Ø}12/100$
300	4, 5, 6	1.69	0.38	0.52	$2\text{Ø}10/100$

Table 4 Beams reinforcement

Beam dimensions (mm)	Location	Bottom reinforcement ratio (%)	Top reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
300 × 700	Edge 1	0.53	1.00	0.67	4Ø8/100
300 × 700	Edge 2	0.82	1.33	0.67	4Ø8/100
300 × 700	Central 1	0.82	1.33	0.67	4Ø8/100
300 × 700	Central 2	0.53	0.83	0.67	4Ø8/100

Table 5 Columns reinforcement

Column dimensions (mm)	Location	Longitudinal reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
700 × 700	Interior and corner	1.00	0.60	5.41Ø10/100

Table 6 Structural walls reinforcement

Web thickness (mm)	Level	Edge reinforcement ratio (%)	Web reinforcement ratio (%)	Transverse reinforcement ratio (%)	Stirrups (mm)
400	1, 2, 3	3.14	0.38	0.80	2Ø16/125
400	4	2.78	0.28	0.51	2Ø14/150
400	5	2.49	0.28	0.51	2Ø14/150
400	6	1.60	0.28	0.38	2Ø12/150
400	7–12	1.00	0.28	0.38	2Ø12/150

It is mentioned that for both buildings the eigenmodes and eigenvalues are obtained using cracked stiffness (50 % of the gross section moment of inertia, according to P100-1/2013). Both structures are considered fixed at the bottom of the ground floor.

3 Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is a form of nonlinear parametric analysis which aims to describe the in-depth behaviour of a structure subjected to earthquake loads (Vamvatsikos and Cornell 2001). A suite of strong motion records is used to excite a comprehensive structural model, and for every structure and every ground motion record a nonlinear time history analysis is performed. The selected accelerograms have increasing intensity measures (i.e., peak ground acceleration—

PGA and/or spectral displacement and spectral acceleration at the fundamental period of vibration of the building). Maximum structural response of the model, in terms of inter-storey drift ratio, base shear force and roof displacement, is recorded for every accelerogram.

3.1 *Nonlinear Time History Analyses*

In order to perform the nonlinear analysis, the two structures were modelled in STERA 3D ver. 5.8. This computer program was chosen because of its high speed of computation and because it allows the modelling of RC structures through fibre elements. The hysteretic model used by the program is a trilinear degrading model based on Takeda Model.

In this paper, for analysis purpose we assumed:

- mean values of the properties of the materials ($f_c = 43$ MPa for C35/45 concrete, $f_c = 33$ MPa for C25/30 concrete and $f_y = 575$ MPa for S500 steel);
- the default parameters for beam element—stiffness degradation ratio 0.5, slip stiffness ratio 0.0, strength degrading ratio 0.0;
- P-delta effects were not considered;
- damping is considered proportional to instantaneous stiffness matrix, changing according to the nonlinearity of structural elements;
- numerical integration method is based on average acceleration.

Slip stiffness ratio and strength degrading ratio are available for user editing only for the beam element.

The nonlinear time-history analyses were conducted using 75 horizontal accelerograms, in order to obtain the maximum storey shear force, maximum displacement at the top and the maximum storey drift associated with the ground motion in question.

The horizontal components of the accelerograms were stochastically simulated for the free surface of INCERC site in Bucharest. In the paper of Pavel et al. (2016), the stochastic simulation of 1554 strong ground motions is performed in conjunction with a 2500 years stochastic catalogue for Vrancea intermediate-depth earthquakes with magnitudes $M_W \geq 5.5$. The catalogue is simulated based on the Monte Carlo approach described by Assatourians and Atkinson (2013) and on the seismicity parameters derived from the ROMPLUS catalogue of the National Institute of Earth Physics (<http://www1.infp.ro/realtime-archive>). Out of 1554 horizontal components of simulated accelerograms, 75 were randomly selected as input for the NTHAs. All the selected accelerograms are compatible with the deep soil profile at INCERC site in Bucharest (Constantinescu and Enescu 1985). Details on the Vrancea intermediate-depth seismic source and the seismic hazard from this seismic source can be found elsewhere (i.e. Ismail-Zadeh et al. 2012; Lungu et al. 2000; Radulian et al. 2000; Vacareanu et al. 2016).

Thus, the input for the NTHAs consisted in 75 strong ground motions already mentioned, grouped in eight bins in ascending order of the intensity measure. The intensity measure initially considered is peak ground acceleration with values in between 0.05g and 0.40g; only two values are larger than 0.4g: 0.59g and 0.71g. The dimension of the bin is 0.04g and the last bin comprises accelerograms with peak ground accelerations in excess of 0.33g. Two other intensity measures are also considered in the analyses, namely spectral acceleration and spectral displacement at the fundamental period of vibration of the buildings.

3.2 Incremental Dynamic Analyses Results

For each NTHAs, the maximum values of base shear force, displacement at the top of the building and inter-storey drift are considered and recorded. In Fig. 2 the results of the IDAs are shown synthetically as the plot of the maximum values of the displacement at the top of the building versus base shear force. The IDAs results are compared in Fig. 2 with the pushover curves obtained with a uniform and a modal distribution of the lateral loading.

The evolution of the structural behaviour from cracking to yielding and eventually collapse can be easily seen in Fig. 2. For both buildings it can be seen that most of the IDA cloud of data fits between the two pushover curves, so the static nonlinear analysis provides a reasonable envelope for the response of the building in terms of base shear versus top displacement. One can notice from Fig. 2 that the push-over curve with modal distribution of lateral loading is the lower bound for IDAs results while the push-over curve with uniform distribution of the lateral loading provides the upper bound for the IDAs results.

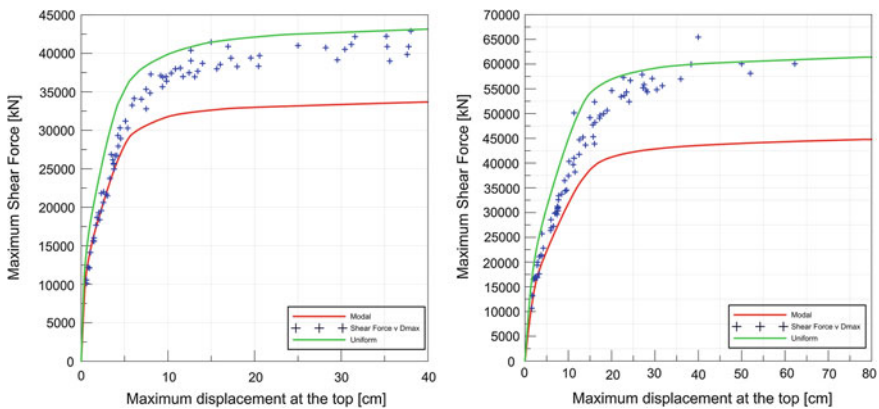


Fig. 2 IDAs results and push-over curves for both structural systems (*left* six storey building; *right* twelve storey building)

4 Seismic Fragility Functions

4.1 Previous Studies

Fragility functions are a powerful tool in assessing the seismic performance of a building structure. In the past decades a great deal of effort was carried out towards improving the defining attributes and quality of such functions. The fragility function relates the probability of reaching or exceeding a certain state of damage to an intensity measure parameter (peak ground acceleration, spectral displacement). Functions parameters usually depend on the structural type, height of structure and material type. There were many attempts to find methodologies suitable for different building types.

One of the first thorough methodologies, which addressed 36 structural categories, was HAZUS (FEMA 2009). These categories were classified according to US practice, and the level of knowledge at the time when design took place was captured through code level approach (i.e. “low code”, “moderate code” etc.). The methodology was integrated with a geographic information system (GIS) in order to obtain exposure and seismic risk maps.

Another important step was the RISK-UE project which took place between 2001 and 2004. The program was meant to assess risk scenarios for European towns located in earthquake prone areas (Mouroux and Le Brun 2006). There were considered 23 building categories and, after building design code and height class was considered, the RISK-UE taxonomy emerged. Some results on the seismic fragility and risk for the RC building stock in Romania can be found in Vacareanu et al. (2004).

SYNER-G is one of the latest European research projects which aim to assess the vulnerability and the seismic risk of buildings, lifelines and infrastructure considering the mutual influence between them (Pitilakis et al. 2014). In order to obtain the fragility functions, one should characterize the analysed structure according to the taxonomy (Hancilar and Taucer 2013). Under the SYNER-G framework the fragility functions can be derived by empirical methods, expert judgement, analytical methods (capacity spectrum method, incremental dynamic analysis) and hybrid methods.

4.2 Results of the Case Studies

In order to assess the probability of exceeding a certain damage state (slight, moderate, extensive or complete) conditioned on the values of peak ground accelerations, spectral displacement and spectral acceleration, the fragility functions are modelled using a log-normal distribution which is described entirely by the

median value of the distribution and the logarithmic standard deviation. Given an intensity measure (IM) of the strong ground motion, the probability of being in or exceeding a damage state, ds , is modelled as:

$$P[ds|IM] = \Phi \left[\frac{1}{\beta_{ds}} \ln \left(\frac{IM}{\overline{IM}_{ds}} \right) \right] \quad (1)$$

where: \overline{IM}_{ds} is the median value of the intensity measure at which the building reaches the threshold of the damage state, ds ;

β_{ds} is the standard deviation of the natural logarithm of the intensity measure of damage state, ds , and

Φ is the standard normal cumulative distribution function.

As described in HAZUS (FEMA 2009), the damage states vary from slight to complete.

Slight structural damage denotes “diagonal hairline cracks on most concrete shear wall surfaces; minor concrete spalling at few locations”, moderate implies that “most shear wall surfaces exhibit diagonal cracks; some shear walls have exceeded yield capacity indicated by larger diagonal cracks and concrete spalling at wall ends”, extensive entails that “most concrete shear walls have exceeded their yield capacities; some walls have exceeded their ultimate capacities indicated by large, through-the-wall diagonal cracks, extensive spalling around the cracks and visibly buckled wall reinforcement or rotation of narrow walls with inadequate foundations” and complete describes the structural collapse or that the structure is in “imminent danger of collapse due to failure of most of the shear walls and failure of some critical beams or columns” (FEMA 2009).

The method employed in this study to obtain the fragility functions parameters is Method B, based on Porter et al. (2007) where different methods for creating fragility functions are presented based on bounding intensity measures (IM). Method B refers to the situation in which a part of the specimens failed (in this study, exceeded the inter-storey or relative drift ratio corresponding to the threshold of the damage state). The damage states are considered as in HAZUS (FEMA 2006), namely Slight, Moderate, Extensive and Complete.

The maximum input values (peak ground acceleration, spectral acceleration and spectral displacement at the fundamental period of vibration of the building) and the corresponding maximum output values (base shear force, top displacement, and inter-storey drift ratio) are organized in bins. Following Porter et al. (2007), each bin must have approximately the same number of parameters and the number of bins is the largest integer less than or equal to the square root of the size of the sample. In this study the size of the sample is 75, thus the number of bins is equal to eight.

For each bin, the number of specimens that failed (number of NTHAs in which the inter-storey drift ratios at the threshold of various damage states are exceeded) is evaluated and the average values of IMs are obtained.

In order to find the number of failed specimens, the inter-storey drift ratios obtained from STERA 3D (calculated at the effective modal height) is compared to the corresponding values at the threshold of the damage states. These thresholds are considered for mid-rise (C2M) and high-rise (C2H) RC structural walls, according to HAZUS (FEMA 2009), and the values are as following:

- C2M: Slight: 0.27 %, Moderate: 0.67 %, Extensive: 2 % and Complete 5.33 %
- C2H: Slight: 0.2 %, Moderate: 0.5 %, Extensive: 1.5 % and Complete 4 %

The parameters of the fragility functions for Slight, Moderate and Extensive damage states, conditioned on peak ground acceleration, are reported in Tables 7 and 8 for six storey building and, respectively, for twelve storey building. Moreover, in Tables 9 and 10, the parameters of the fragility function conditioned on spectral displacement at the fundamental periods of vibration of the buildings are given. The parameters of the fragility functions for Complete damage state are not reported since the authors of the study considered that the values of the logarithmic

Table 7 Parameters of the fragility functions for the six storey building, for $IM = PGA$

Parameters	Slight	Moderate	Extensive	Complete
Median value (cm/s ²), \overline{PGA}_{ds}	120	155	330	827
Logarithmic standard deviation, β_{ds}	0.602	0.527	0.800	1.346

Table 8 Parameters of the fragility functions for the twelve storey building, for $IM = PGA$

Parameters	Slight	Moderate	Extensive
Median value (cm/s ²), \overline{PGA}_{ds}	108	192	704
Logarithmic standard deviation, β_{ds}	0.421	0.546	1.274

Table 9 Parameters of the fragility functions for the six storey building, for $IM = SD(T_1)$

Parameters	Slight	Moderate	Extensive
Median value (cm), $\overline{SD(T_1)}_{ds}$	1.0	1.3	2.9
Logarithmic standard deviation, β_{ds}	0.739	0.649	1.411

Table 10 Parameters of the fragility functions for the twelve storey building, for $IM = SD(T_1)$

Parameters	Slight	Moderate	Extensive
Median value (cm), $\overline{SD(T_1)}_{ds}$	3.0	6.8	55.7
Logarithmic standard deviation, β_{ds}	0.330	0.649	2.049

standard deviation are not well constrained because of the very limited number of cases in which the inter-storey drift ratio at the threshold of the Complete damage state is exceeded.

Some comments on the values of the fragility functions parameters are as follows:

- for the six storey building, the median values of the peak ground acceleration for the Slight and Moderate damage state are closely spaced, showing a very narrow behaviour range within which the structural system might experience quite different types of damage; also, the median value of the peak ground acceleration is only 10 % higher than the design value, revealing that the ultimate limit state given in the P100-1/2013 (MDRAP 2013a) design code is actually exceeded in almost 50 % of the cases for the design acceleration (given that the ultimate limit state must ensure the life safety of the occupants and Complete damage state is far beyond this requirement); the logarithmic standard deviation is increasing as the damage states evolves from Slight to Extensive because of the decreasing number of cases of exceedances of the corresponding inter-storey drift ratio at the thresholds of the damage states;
- for the twelve storey building, the median values of the peak ground acceleration for the Slight and Moderate damage state are widely spaced, showing a large behaviour range within which the structural system might experience quite different types of damage; also, the median value of the peak ground acceleration is more than double than the design value, revealing that the ultimate limit state given in the P100-1/2013 (MDRAP 2013a) design code is actually exceeded in very few cases for the design acceleration (given that the ultimate limit state must ensure the life safety of the occupants and Complete damage state is far beyond this requirement); the logarithmic standard deviation is increasing as the damage states evolves from Slight to Extensive because of the decreasing number of cases of exceedances of the corresponding inter-storey drift ratio at the thresholds of the damage states;
- in the opinion of the authors of this study, the better seismic performance of the twelve storey building (if compared with the six storey building) is due to (i) the 20 % increase in the design seismic force incurred by the importance factor, (ii) its higher redundancy and (iii) the better behaviour of its slenderer RC structural walls if compared with the squatter walls of the six storey building;
- the median values of the spectral displacements (and of the corresponding inter-storey drift ratios) at the thresholds of the damage states obtained in this study are in-between the corresponding values given in HAZUS (FEMA 2009) for RC moment resisting frames and RC structural walls;
- the very high value of the logarithmic standard deviation for the extensive damage state (conditioned upon $IM = PGA$) of the twelve storey building is a consequence of the sampling of PGA values at the high-end of the IM scale; the reason is the very few numbers of runs exceeding the threshold of the extensive damage state (compared to the six storey building) because of the increased

seismic performance brought up by the importance factor of 1.2; nevertheless, very high values of the logarithmic standard deviation, in the same high-end range, are reported in Kaynia (2013);

- the very high values of the logarithmic standard deviation for the extensive damage state [conditioned upon $IM = SD(T_1)$] seems to be a consequence of the change of conditioning IM from PGA (as originally sampled) to $SD(T_1)$ (as formerly transformed), thus losing some accuracy of the sampling and binning since there is no (or poor) correlation between the PGA and $SD(T_1)$ values.

The fragility functions obtained for Slight, Moderate and Extensive damage states for both structures (six-storey building and twelve-storey building) are represented in Fig. 3. The fragility functions for Complete damage state are represented separately in Fig. 4. The intensity measure in Figs. 3 and 4 is the peak ground acceleration.

If one is interested in the probability of a dual RC structural system of being in a given damage state (or undamaged) for a specified value of the intensity measure, the following relations given by Porter et al. (2007) can be applied:

$$\begin{aligned}
 P[DS = ds | IM = im] &= 1 - F_1(im) \rightarrow ds = 0 \\
 &= F_{ds}(im) - F_{ds+1}(im) \rightarrow 1 \leq ds < N \\
 &= F_{ds}(im) \rightarrow ds = N
 \end{aligned}
 \tag{2}$$

where $ds = 0$ for undamaged and $ds = 1 \dots N$ for Slight, Moderate, Extensive and Complete.

Applying relation (2) for $IM = PGA$ and $im = 0.30g$ (the design value of PGA for both buildings), the probabilities are reported in Tables 11 and 12. The values of the

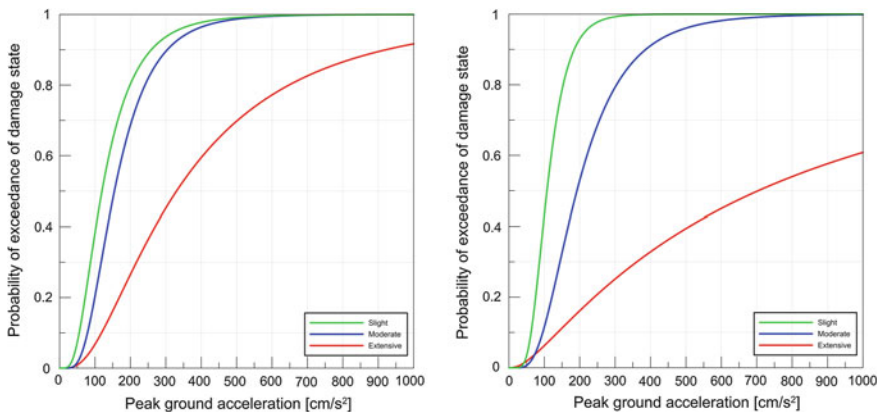


Fig. 3 Fragility functions for slight, moderate and extensive damage states conditioned upon peak ground acceleration (*left* six storey building; *right* twelve storey building)

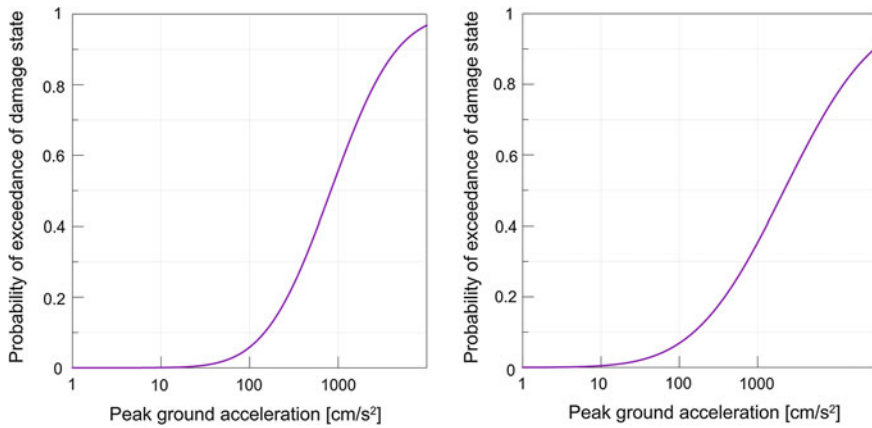


Fig. 4 Fragility functions for Complete damage state conditioned upon peak ground acceleration (*left* six storey building; *right* twelve storey building)

Table 11 Probabilities of being in certain damage states or undamaged for the six storey building, for $PGA = 0.30g$

Undamaged	Slight	Moderate	Extensive	Complete
0.064	0.041	0.443	0.227	0.225

Table 12 Probabilities of being in certain damage states or undamaged for the twelve storey building, for $PGA = 0.30g$

Undamaged	Slight	Moderate	Extensive	Complete
0.007	0.199	0.542	0.082	0.170

probabilities given in Tables 11 and 12 points once again towards a better damage control of the twelve storey building if compared to the six storey building because the probability of being in Extensive or Complete damage states is 0.25 in the first case and 0.45 in the second case.

A set of fragility functions that consider the probability of exceeding serviceability and ultimate limit states conditional upon $IM = PGA$ is also developed using the results of IDAs. The serviceability and ultimate limit states are defined in P100-1/2013 (MDRAP 2013a) in terms of inter-storey drift ratio as 0.5 %, 0.75 % and 1.0 % (depending on the type and connections of the non-structural components) for serviceability and 2.5 % for ultimate limit state, respectively.

Applying the same method as for the previous fragility functions (Method B of Porter et al. (2007), the median values of the IM (in terms of PGA) and logarithmic

standard deviations are obtained and the values are reported in Tables 13 and 14 for the six storey and twelve storey buildings, respectively. The fragility functions for serviceability and ultimate limit states are plotted in Figs. 5 and 6, respectively.

One can notice that for the six storey building the median values of PGA corresponds to one half of the design peak ground acceleration for serviceability limit state and to the design peak ground acceleration for ultimate limit state, matching perfectly the seismic hazard level recommended by the seismic design code in force in Romania for the limit states verifications. On the other hand, the median values are much more scattered for the twelve storey building.

Table 13 Parameters of the fragility functions for the six storey building, for $IM = PGA$

Parameters	SLS (0.5 %)	SLS (0.75 %)	SLS (1.0 %)	ULS (2.5 %)
Median value (cm/s^2), \overline{PGA}_{I_s}	141	143	152	297
Logarithmic standard deviation, β_{I_s}	0.435	0.472	0.455	0.707

Table 14 Parameters of the fragility functions for the twelve storey building, for $IM = PGA$

Parameters	SLS (0.5 %)	SLS (0.75 %)	SLS (1.0 %)	ULS (2.5 %)
Median value (cm/s^2), \overline{PGA}_{I_s}	173	240	330	1251
Logarithmic standard deviation, β_{I_s}	0.483	0.647	0.766	1.653

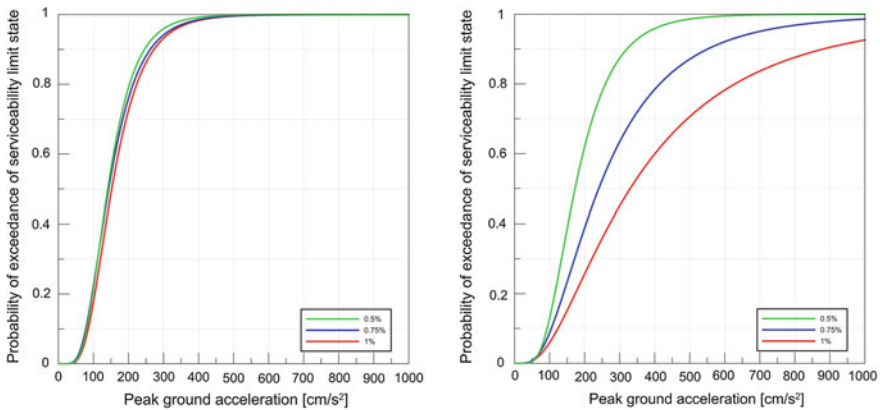


Fig. 5 Fragility functions serviceability limit state conditioned upon PGA (*left* six storey building; *right* twelve storey building)

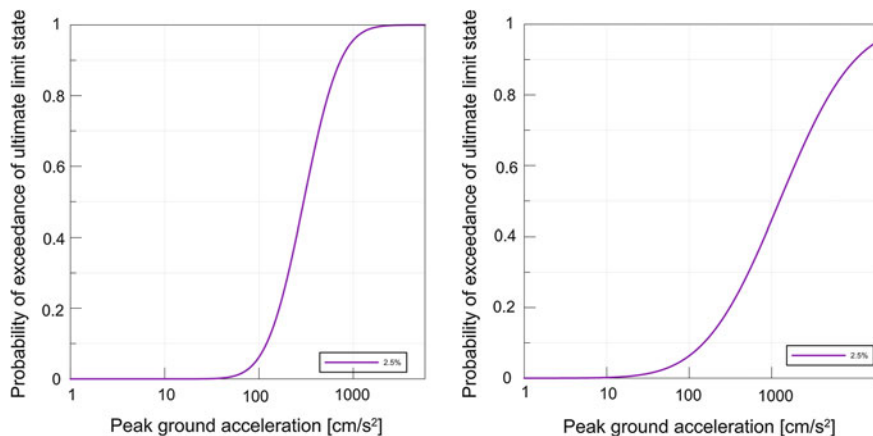


Fig. 6 Fragility functions ultimate limit state conditioned upon PGA (*left* six storey building; *right* twelve storey building)

5 Conclusions

In this study, fragility functions for two model RC dual buildings are obtained. The conditional intensity measures are peak ground acceleration and spectral response values at the fundamental period of vibration of the building.

The fragility functions are developed for damage states and limit states, as well. Although the structural system is the same for the two buildings (moment resisting frames and shear walls), the median values of the fragility functions are different. The higher values of the medians for the twelve storey building (if compared to the six storey building) occur (in the opinion of the authors) because of the:

- importance factor that increase by 20 % the design seismic force for the higher building;
- higher redundancy of the twelve storey structural system;
- better seismic behaviour of the slenderer shear walls of the higher building.

For the twelve storey building, the values of the logarithmic standard deviation are larger than 1.0 for extensive and complete damage states and for the ultimate limit state, as well. Values larger than unity for buildings designed according to high seismic design codes are not reported in HAZUS (FEMA 2009) but are reported in literature and can be found in Kaynia (2013). The reasons for these high values in this study are, in the opinion of the authors, the very limited numbers of accelerograms in the sampling pushing the structural system beyond the thresholds of the extensive and complete damage states and ultimate limit state.

In this paper only the random uncertainties of the seismic excitation are considered. In order to get a full picture of the propagation of the uncertainties into the logarithmic standard deviation of the fragility functions, further studies must

consider other sources of random uncertainties (i.e. mechanical properties of materials, geometrical properties of the structural elements) and of epistemic uncertainties (i.e. alternative structural layouts, softwares to conduct the NTHAs).

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