



Life-Cycle and Seismic Fragility Assessment of Code-Conforming Reinforced Concrete and Steel Structures in Bucharest, Romania

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Published online: 18 April 2018
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Abstract In this article, the fragility of reinforced concrete and steel structures in Bucharest, a city of high seismic hazard, designed using the recent building codes in Romania, is assessed. A total of 24 reinforced concrete and steel structures with heights varying from five stories to 13 stories were analyzed. Their seismic fragility was evaluated using two procedures from the literature. In the first procedure (SPO2FRAG), the fragility was derived based on the pushover curves using the SPO2IDA algorithm, while in the second procedure (FRACAS), the fragility was derived from nonlinear time-history analyses. The analyzed structures were designed for three levels of peak ground accelerations, corresponding to mean return periods of 100, 225, and 475 years. Subsequently, the damage assessment of the analyzed structures was performed using ground motions generated from a Monte-Carlo simulated earthquake catalogue for the Vrancea intermediate-depth seismic source. The damage degrees that were estimated using the SPO2FRAG approach are higher than those estimated using the FRACAS approach. The life-cycle analysis of the structures shows that a further increase of the design peak ground acceleration for Bucharest is feasible from an economic point of view using the SPO2FRAG results. However, based on the FRACAS results, the opposite conclusion can be drawn. Finally, generic lognormal fragility functions are proposed as a function of building height and structural system.

Keywords Bucharest · Building damage degree · Building life-cycle analysis · Monte-Carlo simulated earthquake catalogue · Seismic risk · Vrancea seismic source

1 Introduction

Bucharest, the capital city of Romania, is one of the highest seismic risk cities in Europe because of its proximity to the active Vrancea intermediate-depth seismic source and due to a vulnerable building stock. Within the last 200 years, the Vrancea intermediate-depth seismic source has generated nine earthquakes with moment magnitudes $M_w \geq 7.1$ —the largest one occurred in 1802 and had an estimated $M_w = 7.9$.

Recently, a seismic risk assessment of Bucharest's residential building stock was conducted by Pavel et al. (2017a). The residential buildings were divided into more than 30 typologies as a function of the building material and structural system, height regime, and level of seismic code used in the design process. The high-code (HC), or advanced seismic code typology, includes all the buildings built between 1992 and 2011 when the most recent census was conducted. The HC typology consists of the buildings designed using the 1992 and 2006 versions of the Romanian seismic design code. In 2013, the P100-1/2013 (Ministry of Regional Development and Public Administration 2013) version of the Romanian seismic code was issued. The mean return period and the corresponding design peak ground acceleration (*PGA*) for Bucharest have increased from 50 years ($PGA = 0.20$ g) in the 1992 version of the code, to 100 years ($PGA = 0.24$ g) in the 2006 code, and to 225 years ($PGA = 0.30$ g) in the 2013 version of the seismic design code. In the future, a further increase

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to a mean return period of 475 years for the peak ground acceleration is envisaged.

In this article, we evaluate the seismic fragility of bi-symmetrical reinforced concrete (RC) and steel structures in Bucharest that were designed for three levels of peak ground acceleration—0.24, 0.30, and 0.36 g. The main motivation for the analysis is that several hundred residential and office buildings were built in the past decade or so for which there are very few or no fragility functions that can be used in a seismic risk assessment at city level. Up to now the seismic fragility of these types of buildings was based on HAZUS (FEMA 2012) data. Thus, there is an urgent need for more accurate fragility data.

The seismic fragility of the buildings was assessed using two procedures from the literature, in terms of both spectral accelerations and peak ground accelerations. This approach was motivated by the desire to compare the results from two methodologies and to assess the methodologies' influence on the seismic risk metrics. Subsequently, the seismic losses were evaluated for all the structures using ground motion amplitudes derived from a Monte-Carlo simulated earthquake catalogue for the Vrancea intermediate-depth seismic source that is coupled with a ground motion model derived for the same seismic source. This approach is similar to the one applied by Pavel et al. (2017a). What is novel about this study is the assessment of the seismic fragility of steel structures, which at the moment represent a common solution for both residential and office high-rise buildings in Bucharest and for which there are no fragility functions available. Most of these steel structures were built after the issuance of the 2006 version of the Romanian seismic design code. Thus it is necessary to conduct a fragility assessment for these structures because they have not been considered in any prior seismic risk analysis performed at the city level.

2 Analyzed Reinforced Concrete and Steel Structures

A total of 24 bi-symmetrical examples of reinforced concrete (RC) and steel structures were analyzed in this study. Fifteen structures are made of RC and have a structural system consisting of frames with or without structural walls. The nine steel structures have a structural system consisting of beams, columns, and concentric braces. The planar layouts for all the structures are shown in Figs. 1 and 2. A reinforced concrete slab of 15 cm thickness was assumed for all the structures. The structures were designed using the provisions given in the current seismic design code P100-1/2013 (Ministry of Regional Development and Public Administration 2013). The capacity design procedure from this design code for ductility class high (*DCH*) was

applied to all 24 structures. The relative displacement checks were performed as indicated in the seismic code for both the serviceability limit state (*SLS*) and for the ultimate limit state (*ULS*). A maximum inter-story drift limit of 2.5% is imposed by the Romanian seismic design code for *ULS*. The cross-sections of the structural elements, as well as the number of stories, design peak ground acceleration, and fundamental eigenperiod are shown in Table 1 for the RC structures (denoted as RC) and in Table 2 for the steel structures (denoted as S), respectively. The name of the RC structures also contains the letter F for frames and W for shear walls. For both the RC and the steel structures the design *PGA* and the number of stories are also given in their name. The shear reinforcement was designed so as to avoid any brittle failure. The three 5-story RC structures were also analyzed in a recent article by Pavel et al. (2017b). It is clear that since both of the methodologies used are based on equivalent single-degree-of-freedom (SDOF) systems, they are applicable to structures whose response is dominated by the first eigenmode. Thus, we decided to use in this study only structures for which the modal mass associated to the first eigenmode is in excess of 75%.

3 Assessment of Structural Fragility

The structural fragility of the buildings was assessed using two recently developed procedures. The first approach—SPO2FRAG (Baltzopoulos et al. 2017)—relies on the pushover curve of the structure in order to derive the fragility functions. The second approach—FRACAS (Rossetto et al. 2016)—derives the fragility functions based on the inelastic time-history analyses of equivalent single-degree-of-freedom systems using scaled ground motion recordings. In the SPO2FRAG approach, the results of incremental dynamic analysis are approximated via the SPO2IDA algorithm developed by Vamvatsikos and Cornell (2006) and an equivalent single-degree-of-freedom approximation of the structural system. Four damage states—slight, moderate, extensive, and complete damage—were considered in the assessment of seismic fragility. The inter-story drift limits corresponding to each damage state were taken from HAZUS (FEMA 2012) for each typology (the values used are for high-code structures). The limit drifts vary from 0.25% for the slight damage state to 4% for the complete damage state for the high-rise structures (10- and 13-story structures), and from 0.33 to 5.33% for the medium-rise structures (5- and 7-story structures).

The static pushover curves were obtained for all the structures using the STERA 3D software (Structural Earthquake Response Analysis 3D).¹ The basic

¹ <http://www.rc.ace.tut.ac.jp/saito/software-e.html#p01>.

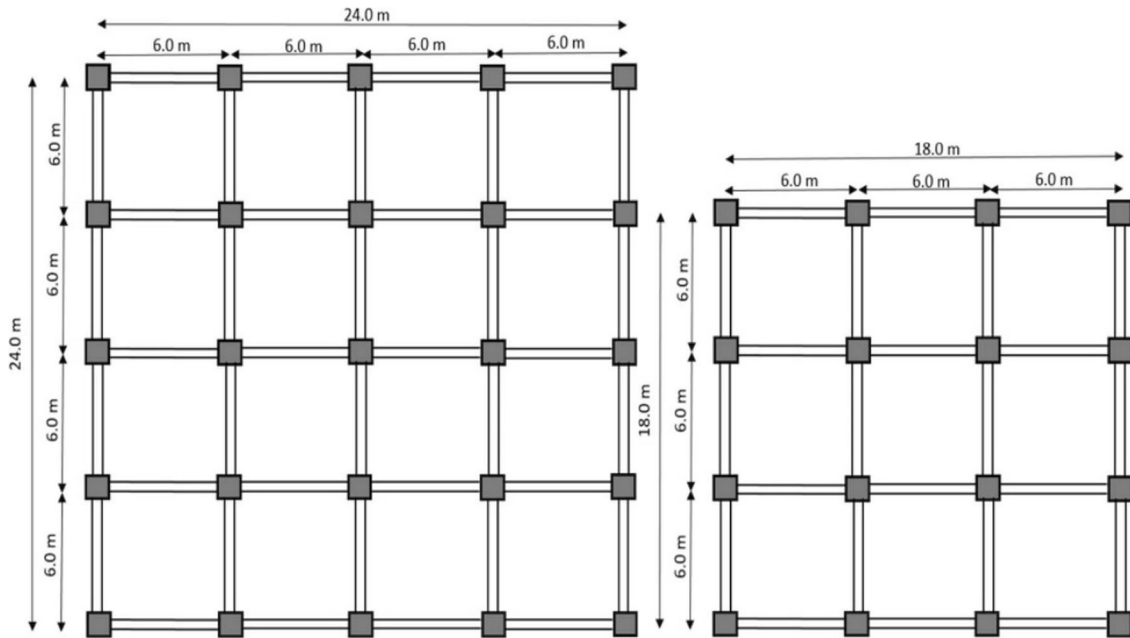


Fig. 1 Planar layout for the 5-story reinforced concrete (RC) frame structures (left), and the 7-story and 10-story RC frame structures (right) in Bucharest, Romania

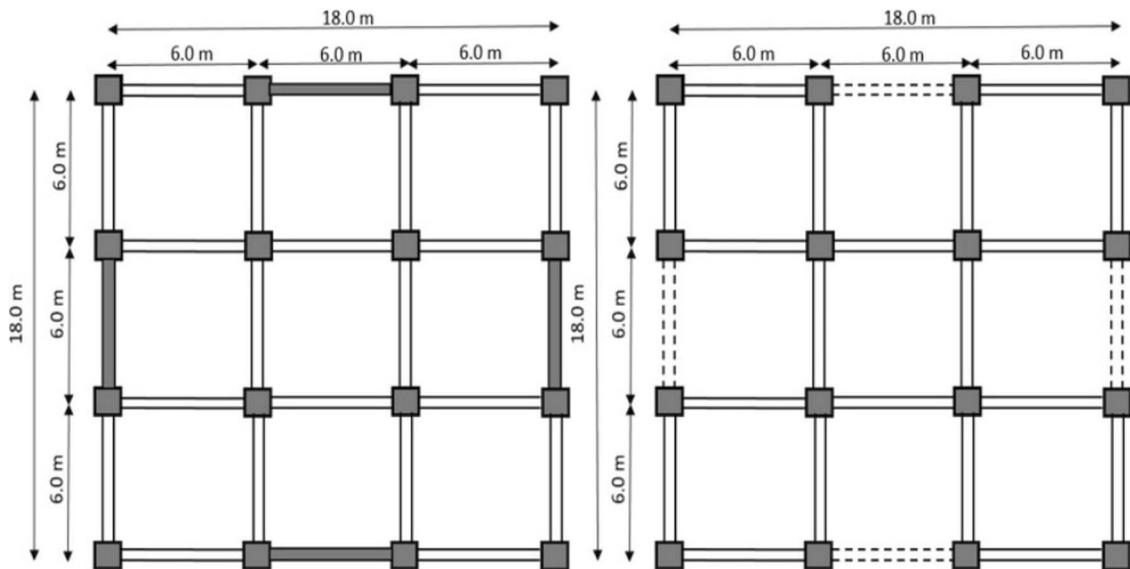


Fig. 2 Planar layout for the 10-story and 13-story reinforced concrete (RC) shear wall structures (left) and for the 7-story, 10-story, and 13-story concentrically braced steel structures (right) in Bucharest, Romania

assumptions employed in the STERA 3D software, similar to the ones applied by Pavel et al. (2017b), are described briefly. All the structural elements are considered as line elements and the floor diaphragms are modelled as rigid in their plane. The concrete and steel strengths are based on the mean strengths of the materials, also taking into account the concrete confinement and the steel hardening. In the case of the steel nonlinear springs, the peak-oriented model is adopted prior to yielding and a trilinear model is

adopted afterwards. The trilinear hysteresis rule is also adopted for the concrete springs with consideration of the strength degradation after the yielding point. Only P-delta effects are considered in the nonlinear static analysis for each model. An example of a moment-rotation curve for a beam hinge is shown in Fig. 3.

Figure 4 shows some examples of pushover curves for the 7-story and 10-story RC frame structures, as well as for the 10-story steel structures. The shape of the pushover

Table 1 Characteristics of the analyzed 15 reinforced concrete (RC) structures in Bucharest, Romania

Structure name	Design <i>PGA</i> (g)	Fundamental period (s)	No. of stories	Cross-section (cm)		
				Columns	Shear wall thickness	Beams
RCF-0.24-5	0.24	0.69	5	50 × 50	–	25 × 50
RCF-0.30-5	0.30	0.59	5	55 × 55	–	25 × 55
RCF-0.36-5	0.36	0.51	5	60 × 60	–	25 × 60
RCF-0.24-7	0.24	0.83	7	55 × 55	–	25 × 50
RCF-0.30-7	0.30	0.76	7	55 × 55	–	25 × 55
RCF-0.36-7	0.36	0.66	7	55 × 55	–	30 × 60
RCF-0.24-10	0.24	1.00	10	65 × 65	–	25 × 55
RCF-0.30-10	0.30	1.00	10	65 × 65	–	25 × 55
RCF-0.36-10	0.36	0.93	10	65 × 65	–	30 × 55
RCW-0.24-10	0.24	0.69	10	60 × 60	20	25 × 50
RCW-0.30-10	0.30	0.65	10	60 × 60	25	30 × 50
RCW-0.36-10	0.36	0.63	10	60 × 60	30	30 × 50
RCW-0.24-13	0.24	0.87	13	70 × 70	25	25 × 50
RCW-0.30-13	0.30	0.83	13	70 × 70	30	30 × 50
RCW-0.36-13	0.36	0.80	13	70 × 70	35	30 × 50

PGA peak ground acceleration, *F* frames, *W* shear walls

Table 2 Characteristics of the analyzed nine steel (S) structures in Bucharest, Romania

Structure Name	Design <i>PGA</i> (g)	Fundamental period (s)	No. of stories	Cross-section		
				Columns	Braces	Beams
S-0.24-7	0.24	0.99	7	HEA320/HEA280/HEA240/HEA200	HEB120/HEB100	IPE240/ IPE180
S-0.30-7	0.30	0.90	7	HEA400/HEA300/HEA280/HEA240	HEB140/HEB120/HEB100	IPE240/ IPE220
S-0.36-7	0.36	0.78	7	HEA600/HEA400/HEA300/HEA280	HEB160/HEB140/HEB120	IPE240
S-0.24-10	0.24	1.07	10	HEA500/HEA400/HEA300/HEA280/ HEA240/HEA200	HEB160/HEB140/HEB120/ HEB100	IPE240/ IPE220
S-0.30-10	0.30	0.97	10	HEA600/HEA500/HEA400/HEA300/ HEA280/HEA240	HEB180/HEB160/HEB140/ HEB120/HEB100	IPE240
S-0.36-10	0.36	0.92	10	HEA700/HEA600/HEA500/HEA400/ HEA300/HEA280	HEB200/HEB180/HEB160/ HEB140/HEB120	IPE270/ IPE240
S-0.24-13	0.24	1.24	13	HEA500/HEA400/HEA320/HEA300/ HEA300/HEA280/HEA240/HEA200	HEB200/HEB180/HEB160/ HEB140/HEB120/HEB100	IPE270/ IPE240
S-0.30-13	0.30	1.09	13	HEA800/HEA600/HEA500/HEA400/ HEA300/HEA280/HEA240	HEB220/HEB200/HEB180/ HEB160/HEB140/HEB120	IPE300/ IPE270/ IPE240
S-0.36-13	0.36	0.98	13	HEA1000/HEA800/HEA700/HEA600/ HEA500/HEA400/HEA300	HEB240/HEB220/HEB200/ HEB180/HEB160/HEB140	IPE300/ IPE270/ IPE240

PGA peak ground acceleration

curve is dictated by the material characteristics and by the moment-rotations curves of the structural elements that are of a bilinear shape, and thus no degradation of the pushover curve occurs.

The pushover curves computed for each of the 24 structures are then transformed into equivalent elastic-perfectly plastic curves that are used as input in both SPO2FRAG (Baltzopoulos et al. 2017) and FRACAS (Rossetto et al. 2016). A lateral load pattern based on the

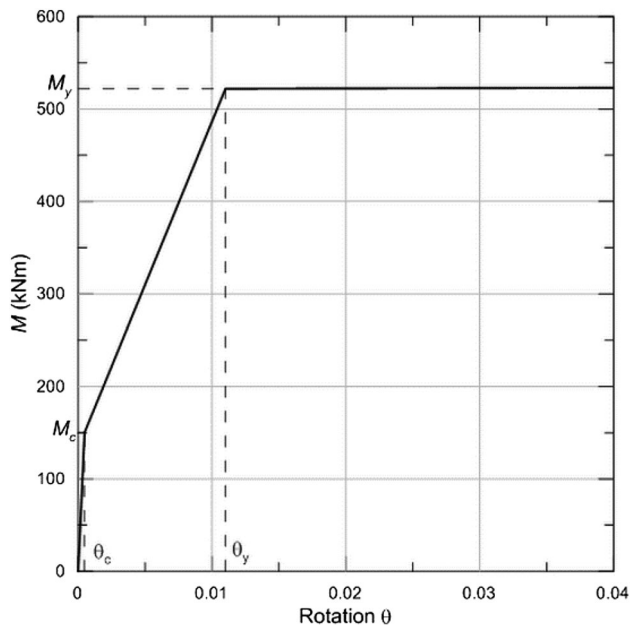


Fig. 3 Example of the moment-rotation relation for beam flexural hinges of the structural models

deformed shape of the fundamental eigenmode was applied in all cases. In SPO2FRAG, the uncertainties regarding the yield force and the ultimate ductility are both taken into account. The nonlinear time-history analyses (*NTHA*) for the equivalent single-degree-of-freedom are performed in FRACAS using a collection of 20 representative horizontal components recorded in the Bucharest area during the Vrancea intermediate-depth seismic events of March 1977 (moment magnitude $M_W = 7.4$, focal depth $h = 94$ km), August 1986 ($M_W = 7.1$, $h = 131$ km), and May 1990 ($M_W = 6.9$, $h = 91$ km). The individual spectral accelerations, as well as the mean value and mean \pm one standard deviation values, are shown in Fig. 5.

The ground motion intensity measure (*IM*) used in SPO2FRAG is the spectral acceleration corresponding to the eigenperiod of the equivalent single-degree-of-freedom, while in the case of FRACAS, we selected the corresponding peak ground acceleration. We aim at assessing the influence of the two methodologies and of the selected *IM* on both fragility parameters and seismic risk metrics. While the spectral acceleration for the fundamental eigenperiod $SA(T_1)$ is a better *IM* than the peak ground acceleration, the latter measure can be more easily related to the seismic design procedure from the Romanian code that employs the peak ground acceleration as *IM*. A second reason for the use of the peak ground acceleration as *IM* is that the seismic hazard curves for *PGA* are more easily computed than the hazard curves for $SA(T_1)$. The latter are cumbersome to obtain when taking into account the local site conditions of Bucharest (deep sedimentary soil layers

of up to several hundred meters thickness) that can generate long-period spectral ordinates in the case of large magnitude Vrancea seismic events. A thorough discussion of the efficiency and sufficiency of various *IMs* can be found in Luco and Cornell (2007).

Consequently, even though FRACAS is able to derive fragility functions for various spectral periods, we decided to employ only the results for *PGA* because this parameter is also employed in the Romanian seismic design code. The parameters of the lognormal fragility functions (median values in g and corresponding standard deviations) are shown in Fig. 6 in terms of peak ground accelerations (FRACAS results), and in Fig. 7 in terms of spectral accelerations corresponding to the equivalent single-degree-of-freedom (SPO2FRAG results). A generalized linear model (GLM) statistical curve fitting approach was applied to obtain the fragility functions in FRACAS.

The results show an expected increase of the median values from the slight damage state to the complete damage state (all the drift limit values were taken from HAZUS for high-code structures). The differences in terms of median values between the same structure designed for three different levels of peak ground acceleration are quite small in the case of the medium- and high-rise reinforced concrete structures and much larger in the case of the steel structures. There is a limited correlation between the median *IM* values and the logarithmic standard deviations obtained using the two methodologies. Two reasons for this issue may be identified. The first is related to the cross-sectional dimensions that, in the case of the RC structures, are almost similar for all the structures that have the same height and structural system. However, in the case of the steel structures, the differences are much larger (as noted in Table 2). The second reason can be related to the fact that the median values and the standard deviations for the extensive, and especially for the complete, damage state are not so well constrained in FRACAS due to the structure of the ground motion database, as the number of ground motions able to generate inter-story drifts associated to the extensive, and especially to the complete, damage state is extremely limited.

4 Evaluation of Seismic Losses

The seismic losses for the 24 analyzed structures were evaluated using the following procedure. As in Pavel et al. (2017a), only the Vrancea intermediate-depth seismic source was considered in the analysis because this source is the dominant contributor to the seismic hazard in southern and eastern Romania. Firstly, the 50,000-year Monte-Carlo earthquake catalogue for the Vrancea intermediate-depth seismic source (containing only events with $M_W \geq 6.0$) that

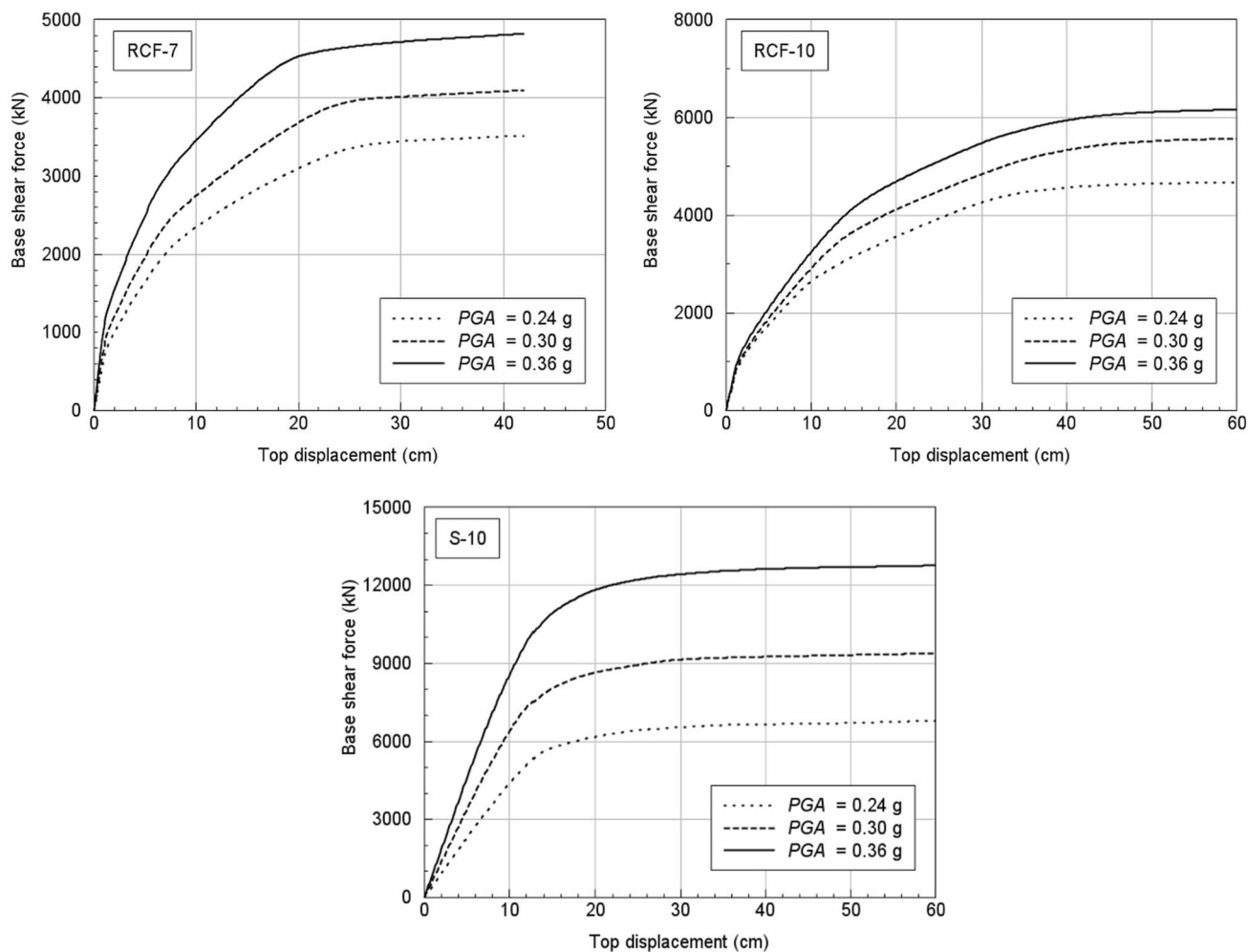


Fig. 4 Examples of pushover curves for the 7-story and 9-story reinforced concrete (RC) frame structures (RCF-7 and RCF-9), as well as for the 10-story steel structures (S-10) in Bucharest, Romania. *PGA* peak ground acceleration

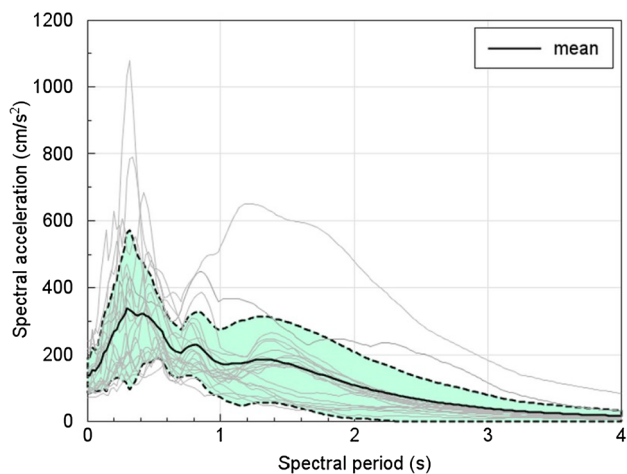


Fig. 5 Absolute acceleration response spectra for the 20 representative horizontal components used for the nonlinear time-history analyses (*NTHA*), as well as the mean value and mean ± one standard deviation values (green contour)

was also employed in Pavel et al. (2017a) was considered. This seismic catalogue includes 1051 individual seismic events with $M_w \geq 7.0$, and an additional 440 seismic events with $M_w \geq 7.5$. Subsequently, the ground motion amplitudes were obtained from the ground motion model of Vacareanu et al. (2015), also taking into account the inter- and intra-event variability.

The conditional probability of being in, or exceeding, a particular damage state (*ds*) given a ground motion intensity parameter is defined using the relation (FEMA 2012):

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

where \overline{IM}_{ds} is the median value of the ground motion intensity measure (either peak ground acceleration or spectral acceleration in this study) at which the building reaches the threshold of damage state (*ds*); β_{ds} is the standard deviation of the natural logarithm of the ground motion intensity parameter corresponding to damage state

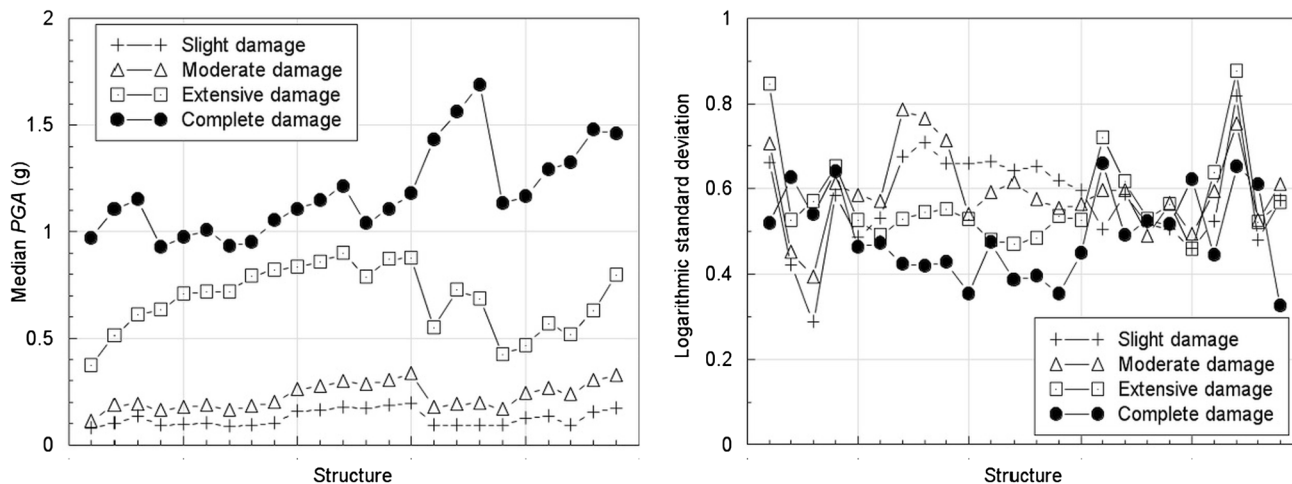


Fig. 6 Median peak ground acceleration (*PGA*) (left plot) and logarithmic standard deviations (right plot) for the fragility curves obtained using FRACAS for structures in Bucharest, Romania

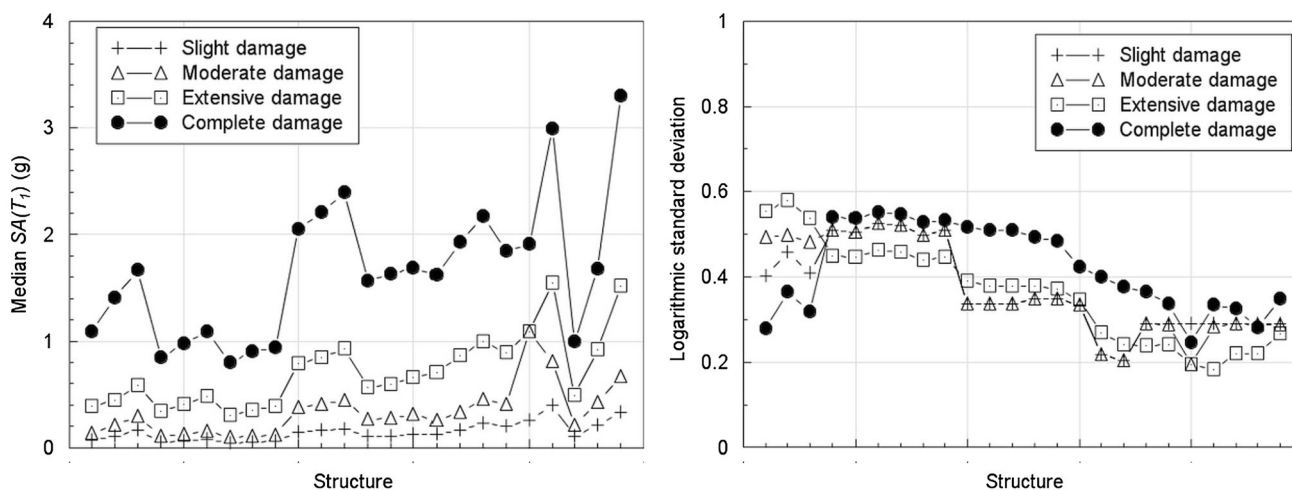


Fig. 7 Median $SA(T_1)$ (left plot) and logarithmic standard deviations (right plot) for the fragility curves obtained using SPO2FRAG for structures in Bucharest, Romania

(ds); and Φ is the standard normal cumulative distribution function.

The median value and the logarithmic standard deviations shown in Figs. 6 and 7 are used in Eq. 1 for the assessment of the conditional probability of damage. The mean annual damage degrees computed using both approaches (FRACAS and SPO2FRAG) are given in Table 3 for the 24 analyzed structures. Table 3 shows that the mean annual damage degrees given by the SPO2FRAG approach are generally larger than the ones obtained from FRACAS, with the exception of the majority of the steel structures.

Figure 8 shows the mean damage degrees obtained using the two methodologies as a function of the earthquake magnitude. The mean values are obtained by averaging the damage degrees computed for all 24 analyzed

structures. The mean damage degree is computed based on the discrete probabilities associated to each damage state. We prefer such a measure since it is more easily understandable than discrete probabilities associated with a particular damage state. It is clear that the mean values obtained from SPO2FRAG are higher than the ones from FRACAS for the entire magnitude range.

The 24 analyzed structures were grouped into eight classes (each containing the same structure designed for three levels of peak ground acceleration). The mean damage degrees as a function of earthquake magnitude are shown in Fig. 9. A sort of capping of the mean damage degree can be inferred from the SPO2FRAG results for magnitudes $M_w \geq 7.5$. In addition, the results from SPO2FRAG appear to be more logical in the sense that the largest damage degrees are expected for the RC frame

Table 3 Comparison of the mean annual damage degrees computed for reinforced concrete (RC) and steel (S) structures in Bucharest, Romania, using the FRACAS and SPO2FRAG approaches

Structure	Mean annual damage degree	
	FRACAS	SPO2FRAG
RCF-0.24-5	0.058	0.061
RCF-0.30-5	0.030	0.045
RCF-0.36-5	0.020	0.027
RCF-0.24-7	0.039	0.069
RCF-0.30-7	0.034	0.063
RCF-0.36-7	0.033	0.054
RCF-0.24-10	0.042	0.063
RCF-0.30-10	0.040	0.059
RCF-0.36-10	0.034	0.058
RCW-0.24-10	0.018	0.024
RCW-0.30-10	0.017	0.021
RCW-0.36-10	0.015	0.018
RCW-0.24-13	0.016	0.036
RCW-0.30-13	0.013	0.034
RCW-0.36-13	0.011	0.030
S-0.24-7	0.037	0.030
S-0.30-7	0.036	0.025
S-0.36-7	0.034	0.019
S-0.24-10	0.039	0.023
S-0.30-10	0.022	0.011
S-0.36-10	0.021	0.004
S-0.24-13	0.038	0.039
S-0.30-13	0.015	0.020
S-0.36-13	0.014	0.004

F frames, *W* shear walls

structures (high-rise and then medium-rise structures), followed by high-rise RC shear wall structures. This observation is due to the significant long-period spectral amplitudes that occur in the event of large magnitude Vrancea intermediate-depth earthquakes (observation confirmed by the 1977 and 1986 seismic events). A larger spread in the damage degrees for all the structures is observed by analyzing the results obtained using SPO2FRAG, as compared to the results from FRACAS.

The results of the comparison of the mean annual damage degrees as a function of the level of the design peak ground acceleration are summarized in Table 4. The relative difference between the mean annual damage degrees from SPO2FRAG and FRACAS for structures designed for $PGA = 0.24$ g and $PGA = 0.36$ g appears as smaller than the difference between the mean annual damage degrees for structures designed for $PGA = 0.30$ g.

Fragility functions for generic structural typologies as a function of the height regime are proposed. Six structural

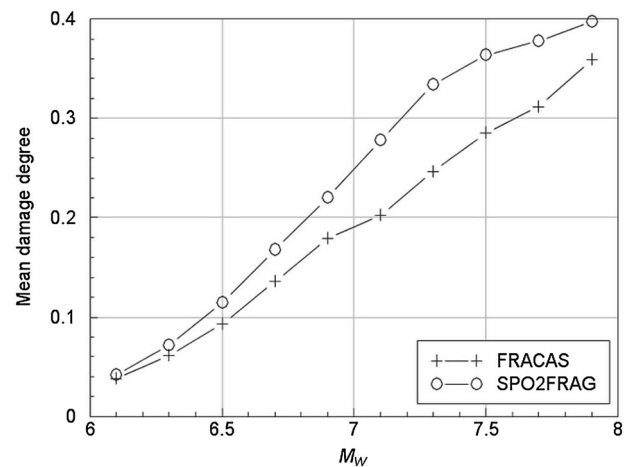


Fig. 8 Comparison of the mean damage degrees of structures in Bucharest, Romania, using the FRACAS and SPO2FRAG methodologies as a function of earthquake magnitude

typologies are defined as follows: RCF-MR (mid-rise RC frames); RCF-HR (high-rise RC frames); RCW-MR (mid-rise RC structures with shear walls); RCW-HR (high-rise RC structures with shear walls); SCBF-MR (mid-rise concentrically braced steel structures); and SCBF-HR (high-rise concentrically braced steel structures). The parameters for the fragility functions are given in Table 5 and are based on the results obtained from nonlinear time-history analyses for the individual structures designed for peak ground accelerations of 0.24 and 0.30 g, respectively (the design peak ground acceleration values for Bucharest from the 2006 and 2013 versions of the Romanian seismic design code). The median values for the six generic typologies have the same order of magnitude as those proposed by Olteanu et al. (2016), but the corresponding standard deviations are smaller in this study. The median values proposed in this study are lower than the ones in Kappos et al. (2006) for the same typology. Figure 10 compares the fragility functions for the six generic typologies and for the four selected damage states.

5 Life-Cycle Analysis

Another focus of this study was to perform the life-cycle analysis for the 24 analyzed structures in order to determine the economic feasibility of a further increase of the mean return period of the seismic action in the Romanian seismic design code from 225 to 475 years. The procedure for life-cycle analysis is the one proposed by Kappos and Dimitrakopoulos (2008). The life-cycle analysis was performed considering both direct and indirect economic losses and using the fragility functions defined both in terms of peak ground acceleration and spectral

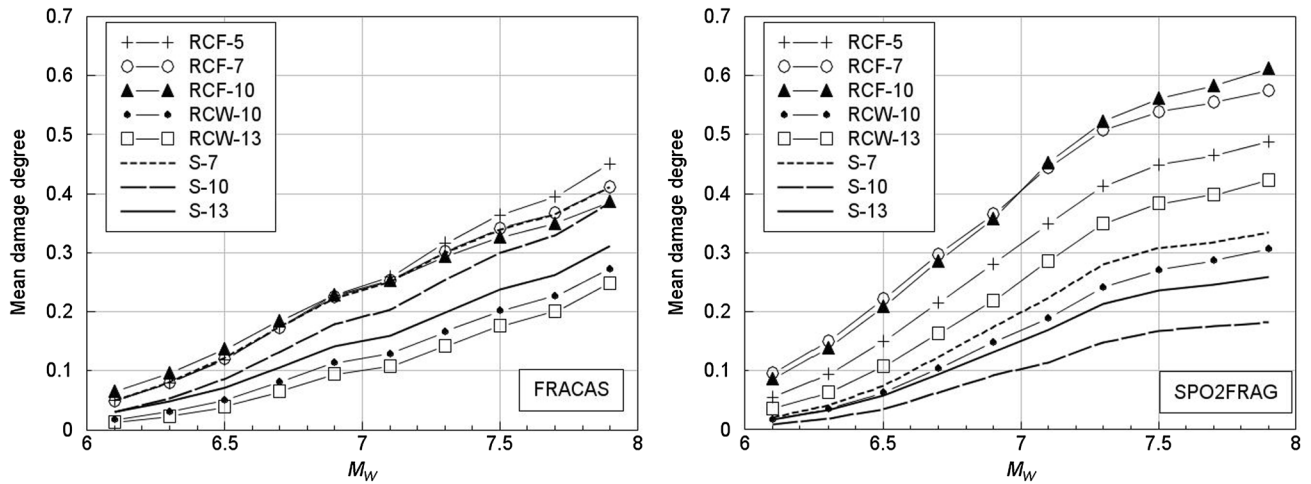


Fig. 9 Comparison of the mean damage degrees for each class of structures in Bucharest, Romania, using the FRACAS and SPO2FRAG methodologies as a function of earthquake magnitude (left–

FRACAS results, right–SPO2FRAG results). *RC* reinforced concrete, *F* frames, *W* shear walls, *S* steel

acceleration. A maximum planning horizon of 50 years was considered in the analysis, while the discount rate has three values—0.02, 0.04, and 0.06. A constant value of 600 Euro/m², which is an approximate mean value for Bucharest, was considered as building replacement cost. The total losses on the vertical axis represent the overall contribution of all the structures designed for the same

level of the peak ground acceleration. A rental loss of 10 Euro/m² per month was also considered in the analysis. The model from HAZUS (FEMA 2012) was considered for the evaluation of the building recovery.

Table 4 Comparison of the mean annual damage degrees for structures in Bucharest, Romania, using the FRACAS and SPO2FRAG methodologies as a function of the level of design peak ground acceleration (*PGA*)

Design <i>PGA</i>	Mean annual damage degree	
	FRACAS	SPO2FRAG
0.24 g	0.043	0.036
0.30 g	0.034	0.026
0.36 g	0.027	0.023

Figure 11 shows, as expected, higher values for the total losses computed using SPO2FRAG (Baltzopoulos et al. 2017), as compared to the ones obtained using FRACAS (Rossetto et al. 2016). What is more interesting is that the results show a contrasting situation—based on the results from FRACAS it was economically feasible to increase the design peak ground acceleration from 0.24 g in the 2006 seismic design code to 0.30 g in the 2013 seismic design code, but it is not economically feasible to make another increase (at least for planning horizons smaller than 50 years). However, if we look at the results from SPO2FRAG, it appears as economically feasible to make another increase of the design *PGA* up to 0.36 g for all the selected discount rate values.

Based on the analysis of the individual results it appears as economically feasible to increase the level of the design

Table 5 Parameters of the fragility functions (median and standard deviation) for the generic structural typologies in Bucharest, Romania

Structure	Damage state							
	Slight damage		Moderate damage		Extensive damage		Complete damage	
	Median	SD	Median	SD	Median	SD	Median	SD
RCF-MR	0.092	0.538	0.159	0.588	0.557	0.637	0.993	0.562
RCF-HR	0.090	0.692	0.170	0.774	0.753	0.537	0.943	0.421
RCW-MR	0.160	0.661	0.268	0.566	0.844	0.504	1.126	0.415
RCW-HR	0.178	0.636	0.293	0.565	0.830	0.510	1.072	0.375
SCBF-MR	0.092	0.546	0.184	0.597	0.637	0.669	1.497	0.574
SCBF-HR	0.115	0.565	0.236	0.584	0.507	0.605	1.274	0.600

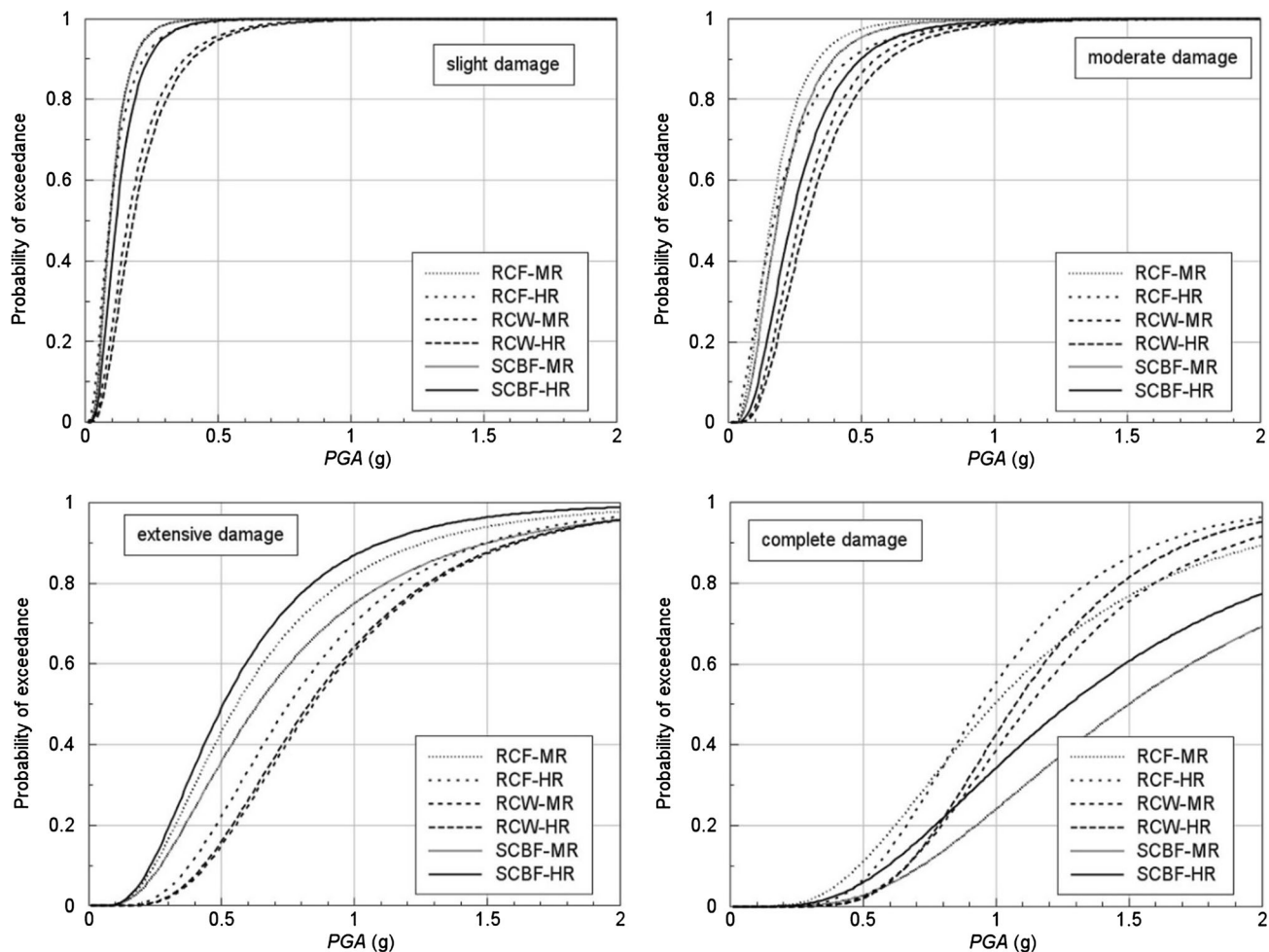


Fig. 10 Fragility curves for four damage states of structures in Bucharest, Romania, obtained for the six generic typologies in terms of *PGA*

peak ground acceleration for the structures at which the difference between the mean damage degrees is significant (for example, mid-rise reinforced concrete frame structures or high-rise steel structures). The economic feasibility is decided based on the total losses for a planning horizon of 50 years, that is if the losses for the structure designed for a peak ground acceleration of 0.36 g are smaller than the losses for the other two structures designed for smaller peak ground accelerations, then it is economically feasible to increase the design level. The reasons for the observed differences can be attributed both to the applied methodology, as well as to the ground motion database used for the nonlinear time-history analyses. Thus, this aspect of the economic feasibility needs further checks (including the use of different hysteretic models) before drawing a final conclusion.

6 Conclusion

In this article, the seismic fragility of 24 RC and steel structures designed for three levels of peak ground acceleration using the current Romanian seismic design code P100-1/2013 (Ministry of Regional Development and Public Administration 2013) is assessed. The assessment of the seismic fragility of high-code reinforced concrete and especially steel structures represents a novel approach. The seismic fragility is computed using two procedures available in the literature. The first approach, SPO2FRAG evaluates the seismic fragility using the pushover curve, while in the second approach, the fragility curves are computed from the nonlinear time-history analyses of an equivalent SDOF. Subsequently, the seismic losses are computed from ground motions simulated for a Monte Carlo earthquake catalogue for the Vrancea intermediate-depth seismic source.

The results show in general larger damage degrees obtained using the SPO2FRAG method, with the exception

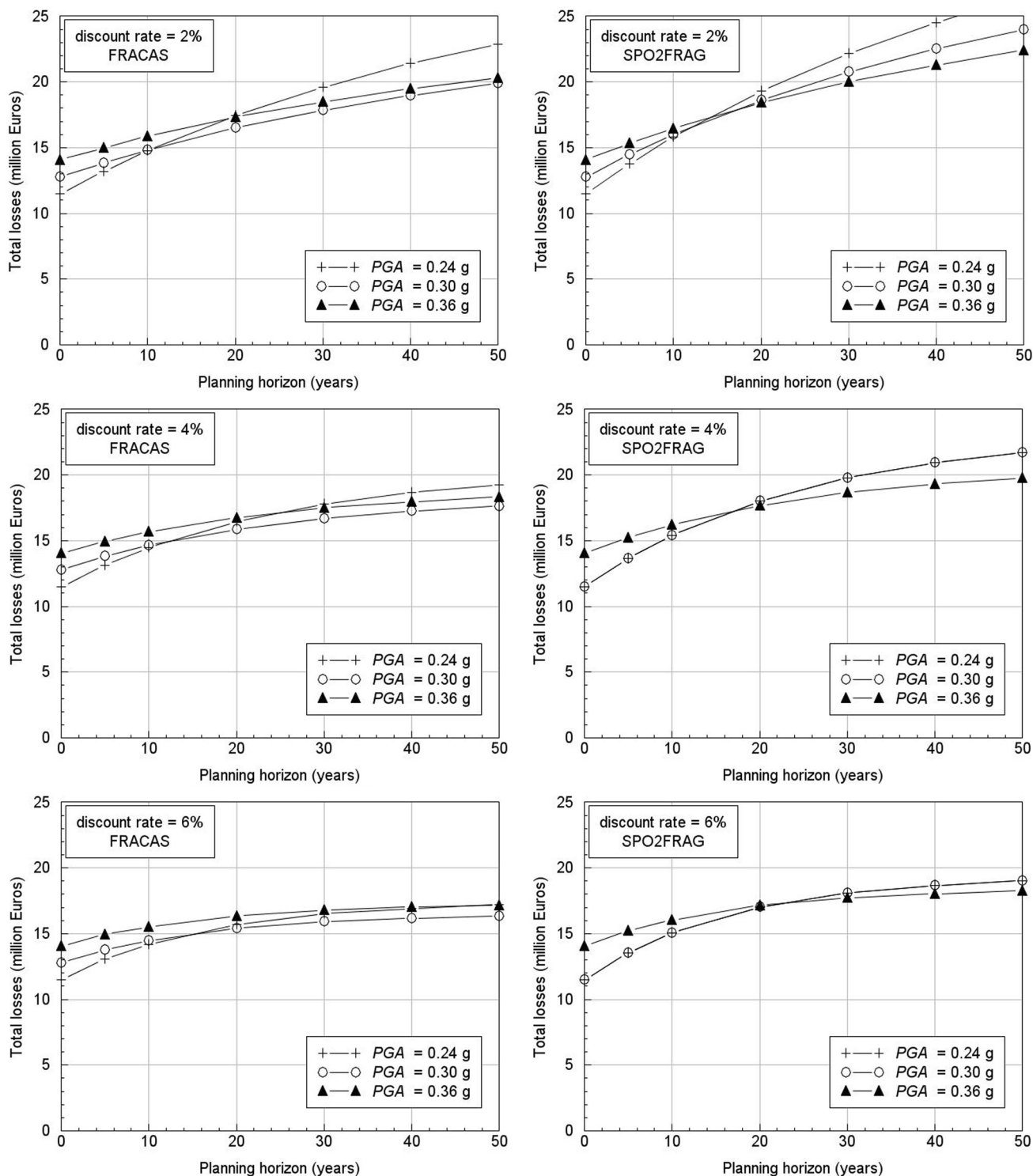


Fig. 11 The results of the life-cycle analysis of structures in Bucharest, Romania, as a function of the discount rate. The results were obtained taken into account the damage degree computed using the FRACAS and SPO2FRAG methodologies. *PGA* peak ground acceleration

of the steel structures. Moreover, the mean damage degrees obtained using the SPO2FRAG approach appear more logical since the most vulnerable buildings appear to be the high-rise and medium-rise RC frame structures. In

addition, based on the computed parameters generic fragility functions for medium-rise and high-rise RC frames, RC structural walls, and concentrically braced steel frames are proposed as a function of the building height. The life-

cycle analysis of the 24 structures shows a contrasting situation—based on the results from FRACAS it is not economically feasible to increase the design peak ground acceleration to 0.36 g, while the results from SPO2FRAG show exactly the opposite. This aspect will need further verification including the use of nonlinear time-history analyses because the aspect of increasing the design peak ground acceleration should also be based on some economic aspects, besides the main issue of increasing the level of safety for the citizens. A particular topic of research interest will be the collapse assessment of structures, especially in the light of a possible future shift towards uniform risk design in contrast with the traditional uniform hazard design.

Acknowledgements The constructive feedback from Dr. John Douglas and from one anonymous reviewer, as well as from the Editor Dr. Ying Li is greatly appreciated and has helped us to considerably improve the quality of the original manuscript.

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